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**COMPARATIVE STUDY OF THE LONG-TERM DEFLECTION OF
CONVENTIONAL AND SELF-COMPACTING CONCRETE WITH
LIGHT-WEIGHT CONCRETE SLABS**

By

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CERTIFICATE OF AUTHORSHIP/ORIGINALITY

I certify that the work in this thesis has not previously been submitted for a degree nor has it been submitted as part of requirements for a degree except as fully acknowledged within the text. I also certify that the thesis has been written by me. Any help that I have received in my research work and the preparation of the thesis itself has been acknowledged. In addition, I certify that all information sources and literature used are indicated in the thesis.

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Behnam Vakhshouri

June 2017

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Notations

The symbols used in this study. Including their definitions, are listed below.

<i>AASHTO</i>	American Association of State Highway and Transportation Officials
<i>ACI</i>	American Concrete Institute
<i>AEA</i>	Air-Entraining Agent
<i>ALWAC</i>	All Light-Weight Aggregate Concrete
<i>AS</i>	Australian standard
<i>ASTM</i>	American Society for Testing and Materials
A_{cr}	Area of section at first cracking
A_e	Effective section area under direct tension
A_g	Gross cross section area
A_{st}	Area of tensile steel bar
<i>BST</i>	Commercial name for the spherical-shaped polystyrene beads with hydrophilic type chemical coating
<i>CC</i>	Conventional concrete
<i>CS</i>	Compressive Strength
<i>CS.CUBE</i>	Compressive strength of cube specimen
<i>CSH</i>	Calcium Silicate Hydrates
<i>CS.CYL</i>	Compressive strength of cylindrical specimen
<i>CSSC</i>	Compressive stress-strain curve
<i>Cyl.</i>	Cylinder
C_b	Concrete cover of bar to the soffit of member
C_m	Smaller of clear bottom cover in reinforced concrete
<i>c/p</i>	Cement to powder ratio
C_s	Concrete cover of bar to the side surface of member
<i>DL</i>	Dead load
<i>D-SCC</i>	Self-compacting concrete containing steel fibre
<i>DS-SCC</i>	Self-compacting concrete containing hybrid fibre (steel and Polypropylene fibre)
D_0^{el}	Initial (undamaged) elastic stiffness of the material
d	Effective depth of tensile bar
<i>dia.</i>	Diameter
d_b	Bar diameter
<i>ECC</i>	Engineered Cementitious Composite
<i>EI</i>	Bending stiffness
<i>EPS</i>	Expanded Poly-Styrene
<i>EPS-LWC</i>	Light Weight Concrete containing Expanded Poly-Styrene Beads
E_c	Modulus of elasticity of concrete

E_{c-14}	Modulus of elasticity of concrete at age 14 days
E_{c-28}	Modulus of elasticity of concrete at age 28 days
E_f	Modulus of elasticity of fibre
E_p	Final slope of the compressive stress-strain curve
E_s	Modulus of elasticity of steel bars
E_0	Initial slope of the compressive stress-strain curve
F	Axial tension in pull out test
FEA	Finite Element Analysis
$FHWA$	Federal Highway Administration
FRP	Fibre reinforced polymer
$FRSCC$	Fibre reinforced self-compacting concrete
f_{ck}	Specified compressive strength of concrete
f_{cm}	Mean concrete cylinder compressive strength
f_{ct}	Splitting tensile strength of concrete
f_{ct-28}	Splitting tensile strength of concrete at age 28 days
f_{cu}	Specified characteristic cube compressive strength
$F_{d,ef}$	Effective design load, per unit area
f_r	Modulus of rupture (flexural tensile strength)
f_{rel}	Relative compressive strength
f_{r-28}	Modulus of rupture at age 28 days
f_s	estimated design service stress in tension reinforcement
f_t	Splitting tensile strength
$ft.$	Foot
f_y	yield strength of reinforcement
f_{yt}	Yield stress of tensile bar
$f(t)$	Time-function parameter
f'_c	Compressive strength of concrete
$f'_t{}^{ef}$	Effective tensile strength derived from the biaxial failure function of concrete
$GGBFS$	Ground Granulated Blast Furnace Slag
gr	Gram
GPa	Giga Pascal
G_c	Absorbed energy per unit volume of the cylindrical specimens under compression
G_{c-28}	Absorbed energy per unit volume of the cylindrical specimens under compression at age 28 days
G_f	Fracture energy needed to create a unit area of stress-free crack
G_{Fi}	Average energy required for unit crack propagation from crack initiation until any time
$HPFRC$	High-Performance Fibre Reinforced Concrete
$HRWRA$	High Range Water Reducing Admixture
$HSLWC$	High-Strength Light-Weight Concrete
$HSNWC$	High-Strength Normal-Weight Concrete
h_r	Rib height in steel bar

I_{cr}	Moment of inertia of section at first cracking
I_e	Effective moment of inertia
I_g	gross moment of inertia of uncracked reinforced concrete section
I_m	Average moment of inertia of section in span length
$J(t, t_0)$	Compliance function of concrete
$LEFM$	Linear Elastic Fracture Mechanics
l	Span length
lb	pound
lb/ft^3	Pound per cubic feet
Lit/m^3	Litre per cubic metre
LL_1	Loading level 1 (Lower loading level)
LL_2	Loading level 2 (Upper loading level)
$LVDT$	Linear variable differential transformer
LWC	Light weight concrete
$LWCC$	Light-Weight Ceramic Concrete
L_{cr}	Cracked length
l_d	Embedment length of steel bar in concrete
L_{ef}	Effective span length
l_n	Length of clear span length measured face to face of supports
$kips$	Imperial unit of force equal to 1000 pounds-force
Kg	Kilo grams
kg/m^3	Kilo gram per cubic meters
KN	Kilo Newton
KPa	Kilo Pascal
Ksi	Size effect factor
k_{cs}	Coefficient of creep and shrinkage in long-term deflection calculations
K_p	Final slope of the compressive stress-strain curve
max	Maximum
min	Minimum
ml/kg	Millilitre per kilo gram
mm	Millimetre
$mm/mm \text{ } ^\circ K$	Millimetre / millimetre grade Kelvin
MM	Moisture migration
MoE	Modulus of elasticity
MoR	Modulus of rupture (flexural tensile strength)
MPa	Mega Pascal
M_a	bending moment due to service loads (service moment)

M_a / M_u	Ratio of service moment to the ultimate moment capacity of reinforced concrete section
M_{cr}	Cracking moment of reinforced concrete section
M_{cr} / M_u	Ratio of cracking moment to the ultimate moment capacity of reinforced concrete section
M_{max}	Maximum bending moment at critical section
M_u	Ultimate bending moment of reinforced concrete section
m_1	Weak axis of the material
m_2	Strong axis of the material
m^3	cubic meters
$M-\kappa$	Moment- curvature
N	Vector of applied loads
$N-SCC$	Normal self-compacting concrete
N/mm^2	Newton per square millimetre
$N(t)$	Resisting force (reaction) in restrained shrinkage test
OPC	Ordinary Portland Cement
$OPKS$	Oil Palm Kernel Shell
pcf	Pound per cubic inch
PET	Poly-Ethylene Terephthalate
PFA	Pulverised Fuel Ash
$PL60-11-3L$	Type of strain gauge to record the strain variation in concrete surface
psi	Pounds per square inch
P_{cr}	Tension force at first cracking under direct tension
RC	Reinforced concrete
RH	Relative Humidity
$RILEM$	Reunion Internationale des Laboratoires et Experts des Materiaux (International Union of Laboratories and Experts in Construction Materials, Systems, and Structures)
RMS	Road and maritime services
RTA	Regional Transportation Authority
$RVDT$	Rotary variable differential transformer
r_{ec}	Reduction factor of the compressive strength
R_{elec}	Electrical resistivity
r_{et}	Reduction factor of the tensile strength
R_{hm}	Rebound value (Schmidt hammer)
R_r	Ratio of the rib height to the rib spacing in ribbed bar
S	Bar spacing
s	Slip of steel bar in concrete
SCC	Self-compacting concrete
$SFRC$	Steel Fibre Reinforced Concrete
$SHCC$	Strain Hardening Cementitious Composites

<i>SLWAC</i>	Sand Light-Weight Aggregate Concrete
<i>SP</i>	Super Plasticiser
<i>S-SCC</i>	Self-compacting concrete containing Polypropylene fibre
<i>STS</i>	Splitting tensile strength
S_0	bond transfer length in short-term loading
S_0^*	bond transfer length in long-term loading
s_m	Slip of steel bar in concrete at peak bond stress
S_r	Final average crack spacing
s_r	Rib spacing in steel bar
<i>t</i>	Time
T_{max}	Ultimate bond force
t_c	Age of concrete where moist-curing ends in days
<i>UDL</i>	Uniformly distributed load
<i>UHPRFC</i>	Ultra High-Performance Fibre Reinforced Concrete
<i>UNSW</i>	University of New South Wales
<i>UPV</i>	Ultrasonic Pulse Velocity
<i>UTS</i>	University of Technology Sydney
U_i	Elastic strain energy
V/S	Ratio of Volume to drying surface area of concrete specimen (Hypothetical thickness)
<i>vol. %</i>	Volumetric percentage
V_{air}	Entrained air content by volume
V_p	Loading rate in pull out test
$V_{\Delta W}$	Rate of weight loss of shrinkage specimen with time
w	Crack width
w/c	Water/cementitious material ratio
<i>WFLA6-11</i>	Type of strain gauge to record the strain variation in steel bars
w_a	Applied service load
w_c	Crack width at the complete release of stress
W_i	Applied work of external force P
w_{max-t}	Maximum crack width at time t
W_0	Initial weight of shrinkage specimen
γ	Density
γ_{RH}	Coefficient of relative humidity in creep coefficient calculation
γ_{VS}	Coefficient of hypothetical thickness in creep coefficient calculation
γ_{t0}	Coefficient of loading time in creep coefficient calculation
γ_{xy}	Shear strain component in plane x and direction y
γ_{yz}	Shear strain component in plane y and direction z
γ_{zx}	Shear strain component in plane x and direction z

ΔU	Vector of current nodal displacement
Δ_{cr}	Deflection due to creep effect
Δ_e	Flexural elastic deflection of slab
Δ_{e-14}	Flexural elastic deflection of slab based on the modulus of elasticity at 14 days
Δ_{e-28}	Flexural elastic deflection of slab based on the modulus of elasticity at 28 days
δ_i	Crack opening displacement
Δ_{inc}	Incremental flexural deflection
Δ_{ins}	Instantaneous flexural deflection
Δ_{ins-14}	Flexural instantaneous deflection of slab subjected to loading at age 14 days
Δ_{i-t}	Flexural deflection of point i at time t
$(\Delta_{ins} / \Delta_e)_{14}$	Ratio of flexural instantaneous deflection to the elastic deflection based on the modulus of elasticity at 14 days
$\Delta_{ins-14} / \Delta_{e-28}$	Ratio of flexural instantaneous deflection at 14 days to the elastic deflection of slab based on the modulus of elasticity at 28 days
Δ_{long}	Long-term flexural deflection
$\Delta_{long} / \Delta_{ins}$	Ratio of long-term to instantaneous flexural deflection
Δ_{sh}	Deflection due to shrinkage (autogeneous and chemical shrinkage)
Δ_{sus}	Time-dependent deflection under sustained loading
Δ_{tmp}	Temperature induced deflection
Δ_{tot}	Total flexural deflection
$\delta\kappa_{0,ts}$	Average instantaneous curvature
ΔW	Weight loss of shrinkage specimen
$\Delta W / W_0$	Ratio of lost weight to the initial weight of shrinkage specimen
ΔL_{ef}	Flexural deflection limit of beam and slab
Δ_1	Vertical deflection of slab at mid-span
Δ_2	vertical deflection of slab at the border of high moment region
Δ_{e1}	Calculated elastic deflection of slab at mid-span
Δ_{e2}	Calculated elastic deflection of slab at the border of high moment region
Δ_{1-SS}	Flexural deflection at midspan of simply supported slab
Δ_{1-FE}	Flexural deflection at midspan of fixed supports slab
Δ_{2-FE}	Flexural deflection at the border of high moment region in fixed supports slab
Δ_{2-SS}	Flexural deflection at the border of high moment region in simply supported slab
ε	Strain
ε'_c	Corresponding strain to the maximum compressive strength of concrete
ε_{cr}	Creep strain
$\varepsilon_{csd.b}$	Basic drying shrinkage strain
$\varepsilon_{csd.b}^*$	Final drying basic shrinkage strain
ε_{cse}^*	Final autogeneous shrinkage strain
ε_e	Elastic strain
ε_p	Plastic strain
ε_s	Tensile strain in steel bar
ε_{sh}	Shrinkage strain
ε_z	Normal strain component in z direction
ε_{cse}	Autogeneous shrinkage strain
ε_{csd}	Drying shrinkage strain

ε_{s2}	Average strain in a tensioned member
ε_y	Ultimate strain of tensile steel bar
ε_u	Ultimate strain of the compressive stress-strain curve
ε_o	Corresponding strain to the maximum compressive strength of concrete
ξ	Scalar stiffness degradation variable
κ	Curvature
κ_{sh}	Shrinkage-induced curvature
$(\kappa_{sh})_{cr}$	Shrinkage-induced curvature on a previously cracked cross section
$(\kappa_{sh})_{uncr}$	shrinkage-induced curvature in uncracked cross-section
λ	Crack spacing
$\mu\varepsilon$	Micro strain
ν	Poisson ratio
ρ	Ratio of tensile steel bar in the section
ρ_b	Ratio of tensile steel bar in the section at balance condition
σ_0	Reference compressive stress of concrete
σ_e	Effective stress
$\sigma-\varepsilon$	Stress-strain
σ_c	Compressive Strength of concrete
σ_z	Normal stress component in direction z
σ_y	Normal stress component in direction y
σ_x	Normal stress component in direction x
σ_{cl}	Concrete stress away from the crack
σ_{sl}	Steel stress away from the crack
σ_{s2}	Steel stress at the crack
σ_{s1}^*	Steel stress away from the crack after all shrinkage cracking
σ_{s2}^*	Steel stress at the crack after all shrinkage cracking
τ	bond stress
$\tau_{b-average}$	Average bond stress to describe the force transfer between steel bar and concrete
τ_{max}	Peak bond stress
τ_{xz}	Shear stress component in plane x normal to z direction
τ_{yz}	Shear stress component in plane y normal to z direction
τ_u	Ultimate bond stress
ϕ_{cc}	Creep coefficient
$\varphi(t, t_0)$	Creep coefficient

$\phi_{c.b}$	Basic creep coefficient
ϕ_u	Ultimate creep coefficient
$^{\circ}K$	Degree Kelvin
$2D$	Two- dimensional
$3D$	Three-dimensional
$\%$	Percentage

ABSTRACT

Long-term deflection of concrete slabs is often the main governing design criteria to determine the appropriate thickness of the slabs to meet the required serviceability limit state. Due to the nonlinear nature of the material, predicting the long-term deflection of concrete structures is complex. Also, the complicated behaviour of concrete in the presence of reinforcement makes the problem more challenging to study.

In general, codes of practice give an overall estimation of the long-term deflection as a multiplier of the short-term or elastic deflection of concrete structures. Such estimation sometimes may lead to an entirely wrong prediction of deflection and consequently unsafe, unrealistic and unprofitable concrete design and construction.

Recent developments in concrete technology have led to produce new construction materials by significant strength and performance features. Many of these developments are engineered solutions to technical and commercial problems by either improvement of the current practices or overcoming of limitations in the existing construction technology. Lightweight and self-compacting concrete are the main two innovative concrete types used in the construction industry along with conventional concrete. This study examines and evaluates the time-dependent deflection of reinforced concrete one-way slabs made of two concrete types versus the conventional concrete.

The presented investigation comprises both experimental and analytical components. The experimental part consists of laboratory investigation of the time-dependent flexural deflection and monitoring of the strains in reinforced concrete slabs. Since lightweight concrete is very prevalent in building construction in Australia, and there are limited studies on its long-term behaviour, the current study examines the lightweight concrete slabs identical to the previously tested conventional concrete slabs (Nejadi, 2005) and self-compacting concrete slabs (Aslani, 2014) subjected to the identical long-term loading.

In addition, an analytical study consists of verification of the recorded experimental data and parametric study of the effective factors in the time-dependent behaviour and deflection of the conventional and self-compacting concrete slabs are conducted. Recently developed high-capable ATENA software is utilized in the numerical analysis. Load-deflection behaviour of the slabs under short-term loading is also recorded and compared with those of self-compacting and conventional concrete slabs.

Finally, the results of this study are used to evaluate and verify the existing and proposed models associated with the parameters that affect the flexural deflection of reinforced concrete slabs with different types of concrete.

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CHAPTER 1

INTRODUCTION

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Chapter 1: Introduction

A general background on the topic and state of the problem is explained. The objectives and expected results and contributions of the study in this area are presented. The research methodology and the layout of the thesis in accordance with the literature review, experimental program and the numerical investigation are described. To emphasise the topic's importance, a sample of the huge economic loss due to the large deflection of one-way slabs in real construction projects is given.

1.1. BACKGROUND

Light-Weight Concrete (LWC) is used worldwide in the construction industry where savings made in the weight of structures is a major factor. The most common applications of the lightweight concrete are in floor systems, bridge decks, masonry and offshore structures. LWC is made by replacing some or all of the normal weight aggregate with lightweight aggregate. Often the coarse fraction is replaced with lightweight aggregate and the fines fraction is normal weight sand. According to the lightweight concrete density and its compressive strength at age 28 days, lightweight concrete can be classified into two broad categories of structural and non-structural concrete.

Lightweight structural concrete is typically made using expanded clay, shale or slate. Lightweight aggregates such as crushed limestone, granite and quartz weigh less than normal weight aggregates due to the porous cellular structure of the individual aggregate particles.

Compressive strength, elastic modulus, splitting tensile strength and other mechanical properties of the lightweight concrete are significantly affected by the material properties of the

lightweight aggregate used. The lightweight aggregate itself must possess desirable properties such as adequate compressive strength, porosity, abrasion resistance and good bonding with the cement paste. For this reason, non-structural lightweight aggregate such as Perlite, Vermiculite, and Styrofoam are not considered suitable for structural concrete, but rather are used in concrete meant for insulation or lightweight filler.

Developments in concrete technology provide engineers, suppliers and contractors with new methods and approaches to engineering problems. Many of these developments are engineered solutions to technical and commercial problems by either improvement of current practices or overcoming limitations in the existing technology.

The long-term deflection of concrete slabs is often the governing design criteria to determine the appropriate thickness to meet the required serviceability limit state. Serviceability limit state places a limit on the maximum deflection and cracking allowed in the slab. If the structure does not meet serviceability state requirements, it may be deemed not fit for the intended purpose. Excessive concrete slab deflections would be deemed as a serviceability failure as the deflection has adverse effects on the functionality of the other connected members, while not being in and of itself a catastrophic structural failure. The negative impacts of excessive slab deflections on the structure include jammed doors and windows, cracking of masonry and plaster partitions and perceivable floor deflection.

An extreme example of a serviceability failure would be the infamous Silverton Centre Office Building constructed in 1983 in Canberra. The flat slab concrete structure was evacuated in 1989 and demolished in 1994 due to the excessive slab deflections and cracking. The excessive deflections of the structure resulted in failure and subsequent leaking of the building's curtain walls. The structure's value was estimated to be worth AUD\$20m in 1994; the lost revenue due to the vacancy of the building is estimated to be AUD\$20m, and the legal costs of all parties involved in the subsequent litigation are estimated to be over AUD\$25m [1]. In addition, a reliable and universally accepted procedure of designing the long-term deflection of concrete slabs has not been developed yet.

1.2. STATE OF THE PROBLEM

Concrete is the world's most broadly used construction material. However, despite all benefits related to the use of concrete, the considerable self-weight of concrete compared to the other construction materials and its workability problems limit its use in some structures.

Deflection is a common concern in almost all design codes and technical drafts in relation to the time-dependent behaviour of the concrete structure. Short-term and long-term deflections are regulated in serviceability limit states. Due to nonlinear nature of the concrete, prediction of long-term deflection is complex. Also, the complicated behaviour of concrete in the presence of reinforcement makes the problem hard to study. To better prediction of the deflection, comprehensive information on time-dependent and nonlinear behaviour of material is required.

In general, there are big differences between the predicted values of deflection and the real values measured in the concrete members under service loads especially in long-term deflection. The codes of practice give an overall estimation of long-term deflection as a multiple of the short-term or elastic deflection of concrete. Such estimation sometimes may lead to an entirely wrong prediction of deflection and consequently unsafe and unprofitable concrete design and construction.

There are different methods in the design codes to control deflection and to ensure the serviceability adequacy of flexural reinforced concrete members over long-term period. Limiting the minimum thickness of a concrete member is one of these simple methods of deflection control. This limit is just an initial estimation of the expected deflection and is not able to guarantee the expected accuracy under the different conditions of loading, environmental effects and impacts of the mechanical properties of utilized materials.

One major problem with the explicit calculation of deflections, particularly long-term deflection, is that the agreement between the calculated deflection and what actually occurs should be checked. It is commonly found that the calculated and measured values differ to a great extent. The deflection of concrete slabs involves numerous factors including the geometric properties of the structure, concrete material properties and load history. Long-term deflections are the result of increase in the short-term deflection over time due to concrete inelastic shrinkage and creep strains. The accurate calculation of long-term deflection in concrete slabs is crucial as it is often the governing design criteria to determine the slab thickness to meet the required serviceability limit state.

Finite element programs are useful to analyse the non-linear behaviour of concrete slabs most notably two-way slab systems. However, they lack the functionality to accurately model the long-term deflections due to complexity of the factors involved. Currently, there is no commonly accepted finite element software to calculate the long-term deflections in concrete slabs. Without industry guidelines, there is an increased risk of structures failing to meet the required serviceability limit state using finite element methods, the consequences of which can be very severe as in the case of the Silverton Centre office building.

There exists a need for both theoretical and experimental research of the critical factors which affect the time-dependent long-term deflection of different types of concrete structures under real day to day service loads.

Most of the equations to predict the time-dependent behaviour in codes of practice are for Conventional Concrete (CC), and there are limited numbers of relationships for Self-Compacted Concrete (SCC) and LWC. SCC and LWC are results of recent technologies in the construction industry. Hence, due to different mechanical properties of the used materials compared to the conventional concrete, calculation and prediction of their short-term and long-term deflection by utilizing the CC models and equations may not be justifiable.

Nejadi Gilbert (2005) [2] tested six one-way slabs made from CC under sustained loads for about 400 days in the University of New South Wales (UNSW). Later, Aslani and Nejadi (2014) [3] conducted a similar experiment on the identical slabs with similar loading, span length and sections with different type of concrete. They used SCC to make the identical one-way slabs.

Since LWC is very prevalent in the building construction in Australia and there are limited studies on the long-term behaviour of this type of concrete, the current study examines the identical LWC slabs with the previously tested CC and SCC slabs under long-term loading. Therefore, to have a comparison with existing data of the previously conducted studies by Nejadi and Gilbert (2005) [2] and Aslani and Nejadi (2014) [3], the same one-way slabs made of LWC, with the similar loading, span length and sections have been investigated.

1.3. OBJECTIVES AND SCOPE OF THE RESEARCH

- 1- To monitor the affecting parameters associated with the fresh and hardened properties of LWC;
- 2- To investigate and compare the fresh properties of CC, SCC and LWC (e.g. slump and fresh concrete density);
- 3- To propose a new relationship between mixture design, slump and compressive strength of SCC and LWC;
- 4- To study the hardened properties of LWC including the mechanical properties, bond characteristics, shrinkage and creep to compare with those of CC and SCC;
- 5- To propose new mathematical models to predict the hardened properties of LWC accurately;

- 6- To study the relationship between the compressive strength and modulus of elasticity of LWC with the mixture design parameters by investigating the wide range of experimental data of the utilized type lightweight concrete in the literature;
- 7- Comparison of the requirements of structural lightweight concrete in the fresh and hardened state in different design codes and guidelines;
- 8- To obtain laboratory controlled data for LWC to calibrate, validate, and extend analytical models that are being developed concurrently with the experimental program;
- 9- To study the mechanisms associated with the flexural deflection of slabs and influence of the factors that affect their short-term and long-term deflections under sustained service loads and proposing design guidelines for deflection calculations of LWC, SCC and CC slabs;
- 10- Verification and validation of the analytical model by utilizing the finite element program by implementing the proposed models of fresh and hardened LWC properties into the ATENA software.

1.4. METHODOLOGY

This research comprises both experimental and analytical components. The experimental part of the current study consists of laboratory investigation of the time-dependent flexural deflection and monitoring of the strains in reinforced concrete elements of LWC. In addition, the analytical part consists of two separate parts of verification of the recorded data of experimental parts and the parametric study of the effective factors in time-dependent behaviour and deflection of CC, SCC and LWC one-way slabs.

1.4.1. Experimental Program

Phase I: Testing of the fresh and hardened properties of LWC in the laboratory-controlled conditions by utilizing the standard cylinders, prisms and cubes according to Australian standards and test methods.

Phase II: Monitoring and recording the short-term (instantaneous) deflection of slab specimens.

Phase III: Monitoring and recording the long-term deflection of slab specimens.

Phase IV: Comparing the results in above mentioned phases with the results of former investigations by Nejadi and Gilbert (2005) and Aslani and Nejadi (2014) on identical slabs with different types of concrete.

1.4.2. Numerical Investigation

Phase I: Parametric study of the short-term and long-term deflection of the one-way simply supported slabs

- For parametric study, recorded data from LWC tests together with the previously recorded data of one-way slabs made from SCC and some other conventional concrete data available in the literature are implemented.

Phase II: Verification of the proposed analytical model to calculate the LWC slabs deflection by utilizing the recorded data of deflection and strain changes during the test period.

- Simulation of the slab specimens by ATENA software and utilisation of the proposed material properties of LWC. ATENA Finite Element software is a powerful tool to investigate the effects of different parameters on LWC slabs deflection accurately.

1.5. LAYOUT OF THE THESIS

This thesis is presented in eleven chapters and two appendices. The current chapter (Chapter one) explains the state of the problem, objectives and methanoyl of the research program.

Chapter Two of this thesis provides a review of previous LWC studies related to history and development, advantages and limitations, fresh properties, mechanical properties, bond characteristics, shrinkage and creep, and available full-scale time-dependent LWC structure investigations.

Chapter Three is a state-of-the-art review of the hardened concrete properties and available empirical equations. Also, in Chapter Three, equations are developed for each of the hardened concrete properties. Besides, to have a comprehensive comparison between the mechanical properties of LWC with CC and SCC, the existing models of predicting the mechanical properties of LWC, SCC and CC have been collected and compared.

Chapter Four is a review of the time-dependent behaviour of hardened concrete and available empirical equations. Also, in Chapter Four, the existing models of shrinkage and creep and bond-slip models in LWC, SCC and CC in the literature are investigated.

Chapter Five reviews the mechanism of flexural deflection in the reinforced concrete structures. Additionally, the structural behaviour of concrete, reinforcement and the interaction between concrete and reinforcement are broadly discussed and the existing models of tension stiffening and the effective moment of inertia in literature are presented.

Chapter Six describes the experimental program for material properties of proposed LWC mixtures. Also, the resultant mechanical properties of LWC have been compared with the fresh and hardened properties of CC and SCC mixtures used in the experiments by Nejadi and Gilbert (2005) and Aslani and Nejadi (2012) respectively.

Chapters Seven, Eight and Nine include the second phase of the experimental program. Chapter Seven describes the experimental program for LWC short-term flexural deflection. Also, a review of the similar experimental programs on CC slabs by Nejadi (2005) [2] and SCC slabs by Aslani (2014) [3] is presented. Results of the current study on the short-term deflection (load-deflection) of LWC slabs are compared with the corresponding experimental results on the above-mentioned CC and SCC slabs.

Chapters Eight and Nine contain the explanations on the structures, testing setup, loading magnitudes, monitoring system and the experimental results of the instantaneous and long-term deflection of slabs, respectively. The experimental results of LWC slabs are compared with the previously conducted experiments on the CC and SCC slabs. Additionally, new models are proposed to calculate the instantaneous and long-term deflection of LWC, SCC and CC slabs.

Chapter Ten evaluates the finite element models simulated in the ATENA program. Also, developed models in the previous Chapters about the LWC mechanical properties, bond characteristics, shrinkage and creep are implemented in the ATENA program. At the end of this Chapter, a parametric investigation using the proposed models to study the time-dependent deflection has been performed and the results are summarised and discussed.

Finally, Chapter Eleven summarizes the conclusions drawn from the investigation of this work and a proposal for future research is recommended.

Additional information including the raw data from creep and shrinkage tests, recorded values by strain gauges embedded in one-way slabs and the deflection measured at two points of each slab (mid-span and the border point of the high moment region of slab span) in LWC slabs are provided in the Appendices.

REFERENCES

1. Gilbert, R., et al. *The Risk and Responsibility of Structural Engineers—A Case Study of a Structural Failure*. in *Proceedings of International Congress-Creating with Concrete*. 1999.
2. Nejadi, S., *Time-dependent cracking and crack control in reinforced concrete structures*. 2005, PhD thesis, The University of New South Wales Sydney, Australia.
3. Aslani, F., *Experimental and numerical study of time-dependent behaviour of reinforced self-compacting concrete slabs*. PhD dissertation, University of Technology Sydney (UTS), 2014.

CHAPTER 2

LITERATURE REVIEW

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Chapter 2: Literature review

2.1. BACKGROUND AND DEVELOPMENT OF LIGHTWEIGHT CONCRETE

2.1.1. History

The first known use of Light-Weight Concrete (LWC) dates back over 2000 years. There are several LWC structures in the Mediterranean region, but the three most notable structures were built during the early Roman Empire and included the Port of Cosa, the Pantheon Dome, and the Coliseum. The Port of Cosa, built in about 273 B.C., used LWC made from natural volcanic materials. The Pantheon, finished in 27 B.C., incorporates concrete varying in density from the bottom to the top of the dome. The Coliseum, built from 75 to 80 A.D., is a gigantic amphitheatre with a seating capacity of 50,000 spectators [1].

LWC has a surprisingly long history. It was first patented in 1923, mainly for use as an insulation material. Although there is evidence that the Romans used air entrainers to decrease density, this was not a real LWC.

Many different materials have been used in LWC since Roman days, and some have shown remarkably better structural qualities than the Roman materials. Up to the 20th century, however, such improvements were associated with the strength of the cementing materials used rather than with improvements to the LWC.

The demand for LWC in modern construction applications, such as high-rise buildings, offshore structures and long-span bridges, is increasing. This interest arises from the decreasing amounts of many load-bearing elements, as well as the superior thermal properties of lightweight compared to Conventional Concrete (CC) [2]. LWC can be obtained by introducing

gas (or foam) into the paste or by wholly or partially replacing the standard aggregate with low-weight and preferentially low-cost components.

The implementation of LWC is usually necessitated by the reduction of the project cost, enhanced functionality, or a combination of both. The first such improvement in the strength of concrete came about in the course of Roman colonization when their need for widely scattered building activity forced the use, on some occasions, of various impure grades of limestone for the preparation of the required burned lime. In those instances where impurities were present in the silica, alumina and iron oxides, the strength of concrete was found to be substantially greater than where pure limes were used. This superior material was referred to as gray lime and subsequently become known through Europe as Roman cement.

Light-Weight Aggregate and LWC are not new materials. LWC has been known since the early days of the Roman Empire: both the Colosseum and the Pantheon were partly constructed with materials that can be characterised as lightweight aggregate concrete (aggregates of crushed lava, crushed brick and pumice). In the United States, over 100 World War II ships were built in LWC, ranging in capacity from 3000 to 140000 tons and their successful performance led, at that time, to an extended use of structural LWAC in buildings and bridges.

The logistic problems created by the entry of the United States into World War I in 1917 were compounded by the shortage of high-grade plate steel for building ships. The United States Fleet Corporation, an arm of the Federal Government, was charged with planning a shipbuilding program using materials other than steel. One of these materials was reinforced concrete, which had already been used in shipbuilding in the Scandinavian countries.

The fundamental problem was the reduction of dead weight, and tests were made of concrete made with natural aggregates, such as pumice, cinders and scoria, but these were found to be unsuitable because of their comparatively low-strengths in addition to their permeability and lack of uniformity.

The first commercial plant dedicated to expanded shale aggregate began operating in Kansas City, Missouri, in 1920 under the name Haydite Company. Even so, there were few design criteria available that could apply to applications of LWC in building construction, and little inclination among architects, engineers and builders to risk their reputations by experimenting with the new material. Two years later, a gymnasium addition to the Westport High School in Kansas City was the first LWC building in history.

The first major project employing structural LWC was undertaken in 1928 and 1929, in the form of an addition to the South-western Bell Telephone Company office in Kansas City. The building was originally built as a fourteen-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 heavyweight concrete.

The first structural LWC high-rise building was the Park Plaza Hotel in St. Louis (now the Chase Park Plaza). Built in 1929, this twenty-eight-story structure made extensive use of structure LWC in both frame and floor systems, as well as fire-proofing.

Among the more spectacular and sensational landmarks in the growth of LWC applications have been its use in some major bridges, and in test buildings utilized in the historic atom bomb tests at Yucca Flats, Nevada, in 1955. In the construction of the San Francisco-Oakland Bay Bridge, for example, the use of LWC floor in the upper deck permitted weight reduction of 25.0 pounds per square foot or a total of 31.6 million pounds for the entire structure.

The term "Lightweight Aggregate" describes a range of particular use aggregates that have an apparent specific gravity considerably below the normal sand and gravel which were at one time used in almost all concrete.

According to Glenn A. Black (2004), the term "Lightweight Aggregate" describes a range of special use aggregates that have an apparent specific gravity considerably below normal sand and gravel which were at one time used in almost all concrete. So it is evident that the lightweight aggregate is one of the critical elements that make concrete flexible and versatile enough to make the overall structural design and specifications to meet the construction requirements as argued by Leif Berntsson Satish Chandra (2002). It is also interesting to note that the lightweight aggregate in the concrete that is made using light-weight materials also provides a detectable level of compressibility as well as possessing the strength that can be defined based on the composition thus making it a versatile and profitable process in production process.

These lightweight aggregates will range from the extremely light materials used for insulative and non-structural concrete all the way to expanded clays and shales used for structural concrete. Since the lightness of these aggregates derives from the air trapped in each 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.

The range of lightweight aggregate is extensive, ranging from extremely light materials used for insulation and non-structural concrete all the way to expanded clays and shales used for structural concrete. This variety makes it clear that the lightweight aggregate in the concrete is mainly aimed at achieving a high level of physical stability and compressibility through effectively utilising the physical qualities of the aggregate materials. This is further justified by the arguments of Leif Berntsson Satish Chandra (2002) who argues that the lightweight

aggregate in the concrete is a significant step towards innovation in the field of engineering itself [3].

2.1.2. Development of Lightweight Concrete

Lightweight and free flowing (high flow-ability) concrete is a suitable material for a wide range of purposes such as, but not limited to, panels and block production, floor and roof screeds, wall casting, complete house casting, sound barrier walls, floating homes, void infill, slope protection, outdoor furniture and many more applications.

The demand for lightweight concrete in modern construction applications, such as high-rise buildings, offshore structures and long-span bridges is increasing. This interest arises from the decreasing volume of many load-bearing elements, as well as the superior thermal properties of lightweight concrete compared to conventional concrete.

Lighter concrete offers design flexibility and substantial cost-saving by providing less dead load, improved seismic structural response, low heat conductivity and lower foundation cost when applied to structures. In recent years, due to these advantages, there has been an interest in production and investigation of the light or reduced-weight concrete.

The accompanying "Concrete Spectrum" in Figure (2.1) shows the relative weights and differing applications of the various lightweight aggregates now in use. 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 240 or 320 kg/m³.

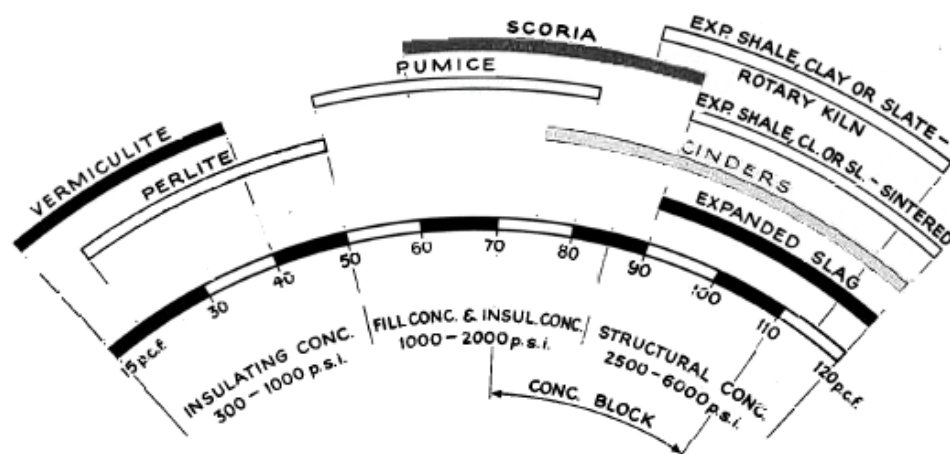


Figure 2.1: Weight spectrum of lightweight concrete (1lb/ft³=16.02 kg/m³) [4]

Air-dry densities of lightweight concrete ranging from 190 to 1900 kg/m³ are shown in Figure (2.2) with different types of lightweight aggregates. Structural lightweight concrete at the right end of the spectrum has strengths of 1800 kg/m³ and above [4].

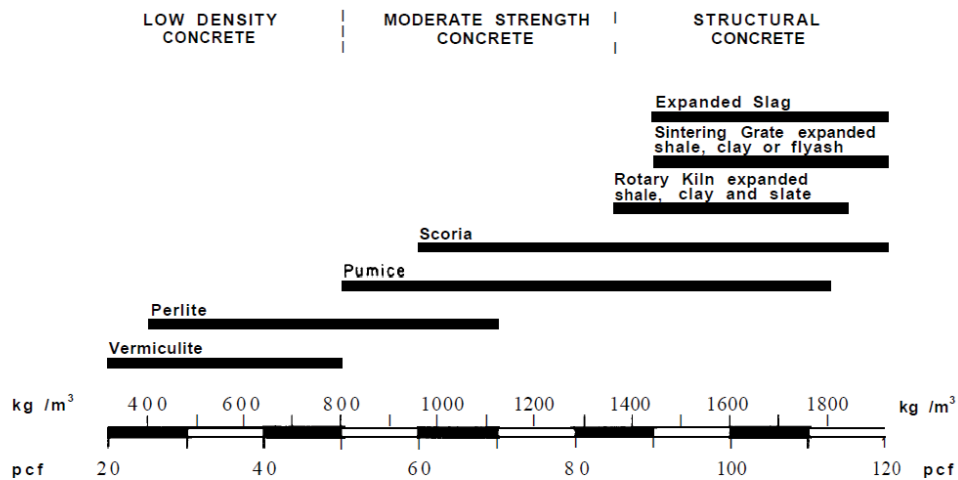


Figure 2.2: Typical ranges of densities of concretes made with various lightweight aggregates (Approximate unit weight and use classification of light weight aggregate concrete) [4]

In general, lightweight aggregates are broadly classified into two types: natural (e.g., pumice, diatomite and volcanic cinder) and artificial (e.g., perlite, clay, sintered fly ash and expanded shale). Lightweight aggregates have many sources. They may be produced using natural materials such as shales, clays, pumice, diatomite, volcanic cinders, and slates. Artificial materials (by-products) such as iron blast furnace slag, clay, sintered fly ash, and shale are other sources of lightweight aggregates [4],[5],[6]. Figure (2.3) shows the sources and types of the lightweight aggregate.

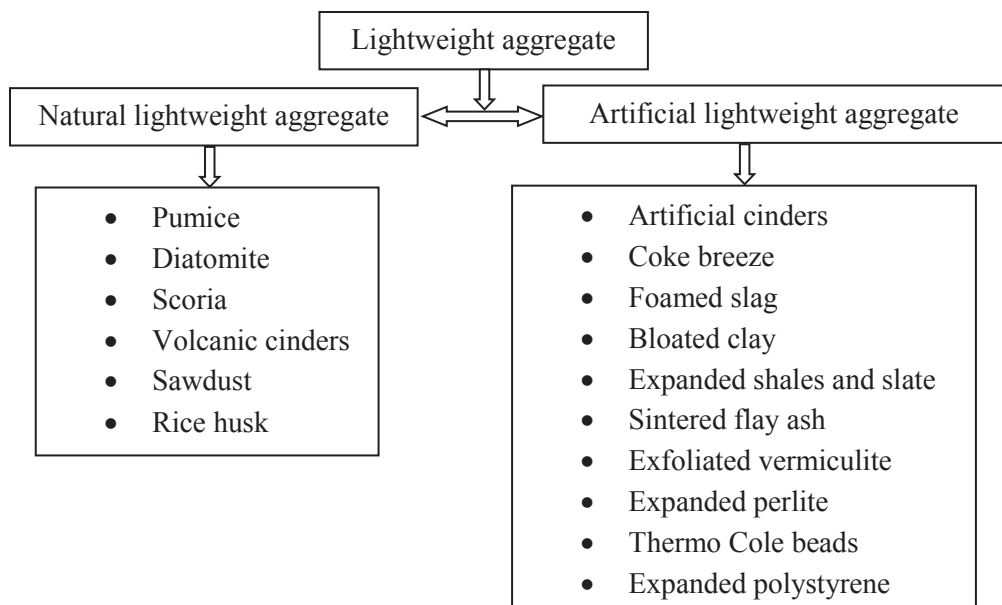


Figure 2.3: Natural and artificial lightweight aggregates used in lightweight concrete

2.2. MIX DESIGN OF LIGHTWEIGHT CONCRETE

The main requirements of structural lightweight concrete from mix design are as follows:

- Compressive strength and density of the hardened concrete for the structural design;
- Consistency or workability for production purposes and entrained air volume.

Other properties may be required for a particular application such as frost resistance, water impermeability, and the rate of chloride diffusion. For design work, formulas exist that are used to calculate, for tensile strength or the modulus of elasticity, based on density and compressive strength.

The aim of mix proportioning of concrete is to present a formula or a recipe according to specifications. The procedure for proportioning is to combine different concrete ingredients to attain the required properties of the fresh, as well as the hardened concrete. The properties of concrete are chosen from the structural design and the requirements of safe and functional structures. The requirements are mostly technical in nature, such as consistency, workability, compressive strength, density, freeze-thaw resistance, water permeability, thermal conductivity, and, not least, the service life of the structure. In many cases, there are requirements which are implicit and expected of good quality concrete, i.e., stability against segregation of aggregate, minor internal water bleeding, homogeneity after transportation, and compaction. Good economy is the main driving force of all kinds of concrete structures.

LWC is quite similar to normal-weight (conventional) concrete, except that the whole or a part of the aggregate consists of porous particles with the density being less than the natural normal-weight aggregate of rock origin. In practice, the designation of LWC is classified according to standardized strength and density classes. Structural LWC has a closed structure which means that all voids between the aggregate particles are filled with a binder component or cement paste[7]. The surfaces of the aggregate particles have to transfer the distributed stresses from the surrounding binder. This is not the case in concrete with an open structure or no-fines concrete where the internal stresses are concentrated at discrete contact points on aggregate particles.

2.2.1. Mix Design Components and Proportioning

The common ingredients (mixture components) of LWC are as following:

- Binder, for example, type of cement and mineral admixture, i.e., silica fume, fly ash, and slag;
- Fine and coarse aggregates, and their properties such as particle size distribution by sieve analysis, bulk density, and particle density of the lightweight aggregate, water content, and, eventually, water absorption;
- Chemical admixtures, such as plasticizers and air-entraining agents;
- Water.

In calculating the optimal solution for the strength and density of LWC at the start of the mix design process, the results are dependent on the strength, density, and size of the available lightweight aggregate. Sometimes, much is to be gained by changing one or two ingredients. Based on the proportioning methods, most of the LWC can be designed for different applications. There are many ways to produce lightweight concrete. Of these, three main types can be identified as following:

2.2.1.1. Monolithic lightweight concrete

Monolithic lightweight concrete is a lightweight aggregate concrete with monolithic structure in which the lightweight aggregate is used instead of normal-weight aggregate. The concrete may be used as structural concrete and non-load-bearing concrete for heat insulation purposes.

2.2.1.2. Aerated lightweight concrete

This type of LWC is an aerated, foamed or gas concrete that relies on introducing large voids within the concrete mass. There are examples where lightweight aggregate has also been added to such mixes.

2.2.1.3. No-fine lightweight concrete

No-fines concrete as a low-density concrete is formed by omitting the fine aggregate, causing a large number of interstitial voids. Coarse aggregate of normal density is used, but lightweight aggregate gives considerably reduced weight and better heat insulation properties. The no-fines concrete of lightweight aggregate is normally composed of the coarse lightweight aggregate of fractions from 4 mm and upwards, for example, 4 to 8 or 10 mm. For no-fines, the fine aggregate is omitted and the concrete consists of cement, water and coarse aggregate only. Quite

often, there is an addition of a mineral admixture such as fly ash or slag and a small addition of filler or fine sand to the binder paste, mostly for economic and technical reasons [8]. No-fines concrete is a lightweight aggregate in bulk with the particles surrounded by a coating of cement paste or binder or mortar. The thickness may be about one millimetre and sometimes a little more; for that reason, large voids exist between the aggregate particles. When graded aggregate is used, a higher bulk density results in comparison to a one-size aggregate. The volume of voids is decreased, resulting in higher strength and thermal conductivity.

2.3. DEFINITION OF LIGHTWEIGHT CONCRETE

There are various definitions and specifications for structural and non-structural LWC in international codes of practice. Some of the most frequently used specifications of lightweight aggregates in codes of practice are described below.

2.3.1. ASTM-C330/M

2.3.1.1. Standard Specification for lightweight aggregates in structural concrete

The main purpose for structural LWC in ASTM-C330/M (2014) [9] is reducing the density while maintaining the compressive strength of the concrete. General specifications of lightweight aggregate for structural concrete are defined in Table (2.1). The grading requirements of allowed lightweight aggregates are presented in Table (2.2).

Table 2.1: Specifications of lightweight aggregate for structural concrete in ASTM-C330/M

	Preparation method	Physical Properties
Covered aggregate types	Prepared by expanding, pelletizing, or sintering products like blast-furnace slag, clay, diatomite, fly ash, shale, or slate	Clay lumps and friable particles less than 2% by dry mass
	Prepared by processing natural materials like pumice, scoria, or tuff	Uniformity of grading change of fineness modulus in different samples should be less than 7%

Table 2.2: Grading requirements of lightweight aggregate for structural concrete in ASTM-C330/M

Nominal size designation	Percentage (mass) passing sieves having square openings									
	25.0mm (1 in.)	19.0 mm (3/4 in.)	12.5 mm (1/2in.)	9.5 mm (3/8in.)	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	300 µm (No. 50)	150 µm (No. 100)	75 µm (No. 200)
Fine aggregate Up to 4.75mm	...	...	...	100	85-100	...	40-80	10-35	5-25	...
Coarse aggregate										
4.75 to 25 mm	95-100	...	25-60	...	0-10	...	...	...	...	0-10
4.75 to 19 mm	100	90-100	...	10-50	0-15	...	...	...	...	0-10
4.75 to 12 mm	...	100	90-100	40-80	0-20	0-10	...	...	...	0-10
2.36 to 9.5 mm	...	...	100	80-100	5-40	0-20	0-10	...	...	0-10
Combined fine and coarse aggregate										
Up to 12.5 mm	...	100	95-100	...	50-80	...	...	5-20	2-15	0-10
Up to 9.5mm	...	...	100	90-100	65-90	35-65	...	10-25	5-15	0-10

Considering the mechanical properties, concrete specimens containing lightweight aggregate under test should meet the requirements in Table (2.3).

Table 2.3: Concrete specimens containing lightweight aggregate in ASTM-C330/M

	Calculated Equilibrium Density max, kg/m ³	Average 28-day minimum Splitting Tensile Strength, , MPa	Average 28-day minimum Compressive Strength, MPa
All lightweight aggregates	1760	2.2	28
	1680	2.1	21
	1600	2.0	17
Combination of normal- Weight and lightweight aggregates	1840	2.3	28
	1760	2.1	21
	1680	2.1	17

The drying shrinkage of concrete specimens prepared, cured, and tested should not exceed 0.07%. Also, the following tests are required for aggregates and structural LWC:

2.3.1.2. Tests on Aggregates:

One representative sample is required for each test of organic impurities, staining, loss of ignition, grading, bulk density, and clay lumps.

2.3.1.3. Tests on Concrete

At least three specimens are required for each of the following tests of concrete: compressive strength, shrinkage, density, resistance to freezing and thawing, and the presence of pop-out materials. At least eight concrete specimens are required for splitting tensile strength tests.

2.3.2. ACI-213R-2003 [1]

Structural aggregate meeting the requirements of ASTM-C330 for bulk density less than 1120 kg/m^3 for fine aggregate and less than 880 kg/m^3 for coarse aggregate is allowed in this standard. The minimum required compressive strength at the age of 28 days is 17 MPa, by an equilibrium density between 1120 and 1920 kg/m^3 . To achieve the requirements, application of entirely lightweight aggregate or a combination of lightweight and normal-density aggregates is allowed.

2.3.3. ACI 211.2-98 [10]

This standard provides applicable methods for selecting and adjusting mixture proportions for structural lightweight concrete. The required specifications for structural LWC are as follows:

- (a) Made with lightweight aggregates in accordance with ASTM-C330;
- (b) Has a compressive strength in excess of 17 MPa at 28 days of age when tested in agreement with methods stated in ASTM C330;
- (c) Has an air dry weight not exceeding 1840 kg/m^3 as determined by ASTM-C567.

In absence of prior information, the proportioning of first trial mixtures can be ascertained by either of these methods:

2.3.3.1. Weight method, specific gravity pycnometer

This method applies to sand-lightweight concrete containing lightweight coarse aggregate and normal weight fine aggregate. To estimate the weight of the LWC batch, the specific gravity factor of lightweight coarse aggregate is determined and the first estimate of the weight of fresh LWC can be made.

2.3.3.2. Volumetric method

This method is applicable for all lightweight and combinations of lightweight and normal weight aggregates. The total volume of aggregates required is measured as the sum of the un-

combined volumes on a damp, loose basis. Of this amount, the loose volume of the fine aggregate may make up 40 to 60 % of the total loose volume.

2.3.4. Experimental Investigations

Yang et al. (2014) [11] conducted a study to develop mixture-proportioning for structural LWC to determine the unit content of each component to achieve the target values for the slump, compressive strength, dry density, and air content of the concrete. Based on the comprehensive database with 347 test results, which were compiled from concrete mixtures with expanded fly ash or clay lightweight aggregates, they formulated the water to cement ratio, and unit water content and provided a simple guide to proportion a trial mixture of structural LWC. Because of the limited range and information of the dataset, the proposed approach was optimized for the following requirements: the compressive strength range between 18 and 50 MPa, the range of dry density of concrete between 1200 kg/m³ and 2000 kg/m³, and the maximum aggregate size being 19 mm or 25 mm.

2.4. CLASSIFICATION OF LIGHTWEIGHT CONCRETE

A classification of LWC can be based on the density of the concrete. There may be a relation between the density and the compressive strength of lightweight aggregate concrete. A low-density concrete has lower strength and a high-density concrete often has higher strength. Structural lightweight aggregate concrete normally has compressive strength from at least 10 to 70 MPa.

The density and compressive strength of structural lightweight concrete are lower than those for conventional concrete. According to Table (2.4), the definition of structural and non-structural lightweight concrete, concerning density and compressive strength limits varies in codes of practice and references.

Table 2.4: Definition structural and non-structural lightweight concrete in codes of practice

Reference	Density (kg/m ³)	Compressive strength (MPa)	Application
ACI 213R-03 [1]	800 to 2240	≥ 17	Structural
ACI 211.2-98 [10]	≤ 1840	≥ 17	Structural
ACI 213R(2014) [12]	1350 to 1900	≥ 17	Structural
TS 2511 (1977) [13]	≤ 1900	≥ 16	Structural
EN 206-1(2003) [14]	800 to 2000	8 to 80	Non structural
AS-3600 (2009) [15]	1800 to 2100		Structural

2.5. ADVANTAGES AND DISADVANTAGES OF LIGHTWEIGHT CONCRETE

2.5.1. Advantages

The general advantages of lightweight concrete are its reduced mass and improved thermal and sound insulation properties, while maintaining adequate strength. The reduced weight has numerous advantages, not the least of them being a reduced demand for energy during construction. In addition to lower density, better reinforcing steel-concrete bond, durability performance, tensile strain capacity and fatigue resistance make the LWC preferable to normal strength concrete.

Kivrak et al. (2006) describe the most common advantages of LWC compared to other types of concrete as the following: [16]

- Reducing the seismic hazard for tall buildings and special concrete structures by reducing the dead load of the structure;
- Reduction in the weight of the structure and consequently reducing the construction costs;
- Facilitating the transport of concrete in the field of precast concrete structures;
- Better thermal insulation compared to ordinary concrete;
- Using artificial aggregates and eliminating the limitations in absence of natural aggregates in all areas;
- High internal porosity which leads to maintaining a low apparent specific gravity.

Considering the massive consumption of natural raw materials in concrete industry, Schackow et al. (2014) [17] recommended replacing the gravel in the concrete with expanded polystyrene. Recycling of materials and waste reduction during the construction of buildings and also in the recycling of demolition materials are the main reasons for their proposition. They also mentioned the low thermal conductivity of LWC as an excellent option for applications requiring good insulation. Lower dead load of structure, ease of cutting and attaching filaments and unique sound isolation are other advantages of LWC in their study.

Since it is possible to achieve similar strength in LWC as conventional concrete, Rakoczy and Andrzej (2014) [18] found LWC is a more efficient concrete in regard to better strength/weight ratio than CC structural elements.

Lightweight concrete (LWC) is as a versatile material that has created keen interest and considerable industrial demand in recent years in a wide range of construction projects. It has been made lighter than conventional (normal weight aggregate) concrete [19], [20]. It has obvious advantages of high strength to weight ratio, good tensile strength, low coefficient of

thermal expansion, and superior thermal and acoustic insulation characteristics due to the air voids within its structure [21]. According to Newman et al. (2003) [22], it is possible to attain the compressive strength over 60 MPa in LWC with oven dry density range of about 300 to not exceed 2000 kg/m³ and thermal conductivities of 0.2 to 1.0 W/mK. These values can be compared to those for normal weight concrete with approximately 2100–2500 kg/m³, greater than 100 MPa and 1.6–1.9 W/mK.

Overall, the advantages of LWC can be summarized as the items in Table (2.5).

Table 2.5: Advantages of lightweight concrete

Advantage	Description
Decreased dead Load	Less mass is required to support additional weight. Structural reinforcement can be the less demanding
Structure efficiency	The design of economic concrete structures is accomplished by increasing the strength/weight ratio. Structural LWC is typically 25 to 35 percent lighter and smaller than conventional concrete. This translates into lighter superstructures and smaller loads for substructure design.
Lower water permeability	Greatly reduced due to the diffusion of closed cells which prevents sponging. Also reduces problems caused by rusting rebar by eliminating the problem at its source
Higher Seismic resistance	In lower densities concrete can absorb shock. LWC is often used in ballistic tests because of this ability. Hammer blows can be absorbed without fracturing the concrete
Higher sound absorption	The transmission of sound is inversely related to the number of air/solid interfaces. LWC has a high number of these interfaces; thus, more sound is absorbed
Greater Insulation	Enhanced R-values, especially in the lower density range. Again, this is due to the increased number of air/solid interfaces. Structures can have much higher thermal efficiency resulting in lower heating or air conditioning costs
Increased fire resistance	Greatly improved due to lower thermal conductivity. Spalling (scaling or flake chipping from heat) is reduced or eliminated
Adaptability	Lighter weight increases options for on-site casting. Forming can be swifter and easier due to the less supported weight
Simplicity	Ordinary tools can be used for alterations. It can be easily sewn and sculpted, and nailed or screwed without pre-drilling
Better handling	Handling capabilities are vastly improved. Concrete does not need to be cold, damp, dense, and hard to work with
Pumping	Fewer failures at the pump from baling or compacting. Air bubbles from the foam act as miniature ball bearings in the mix. There are no settling out the problems
Increased yield	Having a greater yield/ton equates to reduced fuel consumption and lower transit costs for the producer. Adding foam up to 10% volume (and subtracting aggregate and water) will not significantly reduce concrete strength - in some cases, it may improve
No Surface Bleed water	Reduction of excess water in the mix allows for sooner finishing. Flowability is achieved through the foam. Water can be utilized strictly for cement hydration
Placement	Less handling weight makes all aspects of moving concrete easier, resulting in reduced labour costs and quicker turnover

Table 2.5: Advantages of lightweight concrete (continued)

Advantage	Description
Aesthetic improvements	The beauty of concrete is its ability to adapt to any shape, whether angular or curvilinear. If desired, it can be cast in different forms to make it look like wood
Environmental improvements	The problem of deforestation can be reduced by relinquishing the demand for timber and substituting LWC in residential construction. Lightweight concrete in conjunction with tasteful design can be the solution for this environmental dilemma. Deforestation is also a major contributor to global warming because vegetation removes a large quantity of carbon dioxide from the atmosphere
Renovate and repair	One of the most extensive applications of structural LWC is in bridge re-decking where a lower dead load is achieved. This often means that bridge widths, traffic lanes and the thickness of structural slabs can be increased while utilizing existing piers, footings and other structural members. The use of LWC often allows the live load capacity of older structures to be increased
Durability	LWC has been used in structures like bridge decks and building for over 50 years. The excellent in-service performance in these structures as well as in marine structures and ships has demonstrated that lightweight concrete is a durable concrete.
Improved hydration	Lightweight aggregate containing high internal moisture contents may be suitable for conventional aggregates to provide internal curing. Higher cementitious concretes along with a very low water to binder ratio are important parameters in the self-desiccation. These concrete benefits significantly from the added internal moisture of properly pre-wetted lightweight aggregates. Internal curing is particularly useful for concretes containing high volumes of silica fume and other materials known to be sensitive to curing procedures.

2.5.2. Disadvantages

Apart from advantages and benefits of application of LWC in the construction industry, it also has several disadvantages. However, in many ways the benefits of lightweight concrete overshadow its disadvantages.

Lightweight aggregate concrete often has a more complicated production process than Conventional Concrete (CC) because the porous aggregates have excessively high water absorption and low density. Water absorption by the aggregates results in severe slump loss and a rapid setting time for the fresh concrete.

Sim et al. (2014) [23] compared LWC made with artificial lightweight aggregates and conventional concrete and mentioned some weak points of LWC as following:

- Artificial lightweight aggregates used in LWC are commonly weaker and have smoother surfaces than normal-weight aggregates. Hence, LWC has lower tensile strength, lower cohesion between aggregate particles and pastes, and a more brittle

descending branch after peak stress compared with CC of the same compressive strength;

- Cracks in LWC frequently propagate by passing through the aggregates; so the interlocking capacity of aggregates along the crack panels will be significantly reduced. Hence, the crack opening displacement and width of the fracture process zone can be expected to be smaller in LWC than in conventional concrete.

Schackow et al. (2014) [17] used Expanded Polystyrene (EPS) to produce LWC, however they mentioned that the increase in the content of EPS resulted in a significant reduction in the compressive strength. Table (2.6) shows some other disadvantages of using LWC.

Table 2.6: Disadvantages of lightweight concrete application

Disadvantage	Description
Sensitivity of mixture	LWC is very sensitive to the water amount in the mixture
Longer mixing time	Mixing time is longer than conventional concrete to assure a proper mixing
low compressive strength	Compressive strength in LWC is less that of conventional concrete
Lower modulus of elasticity	The modulus of elasticity of the concrete with lightweight aggregates is about 0.5 to 0.75 to that of the normal concrete; hence, more deflection is there in LWC members
Longer drying time	Due to the higher moisture content of LWC, drying times are typically longer than regular concrete to levels that might be considered adequate for application of floor covering materials. Typically, a 0.5 water to cement ratio slurry is used as a base mixture for lightweight concrete.
Thicker reinforcement cover	Usually, the cover in LWC structure is 25mm more than that of normal concrete because of its increased permeability and also concrete carbonates rapidly by which the protection to the steel by the alkaline lime is lost.
Pre-wetting problems	Pre-wetting aggregates require wetting before using, to achieve a high range of saturation. Some concrete producers may not have the capability pre-wetting lightweight aggregates in cold weather if temperature controlled storage is not available
Expensive concrete	LWC is usually more expensive than ordinary concrete, and its production process needs considerably more attention than that of the traditional concrete
More brittle concrete	LWC structure has a tendency to behave in a more brittle way than conventional concrete, special attention should be given to load-bearing mechanisms, where spalling or splitting actions govern the behaviour
Cracking of aggregates	Cracks in LWC often spread by passing through the aggregates not the interlock of the aggregates

2.6. FRESH PROPERTIES OF LIGHTWEIGHT CONCRETE

Although the general producing process of LWC and CC are similar in regard to mixture components and curing conditions, there are considerable differences in properties of fresh concrete between LWC and CC.

Main properties of fresh concrete are consistency, workability, segregation and bleeding. To study these parameters, air content, fresh density, slump and setting time are investigated in engineering and research applications of concrete.

Consistency or fluidity of concrete is a major component of workability and refers to the wetness of the concrete. However, it must not be assumed that the wetter the mix the more workable it is. If a mix is too wet, segregation may occur with resulting honeycombing, excessive bleeding and sand streaking on the formed surface. On the other hand, if the fresh concrete is too dry, it may be difficult to place and compact, and the segregation may occur because of lack of cohesiveness and plasticity of the paste.

2.6.1. Workability

Workability is a term to describe the properties of fresh concrete and consists of the four partial properties of concrete namely, mix-ability, transportability, mould-ability and compatibility. Workability is often defined as the amount of mechanical work required for full compaction of the concrete without segregation.

American Concrete Institute (ACI) defines the workability as the ease of placement of concrete. This term is defined in ASTM-C125 as the property determining the effort required to manipulate a freshly mixed quantity of concrete with minimum loss of homogeneity. The term “manipulate” includes the operations of placing, compacting and finishing the concrete. Neville and Brooks (2010) define the workability as the amount of useful internal work required to produce full compaction [24].

Mehta and Monteiro (2006) indicate that workability is a composite property, with at least two primary components: a) Consistency (or fluidity) which he describes as the “ease of flow” of the fresh concrete, and b) Cohesiveness which he describes the ability of fresh concrete to hold all ingredients together as the “tendency not to bleed or segregate”. Segregation risk increases with increasing flowability which is generally higher in LWC with, respect to conventional concrete. Hence, the control of workability and proper mix design is much more critical for LWC than for conventionally placed concrete [25].

The followings are effective parameters in workability of fresh concrete:

- Water content in the concrete mix;
- Content and properties of cement;
- Shape, size, texture, maximum size and grading of the aggregates;
- Temperature and humidity of concrete mix;
- Compaction and placement method of fresh concrete.

Kupaei et al. (2013) [26] investigated the production of geopolymer LWC using two locally available waste materials. They found that the fibres in the mixture absorb the existing water in the paste and decrease the workability and also reduce the contact area between the lightweight aggregate surface and the mortar, which causes a weak interfacial transition zone. To improve the workability, they used Naphthalene sulphonated super plasticizer.

According to Alengaram et al. (2011) [27], with the same grade of concrete strength, LWC by 20% lower density, shows higher workability, compared to CC.

2.6.2. Segregation

Having a broad conception of the effect of each component in mix design may improve the capacity to reduce the unwanted effects. For example, good knowledge about the optimum water to cement ratio brings down the segregation risk and enhances the flowability of the fresh concrete. Segregation also makes the concrete weaker and less durable.

Expanded clay, granulated slag, perlite or vermiculite, and polymer materials are frequently used as lightweight aggregate in LWC. Due to closed cavities, water absorption is high and it is difficult to estimate the required water volume. Raising the water to the surface during the mixing, accompanied by the tendency of lightweight aggregates to float up, increases the segregation risk [28], [29].

Mazaheripour et al. (2011) [30] recommends applying the mix design method of high-performance concrete for LWC to avoid the segregation problem and to keep the strength of concrete high, despite applying it to light-weight aggregate.

2.6.3. Testing of Fresh Properties

2.6.3.1. Slump

The slump test measures the concrete flow. Higher slump concretes that are still cohesive flow more readily and are easier to pump. However, excessively high slump concrete can separate and cause plugs in the pump or the line.

The addition of a water reducing agent accompanied by a reduction in cement and water can produce a mix that is hard to pump.

Slump and compacting factors can be reliable indicators of workability assessment. Table (2.7) presents the relation between workability level and the range of slump in the concrete. The Compacting Factor (CF) is defined as the weight ratio of partially-compacted to fully-compacted concrete.

Table 2.7: Relation between the workability of concrete and slump value

Workability	Slump (mm)	Compacting factor	Application
Very low	0-25	0.78	Roads-pavement
Low	25-50	0.85	Foundations
medium	25-100	0.92	Reinforced concrete
High	100-175	0.95	Reinforced concrete (highly reinforced)

Measurement of slump in fresh concrete is typically performed using the method presented in ASTM-C143 [31]. Australian code (AS-1012.3.1) [32] sets out the manner for determining the slump of concrete when the nominal size of aggregate does not exceed 40 mm. Figure (2.4) shows the equipment and types of the slump of concrete.

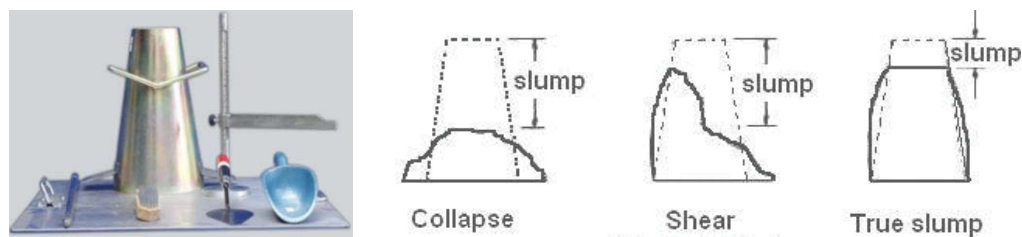


Figure 2.4: Test equipment and types of concrete slump

Dinakar (2013) [33] investigated the effect of water to binder ratio in regard to the workability of conventional concrete and fly ash LWC. He found that for a given binder ratio and the same super plasticizer dosage, the slump in LWC was recorded as slightly more than in the CC. This observation was the same for all water/binder ratios.

2.6.3.2. Air content

The entrained air content (V_{air}) per unit volume of the fresh concrete is a major factor in fresh concrete to achieve the target slump and mechanical properties. The entrained air in the LWC mixture is designed to resist the freezing and thawing cycles. Improved workability of concrete mixtures, especially those containing less water and cement, rough textured aggregates or lightweight aggregates are some other benefits of air-entraining process. Hence, air entrainment is a usual method of making mass concrete and lightweight concrete mixtures [17]. The air voids in LWC also make it less vulnerable to stiffness and strength deterioration compared to CC [34].

Moreno et al. (2014) investigated the effect of air voids (entrapped/entrained air) in compressive strength of LWA and verified their experimental results by Equation (2.1) [35].

$$f_{rel} = 10^{-3.84 V_{air}} \quad \text{Eq. (2.1)}$$

Where; f_{rel} is the relative compressive strength; and V_{air} is air-concrete ratio by volume.

The amount of entrained air recommended for LWC in ACI-211.2.98 [10] is limited as shown in Table 2.8. According to ACI-213R-03 [1], the air content of LWC should not vary by more than $\pm 1\%$ -1/2 % percentage points from the target value to avoid adverse effects on compressive strength, concrete density, workability, and durability.

Table 2.8: Allowed air content of lightweight concrete in ACI-211.2.98

Conditions	Air content (%)	Max. aggr. size (mm)
LWC subjected to freezing and thawing or to deicer salts	4-6	20
	4.5-7.5	10

The measurement of air content in the fresh concrete of normal-density is typically performed using the pressure method (ASTM-C-231) [36]. The volumetric method of measuring air, as described in ASTM-C173 [37], is the most reliable method of measuring air in LWC. This method relies on displacement of air with water in a vessel of pre-calibrated volume. Figure

(2.5) shows the testing equipment for Pressure method (left) and volumetric method to measure the air content in fresh concrete.



Figure 2.5: Testing equipment of pressure and volumetric method to measure air content

Australian standard (AS-1012.4) [38] sets out a method for determining the air content of freshly mixed concrete from observation of the change in volume of the concrete when it is subjected to an increased air pressure. In the absence of testing facilities, this standard also gives a simplified equation to find the air content of the concrete.

2.7. MECHANICAL PROPERTIES OF LIGHTWEIGHT CONCRETE

The early evaluation of hardened concrete properties is crucial and the compressive strength of concrete is a fundamental parameter to estimate its other mechanical properties. However, there is no direct relation to obtaining the compressive strength, and it has to be predicted by experimental studies, destructive and non-destructive tests. Hardening properties of concrete come directly from its fresh properties. The problem is that following the hardening process, the quality and mechanical properties cannot improve. Structural behaviour of concrete relies on mixing proportions and material properties of the composite system which cannot change after hardening. Therefore, obtaining a relation to predict the hardening properties from the fresh state and mixing proportions would be a great achievement in the concrete industry. However, there are numerous models of international standards and codes of practice for the early assessment of hardened properties of concrete at different ages. Sometimes these models give under or overestimated predictions and strongly affect the targeted design and serviceability aspects.

The present study investigates the time-dependent deflection of slabs made with LWC containing Expanded Polystyrene (EPS) beads. This type of lightweight aggregate is commercially called BST aggregates in Australia.

Despite considerable investigation of the mix proportioning and the relationship between mechanical properties and fresh state properties of conventional concrete in the literature, there are limited studies reported in the case of LWC, particularly LWC containing BST aggregates.

In this study, test results reported in the literature are used to evaluate the influence of LWC mixture components and proportioning of the mechanical properties. The collected mechanical properties will be compared with those of CC and Self-Compacting Concrete (SCC). The existing models for mechanical properties of LWC, CC and SCC are presented and their predictions for different properties will be compared. The applicability of the presented prediction models are evaluated in Chapter Three. Other available prediction models for CC and SCC are also assessed to check their capability of reasonably predicting the mechanical properties of LWC. In conclusion, new models are proposed in this study to predict the mechanical properties of LWC.

2.7.1. Compressive Strength

Compressive Strength (CS) gives an overall view of the quality of concrete. It is the most important specification for quality characterization and concrete classification. Usually, the other mechanical properties of concrete such as modulus of elasticity, stress-strain curve, tensile strength, flexural strength (modulus of rupture), and bond strength are expressed in terms of the compressive strength.

The compressive strength of structural LWC is specified according to the design requirements of the structure. Normally, compressive strength of lightweight concrete is ranging from 21 to 35 MPa; and less frequently up to 48 MPa or higher. Although some lightweight aggregates are capable of producing very high strengths consistently, it should not be expected that concrete made with every lightweight aggregate classified as “structural” can consistently attain the higher strength values.

Kim et al. (2012) [39] compared the effect of mixture design on mechanical properties of air entrained LWA and CC and reported the considerable influence of the entrained air content on compressive strength and other mechanical properties of LWC. The air content in the LWC matrix had a decreasing effect on compressive strength. The compressive strength increases with matrix content, for the aggregate used and for air/matrix ratios of approximately 22% and under; otherwise, compressive strength decreases.

2.7.2. Modulus of Elasticity

Modulus of Elasticity (MoE) is one of the most important characteristics of concrete that is required in every design and evaluation process of concrete structures. MoE is also an essential parameter in determination of the strain distributions and displacements, especially when the elasticity considerations govern the concrete structure design.

Due to the brittle behaviour of concrete, various parameters affect MoE. Topcu and Ugurlu (2007) [40] defined the effecting parameters as shown in Figure (2.6).

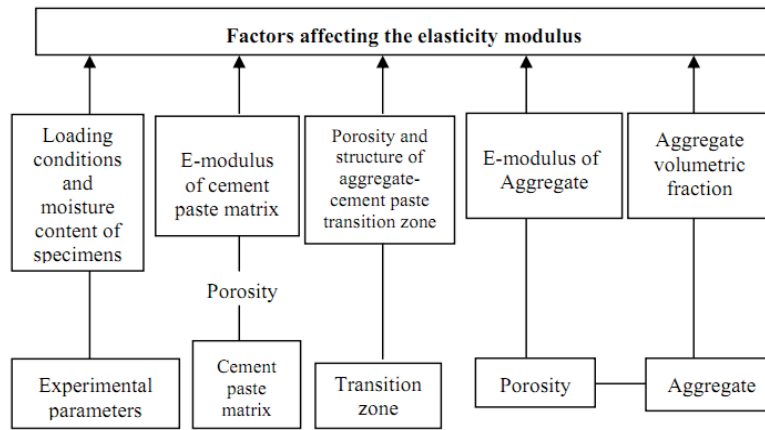


Figure 2.6: Classification of effective factors on modulus of elasticity of concrete

Two-phase (Mehta and Monterio (2006)) and three-phase (Nilsen and Monteir (1993)) models are the most frequent models for the researchers to explain the MoE of concrete. The third phase known as Interfacial Transition Zone (ITZ), deals with the interface between mortar and aggregate in concrete [35].

Nguyen et al. (2014) [41] studied the effect of mixture parameter and volume fraction on thermal and mechanical properties of LWC. According to test results of seven types of fine and coarse lightweight aggregates from three different environments in twelve mixtures, they showed a linear decrease in the modulus of elasticity and compressive strength in reducing the LWC density. Replacing normal-weight fine aggregate by lightweight fine aggregate by 100 kg/m^3 , decreases the compressive strength and the modulus of elasticity by 2.3-3.8 MPa and 1.7-2.6 GPa respectively.

Byrad et al. (2014) [42] compared the cracking tendency of LWC and CC in bridge decks. They used expanded shale, clay, and slate lightweight coarse and fine aggregates to produce the LWC. The results showed a decreasing tendency of MOE by increasing the amount of pre-wetted lightweight aggregate in the concrete. The significantly reduced MOE and coefficient of

thermal expansion also led to a considerable overall delay in early age cracking in bridge deck concrete.

2.7.3. Tensile Strength

Tensile strength of concrete is an effective parameter to evaluate the structural properties of LWC such as the bond strength, shear resistance, anchorage and resistance to cracking. Despite sufficient evidence about the effect of tensile capacity on total strength of concrete, it is usually neglected in design and evaluation of reinforced concrete structures. Due to this tensile capacity, concrete carries tensile strength between the cracks due to bond transfer between concrete and reinforcement [43]. Three types of tests can determine concrete uniaxial tensile strength including direct pull-out test, the modulus of rupture (flexural strength) test, and the splitting tensile (Brazilian) test. Among the existing methods, the method illustrated in ASTM C496/C496M-04 [44], is the most commonly used approach to determine the splitting tensile strength. In this method, a concrete cylinder specimen is laid horizontally between the loading platens of the testing machine and it is compressed along a vertical diameter. Figure (2.7) shows the general arrangement of splitting tensile strength.

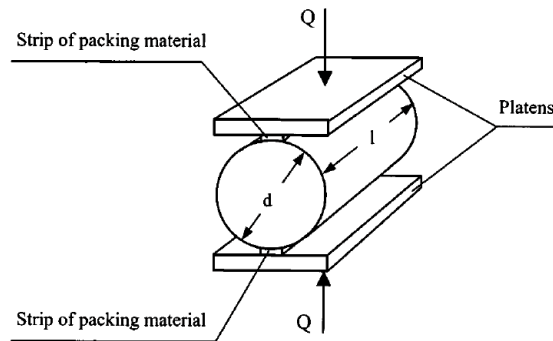


Figure 2.7: Brazilian concrete splitting tensile test setup

The lightweight aggregate content in the mixture influences the splitting tensile strength. According to Gesoglu et al. (2004) [45], the splitting tensile strength in LWC gradually decreases as the lightweight aggregate content increases from 30% to 60%. The maximum drop in splitting tensile strength was in silica fume concrete with water/binder ratio equal to 0.32.

Tang et al. (2008) [46] compared the mechanical properties of high-strength sintered fly ash LWC and crushed limestone CC and reported 25% lower splitting tensile strength in LWC. From a review study on the Oil Palm Kernel Shell (OPKS) LWC, performed by Alengaram et al. (2013) [47], inclusion of fly ash, silica fume and super plasticizer cause an improvement of

compressive strength from the typical range of 13-22 MPa to 37 MPa. Moreover, the compressive strength reaches up to 48 MPa by adding crushed OPKS and limestone powder to the LWC mixture. In this range, the splitting tensile strength was reported in the range of 6-10% of the compressive strength.

Dinakar (2013) [33] used the same mixture proportions by volume for water/binder ratios varying from 0.3 to 0.5 in LWC and CC mixtures and compared the mechanical properties. Use of the natural coarse aggregate and Lytag coarse aggregates were the only differences in the mixtures. As a result, they observed 7-9% variation of the ratio of splitting tensile/ compressive strength in both LWC and CC specimens that was beyond the range given in European code of practice, CEB-FIP.

2.7.4. Modulus of Rupture

In a reinforced concrete element subjected to flexural bending, immediately after the first crack appears there is a sudden significant change in strain of tensile steel bars. Modulus of rupture aims to accurately define of stress corresponding to the modification of concrete from linear elastic condition to non-linear plastic condition. The maximum normal stress applied by the ultimate bending moment under elastic behaviour assumption is defined as modulus of rupture. The illustrated method in ASTM-C78/ C78M-10 [48] with the general arrangement of the test specimen and flexural loading equipment shown in Figure (2.8) is used to define the modulus of rupture in plain concrete.

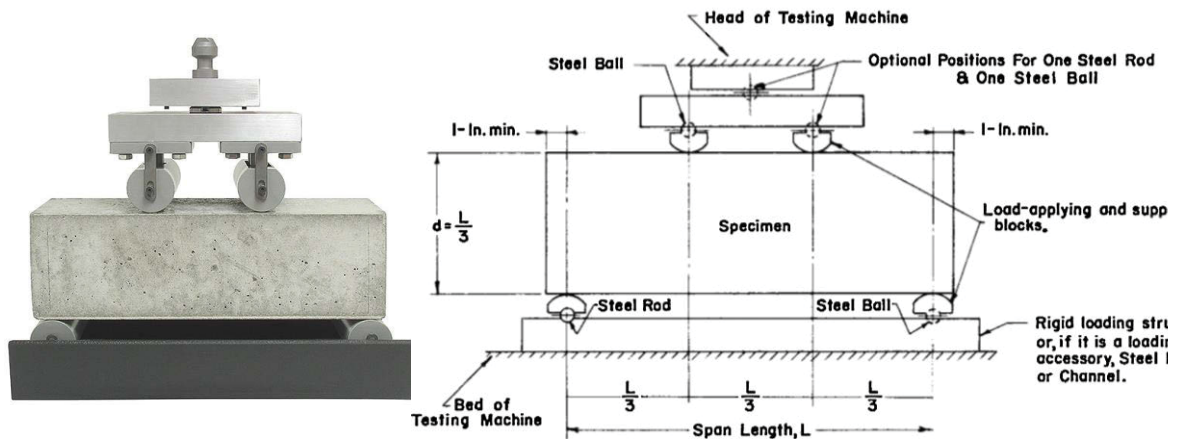


Figure 2.8: ASTM-C78 beam specimen loaded at third-point loading [48].

Tassew and Lubell (2012) [49] investigated the mechanical properties of Light-Weight Ceramic Concrete (LWCC) with a magnesium potassium phosphate binder. This binder was applied to achieve the faster strength gain and lower the overall environmental impacts on the concrete. Their experiment revealed that regardless of the aggregate type, the higher

water/binder ratio produces larger modulus of rupture in LWCC. Based on the existing model in ACI-213(2003) [1] for Portland cement LWC, they also developed an empirical model with a reduced coefficient for LWCC.

Alengaram et al. (2013) [47], concluded that in a high strength LWC made with Oil Palm Kernel Shell (OPKS) LWC, the flexural tensile strength is 8-14% of the compressive strength.

Casanova-del-Angel and Vazquez-Ruiz (2012) [50] studied the production of LWC by replacing the stone coarse aggregate with Poly-Ethylene Terephthalate (PET) lightweight aggregate. Table (2.9) presents the recorded values of compressive strength and modulus of rupture in CC and LWC. It is evident that despite the same range of compressive strength, the modulus of rupture in LWC is lower than that of CC, particularly at higher strength classes.

Table 2.9: Compressive strength and modulus of rupture of conventional and lightweight concrete [50]

	Conventional concrete				Lightweight concrete			
Compressive strength (MPa)	14.99	18.82	24.33	30.69	14.38	18.52	23.12	29.71
Modulus of rupture (MPa)	2.5	2.88	3.21	3.56	2.34	2.64	2.71	2.83

2.7.5. Bond Characteristics of Lightweight Concrete

Due to the presence of steel in reinforced concrete members, bar-concrete bond and interaction consideration is inevitable. The concrete-reinforcement bond between two adjacent cracks carries a certain amount of the tensile force normal to the cracked plane is illustrated by tension stiffening that contributes to the overall stiffness of the member. Since bond stresses in reinforced concrete members arise from the change in the steel force along the length, the effect of the bond becomes more pronounced at end anchorages of reinforcing bars and in the vicinity of the cracks. Figure (2.9) shows a schematic deformation of concrete in a tension region in the vicinity of the deformed bar. Figures (2.10 A, B) show the idealized distribution of stress and strain related to steel-concrete bond and the crack development in sections subjected to flexural moment and direct tension respectively. It is, in fact, idealized distribution of stress and strain between cracks of bond stress, concrete tensile stress and steel tensile stress respectively. As shown in Figure (2.10 B-f), since concrete is carrying some tension between the cracks, the flexural rigidity is clearly greater between the cracks, than at the cracks [51].

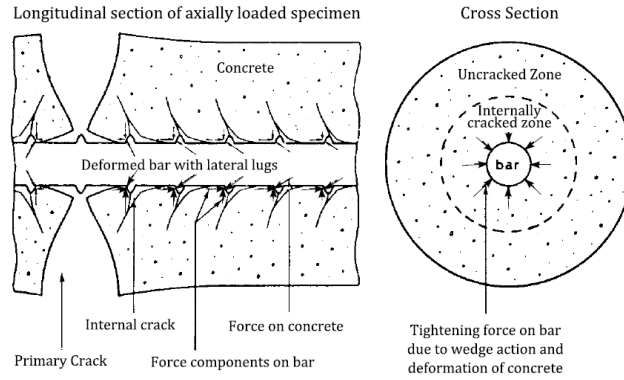


Figure 2.9: Schematic deformation of concrete in a tension specimen

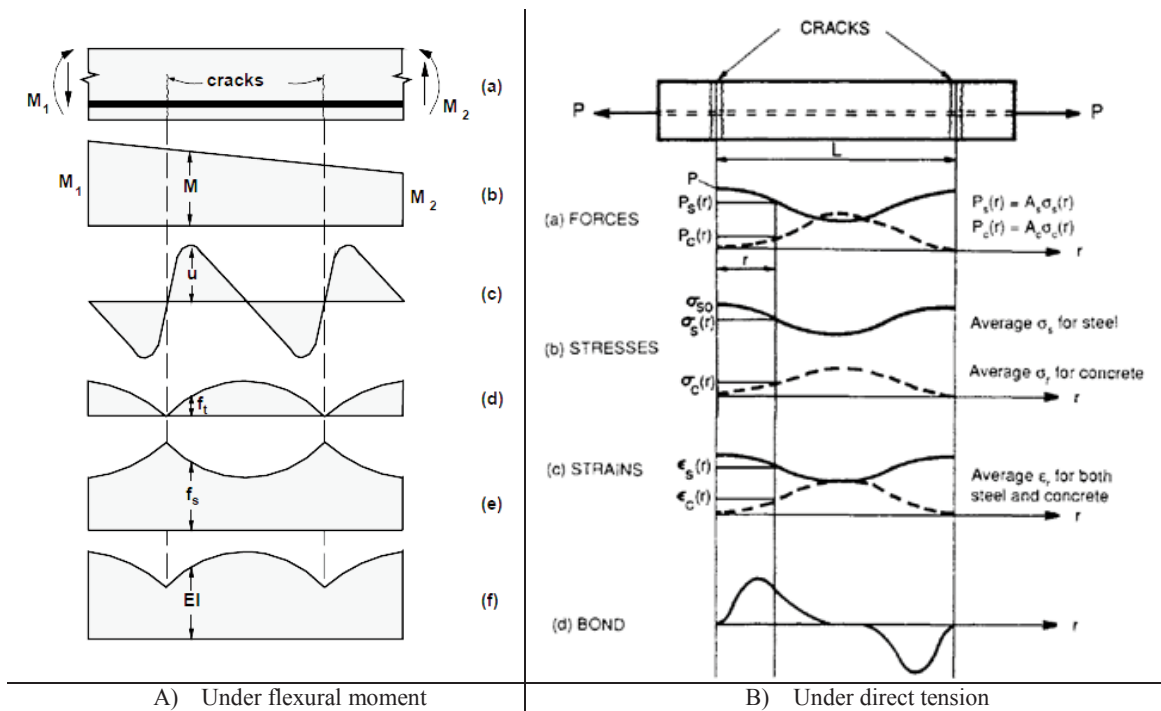


Figure 2.10: Effect of cracking in reinforced concrete member, A) a-portion of member; b- bending moment distribution; c- bond stress distribution; d- concrete tensile stress distribution; e- steel tensile stress distribution; f- flexural stiffness and B) a-forces; b-stress distribution; c- strain distribution and d- bond variation between steel and concrete [51]

From the tension stiffening concept, concrete does not crack suddenly and completely, but undergoes progressive micro-cracking (strain softening). Due to the bond between steel and concrete, the intact concrete between adjacent primary cracks carries considerable tensile force. This bond significantly makes the stress-strain characteristics of tensile concrete in cracked reinforced concrete structures different from the stress-strain characteristics of the plain concrete.

The bond stress between steel and concrete is the combination of friction, adhesion and the support of the ribs in deformed reinforcing steel. When adhesion fails, some relative movement

between steel and concrete and consequently cracking happen and the friction and steel ribs go into action.

Chen et al. (2004) [52] compared the bond behaviour of LWC and CC in two groups of low and high strength concrete. They presented the following results from the tests:

- Shear stirrups in the section improve the nominal bond strength in both LWC and CC about 20%. However, its effect on critical bond strength is less than 20%;
- The bond failure surface through the lightweight aggregates particularly in low-strength LWC, while in CC specimens most aggregates are not broken. For higher strength levels over 40 MPa, the cracking surface is similar in both types of concrete;
- Corresponding bond strength in LWC is relatively lower than that for CC. However in high-strength LWC, the bond strength is greater than that of CC, due to higher mortar strength.

Sancak and Simsek (2001) [53] compared the steel-concrete bond in LWC produced by pumice aggregate with conventional concrete. By replacing mineral additives and silica fume in different ratios with Portland cement, they found the negative effect of silica fume on the bond strength of LWC. Also, the bond strength of non-additive LWC in their experiment was 24% lower than that of CC.

Alengaram et al (2010) [54] compared the bond properties of grade 30 LWC, namely Oil Palm Kernel Shell Concrete (OPKSC) with CC of similar strength. The experimental observation showed a weaker bond between OPKSC and cement matrix, compared to the bond between crushed granite aggregate and cement matrix in CC. They also reported that the steel-concrete bond stress of OPKSC was 86% of the same grade CC.

2.8. FRACTURE ENERGY

Fracture energy is defined as the energy required to open unit area of the crack surface (complete separation of a concrete body) and is found to be increasing as we approach crack tip. It is a material property; however there are some studies in the literature of investigations of the size effect of structures on the fracture energy. Fracture energy can be expressed as the sum of surface creation energy and surface separation energy. It is reprehensive for cracking resistance and fracture toughness of concrete and therefore, is considered as a material property of concrete [55].

According to the energy balance principle, the action of force P on its displacement will be stored in two forms of energy; one part, as strain energy and the other part as used for crack

propagation. Equation (2.2) shows the relation between applied work and stored energy. Figure (2.11) also displays the energy consumption for the entire cracking period.

$$G_{Fi}A_i = W_i - \Delta U_i = \int_0^{\delta_i} P d\delta - \Delta U_i \quad \text{Eq. (2.2)}$$

Where G_{Fi} is the average energy required for unit crack propagation from crack initiation until any time; A_i is the newly formed fracture area for this period; W_i is the applied work of external force P ; U_i is the elastic strain energy, and δ_i is the crack opening displacement

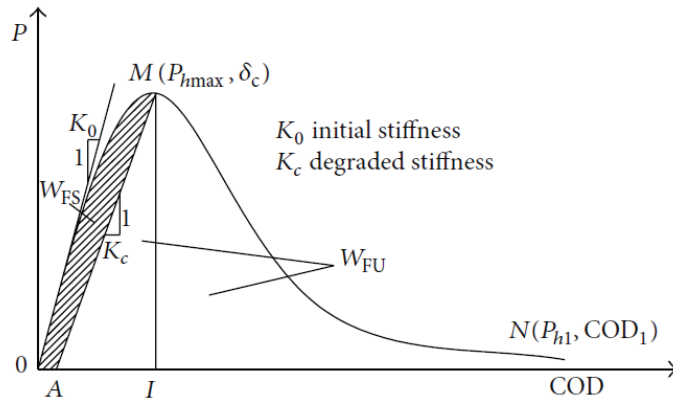


Figure 2.11: Energy consumption for entire crack propagation

Although the “work-of-fracture” method recommended by RILEM [56] is dependent on the size and shape of the structure, it is still the most commonly used method to measure the fracture energy of concrete.

Kaplan (1961) initiated the study of fracture mechanics in concrete and Kesler (1971) and Walsh (1972) reported the inadequacy of classical Linear Elastic Fracture Mechanics (LEFM) for concrete structures. Hilleberg et al. (1976, 1985) made a major advance in the fracture energy estimation and improved the developed the crack model (fictitious crack model) to concrete. This model predicts the flexural failure of un-notched plain concrete beams. Peterson (1981) refined and improved this model. Bazant (2002) presented a brief history of concrete fracture models and studied the size effect on the fracture energy [57].

Sim et al. (2014) [23] compared the effect of aggregates and specimen size on the fracture energy of LWC and CC beams. They tested four types of beam according to RILEM (1985) [56]. As a result, the fracture energy in LWC was lower than that of CC. It was tending to increase as the maximum size of aggregate increased up to 8 mm, beyond which it remains

constant. It is an indication of minor or no contribution of the aggregate interlock to the fracture energy of LWC with the maximum aggregate size above 8 mm.

Dehn (2004) [58] investigated the fracture mechanics of 15 different LWC mixtures with dry densities ranging from 1300 to 1800 kg/m³ and low to high-strength range between 20-100 MPa. They found the effect of coarse aggregate on the maximum splitting tensile strength and fracture energy. Also, the results showed that the fracture mechanism is not dependent to the type of utilized lightweight aggregate. Since the brittleness of LWC increases with decreasing the grain density of the coarse and fine aggregate, they compared conventional concrete by Sand Light-Weight Aggregate Concrete (SLWAC) and All Light-Weight Aggregate Concrete (ALWAC). Natural aggregates own a much higher resistance to cracking than porous and lightweight aggregates; this is why ALWAC can transmit tensile stress over shorter cracks than SLWAC. Figure (2.12) shows the effect of natural coarse aggregate, sand and lightweight aggregate on the roughness, internal friction and cracking of the concrete.

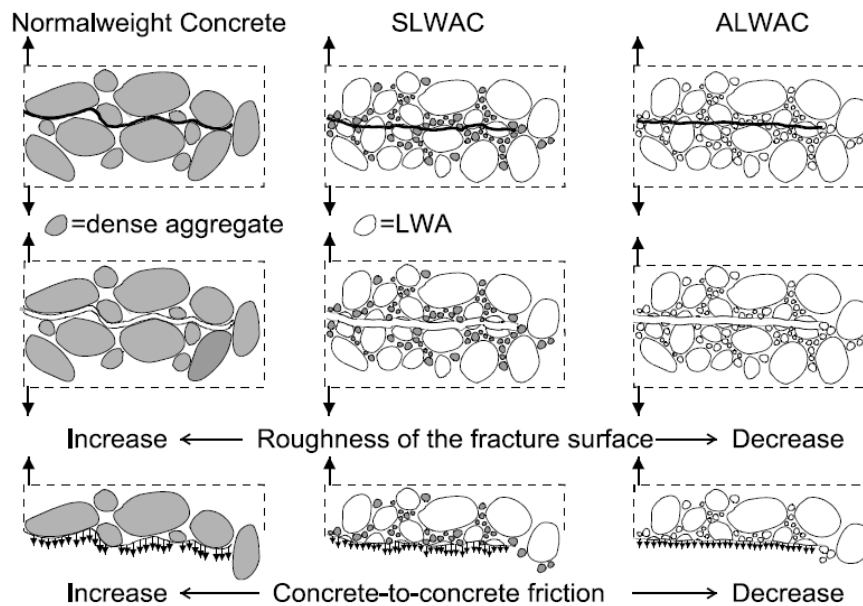


Figure 2.12: Influence of the aggregates on the softening behaviour of lightweight concrete

2.9. SHRINKAGE AND CREEP OF LIGHTWEIGHT CONCRETE

2.9.1. Shrinkage of Lightweight Concrete

2.9.1.1. Drying shrinkage

Withdrawal of water from the concrete stored in the unsaturated air causes the drying shrinkage. A part of this movement is irreversible and should be distinguished from the reversible moisture movement caused by alternating storage under wet and dry conditions. In

this type of shrinkage, based on the thermodynamic equilibrium principle between moisture evaporation and the environment, the shrinkage must stop after all the moisture has evaporated.

2.9.1.2. Autogeneous shrinkage

The chemical processes of Portland cement hydration leads to another type of shrinkage, called the autogeneous shrinkage, which is observed in sealed specimens, i.e., at no moisture loss. It is caused partly by chemical volume changes, but mainly by self-desiccation due to loss of water consumed by the hydration reaction. This type of shrinkage is a limited part of the total shrinkage and terminates once all the constituents have reacted. It amounts to only about 5% of the drying shrinkage in normal concretes, which self-desiccate to about 97% pore humidity. However, it can equal the drying shrinkage in modern high-strength concretes with very low water-cement ratios, which may self-desiccate to as low as 75% humidity [59].

Figure (2.13) shows the possible shrinkage strain (ϵ_{sh}) curves for specimens of various concrete types with identical size, shape, age and environment.

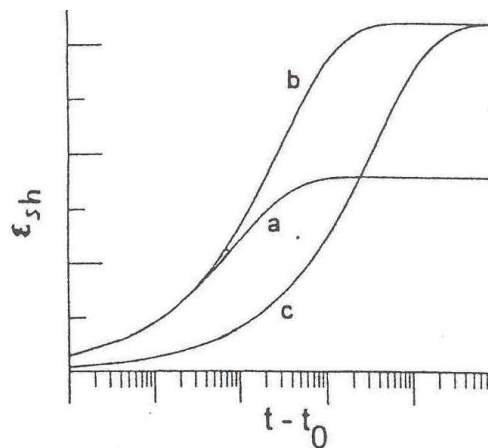


Figure 2.13: Possible shrinkage curves for specimens of various concretes

Considering the effect of mineralogical character of aggregate on shrinkage, Troxell et al. (1958) [60] showed that the shrinkage of concretes containing limestone or quartz aggregate in CC is only about half that of concrete containing sandstone aggregate.

Best and Polivka (1959) [61] conducted 18 months monitoring of shrinkage in expanded shale LWC specimens having compressive strengths of 20 and 35 MPa. They found considerably high drying shrinkage and deformation compared to the sand/gravel CC.

According to Neville (1995) [62], the lightweight aggregate results in higher shrinkage values than normal-weight aggregate. However, Nielsen and Aitcin (1992) [63] found different results

as the drying shrinkage of LWC incorporating expanded shale was 30% to 50% lower than the companion CC after 28 days of curing and 56 days of drying. For LWC with dense fine aggregates, Newman (1993) [64] found similar shrinkage performance with the CC. They also observed less shrinkage cracking in LWC due to the relief of restraint by creep and the continuous supply of moisture from the pores of the lightweight aggregate. The experiment by Kayali et al. (1999) [65] was in agreement with Neville (1995) [62]. They tested LWC made of Lytag aggregates (sintered fly ash) and observed that the drying shrinkage (about 1000µε) was nearly twice that of CC after 400 days.

From a series of experimental tests on expanded shale LWC and CC, Reichard (1964) [66] revealed that the average one-year shrinkages of the lightweight and normal-weight concrete were 720 and 500 micro strain respectively.

Gesoglu et al. (2004) [45] investigated the free and restrained shrinkage behaviour of LWC specimens with different ratios of water/cementitious material (w/c) and different volumes of coarse aggregates in the mixture. The results showed a significantly poor shrinkage cracking of LWC compared to CC. Also, the mixtures containing the higher volume of lightweight aggregates showed higher shrinkage during the first 50 days. By comparing the effect of w/c ratio, the concrete with lower w/c had higher shrinkage, in which the addition of silica fume reduced the difference noticeably.

Shekarchi et al. (2012) [67] tried to develop a method of predicting the long-term shrinkage from short-term measurements of four different mixtures of concrete. They reported the inadequacy of the existing models for all types of concrete, emphasizing that individual coefficients with the general form of Equation (2.3) are required to apply to each type of concrete.

$$\varepsilon_{sh}(t) = \alpha C_1 (f(t))^{C_2} \quad \text{Eq. (2. 3)}$$

Where; α : coefficient of concrete type; $f(t)$: time-function term; C_1 : modification factor and C_2 is used to modify the time-function term.

2.9.2. Creep of Lightweight Concrete

Under sustained load, concrete continues to deform. This phenomenon is called creep. Creep, along with instantaneous (or elastic) strain, is defined as the strain difference between the loaded and load-free specimens. Creep can be defined as the increase in strain under sustained stress. The load-induced strain under conditions of no moisture exchange is known as basic

creep. However, if a specimen is also drying while under load, there is an increase in creep and the extra creep is called drying creep.

Figure (2.14) shows the schematic set-up for creep test in laboratory condition.

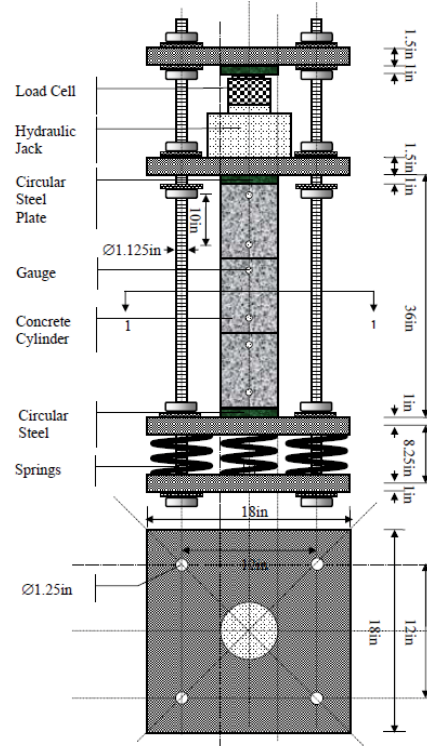


Figure 2.14: Creep test set-up

The creep of concrete, and particularly its dependence on the age of loading and water content, has proven to be a challenging problem. Bazant (2001) [68] introduced other effective parameters in creep of concrete as followings:

- Plastic flow;
- Consolidation theory;
- Load-bearing hindered adsorbed water;
- Bond breakage in slip and its reformation;
- Nonlinear deformations and cracking as a contribution of the Pickett effect;
- Solidification theory for short-term aging;
- Micro-prestress of creep sites in cement gel microstructures.

Hatt (1907) discovered the creep phenomena in concrete. Despite considerable efforts and achievement by the well-known experts such as Troxell et al. (1958) [60], Pickett, McHenry, L'Hermite et al. (1965), Arutyunian, Aleksandrovskii (1959), Powers, Hansen and Mattock (1966), Neville et al., (1983); the creep of concrete is not yet fully understood [69].

Best and Polivka (1959) [61] adjusted the creep relation proposed by Shank (1953), for LWC. According to their findings, LWC exhibits lower modulus of elasticity than CC and therefore, greater elastic strain under a given load.

From a series of experimental tests on CC and LWC samples containing expanded shale, Reichard (1964) [66] revealed that in general, the CC exhibits less creep and shrinkage than the LWC. However, some lightweight concretes exhibited less creep and shrinkage than some normal-weight concretes. He loaded the LWC specimens at 28 hours and CC specimens at different ages. The average adjusted creep of LWC was 0.145% of that at 365 days, while the corresponding average creep for CC was 0.106%. Considering the cement content in the mixture, he also found that in general, the amount of cement needed to develop a given strength has increasing effect on the "creep plus shrinkage".

Different lightweight aggregates show different creep behaviour. According to reported results of Swamy and Ibrahim (1973) [70], Solite LWC shows the same order for creep as for Lytag LWC made with rapid hardening cement, however, it was about 20-37% less than that for creep of expanded clay (Aglite) LWC. They confirmed the results reported by Ross (1937) [71] and Lorman (1940) [72] and found a linear relationship between creep and stress/strength ratio for some value between 0.33 and 0.5.

Arnaouti and Sangakkara (1984) [73] performed an experiment to propose a model to predict the creep and shrinkage of a large reinforced LWC structure from the shrinkage data of a small plain LWC prism. Gamble and Parrott (1978) [74] concluded a linear relationship between drying creep and concurrent shrinkage of LWC specimen. In their experiment, the basic creep after 30 days was not significant.

The creep of LWC and CC is considered as an independent parameter of the stress history. However, Tasdemir et al. (1988) [75] performed some studies to predict the creep strain under various stress levels and found that the history of loading is affecting the creep strain and there is a good logarithm correlation between the age of loading and the creep function.

Birjandi and Clarke (1993) [76] studied the long-term deflection behaviour of reinforced LWC beams using two types of lightweight aggregate namely Lytag and Leca. They found that parallel calculation for $\pm 20\%$ higher or lower shrinkage and creep coefficients produces only $\pm 6\%$ difference in the final deflection values of LWC beams.

Figure (2.15) shows the general presentation of possible time-dependent strains and their general comparison.

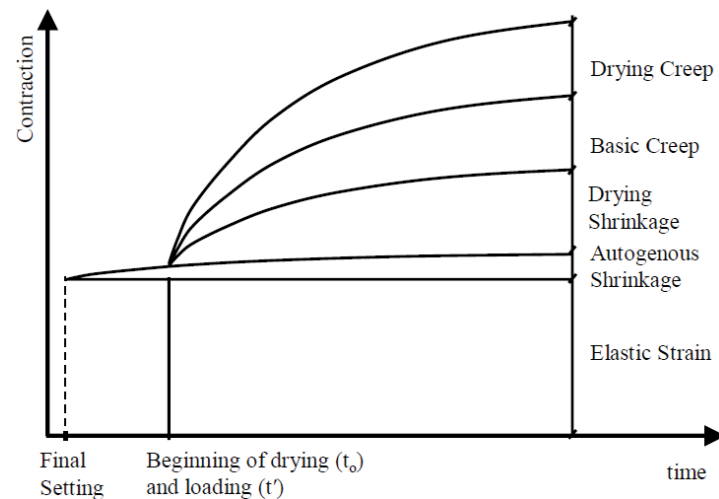


Figure 2.15: Various strains in concrete with time

2.10. REAL-SCALE TIME-DEPENDENT DEFLECTION STUDY

Despite numerous examples in the literature of the long-term deflection of conventional concrete structures, the investigation on the long-term deflection of LWC beams is very rare. This section describes the summary of that experimental study in terms of material properties, test methods, geometry of specimens and the obtained results.

To cover the general objective of this study, the time-dependent deflection on other types of concrete are presented in Chapter Four, and the results are compared with the time-dependent deflection of LWC slabs.

2.10.1. Birjandi and Clarck (1993), Deflection of Lightweight Aggregate Concrete Beams [76]

2.10.1.1. Objectives

The main objectives of the study were:

- Long-term deflection of LWC beams over a period of six months;
- To cover different densities of LWC beams and comparing the results with CC beams;
- To compare the recorded deflection with estimated deflection by using the requirements in BS 8110: Part 2 [77].

Specimens: A total of 12 beam specimens with the same length (3.0 m) and section dimensions (100 mm wide and 200 mm deep) were made for two ratios of tensile reinforcement and three types of concrete. The minimum concrete cover in all beams was 15 mm.

2.10.1.2. Materials

Concrete: Details of the LWC and CC mixture used in the study is given in Table (2.10). All beams were cast in timber moulds, demoulded after 24 hours and then cured under a plastic chamber until the loading age. Humidity and temperature of the lab during the test was kept at 40-60% and 20°C respectively.

Table 2.10: Mixture proportions and the maximum aggregate size of beams

Mixture	Coarse aggregate	Surface dry mixture proportions (kg/m ³)			
		Coarse aggr.	sand	Water	OPC-42.5
LWC-1	Lytag - sintered PFA (3-12 mm)	637	789	241	277
LWC-2	Leca - expanded clay (3-10mm)	305	777	197	447
CC	Normal aggregate (10mm)	930	910	200	270

2.10.1.3. Reinforcement

Tensile reinforcement: two arrangements of ribbed tensile reinforcement (each in six beams) were used in the study. In the first group, 2N10 giving the reinforcement ratio of 0.9% and in the second group, 2N8 giving the reinforcement ratio of 0.58% were used as tensile bars.

Compression reinforcement: In all beams, 2N6 ribbed bars were used in the compression side.

Shear reinforcement: To resist the applied shear in all beams, R6@175 mm mild plain bars were used.

2.10.1.4. Test arrangements and method

Each pair of beams of the same mixture were placed back-to-back (Figure 2.16) with their tension sides facing each other and simply supported. They were then loaded centrally, using a spring system to maintain the applied load as approximately constant. The sustained loading of 7 KN and 10 KN was provided by a hydraulic jack and was kept for duration six months.

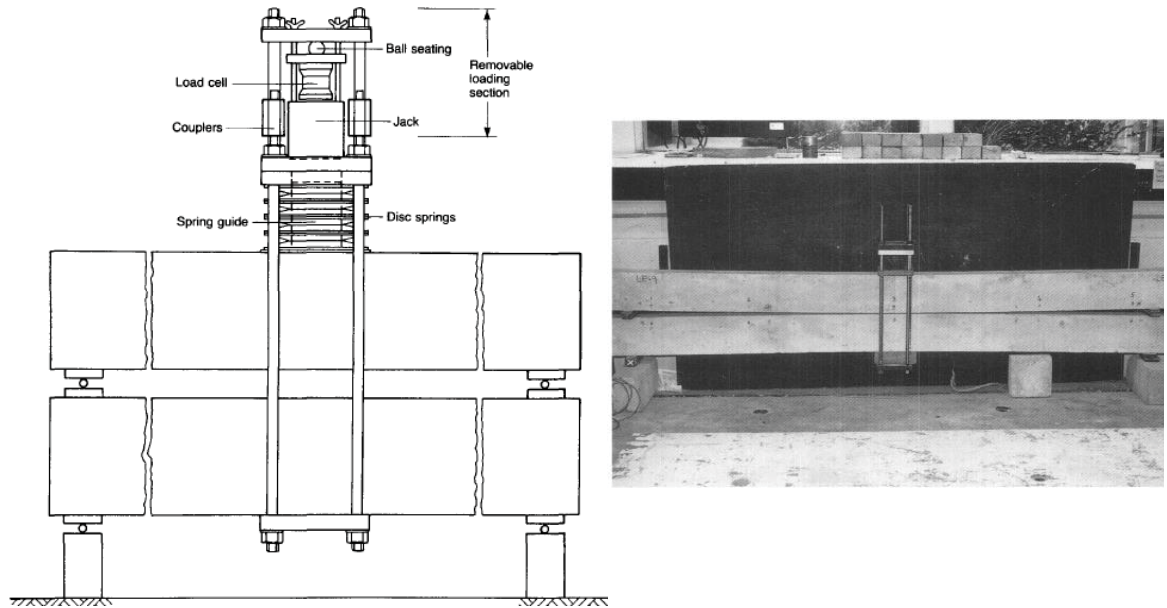


Figure 2.16: Test setup and loading arrangement of the LWC and CC beams [76]

Deflection of each specimen (the pair of beams) was measured at 5 points (5 opposite points on top and bottom beam) along the lengths including the mid-span. The total deflection of each spot was measured between two opposite points on the beams on a vertical line. Thus, the measured value was the sum of the deflection at two beams.

2.10.1.5. Experimental results

The measured mechanical properties of the mixtures are presented in Table (2.11). Table (2.12) shows the recorded and estimated instantaneous deflection (Δ_{ins}) and long-term deflection (Δ_{long}) after six months in the beams.

Table 2.11: Mechanical properties of concrete and tensile bar ratio in LWC and CC beams

Property	Curing	Beam specimen					
		CC-a	CC-b	LWC-1a	LWC-1b	LWC-2a	LWC-2b
Steel ratio %		0.58	0.9	0.9	0.9	0.58	0.58
f_{cu} (MPa)	Air	29.6	28.9	35.6	38.1	31.4	31.4
	water	31	27.8	33.1	35	32.7	31
f_t (MPa)	Air	2.69	2.63	2.73	2.97	2.37	2.55
	water	2.64	2.85	2.86	2.97	2.89	2.73
Density (kg/m ³)	Air	2230	2210	1810	1810	1650	1630
	water	2280	2270	1960	1970	1720	1700
E_c (GPa)		25.7	24.6	17.4	18.9	18.5	17.2
Estimated creep coefficient		1.8	1.8	1.3	1.3	1	1
Estimated shrinkage ($\mu\epsilon$)		300	300	400	400	400	400

Table 2.12: Recorded and calculated deflection of beams

Property		Beam specimen					
		CC-a	CC-b	LWC-1a	LWC-1b	LWC-2a	LWC-2b
Applied load (KN)		7.1	10	7.1	10	7.1	10
Δ_{ins}	Experimental	9.7	14.3	9.5	15.4	14.6	21.6
	Theoretical (measured E_c)	13.5	14.9	10.9	15.9	14.5	22.1
	Theoretical (estimated E_c , BS-8110) [77]	13.5	14.7	11.4	16.8	16.2	23.9
Δ_{long}	Experimental	20.8	24.1	17.8	25.3	21.9	30.4
	Theoretical (measured E_c)	22.7	24.7	21.3	27.2	24.6	33.2
	Theoretical (estimated E_c , BS-8110) [77]	22.7	24.5	22.1	28.7	27	35.8

The ratio of long-term deflection to instantaneous deflection ($\Delta_{long} / \Delta_{ins}$) for LWC and CC beams are presented in Table (2.13). In average, the $\Delta_{long} / \Delta_{ins}$ ratio in CC is higher than that of LWC beams. In addition, this ratio is the smallest in Leca LWC beams. The theoretical methods predict the deflection with $\pm 20\%$ accuracy. The ratio estimated for LWC by the British standard, BS-8110 [77], particularly for the lower density Leca LWC is the smallest among all the beams.

Table 2.13: Ratio of long-term to instantaneous deflection of LWC and CC beams

Property		Beam specimen					
		CC-a	CC-b	LWC-1a	LWC-1b	LWC-2a	LWC-2b
$\Delta_{long} / \Delta_{ins}$	Actual	2.144	1.685	1.874	1.643	1.5	1.407
	Theoretical (measured E_c)	1.68	1.658	1.954	1.71	1.697	1.5
	Theoretical (estimated E_c , BS-8110) [77]	1.68	1.67	1.938	1.7	1.667	1.498

Figure (2.17) compares the time-dependent deflection of Leca LWC and Lytag LWC with CC.

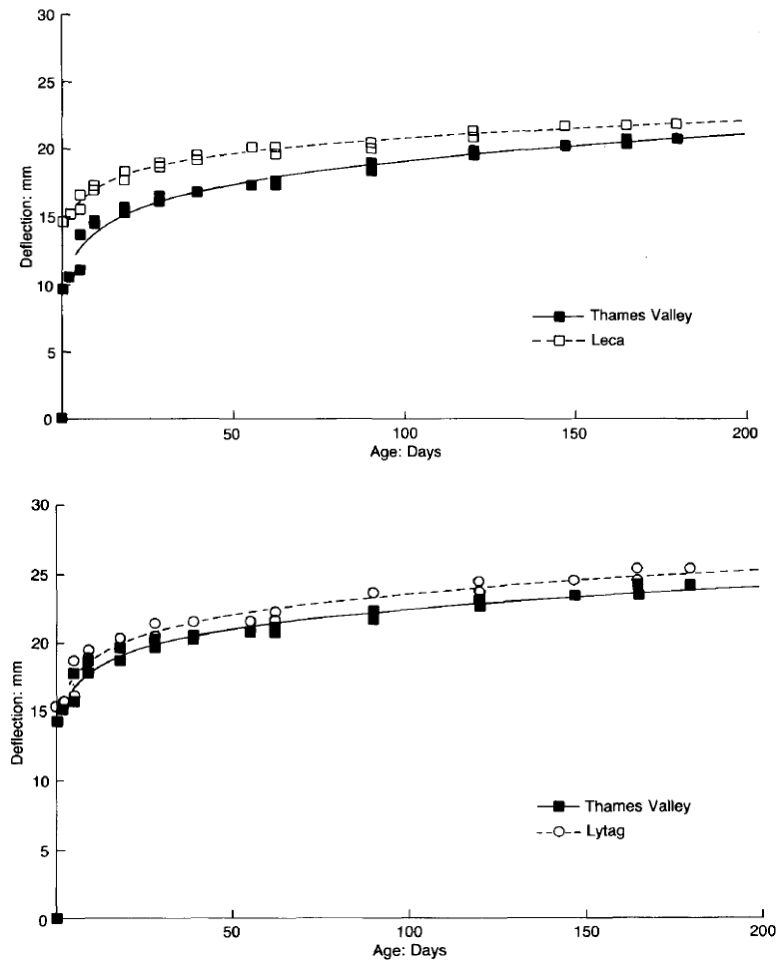


Figure 2.17: Deflection increase of LWC and CC beams with time [76]

2.11. SUMMARY

A brief history of the invention and development of lightweight concrete, together with the current classification of lightweight concrete in codes of practice are given. Also, the definitions and the minimum requirements of structural and non-structural lightweight concrete in the frequently used design codes such as ASTM-C330, ACI-211.2-98, ACI.213R-03, ACI.213R-14, EN-206.1-03, TS-2511-97 and AS-3600-09 are given.

Investigations of performance and economic aspects of LWC are the concerns in producing LWC with new types of natural and artificial lightweight aggregates. Various sources of natural or artificial lightweight aggregates used to produce the lightweight concrete and mixing methods are explained broadly. Advantages and disadvantages of the lightweight concrete regarding the economic issues, mixing and preparation, technical performance, fire protection, seismic load bearing, and some other effective parameters in the selection of the proper concrete type for construction projects are discussed.

The definitions for structural and non-structural LWC in some international standards are compared. Furthermore, the criteria for choosing the proper aggregates in LWC are discussed. Fresh and hardened state characteristics of LWC are studied and the common methods of testing and instruction are given in the first part of this chapter. Fresh LWC density, air entrained content, workability, segregation and slump are some fresh state properties that are discussed and compared with those for Conventional Concrete (CC). According to the investigated case studies, LWC owns better workability than CC, however due to presence of lightweight aggregates; there is a risk of segregation in LWC.

A review of the properties of fresh and hardened lightweight concrete and the relevant testing methods are given. The workability, segregation, slump and air content are the topics related to the fresh lightweight concrete. Also, the compressive strength, modulus of elasticity, tensile strength, modulus of rupture, bond characteristics, and fracture energy are among the most important mechanical properties of lightweight concrete which are discussed and compared to the corresponding properties of the conventional concrete. The compressive strength and density of LWC are less than those of CC in general. Besides, the mechanical properties of LWC in the same range of compressive strength in conventional concrete are of lower values in LWC.

Considering the time-dependent properties, the creep and shrinkage in lightweight concrete are compared with those of the conventional concrete. In general, the shrinkage and creep effects are more considerable in LWC due to its higher water absorption and reduced elasticity in comparison to CC.

Despite the limited number of long-term experimental studies on the mechanical and time-dependent properties of lightweight concrete structures in the literature, collection of all the relevant experimental investigation as a means of strengthening the basis of the current research has been attempted. Therefore, the most relevant experimental studies and the obtained results are discussed. An intention of this study was to investigate the instantaneous and long-term deflection of LWC slabs, however after a comprehensive review, only one example of related case study was found on LWC beams in literature. Some highlights of that experiment in terms of test setup, materials and specimens and the obtained results are presented and evaluated.

There are more investigations of other types of concrete in the literature that will be presented and compared in Chapter Four of this study.

REFERENCES

1. ACI-213R-03, *Guide for Structural Lightweight-Aggregate Concrete*. ACI Committee 213, American Concrete Institute, Farmington Hills, Mich. 48333-9094, 2003.
2. Vakhshouri, B. and S. Nejadi, *Mix design of light-weight self-compacting concrete*. Case Studies in Construction Materials, 2016. **4**: p. 1-14.
3. Nicolas Ali Libre, et al., *Mechanical properties of hybrid fiber reinforced lightweight aggregate concrete made with natural pumice*. Construction and Building Materials 2011. **25**: p. 2458–2464.
4. A.M, N., *Properties of Concrete*. Pitman Publishing , 3rd Edition, 1981.
5. ACI-213R-87, *Guide for Structural Lightweight Aggregate Concrete*. ACI Committee 213, American Concrete Institute, Farmington Hills, Mich. 48333-9094, 1999.
6. Miled K., et al., *Compressive behavior of an idealized EPS lightweight concrete: size effects and failure mode*. Mechanics of materials, 2004. **36**(11): p. 1031–1046.
7. Vakhshouri, B. and S. Nejadi, *Self-compacting light-weight concrete; mix design and proportions*. Structural Engineering and Mechanics, 2016. **58**(1): p. 143-161.
8. Chandra Satish and B. Leif, *Lightweight aggregate concrete, science, technology and application*. William Andrew Publishing/Noyes, 2002.
9. ASTM-C330/C330M-1, *Standard Specification for Lightweight Aggregates for Structural Concrete*. American Society of Testing and materials (ASTM), 2014.
10. ACI-211.2-98, *Standard Practice for Selecting Proportions for Structural Lightweight Concrete*. ACI Committee 211, American Concrete Institute, Farmington Hills, Mich, 1998.
11. Yang, K.-H., G.-H. Kim, and Y.-H. Choi, *An initial trial mixture proportioning procedure for structural lightweight aggregate concrete*. Construction and Building Materials, 2014. **55**: p. 431-439.
12. ACI-213R-03, *Guide for Structural Lightweight-Aggregate Concrete*. ACI Committee 213, American Concrete Institute, Farmington Hills, Mich. 48333-9094, 2003.
13. TS-2511, *Mix Design for Structural Lightweight Aggregate Concrete*. Turkish Standards Institution (TSI), Turkey, Ankara (in Turkish), 1977.
14. EN-206-1, *Specification, performance, production and conformity*. European Standard, 2000: p. 72.
15. AS-3600-09, *Concrete structures*. Australian Standard, 2009.
16. S. Kivrak, M.T., M. I. Onur, G. Arslan and O. Arioz, *An economic perspective of advantages of using light weight concrete in construction*. 31st Conference on OUR WORLD IN CONCRETE & STRUCTURES: 16 - 17 August 2006, Singapore, 2006.
17. Schackow, A., and, et al., *Mechanical and thermal properties of lightweight concretes with vermiculite and EPS using air-entraining agent*. Construction and Building Materials, 2014. **57**: p. 190-197.
18. M., R.A. and A.S. Nowak, *Resistance Factors for Lightweight Concrete Members*. ACI Structural Journal, 2014. **111**(1).
19. El Zareef, M.A., *Conceptual and structural design of buildings made of lightweight and infra lightweight concrete*. PhD thesis, Berlin University of Technology, 2010.
20. Babu, D.S., *Mechanical and deformational properties, and shrinkage cracking behavior of lightweight concretes*. PhD thesis, national university of Singapore, 2008.
21. Abdulkareem, O.A., et al., *Effects of elevated temperatures on the thermal behavior and mechanical performance of fly ash geopolymers paste, mortar and lightweight concrete*. Construction and Building Materials, 2014. **50**: p. 377-387.

22. Newman, J., Choo, B. S. and Owens, P, *Advanced Concrete Technology Processes*. Elsevier Ltd, 2003.
23. Sim, J., Yang, K., Lee, E., and Yi, S, *Effects of Aggregate and Specimen Sizes on Lightweight Concrete Fracture Energy*. J. Materials in Civil Engineering, 2014. **26**(5): p. 845-854.
24. Koehler E.P. and F.W. D., *ICAR Project 108: Aggregates in Self-Consolidating Concrete. Aggregates Foundation for Technology, Research, and Education (AFTRE)*. International Center for Aggregates Research (ICAR), The University of Texas at Austin, 2007.
25. Neshat, M., et al., *A Comparative Study on ANFIS and Fuzzy Expert System Models for Concrete Mix Design*. IJCSI International Journal of Computer Science Issues, 2011. **8**(3).
26. Hosseini Kupaei, R., et al., *Mix design for fly ash based oil palm shell geopolymers lightweight concrete*. Construction and Building Materials, 2013. **43**: p. 490-496.
27. Alengaram, U.J.a., H. Mahmud, and M.Z. Jumaat, *Enhancement and prediction of modulus of elasticity of palm kernel shell concrete*. Materials & Design, 2011. **32**(4): p. 2143-2148.
28. Barrios, F.A., *Acoustic emission techniques and cyclic load testing for integrity evaluation of self-compacting normal and self-compacting lightweight prestressed concrete girders*. 2010.
29. Juradin, S., G. Baloević, and A. Harapin, *Experimental Testing of the Effects of Fine Particles on the Properties of the Self-Compacting Lightweight Concrete*. Advances in Materials Science and Engineering, 2012. **2012**: p. 1-8.
30. Mazaheripour, H., et al., *The effect of polypropylene fibers on the properties of fresh and hardened lightweight self-compacting concrete*. Construction and Building Materials, 2011. **25**(1): p. 351-358.
31. 143, A.-C., *Standard test method for slump of hydraulic-cement concrete*. aMERICAN Society for Testing Materials (ASTM), West Conshohocken, 1997.
32. AS-1012.13, *Methods of testing concrete, method 13-determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory*. Australian standard, 2014.
33. Dinakar, P. *Properties of fly-ash lightweight aggregate concretes*. Proceedings of the ICE - Construction Materials, 2013. **166**, 133-140.
34. Zanjani Zadeh, V. and C.P. Bobko, *Nanomechanical characteristics of lightweight aggregate concrete containing supplementary cementitious materials exposed to elevated temperature*. Construction and Building Materials, 2014. **51**: p. 198-206.
35. Moreno, D., P. Martinez, and M. Lopez, *Practical Approach for Assessing Lightweight Aggregate Potential for Concrete Performance*. ACI Materials Journal, 2014. **111**(2): p. 123-132.
36. ASTM-C231, *Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*. American Society for Testing Materials (ASTM), 2014.
37. C173, A., *Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method*. American Society for Testing Materials (ASTM), 2007: p. 8.
38. AS-1012.4, *methods of testing concrete, method 4.1-determination of air content of freshly mixed concrete- measuring reduction in concrete volume within increased air pressure*. Australian standard, 2014.
39. Kim, Y.-J., et al., *Mechanical properties of aerated lightweight aggregate concrete*. Magazine of Concrete Research, 2012. **64**(3): p. 189-199.
40. İlker Bekir Topcu, A.U., *Elasticity Theory of Concrete and Prediction of Static E-Modulus for Dam Concrete Using Composite Models*. Digest, 2007: p. 1115-1127.
41. L.H. Nguyen, et al., *Influence of the volume fraction and the nature of fine lightweight aggregates on the thermal and mechanical properties of structural concrete*. Construction and Building Materials, 2014. **51**: p. 121-132.
42. Byard, B.E.a., A.K. Schindler, and R.W. Barnes, *Cracking Tendency of Lightweight Aggregate Bridge Deck Concrete*. ACI Materials Journal, 2014. **111**(2).

43. Ian Gilbert, R., *Tension Stiffening in Lightly Reinforced Concrete Slabs*. Journal of Structural Engineering, 2007. **133**(6): p. 899-903.
44. C496/C496M-04, A., *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens*. aMERICAN Society for Testing Materials (ASTM), West Conshohocken, 2004: p. 5.
45. Gesoğlu, M., T. Özturan, and E. Güneyisi, *Shrinkage cracking of lightweight concrete made with cold-bonded fly ash aggregates*. Cement and Concrete Research, 2004. **34**(7): p. 1121-1130.
46. Tang, W.C., T.Y. Lo, and W.K. Chan *Fracture properties of normal and lightweight high-strength concrete*. Magazine of Concrete Research, 2008. **60**, 237-244.
47. Alengaram, U.J., B.A.A. Muhit, and M.Z.b. Jumaat, *Utilization of oil palm kernel shell as lightweight aggregate in concrete – A review*. Construction and Building Materials, 2013. **38**: p. 161-172.
48. C78/C78M-10, A., *Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*. aMERICAN Society for Testing Materials (ASTM), West Conshohocken, 2010: p. 4.
49. Tassew, S. and A. Lubell, *Mechanical properties of lightweight ceramic concrete*. Materials and Structures, 2012. **45**(4): p. 561-574.
50. Casanova-del-Angel, F. and J.L. Vázquez-Ruiz, *Manufacturing Light Concrete with PET Aggregate*. ISRN Civil Engineering, 2012. **2012**.
51. Park, R. and T. Paulay, *Reinforced concrete structures*. 1975: John Wiley & Sons.
52. Chen HJ, Huang CH, and K. ZY, *Experimental investigation on steel–concrete bond in lightweight and normal weight concrete*. Structural Engineering and Mechanics 2004: p. 141-152.
53. Sancak, E., O. Simsek, and A.C. Apay, *A comparative study on the bond performance between rebar and structural lightweight pumice concrete with/without admixture*. Int J Phys Sci, 2001. **6**(14): p. 3437-3454.
54. U. Johnson Alengaram, H.M.a.M.Z.J., *Comparison of mechanical and bond properties of oil palm kernel shell concrete with normal weight concrete*. International Journal of the Physical Sciences 2010. **5**(8): p. 1231-1239.
55. Cifuentes, H., M. Alcalde, and F. Medina, *Measuring the Size-Independent Fracture Energy of Concrete*. Strain, 2013. **49**(1): p. 54-59.
56. TCM-85., R., *Determination of the fracture energy of mortar and concrete by means of three-point bend tests on notched beams*. Materials and Structures, 1985. **18**: p. 287-290.
57. Bazant, Z.P., *Concrete fracture models: testing and practice*. Engineering fracture mechanics 2002. **69**: p. 165-205.
58. F, D., *Fracture mechanical behaviour of lightweight aggregate concrete*. In: Proceedings of the 5th International Conference on Fracture Mechanics of Concrete and Concrete Structures (FramCos-5), 2004. **2**: p. 1097-1104.
59. Bazant, Z.P., *Criteria for rational prediction of creep and shrinkage of concrete*. ACI SPECIAL PUBLICATIONS, 2000. **194**: p. 237-260.
60. Troxell, G.E., J.M. Raphael, and R.E. Davis, *Long-time creep and shrinkage tests of plain and reinforced concrete*. American Society for Testing Materials (ASTM), 1958. **91**.
61. Best, C.H., and Polivka, M. , *Creep of lightweight concrete*. Magazine of Concrete Research, 1959. **11**(33).
62. Neville, A.M., *Properties of Concrete*. 4th ed , Birmingham, UK, 2010.
63. Nilsen, U. and P. Aitcin, *Properties of high-strength concrete containing light-, normal-, and heavyweight aggregate*. Cement, Concrete and Aggregates, 1992. **14**(1).
64. Newman, J.B., *Properties of structural lightweight aggregate concrete*. in: J.L. Clarke (Ed.), Structural Lightweight Aggregate Concrete, Chapman & Hall, London, 1993: p. 19-44.

65. Kayali, O., M.N. Haque, and B. Zhu, *Drying shrinkage of fibre-reinforced lightweight aggregate concrete containing fly ash*. Cement and Concrete Research, 1999. **29**(11): p. 1835-1840.
66. Reichard, T.W., *Creep and drying shrinkage of lightweight and normal weight concretes*. National Bureau of Standards, Washington, D.C., Monograph 74, 1964.
67. Shekarchi, M., F. Ghasemzadeh, and S. Sajedi, *Inverse analysis method for concrete shrinkage prediction from short-term tests*. ACI Materials Journal, 2012. **109**(3).
68. Bazant, Z.P., *Prediction of concrete creep and shrinkage: past, present and future*. Nuclear Engineering and Design 2001. **203**: p. 27-38.
69. TC-69, R., *State of the art in mathematical modeling of creep and shrinkage of concrete*. In: Bazant, Z.P., (Ed.), *Mathematical Modeling of Creep and Shrinkage of Concrete*, Wiley, Chichester and New York, pp. 57–215; and in *prelim. form: State-of-art report on creep and shrinkage of concrete: mathematical modeling*. Fourth RILEM International Conference on Creep and Shrinkage of Concrete, 1986, 1988: p. 41-80.
70. N., S.R. and A. K.L., *Shrinkage and creep properties of high-strength structural concrete*. Civil Engineering and Public Works Review, 1973. **68**(807): p. 855-868.
71. D., R.A., *Concrete creep data*. Structural Engineering, 1937. **15**: p. 314-326.
72. Lorman, W.R., *The theory of concrete creep*. American Society for Testing Materials (ASTM), 1940. **40**: p. 1082-1120.
73. Arnaouti, C. and S.R. Sangakkara, *Creep and shrinkage in lightweight-aggregate concrete*. Magazine of Concrete Research, 1984. **45**(162): p. 43-49.
74. Gamble, B.R. and L.J. Parrott, *Creep of concrete in compression during drying and wetting*. Magazine of Concrete Research, 1978. **30**(104): p. 129-138.
75. Tasdemir, M.A., S. Akyuz, and N. Uzunhasanoglu, *Creep of lightweight aggregate concrete under variable stresses*. Cement, Concrete and Aggregates, 1988. **10**(2): p. 61-67.
76. Birjandi, F. and J. Clarke, *Deflection of lightweight aggregate concrete beams*. Magazine of Concrete Research, 1993. **45**(162): p. 43-49.
77. BS-8110, *Structural use of concrete, Part 1, Code of practice for design and construction; Part 2, Code of practice for special circumstances*. British standard institute, BSI, London, 1985.

CHAPTER 3

HARDENED CONCRETE PROPERTIES

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Chapter 3: Hardened concrete properties

3.1. INTRODUCTION

Mechanical properties of concrete change from a fresh to hardened state with wholly different trend and values. Various investigations have tried to find and establish a relationship between these two phases. The principal properties of hardened concrete which are of practical importance can be listed as following:

- Strength;
- Permeability and durability;
- Shrinkage and creep deformation;
- Response to temperature variation.

The compressive strength is the most important property of concrete and almost all other properties are defined in terms of compressive strength. Concrete is usually used for compressive loads, and the compressive strength is a reliable measure of other properties. The compressive strength of concrete is the most common measure for judging not only the ability of the concrete to withstand load, but also the quality of the hardened concrete.

Durability might be defined as the capacity to maintain satisfactory performance over the extended service life. A durable concrete withstands to a satisfactory degree the effects of service conditions to which it will be subjected. The durability of concrete can be defined also as its resistance to deterioration resulting from external and internal causes. In general, external conditions such as environment and internal conditions like permeability, mix proportions and entrapped air affect the durability of concrete.

Fully-cured hardened concrete must be strong enough to withstand the structural and service loads which will be applied to it and must be durable enough to withstand the environmental exposure for which it is designed. If concrete is made with high-quality materials and is properly proportioned, mixed, handled, placed and finished, it will be the strongest and most durable building material

In Chapter Two, a brief literature review of the mechanical and time-dependent properties of Light-Weight Concrete (LWC) is presented. Also, a general comparison between the characteristics of LWC and Conventional Concrete (CC) is performed.

The overall objective of this study is to compare the instantaneous and time-dependent deflection of one-way slabs made with LWC, CC and self-compacting concrete (SCC). Therefore, in this Chapter, a comparative study of the mechanical properties of LWC, SCC and CC is performed. In this regard, the existing models of international codes of practice and the empirical models developed by researchers to predict the properties of LWC, CC and SCC are compared.

Water, aggregates and cement are the main components of the mixture design of CC, LWC and SCC. However, due to considerable differences in the proportioning of these components, along with the inclusion of different types and volumes of mineral and chemical additives, sometimes there are significant differences in the final composition and the fresh properties of each type of concrete. The major differences can also be seen in the hardened properties of LWC, SCC and CC. Compared to CC and SCC, LWC shows less density and usually lower values of the basic mechanical properties, while SCC is preferred to CC and LWC, in term of performance and the workability requirements of fresh concrete.

Lightweight concrete as described in Chapter Two, involves different types of natural and artificial lightweight aggregates. In this study, a specific type of lightweight aggregate called Expanded Poly-Styrene (EPS) has been used in the LWC mixture. Therefore, a particular review of the application, mixture design and mechanical properties of LWC made with EPS beads is performed. The accurate estimation of essential mechanical properties of EPS-LWC, especially for structural purposes is vital in its application. Hence, based on a massive range of experimental data from literature and the current study, new models are developed to predict the modulus of elasticity, modulus of rupture, splitting tensile strength and fracture energy, stress-strain curve and the steel-concrete bond behaviour for this type of LWC.

The proposed models are compared to the corresponding existing models of LWC with other types of lightweight aggregates. Also, they are compared to the relevant models for CC and SCC. The time-dependent characteristics like creep and shrinkage strain are evaluated in Chapter Four.

3.2. MANUFACTURING PROCESS OF EXPANDED POLYSTYRENE BEADS

Expanded polystyrene beads are manufactured in three main steps of pre-expansion, aging and moulding. A brief summary of each step is given in the following section.

3.2.1. Pre-Expansion Stage

The small expandable beads are subjected to steam, which causes the thermoplastic polystyrene to soften. Increasing vapour pressure caused by the blowing agent causes the beads to expand to up to 40 times their original volume. The pre-foam stage determines the final density of the expanded polystyrene block.

3.2.2. Aging Stage

After pre-expansion, the pre-foam is transferred via fluidized drying bed to large silos for aging. This process is designed to allow for the replacement of the expanding agent by air in the cells of the bead. Aging also provides stabilization and cooling of the pre-foam.

3.2.3. Moulding Stage

Once conditioned by aging, the pre-foam is blown into a mould where further steaming causes the expanded beads to fuse into a block.

Table (3.1) shows the chemical and mechanical properties of expanded polystyrene according to ASTM-C/578 (2015) [1]. Table (3.2) also gives some information about the physical properties of EPS beads not specified in Australian standard, AS-1366.3(1992) [2].

Table (3.3) describes the physical properties of EPS in the American standard ASTM-C578-15.

Table 3.1: Physical properties of expanded polystyrene according to AS 1366, Part 3 – 1992

Physical Property	Unit	Classes						Test Method
		L	SL	S	M	H	VH	
Nominal Density (kg/m ³)		11	13.5	16	19	24	28	
Compressive stress at 10% deformation (min)	KPa	50	70	85	105	135	165	AS-2498.3
Cross-breaking strength (min)	KPa	95	135	165	200	260	320	AS-2498.4
Rate of water vapor transmission (max) measured parallel to rise at 23°C	µg/m2s	710	630	580	520	460	400	AS-2498.5
Dimensional stability of length, width, thickness (max) at 70°C, dry condition 7 days	%	1.0	1.0	1.0	1.0	1.0	1.0	AS-2498.6
Thermal resistance (min) at a mean temperature of 25°C (50mm sample)	M2K/W	1	1.13	1.17	1.2	1.25	1.28	AS-2464.5 or AS-2464.6
Flame propagation characteristics:								
- median flame duration; max	S	2	2	2	2	2	2	AS-2122.1
- eighth value; max	S	3	3	3	3	3	3	
- median volume retained;	%	15	18	22	30	40	50	
- eighth value (min.)	%	12	15	19	27	37	47	

Table 3.2: Other physical properties of expanded polystyrene not specified in AS1366, Part 3-1992

Properties	Class							Test method
Physical Property	Unit	L	SL	S	M	H	VH	
Density - Nominal	Kg/m ³	11	13.5	16	19	24	28	
Compressive stress at 1% deformation (max.)	kPa	14	23	31	42	58	72	ASTM
Compressive stress at 5% deformation (max.)	kPa	33	59	68	95	134	164	ASTM
Flexural strength (min.)	kPa	60	150	178	218	304	337	ASTM C203
Elastic modulus (min.)	kPa	1450	2200	3100	4250	5850	7250	
Water absorption by total immersion (max.)	Volume%	4.0	4.0	4.0	3.0	30	2.0	ASTM C272
Buoyancy force	Kg/m ³	989	986.5	986.5	981	976	972	
Coefficient of thermal expansion	mm/mm °K	6.3e-6	6.3e-5	6.3e-6	6.3e-5	6.3e-6	6.3e-5	ASTM D696

Table 3.3: Physical properties of expanded polystyrene in ASTM C-578-15

Property	ASTM test	Type of EPS (classified by ASTM C-578)				
		Type XI	Type I	Type VIII	Type II	Type IX
Density, kg/m ³						
-Nominal	C303/	12	16	20	24	32
Minimum	D1622	11	15	18	22	29
Compressive strength, KPa						
-at 0.5% strain	C165/	17	24	29	41	55
-at 1.0% strain	D1621	35	48	58	82	110
-at yield point or 10% strain		355	69	90	104	173
Flexural strength, KPa	C203	70	173	208	276	345
Water absorption, Maximum % by volume	C272 (Total immersion)	4.0	4.0	3.0	3.0	2.0
Modulus of elasticity, KPa	D1621	3103	4655	5862	7935	10344
Poisson's ratio		0.05	0.05	0.05	0.05	0.05

3.3. EXPERIMENTAL DATA FROM LITERATURE REVIEW

The database of experimental results from published investigations could be one of the effective tools to propose and verify new models and to compare the real and predicted values. Accurate application of a developed model to particular concrete mixtures needs to use the experiment data consistent with the applied testing methodology. In spite of effectiveness of experimental results from different sources, using them can be problematic owing to the following parameters:

- Insufficient information concerning the exact composition of the concrete mixes;
- Different size and number of the specimens, curing conditions and testing methodology;
- Extracting real data of experimental results from graphs and diagrams.

The collected experimental database of this study has been based on papers presented at conferences and published articles on LWC containing EPS beads. The database contains information about the composition of the mixes, type of chemical admixture such as plasticizer and air entraining agents, curing method, curing age, fine and coarse aggregate, filler type, cement type, and fresh and hardened properties of LWC like density and compressive strength at different ages.

The wide range of experimental data of different mechanical properties of LWC containing EPS beads is included in this study. These data are collected from 55 published papers in journals and conference proceedings, some professional and scientific reports and also the dissertations all around the world since 1976 to 2015. Table (3.4) shows the sources of data and some general information about the collected data. The number of mixtures for LWC containing EPS beads, curing methods, type and maximum size of the natural and lightweight coarse and fine aggregates, maximum size of EPS beads, additional chemical admixtures including the Super Plasticiser (SP), High Range Water Reducing Admixture (HRWRA) and Air-Entraining Agent (AEA) are illustrated in the table. In addition, the types of fibre used in some experimental studies in the literature are given.

3.3.1. Mixture Design of Lightweight Concrete Containing Expanded Polystyrene Beads

In the LWC containing EPS beads, the coarse aggregates are replaced partially or wholly by EPS beads. Table (3.5) presents the mixture components and proportions in some experimental investigations to reach LWC. The resultant fresh density and compressive strength of the mixture at age 28 days are also presented in the table.

Table 3.4: Source of collected data for LWC containing EPS beads

References	No. of EPS mixes	Curing method	Cementitious materials		Aggregates					Mineral Filler	Chemical admixtures			Fibre
					Coarse		Fine		EPS bead					
			Cement	Other	Type	Max Size (mm)	Type	Max size (mm)	Max size (mm)		SP	HRWRA	AEA	
Bagon and Yannas (1976) [3]	11	W, SSS	PC-II		Perlite		Ottawa sand		2.36, 4.75			Pozzolith 200-N	MB-VR neutralized Vinsol Resin solution	
Bischoff et al. (1990) [4]	3	DH	OPC				sand	1.2	2.5-5.5		Cormix SP5			
Ravindrarajah and Tuck (1994)[5]	6	W, CS, CR	GP-PC	SF	crushed basalt	10	Sand		2.36	SF	N.G.	N.G.		
Park and Chisholm (1999) [6]	6	LSW	GP-PC				Puketapu sand	2.75	N.G.	FA				
Babu and Babu (2003) [7]	11	SW, WGB, W	PC-I	SF	crushed blue granite	10, 16	sand	1.18, 2.36	4.75, 6.3	SF	NB			
Babu and Babu (2004) [8]	7	Immersion (SW, W)	PC-I	FA, SF	crushed blue granite	8	sand	2.36	4.75, 6.3	SF	NB		N.G.	
Chen and Liu (2004) [9]	12	FR	OPC	SF	crushed granite	20	River sand	2.85	3, 8	SF	NB			SF
Miled et al. (2004) [10]	2	Oven(60 °C), CR	Saint Vigor				Betocarb P2	N.G.	3,5		Optima 100	Meyco-685		
Roy et al. (2005) [11]	5	Oven(60 °C)	CEM I 52.5	SF (A)			rounded quartz	400µm	1, 3, 6	SF	Chryso THP (PCB)			
Babu et al. (2005) [12]	7	WGB, W	PC-I-C53	FA (F)	crushed blue granite	8	Sand	2.36	4.75, 6.3		NB		N.G.	
Chen and Liu (2005) [13]	24	NAD, W, M	OPC		crushed granite	20	River sand	N.G.	3, 8			styrene-butadiene rubber (SBR)	N.G.	
Babu et al. (2006) [14]	6	N.G.	OPC	FA(F), SF	(crushed blue granite	8	Sand	2.36	1.18, 4.75		N.G.			

Table 3.4: Source of collected data for LWC containing EPS beads (continued)

References	No. of EPS mixes	Curing method	Cementitious materials		Aggregates					Mineral Filler	Chemical admixtures			Fibre
					Coarse		Fine		EPS bead					
			Cement	Other	Type	Max Size (mm)	Type	Max size (mm)	Max size (mm)		SP	HRWRA	AEA	
Haghi et al. (2006) [15]	41	NAD, W	OPC				N.G.	N.G.	3, 6, 8		NB			PA-66F
Bouvard et al. (2007) [16]	8	N.G.	HPC		N.G.	N.G.	N.G.	N.G.	1, 2, 2		N.G.			
Chen and Liu (2007) [17]	24	NAD, W, M	OPC		crushed granite	20	River sand	N.G.	3, 8			styrene-butadiene rubber (SBR)	N.G.	
Miled et al. (2007) [18]	5	NAD, Wrapped in aluminium paper (CR)	CEM I 52.5	SF(A)			rounded quartz	300µm	1, 2.5, 6.3		Optima 175 (PCB)			
Amianti and Botaro (2008) [19]	4	humid chamber	HSC		gravel	6.88	Sand	4.8	N.G.					
Bisschop and Mier (2008) [20]	33	N.G.	CEM I 52.5R		glass beads	0.5, 1, 2, 4, 6	quartz, feldspar	N.G.	N.G.		N.G.			
Lepech et L. (2008) [21]	3	N.G.	OPC	FA(F)			Silica	300µm	N.G.			ASTM C1017		PVAF
Chen and Fang (2009) [22]	12	SC, FR	PC-I	SF			Sand	2.36	600µm	SF	NB			PPF, SF
Bai et al. (2011) [23]	5	N.G.	OPC	SF, FA	stone	N.G.	sand	N.G.	2-8		FDN (NB)			
Chen and Fang (2011) [24]	3	FR	PC-I	SF			Sand	2.36	1, 2.5, 6.3		NB			PPF
Ling and Teo (2011) [25]	4	NAD, W	PC-I	RHA			River sand	1.18	8	RHA	GLENIU M C380			
Madandoust et al. (2011) [26]	12	N.G.	PC-I	SF, Nano-SiO ₂	crushed gravel	12.5	River sand	4.75	5,8	SF		Polycarboxylic ether based(Glenium51)		
Miled et al. (2011) [27]	5	NAD, Wrapped in aluminium paper (CR)	CEM I 52.5	SF (A)			rounded quartz	300µm	1, 2.5, 6.3		Optima 175 (PCB)			

Table 3.4: Source of collected data for LWC containing EPS beads (continued)

References	No. of EPS mixes	Curing method	Cementitious materials		Aggregates					Mineral Filler	Chemical admixtures			Fibre
			Cement	Other	Coarse		Fine		EPS bead		SP	HRWRA	AEA	
					Type	Max Size (mm)	Type	Max Size (mm)	Max Size (mm)					
Sadrmomtazi et al. (2012) [28]	12	Wet cured	PC-I	SF, RHA	gravel	12	Sand	4.75	3.5		PCB			PPF
Xu et al. (2012) [29]	9	CR	OPC		crushed stone	10	River sand	N.G.	3					
Carbonari et al. (2012) [30]	1	W	CEM II-32.5R				Moravac sand	4	5					
Ferrandiz-Mas and Garcia-Alcocel (2012) [31]	12	W	CEM I 52,5R				Silica sand	N.G.	1,3		Rheomix GT 205 MA	Hidroxi propil metil cellulose a	Basf Rheomix 934,	
Liu and Guo (2012) [32]	7	W	PC-I	SF		5	Sand	N.G.	N.G.					
Passa et al. (2012) [33]	3	W	HSP				white marble	4	N.G.					
Zhao et al. (2012) [34]	5	N.G.	N.G.	SF	stone		Sand	N.G.	2-8	SF	FDN (NB)			
Trussoni et al. (2013) [35]	2	plastic cover, M	PC-I		crushed limestone	10, 19	sand	N.G.	1					
Wang and Fu (2013) [36]	7	N.G.	RHSC	FA, MCF	hydrophobic perlite	N.G.			2.5,5	Gum powder	hydrophobic agent	Coagulant, HPMC ether	Silane coupling agent	PPF
Chen and Liu (2013) [37]	4	WGB, FR	PC-I, HAC	SF					1	SF	NB	F-7000 latex powder	animal protein-based	PPF
Elsalah et al. (2013) [38]	6	Oven, W	OPC		crushed stone	5	Sand	N.G.	N.G.					
Ferrandiz-Mas and Garcia-Alcocel (2013) [39]	15	W	CEM I 52.5R				silica sand		1,3		Basf Rheomix GT 205 MA	HPMC	Basf Rheomix 934	
Herki et al. (2013) [40]	6	plastic cover, W	OPC	FA			Natural sand	3	10	FA				
Sadrmomtazi et al. (2013) [41]	9	W	OPC	SF, RHA	crushed limestone	12	River sand	4.75	3.5		PCB			PPF

Table 3.4: Source of collected data for LWC containing EPS beads (continued)

References	No. of EPS mixes	Curing method	Cementitious materials		Aggregates					Mineral Filler	Chemical admixtures			Fibre
					Coarse		Fine		EPS bead					
			Cement	Other	Type	Max Size (mm)	Type	Max size (mm)	Max size (mm)		SP	HRWRA	AEA	
Hongbo et al. (2014) [42]	7	Standard curing					N.G.	N.G.	1-5					
Hidayat and Gunaedi (2014) [43]	18	N.G.	PC-I	FA			Sand	N.G.	1-2					
Kekanović et al. (2014) [44]	1	W	CEM II-32.5R				Moravac sand	4	5					PPF
Liu and Chen (2014) [45]	8	SC, FR	PC-I	SF			Sand	2.36	600 μ m, 4		NB			
Mousavi et al. (2014) [46]	2	NAD	Fly ash						5	Sand powder				GF
Schackow et al. (2014) [47]	8	N.G.	PC-II-Z32		Vermiculite	3	River sand	4.75	3	Vermiculite	Glenium-SCC160			
Tamut et al. (2014) [48]	6	W	OPC		crushed granite	N.G.	River sand	N.G.	8-9					
Nekooie et al. (2015) [49]	4	N.G.	OPC	SF, FA, POFA	crushed stone	12	Sand	N.G.	2-5					
Xu et al. (2015) [50]	16	N.G.	OPC		crushed granite	10	siliceous sand	MEDIUM GRADED	3					
Li et al. (2015) [51]	3	CB	MPC	FA	N.G.	>4.75		>1.18	>2.36, >1.18, >600 μ m					
Lo Monte et al. (2015) [52]	2	CR	CEM II R 42.5		gravel	12	Sand	N.G.	3		N.G.		N.G.	
Pecce et al. (2015) [53]	2	M	CEM II R 42.5	SF	gravel	20	Natural sand	N.G.	3		acrylic-based (Dynamon SX, Mapei)	Viscofluid SCC/10, Mapei	Mapeplast PT1, Mapei	
Ranjbar and Mousavi (2015) [54]		W	PC-I	SF	crushed gravel	12.5	River sand	4.75	4.75, 9.5			Glenium51		

The notations and symbols used in Table (3.4) are illustrated below.

Curing method

Water (W), Fog Room (FR), Natural Dry (ND), Sodium Sulphate Solutions (SSS), Damp Hessian (DH), Chemical Solutions (CS), Curing Room (CR), Moist (M), Curing Box (CB), Natural Air Dried (NAD), Wet Gunny Bags (WGB), Sea Water (SW), Stripped Cover (SC), Lime Saturated Water (LSW)

Cementitious materials and mineral fillers

Cement: Ordinary Portland Cement (OPC), Portland Cement type 1 (PC-I), type 2 (PC-II), Portland Cement type 2 grade 42.5 (CEM-II-R 42.5), Portland Cement type 1 class 53 (PC-I C53), General Purpose (GP), High Performance Cement (HPC), High Strength Cement (HSC), High Alumina Cement (HAC), Magnesia Phosphate Cement (MPC), Rapid Hardening Sulphoaluminate Cement (RHSC), High-Performance cement (HPC)

Other:

Silica Fume (SF), Fly Ash (FA), Rice Husk Ash (RHA), Palm Oil Fuel Ash (POFA), Micro-Cellular Foam (MCF)

Aggregates: Expanded Poly-Styrene (EPS)

Chemical admixtures:

Super Plasticiser (SP): Naphthalene-Based (NB), Poly-Carboxylate-Based (PCB)

High Range Water Reducing Admixture (HRWRA):

Air-Entraining Agent (AEA): Hydroxy-Propyl Methyl Cellulose (HPMC)

EPS: Expanded Poly-Styrene aggregate (EPS bead)

Fibre type: Glass Fibre (GF), Poly-Propylene Fibre (PPF), Steel Fibre (SF), Poly-Vinyl Alcohol Fibre (PVAF), Poly-Amide-66 Fibre (PA-66F)

N.G. in all columns and rows mentioning that the method or material is applied, but the relevant information is not given

* In Sadrmomtazi et al. (2012) [28], aggregates 0-3 mm are classified as sand and aggregates in the range of 3-6 mm and 6-12 mm are considered as coarse aggregates respectively.

Table 3.5: Mixture design of EPS lightweight concrete from literature

Reference	cement	Silica fume	RHA	Fly ash	Nano- sio2	Coarse aggregate	sand	EPS	Water	w/c	SP	AEA	Fibre	γ	f'_c MPa
	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	Lit/m ³	kg/m ³		ml/kg	ml/kg	Vol. %	kg/m ³	MPa
Ravindrarajah and Tuck (1994)[5]	400	0				0	753	690 l/m3	140	0.35				1325	11.9
	400	0				0	713	775	160	0.4				1230	9.8
	400	0				0	673	710	200	0.5				1220	8.7
	400	0				0	633	775	240	0.6				1165	5.6
	400	40				250	507	675	176	0.4				1305	11.8
	360	40				250	563	675	160	0.4				1350	10.9
Park and Chisholm (1999) [6]	350			0			0	1000 l/m3	180	0.51				550	0.8
	230			120			0	1000	180	0.51				520	0.7
	400			0			180	900	180	0.45				820	3.8
	270			130			180	900	180	0.45				760	2.1
	450			0			340	850	180	0.4				1020	6.7
	300			150			340	850	180	0.4				980	4.9
Babu and Babu (2003) [7]	350	3%				325	481	7.07(36.4%)	158	0.44	0.25%			1552	10.2
	362	3%				481	488	5.63(29%)	164	0.44	0.25%			1698	15
	375	3%				648	497	4.07(21%)	170	0.44	0.25%			1979	14
	335	3%					779	6.72(34.4%)	152	0.44	0.25%			1503	8.4
	399	5%				530	498	7.2(22.2%)	185	0.44	0.25%			1856	15.2
	399	5%					1028	7.2(22.2%)	185	0.44	0.25%			1748	17.6
	399	5%					743	10.8(33.2%)	185	0.44	0.25%			1546	12.4
	399	5%				515	337	7.3(28.7%)	185	0.44	0.25%			1762	11.6
	399	5%				515	337	7.3(28.7%)	185	0.44	0.25%			1771	14.2
	605	9%				466	305	6.6(26.3%)	185	0.278	0.25%			1793	15.6
	605	9%				466	305	6.6(26%)	185	0.278	0.25%			1873	19.8

Table 3.5: Mixture design of EPS lightweight concrete from literature (continued)

Reference	cement	Silica fume	RHA	Fly ash	Nano-sio2	Coarse aggregate	sand	EPS	Water	w/c	SP	AEA	Fibre	γ	f'_c MPa
	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	Lit/m ³	kg/m ³		ml/kg	ml/kg	Vol. %	kg/m ³	MPa
Chen and Liu (2004) [9]	472	0				710	392	2.49kg/m ³ (25%)	175	0.37	4.2		0	1820	22.1
	472	0				710	392	2.49 (25%)	175	0.37	4.2		70	1883	21.4
	472	0				455	255	5.01 (40%)	175	0.37	4.5		0	1356	17.6
	472	0				455	255	5.01 (40%)	175	0.37	4.5		70	1403	16.7
	472	0				201	118	6.88 (55%)	175	0.37	5.1		0	876	10.6
	472	0				201	118	6.88 (55%)	175	0.37	5.1		70	910	9.9
	472	35				710	392	2.49 (25%)	175	0.37	4.2		0	1850	25.7
	472	35				710	392	2.49 (25%)	175	0.37	4.2		70	1929	25.9
	472	35				455	255	5.01 (40%)	175	0.37	4.5		0	1370	20.1
	472	35				455	255	5.01 (40%)	175	0.37	4.5		70	1415	20.8
	472	35				201	118	6.88 (55%)	175	0.37	5.1		0	882	11.3
	472	35				201	118	6.88 (55%)	175	0.37	5.1		70	392	10.9
Roy et al. (2005) [11]	287	86					229	23.31kg/m ³	74.5	0.26	1.3%			600	
	330	99					264	21.86	85.7	0.26	1.3%			800	
	416	125					333	19.05	108.1	0.26	1.3%			1000	
	502	150					401	16.05	130.4	0.26	1.3%			1200	
	588	176					470	13.14	152.8	0.26	1.3%			1400	
Babu et al. (2005) [12]	142			142			5.5%	66.5%	185	0.651				582	1.1
	190			190			10%	58%	185	0.487				779	2.3
	224			224			20%	49%	185	0.431				984	3.83
	247			247		16%	20%	38%	214	0.434				1304	6
	274			274		30%	20%	28.5%	225	0.412				1484	7.8
	309			309		50%	20%	16.3%	245	0.396				1723	12.5

Table 3.5: Mixture design of EPS lightweight concrete from literature (continued)

Reference	cement	Silica fume	RHA	Fly ash	Nano-sio2	Coarse aggregate	sand	EPS	Water	w/c	SP	AEA	Fibre	γ	f'_c MPa
	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	Lit/m ³	kg/m ³		ml/kg	ml/kg	Vol. %	kg/m ³	MPa
Babu et al. (2006) [14]	394			169			15%	50%	200	0.355	0.5%			1012	7
	394			169		26%	14%	35%	200	0.355	0.5%			1492	11
	394			169		42%	24%	20%	200	0.355	0.5%			1858	20
	394			169			15%	50%	200	0.355	0.5%			1076	5
	394			169		26%	14%	35%	200	0.355	0.5%			1454	9
	394			169		42%	24%	20%	200	0.355	0.5%			1842	17
Miled et al. (2007) [18]	839.35	251.83					685.86	2.55(12.8%)	231.17		7.24 kg/m ³			2000	
	754	226.39					616.58	4.31(21.7%)	191.64		6.51			1800	
	669.78	200.96					547.3	6.07(30.5%)	170.11		5.78			1600	
	585	175.52					478.02	7.84(39.4%)	148.58		5.05			1400	
	500.22	150.08					408.74	9.6(48.3%)	127.04		4.32			1200	
	839.35	251.83					685.86	2.55(12.8%)	231.17		7.24			2000	
	754	226.39					616.58	4.31(21.7%)	191.64		6.51			1800	
	669.78	200.96					547.3	6.07(30.5%)	170.11		5.78			1600	
	585	175.52					478.02	7.84(39.4%)	148.58		5.05			1400	
	500.22	150.08					408.74	9.6(48.3%)	127.04		4.32			1200	
	839.35	251.83					685.86	2.55(12.8%)	231.17		7.24			2000	
	754	226.39					616.58	4.31(21.7%)	191.64		6.51			1800	
	669.78	200.96					547.3	6.07(30.5%)	170.11		5.78			1600	
	585	175.52					478.02	7.84(39.4%)	148.58		5.05			1400	
	500.22	150.08					408.74	9.6(48.3%)	127.04		4.32			1200	

Table 3.5: Mixture design of EPS light weight concrete from literature (continued)

Reference	cement	Silica fume	RHA	Fly ash	Nano-sio2	Coarse aggregate	sand	EPS	Water	w/c	SP	AEA	Fibre	γ	f'_c MPa
	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	Lit/m ³	kg/m ³		ml/kg	ml/kg	Vol.%	kg/m ³	MPa
Chen and Fang (2009) [22]	437	77					321	18.1(55%)	144	0.28	0.8		1%	980	8.5
	728	128					535	4.95(15%)	240	0.28	1.1		1%	1630	44
	631	111					464	11.5(35%)	208	0.28	1.1		1%	1435	23.5
	437	77					321	18.1(55%)	144	0.28	1		1%	990	9.5
	728	128					535	4.95(15%)	240	0.28	1.2		1%	1590	46
	631	111					464	11.5(35%)	208	0.28	1.2		1%	1418	24.5
	437	77					321	18.1(55%)	144	0.28	1.2		1%	975	10
	728	128					535	4.95(15%)	240	0.28	1		1%	1610	46.8
	631	111					464	11.5(35%)	208	0.28	1		1%	1430	25
	437	77					321	18.1(55%)	144	0.28	1		1%	983	10.5
Bai et al. (2011) [23]	386	29.68		213.5		899	503	10%	184		5.93				
	386	29.68		213.5		728	407	20%	184		5.93				
	386	29.68		213.5		557	311	30%	184		5.93				
	386	29.68		213.5		386	215	40%	184		5.93				
	386	29.68		213.5		215	119	50%	184		5.93				
Chen and Fang (2011) [24]	861	369					768		246	0.2	1.2			1800	130
	944	236					737		295	0.25	1			1400	90
	1017	113					706		339	0.3	0.8			1000	50
	861	369							246		1.2			1800	
	944	236					737		295		1			1400	
	1017	113					706		339		0.8			1000	
	861	369							246		1.2			1800	
	944	236					737		295		1			1400	
	1017	113					706		339		0.8			1000	

Table 3.5: Mixture design of EPS lightweight concrete from literature (continued)

RefereRence	cement	Silica fume	RHA	Fly ash	Nano-sio2	Coarse aggregate	sand	EPS	Water	w/c	SP	AEA	Fibre	γ	f'_c MPa
	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	Lit/m ³	kg/m ³		ml/kg	ml/kg	Vol. %	kg/m ³	MPa
Ling and Teo (2011) [25]	425		0				850	18		0.5	8			1830	13
	403		21				850	18		0.5	8			1810	15
	382		42				850	18		0.5	8			1790	17
	360		64				850	18		0.5	8			1770	13
	340		85				850	18		0.5	8			1750	11
Madandoust et al. (2011) [26]	400	44	3.5			662	783	1.46	196					2185	
	400	44	3.2			594	716	2	196					2040	
	400	44	2.8			491	617	2.52	196					1896	
	400	44	2.5			387	517	2.74	196					1712	
	388	44	3.5			662	783	1.46	196					2187	
	388	44	3.2			594	716	2	196					2105	
	388	44	2.8			491	617	2.52	196					1898	
	388	44	2.5			387	517	2.74	196					1701	
	434	48	3.5			662	783	1.46	183					2235	
	434	48	3.2			594	716	2	183					2077	
	434	48	2.8			491	617	2.52	183					1919	
	434	48	2.5			387	517	2.74	183					1748	
Sadrmomtazi et al. (2012) [28] *	400	0	0			872	540	2.14	170	0.43	N.G.		1.32	1900	33
	400	0	0			696	431	2.85	165	0.41	N.G.		4	1700	16.7
	400	0	0			475	294	3.11	160	0.4	N.G.		6.6	1350	9.8
	400	0	0			249	154	2.24	160	0.42	N.G.		13.26	979	5.4
	360	40	0			848	524	2.08	175	0.44	N.G.		1.32	1900	35
	360	40	0			682	422	2.79	175	0.44	N.G.		4	1650	24.4
	360	40	0			456	282	3	170	0.43	N.G.		6.6	1300	10.2
	360	40	0			227	140	2.04	167	0.42	N.G.		13.26	949	2.9
	320	0	80			750	470	1.85	205	0.51	N.G.		1.32	1850	22.4
	320	0	80			623	385	2.54	205	0.51	N.G.		4	1600	10.6
	320	0	80			395	245	2.59	200	0.5	N.G.		6.6	1250	7.1
	320	0	80			133	82	55%1.19	210	0.53	N.G.		13.26	890	3.5
	0.0231	0				0.0338	0.0411	0.000287	0.0113	0.49	0	0		1920	23.24

Table 3.5: Mixture design of EPS lightweight concrete from literature (continued)

Reference	cement	Silica fume	RHA	Fly ash	Nano-sio2	Coarse aggregate	sand	EPS	Water	w/c	SP	AEA	Fibre	γ	f'_c MPa
	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	kg/m ³	Lit/m ³	kg/m ³		ml/kg	ml/kg	Vol. %	kg/m ³	MPa
Xu et al. (2012) [29]	400						35%	15%		0.45				2060	20.77
	400						38%	20%		0.5				1890	14.79
	400						41%	25%		0.55				1720	11.22
	450						41%	15%		0.45				1880	17.86
	450						35%	20%		0.5				1800	7.85
	450						38%	25%		0.55				1930	15.68
	500						38%	15%		0.45				1730	13.52
	500						41%	20%		0.5				1950	18.56
	500						35%	25%		0.55				1790	12.98
Trussoni et al. (2013) [35]	0.0233	0				0.032	0.0411	0.000299	0.011	0.48	0	0		1920	24.99
	0.0231	0				0.0338	0.0411	0.000287	0.0113	0.49	0	0		1920	23.24
Lo Monte et al. (2015) [52]	643					591	475	5.09	200		7.05	2.12			24.7
	815					310	620	5.32	220		9.11	2.73			26.3
Pecce et al. (2015) [53]	815					367	734	5.32	220	0.27	9.11	2.73			31
	843					591	475	5.09	193	0.3	7.05	2.12			20.5
Ranjbar and Mousavi (2015) [54]	400	44				662	783	10%	196	0.44		3.5		2185	27
	400	44				594	716	15%	196	0.44		3.2		2040	22
	400	44				491	617	22.5%	196	0.44		2.8		1896	19
	400	44				387	517	30%	196	0.44		2.5		1712	17
	434	48				662	783	10%	183	0.38		3.5		2235	31
	434	48				594	716	15%	183	0.38		3.2		2077	26
	434	48				491	617	22.5%	183	0.38		2.8		1919	22
	434	48				387	517	30%	183	0.38		2.5		1748	19

3.4. MECHANICAL PROPERTIES OF LIGHTWEIGHT CONCRETE CONTAINING EXPANDED POLYSTYRENE BEADS

Mechanical properties of LWC containing EPS beads are evaluated in various experimental investigations. Table (3.6) explains the investigated properties of EPS-LWC in the collected publications in this study. Table (3.7) also describes the abbreviations of the characteristics and measured properties of EPS-LWC by different testing methods.

Table 3.6: Investigated characteristics of lightweight concrete containing EPS beads in literature

Column	1	2	2	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Properties	f'_c	E_c	f_r	f_{ct}	$\sigma-\epsilon$	γ	WRK	ϕ_{cc}	ϵ_{sh}	WA (%)	DUR	EAF	BS	UPV	TC	SE	EAP	MM	R_{hm}	R_{elec}
Bagon and Yannas (1976) [3]	✓		✓			✓	✓			✓	✓									
Bischoff et al. (1990) [4]	✓					✓						✓								
Ravindrarajah and Tuck (1994)[5]	✓	✓		✓		✓			✓		✓			✓			✓			
Park and Chisholm (1999) [6]	✓					✓	✓		✓						✓					
Babu and Babu (2003) [7]	✓			✓		✓	✓			✓	✓									
Babu and Babu (2004) [8]	✓					✓				✓	✓									
Chen and Liu (2004) [9]	✓			✓		✓			✓							✓				
Miled et al. (2004) [10]	✓	✓																		
Roy et al. (2005) [11]	✓	✓				✓	✓		✓	✓		✓								
Babu et al. (2005) [12]	✓	✓		✓	✓		✓							✓					✓	
Chen and Liu (2005) [13]	✓		✓			✓														
Babu et al. (2006) [14]	✓			✓		✓	✓			✓								✓		
Haghi et al. (2006) [15]	✓					✓						✓								
Bouvard et al. (2007) [16]	✓				✓	✓											✓			
Chen and Liu (2007) [17]	✓		✓				✓													
Miled et al. (2007) [18]	✓	✓				✓										✓				
Amianti and Botaro (2008) [19]										✓								✓		
Bisschop and Mier (2008) [20]						✓			✓								✓			
Lepech et L. (2008) [21]	✓				✓															
Chen and Fang (2009) [22]	✓			✓		✓	✓		✓	✓										
Bai et al. (2011) [23]	✓				✓							✓								
Chen and Fang (2011) [24]	✓	✓		✓		✓														
Ling and Teo (2011) [25]	✓					✓	✓			✓					✓	✓				
Madandoust et al. (2011) [26]	✓					✓	✓													
Miled et al. (2011) [27]		✓															✓			
Sadrmomtazi et al. (2012) [28]	✓		✓	✓		✓			✓	✓	✓			✓						✓

Table 3.6: Investigated characteristics of lightweight concrete containing EPS beads in literature (continued)

Column	1	2	2	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Properties	f'_c	E_c	f_r	f_{ct}	$\sigma-\epsilon$	γ	WRK	ϕ_{cc}	ϵ_{sh}	WA (%)	DUR	EAF	BS	UPV	TC	SE	EAP	MM	R_{hm}	R_{elec}
Xu et al. (2012) [29]	✓				✓	✓	✓													
Ferrandiz-Mas and Garcia-Alcocel (2012) [31]	✓		✓			✓	✓										✓			
Liu and Guo (2012) [32]						✓			✓	✓		✓			✓					
Passa et al. (2012) [33]	✓								✓						✓					
Zhao et al. (2012) [34]					✓							✓								
Trussoni et al. (2013) [35]	✓	✓			✓							✓								
Wang and Fu (2013) [36]	✓					✓	✓		✓						✓					
Chen and Liu (2013) [37]	✓			✓	✓	✓	✓								✓					
Elsalah et al. (2013) [38]			✓						✓	✓										
Ferrandiz-Mas and Garcia-Alcocel (2013) [39]										✓	✓									
Herki et al. (2013) [40]	✓					✓								✓						
Sadrmomtazi et al. (2013) [41]	✓																			
Hongbo et al. (2014) [42]	✓		✓		✓	✓				✓		✓								
Wang and Arellano (2014) [55]	✓		✓			✓		✓												
Gao et al. (2014) [56]	✓		✓			✓									✓					
Hidayat and Gunaedi (2014) [43]	✓					✓	✓													
Kekanović et al. (2014) [44]	✓	✓	✓			✓			✓		✓				✓					
Liu and Chen (2014) [45]	✓	✓	✓	✓		✓								✓						
Mousavi et al. (2014) [46]	✓				✓	✓														
Schackow et al. (2014) [47]	✓					✓	✓			✓					✓					
Tamut et al. (2014) [48]	✓		✓	✓			✓													
Nekooie et al. (2015) [49]	✓						✓			✓										
Xu et al. (2015) [50]	✓	✓		✓		✓														
Li et al. (2015) [51]	✓			✓		✓			✓	✓	✓									
Lo Monte et al. (2015) [52]	✓			✓	✓	✓				✓				✓	✓					
Pecce et al. (2015) [53]	✓	✓				✓	✓						✓							
Ranjbar and Mousavi (2015) [54]	✓			✓		✓	✓			✓	✓									✓

Full description and symbols for properties illustrated in the columns of Table (3.6) are presented in Table (3.7).

Table 3.7: Symbols and notations for collected properties of EPS-LWC in Table 3.6

Column	Measured property	Abbreviation
1	Compressive strength	f'_c
2	Modulus of elasticity	E_c
3	Flexural tensile strength (modulus of rupture)	f_r
4	Splitting tensile strength	f_{ct}
5	Stress-strain	$\sigma - \varepsilon$
6	Density	γ
7	Slump or workability	WRK
8	Creep coefficient	ϕ_{cc}
9	Shrinkage strain	ε_{sh}
10	Water absorption	WA (%)
11	Durability test	DUR
12	Energy absorption or fracture energy	EAF
13	Bond-slip between concrete and bars	BS
14	Ultrasonic Pulse Velocity	UPV
15	Thermal conductivity or fire resistance	TC
16	Size effect of specimen or aggregates	SE
17	Entrapped air or porosity	EAP
18	Moisture migration	MM
19	Rebound value (Schmidt hammer)	R_{hm}
20	Electrical resistivity	R_{elec}

The experimental data from previous experiments in the literature is always a reliable method of developing new models and evaluating the existing models to predict the mechanical properties of concrete. The main difference between conventional concrete and LWC containing EPS beads is replacement of some or the whole part of the coarse aggregates with various sizes of EPS beads.

Table (3.8) presents the recorded values of mechanical properties of EPS-LWC in the hardened state from previously conducted experiments presented in Table (3.4). Compressive strength (f'_c), modulus of elasticity (E_c), modulus of rupture (f_r) and splitting tensile strength (f_{ct}) are the main mechanical properties given in Table (3.8).

Table (3.8) also explains the volume of EPS beads in the LWC mixtures. The maximum size of EPS beads and fine and coarse aggregates are previously illustrated in Table (3.4).

Table 3.8: Mechanical properties of LWC containing EPS beads from experiments in literature

References	EPS	f'_c	E_c	f_r	f_{ct}
	Vol. %	(MPa)	(GPa)	(MPa)	(MPa)
Bagon and Yannas (1976) [3]		3.3	3.92	1.37	
		4.6	3.92	1.76	
		4.6	3.92	1.66	
		4.6	4.89	1.96	
		3.9	3.92	1.57	
		4.6	3.92	1.76	
		4.8	3.92	1.76	
		4.8	5.87	1.76	
		3.6	3.92	1.37	
		5	5.87	1.76	
		4.5	4.89	1.66	
Ravindrarajah and Tuck (1994)[5]	36.3	11.9	11.8		1.7
	36.3	9.8	8.7		1.5
	36.9	8.7	8.4		1.2
	36.4	5.6	5.4		0.8
	33	11.8	11.6		1.6
	34.2	10.9	11.9		1.5
Babu and Babu (2003) [7]	36.4	10.2			1.53
	29	15			2.04
	21	14			2.16
	34.4	8.4			2.1
	22.2	15.2			2.23
	22.2	17.6			2.2
	33.2	12.4			2.15
	28.7	11.6			1.83
	28.7	14.2			2.02
	26.3	15.6			2.22
	26	19.8			2.32
Chen and Liu (2004) [9]	25	22.1			2.31
	25	21.4			2.62
	40	17.6			2.14
	40	16.7			2.53
	55	10.6			1.32
	55	9.9			2.08
	25	25.7			2.4
	25	25.9			2.73
	40	20.1			2.22
	40	20.8			2.68
	55	11.3			1.73
	55	10.9			2.31
Miled et al. (2004) [10]		8.1	5.633		
Babu et al. (2005) [12]	66.5	1.1			0.64
	58	2.3	2.1		0.89
	49	3.83	4.33		1.15
	38	6	8.45		1.4
	28.5	7.8	9		2.04
	16.3	12.5	16		2.34
Chen and Liu (2005) [13]		14		2.4	
		16.5		3.8	
		17		4.5	
		15.2		4.6	
		12.2		3.9	

Table 3.8: Mechanical properties of LWC containing EPS beads from experiments in literature (continued)

References	EPS	f'_c	E_c	f_r	f_{ct}
	Vol. %	(MPa)	(GPa)	(MPa)	(MPa)
Chen and Liu (2005) [13]		16.8			
		16.3			
		15.5			
		13.6			
		11.2			
		22.6			
		20.7			
Babu et al. (2006) [14]	50	7			1.4
	35	11			1.65
	20	20			2.7
	50	5			0.87
	35	9			1.45
	20	17			2.87
Chen and Liu (2007) [17]		16.5		3.8	
		17		4.5	
		15.2		4.6	
		12.2		3.9	
		16.3		3.4	
		15.5		3.6	
		13.6		3.5	
		11.2		3.3	
		20.7		4	
		19.2		3.9	
		17		3.9	
		14.7		3.3	
		24.3		4.2	
		25.4		4.3	
		20.4		4.4	
		18.8		4.2	
		20.3		4.8	
		18.6		5.2	
		15.5		4.9	
		13.4		4.2	
		20.3		3.9	
		17		4.3	
		16.2		4.4	
		13.8		4.2	
Chen and Fang (2009) [22]	15	42			4.5
	35	21.5			3
	55	8.5			1.55
	15	44			4.9
	35	23.5			3.3
	55	9.5			1.8
	15	46			5
	35	24.5			3.5
	55	10			2
	15	46.8			5.3
	35	25			3.55
	55	10.5			2.1
Chen and Fang (2011) [24]	19.4	40			4.34
	19.4	48			4.89
	19.4	31			3.67

Table 3.8: Mechanical properties of LWC containing EPS beads from experiments in literature (continued)

References	EPS	f'_c	E_c	f_r	f_{ct}
	Vol. %	(MPa)	(GPa)	(MPa)	(MPa)
Chen and Fang (2011) [24]	19.4	37			4.12
	19.4	26			3.27
	19.4	33			3.82
	37.6	18			2.57
	37.6	22			2.93
	37.6	12			1.97
	37.6	15			2.28
	37.6	10			1.75
	37.6	13			2.08
	55.8	6			1.25
	55.8	8			1.51
	55.8	4			0.96
	55.8	5			1.11
	55.8	3.5			0.88
	55.8	4.5			1.04
Madandoust et al. (2011) [26]	10	26.58			
	15	28.86			
	22.5	31.14			
	30	22.78			
	10	24.30			
	15	25.82			
	22.5	18.99			
	30	20.51			
	10	22.03			
	15	17.47			
	22.5	18.23			
Sadrumontazi et al. (2012) [28]	15	33		3.30	3.94
	25	16.7		2.26	2.71
	40	9.8		2.39	2.82
	55	5.4		2.52	3.12
	15	35		1.57	2.35
	25	24.4		2.04	2.94
	40	10.2		1.26	1.71
	55	2.9		1.30	1.82
	15	22.4		0.96	1.35
	25	10.6		1.00	0.94
	40	7.1		0.70	0.82
	55	3.5		0.74	0.76
Xu et al. (2012) [29]	15	20.77			
	20	17.79			
	25	11.22			
	25	17.86			
	15	7.85			
	20	15.68			
	20	13.52			
	25	18.56			
Trussoni et al. (2013) [35]	13	25	17.84		
	12.4	23.25	15.33		
Chen and Liu (2013) [37]	5	1.76			0.23
	10	2.07			0.17
	15	2.76			0.19
	20	2.37			0.15
	5	10.5			1.42
	10	13			1.44
	15	12.5			1.37
	20	12			1.35

Table 3.8: Mechanical properties of LWC containing EPS beads from experiments in literature (continued)

References	EPS	f'_c	E_c	f_r	f_{ct}
	Vol. %	(MPa)	(GPa)	(MPa)	(MPa)
Herki et al. (2013) [40]	60	11			
	100	5			
	60	8			
	100	3.7			
	60	5.5			
	100	3			
Sadrmomtazi et al. (2013) [41]	15 kg/m ³	33			
	25	16.7			
	40	9.8			
	15	27.8			
	25	24.4			
	40	10.2			
	15	22.4			
	25	10.6			
	40	6.7			
Hidayat and Gunaedi (2014) [43]	10	18.12			
	10	18.87			
	10	19.63			
	20	16.23			
	20	16.99			
	20	17.55			
	30	14.91			
	30	15.66			
	30	16.04			
Kekanović et al. (2014) [44]	0.49kg/m ³	7.73	6.6	2.52	
	0.51	2.53		0.82	
	0.53	6.2		0.39	
	0.53	6.7			
Liu and Chen (2014) [45]	2.64kg/m ³	46.3		9.11	6.85
	5.76	23.6		6.87	5.37
	8.75	20.3		5.79	4.24
	10.9	12.2		3.72	2.4
	2.47	37.8		6.21	5.77
	5.18	17.4		5.4	4.53
	7.8	11.6		3.99	2.77
	9.59	7.5		3.39	1.38
Mousavi et al. (2014) [46]	13.2kg/m ³	5.5			
	7	6.3			
Schackow et al. (2014) [47]	55	16			
	56	14			
	58	12			
	60	10			
	64	8			
Tamut et al. (2014) [48]	5	33.75			
	10	28.75			
	15	26.7			2.8
	20	22.9			
	25	20.4			
	30	17.1			2.4

Table 3.8: Mechanical properties of LWC containing EPS beads from experiments in literature (continued)

References	EPS	f'_c	E_c	f_r	f_{ct}
	Vol. %	(MPa)	(GPa)	(MPa)	(MPa)
Nekooie et al. (2015) [49]	50	10.3			
	50	15.54			
	50	14.41			
	50	14.07			
Xu et al. (2015) [50]	5	33	29.71		3.1
	10	25.9	24.46		2.8
	15	16.8	20.5		2.2
	20	12.5	14.47		2.1
	25	9.1	12.49		1.3
	30	3.8	8.71		1.1
	35	3.6	6.15		0.9
	40	1.9	3.6		0.7
	5	28.6	27.36		2.8
	10	22.3	23.4		2.6
	15	18	21.36		2.3
	20	14.8	14.95		1.6
	25	12.5	13.73		1.6
	30	10.7	11.29		1.5
	35	8.8	9.2		1.1
	40	5.3	6.86		1
Li et al. (2015) [51]	20	37			4.2
	40	22			3
	60	11			1.87
Pecce et al. (2015) [53]	5.32kg/m ³	31	19.89		
	5.09	20.5	18.55		
Ranjbar and Mousavi (2015) [54]	10	23			2.2
	15	20			2.02
	22.5	16			1.81
	30	15			1.77
	10	31			2.33
	15	25			2.23
	22.5	22			2.14
	30	19			1.95

3.4.1. Modulus of Elasticity

3.4.1.1. Experimental and analytical database for modulus of elasticity

Tables (3.4) to (3.8) present a comprehensive summary of the previously conducted experiments in the literature. They show the components and proportions of the mixtures involved in making LWC containing EPS beads. Different types of chemical and mineral admixture are included in the mixtures. Also different methods have been applied to cure the samples before testing. Various types of cementitious materials and natural and artificial aggregates in the mixtures give different ranges of mechanical properties for hardened concrete.

The main properties of hardened concrete investigated in the present study include compressive strength, modulus of elasticity, modulus of rupture, splitting tensile strength and

stress-strain curve in the EPS-LWC mixtures. However, a wide range of existing models for mechanical properties of conventional concrete and self-compacting concrete have been collected from the existing literature. These data are compared and evaluate with the outcomes of this study.

A general comparison between the mechanical properties of LWC made with different types of lightweight aggregates and the conventional concrete is presented in Chapter Two. In this chapter, mechanical properties of LWC containing EPS beads are investigated. New models are developed for different properties of hardened concrete and evaluated by the existing models.

Considering the modulus of elasticity, Table (3.8) gives a considerable amount of experimental data of EPS-LWC. In this section, the proposed model to predict the modulus of elasticity (E_c) of EPS-LWC is based on the regression analysis of the experimental data. The power type models in most of the existing codes and empirical equations are selected as the basis of the newly developed models in this study.

3.4.1.2. Proposed model of modulus of elasticity in this study

An important part of the experimental datasets for EPS-LWC is classified as non-structural LWC. Hence, the models are proposed for three groups of non-structural EPS-LWC with the compressive strength not exceeding 17 MPa, structural LWC with compressive strength above 17 MPa and finally LWC containing all ranges of the compressive strength. According to the collected experimental data, the upper limit of the compressive strength of the proposed models is different from the various mechanical properties of LWC mixtures that are mentioned in the proposed models.

Equation (3.1) is a general form of the proposed model to predict MoE from compressive strength. The proposed model includes different types of natural and artificial aggregates, fillers and chemical admixtures, different size and content of EPS beads along with the variable water to binder ratios in the mixtures. Table (3.9) gives the specialized coefficients of the proposed model for each group of the EPS-LWC. To evaluate the accuracy of proposed model, correlation factor (R^2) in the least square method of regression analysis is given in the table.

$$E_c = \alpha. (f'_c)^\beta \quad \text{Eq. (3. 1)}$$

Where; E_c is modulus of elasticity (GPa), f'_c is compressive strength (MPa) and α , β are coefficients.

Table 3.9: Coefficients of equation (3.1) and the compressive strength limit

	α	β	R^2	f'_c limit (MPa)
Non-Structural EPS-LWC	0.9101	1.026	0.93	≤ 17
Structural EPS-LWC	4.21	0.508	0.22	> 17
EPS-LWC	1.4447	0.8511	0.89	3-35

Figure (3.1) shows the predicted versus experimental values of modulus of elasticity in two ranges of compressive strength. As shown, the proposed equations accurately predict the modulus of elasticity of EPS-LWC, especially for non-structural concrete.

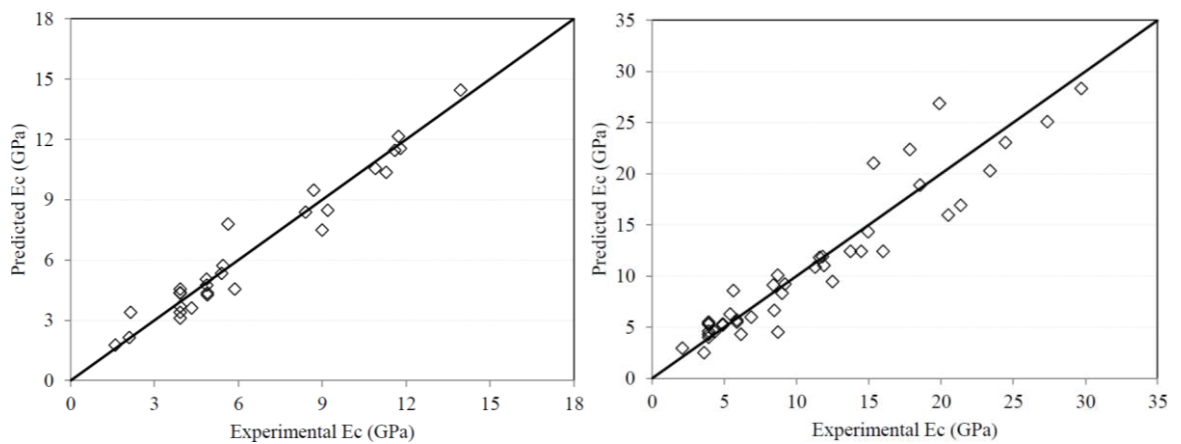


Figure 3.1: Comparison of the experimental data with the predicted modulus of elasticity in EPS-LWC

3.4.1.3. Comparison of the analytical models of modulus of elasticity

In absence of accurate testing measurements, there are several analytical and numerical models in the literature used to predict the modulus of elasticity of different types of concrete mixtures. Tables (3.10) show some of these existing models to estimate the MoE of LWC. These models vary in complexity and precision in the calculations.

Figure (3.2) shows the MoE versus compressive strength in LWC. Comparison of the proposed models in Table (3.9) for EPS-LWC in non-structural data and all data ranges of compressive strength are also presented.

This study utilizes EPS-LWC with a density of 2000 kg/m^3 to manufacture the testing specimens and one-way slabs for instantaneous and long-term deflection. Therefore, the comparison is performed for that specified density in Figure (3.2).

As shown in Figure (3.2), the models in selected codes of practice give a comparable prediction for MoE in LWC in all ranges of compressive strength. The empirical model

proposed by Min and Gjørsv (1991) significantly underestimates the MoE in LWC with the density of 2000 kg/m³. The proposed models in this study underestimate MoE in the range of compressive strength below 17 MPa (non-structural LWC) and over-estimate the MoE for compressive strength above 17 MPa. The differences in overestimated MoE values by the proposed models are considerable.

Since the density of LWC is a major parameter in the prediction of MoE, Figure (3.3) compares predictions of the models in most frequently used codes of practice for the lower limit of density (1280 kg/m³). Comparing Figures (3.2) and (3.3) show the variation of MoE values by changing the density in LWC. Even, when considering similar prediction of the selected codes for LWC with density of 2000 kg/m³, the same models estimate different MoE values for LWC with density of 1280 kg/m³.

Table 3.10: Existing models to predict the modulus of elasticity of lightweight concrete

Reference	$E_c = Aw^B \cdot (f'_c)^C + D$: Ec(GPa), f'_c (Mpa)	Notes
AASHTO LRFD (2007) [57]	$E_c = 0.043k_1\gamma^{1.5}(f'_c)^{0.5}$ kg/m ³ and MPa $K_1 = 1.0$ unless determined by physical test	1280 < γ < 2400 kg/m ³ 14 ≤ f'_c ≤ 48 MPa
Rizkalla (NCHRP)(2007) [58]	$E_c = 310000k_1\gamma^{2.5} \cdot (f'_c)^{0.33}$	1440 < γ < 2800 kg/m ³ f'_c < 165 MPa E_c < 74 GPa
ACI 318-08 [59]	$E_c = 0.043w^{1.5}(f'_c)^{0.5}$ MPa, GPa, kg/m ³	for 1440 < γ < 2500 kg/m ³
Modified ACI 318-95 [60]	$E_c = 30.16\gamma^{1.5}(f'_c)^{0.5} + 484200$	Ec(psi), f'_c (psi), w(pcf)
ACI-209-2R-08 [61]	$E_c = 0.043\gamma^{1.5}(f'_c)^{0.5}$ MPa, kg/m ³ $E_c = 4730(f'_c)^{0.5}$ MPa	1440 < γ < 2500 kg/m ³
ACI 312-92 [62]	$E_c = 14000 + 3250(f'_c)^{0.5}$ $E_c = 9500(f'_c + 8)^{1/3}$ $E_c = 0.043\gamma^{1.5}(f'_c)^{0.5}$	GPa, MPa
CSA A23.3-04 [63]	$E_c = (3300(f'_c)^{0.5} + 6900)(\gamma/2300)^{1.5}$ $E_c = 4500(f'_c)^{0.5}$	20 ≤ f'_c ≤ 40 MPa 1500 < γ < 2500 kg/m ³
NZS-3101-95 [64]	$E_c = (3320(f'_c)^{0.5} + 6900)(W/2300)^{1.5}$	E_c in MPa
AS-3600-2900 [65]	$E_c = 0.043\gamma^{1.5}(f'_c)^{0.5}$ $E_c = 5050(f'_c)^{0.5}$ normal weight concrete	f'_c < 40 MPa 0 < Secant < 0.4 f_{cm}
NS-3473 (1992) [66]	$E_c = 9.5(f'_c)^{0.3}(\frac{\gamma}{2400})^{1.5}$ GPa, kg/m ³ $E_c = 9500(f'_c)^{0.3}$	
NS-3473 (1992) [66]	$E_c = 9.5(f'_c)^{0.3}(\frac{\gamma}{2400})^{1.5}$ GPa, kg/m ³ $E_c = 9500(f'_c)^{0.3}$	

Table 3.10: Existing models to predict the modulus of elasticity of lightweight concrete (continued)

Reference	$E_c = Aw^B \cdot (f'_c)^C + D : E_c(\text{GPa}), f'_c(\text{Mpa})$	Notes
Architecture institute of Japan (AIJ) [67]	$E_c = 2.1 \times 10^5 (\gamma/2.3)^{1.5} (f'_c/200)^{0.5}$	$E_c, f'_c: \text{kgf/cm}^2, \gamma: \text{t/m}^3$
Yusuf, I T; Jimoh, Y A (2013) [68]	$E_c = 0.001(f'_c)^{1.549}$	
JSCE (2007) [69]	$E_c = 4700(f'_c)^{0.5} \quad (ACI-318-08)$ $E_c = 10.792 \ln(f'_c) - 9.0675 \quad \text{best fit}$	$18 \leq f'_c \leq 80 \text{ MPa}$
RakMK-D3-2012 [70]	$E_c = 5000(\gamma/2400)(f'_c)^{0.5}$	
NTE E.060(2009) [71]	$E_c = 0.043\gamma^{1.5}(f'_c)^{0.5} \quad \text{MPa, kg/m}^3$ $E_c = 4730(f'_c)^{0.5} \text{ MPa}$	
BS 8110 (1997) [72]	$E_c = (20 + 0.2f'_c)(\gamma/2400)^2 \text{ GPa}$	$20 \leq f'_c \leq 60 \text{ MPa}$
BS 5400-4(1990)[73]	$E_c = 8.6475(f'_c)^{0.348}$	$20 \leq f'_c \leq 60 \text{ MPa}$
SP 52-101-2003 [74]	$E_c = 11.625 \ln(f'_c) - 7.4713$	$10 \leq f'_c \leq 60 \text{ MPa}$
Carrasquillo, et al.(1981) [75]	$E_c = 3.320(f'_c)^{0.5} + 6.900 (\gamma/2346)$ $E_c = 3.320(f'_c)^{0.5} + 6.900$	GPa, kg/m^3
Soleymani (2006) [76]	$E_c = (\gamma/2300)^{1.5} (3000(f'_c)^{0.5} + 6900)$ $E_c = (\gamma/2300)^{1.5} (3300(f'_c)^{0.5} + 6900)$	Best fit with: - Eq. 8-6 in CSA-A23.3(1994) - Eq. 8-6 in CSA-23.3(1994)
Haranki (2009) [77]	$E_c = 31.92 \times \gamma^{1.5} (f'_c)^{0.5} + 345,328$	psi, lb/ft^3
Ahmad and Shah (1985) [78]	$E_c = 3.38 \times 10^{-5} \times \gamma^{2.5} (\sqrt{f'_c})^{0.65}$	
Jobse and Mustafa (1984) [79]	$E_c = 0.103\gamma^{1.5} (\sqrt{f'_c})^{0.5}$	
Cook (1989) [80]	$E_c = 3.22 \times 10^{-5} \times \gamma^{2.5} (f'_c)^{0.315}$	$\text{kg/m}^3, \text{MPa}$
Min and Gjørsv (1991) [81]	$E_c = 1.19(f'_c)^{2/3} \text{ GPa, MPa}$	For light-weight concrete
Jensen (1943) [82]	$E_c = 6 \times 10^6 / (1 + 2000/f'_c) \text{ psi}$	$1280 < \gamma < 2400 \text{ kg/m}^3$ $14 \leq f'_c \leq 48 \text{ MPa}$
Pauw (1960) [83]	$E_c = 13.82\gamma^{1.79} (f'_c)^{0.44}$	
Slate et al. (1986) [84]	$E_c = (3320(f'_c)^{0.5} + 6895)(\gamma/2320)^{1.5}$	
Ravindrarajah and Tuck (1994) [5]	$E_c = 1.146(f'_c)^{0.5}$	
Khan (1990)	$E_c = (3.32(f'_c)^{0.5} + 6.9)(\gamma/2325)^{1.5}$	MPa
Morales (1982) [85]	$E_c = (40000(f'_c)^{0.5} + 1000000)(\gamma/145)^{1.5}$	psi
Meyer (2002) [86]	$E_c = (33000(f'_c)^{0.5} + 4000000)(\gamma/242)^{0.9}$	psi
Alengaram et al. (2011) [87]	$E_c = 5(f'_c)^{1/3} (\gamma/2400)^2$	
Ahmmad et al. (2014) [88]	$E_c = 0.0026(f'_c)^{2.5546} \text{ GPa}$	
Dinakar (2012) [89]	$E_c = 4.2(f'_c)^{0.5} \text{ GPa}$	
Tassw and Lubell (2012) [90]	$E_c = 0.031f'_c^{0.5}\gamma^{1.5}$	

The presented study investigates the mechanical properties of lightweight concrete. Also, the short-term and long-term behaviour of lightweight concrete slabs are evaluated

comprehensively. To better analysis of the mechanical properties, it is tried to collect all the published models in the articles and the technical reports in literature.

In total, 37 models to estimate the modulus of elasticity from compressive strength in lightweight concrete are collected. The oldest model is developed in 1943 for the wide range of density and compressive strength of lightweight concrete. Considering the lower density of lightweight concrete in comparison with conventional concrete, the density limit is almost applied in all the models. Also, limitations on the compressive strength are applied in most of the models. Some models are also applicable for conventional concrete by changing the density related coefficients.

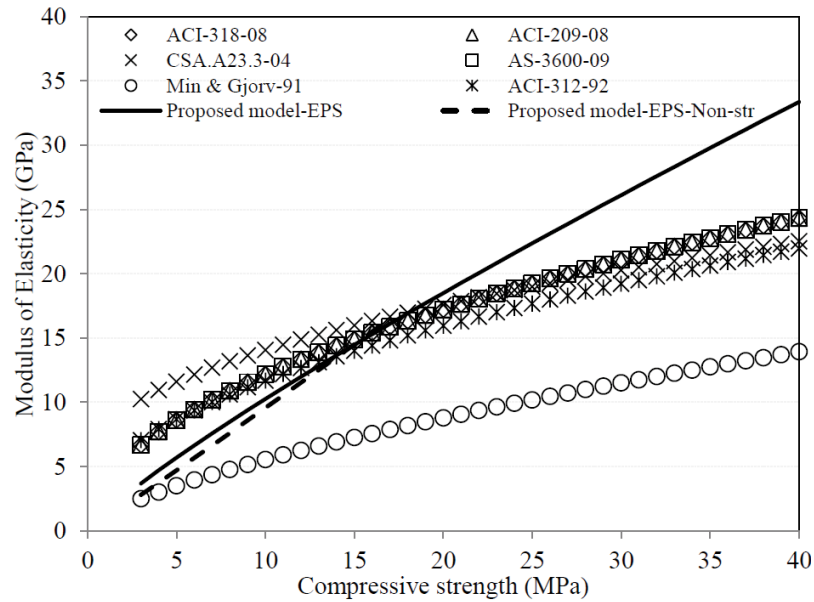


Figure 3.2: Comparison of the existing MoE models with the proposed models (Eq.3.1) (density 2000 kg/m³)

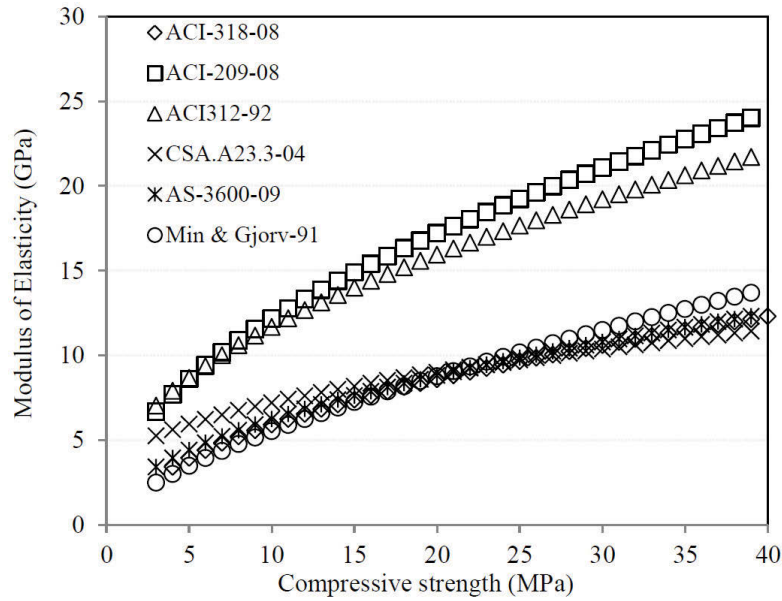


Figure 3.3: Comparison of the existing MoE models with the proposed models (Eq. 3.1) (density 1280 kg/m³)

3.4.2. Modulus of Rupture in Lightweight Concrete

3.4.2.1. Experimental and analytical database in EPS-LWC

Considering the modulus of elasticity, Table (3.8) presents a large number of experimental data relating to EPS-LWC. In this section, the proposed model to predict the Modulus of Rupture (MoR) of EPS-LWC is based on regression analysis of the experimental data. The power type models in most of the existing codes of practice and empirical equations are selected as the basis of the newly developed models in this study.

3.4.2.2. Proposed analytical model in this study

From the experimental results, some equations have been developed to predict the modulus of rupture from compressive strength. According to Chapter 2, the compressive strength above 17 MPa is considered as a minimum strength requirement of structural LWC. Hence, the developed equations are presented for classified ranges and the whole range of compressive strength respectively.

Equation (3.2) is a general form of the proposed model of predicting MOR from compressive strength. The proposed models include different types of natural and artificial aggregates, fillers and chemical admixtures, different size and content of EPS beads along with the variable ratios of water to binder in the mixtures. Table (3.11) gives the specialized coefficients of the proposed model for each group of the EPS-LWC. To evaluate the proposed models, correlation factor (R^2) in the least square method of regression analysis is given in the table as well.

$$f_r = \varphi \cdot (f'_c)^\Pi \quad \text{Eq. (3.2)}$$

Where; f_r is modulus of rupture (MPa), f'_c is compressive strength (MPa) and φ, Π are coefficients.

Table 3.11: Coefficients of Equation (3.2) and the compressive strength limit

	φ	Π	R^2	Range of f'_c (MPa)
Non-Structural EPS-LWC	0.524	0.7398	0.91	≤ 17
Structural EPS-LWC	1.266	0.4062	0.52	> 17
EPS-LWC	0.5769	0.6756	0.83	3-40

Figure (3.4) shows the predicted versus experimental values of MoR of EPS-LWC in two ranges of compressive strength. As shown, the developed equations accurately predict MoR of EPS-LWC, especially for non-structural concrete. However, more experimental data is required to develop accurate models for structural parts.

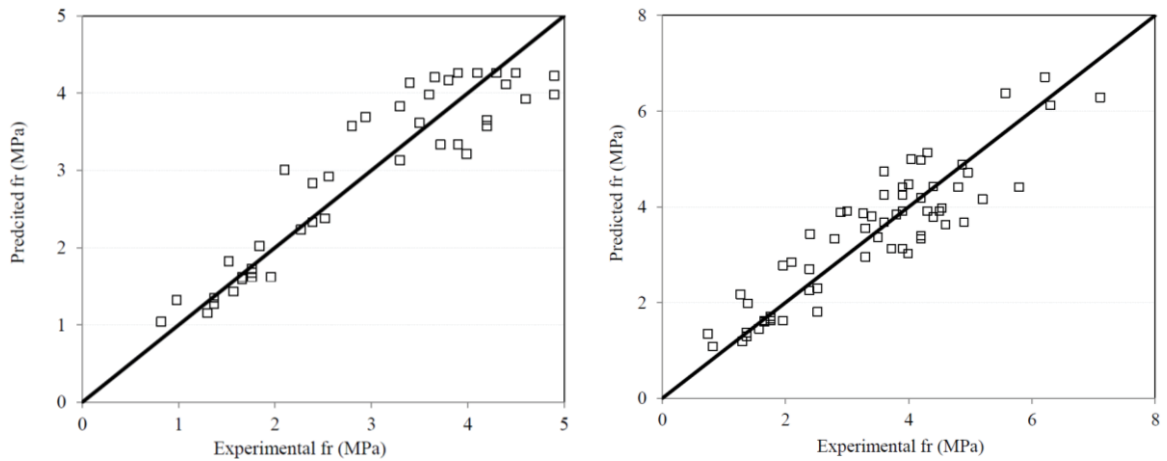


Figure 3.4: Comparison of the modulus of rupture in experimental data and prediction of Eq. (3.2)

3.4.3. Comparison of the Analytical Models of Modulus of Rupture

Modulus of rupture is predicted from compressive strength in various types of empirical equations and the models in codes of practice for different types of concrete. Table (3.13) gives some existing models in literature to estimate the MOR of LWC in the absence of accurate testing methods. The calculations of these models vary in complexity and precision.

Figure (3.3) shows the MoR versus compressive strength of LWC and compares the proposed models in Table (3.12) for EPS-LWC in non-structural data range and all data ranges of the compressive strength.

Table 3.12: Existing models to predict the modulus of rupture of lightweight concrete

Reference	$f_r = A(f'_c)^B + C$	Notes
Slate et al. (1986) [84]	$f_r = 0.35(f'_c)^{0.5}$	$20 \leq f'_c \leq 60$ MPa
Yusuf, I T; Jimoh, Y A (2013) [68]	$f_r = 0.399(f'_c)^{0.591}$	
Cousins et al. (2013)[91]	$f_r = k(f'_c)^{0.64}$	$0.26 \leq k \leq 0.36$
Alengaram et al. (2013)[92]	$f_r = 0.3(f'_c)^{2/3}$	
ACI 213R-03 [93]	$f_r = 0.45f'_c^{0.5}$	
Tassew and Lubell (2012) [90]	$f_r = 0.33f'_c^{0.5}$	
ACI 318-95 [60]	$f_r = 0.46f'_c^{0.5}$	
FHWA (2013) [94]	$f_r = 0.46f'_c^{0.5}$	
ACI-318-08 [59]	$f_r = 0.465f'_c^{0.5}$ $f_r = 0.527f'_c^{0.5}$	All LWC Sand LWC
AASHTO (2012) [95]	$f_r = 0.17f'_c^{0.5}$	
Bogas (2014) [96]	$f_r = 0.69f'_c^{0.5}$	

The total number of 11 models since 1986 in the codes of practice and the published research articles and technical reports are collected. The models are used to estimate the modulus of rupture from compressive strength in lightweight concrete. Majority of the models apply a power type relationship to estimate the modulus of rupture from compressive strength. However, effect of the concrete density is considered in some models. Also, limitation on the compressive strength is applied in some models.

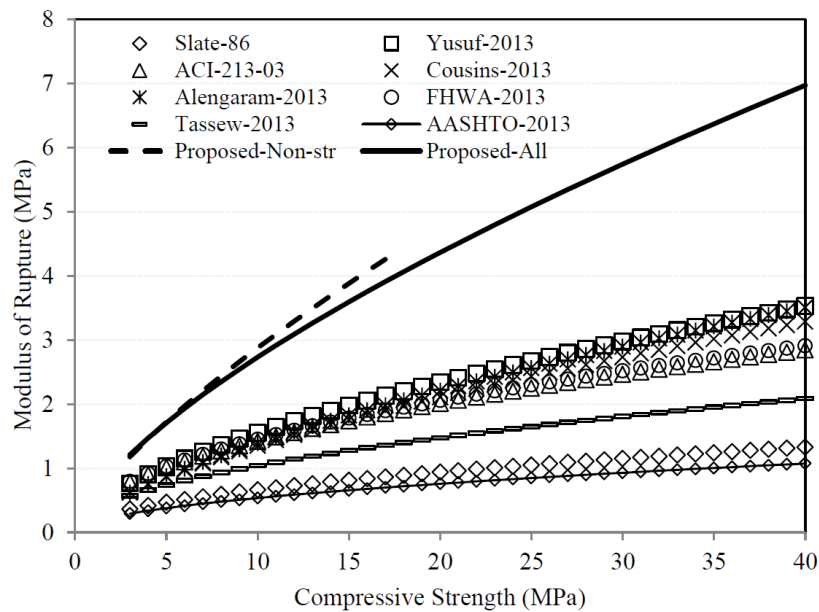


Figure 3.5: Comparison of the existing MoR models with the proposed models (density 1280 kg/m³)

From Figure (3.5), it is evident that the proposed model for non-structural EPS-LWC and EPS-LWC in all ranges of the compressive strength, predict higher values of MoR than other models presented in Table (3.9). Among the existing models, AASHTO (2013) and Slate (1986) models give the lowest predictions, while the models presented in ACI-213-03, FHWA (2013), Cousins et al. (2013) and Alengaram et al. (2013) predict close values of MoR for structural and non-structural LWC.

3.4.4. Splitting Tensile Strength in Lightweight Concrete

3.4.4.1. Experimental and analytical database in EPS-LWC

Considering the modulus of elasticity, Table (3.5) gives a considerable amount of experimental data in regard to EPS-LWC. In this section, the proposed model of predicting the Splitting Tensile Strength (STS) of EPS-LWC is based on the regression analysis of the experimental data. The power type relationships in most of the existing codes and empirical equations are selected as the basis of developing new models.

3.4.4.2. Proposed analytical model in this study

From the experimental results some equations are developed to predict STS of LWC from compressive strength. The developed equations are presented for classified ranges of compressive strength.

Equation (3.3) is a general form of the proposed model to predict STS from compressive strength. The proposed models include different types of natural and artificial aggregates, fillers and chemical admixtures, different size and content of EPS beads, along with the variable ratios of water to binder in the mixtures. Table (3.13) gives the specialized coefficients of the proposed model for each group of the EPS-LWC. To evaluate the proposed models, correlation factor (R^2) in the least square method is given in the table also.

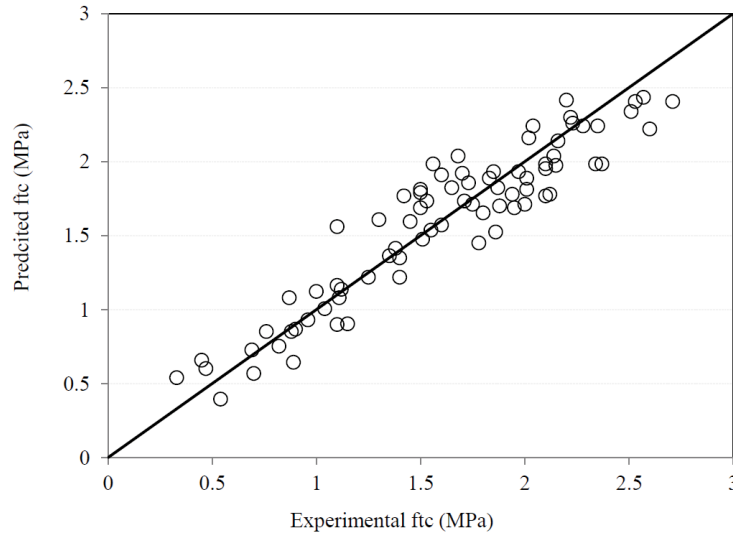
$$f_{ct} = \varphi \cdot (f'_c)^\omega \quad \text{Eq. (3. 3)}$$

Where; f_{ct} is splitting tensile strength (MPa), f'_c is compressive strength (MPa) and φ , ω are coefficients.

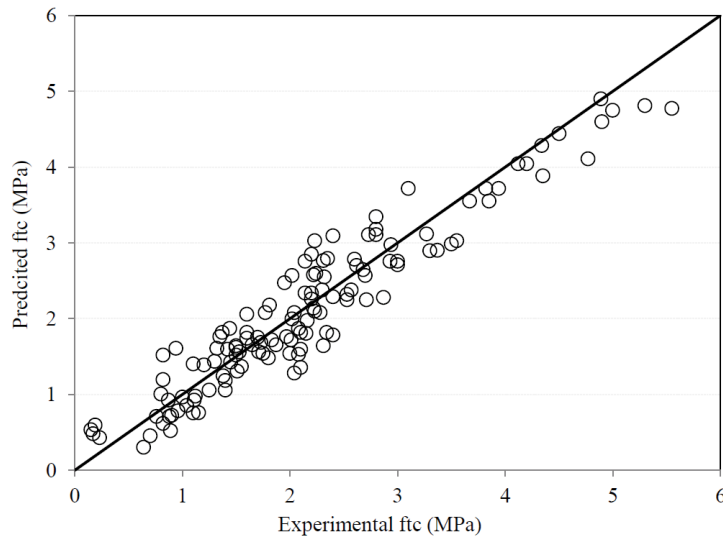
Table 3.13: Coefficients of equation (3.3) and the compressive strength limit

	φ	ω	R^2	f'_c limit (MPa)
Non-Structural EPS-LWC	0.3705	0.6644	0.98	≤ 17
Structural EPS-LWC	0.1862	0.8655	0.94	> 17
EPS-LWC	0.2818	0.7378	0.811	1-50

Figure (3.6) shows the predicted via experimental values of splitting tensile strength into two distinct ranges of compressive strength. As shown, the developed equations accurately predict the splitting tensile strength of EPS-LWC, especially for non-structural concrete.



(a) $f'_c < 17$ MPa



(b) $f'_c \geq 17$ MPa

Figure 3.6: Comparison of the experimental and predicted splitting tensile strength in EPS-LWC

3.4.4.3. Comparison of the STS analytical models

Splitting tensile strength for different types of concrete are predicted based on the compressive strength in various types of empirical equations and the models in some codes of practice. Table (3.10) gives some existing models in literature to estimate the STS of LWC in the absence of accurate testing methods. These models vary in complexity and precision in the calculations.

Figure (3.7) shows the STS versus compressive strength of LWC, and compares the proposed models in Table (3.14) in non-structural EPS-LWC and all data ranges of compressive strength in EPS-LWC.

Table 3.14: Existing models to predict the splitting tensile strength of lightweight concrete

Reference	$f_{ct} = A(f'_c)^B + C$	Notes
Shafigh et al. (2013) [97]	$f_{ct} = 0.23(f'_c)^{0.64}$	
Cousins et al. (2013)[91]	$f_{ct} = 0.21(f'_c)^{0.5}$	
Dinakar (2013) [89]	$f_{ct} = 0.268(f'_c)^{0.675}$	
Slate et al. (1986) [84]	$f_{ct} = 0.41(f'_c)^{0.5}$	$21 \leq f'_c \leq 62 \text{ Mpa}$
EC-2 (2002) [98]	$f_{ct} = 0.33(f'_c)^{0.67} (0.4 + \frac{0.6\gamma}{2200})$	$f'_c \geq 50 \text{ MPa}$
Wegian (2012) [99]	$f_{ct} = 0.087f'_c - 0.87$	
Bogas (2014) [96]	$f_{ct} = 0.47(f'_c)^{0.5}$	
Yew et al. (2014) [100]	$f_{ct} = 0.47(f'_c)^{0.5}$	
Kou et al. (2009) [101]	$f'_c = 15.36f_{ct} - 6.93$ $f_{ct} = 0.24(f'_c)^{0.69}$	$20 \leq f'_c \leq 41$

To estimate the splitting tensile strength from compressive strength, totally 9 models in the design codes and empirical equations since 1986 are presented. The power type relationship is used in majority of the models. Also, the compressive strength is limited in some models. Density is the only other parameter included in the models.

Figure (3.7) shows the splitting tensile strength versus compressive strength of EPS-LWC. It compares the proposed models of EPS-LWC with existing models presented in Table (3.14).

According to Figure (3.7), it is evident that the proposed model for the structural and non-structural EPS-LWC, predicts higher values of STS than other models presented in Table (3.10). The models presented by Cousins et al. (2013) and Wegian (2012) give the lowest estimation of STS for LWC in whole ranges of compressive strength. All other models in Figure (3.7) have comparable prediction for STS for structural and non-structural LWC.

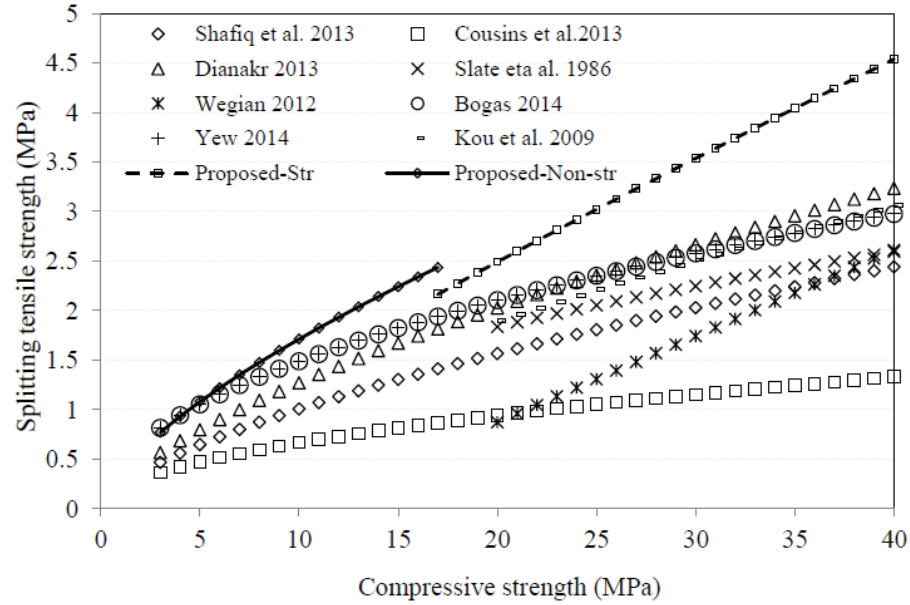


Figure 3.7: Comparison of the proposed and existing models of splitting tensile strength (density 2000 kg/m³)

3.4.5. Relationship between Modulus of Rupture and Modulus of Elasticity

The relationship between MoE and MoR in EPS-LWC is illustrated in Eq. (3.4). The related coefficients and the ranges of applied compressive strength are presented in Table (3.15).

$$f_r = \rho \cdot (E_c)^\sigma \quad \text{Eq. (3.4)}$$

Where; f_r is modulus of rupture (MPa), E_c is modulus of elasticity (GPa) and ρ , σ are coefficients.

Table 3.15: Coefficients of Equation (3.4) and compressive strength limit

	ρ	σ	R^2	f'_c limit (MPa)
EPS-LWC	0.7027	0.5591	0.78	≤ 50 MPa

3.4.6. Relationship between splitting tensile strength and modulus of elasticity

The relationship between MoE and STS in EPS-LWC is illustrated by Equation (3.5). The relevant coefficients and the ranges of applied compressive strength are also presented in Table (3.16).

$$f_{ct} = \chi \cdot (E_c)^\psi \quad \text{Eq. (3.5)}$$

Where; f_{ct} is splitting tensile strength (MPa), E_c is modulus of elasticity (GPa) and φ , ω are coefficients.

Table 3.16: Coefficients of Equation (3.5)

	χ	ψ	R^2	Range of f'_c
<i>EPS-LWC</i>	0.3867	0.5746	0.95	$\leq 50 \text{ MPa}$

3.5. MECHANICAL PROPERTIES OF CONVENTIONAL AND SELF-COMPACTING CONCRETE

In the absence of accurate testing systems, the models proposed in international codes of practice and the empirical models developed by some experimental investigations are useful tools in estimating the mechanical properties of concrete from compressive strength.

Along with the models presented for LWC in the previous parts of this chapter, the models for different properties of conventional concrete and self-compacting concrete are collected and presented also.

Modulus of elasticity, modulus of rupture and splitting tensile strength are the basic properties of concrete used to investigate structural behaviour. In addition, the compressive stress-strain curve and creep and shrinkage are other properties of concrete that are studied in the next sections.

3.5.1. Conventional Concrete

3.5.1.1. Modulus of elasticity

Table (3.17) shows the existing models used in some codes of practice to predict the modulus of elasticity in conventional concrete.

Table (3.18) presents some empirically developed models to predict the modulus of elasticity of conventional concrete. The experimental models are modified form of the models in the most frequently used codes of practice worldwide.

Table 3.17: Existing models of the MoE of conventional concrete in codes of practice

Design code	Model	Limits and coefficients
ACI 318- 08 [59]	$E_c = 0.043\gamma^{1.5}(f'_c)^{0.5}$ MPa, kg/m ³ $E_c = 4730(f'_c)^{0.5}$ MPa	1440 < γ < 2500 kg/m ³
OHBDC(1983) [102]	$E_c = 5000(f'_c)^{0.5}$ psi	
AIJ (1985) [67]	$E_c = 2.1 \times 10^5 (\gamma/2.3)^{1.5} (f'_c/200)^{0.5}$	E_c, f'_c : kgf/cm ² , γ = t/m ³
CEB-FIP (1990) [103]	$E_c = 21500\alpha(f'_c/10)^{1/3}$	$\alpha=1.2$ basalt, dense limestone, 1= quartzite , 0.9 limestone, 0.7 sandstone aggregate
NS-3473(1992) [66]	$E_c = 9.5(f'_c)^{0.3}(\frac{\gamma}{2400})^{1.5}$ GPa, kg/m ³ $E_c = 9500(f'_c)^{0.3}$	
ACI 312-92 [62]	$E_c = 14000 + 3250(f'_c)^{0.5}$ $E_c = 9500(f'_c + 8)^{1/3}$ $E_c = 0.043\gamma^{1.5}(f'_c)^{0.5}$	
CEB (1993) [104]	$E_c = 10000(f'_c + 8)^{1/3}$	
GBJ 11-89 (1994) [105]	$E_c = 10^2 / [2.2 + (34.7/f'_c)]$	
modified ACI 318-95 [60]	$E_c = 30.16\gamma^{1.5}(f'_c)^{0.5} + 484200$	E_c in psi; f'_c in psi; γ in pcf
NZS-3101-95 [64]	$E_c = (3320(f'_c)^{0.5} + 6900)(\gamma/2300)^{1.5}$	E_c, f'_c : MPa
ACI 363R-92, (1997) [106]	$E_c = 3320(f'_c)^{0.5} + 6890$	21 < f'_c < 83 MPa
BS 8110 (1997) [72]	$E_c = 20 + 0.2f'_c$	20 ≤ f'_c ≤ 60 MPa
EHE (1998) [107]	$E_c = 10,000\sqrt[3]{f'_c}$	
BS 5400-4(1999) [73]	$E_c = 8.6475(f'_c)^{0.348}$	20 ≤ f'_c ≤ 60 MPa
FHWA (2000) [108]	$E_c = 3837(f'_c)^{0.5}$	28 ≤ f'_c ≤ 193 MPa
TS-500 (2000) [109]	$E_c = 3.25(f'_c)^{0.5} + 14$	
IS 456 (BIS, 2000) [110]	$E_c = 5000(f'_c)^{0.5}$	
GDC (2000) [111]	$E_c = 4.76(f'_c)^{0.5}$	
EC2-02 (2002) [98]	$E_c = 22000(f'_c/10)^{0.3}$ MPa	Secant between 0 and $0.4f_{cm}$
NCHRP-2003 [112]	$E_c = 33000k_1k_2(0.14 + f'_c/1000)^{1.5} \cdot (f'_c)^{0.5}$ ksi	$K_1=1.0$, K_2 =90th percentile upper bound and the 10th percentile lower bound 2320 < γ < 2480 kg/m ³
NBR-6118 (2003) [113]	$E_c = 5600(f'_c)^{0.5}$	
IDC 3274 (2003) [114]	$E_c = 5.7(f'_c)^{0.5}$	

Table 3.17: Existing models of the MoE of conventional concrete in codes of practice (continued)

Design code	Model	Limits and coefficients
SP 52-101(2003) [74]	$E_c = 11.652 \ln(f'_c) - 7.4713$	$10 \leq f'_c \leq 60$ MPa
CSA A23.3-04 [63]	$E_c = (3300(f'_c)^{0.5} + 6900)(\gamma/2300)^{1.5}$ $E_c = 4500(f'_c)^{0.5}$	E_c in MPa $20 \leq f'_c \leq 40$ MPa $1500 < \gamma < 2500$ kg/m ³
SABS-0100 (1992) Modified (2006) [115]	$E_c(\text{GPa}) = K_0 + \alpha f_{cu}$	K_0 (GPa) = 17 (ferro quartzite); 20 (Jukskei granite); 29 (Eikenhof andesit α (GPa/MPa) = 0.4 (ferro quartzite); 0.2 (Jukskei granite and Eikenhof andesit
JSCE (2007) [69]	$E_c = 4700(f'_c)^{0.5}$ (ACI-318-08) $E_c = 10.792 \ln(f'_c) - 9.0675$ best fit	$18 \leq f'_c \leq 80$ MPa
AASHTO-LRFD (2007) [57]	$E_c = 0.043 k_1 \gamma^{1.5} (f'_c)^{0.5}$ $K_1 = 1.0$ unless determined by physical test	kg/m ³ and MPa $1280 < \gamma < 2400$ kg/m ³ $14 \leq f'_c \leq 48$ MPa
ACI-209-2R-08 [61]	$E_c = 0.043 \gamma^{1.5} (f'_c)^{0.5}$ MPa, kg/m ³ $E_c = 4730(f'_c)^{0.5}$ MPa	for $1440 < \gamma < 2500$ kg/m ³
NTE E.060(2009) [71]	$E_c = 0.043 \gamma^{1.5} (f'_c)^{0.5}$ $E_c = 4730(f'_c)^{0.5}$	MPa, kg/m ³ MPa
AS-3600(2009) [65]	$E_c = 0.043 \gamma^{1.5} (f'_c)^{0.5}$ $E_c = 5050(f'_c)^{0.5}$	$f'_c < 40$ MPa, Secant between 0 and $0.4f_{cm}$
AS-3600(2009) [65]	$E_c = \gamma^{1.5} (0.024(f'_c)^{0.5} + 0.12)$	$f'_c > 40$ MPa
RakMK-D3(2012) [70]	$E_c = 5000(\gamma/2400)(f'_c)^{0.5}$	

The total number of 53 models since 1943 in the codes of practice and the published research articles and technical reports are collected to compare with the models for modulus of elasticity in the lightweight concrete and self-compacting concrete. Majority of the models are the power type relationship between the compressive strength and modulus of elasticity. Density of concert, type of aggregates and the limitation on the applied compressive strength in the models are the other effective parameters.

Table 3.18: Empirical models to predict the modulus of elasticity of conventional concrete

Researchers	Model	Limits and coefficients
Jensen (1943) [82]	$E_c = 6 \times 10^6 / (1 + 2000/f'_c)$ psi	$1280 < \gamma < 2400 \text{ kg/m}^3$ $14 \leq f'_c \leq 48 \text{ MPa}$
Pauw (1960) [83]	$E_c = 13.82\gamma^{1.79}(f'_c)^{0.44}$	
Carrasquillo, et al.(1981) [75]	$E_c = 3.320(f'_c)^{0.5} + 6.900$ ($W/2346$) $E_c = 3.320(f'_c)^{0.5} + 6.900$	GPa , kg/m3
Jobse and Mustafa (1984) [79]	$E_c = 0.103\gamma^{1.5}(\sqrt{f'_c})^{0.5}$	
Ahmad and Shah (1985) [78]	$E_c = 3.38 \times 10^{-5} \times \gamma^{2.5}(\sqrt{f'_c})^{0.65}$	
Ravindrarajah et al. (1985) [116]	$E_c = 4.630(f'_c)^{0.5}$	
Cook (1989) [80]	$E_c = 3.22 \times 10^{-5} \times \gamma^{2.5}(f'_c)^{0.315}$	$\text{kg/m}^3 \cdot \text{MPa}$
Zhang and Gjorv (1991) [81]	$E_c = 1.19(f'_c)^{2/3}$ GPa , MPa	
Gardner and Zhao(1991) [117]	$E_c = 9(f'_c)^{1/3}$	$f'_c > 27 \text{ MPa}$
Gutierrez and Canovas (1995) [118]	$E_c = 8340\sqrt[3]{f'_c}$	
Rashid et al. (2002) [119]	$E_c = 8900(f'_c)^{0.33}$	$20 < f'_c < 130 \text{ MPa}$
Levtchitch et el. (2004) [120]	$E_c = 10000(1.9 + .045f'_c)$	MPa
Leemann and Hoffmann (2005) [121]	$E_c = 5480(f'_c)^{0.5}$	
Kheder and Al-Windawi (2005) [122]	$E_c = 5.323(f'_c)^{0.453}$	GPa, MPa
Soleymani (2006) [76]	$E_c = (\gamma/2300)^{1.5}(3000(f'_c)^{0.5} + 6900)$ $E_c = (\gamma/2300)^{1.5}(3300(f'_c)^{0.5} + 6900)$ $E_c = 4500(f'_c)^{0.5}$	-Best fit with eq. 8-6 in CSA-A23.3(1994) -Best fit with eq. 8-6 in CSA-23.3(1994) -worst fit with eq. 8-7 in CSA-A23.3(1994)
YanJun Liu (2007) [123]	$E_c = \alpha(f'_c)^{0.5}$ E_c, f'_c in psi	$\alpha=55949$ Miami oolite limestone, 62721 for Georgia granite , 43777 for stalite lightweight aggregate
Dinakar (2008) [124]	$E_c = 4.55(f'_c)^{0.5}$	
Noguchi et al. (2009) [125]	$E_c = 3.48 \times 10^{-3}\gamma^{1.89}(f'_c)^{1/3}$	MPa, kg/m^3 , $40 \leq f'_c \leq 160 \text{ MPa}$
Haranki (2009) [77]	$E_c = 31.92 \times \gamma^{1.5}(f'_c)^{0.5} + 345,328$	psi , lb/ft^3
San Luis Obispo (2011) [126]	$E_c = 6.59(f'_c)^{0.38}$	
Aslani and Nejadi (2012) [127]	$E_c = 4835(f'_c)^{0.49}$	

3.5.1.2. Modulus of rupture

Table (3.19) presents number of empirically developed models used to predict the modulus of rupture of conventional concrete, together with the models in some codes of practice. The experimental models are the modified form of the models employed in the most frequently used codes of practice worldwide.

Table 3.19: Existing models to predict the modulus of rupture of conventional concrete

Reference	Modulus of rupture (MPa)
EC2-02 [128]	$f_r = 0.435(f'_c - 8)^{2/3}$
CSA A23.3-04 [63]	$f_r = 0.6(f'_c)^{0.5}$ $f_r = 0.6\lambda(f'_c)^{0.5}$
ACI-363R-92 [129]	$f_r = 0.94(f'_c)^{0.5}$ $21 \leq f'_c \leq 83$ MPa
Rashid et al. (2002) [119]	$f_r = 0.42(f'_{c-150})^{0.68}$ $5 \leq f'_c \leq 120$ MPa
IS-456 (BIS, 2000) [130]	$f_r = 0.7(f'_c)^{0.5}$
Ahmad and shah (1985) [78]	$f_r = 0.44(f'_c)^{0.5}$ $f'_c < 84$ MPa
Jerome (1984)	$f_r = 0.44(f'_c)^{2/3}$ $f'_c < 62$ MPa
ACI-435 (1968) [131]	$f_r = 0.62$ to $0.99(f'_c)^{0.5}$
Carrasquillo et al. (1981) [132]	$f_r = 0.97(f'_c)^{0.5}$
Bakhsh et al -(1990) [133]	$f_r = 0.8(f'_c)^{0.5}$
Raphael, J.M.- (1984) [134]	$f_r = 0.44(f'_c)^{2/3}$
Lane-(1998) [135]	$\log f_r = 0.555(\log f'_c) - 0.197$

The collected 12 models to predict the modulus of rupture of conventional concrete cover the period since 1968 up to now. The power type relationship is the governing type model. Limitation on the applied range of compressive strength is effective parameter in some models.

3.5.1.3. Splitting tensile strength

Table (3.20) presents some empirically developed models to predict the splitting tensile strength of conventional concrete together with the models used in some codes of practice. The experimental models are the modified form of the models employed in the most frequently used codes of practice universally.

For prediction of the splitting tensile strength, totally 26 models in the design codes and empirical equations since 1962 are presented. The power type relationship is used in majority of the models. Also, the compressive strength is limited in some models.

3.5.2. Self-compacting Concrete

3.5.2.1. Modulus of elasticity

Self-compacting concrete is a comparatively new construction material in comparison to conventional and light-weight concrete and the research to discover the most accurate models of predicting its mechanical properties are still continuing. Therefore, there are fewer models for modulus of elasticity, splitting tensile strength and modulus of rupture of self-compacting concrete in literature. Table (3.21) presents some existing models of predicting the modulus of elasticity of self-compacting concrete in design codes and also the empirical models developed based on the experimental investigations. The experimental models are the modified form of the models in the most frequently used codes of practice globally.

Table 3.20: Existing models to predict the splitting tensile strength of conventional concrete

References	Splitting tensile strength (MPa)
Akawaza (1953) [136]	$f_{ct} = 0.209(f'_c)^{0.73}$
Carneriro and Barcellos (1953) [137]	$f_{ct} = 0.185(f'_c)^{0.735}$
Vinayaka (1959) [138]	$f_{ct} = 0.88(f'_c)^{0.716}$
Sen and Desayi (1962) [139]	$f_{ct} = 0.682(f'_c)^{0.73}$
Chapman, G.P., 1968)	$f_{ct} = 0.55(f'_c)^{0.5}$
Mirza, S.A et al. (1979)	$f_{ct} = 0.3(f'_c)^{2/3}$
Carino and Lew (1982) [140]	$f_{ct} = 0.272(f'_c)^{0.71}$
Raphael (1984) [134]	$f_{ct} = 0.313(f'_c)^{0.667}$
Ahmad and shah (1985) [78]	$f_{ct} = 0.46(f'_c)^{0.5}$ Mpa $f'_c < 84$ MPa
Gardner (1990) [141]	$f_{ct} = 0.313(f'_c)^{0.667}$
Olokun (1991) [142]	$f_{ct} = 0.206(f'_c)^{0.69}$
ACI-363-R (1992) [129]	$f_{ct} = 0.59f'_c^{0.5}$ $21 \leq f'_c \leq 83$ MPa
Burg and Ost (1992) [143]	$f_{ct} = 7.3(f'_c)^{0.5}$
CEB-FIP (1993) [144]	$f_{ct} = 0.3(f'_c)^{2/3}$
ACI318 (1995) [60]	$f_{ct} = 0.56(f'_c)^{0.5}$
Lane (1998) [135]	$\log f'_c = 1.096(\log f_{ct}) + 0.85$
EC2-02 (2002) [128]	$f_{ct} = 1/3(f'_c - 8)^{2/3}$ MPa
CSA A23.3-04 [63]	$f_{ct} = 0.67(f'_c)^{0.5}$
NEN-6722 (2000) [145]	$f_{ct} = 1 + 0.05f'_c$
Rashid et al. (2002) [119]	$f_{ct} = 0.47(f'_c)^{0.56}$ $5 \leq f'_c \leq 120$ MPa
NEN-6722 (2000) [145]	$f_{ct} = 1 \frac{N}{mm^2} + 0.05f'_c$
Hueste et al. (2004) [146]	$f_{ct} = 0.55(f'_c)^{0.5}$
AASHTO (2006) [147]	$f_{ct} = 0.59f'_c^{0.5}$
Dinakar et al. (2008) [124]	$f_{ct} = 0.65(f'_c)^{0.5}$
Dinakar (2012) [89]	$f_{ct} = 0.29(f'_c)^{0.675}$
Aslani and Nejadi (2012) [127]	$f_{ct} = 0.19f'_c^{0.78}$

Table 3.21: Existing models of the modulus of elasticity of self-compacting concrete

References	Modulus of elasticity (SI units, Mpa)
Gardener and Zhao (1991) [117]	$E_c = 9000^3 \sqrt{f'_c}$ $f'_c > 27$ MPa
NS 3473 (1992) [148]	$E_c = 9.5 \left(\frac{\gamma}{2400} \right)^{1.5} (f'_c)^{0.3}$, $25 < f'_c < 85$ MPa
ACI 363R-92 [129]	$E_c = 3300(f'_c)^{0.5} + 6900$
CEB-FIP (1993) [144]	$E_c = 21.5^3 \sqrt{\frac{f'_c}{10}}$ GPa, MPa
EHE (1998) [107]	$E_c = 10000^3 \sqrt{f'_c}$
ACI-318-1999 [60]	$E_c = 43\gamma^{1.5} (f'_c)^{0.5} \times 10^{-6}$
Persson (2001) [149]	$E_c = 3750(f'_c)^{0.5}$
EC-2 (2002) [128]	$E_c = 22 \left(\frac{f'_c}{10} \right)^{0.3}$ GPa, MPa
NBR 6118 (2003) [113]	$E_c = 5600(f'_c)^{0.5}$
Huest et al. (2004) [146]	$E_c = 5230(f'_c)^{0.5}$
CSA A23.3-04 [63]	$E_c = 4500(f'_c)^{0.5}$ Mpa
Leemann and Hoffmann (2005)[121]	$E_c = 4740(f'_c)^{0.5}$
NZS 3101 (2006) [64]	$E_c = 3320(f'_c)^{0.5} + 6900$ MPa
Felekoglu et al.(2007) [150]	$E_c = 1.57(f'_c)^{0.8}$ MPa, GPa
AASHTO LRFD (2007)[147]	$E_c = 0.043K_1 \cdot \gamma^{1.5} (f'_c)^{0.5}$ MPa and kg/m ³ units 1440 < γ < 2480 kg/m ³ , $f'_c < 103$ MPa
Atahan et al. (2007) [151]	$E_c = 10150 \left(\frac{\gamma}{2400} \right)^{3.3} (f'_c)^{0.343}$ MPa , kg/m ³
Dinakar et al. (2008) [124]	$E_c = 4180(f'_c)^{0.5}$
Kim (2008) [152]	SCC mixtures containing river gravel: $E_c = 8981(f'_c)^{0.378} (\gamma/2400)^1$ $E_c = 8278(f'_c)^{0.39} + 424$ For limestone SCC mixture: $E_c = 9420(f'_c)^{0.383} (\gamma/2400)^{2.3}$ To compare the MOE between SCC and CC containing limestone coarse aggregate, the following equation is proposed: $E_c = 7120(f'_c)^{0.40} + 621$ For the unified design equations, the following equation is proposed for SCC mixtures: $E_c = 10153(f'_c)^{0.343} (\gamma/2400)^{3.3}$
Dinakar (2008) [124]	$E_c = 4.18(f'_c)^{0.5}$ in fsp
ACI 318-08 [153]	$E_c = 4700(f'_c)^{0.5}$ MPa
Vilanova et al. (2011) [154]	$E_c = 5.88(f'_c)^{0.44}$ GPa, Mpa
Hasan Yıldırım et al.(2011) [155]	$E_c = 5.85(f'_c)^{0.5} - 13.5$
Debashis Das (2011) [156]	$E_c = 3615.6(f'_c)^{0.5} + 6887.6$ applicable for various ages
Aslani and Nejadi (2012) [127]	$E_c = 4150(f'_c)^{0.525}$

The collected 24 models to predict the modulus of elasticity of self-compacting concrete are developed since 1985 up to now. The power type relationship is used in most of the presented models. Effect of the aggregate type, density of concrete and the compressive strength limit are some effective parameters included in the model.

3.5.2.2. Modulus of rupture

Table (3.22) presents some existing models of predicting the modulus of rupture of self-compacting concrete in design codes and also the empirical models developed based on the experimental investigations. The experimental models are the modified form of the models in the most frequently used codes of practice worldwide.

Table 3.22: Modulus of rupture of self-compacting concrete in codes of practice and empirical equations

References	Modulus of rupture (MPa)
Vilanova et al. (2011) [154]	$f_r = 0.43f_c'^{0.68}$
EC-2 (2002) [128]	$f_r = 0.435(f_c' - 8)^{2/3}$
CSA A23.3-04 [63]	$f_r = 0.6\sqrt{f_c'}$
NZS-3101 (2006) [64]	$f_r = 0.8\sqrt{f_c'}$
Atahan et al. (2007) [151]	$f_r = 375 \times 10^{(3.7 \times 10^{-5} f_c')}$ psi $f_r = 2.59 \times 10^{(5.3 \times 10^{-3} f_c')}$
ACI-318-08 [153]	$f_r = 0.94\sqrt{f_c'}$

The models to predict the modulus of rupture of self-compacting concrete are very rare and just 6 models are found in the published articles and technical reports. These models are proposed since 2002. Similar to the conventional concrete, the models are of mainly power type relationship.

3.5.2.3. Splitting tensile strength

Some models to estimate the splitting tensile strength of self-compacting concrete are presented in Table (3.23). Some models are proposed in codes of practice, while some models are based on the experimental investigations. The majority of the empirical models are modified versions of the most frequently used models in the literature.

To predict the splitting tensile strength, totally 21 models developed since 1985 are presented in the study. Majority of the models are based on the power type relationship between the compressive strength and modulus of elasticity. In all models, the splitting tensile strength is derived from the compressive strength.

3.6. COMPRESSIVE STRESS-STRAIN CURVE IN LIGHTWEIGHT, CONVENTIONAL AND SELF-COMPACTING CONCRETE

The stress-strain curve is a particularly important graphical presentation of the mechanical properties of concrete. Experimental stress-strain relationships dealing with different stress regions are the most usual representation of material properties [157]. The curve is unique for each type of concrete and is obtained from compression testing of cylinders with a controlled rate of deformation (displacement) in the concrete specimen subjected to compression. To draw the stress-strain curve, the deformation (strain) at distinct intervals of compressive loading (stress) is recorded. These curves expose many of the properties of a material such as the modulus of elasticity and energy absorption of the concrete specimen. Ductility and brittleness of the concrete can be found from the stress-strain curves.

Table 3.23: Splitting tensile strength of self-compacting concrete in codes of practice and experimental studies

References	Splitting tensile strength (MPa)
Ahmad and Shah (1985) [78]	$f_{ct} = 4.34f_c'^{0.55}$
Olokun (1991) [142]	$f_{ct} = 0.295f_c'^{0.69}$
Burg and Ost (1992) [143]	$f_{ct} = 7.3f_c'^{0.5}$
CEB-FIP, (1993) [144]	$f_{ct} = 0.301f_c'^{0.67}$
CEB-FIP (1993) [144]	$f_{ct} = 1.56 \sqrt[3]{\left(\frac{f_c' - 8}{10}\right)^2}$
EHE (1998) [107]	$f_{ct} = 0.21 \sqrt[3]{f_c'^2}$
ACI-318 (1999) [60]	$f_{ct} = 0.556f_c'^{0.5}$
EC-2 (2002) [128]	$f_{ct} = (1/3 (f_c' - 8))^{2/3}$
NBR- 6118 (2003) [113]	$f_{ct} = 0.3 \sqrt[3]{f_c'^2}$
CSA A23.3-04 [63]	$f_{ct} = 0.67f_c'^{0.5}$
NZS- 3101 (2006) [64]	$f_{ct} = 0.54f_c'^{0.5}$
Atahan et al. (2007) [151]	$f_{ct} = 0.295f_c'^{0.69}$
Felekoglu et al. (2007) [150]	$f_{ct} = 0.43f_c'^{0.6}$
Dinakar et al. (2008) [124]	$f_{ct} = 0.82f_c'^{0.5}$
ACI-318-08 [153]	$f_{ct} = 0.59f_c'^{0.5}$
Sukumar et al. (2008) [158]	$f_{ct} = 0.0843f_c' + 0.818$
Kim (2008) [152]	$f_{ct} = 0.68f_c'^{0.5}$
Topcu and Uygungözü (2010) [159]	$f_{ct} = 0.062f_c' + 0.2009$
Parra et al. (2011) [160]	$f_{ct} = 0.28f_c'^{0.67}$
Vilanova et al. (2011) [154]	$f_{ct} = 0.26f_c'^{0.71}$
Aslani and Nejadi (2012) [127]	$f_{ct} = 0.115f_c'^{0.893}$

Increasing the strain capacity of concrete causes an increase in the area under the stress-strain diagram and improves the energy absorption capacity. The enhanced energy absorption capacity changes the concrete into a more ductile material [100]. The greater strain capacity also improves the resistance to cracking due to restrained shrinkage [161].

Stress–strain curves of various concrete types vary widely; and different compression tests conducted on the same concrete yield different results, depending upon the temperature of the specimen and the speed of the loading. Even for the same concrete mixture, these curves are sensitive to some other parameters. Concrete characteristics and testing conditions are two major effective parameters in stress–strain curves of concrete. Shape and size of the specimen, stiffness of the testing machine, type of the strain gauge, gauge length and strain rate are relevant to testing conditions. Also, the test age and mixture components can change the stress-strain curve [162].

Mathematical models proposed for the stress-strain relationship of concrete can be divided into three categories[163]:

- Continuous function for the ascending and descending branches of the stress-strain curve;
- Separate curves for the ascending and descending parts meeting at the peak stress;
- Parabolic curve for the ascending branch together with a bilinear falling curve.

Figure (3.8) shows the general relationship between stress and strain in ductile and brittle failure of the concrete.

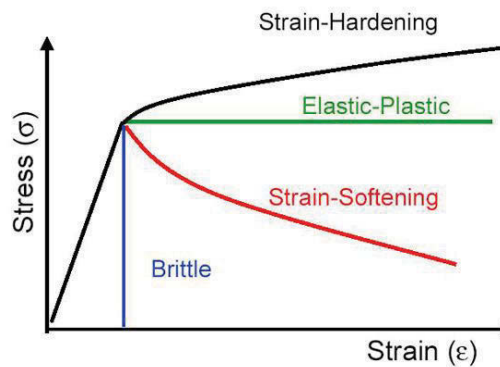


Figure 3.8: Different types of concrete failure in compression

The relationship between the peak compressive stress and corresponding axial strain is reported as linear in some investigations [163, 164], while some other studies such as Tasnimi

(2004) [165] propose a polynomial function to describe the relation between compressive stress and strain in concrete.

Lower stiffness of lightweight aggregate particles and higher cement contents in LWC result in larger deformations. However, these effects are alleviated by lower density so small scaled tests in the laboratory can provide pessimistic data, compared to behaviour on site. The stress-strain relationship for LWC is more linear and brittle than for normal-weight concrete. Such a response is probably attributable to greater compatibility [100], [166]. Figure (3.9) shows the general failure mode of LWC and CC. It is evident that for the same grade of concrete, LWC experiences a brittle failure.

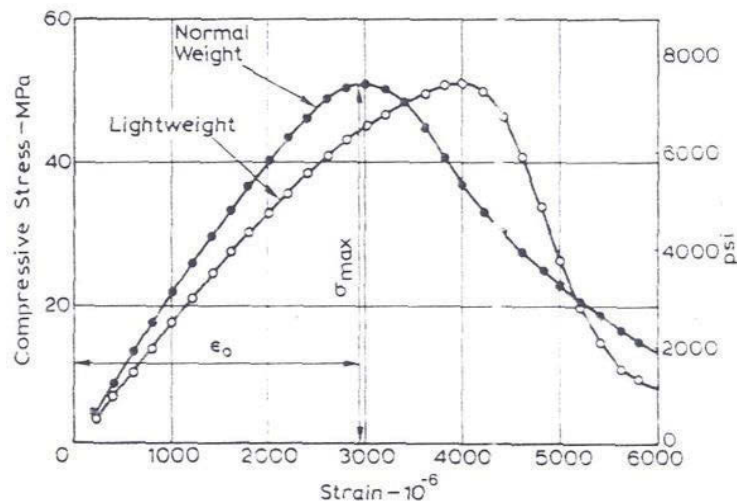


Figure 3.9: Comparing the failure modes of LWC and CC [166]

Yew et al. (2015) compared the stress-strain curves of Oil Palm Shell (OPS) LWC and conventional concrete. The stress-strain curves of most LWCs for both normal and high-strength levels were typically linear to the levels approaching 90% or greater of the failure strength [100].

Lo Monte et al. (2015) [52] investigated the compressive and tensile stress-strain curves of two mixes of LWC containing EPS beads and one CC mix. They applied thermal cycles at the temperature of 20°C (room temperature), 150°C, 300°C, 500°C, and 700°C. They found that both in virgin and slowly heated specimens, at any temperature, the stress-strain curves present a distinct linear ascending branch, with the strain at the peak of 1.8 to 2.5% at 20°C and 5.5–8.0% at 700°C, as in CC. The descending branch in conventional concrete was smoother at higher temperatures.

Santiago et al. (2009) [167] reported a similar stress-strain curve for all the recycled aggregate LWC and the natural aggregate concrete. The post-cracking branch of stress-strain curve of LWC manufactured with waste Ethylene Vinyl Acetate (EVA) was softer. Also, the slope of the descending branch decreased in their experiment.

3.6.1. Compressive Stress-Strain Curve in Lightweight Concrete

So far, several analytical and numerical models have been developed to simulate the Compressive Stress-Strain Curve (CSSC) for different types of concrete. Table (3.24) shows some existing models for calculating CSSC of LWC. Complexity and number of the included effective parameters affect the precision of the model in calculation of the relation between stress and strain in these models.

The models to estimate the compressive stress-strain of lightweight concrete are very rare in the literature. The existing 4 models in the literature since 1975 are collected and compared in this study. Compressive strength, initial and final slope of the curve, stress and strain at the peak point and shape factor of the curve are the main parameters in the models. The first developed models generally contain less parameter and predict the stress-strain behaviour by a simple cure.

Table 3.24: Existing models of compressive stress-strain curve in lightweight concrete

References	CSSC model	Notes
Richard and Abbott (1975) [168]	$\sigma_c = \frac{(K - K_p)\epsilon_c}{\left[1 + \left(\frac{(K - K_p)\epsilon_c}{\sigma_0}\right)^n\right]^{1/n}} + K_p\epsilon_c$ $n = -\frac{\ln 2}{\ln\left(\frac{\sigma_1}{\sigma_0} - \frac{K_p}{K - K_p}\right)}$ $\sigma_1 = f'_c \left[2 \frac{\epsilon_1}{\epsilon_0} - \left(\frac{\epsilon_1}{\epsilon_0}\right)^2\right]$ $\epsilon_1 = \frac{0.65\sigma_0}{K - K_p}$ $\sigma_0 = 19.1 + 1.3f'_c - K_p\epsilon_0$ $K_p = 1374.5 - 871.1f'_c \text{ for } f'_c \geq 15 \text{ MPa}$ $K = E_c = 180.9 f'_c + 7770.7$ $\epsilon_0 = (0.2f'_c + 13.06) \times 10^{-4}$	σ_c is the plain concrete stress K : the initial slope of the curve K_p : the final slope of the curve σ_0 : reference stress n : curve-shape parameter
Wang et al. (1978) [169]	$K = E_c = 180.9f'_c + 7770.7$ Initial slope of curves for Wang et al(1978)	
Almussalam and alsayed (1995) [163]	$n = -\frac{\ln 2}{\ln\left(\frac{f_1}{f_0} - \frac{K_p}{K - K_p}\right)}$ $f_1 = f'_c \left[2 \frac{\epsilon_1}{\epsilon_0} - \left(\frac{\epsilon_1}{\epsilon_0}\right)^2\right]$ $\epsilon_1 = \frac{\sigma_0}{K - K_p} \text{ for CC}$ $\epsilon_1 = \frac{0.65f_0}{K - K_p} \text{ for LWC}$ $\sigma_0 = 5.6 + 1.02f'_c - K_p\epsilon_c \text{ for CC}$ $\sigma_0 = 19.1 + 1.3f'_c - K_p\epsilon_c \text{ for LWC}$ $K_p = 5470 - 375f'_c \text{ for } f'_c \leq 55 \text{ MPa}$ $K_p = 16398.23 - 676.82f'_c$ $\text{for } f'_c > 55 \text{ MPa}$ $K_p = 1374.5 - 871.1f'_c$ $\text{for } f'_c \geq 15 \text{ MPa for LWC}$	
Hussin et al. (2013) [170]	$\sigma_c = \frac{f'_c \beta (\epsilon / \epsilon_0)}{[(\beta - 1 + (\epsilon / \epsilon_0)^\beta)]} \text{ if } \epsilon \leq \epsilon_0$ $\sigma_c = \frac{f'_c \cdot n \beta (\epsilon / \epsilon_0)}{[(n\beta - 1 + (\epsilon / \epsilon_0)^{n\beta})]} \text{ if } \epsilon > \epsilon_0$ $\beta = \frac{1}{1 - \frac{f'_c}{E_0 \epsilon_0}}$ $n = 0.0012P^2 - 0.0846P + 1.551$ $f'_c = 149.55e^{-0.1P}$ $\epsilon_0 = (2f'_c) \times 10^{-5} + 0.0008$	For permeable concrete σ_c, ϵ: concrete stress and strain f'_c, ϵ_0: Maximum stress and corresponding strain β: Material parameter E_0 : Initial tangent modulus of elasticity P: porosity of concrete

3.6.2. Compressive Stress-Strain Models in Conventional Concrete

Table (3.25) shows the existing models of predicting the relationship between stress and strain under compression in conventional concrete.

Table 3.25: Existing models of compressive stress-strain curve in conventional concrete

References	CSSC model	Notes
Hognestad (1951) [171]	$\sigma_c = f'_c \left[2 \left(\frac{\epsilon}{\epsilon_0} \right) - \left(\frac{\epsilon}{\epsilon_0} \right)^2 \right] \quad ; \quad \epsilon \leq \epsilon_0$ $\sigma_c = f'_c \left[1 - 0.15 \left(\frac{\epsilon - \epsilon_0}{\epsilon_u - \epsilon_0} \right) \right] \quad , \quad \epsilon > \epsilon_0$ $E_c = 12680 + 460 f'_c$	f'_c , ϵ_0 : Maximum stress and strain
Smith and Young (1955) [172]	$\sigma_c = E_c \epsilon \epsilon_0^{(1-\frac{\epsilon}{\epsilon_0})}$	
Desayi and Krishnan (1964) [173]	$\sigma_c = \left[\frac{E_c \epsilon}{1 + \left(\frac{\epsilon}{\epsilon_0} \right)^2} \right]$	
Saenz (1964) [174]	$\sigma_c = f'_c \left[\frac{E_c \epsilon}{1 + \left(\frac{E_c}{E_p} - 2 \right) \left(\frac{\epsilon}{\epsilon_0} \right) + \left(\frac{\epsilon}{\epsilon_0} \right)^2} \right]$ $E_p = \frac{f'_c}{\epsilon_0}$	
Richard and Abbott (1975) [168]	$\sigma_c = \frac{(K - K_p) \epsilon_c}{\left[1 + \left(\frac{(K - K_p) \epsilon_c}{\sigma_0} \right)^n \right]^{1/n}} + K_p \epsilon_c$ $n = - \frac{\ln 2}{\ln \left(\frac{\sigma_1}{\sigma_0} - \frac{K_p}{K - K_p} \right)}$ $\sigma_1 = f'_c \left[2 \frac{\epsilon_1}{\epsilon_0} - \left(\frac{\epsilon_1}{\epsilon_0} \right)^2 \right]$ $\epsilon_1 = \frac{\sigma_0}{K - K_p}$ $\sigma_0 = 5.6 + 1.02 f'_c - K_p \epsilon_0$ $K_p = 5470 - 375 f'_c \quad ; \quad f'_c \leq 55 \text{ MPa}$ $K_p = 16398.23 - 676.82 f'_c$ $f'_c > 55 \text{ MP}$ $K = E_c = 3320 \sqrt{f'_c} + 6900$ $\epsilon_0 = (0.2 f'_c + 13.06) \times 10^{-4}$	σ_c is the plain concrete stress K : the initial slope of the curve K_p : the final slope of the curve σ_0 : reference stress n : curve-shape parameter (for normal and high strength concrete)
Carreira and Chu (1985) [164]	$\sigma_c = \frac{f'_c \beta (\epsilon / \epsilon_0)}{[(\beta - 1) (\epsilon / \epsilon_0)]^\beta}$ $\beta = \frac{1}{\left[1 - \left(\frac{f'_c}{E_0 \epsilon_0} \right) \right]}$ $\epsilon_0 = (7.1 f'_c + 1680) \times 10^{-6}$	σ_c, ϵ : concrete stress and strain f'_c, ϵ_0 : Maximum stress and corresponding strain β : Material parameter E_0 : Initial tangent modulus of elasticity
Mazars and Pijaudier- Cabot (1989) [175]	$\sigma_c = E_c \epsilon \quad ; \quad \epsilon \leq \epsilon_0$ $\sigma_c = \left[(1 - \alpha) \epsilon_0 + \alpha \epsilon e^{\left(\frac{\epsilon_0 - \epsilon}{\epsilon'_c} \right)} \right] E_c \quad ; \quad \epsilon > \epsilon_0$ $\alpha = \frac{f'_c - E_c \epsilon_0}{E_c (\epsilon'_c e^{\frac{\epsilon_0 - \epsilon}{\epsilon'_c}} - \epsilon_0)}$	

Table 3.25: Existing models of compressive stress-strain curve in conventional concrete (continued)

References	CSSC model	Notes
Collins and Mitchell (1991) [176]	$\sigma_c = f'_c \left[\frac{n \left(\frac{\varepsilon}{\varepsilon_0} \right)}{n - 1 + \left(\frac{\varepsilon}{\varepsilon_0} \right)^{nk}} \right]$ $\varepsilon \leq \varepsilon_0 ; k = 1$ $\varepsilon > \varepsilon_0 ; k = 0.67 + \frac{f'_c}{62} , MPa$ $n = 0.8 + \frac{f'_c}{17} , \quad \varepsilon_0 = \frac{f'_c}{E_c} \frac{n}{n - 1}$	
CEB-FIP (1990) [103]	$\sigma_c = f'_c \left[\frac{\left(\frac{E_{it}}{E_0} \right) \left(\frac{\varepsilon}{\varepsilon_0} \right) - \left(\frac{\varepsilon}{\varepsilon_0} \right)^2}{1 + \left(\frac{E_{it}}{E_0} - 2 \right) \left(\frac{\varepsilon}{\varepsilon_0} \right)} \right] ; 0 \leq \varepsilon \leq \varepsilon_{\max}$ $\varepsilon_{\max} = \varepsilon_0 \left[\frac{1}{2} \left(\frac{E_{it}}{E_0} + 1 \right) + \sqrt{\frac{1}{2} \left(\frac{1}{2} \frac{E_{it}}{E_0} + 1 \right)^2 - \frac{1}{2}} \right]$ $\sigma_c = \frac{f'_c}{\left[\frac{1}{(\varepsilon_{\max}/\varepsilon_0)} \varphi - \frac{2}{(\varepsilon_{\max}/\varepsilon_0)^2} \right] \left(\frac{\varepsilon}{\varepsilon_0} \right)^2 + \left[\frac{4}{(\varepsilon_{\max}/\varepsilon_0)} - \varphi \right] \frac{\varepsilon}{\varepsilon_0}} ; \varepsilon > \varepsilon_{\max}$ $\varphi = \frac{4 \left[\left(\frac{\varepsilon_{\max}}{\varepsilon_0} \right)^2 \left(\frac{E_{it}}{E_0} - 2 \right) + 2 \left(\frac{\varepsilon_{\max}}{\varepsilon_0} \right) - \frac{E_{it}}{E_0} \right]}{\left[\left(\frac{\varepsilon_{\max}}{\varepsilon_0} \right) \left(\frac{E_{it}}{E_0} - 2 \right) + 1 \right]^2}$ $E_{it} = 21500 \sqrt[3]{f'_c/10} , \quad E_0 = \frac{f'_c}{0.0022} , \quad \varepsilon_0 = 0.0022$	
(Almusallam & Alsayed (1995) [163]	$\sigma_c = \frac{f'_c \beta (\varepsilon/\varepsilon_0)}{[(\beta - 1 + (\varepsilon/\varepsilon_0)^\beta)]}$ $\beta = \frac{1}{\left[1 - \left(\frac{f'_c}{E_0 \varepsilon_0} \right) \right]}$ $\varepsilon_0 = (0.398 f'_c{}^{0.44} - 6.748) \times 10^{-5}$	σ_c, ε : concrete stress and strain f'_c, ε_0 : Maximum stress and corresponding strain β : Material parameter E_0 : Initial tangent modulus of elasticity
Wee et al. (1996) [177]	$\sigma_c = f'_c \left[\frac{k_1 \beta (\varepsilon/\varepsilon_0)}{k_1 \beta - 1 + (\varepsilon/\varepsilon_0)^{k_2 \beta}} \right]$ $k_1 = (50/f'_c)^3 , \quad k_2 = (50/f'_c)^{1.3} ; f'_c \leq 50 MPa$ $k_1 = k_2 = 1 , \quad \text{for } f'_c > 50 MPa$ $\varepsilon_0 = 0.00078 (f'_c)^{0.25}$ $\beta = \frac{1}{\left[1 - \left(\frac{f'_c}{E_0 \varepsilon_0} \right) \right]}$ $E_c = 10200 \sqrt[3]{f'_c}$	For normal and high strength concrete σ_c, ε : concrete stress and strain f'_c, ε_0 : Maximum stress and corresponding strain β : Material parameter E_0 : Initial tangent modulus of elasticity
Tasnimi(2004) [162]	$\sigma_c = \frac{f'_c \beta (\varepsilon/\varepsilon_0)}{[(\beta - 1 + (\varepsilon/\varepsilon_0)^\beta)]}$ $\beta = \frac{1}{\left[1 - \left(\frac{f'_c}{E_0 \varepsilon_0} \right) \right]}$ $\varepsilon_0 = (65.57 f'_c{}^{0.44} - 6.748) \times 10^{-5}$	σ_c, ε : concrete stress and strain f'_c, ε_0 : Maximum stress and corresponding strain β : Material parameter E_0 : Initial tangent modulus of elasticity
Chandrasekhar et al. (2011) [178]	$\sigma_c = f'_c \left[\frac{\alpha \left(\frac{\varepsilon}{\varepsilon_0} \right)}{1 + \beta \left(\frac{\varepsilon}{\varepsilon_0} \right)} \right]$ $\frac{\varepsilon}{\varepsilon_0} = 1 \text{ then } \sigma_c = f'_c$ $\frac{\varepsilon}{\varepsilon_0} = 1 ; \quad d(\sigma_c/f'_c) \left(\frac{\varepsilon}{\varepsilon_0} \right) = 1$ Ascending portion: $\alpha=1.486$, $\beta=0.486$ Descending portion: $\alpha=2.462$, $\beta=1.462$	

Total number of 13 models to estimate the compressive stress-strain curve in the conventional concrete is collected. There are major differences in the simplicity and complexity of the models to include different parameters in the model. The oldest model is developed in 1951. Wide range of the limitation of the applicable compressive strength, boundary of the linear part of the curve, peak stress and strain and the ultimate strain are the main parameters in almost all the models. The first developed models generally contain less parameter and predict the stress-strain behaviour by a simple curve with limited number of parameters.

3.6.3. Compressive Stress-Strain Models in Self-compacting Concrete

Table (3.26) shows the existing models of predicting the relationship between stress and strain under compression in self-compacting concrete.

Compared to the conventional and lightweight concrete, self-compacting concrete is a new construction material and the models to estimate the compressive-stress-strain are very rare in the literature. In this study a total number of 4 models are presented to estimate the variation of stress-strain diagram in self-compacting concrete under compression. The included parameters are similar to the conventional concrete. However, the applied coefficients are different. The stress and strain at the peak point, initial slope, the ultimate strain and the shape factor of curve are the main parameters in the developed models.

Table 3.26: Existing models of compressive stress-strain curve for self-compacting concrete

References	CSSC model
Prasad et al. (2009) [179]	$\sigma_c = f'_c \left[\frac{2.8662(\varepsilon/\varepsilon_0)}{1 + 0.8662\left(\frac{\varepsilon}{\varepsilon_0}\right) + (\varepsilon/\varepsilon_0)^2} \right] : \varepsilon \leq \varepsilon_0$ $\sigma_c = f'_c \left[\frac{2.8662(\varepsilon/\varepsilon_0)}{1 + 0.8662\left(\frac{\varepsilon}{\varepsilon_0}\right) + (\varepsilon/\varepsilon_0)^2} \right] : \varepsilon > \varepsilon_0$
Kumar et al. (2011) [180]	$\sigma_c = f'_c \left[\frac{\alpha\left(\frac{\varepsilon}{\varepsilon_0}\right) + (k\beta - 1)\left(\frac{\varepsilon}{\varepsilon_0}\right)^2}{1 + (\alpha - 2)\left(\frac{\varepsilon}{\varepsilon_0}\right) + k\beta\left(\frac{\varepsilon}{\varepsilon_0}\right)^2} \right]$ $\beta = \frac{\left(\frac{\varepsilon_{80}}{\varepsilon_0}\right)^2 - (0.2\alpha + 1.6)\left(\frac{\varepsilon_{80}}{\varepsilon_0}\right)}{0.2\left(\frac{\varepsilon_{80}}{\varepsilon_0}\right)^2}$ $\alpha = E_0 \frac{\varepsilon_0}{f'_c} ; k = 1.85 - 0.025f'_c$ $\varepsilon_0 = 0.0006\sqrt[3]{f'_c} ; E_c = 5300(f'_c)^{0.46}$ $\varepsilon_{80} = (0.0015)(f'_c)^{0.16}$
Chandrasekhar et al. (2011) [178]	$\sigma_c = f'_c \left[\frac{\alpha\left(\frac{\varepsilon}{\varepsilon_0}\right)}{1 + \beta\left(\frac{\varepsilon}{\varepsilon_0}\right)} \right]$ $\frac{\varepsilon}{\varepsilon_0} = 1 \text{ then } \sigma_c = f'_c$ $\frac{\varepsilon}{\varepsilon_0} = 1 ; d(\sigma_c/f'_c)\left(\frac{\varepsilon}{\varepsilon_0}\right) = 1$ Ascending portion: $\alpha=1.331$, $\beta=0.331$ Descending portion: $\alpha=2.462$, $\beta=1.462$
Aslani and Nejadi (2012) [127]	$\sigma_c = f'_c \frac{n\left(\frac{\varepsilon}{\varepsilon'_c}\right)}{n - 1 + \left(\frac{\varepsilon}{\varepsilon'_c}\right)^n}$ $n = n_1 \left[1.02 - 1.17 \left(\frac{E_{sec}}{E_c} \right) \right]^{-0.74} ; \varepsilon \leq \varepsilon'_c$ $n = n_2 = n_1 + (\lambda + 28\mu) ; \varepsilon > \varepsilon'_c$ $\lambda = (135.16 - 0.1744f'_c)^{-0.46}$ $\mu = 0.83 \exp\left(-\frac{911}{f'_c}\right)$ $E_{sec} = f'_c / \varepsilon'_c$ $\varepsilon'_c = \left(\frac{f'_c}{E_c} \right) \left(\frac{\psi}{\psi - 1} \right), \psi = \frac{f'_c}{17} + 0.8$

3.7. STEEL-CONCRETE BOND CHARACTERISTICS

The bond stress between reinforcing bars and the surrounding concrete is the shearing force that results from the chemical adhesion, friction and mechanical interaction between concrete and steel. The first two parameters are typical in plain and deformed bars, and the last parameter is mostly the matter of deformed bars.

3.7.1. Bond Characteristics in Lightweight Concrete

The bond strength of LWC is similar to that of normal-weight concrete. Anchorage in LWC is lower than that of conventional concrete due to the lower bearing strength caused by the relative weakness of lightweight aggregates [166].

According to BS-8110 [72], the bond stresses due to determined lap and anchorage lengths for LWC can be taken as being as conservative as 80% of those for the same grade of compressive strength in CC. Table (3.27) shows some existing models of predicting the bond between reinforcing steel bars and surrounding concrete in lightweight concrete.

Despite numerous models to predict different mechanical properties of concrete, only one model is found in the literature to describe the relationship between bond stress between reinforcement and concrete. This model is particularly developed for lightweight concrete in 2013. Bar diameter, compressive strength of concrete and the embedded length of bar are the main included parameters in the models.

Table 3.27: Models to estimate the steel-concrete bond stress in light-weight concrete

Reference	Bond stress (MPa)	Notes
Kim et al. (2013) [181]	$\tau_{max} = \left[\frac{37.5}{(d_b + l_d)^{0.25}} - 9.4 \right] \sqrt{f'_c}$	d_b : rebar diameter l_d : embedded length

3.7.2. Bond Characteristics in Conventional Concrete

Table (3.28) shows some existing models of predicting the bond stress between reinforcing steel bars and surrounding concrete in conventional concrete.

The bond stress between the concrete and embedded steel bar is widely investigated in the literature since 1977 and 25 models are collected in this study. Diameter and embedded length of steel bar, compressive strength of surrounding concrete and the concrete cover are almost the main parameters to define the bond stress in the models. However, in some models the tensile strength of steel bar, plane or deformed bar and the ribs on the bar are included.

Table 3.28: Existing models to estimate the steel-concrete bond stress in conventional concrete

References	Bond model	Notes
Orangun et al (1977) [182]	$\tau_{max} = \left[1.22 + 3.23 \frac{c}{d_b} + 53 \frac{d_b}{l_d} \right] (f'_c)^{0.5}$ $\tau_{max} = \left[0.1 + 0.25 \left(\frac{C_m}{d_b} \right) + 4.15 \left(\frac{d_b}{l_d} \right) \right] (f'_c)^{0.5}$	d_b : rebar diameter l_d : embedded length c_m : smaller of clear bottom cover to main reinforcement or half clear spacing between bars
Kemp and Wilhelm (1979) [183]	$\tau_{max} = \left(0.55 + 0.24 \frac{c}{d_b} \right) (f'_c)^{0.5} + 0.191 \frac{A_t f_y}{s \cdot d_b}$	
Eligehausen et al. (1983) [184]	$\tau_{max} = 0.75 \sqrt{\frac{c \cdot f'_c}{d_b}}$	
Kemp (1986) [185]	$\tau_{max} = 2322 + 2.716 \frac{c}{d_b} (f'_c)^{0.5}$	
Robins and Austin (1986) [186]	$\tau_{max} = 1.38 \sqrt{f'_c}$	
Chapman and Shah (1987) [187]	$\tau_{max} = \left[3.5 + 3.4 \frac{c}{d_b} + 57 \frac{d_b}{l_d} \right] (f'_c)^{0.5}$	
Harajli (1994) [188]	$\tau_{max} = \left[1.2 + 3 \frac{c}{d_b} + 50 \frac{d_b}{l_d} \right] (f'_c)^{0.5}$	
Huang et al. (1996) [189]	$\tau_{max} = 0.45 f'_c$	
Esfahani and Rangan (1998) [190]	$\tau_{max} = \left(2.695 \times \frac{\frac{c}{d_b} + 0.5}{\frac{c}{d_b} + 5.5} \right) (f'_c)^{0.5}$	
Pillai et al. (1999) [191]	$\tau_{max} = \left[0.1 + 0.25 \frac{c}{d_b} + 4.2 \frac{d_b}{l_d} + 0.024 \frac{A_{tr} f_y}{s \cdot d_b} \right] (f'_c)^{0.5}$	
CEB-FIP (1990) [103]	$\tau_{max} = 2.5 \sqrt{f'_c}$ $\tau_{max} = 2.0 \sqrt{f'_c}$	Confined Unconfined
Zuo and Darwin (2000) [192]	$\frac{T_{max}}{(f'_c)^{0.25}} = [1.44 l_d (c + 0.5 d_b) + 56.3 A_s] \times \left(0.1 \frac{C_m}{c} + 0.9 \right)$	T_{max} : ultimate bond force (N) A_s : cross section area C_m : maximum of clear bottom cover to main reinforcement and half clear spacing between bars
ACI-408R (2003) [193]	$\tau_{max} = 20.23 \frac{\sqrt{f'_c}}{d_b}$	
Sancak (2009) [194]	$\tau_{max} = 0.63 (f'_c)^{0.714}$	At 90 days
Tanyildizi (2009) [195]	$\tau_{max} = 0.1796 f'_c - 0.3845$	
Desnerck (2010) [196]	$\tau_{max} = \left(1.94 + 0.0291 \frac{c}{d_b} \right) \sqrt{f'_c}$	

Table 3.28: Models to estimate the steel-concrete bond stress in conventional concrete (continued)

References	Bond model	Notes
Aslani and Nejadi (2012) [197]	$\tau_{max} = \left[0.7 \left(\frac{c}{d_b} \right)^{0.6} + 4 \left(\frac{d_b}{l_d} \right) \right] (f'_c)^{0.23}$ $\tau_{max} = \left[0.7 \left(\frac{c}{d_b} \right)^{0.6} + 4 \left(\frac{d_b}{l_d} \right) \right] (f'_c)^{0.23}$	Plain rebar Deformed rebar
Kim et al. (2013) [181]	$\tau_{max} = \left[\frac{37.5}{(d_b + l_d)^{0.25}} - 9.4 \right] \sqrt{f'_c}$	
Nilson (1971) [198]	$\tau_{max} = (1.43x + 1.5)\sqrt{f'_c}$	
ACI-408R (2003) [193]	$\tau_{max} = 20.23 \frac{\sqrt{f'_c}}{d_b}$	d_b : bar diameter of the (mm) f'_c : compressive strength (MPa).
Wu and Zhou (2012) [199]	$\tau_{max} = \left[\frac{2.5}{1 + 3.1e^{-0.47(K_{co} + 33K_{st})}} \right] \sqrt{f'_c}$	$K_{co} = c/d_b$ combined parameter of bar diameter and concrete cover $K_{st} = A_{st}/nS_{st} d_b$ confinement factor
Diab et al. (2014)[200]	$\tau_{max} = [0.18468 + 0.09234(c/d_b) + 1.6038(d_b/l_d) + 0.6318(h_r/s_r)]\sqrt{f'_c} \quad \text{for } f'_c < 80 \text{ MPa}$ $\tau_{max} = [0.08262 + 0.09234(c/d_b) + 1.6038(d_b/l_d) + 0.6318(h_r/s_r)]\sqrt{f'_c} \quad \text{for } f'_c \geq 80 \text{ MPa}$	C : $\min(c_x, c_y, c_s/2)$ d_b : bar diameter l_d : embedded length h_r : rib height s_r : rib spacing
BS-8110 (1997) [72]	$\tau_{max} = 0.4\sqrt{f'_c}$	
EN-92 (2002) [128]	$\tau_{max} = 2.25\eta_1\eta_2f_{ct}$ $f_{ct} = \frac{0.2f'_c{}^{2/3}}{\gamma c} \quad ; \quad f'_c \leq 60 \text{ MPa}$ $f_{ct} = \frac{1.48 \ln \left(\frac{(1 + (f'_c + 8))}{10} \right)}{\gamma c} \quad ; \quad f'_c \leq 60 \text{ MPa}$	
Marti et al. (1998) [201]	$\tau_{max} = 0.6 f'_c{}^{2/3} \quad : \quad \varepsilon_s \leq \varepsilon_y$ $\tau_{max} = 0.3 f'_c{}^{2/3} \quad : \quad \varepsilon_s > \varepsilon_y$	Tension chord model
Oh and Kim (2007) [202]	$\tau_{max} = 2.5 f'_c{}^{0.6}$	

3.7.3. Bond Characteristics in Self-compacting Concrete

Table (3.29) shows some existing models of predicting the steel-concrete bond behaviour in conventional concrete. Only 3 empirical models since 2007 are found in the literature to describe the relationship between the bond stress between the steel bars and the surrounding

self-compacting concrete. Bar diameter, compressive strength of concrete, and the embedded bar length are the effective parameters in the models.

Table 3.29: Models to estimate the steel-concrete bond stress in self-compacting concrete

References	Bond model	Notes
Bae (2006) [203]	$\tau_{max} = A \left(\frac{c}{d_b} \right)^B \times (f'_c)^\alpha$	$\alpha = 0.21, 0.58, 0.45$ Plain, deformed and GFRP rebar GFRP: Glass Fibre Reinforced Polymer
	Lightweight SCC	
	deformed	
	GFRP	
	plain	
	A	0.85
	B	0.17
	0.48	0.3
	0.68	0.88
	0.74	0.52
	0.52	
Embedded length < 15 d _b		
Desnerck (2010) [196]	$\tau_{max} = \left(1.762 + 0.514 \frac{c}{d_b} \right) \sqrt{f'_c}$	
Aslani and Nejadi (2012) [197]	$\tau_{max} = \left[0.679 \left(\frac{c}{d_b} \right)^{0.6} + 3.88 \left(\frac{d_b}{l_d} \right) \right] (f'_c)^{0.55}$	Plain rebar
	$\tau_{max} = \left[0.672 \left(\frac{c}{d_b} \right)^{0.6} + 4.8 \left(\frac{d_b}{l_d} \right) \right] (f'_c)^{0.55}$	Deformed rebar

3.8. SUMMARY

The lightweight concrete containing Expanded Poly-Styrene (EPS) beads is the particular type of concrete that used to manufacture the testing samples and slab specimens of this study. General specifications of the EPS beads are given in the technical documentations. Almost all the existing investigations about EPS lightweight concrete since 1976, including the experimental data are presented. The collected data contains the information of curing method, type of fine and coarse aggregates, mineral fillers, chemical admixtures, and fibres in each experiment. The mixture proportions including the size and volume of EPS beads in all experiments, together with the obtained density and compressive strength of the concrete, are presented.

Mechanical properties of EPS lightweight concrete from 154 mixture design in 55 experimental programs are also described. In the presented tables, the volume of EPS beads corresponding to the reported mechanical properties is also given.

The lightweight concrete is classified into structural and non-structural types in many codes of practice. Therefore, the collected data was investigated in the corresponding categories. From

the regression analysis of the experimental data, new relationships are developed between the compressive strength and other mechanical properties of EPS lightweight concrete.

In the absence of reliable testing facility, mechanical properties are extracted from the existing models in the literature. Although there are a wide range of relationships in the codes of practice and empirical equations for predicting the mechanical properties of lightweight and conventional concrete, they may give entirely different predictions for the same mixture of concrete. In addition, Chapter Three presents existing models from the literature to estimate the modulus of elasticity, the modulus of rupture and the splitting tensile strength in conventional concrete, lightweight concrete and self-compacting concrete.

The proposed models are compared and verified by the most frequently used models in the design codes and experimental data. The results show a good agreement between the experimental data and the predictions of the proposed models. Table (3.31) summarizes the developed models for the different mechanical properties of structural EPS-LWC. The models for non-structural EPS-LWC are presented in this chapter also.

Table 3.30: Proposed models for structural EPS lightweight concrete

Parameter	Equation
Modulus of elasticity (GPa, MPa)	$E_c = 4.21(f'_c)^{0.508}$
Modulus of rupture (MPa)	$f_r = 1.266 \cdot (f'_c)^{0.4062}$
Splitting tensile strength (MPa)	$f_{ct} = 0.1862(f'_c)^{0.8655}$

According to the topic of this research program, a broad range of the existing models to predict the mechanical properties of conventional concrete and self-compacting concrete are also presented and evaluated in this chapter.

REFERENCES

1. ASTM-C578, *Standard specification for rigid, cellular polystyrene thermal insulation*. American society for Testing Materials (ASTM), C16.22, 2015. **04.06**.
2. AS-1366.3, *Rigid cellular plastics sheets for thermal insulation PART 3: Rigid cellular polystyrene Moulded (RC/PS—M)*. Australian Standards, 1992.
3. Bagon, C. and S. Frondistou-Yannas, *Marine floating concrete made with polystyrene expanded beads*. Magazine of Concrete Research, 1976. **28**(97): p. 225-229.
4. Bischoff, P., K. Yamura, and S. Perry. *Polystyrene aggregate concrete subjected to hard impact*. in *ICE Proceedings*. 1990. Thomas Telford.
5. Ravindrarajah, R.S. and A. Tuck, *Properties of hardened concrete containing treated expanded polystyrene beads*. Cement and Concrete Composites, 1994. **16**(4): p. 273-277.
6. Park, S.G. and D.H. Chisholm, *Polystyrene Aggregate Concrete*. STUDY REPORT, No. 85 , Building Research Levy, BRANZ, 1999.
7. Babu, K.G. and D.S. Babu, *Behaviour of lightweight expanded polystyrene concrete containing silica fume*. Cement and Concrete Research, 2003. **33**(5): p. 755-762.
8. Babu, K.G. and D.S. Babu, *Performance of fly ash concretes containing lightweight EPS aggregates*. Cement and Concrete Composites, 2004. **26**(6): p. 605-611.
9. Chen, B. and J. Liu, *Properties of lightweight expanded polystyrene concrete reinforced with steel fiber*. Cement and Concrete Research, 2004. **34**(7): p. 1259-1263.
10. Miled, K., et al., *Compressive behavior of an idealized EPS lightweight concrete: size effects and failure mode*. Mechanics of materials, 2004. **36**(11): p. 1031-1046.
11. Le Roy, R., E. Parant, and C. Boulay, *Taking into account the inclusions' size in lightweight concrete compressive strength prediction*. Cement and Concrete Research, 2005. **35**(4): p. 770-775.
12. Babu, D.S., K.G. Babu, and T. Wee, *Properties of lightweight expanded polystyrene aggregate concretes containing fly ash*. Cement and Concrete Research, 2005. **35**(6): p. 1218-1223.
13. Chen, B. and J. Liu, *Mechanical properties of polymer-modified concretes containing expanded polystyrene beads*. Construction and Building Materials, 2005. **xxx**: p. 1-5.
14. Babu, D.S., K.G. Babu, and W. Tiong-Huan, *Effect of polystyrene aggregate size on strength and moisture migration characteristics of lightweight concrete*. Cement and Concrete Composites, 2006. **28**(6): p. 520-527.
15. Haghi, A., M. Arabani, and H. Ahmadi, *Applications of expanded polystyrene (EPS) beads and polyamide-66 in civil engineering, Part One: Lightweight polymeric concrete*. Composite Interfaces, 2006. **13**(4-6): p. 441-450.
16. Bouvard, D., et al., *Characterization and simulation of microstructure and properties of EPS lightweight concrete*. Cement and Concrete Research, 2007. **37**(12): p. 1666-1673.
17. Chen, B. and J. Liu, *Mechanical properties of polymer-modified concretes containing expanded polystyrene beads*. Construction and Building Materials, 2007. **21**(1): p. 7-11.
18. Miled, K., K. Sab, and R. Le Roy, *Particle size effect on EPS lightweight concrete compressive strength: experimental investigation and modelling*. Mechanics of Materials, 2007. **39**(3): p. 222-240.
19. Amianti, M. and V.R. Botaro, *Recycling of EPS: A new methodology for production of concrete impregnated with polystyrene (CIP)*. Cement and Concrete Composites, 2008. **30**(1): p. 23-28.
20. Bisschop, J. and J.G. van Mier, *Effect of aggregates and microcracks on the drying rate of cementitious composites*. Cement and Concrete Research, 2008. **38**(10): p. 1190-1196.
21. Lepech, M.D., et al., *Design of green engineered cementitious composites for improved sustainability*. ACI Materials Journal, 2008. **105**(6): p. 567-575.
22. Chen, B. and C. Fang, *Contribution of fibres to the properties of EPS lightweight concrete*. Magazine of concrete research, 2009. **61**(9): p. 671-678.
23. Bai, E.L., J.Y. Xu, and Z.G. Gao. *Study on Deformation Property of EPS Concrete Under Impact Loading*. in *Applied Mechanics and Materials*. 2011. Trans Tech Publ.
24. Chen, B. and C. Fang, *Mechanical properties of EPS lightweight concrete*. Proceedings of the ICE-Construction Materials, 2011. **164**(4): p. 173-180.
25. Ling, I. and D. Teo, *Properties of EPS RHA lightweight concrete bricks under different curing conditions*. Construction and Building Materials, 2011. **25**(8): p. 3648-3655.

26. Madandoust, R., M.M. Ranjbar, and S.Y. Mousavi, *An investigation on the fresh properties of self-compacted lightweight concrete containing expanded polystyrene*. Construction and Building Materials, 2011. **25**(9): p. 3721-3731.
27. Miled, K., K. Sab, and R. Le Roy, *Effective elastic properties of porous materials: Homogenization schemes vs experimental data*. Mechanics Research Communications, 2011. **38**(2): p. 131-135.
28. Sadrmomtazi, A., et al., *Properties of multi-strength grade EPS concrete containing silica fume and rice husk ash*. Construction and Building Materials, 2012. **35**: p. 211-219.
29. Xu, Y., et al., *Mechanical properties of expanded polystyrene lightweight aggregate concrete and brick*. Construction and Building Materials, 2012. **27**(1): p. 32-38.
30. Carbonari, G., et al., *Flexural behaviour of light-weight sandwich panels composed by concrete and EPS*. Construction and Building Materials, 2012. **35**: p. 792-799.
31. Ferrándiz-Mas, V. and E. García-Alcocel, *Physical and mechanical characterization of Portland cement mortars made with expanded polystyrene particles addition (EPS)*. Materiales de construcción, 2012. **62**(308): p. 547-566.
32. Liu, A.F. and S.R. Guo. *Research on Application of New Energy-Saving Building Materials*. in *Applied Mechanics and Materials*. 2012. Trans Tech Publ.
33. Passa, D.S., et al. *Thermal and drying cyclic loading for cement based mortars and expanded polystyrene foam layers*. in *Applied Mechanics and Materials*. 2012. Trans Tech Publ.
34. Zhao, D.H., J.Y. Xu, and E.L. Bai. *Study on Toughening Effect in EPS Concrete*. in *Applied Mechanics and Materials*. 2012. Trans Tech Publ.
35. Trussoni, M., C.D. Hays, and R.F. Zollo, *Fracture Properties of Concrete Containing Expanded Polystyrene Aggregate Replacement*. ACI Materials Journal, 2013. **110**(5).
36. Wang, H.Z. and D.G. Fu. *Experimental study of porous concrete material thermal insulation material lightweight polystyrene particles*. in *Applied Mechanics and Materials*. 2013. Trans Tech Publ.
37. Chen, B. and N. Liu, *A novel lightweight concrete-fabrication and its thermal and mechanical properties*. Construction and Building Materials, 2013. **44**: p. 691-698.
38. Elsalah, J., et al. *The influence of recycled expanded polystyrene (EPS) on concrete properties: Influence on flexural strength, water absorption and shrinkage*. in *3RD INTERNATIONAL ADVANCES IN APPLIED PHYSICS AND MATERIALS SCIENCE CONGRESS*. 2013. AIP Publishing.
39. Ferrándiz-Mas, V. and E. García-Alcocel, *Durability of expanded polystyrene mortars*. Construction and Building Materials, 2013. **46**: p. 175-182.
40. Herki, A., J. Khatib, and E. Negim, *Lightweight concrete made from waste polystyrene and fly ash*. World Applied Sciences Journal, 2013. **21**(9): p. 1356-1360.
41. Sadrmomtazi, A., J. Sobhani, and M. Mirgozar, *Modeling compressive strength of EPS lightweight concrete using regression, neural network and ANFIS*. Construction and Building Materials, 2013. **42**: p. 205-216.
42. Hongbo, Z., et al., *Impact resistance of a novel expanded polystyrene cement-based material*. Journal of Wuhan University of Technology-Mater. Sci. Ed., 2014. **29**(2): p. 284-290.
43. Hidayat, I. and Gunaedil, *The Influence of Fly Ash in Concrete Compressive Strength by using Expanded Polystyrene as a Partial Substitution of Sand*. International Journal of Applied Engineering Research, 2014. **9**(21): p. 9729-9738.
44. Kekanović, M., et al., *Lightweight concrete with recycled ground expanded polystyrene aggregate*. Tehnički vjesnik, 2014. **21**(2): p. 309-315.
45. Liu, N. and B. Chen, *Experimental study of the influence of EPS particle size on the mechanical properties of EPS lightweight concrete*. Construction and Building Materials, 2014. **68**: p. 227-232.
46. Mousavi, S.A., S.M. Zahrai, and A. Bahrami-Rad, *Quasi-static cyclic tests on super-lightweight EPS concrete shear walls*. Engineering Structures, 2014. **65**: p. 62-75.
47. Schackow, A., et al., *Mechanical and thermal properties of lightweight concretes with vermiculite and EPS using air-entraining agent*. Construction and Building Materials, 2014. **57**: p. 190-197.
48. Tamut, T., et al., *PARTIAL REPLACEMENT OF COARSE AGGREGATES BY EXPANDED POLYSTYRENE BEADS IN CONCRETE*. International Journal of Research in Engineering and Technology, 2014. **3**(2): p. 238-241.
49. Nekooie, M.A., H. Goodall, and M.I. Mohamad, *Selection of sustainable materials for lightweight concrete*. 2015.

50. Xu, Y., et al., *Prediction of compressive strength and elastic modulus of expanded polystyrene lightweight concrete*. 2015.
51. Li, Y., N. Liu, and B. Chen, *Properties of lightweight concrete composed of magnesia phosphate cement and expanded polystyrene aggregates*. Materials and Structures, 2015. **48**(1-2): p. 269-276.
52. Lo Monte, F., P. Bamonte, and P.G. Gambarova, *Physical and mechanical properties of heat-damaged structural concrete containing expanded polystyrene synthesized particles*. Fire and Materials, 2015. **39**(1): p. 58-71.
53. Pecce, M., et al., *Steel-concrete bond behaviour of lightweight concrete with expanded polystyrene (EPS)*. Materials and Structures, 2015. **48**(1-2): p. 139-152.
54. Ranjbar, M.M. and S.Y. Mousavi, *Strength and durability assessment of self-compacted lightweight concrete containing expanded polystyrene*. Materials and Structures, 2015. **48**(4): p. 1001-1011.
55. Wang, C. and D. Arellano. *Are the Mechanical Properties of Recycled-Content Expanded Polystyrene (EPS) Comparable to Nonrecycled EPS Geofoam?* in *Geo-Congress 2014 Technical Papers@sGeo-characterization and Modeling for Sustainability*. 2014. ASCE.
56. Gao, T., et al., *Aerogel-incorporated concrete: An experimental study*. Construction and Building Materials, 2014. **52**: p. 130-136.
57. AASHTO, *LRFD, Bridge Design Specifications*. Fourth Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 2007.
58. Rizkalla, S., *Application of the LRFD bridge design specifications to high-strength structural concrete: Flexure and compression provisions*. Vol. 595. 2007: Transportation Research Board.
59. ACI-318-08, *Building code requirements for structural concrete*. American Concrete Institute, International Organization for Standardization, Farmington Hills MI): American Concrete Institute, 2008.
60. ACI-318-99, *Building code requirements for structural concrete*. ACI Committee, Standard ACI 318R-95. American Concrete Institute, Farmington Hills, Mich, 1999.
61. ACI-209-2R, *Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete*. Reported by ACI Committee 209, 2008.
62. 312-92, A.-. *Building code requirements for reinforced concrete*. ACI manual of concrete practice, part 3, 1994.
63. CSA.A23.3-04, *Design of concrete structures*. Canadian Standards Association (CSA), 2010.
64. NZS-3101, *Concrete Structures Standard, Part 1- The Design of Concrete Structures*. New Zealand Standard, Standards New Zealand, Wellington, New Zealand, 2006.
65. AS-3600-09, *Concrete structures*. Australian Standard, 2009.
66. NS.3473, *Concrete structures-design rules*. Norwegian Council for Standardization, Norges Standardiserings Forbund, 1992.
67. AIJ, *Standard for Structural calculations of steel reinforced concrete structures*. Architectural Institute of Japan, 1985.
68. Yusuf, I. and Y. Jimoh, *The transfer models of compressive to tensile, flexural and elastic properties of palm kernel shell concrete*. International Journal of Engineering, IJE, 2013. **11**(2): p. 195-200.
69. JSCE, *Guidelines for concrete, Standard specifications for concrete structures- Design*. Japan Society of Civil Engineering, JSCE, 2010.
70. RakMK-D3-12, *Rakennusten energiatehokkuus. Määräykset ja ohjeet* Finnish building code , Suomen rakentamismääräyskokoelma, Helsinki, Ympäristöministeriö, 2012.
71. NTE-E.060, *Reglamento nacional de edificaciones*. Norma Técnica de Edificaciones E.060 Concreto Armado, Perú, 2009.
72. BS-8110-1, *Structural use of concrete, Part14: Code of practice for design and construction*. Committee reference B/525/2 Draft for comment 95/105430 DC, 1997.
73. BS-5400-4, *Steel, concrete and composite bridges, Part 4: Code of practice for design of concrete bridges*. Committee reference CSB/59, Draft for comment 88/11190 DC , British Standard, 1999.
74. SP-52-101, *Concrete and reinforced concrete structures without prestressing*. Russian standard, Code of Practice in building construction (SP), 2003.
75. Carrasquillo, R.L., H.H. Nilson, and F.O. Slate, *Properties of high strength concrete subjected to short term loads*. ACI J.Proc, 1981. **78**(3): p. 171-178.
76. Soleymani, H.R., *Structural design properties of concrete for a bridge in Alberta*. Canadian Journal of Civil Engineering, 2006. **33**(2): p. 199-205.

77. Haranki, B., *Strength, modulus of elasticity, creep and shrinkage of concrete used in florida*. 2009, University of Florida.
78. Ahmad, S.H. and S.P. Shah, *Structural properties of high-strength concrete and its implications for precast concrete*. 1985.
79. Jobse, H. and S. Moustafa, *Applications of high strength concrete for highway bridges*. PRECAST/PRESTRESSED CONCRETE INSTITUTE. JOURNAL, 1984. **29**(3).
80. Cook, J.E., *10,000 psi Concrete*. Concrete International, 1989. **11**(10): p. 67-75.
81. Zhang, M.H. and O.E. Gjvovr, *Mechanical properties of high-strength lightweight concrete*. ACI Materials Journal, 1991. **88**(3).
82. Jensen, V. *The Plasticity Ratio of Concrete and It's Effect on the Ultimate Strength of Beamse*. in *ACI Journal Proceedings*. 1943. ACI.
83. Pauw, A. *Static modulus of elasticity of concrete as affected by density*. in *ACI Journal Proceedings*. 1960. ACI.
84. Floyd O. Slate, A.H.N. and S. Martinez. *Mechanical properties of high-strength lightweight concrete*. in *ACI Journal Proceedings*. 1986. ACI.
85. Morales, S.M., *Short-term mechanical properties of high-strength light-weight concrete*. NASA STI/Recon Technical Report N, 1982. **84**: p. 10387.
86. Meyer, K.F. and L.F. Kahn, *Lightweight concrete reduces weight and increases span length of pretensioned concrete bridge girders*. PCI journal, 2002. **47**(1): p. 68-75.
87. Alengaram, U.J., H. Mahmud, and M.Z. Jumaat, *Enhancement and prediction of modulus of elasticity of palm kernel shell concrete*. Materials & Design, 2011. **32**(4): p. 2143-2148.
88. Ahmmad, R., et al., *Ductility performance of lightweight concrete element containing massive palm shell clinker*. Construction and Building Materials, 2014. **63**: p. 234-241.
89. Dinakar, P., *Properties of fly-ash lightweight aggregate concretes*. Proceedings of the ICE-Construction Materials, 2012. **166**(3): p. 133-140.
90. Tassew, S.T. and A.S. Lubell, *Mechanical properties of lightweight ceramic concrete*. Materials and structures, 2012. **45**(4): p. 561-574.
91. Cousins, T.E., C.L. Roberts-Wollmann, and M.C. Brown, *High-performance/high-strength lightweight concrete for bridge girders and decks*. Vol. 733. 2013: Transportation Research Board.
92. Alengaram, U.J., B.A. Al Muhit, and M.Z. bin Jumaat, *Utilization of oil palm kernel shell as lightweight aggregate in concrete—a review*. Construction and Building Materials, 2013. **38**: p. 161-172.
93. 213R-03, A., *Guide for Structural Lightweight-Aggregate Concrete*. American Concrete Institute, ACI Committee 213, 2003.
94. FHWA, *Lightweight Concrete: Mechanical Properties*. Federal Highway Administration (FHWA), Highway Research Center, FHWA Publication No. FHWA-HRT-13-061, 2013.
95. AASHTO, *AASHTO LRFD Bridge Design Specifications*. American Association of State Highway and Transportation Officials 2012. **6th Edition**.
96. Bogas, J.A. and R. Nogueira, *Tensile strength of structural expanded clay lightweight concrete subjected to different curing conditions*. KSCE Journal of Civil Engineering, 2014. **18**(6): p. 1780-1791.
97. Shafigh, P., et al., *Engineering properties of oil palm shell lightweight concrete containing fly ash*. Materials & Design, 2013. **49**: p. 613-621.
98. (EC-2), E., *Design of Concrete Structures. Part 1-General Rules and Rules for Buildings*. German version EN 1992-1-1:2004 + AC: 2010, The Concrete Centre: Blackwater, Camberley, UK, , 2002.
99. Wegian, F., *Strength properties of lightweight concrete made with LECA grading*. Australian Journal of Civil Engineering, 2012. **10**(1): p. 11.
100. Yew, M.K., et al., *Effects of Low Volume Fraction of Polyvinyl Alcohol Fibers on the Mechanical Properties of Oil Palm Shell Lightweight Concrete*. Advances in Materials Science and Engineering, 2014.
101. Kou, S., et al., *Properties of lightweight aggregate concrete prepared with PVC granules derived from scraped PVC pipes*. Waste Management, 2009. **29**(2): p. 621-628.
102. OHBDC, *Ontario Highway Bridge Design Code*. Ministry of Transportation, Downs view, Ontario, Canada, 2nd Edition, 1983.
103. CEB-FIP, *High-strength concrete state of the art report*. London: Thomas Telford, 1990.
104. Beton, C.E.-I.d., *CEB-FIP model code 1990*. London. Thomas Telford, 1993.
105. GBJ-11-89, *National standard of the People's Republic of China, Chinese code for seismic design of buildings*. Chinese Design Code, Beijing: NewWorld Press, 1994.

106. ACI363-R, *State of the art report on high-strength concrete*. Farmington Hills (Michigan): American Concrete Institute,, 1992.
107. EHE, *Spanish code for structural concrete*. Real Decreto 2661/1998, Madrid; [in Spanish], 1998.
108. FHWA, *Material Property Characterization of Ultra-High Performance Concrete*. Federal Highway Administration (FHWA), Highway Research Center, McLean, VA 22101-2296, 2000.
109. TS-500, *Building code requirements for reinforced concrete*. Turkish Standards Institute, Ankara, Turkey, 2000.
110. IS.456, *Plain and Reinforced concrete- code of practice*. Indian Standard, ICS, 91.100.30, New Delhi, July 2000.
111. GDC-2000, *Greek Design Code*. Greek National Code Center, Athens, Greece, 2000.
112. NCHRP, *National Cooperative Highway Research Program*. 2003.
113. NBR.6118, *Design of concrete structures*. Brazilian association of technical standards , Rio de Janeiro; [in Portuguese], 2003.
114. IDC-3274, *Come Modificato Dall'OPCM 3431*. Italian Design Code, Italy: Public and Health Ministry, 2003.
115. Fanourakis, G. and Y. Ballim, *An assessment of the accuracy of nine design models for predicting creep in concrete: technical paper*. Journal of the South African Institution of Civil Engineering= Joernaal van die Suid-Afrikaanse Instituut van Siviele Ingenieurswese, 2006. **48**(4): p. 2-8.
116. Ravindrarajah, R.S. and C. Tam, *Properties of concrete made with crushed concrete as coarse aggregate*. Magazine of Concrete Research, 1985. **37**(130): p. 29-38.
117. Gardner, N. and J. Zhao. *Mechanical properties of concrete for calculation of long term deformations*. in *Proceedings of the Second Canadian Symposium on Cement and Concrete*. 1991. University of British Columbia Press, Vancouver, British Columbia, Canada.
118. Gutierrez, P.A. and M.F. Canovas, *The modulus of elasticity of high performance concrete*. Materials and Structures, 1995. **28**(10): p. 559-568.
119. Rashid, M., M. Mansur, and P. Paramasivam, *Correlations between mechanical properties of high-strength concrete*. Journal of Materials in Civil Engineering, 2002. **14**(3): p. 230-238.
120. Levitchitch, V., et al. *Seismic Performance Capacities of Old Concrete*. in *13 WCEE: 13 th World Conference on Earthquake Engineering Conference Proceedings*. 2004.
121. Leemann, A. and C. Hoffmann, *Properties of self-compacting and conventional concrete– differences and similarities*. Magazine of Concrete Research, 2005. **57**(6): p. 315-319.
122. Kheder, G. and S. Al-Windawi, *Variation in mechanical properties of natural and recycled aggregate concrete as related to the strength of their binding mortar*. Materials and Structures, 2005. **38**(7): p. 701-709.
123. Liu, Y., *Strength, modulus of elasticity, shrinkage and creep of concrete*. 2007: ProQuest.
124. Dinakar, P., K. Babu, and M. Santhanam, *Mechanical properties of high-volume fly ash self-compacting concrete mixtures*. Structural concrete, 2008. **9**(2): p. 109-116.
125. Noguchi, T., et al., *A practical equation for elastic modulus of concrete*. ACI Structural Journal, 2009. **106**(5).
126. Obispo, S.L., *Shrinkage and modulus of elasticity in concrete with recycled aggregates*. Master thesis, California Polytechnic State University,, 2011.
127. Aslani, F. and S. Nejadi, *Mechanical properties of conventional and self-compacting concrete: An analytical study*. Construction and Building Materials, 2012. **36**(0): p. 330-347.
128. EC-2, E., *Design of Concrete Structures, Part 1: General Rules and Rules for buildings*. Commission of European Communities ENV, German version EN 1992-1-1:2004 + AC: 2010, The Concrete Centre: Blackwater, Camberley, UK, . 2004: p. 1-1.
129. ACI-363-R, *State of the art report on high-strength concrete*. armington Hills (Michigan): American Concrete Institute (ACI), 1992.
130. IS.456, *Plain and Reinforced concrete- code of practice*. Indian Standard, ICS, 91.100.30, New Delhi, 2000.
131. ACI-435, *Deflections of reinforced concrete flexural member*. American Concrete Institute (ACI), Concrete International, 1968.
132. Carrasquillo, R.L. and A.H. Nilson. *Properties of high strength concrete subject to short-term loads*. in *ACI Journal Proceedings*. 1981. ACI.
133. Bakhsh, A.H., F.F. Wafa, and A.A. Akhtaruzzaman, *Torsional behavior of plain high-strength concrete beams*. ACI Structural Journal, 1990. **87**(5).
134. Raphael, J.M. *Tensile strength of concrete*. in *ACI Journal Proceedings*. 1984. ACI.
135. Lane, D.S., *Evaluation of concrete characteristics for rigid pavements*. 1998.
136. Akazawa, T., *Tension test method for concrete*. Bulletin, 1953. **16**.

137. Carneiro, F. and A. Barcellos, *Concrete tensile strength*. RILEM Bulletin, 1953. **13**: p. 103-107.
138. Vinayaka, M., *The relation between crushing strength and tensile splitting strength of concrete*. CBJ. 29th Session, Hyderabad, 1959.
139. Sen, B. and P. Desayi, *Determination of tensile strength of concrete by splitting a cube along its diagonal plane*. Indian Concrete Journal, 1962. **36**(7): p. 249-252.
140. Carino, N.J. and H. Lew. *Re-examination of the relation between splitting tensile and compressive strength of normal weight concrete*. in *ACI Journal Proceedings*. 1982. ACI.
141. Gardner, N., *Relationship of the punching shear capacity of reinforced concrete slabs with concrete strength*. ACI Structural Journal, 1990. **87**(1).
142. Oluokun, F., *Prediction of concrete tensile strength from its compressive strength: an evaluation of existing relations for normal weight concrete*. ACI Materials Journal, 1991. **88**(3).
143. Burg, R.G. and B.W. Ost, *Engineering properties of commercially available high-strength concretes*. 1992.
144. Beton, C.E.-I.d., *CEB-FIP model code 1990*. London, Thomas Telford, 1993.
145. NEN-6722, *Regulations concrete. Construction* (VBU 1988), with correction sheet; 2000, 2000.
146. Hueste, M.B.D., et al., *Mechanical properties of high-strength concrete for prestressed members*. ACI Structural Journal, 2004. **101**(4).
147. AASHTO-07, *Specifications, LRFD Bridge Design*. American Association of State Highway and Transportation Officials, Washington DC, 2007. **20001**.
148. NS.3473, *Concrete structures-design rules*. Norwegian Council for Standardization, Norges Standardiserings Forbund., 1992.
149. Persson, B., *A comparison between mechanical properties of self-compacting concrete and the corresponding properties of normal concrete*. Cement and concrete Research, 2001. **31**(2): p. 193-198.
150. Felekoğlu, B., S. Türkel, and B. Baradan, *Effect of water/cement ratio on the fresh and hardened properties of self-compacting concrete*. Building and Environment, 2007. **42**(4): p. 1795-1802.
151. Atahan, H., D. Trejo, and M. Hueste, *Applicability of standard equations for predicting mechanical properties of SCC*. ACI Special Publication, 2007. **247**.
152. Kim, Y.H., *Characterization of self-consolidating concrete for the design of precast, pretensioned bridge superstructure elements*. 2008: TEXAS A&M UNIVERSITY.
153. ACI-318-08. *Building code requirements for structural concrete (ACI 318-08) and commentary*. 2008. American Concrete Institute, International Organization for Standardization.
154. Vilanova, A., J. Fernandez-Gomez, and G.A. Landsberger, *Evaluation of the mechanical properties of self compacting concrete using current estimating models: Estimating the modulus of elasticity, tensile strength, and modulus of rupture of self compacting concrete*. Construction and Building Materials, 2011. **25**(8): p. 3417-3426.
155. Yıldırım, H. and O. Sengul, *Modulus of elasticity of substandard and normal concretes*. Construction and Building Materials, 2011. **25**(4): p. 1645-1652.
156. Das, D. and A. Chatterjee, *A comparison of hardened properties of fly-ash-based self-compacting concrete and normally compacted concrete under different curing conditions*. Magazine of Concrete Research, 2011. **64**(2): p. 129-141.
157. Hu, W., et al., *Constitutive models for regression of various experimental stress-strain relations*. International Journal of Mechanical Sciences, 2015. **101**: p. 1-9.
158. Sukumar, B., K. Nagamani, and R.S. Raghavan, *Evaluation of strength at early ages of self-compacting concrete with high volume fly ash*. Construction and Building Materials, 2008. **22**(7): p. 1394-1401.
159. Topçu, İ.B. and T. Uygunoğlu, *Effect of aggregate type on properties of hardened self-consolidating lightweight concrete (SCLC)*. Construction and Building Materials, 2010. **24**(7): p. 1286-1295.
160. Parra, C., M. Valcuende, and F. Gómez, *Splitting tensile strength and modulus of elasticity of self-compacting concrete*. Construction and Building materials, 2011. **25**(1): p. 201-207.
161. Turatsinze, A. and M. Garros, *On the modulus of elasticity and strain capacity of self-compacting concrete incorporating rubber aggregates*. Resources, conservation and recycling, 2008. **52**(10): p. 1209-1215.
162. Tasnimi, A.A. *Mathematical model for complete stress-strain curve prediction of normal, light-weight and high-strength concretes*. Magazine of Concrete Research, 2004. **56**, 23-34.
163. Almusallam, T. and S. Alsayed, *Stress-strain relationship of normal, hight and lightweight concrete*. Magazine of Concrete Research, 1995. **47**(170): p. 39-44.

164. Carreira, D.C., *KH Stress-Strain Relationship for plain concrete in compression* Journal of the American Concrete Institute, 1985. **82**(6): p. 797-804.
165. Tasnimi, A.A., *Mathematical model for complete stress-strain curve prediction of normal, light-weight and high-strength concretes*. Magazine of Concrete Research, 2004. **56**(1): p. 23-34.
166. Clarke, J.L., *Structural light weight aggregate concrete*. Blackie Academic and professional, an imprint of Chapman and Hall, Wester Cleddens Road, Bishpbriggs, Glasgow G642NZ, 1993.
167. Santiago, E.Q.R., et al., *Mechanical behavior of recycled lightweight concrete using EVA waste and CDW under moderate temperature*. Ibracon structures and materials journal, 2009. **2**(3): p. 211-221.
168. Richard, R.M. and B.J. Abbott, *Versatile elastic-plastic stress-strain formula*. Journal of the Engineering Mechanics Division, 1975. **101**(4): p. 511-515.
169. Wang, P., S. Shah, and A. Naaman. *Stress-strain curves of normal and lightweight concrete in compression*. in *ACI Journal Proceedings*. 1978. ACI.
170. Hussin, M., et al. *A mathematical model for complete stress-strain curve prediction of permeable concrete*. in *Proceedings of the 22nd Australasian Conference on the Mechanics of Structures and Materials (ACMSM 22)*. 2013. Taylor & Francis (CRC Press)/Balkema.
171. Hognestad, E., *Study of combined bending and axial load in reinforced concrete members*. University of Illinois. Engineering Experiment Station. Bulletin; no. 399, 1951.
172. Smith, G. and L. Young. *Ultimate theory in flexure by exponential function*. in *ACI Journal Proceedings*. 1955. ACI.
173. Desayi, P. and S. Krishnan. *Equation for the stress-strain curve of concrete*. in *ACI Journal Proceedings*. 1964. ACI.
174. Saenz, L., *Equation for the stress-strain curve of concrete (Discussion)*. J ACI, 1964. **61**(9): p. 1229-35.
175. Mazars, J. and G. Pijaudier-Cabot, *Continuum damage theory-application to concrete*. Journal of Engineering Mechanics, 1989. **115**(2): p. 345-365.
176. Collins, M.P. and D. Mitchell, *Prestressed concrete structures*. Vol. 9. 1991: Prentice Hall Englewood Cliffs, NJ.
177. Wee, T., M. Chin, and M. Mansur, *Stress-strain relationship of high-strength concrete in compression*. Journal of Materials in Civil Engineering, 1996. **8**(2): p. 70-76.
178. Chandrasekhar, M., M.S. Rao, and M. Janardhana, *Studies on stress-strain behavior of SFRSCC and GFRSCC under axial compression*. International Journal of Earth Sciences and Engineering, ISSN, 2011: p. 0974-5904.
179. Prasad, M., P.R. Kumar, and T. Oshima, *Development of Analytical Stress-Strain Model for Glass Fiber Reinforced Self Compacting Concrete*. International Journal of Mechanics & Solids, 2009. **4**(1).
180. Kumar, R., B. Singh, and P. Bhargava, *Flexural capacity predictions of self-compacting concrete beams using stress-strain relationship in axial compression*. Magazine of Concrete Research, 2011. **63**(1): p. 49-59.
181. Kim, D.-J., et al., *Bond strength of steel deformed rebars embedded in artificial lightweight aggregate concrete*. Journal of Adhesion Science and Technology, 2013. **27**(5-6): p. 490-507.
182. Orangun, C., J. Jirsa, and J. Breen. *A reevaluation of test data on development length and splices*. in *ACI Journal Proceedings*. 1977. ACI.
183. Kemp, E. and W. Wilhelm. *Investigation of the parameters influencing bond cracking*. in *ACI Journal Proceedings*. 1979. ACI.
184. Eligehausen, R., *Übergreifungsstöße zugbeanspruchter Rippenstäbe mit geraden Stabenden*. Deutscher Ausschuss für Stahlbeton, 1979(301).
185. Kemp, E.L. *Bond in reinforced concrete: behavior and design criteria*. in *ACI Journal Proceedings*. 1986. ACI.
186. Robins, P. and S. Austin, *Bond of lightweight aggregate concrete incorporating condensed silica fume*. ACI Special Publication, 1986. **91**.
187. Chapman, R.A. and S.P. Shah, *Early-age bond strength in reinforced concrete*. ACI Materials Journal, 1987. **84**(6).
188. Harajli, M., *Development/splice strength of reinforcing bars embedded in plain and fiber reinforced concrete*. ACI structural Journal, 1994. **91**(5).
189. Huang, Z., B. Engström, and J. Magnusson, *Experimental investigation of the bond and anchorage behaviour of deformed bars in high strength concrete*. Goteborg: Division of Concrete Structures, Chalmers University of Technology, 1996.
190. Esfahani, M.R. and B.V. Rangan, *Bond between normal strength and high-strength concrete (HSC) and reinforcing bars in splices in beams*. ACI Structural Journal, 1998. **95**(3).

191. Pillai, S.U., D.W. Kirk, and M.-A. Erki, *Reinforced concrete design*. 1988: McGraw-Hill Ryerson.
192. Zuo, J. and D. Darwin, *Splice strength of conventional and high relative rib area bars in normal and high-strength concrete*. ACI structural Journal, 2000. **97**(4).
193. Committee, A., *Bond and Development of Straight Reinforcing Bars in Tension (ACI 408R-03)*. American Concrete Institute, Detroit, Michigan, US, 2003.
194. Sancak, E., *Prediction of bond strength of lightweight concretes by using artificial neural networks*. Sci. Res. Essays, 2009. **4**(4): p. 256-266.
195. Tanyildizi, H., *Fuzzy logic model for the prediction of bond strength of high-strength lightweight concrete*. Advances in Engineering Software, 2009. **40**(3): p. 161-169.
196. Desnerck, P., G. De Schutter, and L. Taerwe, *Bond behaviour of reinforcing bars in self-compacting concrete: experimental determination by using beam tests*. Materials and Structures, 2010. **43**(1): p. 53-62.
197. Aslani, F. and S. Nejadi, *Bond behavior of reinforcement in conventional and self-compacting concrete*. Advances in Structural Engineering, 2012. **15**(12): p. 2033-2052.
198. Nilson, S., *Bond Stress-Slip Relationships in Reinforced Concrete*. DepL of Structure Engineering, Cornell Univ., Ithaca, NY, 1971.
199. Wu, Y.-F. and X.-M. Zhao, *Unified bond stress-slip model for reinforced concrete*. Journal of Structural Engineering, 2012. **139**(11): p. 1951-1962.
200. Diab, A.M., et al., *Bond behavior and assessment of design ultimate bond stress of normal and high strength concrete*. Alexandria Engineering Journal, 2014. **53**(2): p. 355-371.
201. Marti, P., et al., *Tension chord model for structural concrete*. Structural Engineering International, 1998. **8**(4): p. 287-298.
202. Oh, B.H. and S.H. Kim, *Realistic models for local bond stress-slip of reinforced concrete under repeated loading*. Journal of Structural Engineering, 2007. **133**(2): p. 216-224.
203. Bae, S. *Mix design, formwork pressure and bond characteristics of special self-consolidating concrete*. in *Masters Abstracts International*. 2006.

CHAPTER 4

TIME-DEPENDENT BEHAVIOUR OF CONCRETE

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Chapter 4: Time-dependent properties of concrete

4.1. INTRODUCTION

Generally, when a concrete structure is subjected to an external loading, its response is studied under instantaneous and time-dependent conditions separately. Regarding the time-dependent behaviour of concrete structure, the following items could be addressed: [1]

- Time-dependent properties of concrete, especially creep and shrinkage, with their dependence on a multiplicity of variables;
- Time-dependency of the concrete properties in regard to the processes of structural analysis;
- The time-dependent stress and strain analysis of reinforced concrete sections;
- Integration of the sectional effects to determine the forces, deformations and deflections of structural members.

There are a considerable number of investigations about the time-dependent behaviour of concrete in the literature since it was first recognized and reported almost a century ago. However, there is no distinctive boundary in Reinforced Concrete (RC) members as to when the instantaneous (short-term) behaviour ends and long-term begins. In addition, there is no end for time-dependent deflection of reinforced concrete structures during the service life that makes the prediction of total deflection inaccurate.

To determine the long-term deflection of a reinforced concrete member, the time-delayed effects of shrinkage, creep and the consequence reduction in tension stiffening need to be accurately considered in the design. Additional factors which can contribute to the increased

long-term deflection including the development of new cracks, widening of the earlier cracks, and effects of the repeated load cycles should be addressed too [2].

Shrinkage induces strain that develops with time and causes additional curvature known as shrinkage warping. Deflections also increase due to the effects of creep in the concrete member. The tension stiffening effect is responsible for a significant proportion of concrete stiffness. It decreases over time due to the effects of creep and shrinkage strains [3].

The time-dependent deflection also increases with increasing cracking of the cross-sections. Cracking is caused when the tensile stress induced by flexure, shrinkage and thermal effects, acting individually or simultaneously, becomes greater than the tensile strength of the concrete. As the concrete drying shrinkage increases with time, cracking can also be expected to increase with time. When a high level of concrete tensile stress is maintained for some time, more and more cracks will occur. Therefore, an RC member that is essentially uncracked at the time of stripping will gradually crack. It causes the effective moment of inertia to decrease, and this will contribute to the long-term deflection [4]. Accordingly, a general equation to predict the long-term deflection of concrete members can be written as Equation (4.1).

$$\Delta_{tot} = \Delta_{ins} + \Delta_{sh} + \Delta_{cr} + \Delta_{tmp} \quad \text{Eq. (4. 1)}$$

Where Δ_{tot} is total deflection, Δ_{ins} is instantaneous deflection, Δ_{sh} is deflection due to shrinkage (autogeneous and chemical shrinkage), Δ_{cr} is deflection due to creep effect and Δ_{tmp} is temperature induced deflection.

The time-dependent effects of creep, shrinkage and temperature variation is the source of the nonlinear behaviour of concrete structures that generally is not taken into account in instantaneous deflection calculations of RC structures [5]

The creep and shrinkage characteristics of concrete are critical in design of any concrete structural member which will be under a sustained load. These characteristics are essential, since deformation of the member with time can be three or more times as great as the elastic deformation occurring upon application of the load [6].

Despite the dependent nature of creep and shrinkage on each other [7], in most design purposes they are assumed to be independent. Therefore, their individual effects are being added to the calculation of the total strain in RC structure. Considering the effective parameters in Equation (4.1), the total strain in the concrete may be considered as the summation of the strains induced by all the affecting parameters.

Generally, shrinkage is assumed to be a time-dependent parameter; however, creep is changing by the applied stress level and time. Most of the investigations show that creep is very sensitive to the curing process, environmental variations, and especially the concrete composition [8].

Generally, varying deformation can be observed in an RC member subjected to a constant sustained loading and constant temperature over time. This behaviour may be illustrated by the effects of creep and shrinkage and the constant stress applied at time t_0 . The shrinkage consists of two parts of autogenous and drying shrinkage in general. After the concrete sets or moist curing has finished, concrete undergoes autogenous shrinkage (before drying) and subsequent drying shrinkage. The shrinkage strain increases continuously at a decreasing rate.

However, creep includes basic creep and drying creep parts in general. Immediately after applying the loading, drying creep and basic creep take place [9].

Figure (4.1) shows the terms relating to creep and shrinkage of concrete. It displays the general trend and comparative magnitude of the creep and shrinkage components under constant sustained load (stress).

In Figure (4.1), in contrast with the drying effects, the swelling portion of the diagram deals with the water absorption and expansion of the concrete.

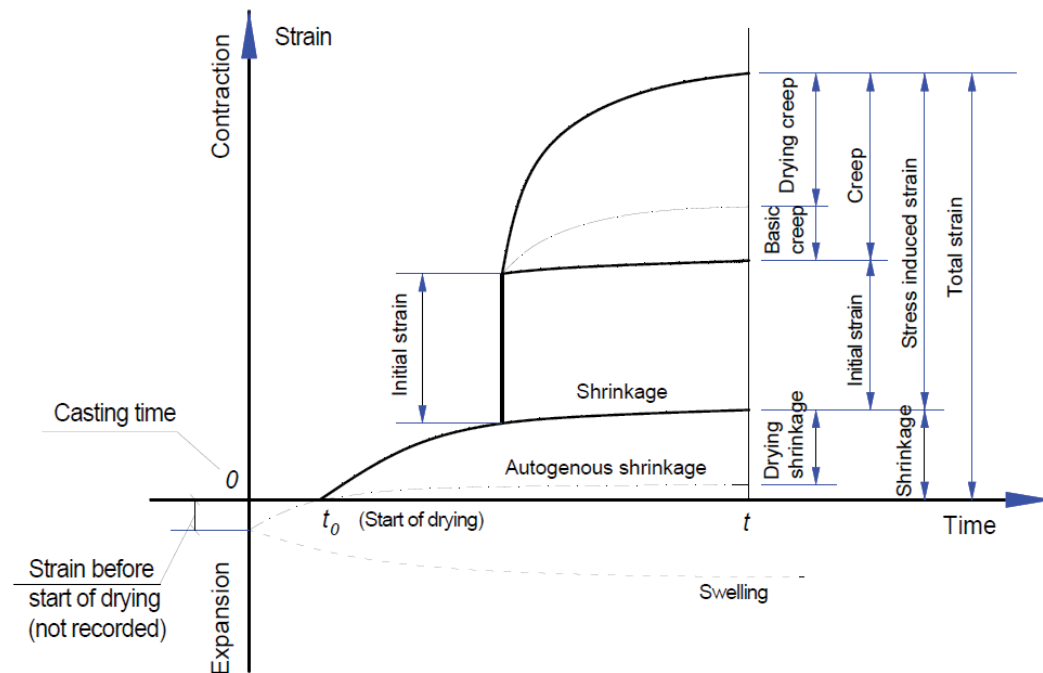


Figure 4.1: General trend of shrinkage and creep strain in concrete under sustained loading

In real conditions, there are more complicated parameters affecting the time-dependent deflection of concrete. For instance, changing of the stress history in the concrete member, applied restraints and unstable environmental conditions e.g. temperature and humidity may affect the deflection of a concrete structure. These uncertainties may strongly influence the predicted values of creep and shrinkage induced strains for the time-dependent deflection calculations.

4.2. CREEP STRAIN

Creep of concrete is the increase in strain under sustained loads. It is composed of two components, the basic creep or deformation under the loads without moisture loss, and the drying creep or deformation under drying conditions only. Creep of concrete is mainly due to the creep of cement paste. For Portland cement, creep is primarily determined by the creep of Calcium Silicate Hydrates (CSH) [10]. The time-dependent deformation due to creep is attributed to the movement of water between different phases of concrete. When an external load is applied, it changes the attraction forces between the cement gel particles. This modification of forces causes an imbalance in the attractive and disjoining forces. However, the imbalance is gradually eliminated by moisture transfer into the pores in cases of compression, and away from the pores in instances of tension.

Creep influences both the strain (deflection) and stress distribution in the reinforced concrete structures. Figure (4.2) shows the total load induced strain per unit stress caused by the unit of uniaxial sustained load (also called as compliance function, $J(t, t_0)$) applied at age t since loading age t_0 .

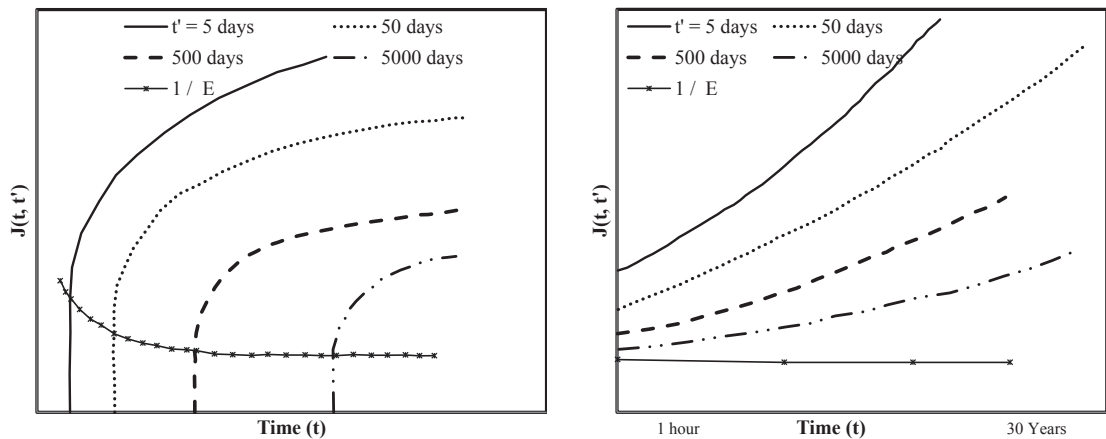


Figure 4.2: Compliance curves for various ages (t') at loading [11]

The gradual increase of creep strain in the compression zone of an RC member cross-section increases the curvature and a consequent increase in deflection of the member. For a plain concrete member without reinforcing bars, the strain development at every point on the section is proportional to the creep coefficient, $\varphi(t)$. Hence, it is proportional to the increase in curvature as well.

Considering the cracked or uncracked RC section, the interaction between the reinforced concrete section curvature and development of creep strain is graphically illustrated in Figure (4.3). The creep strain influences the total strain distribution on the section and consequently the curvature of the RC member.

Figure (4.3.a) shows the uncracked singly reinforced concrete section. The reinforcement in tensile zone applies some restraint to the creep. Depending on the number and area of the tensile steel bars, the increase in curvature due to creep is proportional to a large fraction of the creep coefficient. Usually, this creep induced curvature is between $0.7 \varphi(t)$ and $0.95 \varphi(t)$ [12].

Figure (4.3.b) shows the fully cracked singly reinforced concrete section. Comparing with Figure (4.3a), the initial curvature is considerably larger. Assuming that the cracked tensile concrete below the neutral axis carries no stress; no creep will happen in that tensile concrete zone. On the compression side, the creep reduces the neutral axis depth due to extension of the crack, and consequently causes a dropping of the compressive stress level. The lower compressive stress level reduces the creep strain, so the increase in curvature is proportional to a small fraction of the creep coefficient. In a fully cracked RC section, the creep induced curvature is usually less than $0.25 \varphi(t)$. Although the total deflection of the cracked flexural member is significantly high, the creep effect in total deflection in an uncracked RC section is relatively higher e.g. $0.25 \varphi(t) < 0.7 \varphi(t)$ [12].

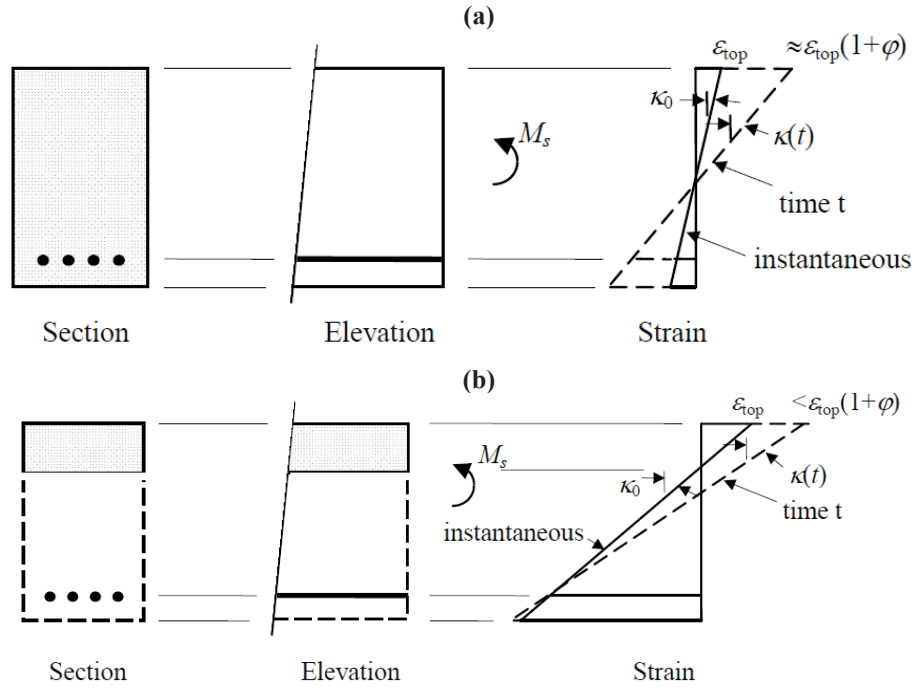


Figure 4.3: Creep effects on the strain distribution of a singly reinforced cross-section [12]

4.3. CREEP COEFFICIENT

The ratio of creep strain to the initial strain or identically, the ratio of creep compliance to the compliance obtained at early ages is called the creep coefficient [13]. It typically shows the capacity of concrete to creep and is described by the general form of Equation (4.2).

Figure (4.4) exhibits the curves of stress versus creep strains for various constant load durations. In the range of stress up to 40% - or 50% of the compressive strength (the typical service stress range), the curves in Figure (4.4) are approximately linear. Therefore, creep coefficient can be written as follows [11]:

$$\phi(t, t_0) = \frac{\epsilon_c(t - t_0)}{\epsilon_e(t_0)} \quad \text{Eq. (4. 2)}$$

Where; $\epsilon_c(t - t_0)$ is the creep strain in duration of $(t - t_0)$ and $\epsilon_e(t_0)$ is the initial elastic strain.

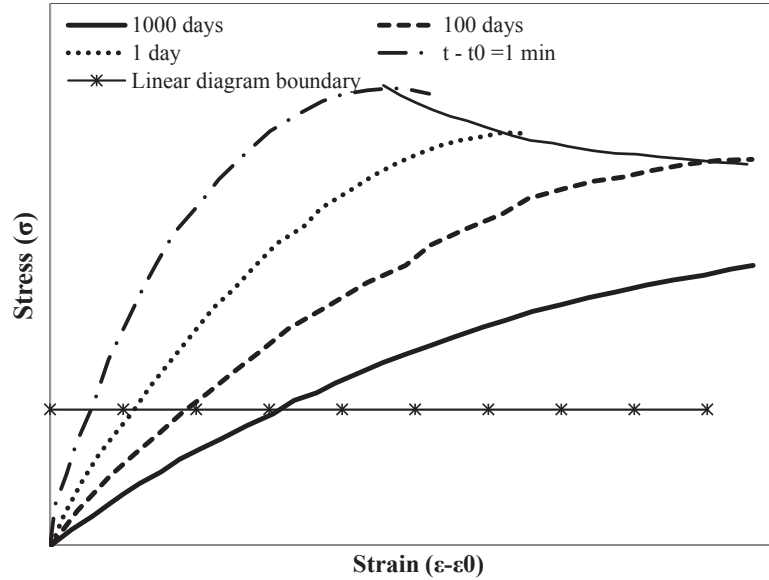


Figure 4.4: Creep isochrones in concrete [11]

4.4. SHRINKAGE STRAIN

As the concrete shrinks, it compresses the steel reinforcement. The steel, in turn, imposes an equal and opposite tensile force, ΔT on the concrete. This progressively increasing tensile force, acting at some eccentricity to centroid of the concrete cross-section, produces elastic plus creep strains and a resulting curvature of the section. The shrinkage-induced curvature (κ_{sh}) often leads to significant load-independent deflection of the member. Magnitude of ΔT and hence the shrinkage-induced curvature depends on the quantity and position of the reinforcement and the size of the compressive concrete in the cross section, and hence on the extent of cracking. The extent of cracking depends, of course, on the magnitude of the applied moment. Although shrinkage strain is independent of stress, the shrinkage-induced curvature is not independent of the external load. The effect of shrinkage-induced curvature on a previously cracked cross section (κ_{sh}/cr) is considerably greater than an uncracked cross-section ($\kappa_{sh}/uncr$) as shown in Figure (4.5) [12].

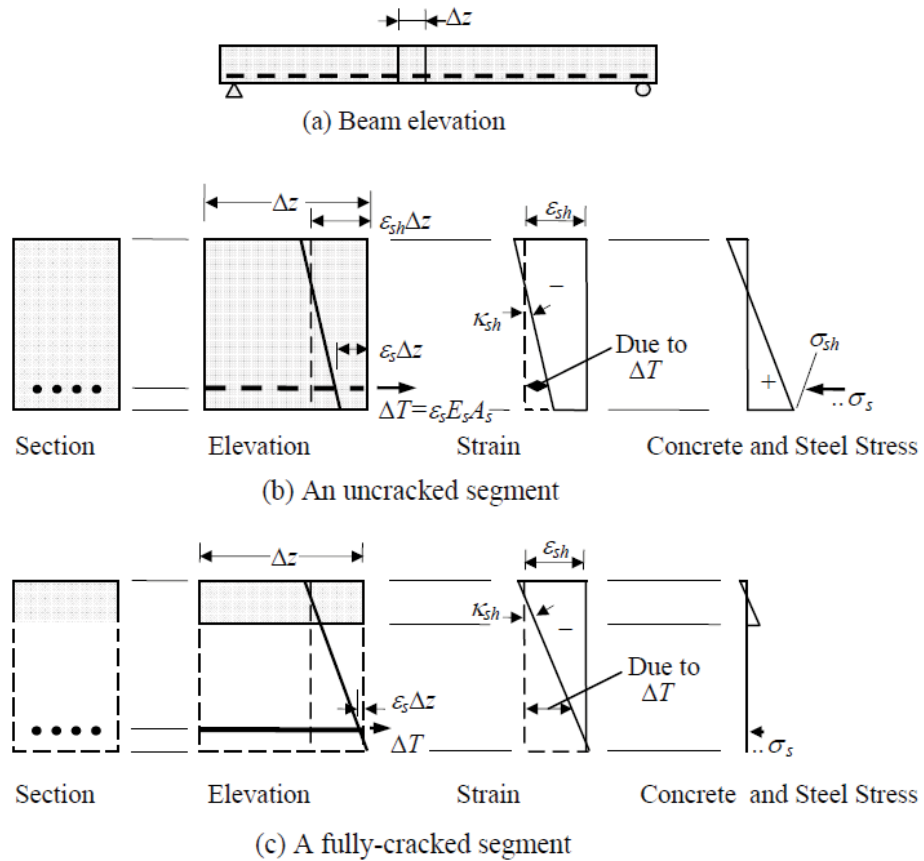


Figure 4.5: Shrinkage-induced deformation and stresses in a singly reinforced beam [12]

4.5. EXISTING CREEP AND SHRINKAGE MODELS FOR LIGHTWEIGHT, SELF-COMPACTING AND CONVENTIONAL CONCRETE

4.5.1. Creep and Shrinkage Models for Lightweight Concrete

4.5.1.1. Creep models for lightweight concrete

In the absence of testing to determine the creep strain and creep coefficient, there are several models in the literature to use for design purposes. These models are developed by the researchers and international codes, based on extensive experimental results. However, these empirical models vary according to the application techniques, properties of the materials and loading age as input data to ascertain the creep strain changes by time and related creep coefficient.

Table (4.1) shows the existing models in codes of practice and some empirical models to calculate the creep strain and coefficient in LWC. Since these models, especially the empirical models have been developed for specific materials and testing setup, they may vary in accuracy and complexity in calculations.

Table 4.1: Existing creep models of lightweight concrete

Reference	Creep model	Notes																																				
Best and Polivka (1959) [14]	$\varepsilon_c = 231t^{1/4.1}$	ε_c : creep strain, t the time under load																																				
Ben Sabaa and Ravindrarajah (1997) [15]	$\phi(t, t_0) = \frac{(t - t_0)}{A + B(t - t_0)}$ <table><tr><th>Mix</th><th>A (x10-2)</th><th>B (x10-2)</th><th>A/B</th></tr><tr><td>A-0</td><td>2.9086</td><td>1.0346</td><td>28.1</td></tr><tr><td>A-30</td><td>4.2501</td><td>1.1101</td><td>38.3</td></tr><tr><td>A-50</td><td>4.3306</td><td>0.9874</td><td>43.9</td></tr><tr><td>A-70</td><td>5.212</td><td>1.2102</td><td>43.1</td></tr><tr><td>B-0</td><td>5.9703</td><td>1.1395</td><td>52.4</td></tr><tr><td>B-30</td><td>7.7086</td><td>1.2608</td><td>61.1</td></tr><tr><td>B-50</td><td>6.0769</td><td>1.3866</td><td>43.8</td></tr><tr><td>B-70</td><td>8.9621</td><td>1.2528</td><td>71.5</td></tr></table>	Mix	A (x10-2)	B (x10-2)	A/B	A-0	2.9086	1.0346	28.1	A-30	4.2501	1.1101	38.3	A-50	4.3306	0.9874	43.9	A-70	5.212	1.2102	43.1	B-0	5.9703	1.1395	52.4	B-30	7.7086	1.2608	61.1	B-50	6.0769	1.3866	43.8	B-70	8.9621	1.2528	71.5	Ross hyperbolic expression A/B = 50 A=5.14E-2 , The polystyrene aggregate concrete mixtures were produced by replacing 30, 50 and 70% of the solid volume of the coarse aggregate with the polystyrene aggregate.
Mix	A (x10-2)	B (x10-2)	A/B																																			
A-0	2.9086	1.0346	28.1																																			
A-30	4.2501	1.1101	38.3																																			
A-50	4.3306	0.9874	43.9																																			
A-70	5.212	1.2102	43.1																																			
B-0	5.9703	1.1395	52.4																																			
B-30	7.7086	1.2608	61.1																																			
B-50	6.0769	1.3866	43.8																																			
B-70	8.9621	1.2528	71.5																																			
ACI-209R-92 (1997) [16] (LWC and CC)	$\phi(t, t_0) = \frac{(t - t_0)}{f + (t - t_0)} \phi_u \gamma_{RH} \gamma_{VS} \gamma_{t_0}$ $f = 26e^{\{1.42 \times 10^{-2} (\frac{V}{S})\}}$ $\gamma_{RH} = 1.27 - 0.67RH \text{ for } h \geq 0.4$ $\gamma_{VS} = (1 + 1.13e^{(-0.0213\frac{V}{S})})$ $\gamma_{t_0} = 1.25t_0^{-0.118}$	(V/S) = 37.5 for a cylinder size of 150 by 300 mm ϕ_u = 2.35 ultimate creep coefficient RH: relative humidity																																				
CEB MC-99 (1999) [17]	$\phi(t, t_0) = \phi_0 \beta_c(t, t_0)$ $\phi_0 = \phi_{RH,T} \beta(f_{cm-28}) \beta(t_0)$ $\beta_0(t - t_0) = \left[\frac{t - t_0}{\beta_{H,T} + (t - t_0)} \right]^{0.3}$ $\phi_{RH,T} = \phi_T + [\phi_{RH} - 1] \phi_T^{1.2}$ $\phi_T = e^{[0.015(T-20)]}$ $\beta_{H,T} = \beta_H \beta_T$ $\phi_{RH} = [1 + \frac{1 - RH}{\sqrt[3]{0.1 \left[\frac{V}{S} \right]}}] \alpha_1 \alpha_2$ $\beta(f_{cm-28}) = \frac{5.3}{\sqrt{f_{cm-28}/10}}$ $\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}}$ $\beta_H = 150[1 + (1.2h)^{18}] \left(\frac{V}{S} \right) + 250 \alpha_3 \leq 1500 \alpha_3$ $\beta_T = e^{\left[\frac{1500}{273+T} - 5.12 \right]}$ $\alpha_1 = \left[\frac{35}{f_{cm-28}} \right]^{0.7}, \alpha_2 = \left[\frac{35}{f_{cm-28}} \right]^{0.2}, \alpha_3 = \left[\frac{35}{f_{cm-28}} \right]^{0.5}$	T: temperature f_{cm-28} : mean compressive strength ϕ_0 : notional creep coefficient																																				
CEB-MC90 (1993)[18]	Similar to CEB MC-99 model except: $\alpha_1 = \alpha_2 = \alpha_3 = 1$																																					

Table 4.1: Existing creep models of lightweight concrete (continued)

Reference	Creep model	Notes
EC-2 (2002) [19]	$\phi(t, t_0) = \phi_0 \beta_c(t, t_0)$ $\phi_0 = \phi_{RH} \beta(f_{cm-28}) \beta(t_0)$ $\beta_0(t - t_0) = \left[\frac{t - t_0}{\beta_H + (t - t_0)} \right]^{0.3}$ $\phi_{RH} = \left[1 + \frac{1 - h}{\sqrt[3]{0.1 \left[\frac{V}{S} \right]}} \alpha_1 \right] \alpha_2$ $\beta(f_{cm-28}) = \frac{5.3}{\sqrt{f_{cm-28}/10}}$ $\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}}$ $\beta_H = 150 \left[1 + (1.2h)^{18} \right] \left(\frac{V}{S} \right) + 250 \alpha_3 \leq 1500 \alpha_3$ $\alpha_1 = \left[\frac{35}{f_{cm-28}} \right]^{0.7}, \alpha_2 = \left[\frac{35}{f_{cm-28}} \right]^{0.2}, \alpha_3 = \left[\frac{35}{f_{cm-28}} \right]^{0.5}$	t_c is age of concrete where moist-curing ends in days
GL2000 (Gardner) model (2004) [20]	$\phi(t, t_0)$ $= \phi(t_c) \left[\frac{2(t - t_0)^{0.3}}{(t - t_0)^{0.3} + 14} + \left(\frac{7}{t_0} \right)^{0.5} \left(\frac{(t - t_0)}{(t - t_0) + 7} \right)^{0.5} + 2.5(1 - 1.086h^2) \left(\frac{(t - t_0)}{(t - t_0) + 77(V/S)^2} \right)^{0.5} \right]$ $\phi(t_c) = \left[1 - \left(\frac{(t - t_0)}{(t - t_0) + 0.12(V/S)^2} \right)^{0.5} \right]^{0.5}$	First two terms calculates the basic creep and the third term is for drying creep
FIB-2010 (2013) [21]	$\phi(t, t_0) = \phi_{bc}(t, t_0) + \phi_{dc}(t, t_0)$ $\phi_{bc}(t, t_0) = \beta_{bc}(f_{cm}) \cdot \beta_{bc}(t, t_0)$ $\beta_{bc}(f_{cm}) = \frac{1.8}{(f_{cm})^{0.7}}$ $\beta_{bc}(t, t_0) = \ln \left(\left(\frac{30}{t_{0,adj}} + 0.035 \right)^2 \cdot (t - t_0) + 1 \right)$ $\phi_{dc}(t, t_0) = \beta_{dc}(f_{cm}) \cdot \beta(R.H) \beta_{dc}(t_0) \cdot \beta_{dc}(t, t_0)$ $\beta_{dc}(f_{cm}) = \frac{412}{(f_{cm})^{1.4}}$ $\beta_{R.H} = \frac{1 - RH/100}{\sqrt[3]{0.1h/100}}$ $\beta_{dc}(t_0) = \frac{1}{0.1 + t_{0,adj}^2}$ $\beta_{dc}(t, t_0) = \left[\frac{t - t_0}{\beta_h + (t - t_0)} \right]^{\gamma_{t_0}}$ $\gamma_{t_0} = \frac{1}{2.3 + \frac{3.5}{\sqrt{t_{0,adj}}}}$ $\beta_h = 1.5h + 250 \alpha_{c,v}$	

4.5.1.2. Shrinkage models of lightweight concrete

Table (4.2) shows the existing models in codes of practice and some empirical models to predict the shrinkage strain for LWC. Since these models, especially the empirical models have been developed for specific materials and testing setup, they may vary in accuracy and complexity in calculations.

Table 4.2: Existing models to predict shrinkage strain in lightweight concrete

Reference	Shrinkage model	Notes
Best and Polivka (1959) [14]	$\varepsilon_{sh} = 231t^{1/4.1}$	
AS-3600 (2009) [22]	$\varepsilon_{sh} = \varepsilon_{cse} + \varepsilon_{csd}$ $\varepsilon_{cse} = \varepsilon_{cse}^* \times (1.0 - e^{-0.1t})$ $\varepsilon_{cse}^* = (0.06f'_c - 1.0) \times 50 \times 10^{-6}$ $\varepsilon_{csd} = k_1 k_4 \varepsilon_{csd.b}$ $\varepsilon_{csd.b} = (1.0 - 0.008f'_c) \times \varepsilon_{csd.b}^*$ $\varepsilon_{csd.b}^* = (800, 900, 1000) \times 10^{-6}$ For Sydney, Brisbane and Melbourne respectively	
ACI-209R-92 (2008) [13](LWC and CC)	$\varepsilon_{sh} = \frac{(t - t_0)^\alpha}{f + (t - t_0)^\alpha} \varepsilon_{sh.u}$ $\varepsilon_{sh.u} = 780 \times 10^{-6} \gamma_{sh}$ $f = 26.0e^{\{1.42 \times 10^{-2}(V/S)\}}$ $\gamma_{sh} = \gamma_{sh,tc} \gamma_{sh,RH} \gamma_{sh,VS} \gamma_{sh,d} \gamma_{sh,\Psi} \gamma_{sh,c} \gamma_{sh,a}$ $\gamma_{sh,tc} = 1.202 - 0.2337 \log(t_0)$ $\gamma_{sh,RH} = 1.4 - 1.02h \text{ for } 0.40 \leq h \leq 0.8$ $\gamma_{sh,RH} = 3.0 - 3.0h \text{ for } 0.80 \leq h \leq 1$ $\gamma_{sh,VS} = 1.2e^{\{-0.00472(V/S)\}}$ $\gamma_{sh,\Psi} = 0.3 + 0.014\Psi$ $\gamma_{sh,c} = 0.75 + 0.00061c \geq 1$ $\gamma_{sh,a} = 0.95 + 0.008a \geq 1$ $\gamma_{sh,d} = 1.23 - 0.0015d \text{ for } t - t_0 \leq 1 \text{ year}$ $\gamma_{sh,d} = 1.17 - 0.00114d \text{ for } t - t_0 > 1 \text{ year}$	

4.5.2. Creep and Shrinkage Models of Conventional Concrete

4.5.2.1. Creep models for conventional concrete

Table (4.3) shows the existing models in codes of practice and some empirical models to estimate the creep coefficient in conventional concrete. Since these models especially the empirical models, have been developed for specific materials and testing systems, they may vary in accuracy and complexity in calculations.

Table 4.3: Existing creep models of conventional concrete

Reference	Creep model	Notes
ACI 209R-92 (1997) [16] (LWC and CC)	$\phi(t, t_0) = \frac{(t - t_0)}{f + (t - t_0)} \phi_u \gamma_{RH} \gamma_{VS} \gamma_{t_0}$ $f = 26e^{\{1.42 \times 10^{-2} (\frac{V}{S})\}}$ $\gamma_{RH} = 1.27 - 0.67h \text{ for } h \geq 0.4$ $\gamma_{VS} = (1 + 1.13e^{(-0.0213 \frac{V}{S})})$ $\gamma_{t_0} = 1.25t_0^{-0.118}$	(V/S) = 37.5 for a cylinder size of 150 by 300 mm $\phi_u = 2.35$ ultimate creep coefficient h: relative humidity
CEB MC-99 (1999) [17]	$\phi(t, t_0) = \phi_0 \beta_c(t, t_0)$ $\phi_0 = \phi_{RH,T} \beta(f_{cm-28}) \beta(t_0)$ $\beta_0(t - t_0) = \left[\frac{t - t_0}{\beta_{H,T} + (t - t_0)} \right]^{0.3}$ $\phi_{RH,T} = \phi_T + [\phi_{RH} - 1] \phi_T^{1.2}$ $\phi_T = e^{[0.015(T-20)]}$ $\beta_{H,T} = \beta_H \beta_T$ $\phi_{RH} = [1 + \frac{1-h}{\sqrt[3]{0.1 \left[\frac{V}{S} \right]}} \alpha_1] \alpha_2$ $\beta(f_{cm-28}) = \frac{5.3}{\sqrt{f_{cm-28}/10}}$ $\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}}$ $\beta_H = 150[1 + (1.2h)^{18}] \left(\frac{V}{S} \right) + 250\alpha_3 \leq 1500\alpha_3$ $\beta_T = e^{\left[\frac{1500}{273+T} - 5.12 \right]}$ $\alpha_1 = \left[\frac{35}{f_{cm-28}} \right]^{0.7}, \alpha_2 = \left[\frac{35}{f_{cm-28}} \right]^{0.2}, \alpha_3 = \left[\frac{35}{f_{cm-28}} \right]^{0.5}$	T: temperature f_{cm-28}: mean compressive strength ϕ_0: notional creep coefficient
CEB MC90 (1993)[18]	Similar to CEB MC-99 model except: $\alpha_1 = \alpha_2 = \alpha_3 = 1$	
EC2 (2002) [19]	$\phi(t, t_0) = \phi_0 \beta_c(t, t_0)$ $\phi_0 = \phi_{RH} \beta(f_{cm-28}) \beta(t_0)$ $\beta_0(t - t_0) = \left[\frac{t - t_0}{\beta_H + (t - t_0)} \right]^{0.3}$ $\phi_{RH} = [1 + \frac{1-h}{\sqrt[3]{0.1 \left[\frac{V}{S} \right]}} \alpha_1] \alpha_2$ $\beta(f_{cm-28}) = \frac{5.3}{\sqrt{f_{cm-28}/10}}$ $\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}}$ $\beta_H = 150[1 + (1.2h)^{18}] \left(\frac{V}{S} \right) + 250\alpha_3 \leq 1500\alpha_3$ $\alpha_1 = \left[\frac{35}{f_{cm-28}} \right]^{0.7}, \alpha_2 = \left[\frac{35}{f_{cm-28}} \right]^{0.2}, \alpha_3 = \left[\frac{35}{f_{cm-28}} \right]^{0.5}$	t_c is age of concrete where moist-curing ends in days

Table 4.3: Existing creep models for conventional concrete (continued)

Reference	Creep model in CC	Notes
GL2000 (Gardner) model (2004) [20]	$\phi(t, t_0)$ $= \phi(t_c) \left[\frac{2(t - t_0)^{0.3}}{(t - t_0)^{0.3} + 14} + \left(\frac{7}{t_0}\right)^{0.5} \left(\frac{(t - t_0)}{(t - t_0) + 7}\right)^{0.5} + 2.5(1 - 1.086h^2) \left(\frac{(t - t_0)}{(t - t_0) + 77(V/S)^2}\right)^{0.5} \right]$ $\phi(t_c) = \left[1 - \left(\frac{(t - t_0)}{(t - t_0) + 0.12(V/S)^2}\right)^{0.5} \right]^{0.5}$	First two terms calculates the basic creep and the third term is for drying creep
FIB-2010 (2013) [21]	$\phi(t, t_0) = \phi_{bc}(t, t_0) + \phi_{dc}(t, t_0)$ $\phi_{bc}(t, t_0) = \beta_{bc}(f_{cm}) \cdot \beta_{bc}(t, t_0)$ $\beta_{bc}(f_{cm}) = \frac{1.8}{(f_{cm})^{0.7}}$ $\beta_{bc}(t, t_0) = \ln \left(\left(\frac{30}{t_{0,adj}} + 0.035 \right)^2 \cdot (t - t_0) + 1 \right)$ $\phi_{dc}(t, t_0) = \beta_{dc}(f_{cm}) \cdot \beta(R.H) \beta_{dc}(t_0) \cdot \beta_{dc}(t, t_0)$ $\beta_{dc}(f_{cm}) = \frac{412}{(f_{cm})^{1.4}}$ $\beta_{R.H} = \frac{1 - RH/100}{\sqrt[3]{0.1h/100}}$ $\beta_{dc}(t_0) = \frac{1}{0.1 + t_{0,adj}^2}$ $\beta_{dc}(t, t_0) = \left[\frac{t - t_0}{\beta_h + (t - t_0)} \right]^{\gamma_{t_0}}$ $\gamma_{t_0} = \frac{1}{2.3 + \frac{3.5}{\sqrt{t_{0,adj}}}}$ $\beta_h = 1.5h + 250\alpha_{c,v}$	
ACI 435-R95 (2003) [23]	$\phi(t, t_0) = \frac{t^{0.6}}{10 + t^{0.6}} \phi_u$ $\phi_u = 2.35 \gamma_{CR}$ $\gamma_{CR} = K_h^c K_d^c K_s^c K_f^c K_{ac}^c K_{t_0}^c = 1$ For standard conditions	C_u : the ultimate creep coefficient K_h^c : relative humidity factor K_d^c : minimum member thickness factor K_s^c : concrete consistency factor K_f^c : fine aggregate content factor K_{ac}^c : air content factor $K_{t_0}^c$: age of concrete at load applications factor
AS-3600 (2900) [22]	$\phi(t, t_0) = k_2 k_3 k_4 k_5 \phi_{c,b}$ $k_2 = \frac{\alpha_2 t^{0.8}}{t^{0.8} + 0.15 t_h}$ $\alpha_2 = 1.0 + 1.12 e^{-0.008 t_h}$ $k_3 = \frac{2.7}{[1 + \log(\tau)]} \text{ for } \tau \geq 1 \text{ day}$ $k_4 = 0.7, 0.65, 0.6, 0.5$ For arid, interior, temperate inland and tropical envir. $k_5 = 1 : f'_c \leq 50 \text{ MPa}$ $k_5 = (2.0 - \alpha_3) - 0.02(1.0 - \alpha_3) f'_c$ $\alpha_3 = 0.7 / (k_4 \alpha_2)$ $\phi_{c,b} = 50.036 f'_c{}^{-0.771} : 20 \leq f'_c \leq 100 \text{ MPa}$	

4.5.2.2. Shrinkage models for conventional concrete

Table (4.4) shows the existing models in codes of practice and some empirical models to calculate the shrinkage strain in CC. Since these models especially the empirical models have been developed for specific materials and testing setup, they may vary in accuracy and complexity in calculations.

Table 4.4: Existing shrinkage models for conventional concrete

Shrinkage	Shrinkage models in CC	Notes
ACI-435-R95 (2003) [23]	$\epsilon_{sh} = \left(\frac{t}{35 + t} \right) (\epsilon_{sh})_u$ $\epsilon_{sh} = \left(\frac{t}{55 + t} \right) (\epsilon_{sh})_u$ $(\epsilon_{sh})_u \max = 780 \times 10^{-6} \gamma_{sh}$ $\gamma_{sh} = K_h^s K_d^s K_s^s K_f^s K_{ac}^s K_b^s = 1$ For standard conditions	Free shrinkage- after 7 days moist curing Free shrinkage- after 1-3 days moist curing K_b^s: coefficient for cement content All other K are the same as defined for creep
ACI-209R-92 (2008) [13] (LWC and CC)	$\epsilon_{sh}(t, t_0) = \frac{(t - t_0)^\alpha}{f + (t - t_0)^\alpha} \epsilon_{sh,u}$ $\epsilon_{sh,u} = 780 \times 10^{-6} \gamma_{sh}$ $f = 26.0 e^{\{1.42 \times 10^{-2} (V/S)\}}$ $\gamma_{sh} = \gamma_{sh,tc} \gamma_{sh,RH} \gamma_{sh,VS} \gamma_{sh,d} \gamma_{sh,\Psi} \gamma_{sh,c} \gamma_{sh,a}$ $\gamma_{sh,tc} = 1.202 - 0.2337 \log(t_0)$ $\gamma_{sh,RH} = 1.4 - 1.02h \text{ for } 0.40 \leq h \leq 0.8$ $\gamma_{sh,RH} = 3.0 - 3.0h \text{ for } 0.80 \leq h \leq 1$ $\gamma_{sh,VS} = 1.2 e^{\{-0.00472 (V/S)\}}$ $\gamma_{sh,\Psi} = 0.3 + 0.014 \Psi$ $\gamma_{sh,c} = 0.75 + 0.00061c \geq 1$ $\gamma_{sh,a} = 0.95 + 0.008a \geq 1$ $\gamma_{sh,d} = 1.23 - 0.0015d \text{ for } t - t_0 \leq 1 \text{ year}$ $\gamma_{sh,d} = 1.17 - 0.00114d \text{ for } t - t_0 > 1 \text{ year}$	
AS-3600 (2900) [22]	$\epsilon_{sh} = \epsilon_{cse} + \epsilon_{csd}$ $\epsilon_{cse} = \epsilon_{cse}^* \times (1.0 - e^{-0.1t})$ $\epsilon_{cse}^* = (0.06 f_c' - 1.0) \times 50 \times 10^{-6}$ $\epsilon_{csd} = k_1 k_4 \epsilon_{csd,b}$ $\epsilon_{csd,b} = (1.0 - 0.008 f_c') \times \epsilon_{csd,b}^*$ $\epsilon_{csd,b}^* = (800, 900, 1000) \times 10^{-6}$ For Sydney, Brisbane and Melbourne $k_1 = \frac{\alpha_1 t^{0.8}}{t^{0.8} + 0.15 t_h}$ $\alpha_1 = 0.8 + 1.2 e^{-0.005 t_h}$ $k_4 = 0.7, 0.65, 0.6$ For arid, interior and temperate inland environment	

4.5.3. Creep and Shrinkage Models in Self-compacting Concrete

4.5.3.1. Creep models of self-compacting concrete

Table (4.5) shows the existing models in codes of practice and some empirical models to estimate the creep coefficient in self-compacting (self-consolidating) concrete. Since these models, especially the empirical models have been developed for specific materials and testing systems, they may vary in accuracy and complexity in calculations.

Table 4.5: Existing creep models for self-compacting concrete

Reference	Creep model	Notes
Poppe and De Schutter (2005)[24]	$\varepsilon_{cr}(t, t_0) = \frac{\sigma_c(t_0)}{E_{ci}} \times \left[1 + \frac{(1 - RH/RH_0)}{0.46(h/h_0)^{1/3}} \right] \times \frac{5.3}{\sqrt{(f_{cm}/f_{cm0})}} \times \frac{1}{(0.1 + (t_0/t_1)^{0.2})}$ $\times \left[\frac{(t - t_0)/t_1}{\left(150 \left(1 + \left(1.2^{RH/RH_0} \right)^{18} \right) h/h_0 + 250 \right) + (t - t_0)/t_1} \right]^{0.3} \left[\frac{1}{0.01 + 1.37(c/p)} \right]$	The model and related coefficients are based on CEB-FIP model
Larson (2006) [25]	$\phi(t, t_0) = \frac{t^{0.7}}{16 + t^{0.7}} \phi_u \quad : \quad \phi_u = 1.75$ $\phi(t, t_0) = \frac{t^{0.6}}{24 + t^{0.6}} \phi_u \quad : \quad \phi_u = 2.0$	square and cylinder specimens loaded at 1 day square specimens loaded at 28 days (based on ACI-209-R(97))
Cordoba (2007) [26]	$\phi(t, t_0) = \frac{t^\alpha}{d + t^\alpha} \phi_u$	The model and related coefficients are based on ACI-209-R(97) model
Aslani and Maia (2013) [27]	$\varepsilon_{cr}(t, t', t_0) = \sigma'_{cp} \times \varepsilon'_{cr} [1 - \exp\{-0.09(t - t')^{0.54}\}] \times \left(0.015 + 1.35 \left(\frac{c}{p} \right) \right)^{-1}$ $: \text{for } \frac{c}{p} < 0.65$ $\varepsilon_{cr}(t, t', t_0) = \sigma'_{cp} \times \varepsilon'_{cr} [1 - \exp\{-0.09(t - t')^{0.54}\}] \times \left(0.015 + 1.05 \left(\frac{c}{p} \right) \right)^{-1}$ $: \text{for } \frac{c}{p} \geq 0.65$ $\sigma'_{cp} = \frac{\mu + \lambda \cdot \sigma(t, t_0)^a}{1 - \kappa}$ $\sigma(t, t_0) = \frac{\sigma(t_0)}{f_{cm}(t)}$ $f_{cm}(t) = f'_{c,28} \cdot \exp \left[s' \left(1 - \left(\frac{28}{t} \right)^n \right) \right]$	$\lambda=1.80$, $\mu=0.9$, $\alpha=2.1$ experimental parameters

4.5.3.2. Shrinkage models of self-compacting concrete

Table (4.6) shows the existing models in codes of practice and some empirical models to calculate the shrinkage strain in SCC. Since these models, especially the empirical models have been developed for specific materials and testing setup, they may vary in accuracy and complexity in calculations.

Table 4.6: Existing shrinkage models for self-compacting concrete

Reference	Shrinkage model	Notes
Poppe and De Schutter (2005) [24]	$\varepsilon_{sh}(t, t_0)$ $= \left[\frac{160}{1 - \alpha(w/c)} + 10\beta_{sc} \left(9 - \frac{f_{cm}}{f_{cm0}} \right) \right]$ $\times [-1.55(1 - (RH/RH_0)^3)]$ $\times \left[\frac{(t - t_0)/t_1}{45.5(h/h_0)^2 + (t - t_0)/t_1} \right]^\gamma$ $\alpha = 4.1 \left(\frac{c}{p} \right) - 1.8$ $\gamma = -2.5 \left(\frac{c}{p} \right) + 2.6$	Model and coefficients are based on CEB-FIP(90)
Larson (2006) [25]	$(\varepsilon_{sh})_t = \frac{t}{20 + t} \times 550 \times 10^{-6}$ $(\varepsilon_{sh})_t = \frac{t}{20 + t} \times 600 \times 10^{-6}$	Square specimens Cylinder specimens
Cordoba (2007) [26]	$(\varepsilon_{sh})_t = \frac{t^\alpha}{f + t^\alpha} (\varepsilon_{sh})_u$	Model and coefficients are based on ACI-209R(97)
Khayat and Long (2010) [28]	$\varepsilon_{sh} = -k_s k_h \left(\frac{t}{55 + t} \right) (0.56 \times 10^3) \times A$ $k_s = \left[\frac{\frac{t}{26e^{0.0142(V/S)} + t}}{\frac{t}{45 + t}} \right] \left[\frac{1064 - 3.7(V/S)}{923} \right]$ $A = 0.918, 1.065$	Model and coefficients are based on AASHTO (2004) A: Cement type factor Steam-cured condition
Aslani and Maia (2013) [27]	$\varepsilon_{sh}(t, t_0) = \varepsilon_{sh} \left[1 - \exp \left\{ -0.1 \left(t - t_0 \right)^{(-2.4(\frac{c}{p}) + 2.3)} \right\} \right]$ $\varepsilon_{sh} = \left[-50 + 78 \left\{ 1 - \exp \left(\frac{RH}{100} \right) \right\} + \right.$ $\left. 38.3 \ln w - 0.92 \ln \left(\frac{w}{c} \right) - 5 \left[\ln \left(\frac{V/S}{10} \right) \right]^2 \right] \times$ $(10^{-5}): \text{for } \frac{c}{p} < 0.65$	RH: relative humidity w/c : water to cement ratio V/S volume to surface ratio c/p: cement to powder ratio

4.6. SOME EXPERIMENTS ON CREEP AND SHRINKAGE OF LIGHTWEIGHT CONCRETE IN THE LITERATURE

Best et al. (1959) [14] reported the creep data in a concrete made with expanded shale aggregates. The monitoring period was 18 months and the compressive strength of concrete at 28 days was in the range of 20 to 35 MPa. They found that contrary to current opinion, creep in lightweight concrete was essentially equal to or less than that in conventional concrete of comparable strength. All 20.6 MPa lightweight concretes had a lower creep strain and total strain (instantaneous plus shrinkage plus creep) than the corresponding conventional concretes.

The total strains of the 35 MPa LWC specimens were equal to or lower than the total strain of the corresponding CC.

Drying shrinkage of the 35 MPa LWC was only 36% to 46 % of the drying shrinkage of the CC. The instantaneous strains of LWC were considerably greater, particularly for the 20MPa concrete. Also, the instantaneous strains of the 20 MPa LWC were about 42 % to 93 % higher than the instantaneous strain of CC.

4.7. BOND TRANSFER LENGTH IN LIGHTWEIGHT CONCRETE

Due to presence of steel in the reinforced concrete member, consideration of bond between the steel bar and concrete and their interaction is inevitable. The concrete-reinforcement bond between two adjacent cracks carries a certain amount of tensile force normal to the cracked plane contributes to the overall stiffness of the member. With the perfect bond, no slip occurs between concrete and reinforcement, whereas with a poor bond, relative displacement (slip) can occur [29].

When the tensile force carried by concrete is transferred to the steel bar, the steel stress at the cracked section increases under restriction by bond forces developed along a certain length of the steel bars called bond transfer length on either side of the crack [30]. There are just limited studies in the short-term definition of bond transfer length; and long-term variation is almost unknown. By increasing the cracks, the service loading gradually moves the bond transfer region toward the unloaded end. From the resulting strain curves, a more or less bilinear decreasing trend are seen, in which, the transition point happening at the limit of the initial transfer region [31].

Beeby (1972) assumed a fixed transfer length (S_0) in short-term loading to transfer the bond stress through the concrete to steel. He investigated the mechanism of primary crack formation to reach the final pattern in this regard [5], [32]. Considering the serviceability of structure,

time-dependent bond transfer length significantly affects the crack width, crack distribution and deflection of concrete structure. Even in the case of inadequate provided development/splice length, bonding may determine the ultimate capacity of the reinforced concrete member [33].

Measurement of the local bond stress and local slip along a stressed steel bar in a tension member is difficult to ascertain and very sensitive to experimental errors [34]. For example, based on different relationships, tensile strength will differ because of differences in stress distribution. A comparison is made in Figure (4.6) which also shows that the tensile strength increases with increasing concrete strength, but not at the same rate [4]. As explained in Chapter Two, the tensile strength due the splitting strength is denoted f_t and the tensile strength due to the flexural test is known as the modulus of rupture f_r .

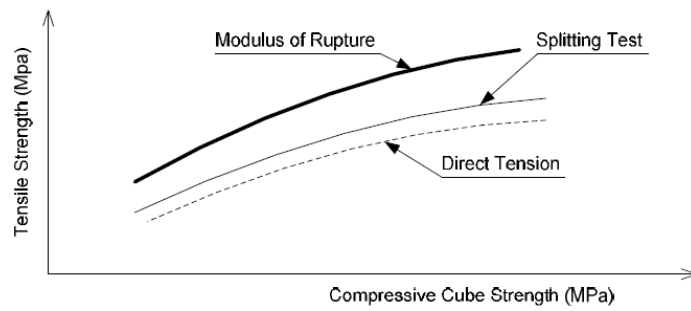


Figure 4.6: Direct and indirect tensile strength and modulus of rupture vs. compressive strength [4]

The actual bond stress distribution is shown in Figure (4.7). However, an average value ($\tau_{b-average}$) is typically used to describe the force transfer between the materials. The total force transferred to concrete is equivalent to the difference in tensile forces between adjacent cracks. By allowing a reduction in tensile force between the cracks, the contribution of concrete stiffens the response of reinforcement. The concept is called “tension stiffening” and is significantly influenced by the cracks spacing [35].

Solokov et al. (2010) [36] noted that for beams with average and high reinforcement ratios ($\rho > 0.5\%$), accurate predictions of tension stiffening by all the methods yields an error (standard deviation) from 8.8% to 10.3%. However, predictions for the lightly reinforced beams ($\rho < 0.5\%$) were far less accurate. These inaccuracies are due to the increased effect of the tensile concrete which is a highly dispersed value.

Beeby and Scott (2004) [30] proposed the Equation (4.3) to calculate the average strains ϵ_{s2} in a tension member based on bond transfer length S_0 and crack spacing λ for short-term loading.

$$\lambda = S_0 \epsilon_{s2} \quad \text{Eq. (4. 3)}$$

Where: ϵ_{s2} is the average strain in the reinforcement at a crack (concrete carries no tension), S_o is bond transfer length and λ is crack spacing.

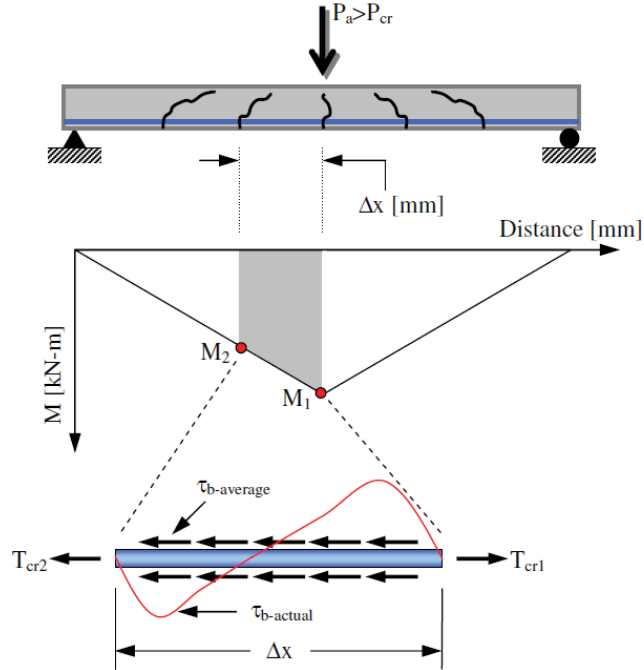


Figure 4.7: Bond transfer of tension reinforcement in concrete

Beeby and Scott (2004) [30] mentioned that the stiffness of an axially reinforced tension member is directly related to the number of cracks. They also found that the bond transfer length is shown to be proportional to the cover, and the variation in reinforcement strain is linear in the region where it is affected by a crack. This was in agreement with their previous studies. The number and the extent of cracks are controlled by the size and placement of the reinforcing steel.

Bizindavy et al. (1999) [31] concluded the existence of an approximated bilinear relationship between the bond transfer length and the relative load level with a constant value in the range of service load levels (loads lower than the initial cracking load) followed by a linearly increasing function up to failure. In this regard, an experimental study in the University of Durham shows approximately constant bond stress over time under sustained loading [37].

4.7.1. Time-dependent Bond Transfer Length

From the tension stiffening concept, concrete does not crack suddenly and completely, but undergoes progressive micro-cracking (strain softening). Due to the bond between steel and concrete, the intact concrete between adjacent primary cracks carries considerable tensile force

[38]. This bond makes the stress-strain characteristics of tensile concrete in cracked reinforced concrete significantly different from the stress-strain characteristics of plain concrete. Since bond stresses arise from the changes in the steel force along the length, the effect of bond becomes more pronounced at end anchorages of reinforcing bars and in the vicinity of the cracks.

As shown in Figures (4.8 a, b), the schematic stress-strain diagram for steel and concrete in reinforced concrete under uniaxial tension is divided into three regions; pre-cracking, crack development stage, and post-cracking. Under sustained load, σ_c gradually reduces; due to cracking and bond breakdown caused by drying shrinkage and, to a lesser extent, due to tensile creep. Early shrinkage reduces the cracking load. Under sustained service loads shrinkage makes additional primary cracks with time and causes a time-dependent decay of the steel-concrete bond. This decay of the bond is a short-term effect which occurs particularly rapidly in large diameter specimens [39].

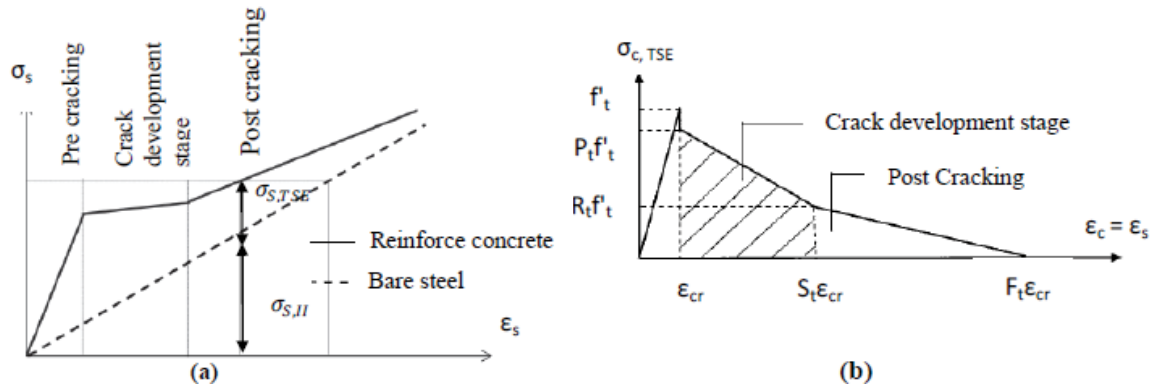


Figure 4.8: a) Tension stiffening model derived from uniaxial tension b) the equivalent concrete stress-strain relationship [40]

Nilson (1968) developed a finite element model for reinforced concrete, considering the influence of reinforcement, progressive cracking, bond stress transfer and non-linear material properties. He provided a significant improvement on the Ngo and Scordelis (1967) model. Goto (1971) applied tension to the reinforcement in a series of uniaxially reinforced concrete prisms and presented the internal crack mechanisms and determination of the schematic deformation diagram shown in Figure (4.9).

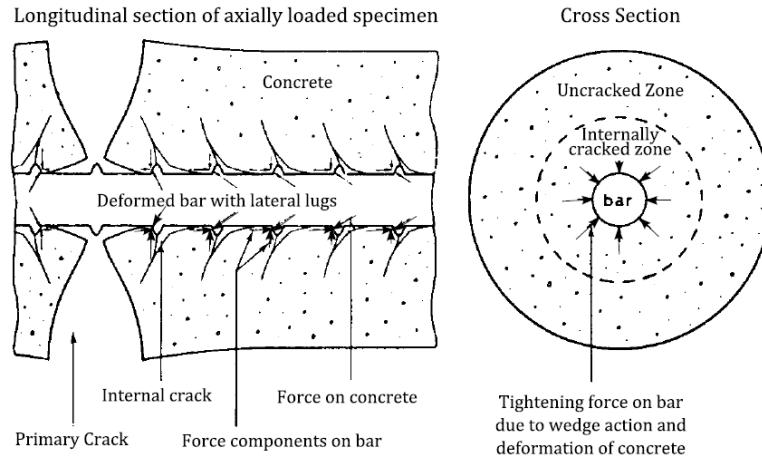


Figure 4.9: Schematic deformation of concrete in a tension specimen [41]

4.7.2. Time-dependent Effects of Creep and Shrinkage on Bond Mechanism

Creep and shrinkage over time influence the behaviour of reinforced concrete members, especially in cracked members [5]. Similar results in cracked and uncracked sections are evident in the experimental investigation performed by Divakar and Dilger (1988), emphasising the need to account for shrinkage and creep effects in long-term study of RC members [4].

Creep and shrinkage effects on the behaviour of the cracked RC members has been recognised from the beginning of the 2nd half of 20th century [36]. There are extensive analytical and numerical works carried out in the last two decades on the long-term behaviour of composite members. However, very limited benchmark experimental data is currently available in literature, e.g. (Alsamsam 1991; Bradford and Gilbert 1991; Johnson 1987; Roll 1971; Wright et al. 1992 and Jiansheng et al. 2010)[12].

Tension-stiffening relationships were coupled with shrinkage and accompanying creep effects. In most cases, tension stiffening relationships were obtained from shrunk experimental RC members [36]. Concrete shrinkage strain continues to increase with time at a decreasing rate. Other internal events such as bond slip or crack development around the bar reduce the tension stiffening. There is no reason to suppose that this reduction will occur at the same rate as creep or shrinkage. Shrinkage induced stresses significantly drop after cracking and they almost disappear with yielding of reinforcement [42].

Scanlon (1971) presented a model based on the modification of the tensile branch of concrete stress–strain relationship. This basic model was later considered and modified in different investigations. Lin and Scordelis (1975), Gilbert and Warner (1978), Cope et al. (1979), Damjanic and Owen (1984), Bazant and Oh (1984), Carreira and Chu (1986), Massicotte et al. (1990), Prakhya and Morley (1990), and Kaklauskas and Ghaboussi (2001) are some

researchers who have modified and developed the model. They describe cracking by a smeared model assuming an average concrete tensile stress in a direction perpendicular to cracks [18].

Since assuming a constant bond stress and tension stiffening over the time cannot be an adequate model for reinforced concrete, there are considerable time-dependent changes in the average tensile stress caused in concrete. These changes are caused by tensile creep, restrained shrinkage, and the time-dependent breakdown of bond due to shrinkage-induced and cover-controlled cracking between the primary cracks [43]. Webster and Brooker (2006) [44] found that effective tensile strength of the concrete, can even be the degree of restraint of shrinkage movements.

The time-dependent effects of creep, shrinkage and temperature variation also exceed the nonlinear behaviour of the RC member due to bond-slip, aggregate interlock at a crack and dowel action of the reinforcing steel crossing a crack [45].

4.7.3. Bond Behaviour After Cracking and Before Yielding of Steel

First cracking occurs at the weakest cross-section where the concrete tensile stress reaches the lower characteristic value of the tensile strength. Therefore, the stress in the concrete at the crack drops to zero. The bond characteristics of materials determine the position of subsequent cracks relative to the first cracks [46]. The zero tensile stress of concrete at each crack is rising with the distance from the crack due to the steel-concrete bond of a maximum value (less than the tensile strength of concrete) of mid-way between the adjacent cracks. The next crack will then not form within S_0 (see Figure (4.10)) of the first crack as the stresses in the concrete are lower within this limit than outside. Slip at the concrete-steel interface in the region of significant bond stress (S_0 on either side of the crack) causes the crack to open. When the bond from steel to concrete can no longer transfer sufficient tensile force to form an additional crack between two existing cracks, the final established cracking state is reached.

Scott and Gill (1971) studied seven tensile specimens with gauged reinforcing bars and found a linear variation in steel strain adjacent to a crack location and linear relation between bond stresses with the load. This linear variation of stress implies a constant bond stress, which initially suggests some form of plastic behaviour [37].

Figure (4.10a) indicates the mechanism by which cracks form in a uniaxial reinforced concrete prism under tension. Scott and Beeby (2004) presented the schematic variation of strain and stress in the region of a crack, as shown in Figure (4.10b) [30].

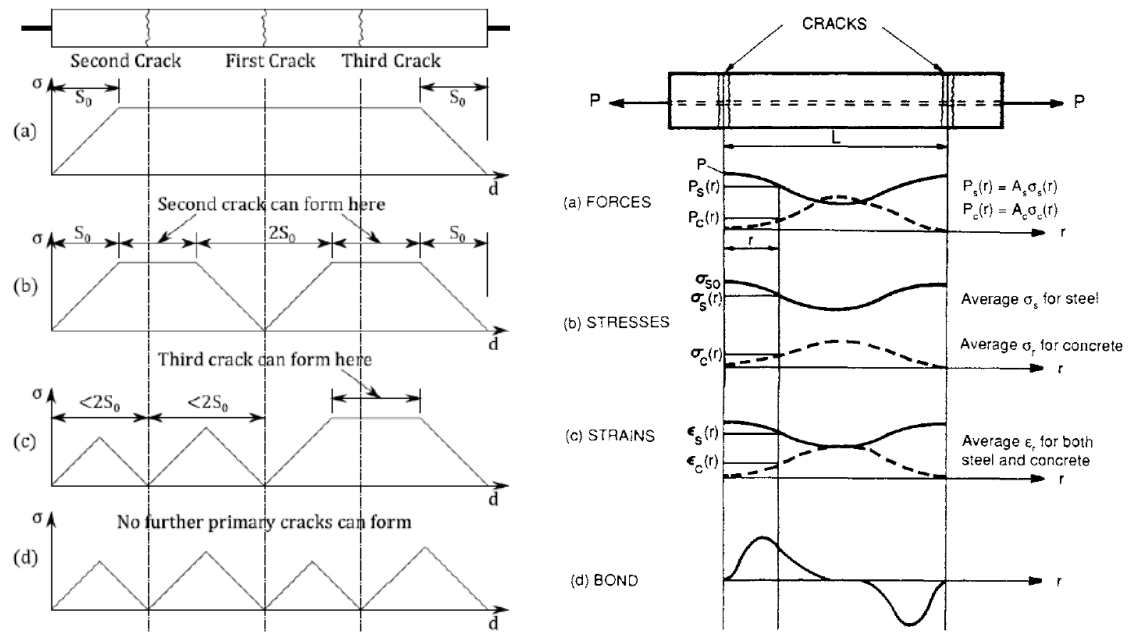


Figure 4.10: Mechanism of cracking formation in a uniaxially reinforced concrete prism under tension [30]

Schematic distribution of forces, strains, normal stresses and bond stresses along a cracked reinforced concrete member are illustrated in Figure (4.11)

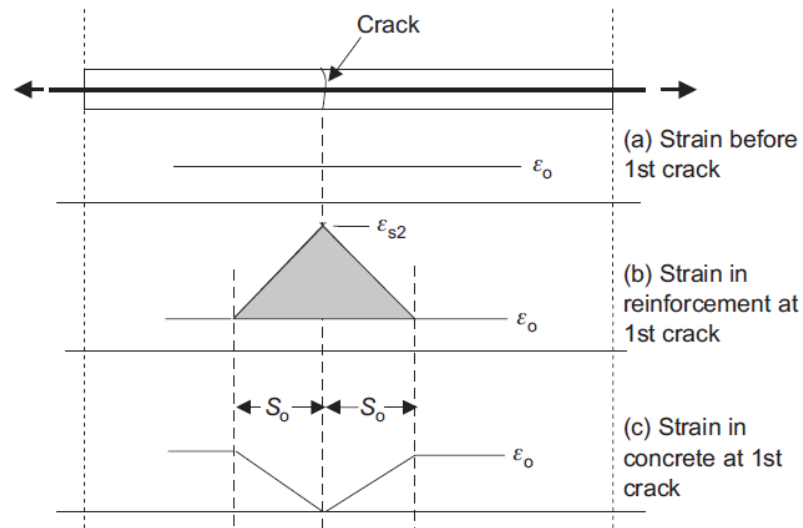


Figure 4.11: Schematic distribution of forces, strains, normal stresses, and bond stresses in cracked member

[47]

4.7.4. Experimental Investigation on the Ratio of Long-term Bond Transfer Length to Initial Bond Transfer Length in Conventional Concrete

As part of the long-term cracking study at the University of New South Wales (UNSW) by Nejadi and Gilbert (2004) [48], a total of eight fully restrained slab specimens with four different reinforcement layouts were monitored for up to 150 days. They measured the effect of shrinkage on development of time-dependent direct tension cracking due to restrained deformation. Details of bar arrangement and concrete cover for the specimens are shown in Table (4.7). Figure (4.12) displays the plan and elevation of the test specimens under pure tension. The slab specimens are effectively anchored at both ends to ensure the complete restraining of specimens. At mid-span of each specimen, the section was locally reduced to make sure that first cracking always occur at this location.

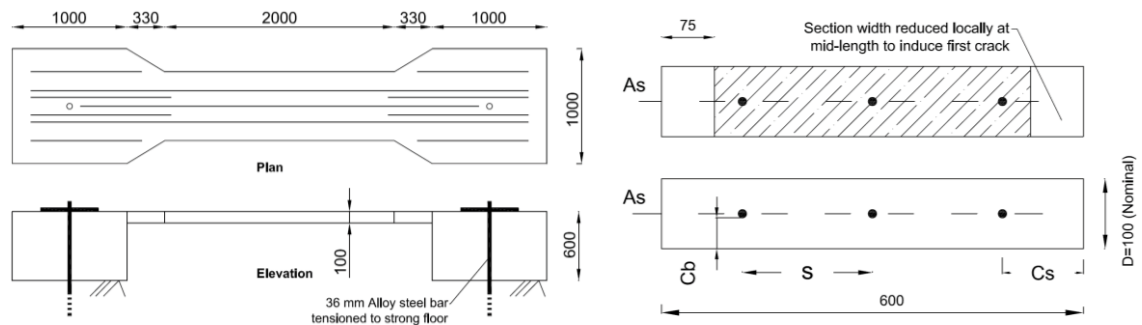


Figure 4.12: Details of specimen in long-term bond transfer length test [48]

To delay the onset of drying shrinkage, the specimens were kept in their moulds for three days and moist-cured continually in lab conditions. All slabs were anchored to the support at age 3 days. The drying process also commenced after the same period. The time-dependent development of drying shrinkage strain is also measured on unrestrained specimens of similar dimension and thickness to the slab specimens.

All tests of companion specimens were carried out in agreement with the relevant parts of the Australian Standard AS-1012.13 [49]. Two different ready mix batches of concrete were used for slabs through the test according to Table (4.8).

Table 4.7: Details of test specimens (reinforcement and dimensions)

Slab	No. of bars	Bar dia.(mm)	A_{st} (mm ²)	C_s (mm)	C_b (mm)	S (mm)
RS1-a	3	12	339	109	44	185
RS1-b	3	12	339	109	44	185
RS2-a	3	10	236	110	46	185
RS2-b	3	10	236	110	46	185
RS3-a	2	10	157	145	46	300
RS3-b	2	10	157	145	46	300
RS4-a	4	10	314	115	46	120
RS4-a	4	10	314	115	46	120

Table 4.8: Material properties of concrete batch 1 (B.1) and batch 2 (B.2)

Age (days)	3		7		14		21		28	
Concrete batch	B.1	B.2	B.1	B.2	B.1	B.2	B.1	B.2	B.1	B.2
Compressive strength (MPa)	8.17	10.7	13.7	17.4	20.7	25	22.9	27.5	24.3	28.4
Flexural tensile strength (MPa)	1.91	2.47	3.15	3.1	3.43	3.77	3.77	3.97	3.98	4.04
Indirect tensile strength (MPa)			1.55	1.6					1.97	2.1
Modulus of elasticity (MPa)	13240	16130	17130	18940	21080	21750	22150	22840	22810	23210

4.7.5. Analytical Model of Bond Transfer Length

In a fully restrained RC member as shown in Figure (4.13), as concrete shrinks, the restraining force $N(t)$ gradually increases until the first crack appears. The resulting force reduces to N_{cr} , immediately after the first crack appears and the stress away from the crack is less than the tensile strength of concrete, $f_t(t)$. On either side of the crack, the concrete shortens elastically, allowing the crack to open to a width w .

The slight increase in crack width with distance from the steel bar can be obtained by integrating the reduction in tensile elastic strain in the concrete (from that at the level of the bar) over the half crack spacing on either side of the crack.[50]

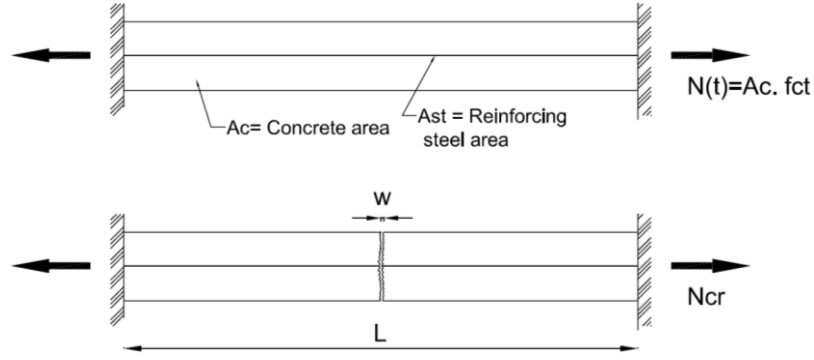


Figure 4.13: First cracking in a restrained direct tension member [51]

According to Figure (4.14), Gilbert (1992) [51] derived the following expressions in Equations (4.4) to (4.11) for the restraining force N_{cr} , the concrete and steel stresses away from the crack σ_{c1} and σ_{s1} respectively, and the steel stress at the crack σ_{s2} .

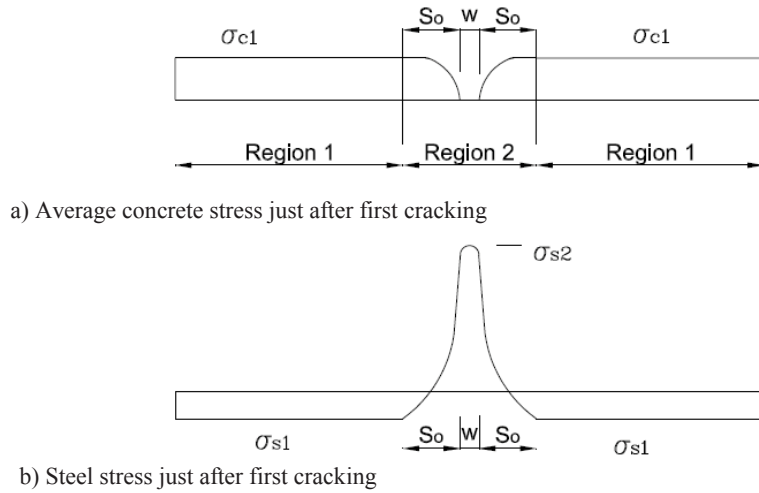


Figure 4.14: Variation of average stress in steel and concrete

$$N_{cr} = \frac{n\rho f_{ct} A_{ct}}{C_1 + n\rho(1+C_1)} \quad \text{Eq. (4.4)}$$

$$\sigma_{c1} = \frac{N_{cr} - \sigma_{s1} A_{st}}{A_c} = \frac{N_{cr}(1+C_1)}{A_c} \quad \text{Eq. (4.5)}$$

$$\sigma_{s1} = -\frac{2s_0}{3L-2s_0} \sigma_{s2} = -C_1 \sigma_{s2} \quad \text{Eq. (4.6)}$$

$$\sigma_{s2} = \frac{N_{cr}}{A_{st}} \quad \text{Eq. (4.7)}$$

$$C_1 = \frac{2s_0}{3L-2s_0} \quad \text{Eq. (4.8)}$$

$$\frac{\sigma_{s1}^*}{E_s} L + m \frac{\sigma_{s2}^* - \sigma_{s1}^*}{E_s} \left(\frac{2}{3} s_0 + w \right) = \Delta u \quad \text{Eq. (4.9)}$$

$$\sigma_{s1}^* = \frac{-2s_0m}{3L-2s_0m} \sigma_{s2}^* + \frac{3\Delta u E_s}{3L-2s_0m} \quad \text{Eq. (4. 10)}$$

$$\sigma_{s2}^* = \frac{N(\infty)}{A_{st}} \quad \text{Eq. (4. 11)}$$

The bond transfer length S_0 as presented in Equation (4.12) is related to bar diameter and ratio of reinforcement. This expression was proposed earlier by Faver et al. (1983) [52] for a member that contained deformed bars or welded wire mesh. Base and Murray (1982) [29] used a similar expression.

$$S_0 = \frac{d_b}{10\rho} \quad \text{Eq. (4. 12)}$$

Where; d_b is bar diameter (mm) and ρ is tensile reinforcement ratio in the section.

Bamonte and Gambarova (2007) [53] compared the bond-slip in a plain and a deformed bar and showed the difference as illustrated in Figure (4.15).

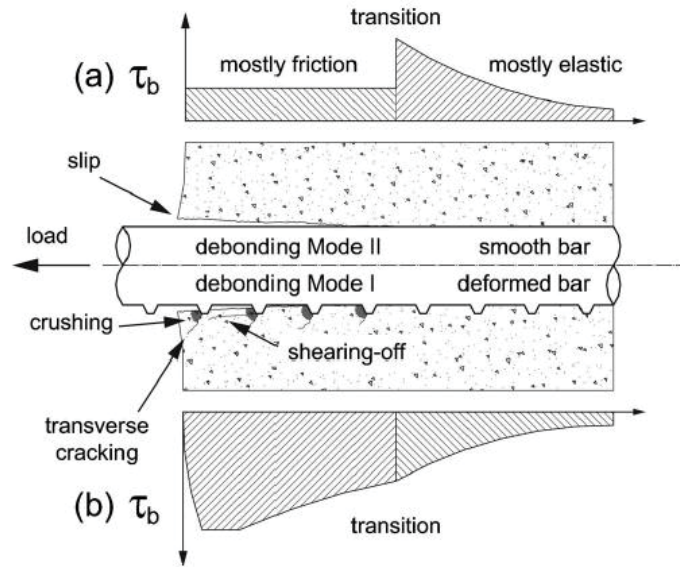


Figure 4.15: Bond-slip mechanisms in a) smooth bars; b) deformed bars [53]

The code alterations regarding tension stiffening proposed by Beeby, Scott and Jones (2005) are supporting the effect of bar diameter and reinforcement ratio on bond transfer length [54]. The good bond between steel and concrete increases the stiffening effect and is more significant for low reinforcement ratios than for higher ones, reported by Massicotte et al. (1990) in Figure.

(4.16) [29]. Horizontal and vertical axes in Figure (4.16) are presenting the deflection (Δ) and load (T), respectively.

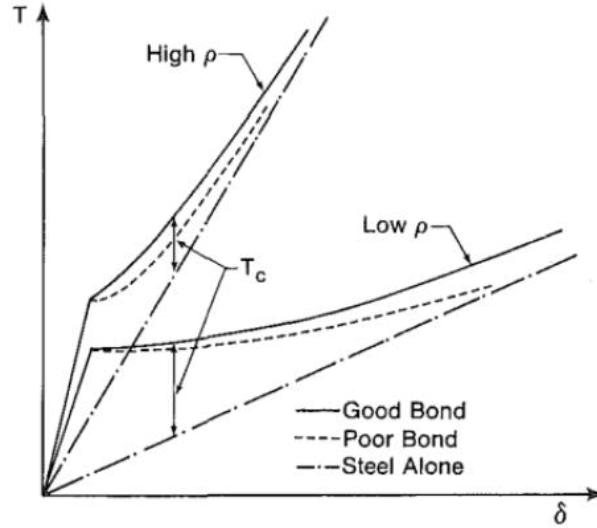


Figure 5.16: Tension stiffening effect in strain-stress diagram respect to bond quality and steel ratio [29]

Sokolovas et al. (2010) [36] noted that the free shrinkage tension stiffening effect is strongly related to the reinforcement ratio and is considerably more pronounced for beams with smaller reinforcement ratios.

Figures (4.17 a, b and c) respectively show a portion of a fully restrained member under direct tension when a) all the shrinkage has occurred, and b) the final pattern of cracks has been established, and c) it illustrates the average stresses in the concrete and steel caused by shrinkage [41].

Gilbert (1992) [51] derived an expression for the final average crack spacing S_r and width in a fully restrained member, the final restraining force in a member, and the final concrete and steel stresses by enforcing the requirements of compatibility and equilibrium. With these derivations, the bond transfer length S_0 was assumed to remain constant over time, and the supports of the member were presumed to be immovable. However, these assumptions may introduce significant error, because the results from this experimental program indicate the shrinkage effects on the deterioration of the bond at the concrete-steel interface and a gradual increase in S_0 with time.

From the average steel strain in the region along the steel bars, the steel stress remote from the cracks after the first crack (σ_{s1}) and after all shrinkage cracking (σ_{s1}^*) is calculated. Similarly, from the measured steel strains in region 2 (measured by electric strain gauges) and obtaining the average value for these strain gauges between the reinforcement bars, the steel

stress in the vicinity of the first crack immediately after the first cracking of σ_{s2} , and after all shrinkage cracking σ_{s2}^* is also calculated.

By using Equations (4.4) to (4.11), the bond transfer length immediately after the first cracks S_0 , and, after all shrinkage cracking S_0^* , was obtained. The results and ratio of S_0^*/S_0 are presented in Table (4.9). Batches B1 and B2 are related to index 'a' and 'b' respectively.

Table 4.9: Measured no-bond length after first and all shrinkage cracking

Slab	Measured bond transfer length (mm)							
	RS1- a	RS1- b	RS2- a	RS2- b	RS3- a	RS3- b	RS4- a	RS4- b
After first cracking (S_0)	261	215	228	201	246	342	256	259
After 150 days shrinkage	366	290	290	292	323	454	332	319
S_0^*/S_0	1.4	1.35	1.27	1.45	1.31	1.33	1.29	1.23

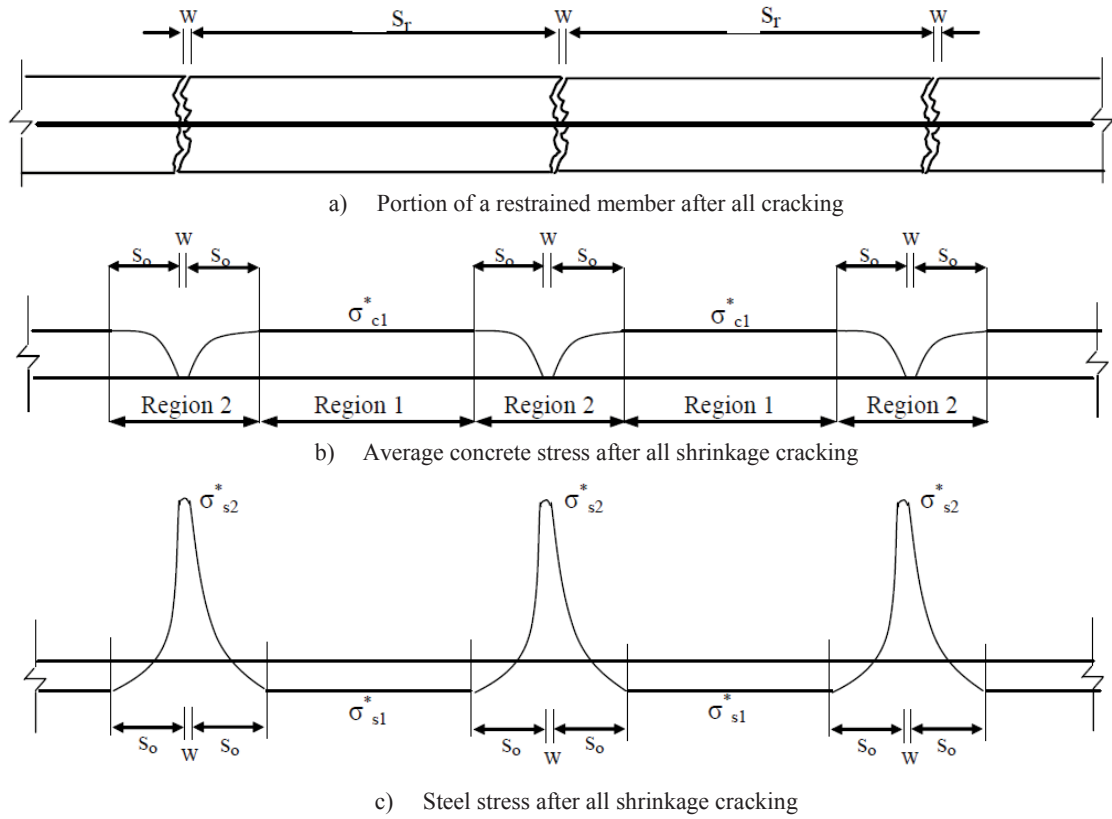


Figure 4.17: Final concrete and steel stresses after direct tension cracking [51]

From Table (4.9), the average ratio of the final bond transfer length S_0^* to the initial value after first cracking S_0 is 1.33. Therefore, the final bond transfer length for long-term calculations may be expressed as Equation (4.13)

$$S_0^* = 1.33 S_0 \quad \text{Eq. (4. 13)}$$

From the test results, the extent to which shrinkage cracking can be controlled depends on limiting the desired crack width and the amount and distribution of bonded reinforcement across the crack.

The final crack width, crack spacing and steel stress at the crack are dependent on the steel area, or more precisely, the reinforcement ratio. An increase in the steel area reduces the final crack width and with more cracks developing, reduces the crack spacing.

With an increase in the steel area, the loss of stiffness at first cracking is reduced and, therefore the restraining force after cracking is greater, but stress in the steel decreases at each crack. With a larger restraining force, the stress in the concrete away from a crack tends to be higher and consequently further cracking is more likely

4.8. EXISTING MODELS OF BOND-SLIP FOR LIGHTWEIGHT, CONVENTIONAL AND SELF-COMPACTING CONCRETE

4.8.1. Bond-Slip Models for Lightweight Concrete

Table (4.10) shows the existing models used to predict the relationship between bond strength and slip between the reinforcing bars and surrounding light weight concrete.

Table 4.10: Existing bond-slip models of light-weight concrete in literature

Reference	Model for bond-slip	Notes
Malvar et al. (2003) [55]	Descending $\begin{cases} \frac{\tau_{max}}{f_t} = A + \frac{B\sigma}{f_t} \\ \frac{s_m}{d_b} = C + \frac{D\sigma}{f_t} \end{cases}$ $\{\tau = \tau_{max}$ Ascending $\tau = \tau_{max} \left(\frac{s}{s_m} \right)^\alpha$	τ_{max} : peak bond stress σ : confining axisymmetric radial pressure f_t : tensile strength s_m : slip at peak bond stress d_b : nominal bar diameter α , sr: empirical coefficients

Mitchel and Marzouk (2007) [56] investigated the bond-slip behavior of High-Strength Light-Weight Concrete (HSLWC) and compared with the previous work at the same concrete

lab by Alavi-Fard and Marzouk (2004) [57] on High-Strength Normal-Weight Concrete (HSNWC). The same specimen size, casting position and test setup was used for all HSLWC and HSNWC specimens. The experimental program consisted pull-out testing of 72 specimens using 25 and 35 mm deformed reinforcement embedded in the specimens. The compressive strength of specimens was about 80 MPa. The bond-slip behaviour in 25 mm and 35 mm steel bars in HSLWC are presented in Figure (4.18).

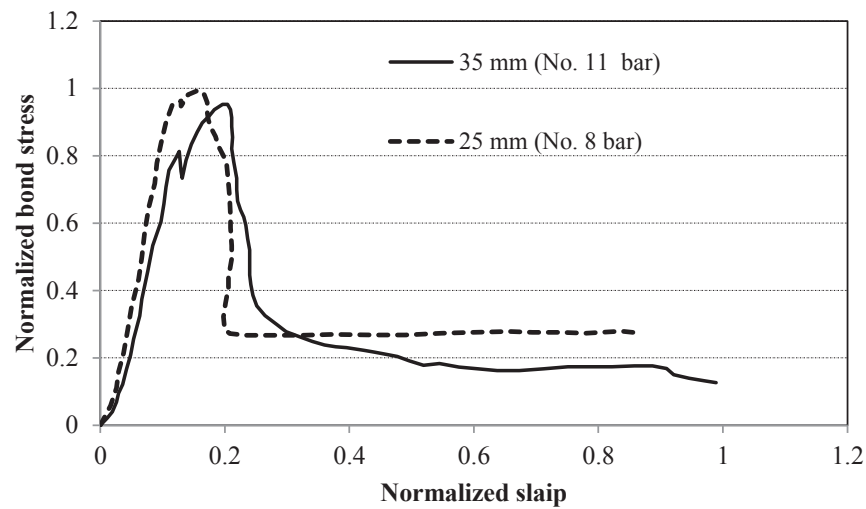


Figure 4.18: Normalized bond stress-slip for 25 and 35 mm bars under compression [56]

In comparison with HSNWC, they observed a similar manner in the bond-slip behaviour of HSLSC and HSNWC. However, the maximum bond stress value (τ_{max}) for 25 mm and 35 mm bars were 10% lower and 10% greater in LWHSC than those for HSNWC respectively. The HSLWC also showed a brittle failure during the pull-out tests.

The final slip value associated with bond failure was approximately five times greater than the slip at the point of τ_{max} . However, the shape of the bond-slip curve in both types of concrete specimens was similar. The decrease in bond stress in the descending part was 50% to 60% for HSLWC compared to 30% to 40% for HSNWC.

Based on the experimental results on the lightweight concrete containing expanded polystyrene beads, a new model is proposed and verified in Chapter Five of this study.

4.8.2. Bond-Slip Models in Conventional Concrete

Table (4.11) shows the existing models used to predict the relationship between bond strength and slip between the reinforcing bars and surrounding conventional concrete.

Table 4.11: Existing bond-slip models of conventional concrete in literature

Reference	Bond-slip models of conventional concrete	Notes
Gupta and Maestrini (1990) [58]	$\frac{d^2s}{dx^2} = \frac{p\tau}{A_s E_s} (1 + n\rho)$ $n = \frac{E_s}{E_c}$, $\rho = \frac{A_s}{A_c}$	p represents the perimeter (circumference) of the steel bar
Kankam(1997) [59]	$\tau = (41.7 - 0.2f_s)s^{0.8}$ Plain round bar $\tau = (55 - 0.5x)s^{0.8}$ Cold-worked ribbed bar $\tau = (35 - 0.3x)s^{0.5}$ Hot-rolled ribbed bar	Where τ =bond stress, in N/mm ² , x =distance from centre of embedment, in mms; and s =local slip of bar, in mms, f_s =steel stress
Nilson (1971)	$\tau = 3100(1.43x + 1.5)s\sqrt{f'_c}$	Where X = distance from loaded face; and $\sqrt{f'_c}$ = measure of tensile strength of concrete.
Mirza and Houde (1979) [60]	$\tau = 1.95 \times 10^6 s - 2.35 \times 10^9 s^2 + 1.39 \times 10^{12} s^3 - 0.33 \times 10^{15} s^4$	s =local slip of bar, in mm
Rehm (1961) [61]	$\tau = f_{c.cub}(\phi s^\alpha \pm \Psi_s)$	where Φ , ψ , α are theoretical or experimental constants
Nilson (1968) [62]	$\tau = 998.4s - 58400s^2 + 852200s^3$	
Martin (1973) [63]	$\tau = \tau_0 + \alpha s^\beta$	
Mirza and Houde (1979) [60]	$\tau = 539.8s - 25610s^2 + 592200s^3 - 5574000s^4$	
Eligehausen et al. (1983) [64]	$\tau = \tau_{max} \left(\frac{s}{s_1}\right)^\alpha \quad 0 \leq s \leq s_1$	s, s ₁ (Figure 4.19)
CEB-FIP (1993) [18] Huang et al. (1996) [65] Harajli et al (1995) [66]	$\tau = \tau_{max} : s_1 \leq s \leq s_2$ $\tau = \tau_{max} - (\tau_{max} - \tau_f) \left(\frac{s - s_2}{s_3 - s_2}\right) : s_2 \leq s \leq s_3$ $\tau = \tau_f : s_3 < s$	s, s ₁ , s ₂ , s ₃ (Figure 4.19)

Table 4.11: Existing bond-slip models of conventional concrete in literature (continued)

Reference	Bond-slip models of conventional concrete	Notes
Wu and Zhao (2012)	$\tau = \frac{\tau_{max}}{[e^{-B \ln(B/D)/(B-D)} - e^{-D \ln(B/D)/(B-D)}] (e^{Bs}) - e^{Ds}}$ $B = \frac{0.0254 + K_{st}}{-0.0232 - 8.34 K_{st}}$ $-1.1 \leq B \leq -0.12$ $D = 3 \ln \left(\frac{0.7315 + K}{5.176 + 0.3333 K} - 0.13 \right) - 3.375$ $K = (K_{co} + F K_{st}) K_{si}$ $K_{si} = \left(\frac{d_b}{25.4} \right)^{0.2 - 1.8 R_r}$	$K_{st} = A_{st} / n S_{st} d_b$ confinement factor $K_{co} = c/d_b$ combined parameter of bar diameter and concrete K_{si} : size effect factor R_r: ratio of the rib height to the rib spacing
Murcia-Delso et al. (2011) [67]	$\tau = \frac{4\tau_{max}}{S_{peak}} s : s < 0.1 s_{peak}$ $\tau = \tau_{max} \left[1 - 0.6 \left(\frac{s - s_{peak}}{0.9 s_{peak}} \right)^4 \right] : 0.1 s_{peak} \leq s \leq s_{peak}$ $\tau = \tau_{max} : s_{peak} \leq s \leq 1.1 s_{peak}$ $\tau = \tau_{max} \left[1 - 0.75 \left(\frac{s - 1.1 s_{peak}}{s_R - 0.9 s_{peak}} \right) \right] : 1.1 s_{peak} \leq s \leq s_R$ $\tau = 0.25 \tau_{max} : s \geq s_R$	For S_{peak} and S_R , see Figure (4.20)
Barbosa (2001) [68]	$\tau = 19.36 s^{0.51} : f'_c < 50 \text{ MPa}$ $\tau = 32.58 s^{0.48} : f'_c \geq 50 \text{ MPa}$	
Oh and Kim (2007) [69]	$\tau = 2.5 f'_c{}^{0.6} \left(\frac{s}{s_{p1}} \right)^\alpha$ $\tau_n = 0 : s < s_{r(n-1)}$ $\tau_n(s) = \tau_{max} \left(\frac{s - s_{r(n-1)}}{s_{p1}} \right) : s \geq s_{r(n-1)}$	s_{p1}: slip value at τ_{max} A: empirical factor τ_n: bond stress for repeated loading

The parameters s , s_1 , s_2 and s_3 in the Equation presented by CEB-FIP (1993) are shown in Figure (4.19). This model is approved later by Huang et al. (1996), and Harajli et al (1995).

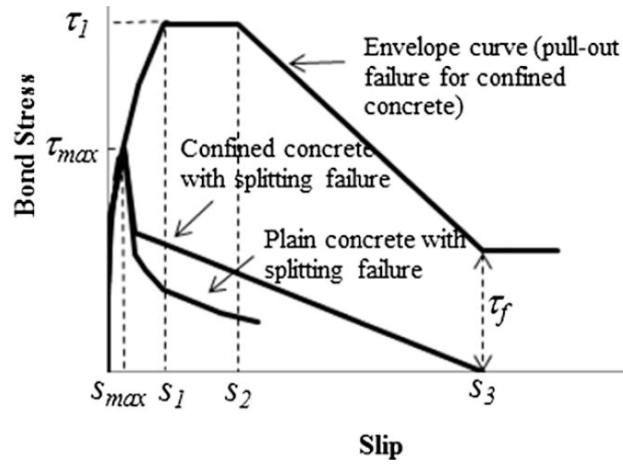


Figure 4.19: Bond-slip model in reinforced concrete (CEB-FIP, 1993)[18]

The parameters in the equation presented by Murcia-Delso et al. (2011) are shown in Figure (4.20).

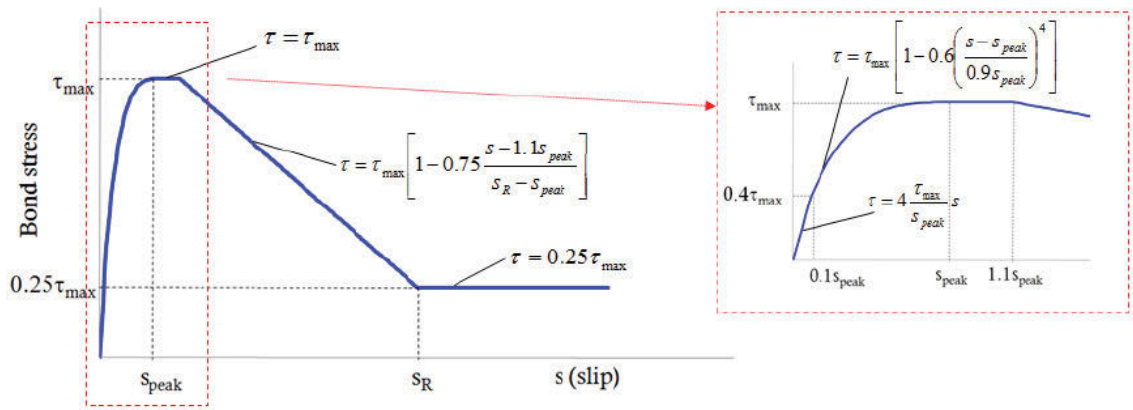


Figure 4.20: The parameters for analytical models developed by Murcia-Delso et al. (2011)[67]

4.8.3. Bond-Slip Models in Self-compacting Concrete

Table (4.12) shows the existing models of predicting the relationship between bond strength and slip between the reinforcing bars and surrounding self-compacting concrete.

Table 4.12: Existing bond-slip models for self-compacting concrete in literature

Reference	Model	Notes																					
Almeida Filho et al. (2008) [70]	$\tau = \tau_{max} \cdot \left(\frac{s}{s_1}\right)^{\alpha}$ $0 \leq s \leq s_1$ $\tau = \tau_{max}$ $s_1 \leq s \leq s_2$ $\tau = \tau_{max} - (\tau_{max} - \tau_u) \left(\frac{s - s_2}{s_3 - s_2}\right)$ $s_2 \leq s \leq s_3$ $\tau = \tau_u$ $s_3 < s$	Model based on Ceb-Fip 195/197 (1993) [18]																					
		<table><tr><td></td><td>Confined concrete</td><td>Non-Confined concrete</td></tr><tr><td>s_1</td><td>1.0 mm</td><td>0.6 mm</td></tr><tr><td>s_2</td><td>3.0 mm</td><td>0.6 mm</td></tr><tr><td>s_3</td><td>Ribs interval</td><td>1.0 mm</td></tr><tr><td>α</td><td>0.4</td><td>0.4</td></tr><tr><td>τ_{max}</td><td>$2.5\sqrt{f'_c}$</td><td>$2.0\sqrt{f'_c}$</td></tr><tr><td>τ_u</td><td>$0.4 \tau_{max}$</td><td>$0.15 \tau_{max}$</td></tr></table>		Confined concrete	Non-Confined concrete	s_1	1.0 mm	0.6 mm	s_2	3.0 mm	0.6 mm	s_3	Ribs interval	1.0 mm	α	0.4	0.4	τ_{max}	$2.5\sqrt{f'_c}$	$2.0\sqrt{f'_c}$	τ_u	$0.4 \tau_{max}$	$0.15 \tau_{max}$
			Confined concrete	Non-Confined concrete																			
		s_1	1.0 mm	0.6 mm																			
		s_2	3.0 mm	0.6 mm																			
		s_3	Ribs interval	1.0 mm																			
		α	0.4	0.4																			
		τ_{max}	$2.5\sqrt{f'_c}$	$2.0\sqrt{f'_c}$																			
		τ_u	$0.4 \tau_{max}$	$0.15 \tau_{max}$																			
		Model based on Huang et al. (1996) [65]																					
		<table><tr><td></td><td>Confined concrete</td><td>Non-Confined concrete</td></tr><tr><td>s_1</td><td>0.5 mm</td><td>1.0 mm</td></tr><tr><td>s_2</td><td>1.5 mm</td><td>3.0 mm</td></tr><tr><td>s_3</td><td>Ribs interval</td><td>Ribs interval</td></tr><tr><td>α</td><td>0.3</td><td>0.4</td></tr><tr><td>τ_{max}</td><td>$0.4f_{cm}$</td><td>$0.4f_{cm}$</td></tr><tr><td>τ_u</td><td>$0.4 \tau_{max}$</td><td>$0.15 \tau_{max}$</td></tr></table>		Confined concrete	Non-Confined concrete	s_1	0.5 mm	1.0 mm	s_2	1.5 mm	3.0 mm	s_3	Ribs interval	Ribs interval	α	0.3	0.4	τ_{max}	$0.4f_{cm}$	$0.4f_{cm}$	τ_u	$0.4 \tau_{max}$	$0.15 \tau_{max}$
			Confined concrete	Non-Confined concrete																			
		s_1	0.5 mm	1.0 mm																			
		s_2	1.5 mm	3.0 mm																			
		s_3	Ribs interval	Ribs interval																			
α	0.3	0.4																					
τ_{max}	$0.4f_{cm}$	$0.4f_{cm}$																					
τ_u	$0.4 \tau_{max}$	$0.15 \tau_{max}$																					

4.9. SUMMARY

Following the description of the mechanical properties of EPS lightweight concrete in Chapter 3, the time-dependent properties are discussed in Chapter 4. A brief description of the creep and shrinkage effects on the deflection behaviour of reinforced concrete structures over time is given also. Other effective parameters in the moment-curvature diagram of reinforced concrete members subjected to the simultaneous effects of the sustained loading and creep and shrinkage are explained.

The existing models used to predict the creep and shrinkage behaviour of lightweight concrete are presented. Also the existing models for conventional and self-compacting concrete are given and compared with the models for lightweight concrete. The experimental investigations and the existing models show a higher range of creep and shrinkage for lightweight concrete compared to the conventional and self-compacting concrete.

Characteristics of the steel-concrete interface are discussed as an effective parameter both in instantaneous and time-dependent deflection of reinforced concrete structures. Some existing models to estimate the bond-slip behaviour and the maximum bond stress in lightweight, conventional and self-compacting concrete are given as well.

The effect of bond transfer length on the deflecting and cracking behaviour of reinforced concrete structures are also investigated and the relative models in the literature are presented.

The results of an experimental study by Nejadi and Gilbert (2004) to measure the time-dependent bond-transfer length in a series of conventional concrete slabs are broadly evaluated.

The bond-slip changes between the reinforcing bars and the surrounding concrete is among the most important parameters to influence the time-dependent and instantaneous deflection behaviour of concrete structures. The existing models for lightweight, conventional and self-compacting concrete are presented and compared.

REFERENCES

1. Gilbert, R.I., *Time effects in concrete structures*. Developments in civil engineering, 1988. **23**.
2. Unnikrishna Pillai, S. and D. Menon, *Reinforced Concrete Design* Tata Mc Graw Hill, New Delhi, India,, 2003.
3. Nejadi, S., *Time-dependent cracking and crack control in reinforced concrete structures*. 2005, PhD thesis, The University of New South Wales Sydney, Australia.
4. Eigelaar, E.M., *Deflections of reinforced concrete flat slabs*. 2010, Stellenbosch: University of Stellenbosch.
5. Nie, J. and C.S. Cai, *Deflection of cracked RC beams under sustained loading*. Journal of Structural Engineering, 2000. **126**(6): p. 708-716.
6. Reichard, T.W., *Creep and drying shrinkage of lightweight and normal weight concretes*. National Bureau of Standards, Washington, D.C., Monograph 74, 1964.
7. Aslani, F., *Experimental and numerical study of time-dependent behaviour of reinforced self-compacting concrete slabs*. PhD dissertation, University of Technology Sydney (UTS), 2014.
8. Bazant, Z.P., *Criteria for rational prediction of creep and shrinkage of concrete*. ACI SPECIAL PUBLICATIONS, 2000. **194**: p. 237-260.
9. Chia, K.-S., et al., *Experimental study on creep and shrinkage of high-performance ultra-lightweight cement composite of 60 MPa*. Structural Engineering and Mechanics, 2014. **50**(5): p. 635-652.
10. Zhang, Q., et al. *Long-term creep properties of cementitious materials—comparing compression tests on concrete with microindentation tests on cement*. in *American Society of Civil Engineers*. 2013.
11. Bazant, Z.P. and R. L'Hermite, *Mathematical modeling of creep and shrinkage of concrete*. 1988: Wiley Chichester.
12. Gilbert, R.I., *Creep and Shrinkage Induced Deflections in RC Beams and Slabs*. ACI , Special Publication, 2012. **284**(13): p. 1-16.
13. ACI-209-2R, *Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete*. Reported by ACI Committee 209, 2008.
14. Best, C.H. and M. Polivka *Creep of lightweight concrete*. Magazine of Concrete Research, 1959. **11**, 129-134.
15. Sabaa, B. and R.S. Ravindrarajah. *Engineering properties of lightweight concrete containing crushed expanded polystyrene waste*. in *Materials Research Society, 1997, Fall Meeting, Symposium MM, Advances in Materials for Cementitious Composites December*. 1997.
16. ACI-209-2R, *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures*. Reported by ACI Committee 209, 1997.
17. CEB-99, *Concrete, Structural, Textbook on Behaviour, Design and Performance. Updated knowledge of the CEB/FIP Model Code 1990*. fib Bulletin 2, V.2, Fédération Internationale du Béton, Lausanne, Switzerland, 1999. **1**: p. 37-52.
18. béton, C.e.-i.d., *CEB-FIP model code 1990: design code*. 1993: CEB Bulletin d'Information No.213/214, Comité EuroInternational du Béton, Lausanne, Switzerland, Telford.
19. EC-2, E., *Design of Concrete Structures, Part 1: General Rules and Rules for buildings*. Commission of European Communities ENV, German version EN 1992-1-1:2004 + AC: 2010, The Concrete Centre: Blackwater, Camberley, UK, . 2004: p. 1-1.
20. Gardner, N., *Comparison of prediction provisions for drying shrinkage and creep of normal-strength concretes*. Canadian Journal of Civil Engineering, 2004. **31**(5): p. 767-775.
21. du Béton, F.I., *fib Model Code for Concrete Structures 2010*. 2013, Lausanne, fib - federation internationale du beton, John Wiley & Sons, 1 Oct 2013 - Technology & Engineering - 500 pages.

22. AS-3600-09, *Concrete structures*. Australian Standard, 2009.
23. ACI-435-R95, *Control of Deflection in Concrete Structures*. American Concrete Institute (ACI) Committee 435, 2003.
24. Poppe, A.-M. and G. De Schutter. *Creep and shrinkage of self-compacting concrete*. in *First International Symposium on Design, Performance and Use of Self-Consolidating Concrete, China*. 2005.
25. Larson, K.H., *Evaluating the time-dependent deformations and bond characteristics of a self-consolidating concrete mix and the implication for pretensioned bridge applications*. 2006, Kansas State University.
26. Cordoba, B., *Creep and shrinkage of self-consolidating concrete (SCC)*. 2007: University of Wyoming.
27. Aslani, F. and L. Maia, *Creep and shrinkage of high-strength self-compacting concrete: experimental and analytical analysis*. Magazine of Concrete Research, 2013. **65**(17): p. 1044-1058.
28. Khayat, K.H. and W.J. Long, *Shrinkage of precast, prestressed self-consolidating concrete*. ACI Materials Journal, 2010. **107**(3).
29. Massicotte, B., A.E. Elwi, and J.G. MacGregor, *Tension-stiffening model for planar reinforced concrete members*. Journal of Structural Engineering, 1990. **116**(11): p. 3039-3058.
30. Beeby, A. and R. Scott, *Cracking and deformation of axially reinforced members subjected to pure tension*. Magazine of concrete research., 2005. **57**(10): p. 611-621.
31. Bizindavyi, L. and K. Neale, *Transfer lengths and bond strengths for composites bonded to concrete*. Journal of composites for construction, 1999. **3**(4): p. 153-160.
32. Alih, S. and A. Khelil. *Tension Stiffening Parameter in Composite Concrete Reinforced with Inoxydable Steel: Laboratory and Finite Element Analysis*. in *Proceedings of World Academy of Science, Engineering and Technology*. 2012. World Academy of Science, Engineering and Technology.
33. Wang, H., *An analytical study of bond strength associated with splitting of concrete cover*. Engineering Structures, 2009. **31**(4): p. 968-975.
34. Chan, H., Y. Cheung, and Y. Huang, *Crack analysis of reinforced concrete tension members*. Journal of Structural Engineering, 1992. **118**(8): p. 2118-2132.
35. Vogel, H.M., *Serviceability of concrete beams reinforced with FRP and concrete prisms prestressed with CFRP*. University of Manitoba (Canada), 2011. **2011.NR78922**. .
36. Sokolovas, A., et al., *Tension-Stiffening Model Based on Test Data of RC Beams*. Modern building materials, structures and techniques, 2010. Vilnius, Lithuania, The 10th international Conference: p. 5., 2010.
37. Wenkenbach, I., *Tension Stiffening in Reinforced Concrete Members with Large Diameter Reinforcement*. Durham University, undergraduate thesis. Available at Durham E-Theses Online:< <http://etheses.dur.ac.uk/3250/>>[Accessed 15/08/2012], 2011.
38. Behfarnia, K., *The effect of tension stiffening on the behaviour of R/C beams*. Asian Journal of Civil Engineering (Building and Housing), 2009. **10**(3): p. 243-255.
39. Vakhshouri, B. and S. Nejadi, *Time-dependent bond transfer length under pure tension in one way slabs*. Structural Engineering and Mechanics, 2016. **60**(2).
40. Ian Gilbert, R., *Tension Stiffening in Lightly Reinforced Concrete Slabs*. Journal of Structural Engineering, 2007. **133**(6): p. 899-903.
41. Vakhshouri, B. and S. Nejadi, *Experimentla study of time-dependant bond-transfer length under pure tension in slabs*. IOS press, Construction materilas and structures, proceeding of the 1st int. conferenec on construction materials and structures 2014: p. 1044-1051.
42. Kaklauskas, G. and J. Ghaboussi, *Stress-strain relations for cracked tensile concrete from RC beam tests*. Journal of Structural Engineering, 2001. **127**(1): p. 64-73.

43. Gilbert, R.I., *Closure to "Tension Stiffening in Lightly Reinforced Concrete Slabs" by R. Ian Gilbert*. Journal of structural engineering, 2008. **134**(7): p. 1264-1265.
44. Bond, A., et al., *How to design concrete structures using Eurocode 2*. 2006: Concrete Centre.
45. Kwak, H.-G. and F.C. Filippou, *Finite element analysis of reinforced concrete structures under monotonic loads*. 1990: Department of Civil Engineering, University of California Berkeley, CA, USA.
46. Mihai, P., I. Hirhui, and B. Rosca, *Reinforcement using the Finite Element Method*. Journal of Applied Sciences, 2010. **10**(9): p. 738-744.
47. Vakhshouri, B. and S. Nejadi. *Experimental study of time dependent bond transfer length under pure tension in slabs*. in *International Conference on Construction Materials and Structures*. 2014. IOS Press BV.
48. Nejadi, S. and R.I. Gilbert, *Shrinkage cracking and crack control in restrained reinforced concrete members*. ACI Structural Journal, 2004. **101**(6).
49. AS-1012.13, *Methods of testing concrete, method 13-determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory*. Australian standard, 2014.
50. Wu, H.Q. and R.I. Gilbert, *Modeling short-term tension stiffening in reinforced concrete prisms using a continuum-based finite element model*. Engineering Structures, 2009. **31**(10): p. 2380-2391.
51. Gilbert, R.I., *Shrinkage cracking in fully restrained concrete members*. ACI Structural Journal, 1992. **89**(2).
52. Faver, R., M. Koprna, and A. Radojicic, *Effects differes, fissuration et deformations des structures en beton*. Manual du Euro-International Du beton, Ecole Polytechnique Federale de Lausanne, Editions Georgi, Saint-Saphorin, Suisse, 1983.
53. Bamonte, P. and P. Gambarova, *High-bond bars in NSC and HPC: study on size effect and on the local bond stress-slip law*. Journal of Structural Engineering, 2007. **133**(2): p. 225-234.
54. Beeby, A., R. Scott, and A. Jones, *Revised code provisions for long-term deflection calculations*. Proceedings of the ICE-Structures and Buildings, 2005. **158**(1): p. 71-75.
55. Malvar, L., J. Cox, and K.B. Cochran, *Bond between carbon fiber reinforced polymer bars and concrete. I: Experimental study*. Journal of composites for construction, 2003. **7**(2): p. 154-163.
56. Mitchell, D.W. and H. Marzouk, *Bond characteristics of high-strength lightweight concrete*. ACI Structural Journal, 2007. **104**(1): p. 22.
57. Alavi-Fard, M. and H. Marzouk, *Bond of high-strength concrete under monotonic pull-out loading*. Magazine of Concrete Research, 2004. **56**(9): p. 545-557.
58. Gupta, A.K. and S.R. Maestrini, *Tension-stiffness model for reinforced concrete bars*. Journal of Structural Engineering, 1990. **116**(3): p. 769-790.
59. Kankam, C.K., *Relationship of bond stress, steel stress, and slip in reinforced concrete*. Journal of Structural Engineering, 1997. **123**(1): p. 79-85.
60. Mirza, S.M. and J. Houde. *Study of bond stress-slip relationships in reinforced concrete*. in *ACI Journal Proceedings*. 1979. ACI.
61. Rehm, G., *On the fundamentals of steel-concrete bond*. Deutscher Ausschuss für Stahlbeton, 1961. **138**: p. 1-59.
62. Nilson, A.H. *Nonlinear analysis of reinforced concrete by the finite element method*. in *ACI Journal Proceedings*. 1968. ACI.
63. Martin, H., *On the interrelation among surface roughness, bond and bar stiffness in the reinforcement subject to short-term loading*. Deutscher Ausschuss Stahlbeton, 1973. **228**: p. 1-50.
64. Eligehausen, R., E.P. Popov, and V.V. Bertero, *Local bond stress-slip relationships of deformed bars under generalized excitations*. 1982.

65. Huang, Z., B. Engström, and J. Magnusson, *Experimental investigation of the bond and anchorage behaviour of deformed bars in high strength concrete*. Goteborg: Division of Concrete Structures, Chalmers University of Technology, 1996.
66. Harajli, M., M. Hout, and W. Jalkh, *Local bond stress-slip behavior of reinforcing bars embedded in plain and fiber concrete*. ACI Materials Journal, 1995. **92**(4).
67. Murcia-Delso, J., A. Stavridis, and B. Shing, *MODELING THE BOND-SLIP BEHAVIOR OF CONFINED LARGE-DIAMETER REINFORCING BARS*. COMPDYN 2011 III ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.) Corfu, Greece, 25–28 May, 2011.
68. Barbosa, M., *Evaluation of the behavior of the bond in ordinary and high strength concrete*. 2001, Doctoral Thesis, COPPE/UFRJ (in Portuguese).
69. Oh, B.H. and S.H. Kim, *Realistic models for local bond stress-slip of reinforced concrete under repeated loading*. Journal of Structural Engineering, 2007. **133**(2): p. 216-224.
70. de Almeida Filho, F.M., K. Mounir, and A.L.H. El Debs, *Bond-slip behavior of self-compacting concrete and vibrated concrete using pull-out and beam tests*. Materials and Structures, 2008. **41**(6): p. 1073-1089.

CHAPTER 5

MECHANISM OF FLEXURAL DEFLECTION IN REINFORCED CONCRETE STRUCTURES

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Chapter 5: Mechanism of flexural deflection in reinforced concrete structures

5.1. INTRODUCTION

Deflection is a common concern of almost all design codes and technical drafts in regard to the time-dependent behaviour of concrete. Short-term and long-term deflections are considered in serviceability limit states. Due to the nonlinear nature and complicated behaviour of concrete in the presence of reinforcement, deflection prediction is full of unknown parameters. For accurate prediction of the deflection, comprehensive information on the time-dependent and nonlinear material behaviour is required.

There are considerable differences between the predicted values of deflection and the experimental values in the Reinforced Concrete (RC) members subjected to service loads, particularly in long-term deflection of concrete structures. Existing codes of practice give an overall estimation of the long-term deflection as a multiple of the short-term or elastic deflection of concrete. Such estimation sometimes may lead to an entirely wrong prediction of deflection and consequently, unsafe or non-economical concrete construction.

The majority of the equations in codes of practice are applied to predict the time-dependent behaviour of the conventional concrete. However, there is also some relevance to other types of concrete such as Light-Weight Concrete (LWC). The modern design codes have approved the limit states method in structural design, in which different limit states or the design requirements such as adequate strength and serviceability which need to be fulfilled, simultaneously. The required minimum performance limits are specified for each limit state and each one may become critical of the design of a particular member [1].

Table (5.1) explains the descriptions and major classifications of the limit state design in various codes of practice.

Table 5.1: Limit state definition and major classifications in codes of practice

Code	Limit states description
ACI 209.2R-08 [2]	To predict the strength and serviceability of reinforced and prestressed concrete structures, the structural engineer requires an appropriate description of the mechanical properties of the materials, including the prediction of the time-dependent strains of the hardened concrete. The prediction of shrinkage and creep is important to assess the risk of concrete cracking, and deflections due to stripping-reshearing.
BS-5400-4 (1990) [3]	Serviceability limit state: The design should be such that the structure will not suffer local damage which would shorten its intended life or incur excessive maintenance costs. In particular, calculated crack widths should not exceed those permitted in 4.1.1.1 - <i>Cracking.</i> Cracking of concrete should not adversely affect the appearance or durability of the structure. - <i>Stress limitations.</i> To prevent unacceptable deformations occurring, compressive stresses in <i>concrete</i> and stresses in steel should be calculated by linear elastic analysis - <i>Deflections.</i> The deflection of the structure, or any part of it, should not be such as to affect adversely the appearance or efficiency of the structure. Ultimate limit states: The strength of the structure should be sufficient to withstand the design loads, taking due account of the possibility of overturning or buckling. The assessment should ensure that collapse will not occur as a result of rupture of one or more critical sections, by overturning or by buckling caused by elastic or plastic instability, having due regard to the effects of sway when appropriate. - <i>Rupture or instability.</i> The assessment of the structure under the design loads appropriate to this limit state should ensure that prior collapse does not occur as a result of rupture of one or more critical sections, buckling caused by elastic or plastic instability, or overturning. The effects of creep and shrinkage of concrete, temperature difference and differential settlement need not be considered at the ultimate limit state provided that these effects have been included in the appropriate load combinations to check the stress limitations given in 4.1.1.3 for the serviceability limit state.
CEB-FIP-90 [4]	Ultimate Limit State Attainment of the bearing capacity of a structural part or the structure as a whole is classified as ultimate limit state. The ultimate limit states considered are distinguished by the consequences of their exceedance. The loss of static equilibrium (Section 1.6.5) when a structural part or the whole structure (considered as rigid bodies) is overturned, is lifted or slides. Exceeding the resistance of one or more critical regions of the structure. Serviceability limit states: The serviceability limit states associated with the general requirements refer to: limited local structural damage such as excessive cracking or excessive compressive stresses, producing irreversible strains and micro cracks; Deformations which produce unacceptable damage in non-structural elements Vibrations resulting in discomfort, alarm or loss of Utility
CSA-A23.3-04 [5]	Design Limit states: Conditions of a structure at which it ceases to fulfil the designed functions. 1- Durability: Concrete structures shall satisfy the durability requirements of CSA A23.1, CSA A23.4, or CAN/CSA-S413, as applicable, for the intended use and exposure conditions. 2- Fire resistance: Concrete structures shall satisfy the fire resistance requirements of the applicable building code. 3- Ultimate limit states: Structures, structural members, and connections shall be designed such that factored resistance is greater than or equal to the effect of factored loads, with the effect of factored loads

Table 5.1: Limit state definitions and major classification in codes of practice (continued)

Code	Limit states description
CSA-A23.3-04 [5]	with the effect of factored loads being determined as specified in Clauses 8.2 and 8.3 and the factored resistance being determined as specified in Clause 8.4. 4- Serviceability limit states 4.1 Deflections: Structures and structural members shall be designed to satisfy the deflection control requirements of Clauses 9.8 and 13.2.7, with loadings as specified in Clause 8.3.3. 4.2 Local damage and cracking: Structural members and connections shall be designed to meet the minimum reinforcement area and maximum reinforcement spacing requirements of this Standard as well as the requirements of Clauses 10.6 and 18.1 to 18.4, with loadings as specified in Clause 8.3.3. Note: <i>This Standard does not specifically limit crack widths.</i> 4.3 Vibrations: In the design of structures and structural members, consideration shall be given to controlling vibrations within acceptable limits for the intended use. 5- Structural integrity: Consideration shall be given to the robustness of the overall structural system to minimize the likelihood of a progressive type of collapse.
EN 1992-1-1:2004 (E) [6]	SERVICEABILITY LIMIT STATES (SLS) - stress limitation (section 7.2): The compressive stress in the concrete shall be limited in order to avoid longitudinal cracks, micro-cracks or high levels of creep, where they could result in unacceptable effects on the function of the structure. - Crack control (section 7.3): Cracking shall be limited to an extent that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable. - Deflection control: Deformations should not exceed those that can be accommodated by other connected elements such as partitions, glazing, cladding, services or finishes. In some cases limitation may be required to ensure the proper functioning of machinery or apparatus supported by the structure, or to avoid ponding on flat roofs.
JSCE (2007) [7]	Limit state design: Safety limit state: Cross sectional failure, fatigue failure, displacement, deformation, mechanisms Serviceability limit state: Appearances, vibration, comfort of vehicle ride, etc. Water tightening, resistance against damage (maintenance of functionality)
IS 456 : (2000) [8]	The acceptable limit for the safety and serviceability requirements before failure occurs is called a 'limit state'. Limit State of Collapse: rupture, buckling due to elastic or plastic instability (including the effects of sway where appropriate) or overturning. The resistance to bending, shear, torsion and axial loads at every section Limit States of Serviceability: <i>Deflection, Cracking</i> Other Limit States
AS-3600-09 [9]	Limit state design: Ultimate limit state Serviceability limit state: Deflection, cracking, Vibration
AS/NZS 1170.0(2002) [10]	Limit state design : Serviceability limit states: deflection , cracking Ultimate limit states: For Australia as AS-3600 and For New Zealand: as described in Table 3-3

5.2. DEFLECTION CONTROL IN SERVICEABILITY DESIGN

According to limit state definitions in codes of practice and literature, serviceability limit states can be classified into three categories as described in the following terms:

1. Motion perception (vibration and acceleration): common samples include human discomfort caused by machinery or the wind, particularly if resonance occurs. Floor vibrations from people or machinery and acceleration, particularly in tall buildings under wind load are usual areas of concern in this category;
2. Deterioration: Included are such items as corrosion, weathering, efflorescence, discoloration, rotting, and fatigue;
3. Deformation (deflection, curvature and cracking): Common examples include local damage to non-structural elements (e.g., ceilings, cladding, and partitions) due to deflections under dead, live, wind, or seismic loads, and damage from temperature change, moisture, shrinkage, or creep effects.

The effects of the excessive deflection on performance of the reinforced concrete structures range from a compromised performance of the structure to unsightly crack openings within the roofs and walls. Excessive deflection may cause damage to adjacent parts of the building (cracking and spalling of brittle partitions) and non-load bearing walls that affect a building's appearance (visible sag of floors and ceilings, visible leaning of walls and columns). Also building usage and function may be influenced by the curvature of floors, and inclinations may cause problems such as tilting of furniture, movement of trolleys and slipping. [11].

Time-dependent deformations of concrete members depend on many factors including the properties of concrete, the geometry of the member and ambient conditions such as temperature and relative humidity. The deflections due to these time-dependent effects can be significant, especially for cracked beams. Cracking has been commonly observed in RC beams and slabs in service. In cases where deflection control is critical, it is important to predict the long-term and time-dependent deflections accurately.

As a general conclusion, deflection of RC flexural member increases with time, mainly due to:

- a) Differential shrinkage (causing differential strains across the cross-section, resulting in curvature);
- b) Creep under sustained loading.

Other effective parameters of the deflection of RC structures have been described comprehensively in Chapter Four of this study.

5.3. RATIO OF LONG-TERM TO SHORT-TERM DEFLECTION

There is no guidance as to when the short-term deflection ends and long-term deflection begins. The methods of predicting or calculating the deflections are deterministic in nature. Even, with the most sophisticated methods of analysis using experimentally determined material properties, the range of variation between the measured and computed results for the instantaneous (short-term) as well as long-term deflection are high. Branson (1977) [12] showed that the coefficient of variation for such deflections is of the order of 15% to 20% and higher. Jokinen and Scanlon (1987) [13] analysed experimental data from several identical slabs in a multi-story building. They reported a coefficient of variation between the calculated and measured deflections in the range of 30% for both short-term and long-term deflections.

Generally, to determine the long-term deflection of a concrete member, the time-delayed effects of shrinkage, creep and the resulting reduction in tension stiffening need to be considered accurately. Additional factors which can contribute to the increased long-term deflection include the development of new cracks, widening of the initial cracks, and effects of the repeated load cycles [14].

Shrinkage induced strain that develops with time causes additional curvature known as shrinkage warping. Deflections also increase due to the effects of creep in the concrete member. The tension stiffening effect is responsible for a significant proportion of the concrete member stiffness. It decreases over time due to the effects of creep and shrinkage strains [15]. Therefore, the RC member that is essentially uncracked at the stripping time will gradually become more cracked; the effective moment of inertia will decrease, and this will contribute to the long-term deflection.

Both compressive and tensile concrete creep under an effective constant sustained load also contribute to the rate of decreasing the long-term deflection with increasing time for several years after casting. Taylor and Heiman (1977) [16] showed that the deflection of slabs and beams increases with age for five to nine years after stripping. The decreasing rate of time-dependent deflection in their experiment is given in Figure (5.1).

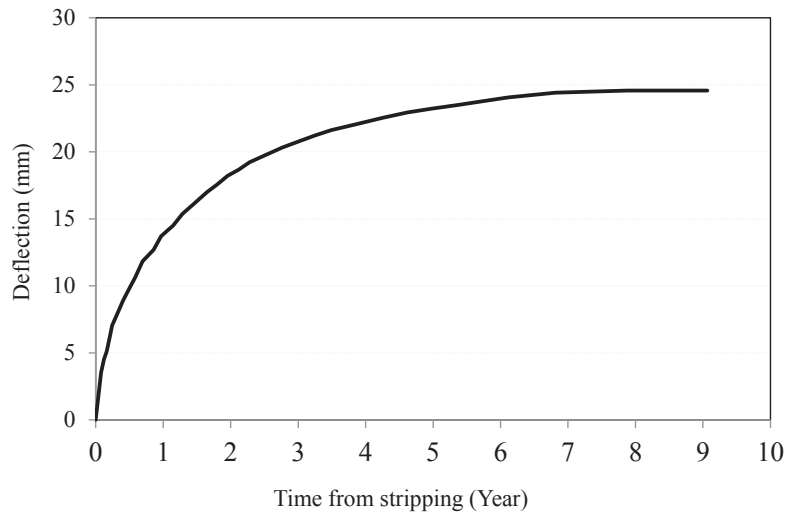


Figure 5.1: Long-term deflection of actual structure after stripping [16]

The coefficient of creep and shrinkage (k_{cs}) in AS-3600 (2009) acts as the multiplier of the instantaneous deflection to calculate the total deflection after a long time. However, using a single multiplier to predict the time-dependent deformations of concrete members cannot cover all uncertainties. Gilbert (2001) also noted that the method of multiplying the ‘load induced short-term deflection’ by a multiplier, especially in slabs is fundamentally wrong [17] because the shrinkage warping occurs irrespective of the load applied and even occurs in unloaded concrete slabs and contributes significantly to long-term deflections [15].

The uncertainties may arise from changing the material properties, the geometry of the member, and the ambient conditions such as temperature and relative humidity cannot yield consistent and accurate values for the calculated and long-term deflection ([17],[18],[19], [20]). In this regard, Gilbert (2007) [20] criticised the use of creep and shrinkage factor in AS-3600 as a ‘ball-park estimate’, that does not take into account the shrinkage and creep properties of the concrete, the load history, the age at first loading and environmental effects, hence often failing to adequately predict the long-term deflection.

5.4. REAL-SCALE TIME-DEPENDENT DEFLECTION MONITORING IN LITERATURE

There are limited numbers of reported in-situ monitoring of deflection behaviour of RC structures under service loads. Table (5.2) illustrates some case studies in which the deflection has been measured in the actual conditions, real-scale elements in the real conditions of temperature, environment effects, changing the loading level and every possible situation during and after the construction. The type of structure and monitored element, loading duration and a brief comparison with predictions of codes of practice and laboratory investigations are given in each case.

Table 5.2: Real-scaled monitoring of time-dependent deflection of reinforced concrete structures

Case study	Specimen type	Duration	Conclusions
Heiman (1974) [21]	1-Two story commercial building-suspended floor of beams and slabs, flat plate roof 2- Three story car park- flat slab 3-Fifty story circular building- radial tapered beam floor 4-Four story motel building- flat slab floor Sydney, Australia	-13 days -56 days - 25to 32 days -28days	ACI Code method underestimates the long-term deflections of flexural members. In shallow members with little or no compressive reinforcement in the mid-span region, difference between real and predicted values increases and separate calculations of long-term deflection due to creep and shrinkage give more accurate results. Existing codes don't consider the heavy construction loads that strongly affect the long-term deflection. Lab tests-based calculations may not be sufficient to predict the real behaviour of RC elements.
Kaminetzky and Stivaros (1992) [22]	Two low-rise (3 story) reinforced concrete buildings slabs have been overloaded during construction, USA	10 months	ACI-318 does not adequately account for the early-age high construction loads and the time-dependent cracking. Most of the estimated deflections for the various slab areas do not compare well with the measured values.
Vollum et al. (2002) [23], European Concrete Building projects, (2004) [24]	St George Wharf case study - slab deflection, Cardington, UK	5 months	Reliance on span/ depth ratio may be insufficient. Longer spans and more slender flat slab structures are more important regarding the serviceability limit states of deflection and vibration.
French et al. (2010) [25]	Precast Composite Slab Span System (PCSSS), implemented for short to moderate span bridges (6-15 m. range). Minnesota-USA	24 months	Cracking and strain distribution are more uniform in lab conditions.
Hirsch et al. (2012) [26]	1- West Washington Street building (No.215) 2-Fifty story building- 2way concrete flat slab 3-Aqua high-rise project, 82 story Chicago , USA	35 days	Considering a cracked slab as uncracked is a significant danger. If the factors that contribute to cracking (load history, column/middle strip effects, restrained shrinkage, shrinkage warping) cannot be included directly into the calculation, conservative adjustments need to be made to the calculations to ensure that cracking that occurs is detected. Caution should be exercised in span to depth ratios for preliminary design of members with unusually high loads or irregular load histories.

5.5. EXPERIMENTAL MONITORING OF DEFLECTIONS IN LABORATORY CONDITIONS FROM LITERATURE

In the real-scaled case studies of the real structures presented in Table (5.2), the recording period of deflection and time-dependent parameters are too short and could not be extended to the whole service life of the RC structure. They monitor just a specified duration of the service life, while in the RC structures, due to the nonlinear and complex behaviour of composite materials, the curing, construction and loading conditions before the monitoring time strongly affect the time-dependent behaviour. These effects may sometimes significantly change the predictions based on the laboratory investigations and the relationships developed in design codes.

5.5.1. Experiment by Uy and Bradford (1994, 1997)

Uy and Bradford (1994) [27] and Uy (1997) [28] conducted experiments to investigate the time-dependent behaviour of simply supported profiled composite beams for 450 days. The results were compared with the predictions of BS-5950 [29] to verification purposes.

The profiled composite slab in a steel framed building and a profiled composite beam and slab in a concrete framed building are shown in Figure (5.2). The flexural test set-up is presented in Figure (5.3).

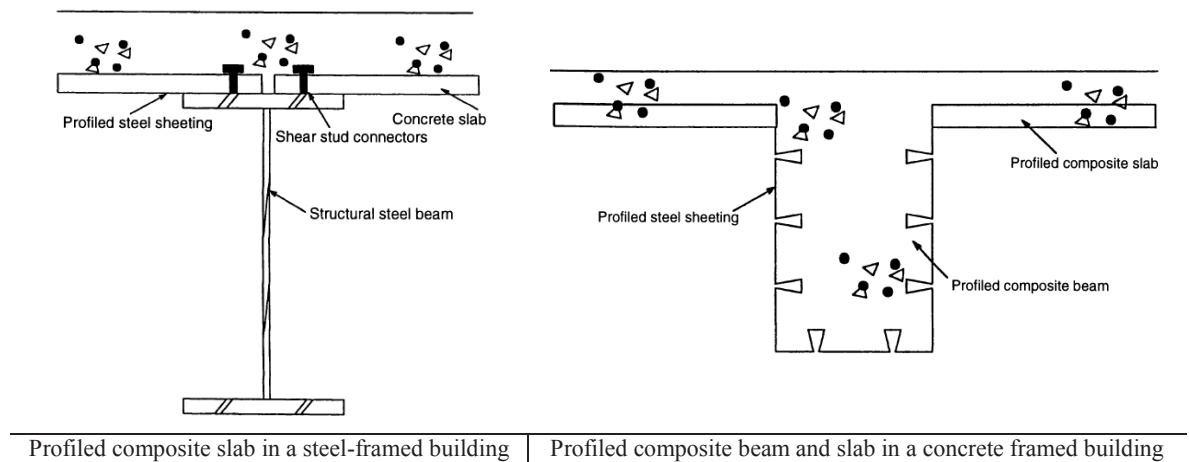


Figure 5.2: Profiled composite slab and beam [27]

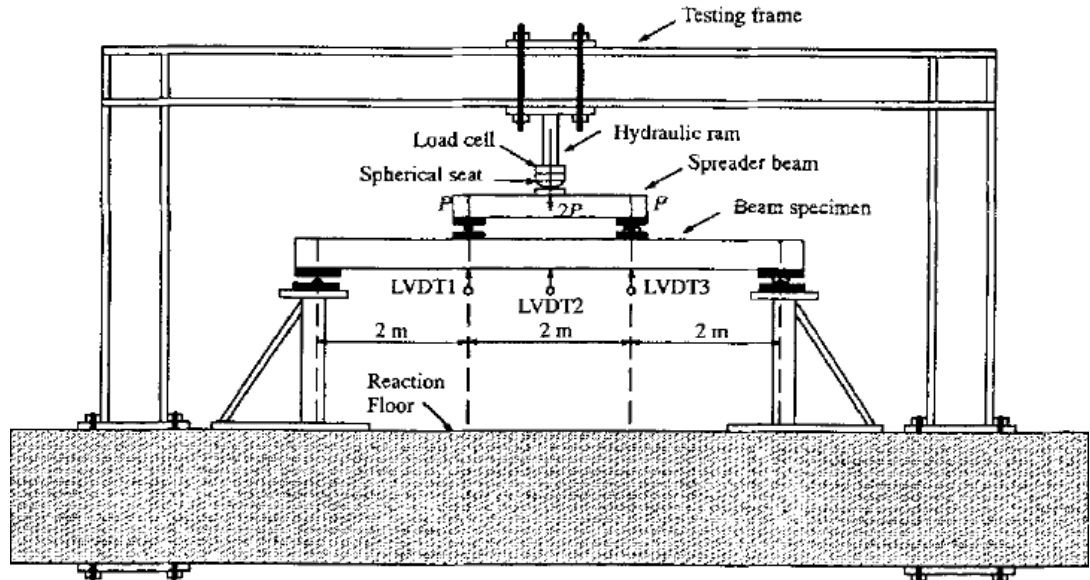


Figure 5.3: Test set-up for long-term flexural deflection of beams and slabs [27]

Figure (5.4) shows the long-term deflection of beams over 450 days. The beams R_1 and P_1 were subjected to self-weight only to study the effect of shrinkage deformations on the behaviour. The beams R_2 and P_2 were subjected to self-weight with an additional 5 kN/m sustained load to study the effect of shrinkage and creep deformations.

According to Figure (5.4), the long-term deflection after 450 days is about 3 to 4 times greater than the instantaneous deflection in all the specimens. The increasing trend of deflection in the beams subjected to self-weight and additional sustained load are slightly increasing over time especially after 100 days of first loading. In other words, the distance between the pair of curves is increasing by time. It is due to the increasing effect of creep strain under the higher sustained load in the beams subjected to higher loading level.

Other conclusions from their study are described as following:

- The span to depth ratio is not adequate to limit the long-term deflection of composite beams and slabs;
- The beneficial effect of lateral restraints of supporting beams on the composite slab stiffness and long-term behaviour are ignored in design of these types of structures.

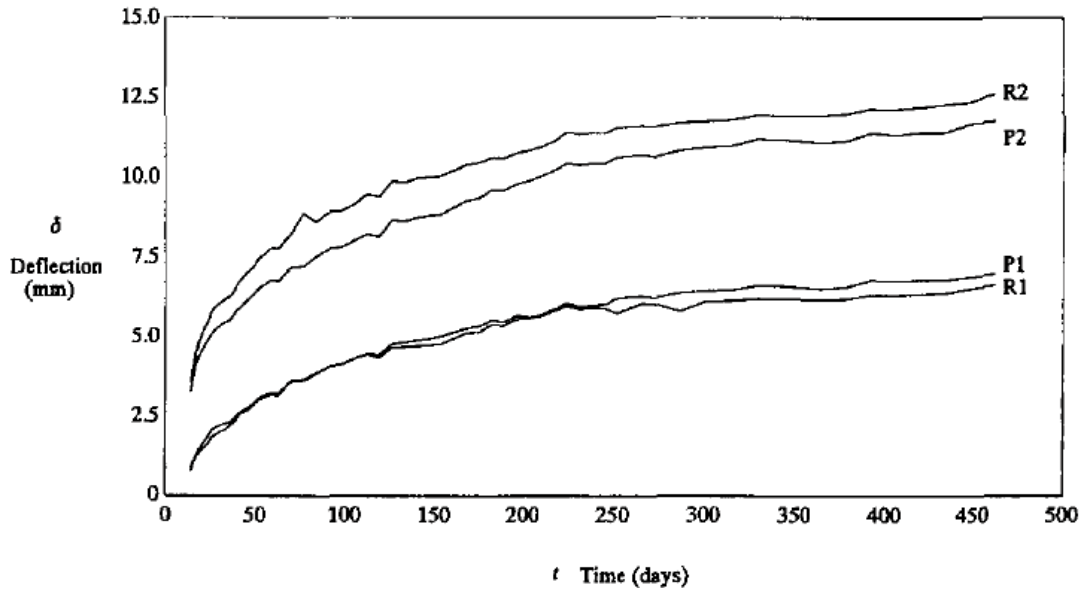


Figure 5.4: Long-term deflections of beams at mid-span [27]

5.5.2. Experiment by Gilbert and Guo (2005)

Gilbert and Guo (2005) [30] performed a three-year experimental program by testing of 7 specimens of large-scale reinforced concrete flat slabs. Overall plan dimensions of the slabs were 6200 mm by 7200 mm continuous over two spans in each direction supported by columns. Applied different environmental conditions and loading history to the specimens were as illustrated in Figure (5.5).

Deflections of slabs were recorded at different ages over the 750 days testing period. The ratio of deflection in different ages to instantaneous deflection after loading is presented in Figure (5.6).

Generally, the long-term to short-term (instantaneous) deflection ratio may be as large as two to three [14], according to the design codes. However, the ratios in this experimental program vary in the range of 5 to 9 at the end of the test, or at the age just before the loading blocks were removed. The ratios shown in Figure (5.6) are recorded under different conditions of loading histories and wetting and drying periods. These high ratios in various slabs depend on the loading history and various curing conditions.

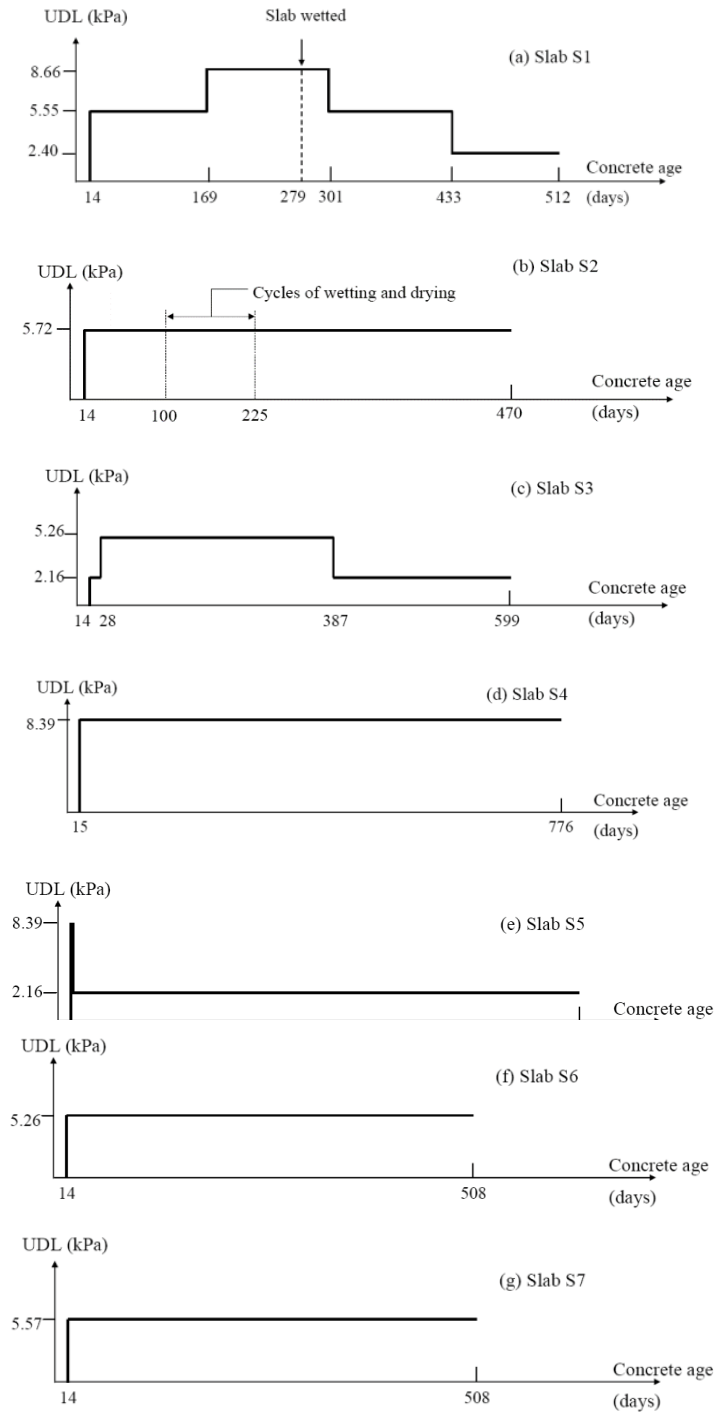


Figure 5.5: Loading history of slab specimens [30]

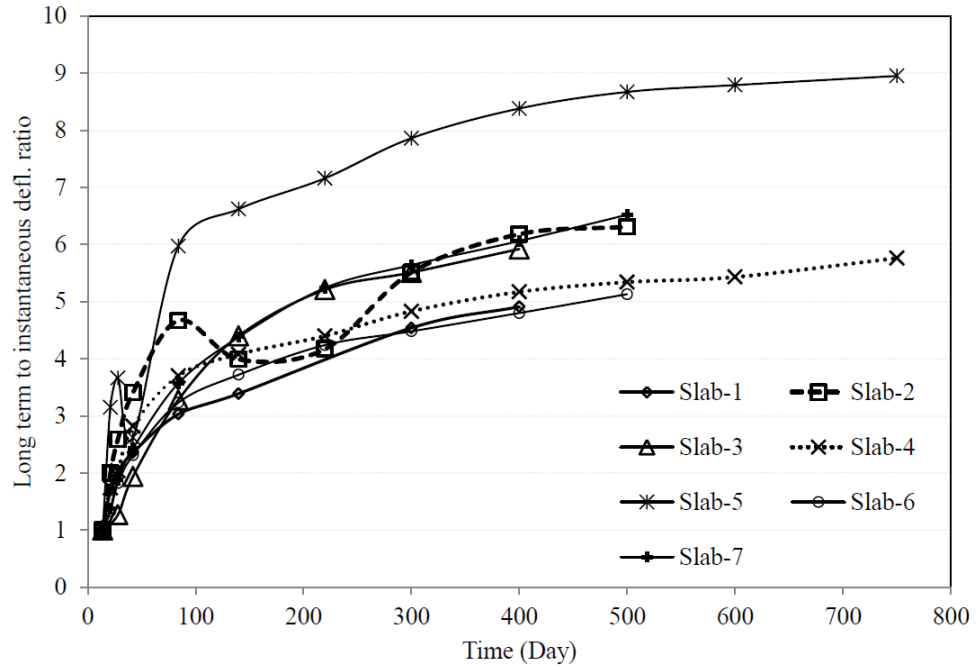


Figure 5.6: Ratio of long-term to instantaneous deflection in concrete slabs [30]

Gilbert and Guo (2005) also investigated the effect of wetting and drying cycles on the deflection behaviour of slabs. They found that the immediate effect of wetting after a long period of drying is comparable to the effect of the full-service load and in-service behaviour of the slabs which are very sensitive to the degree of exposure (15% reduction of the ratio in slab S2 after 3 months). The duration and magnitude of warping due to drying shrinkage is directly proportional to the free drying shrinkage of the concrete. It continues at a decreasing rate with increasing time for several years after casting and is independent of loading [31]. For both slabs S1 and S2, within 24 hours after the top surface was thoroughly wetted, the deflection was reduced significantly due to the redistribution of internal actions.

According to Figure (5.6), deflection in all slabs was rapidly extended in first ages and continued with slow growth without stopping even after 750 days. This conclusion confirms the test results obtained by Taylor and Heiman (1977) [16] previously.

5.5.3. Experiment by Park et al. (2012)

Park et al. (2012) [32] investigate the instantaneous and long-term deflections of RC slabs subjected to construction load at their early ages. The concrete was cured at low-temperature and the loading was sustained for 291 days after initial loading. The test program included seven one-way slabs and one two-way slab.

Dimensions of the simply supported one-way slab were 4500 mm length $\times$ 800 mm width $\times$ 160 mm depth. Two concentrated loads were applied at the mid-span. The shape of two-way slab was cruciform, which was vertically supported at four ends. Figure (5.7) shows the section and dimensions of the specimens and the testing set-up view.

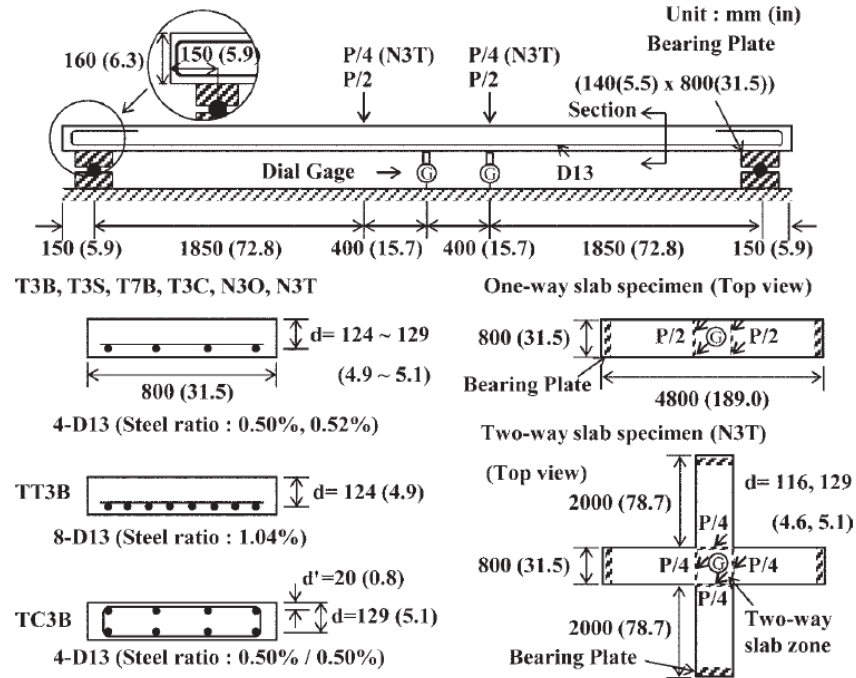


Figure 5.7: Specimens dimension and test set-up for long-term flexural deflection [32]

They studied the effect of changing load value during the test on the deflection changes in the specimens. Figure (5.8) shows the loading history of the specimens for one-step and multi-step loading cases.

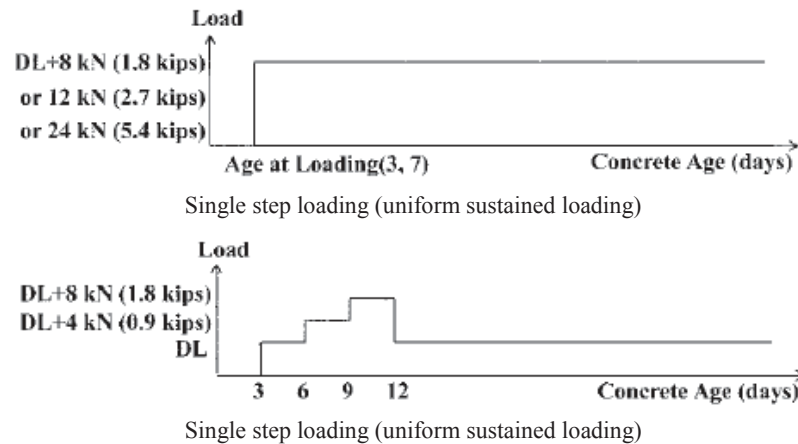


Figure 5.8: Loading history of the specimens [32]

The following items were concluded from their study:

- The lower curing temperature strongly reduces the compressive strength and modulus of elasticity of concrete by 30%;
- The early age loading at three days increases the instantaneous deflection by 31%;
- The ratio of long-term deflection and total deflection to the instantaneous deflection after 110 days were in the range of 1 to 2 and 2 to 3 in different slabs respectively. In other words, the time-dependent deflection multiplier was affected by loading history, curing condition and initial loading age. These ratios also are being affected by the ratio of the applied service moment to the cracking moment (M_a / M_{cr}).

5.6. MINIMUM THICKNESS PROVISIONS

There are different simplified methods in design codes to control the deflection and to ensure the serviceability of flexural RC elements. Limiting the minimum thickness is one of these simple methods of deflection control. This limit is just a general estimation of the deflection and is not able to guarantee the expected behaviour under different conditions of loading, material technology and environmental effects. Table (5.3) summarize the limitations proposed in various codes of practice to control the deflection of RC elements. The limitation suggested by Lee and Scanlon (2010) [33] is presented in the Table (5.3) also.

Table 5.3: Minimum thickness provisions for slabs to control the time-dependent deflection

Code	Minimum thickness of non-prestressed one-way slabs unless deflections are calculated			
ACI 318-08 [34]	Simply supported	One end continuous	Both ends continuous	Cantilever
	$l/20$	$l/24$	$l/28$	$l/10$
	Minimum thickness of two-way slabs without interior beams			
	Without drop panels			
	f_y, psi	Exterior panels		Interior panels
		Without edge beams	With edge beams	-
	60,000	$l_n/30$	$l_n/33$	$l_n/33$
	l is span length of one-way slab , l_n is length of clear span measured face-to-face of supports			
	For slabs with beams spanning between supports on all sides			
	$\alpha_{fm} \leq 0.2$	Slabs without drop panels - 5 in. and Slabs with drop panels - 4 in.		
	$0.2 \leq \alpha_{fm} \leq 2$	$h = \frac{l_n(0.8 + \frac{f_y}{200000})}{36 + 5\beta(\alpha_{fm} - 0.2)}$ Not less than 5 in.		
	$\alpha_{fm} > 2$	$h = \frac{l_n(0.8 + \frac{f_y}{200000})}{36 + 9\beta}$ Not less than 3.5 in.		
α_{fm} is average value of αf for all beams on edges of panel αf is ratio of flexural stiffness of beam section to flexural stiffness of width of slab				
AS 3600-09 [9]	$\frac{L_{ef}}{d}$ $= k_3 k_4 \left[\frac{(\Delta/L_{ef}) E_c}{F_{d,ef}} \right]^{1/3}$	L_{ef} is effective span, taken as less of $(l_n + d)$ and l ; d is effective depth of cross section; Δ/L_{ef} is deflection limit; $F_{d,ef}$ is effective design load, per unit area; $k_3 = 1.0$ for one-way slab, rectangular slabs supported on four sides $= 0.95$ for two-way flat slab without drop panels; k_4 is deflection constant which may be taken as: (a) for simply supported slabs, 1.6; or (b) for continuous slabs, where in adjoining spans ratio of longer span to shorter span does not exceed 1.2 where no end span is longer than an interior span-(i) 2.0 in an end span; or (ii) 2.4 in interior spans; and (c) for edge-supported slabs k_4 varies from 1.7 to 2.0 (Table 9.3.4.2 in AS 3600-2001).		
BS 8110-1-1997 [35]	Basic span/effective depth ratios for rectangular section		Modification factor for tension reinforcement :	$= 0.55 + \frac{(477 - f_s)}{120(0.9 + \frac{M_u}{bd^2})}$ $\leq 2.0, \quad f_s = 2/3f_y$
	Support conditions	Rectangular section	M_u is design ultimate moment at centre of span or, for cantilever, at support; f_s is estimated design service stress in tension reinforcement; f_y is yield strength of reinforcement; b is effective width of rectangular beam; d is effective depth; For spans exceeding 10 m, Table 3.9 should be multiplied by 10/span, except for cantilevers where the design should be justified by calculation. For flat plate, span/effective depth ratio should be multiplied by 0.9.	
	Cantilever	7		
	Simply supported	20		
	Continuous	26		

Table 5.3: Minimum thickness provisions for slabs to control the time-dependent deflection (continued)

Code	Minimum thickness of non-prestressed one-way slabs unless deflections are calculated		
Eurocode 2-04 [6]	If $\rho \leq \rho_o$	$\frac{l}{d} = K \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_o}{\rho} + 32\sqrt{f_{ck}} \left(\frac{\rho_o}{\rho} - 1 \right)^{3/2} \right]$	K is factor to take into account different structural systems: (a) simply supported 1.0; (b) one end continuous 1.3; and (c) both end continuous 1.5; ρ_o is reference reinforcement ratio, $= \sqrt{f_{ck}}^{10^{-3}}$; ρ is required tension reinforcement ratio at mid-span to resist moment due to design loads (at support for cantilevers); ρ' is required compression reinforcement ratio at mid-span to resist moment due to design loads (at support for cantilevers); f_{ck} is specified compressive strength of concrete, in MPa; and l is span length.
	If $\rho > \rho_o$	$\frac{l}{d} = K \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_o}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}} \sqrt{\frac{\rho'}{\rho_o}} \right]$	
Lee and Scanlon (2010) [33]	$\frac{l_n}{h} = \beta \left[\left(\frac{\Delta_{inc}}{l} \right)_{allow} \frac{2400K_{DP}E_c(b/12)}{\kappa k_{AR}k_{SS}(\lambda W_s + W_L(add))} \right]^{1/3}$ (U.S. Customary Units)		W_s is sustained load (psf [slabs]; plf [beams]); (Pa [slabs]; N/m [beams]); $W_L(add)$ is additional live load (psf [slabs]; plf [beams]); (Pa[slabs]; N/m [beams]); $\beta = 1$, except β is long span/short span ≤ 2.0 for edge-supported slabs; κ is deflection coefficient depending on support condition: equals 5 for simply supported, 1.4 for both ends continuous, 2 for one end continuous, and 48 for fixed end cantilever; $k_{DP} = 1$, except $k_{DP} = 1.35$ for slab with drop panels; $k_{SS} = 1$, except $k_{SS} = 1.35$ for column supported two-way slab systems; $k_{AR} = 1$, except $k_{AR} = 0.2 + 0.4\beta$ for edge-supported slabs; $b = 12$ in. (1000 mm for SI) for one- and two-way slabs = beam width (= web width, b_w for T-beams) (in. for U.S. Customary Units and mm for SI Units); $(\Delta_{inc})_{allow}$ is required incremental deflection limit; and λ is long-time multiplier for sustained loads (ACI 318, Section 9.5.2.5).
	$\frac{l_n}{h} = \beta \left[\left(\frac{\Delta_{inc}}{l} \right)_{allow} \frac{0.0167K_{DP}E_cb}{\kappa k_{AR}k_{SS}(\lambda W_s + W_L(add))} \right]^{1/3}$ (SI Units)		

5.7. EFFECTIVE PARAMETERS IN INSTANTANEOUS DEFLECTION

The principle factors which affect the initial or short-term deflection of RC flexural members under service loads are the modulus of elasticity, load distribution and support conditions. Other

parameters are variable cross-section and hence, variable moment of inertia, load level and the degree of cracking along the member.

Cracking and tension-stiffening parameters probably have the most significant effect on numerical results of concrete members subjected to short-term loading. The free shrinkage strain of concrete may be of a magnitude well-exceeding the cracking strain, even at first loading. The shrinkage might significantly affect the cracking resistance and deformations of RC elements subjected to short-term loading. Bischoff (1983), Gilbert (2001), Kaklauskas and Gribniak (2005), Gribniak et al. (2008), Gribniak (2009) and Kaklauskas et al. (2009) discussed the necessity to assess the shrinkage influence on short-term deformations of RC members [36].

Obviously, for a cracked member, assuming every cross-section is uncracked leads to underestimated deformation and assuming that is going to be fully cracked leads to even sometimes grossly overestimated deformation [37].

5.8. EFFECTIVE PARAMETERS IN LONG-TERM DEFLECTION

The time-dependent nature of the effective parameters and the interaction between the parameters, along with the unknown properties of concrete at the design stage and the lack of experimental data to compare the results of the analysis with different techniques and codes, make the deflection prediction of RC members, particularly slabs a challenging problem. Mechanical properties of concrete affect the deflection directly by affecting the structural stiffness of the element and indirectly to define the moment redistribution due to cracking. So implementing the accurate mechanical properties of concrete and incorporating uncertainties is a crucial factor in deflection calculations.

Several models for the non-linear and time-dependent analysis of RC flexural members have been developed, which take into account the non-linearity of materials and the effects of creep and shrinkage of concrete. Sectional analyses with different levels of accuracy have been performed by Clarke et al. (1988) [38], Gilbert (1988) [39], Murcia (1991) [40], Favre and Charif (1994) [41], Nie and Cai (2000) [42], Chiorino (2005) [43] and Ghali et al (2006) [44]. At the structural level, general methods based on the layer filament beam elements have been developed by Kang and Scordelis (1980) [45], Marí (1984) [46], Elbadry and Ghali (1985) [47], Cruz (1998), and Ulm (1994) [48]. These methods include the non-linearity of materials such as cracking, yielding, crushing and load reversals, second order effects, time-dependent effects of creep and shrinkage and their interactive effects.

5.9. TENSION STIFFENING

Tension stiffening is attributed to the fact that concrete does not crack suddenly and entirely but undergoes progressive micro-cracking (strain softening). After first cracking in the RC member under tension or flexure, the intact concrete between adjacent primary cracks takes the significant portion of the tensile force, mainly in direction of the reinforcement. This is the result of the existing bond between the steel and the surrounding concrete.

Generally in the design and evaluation of RC elements, concrete poses no tension capacity after cracking in the tensile zone. Owing to the effect of tension stiffening, the bending stiffness of the member is considerably higher than that based on a fully cracked section, where concrete in tension is assumed to take zero stress. The average tensile stress in the concrete is a major percentage of the tensile strength of concrete. Figure (5.9) shows comparative diagrams of the compression and tension capacity of concrete regarding the stress-strain relationship. These empirical models vary in the applied techniques, properties of the materials and loading age as input data to ascertain the creep strain changes by time and related creep coefficient.

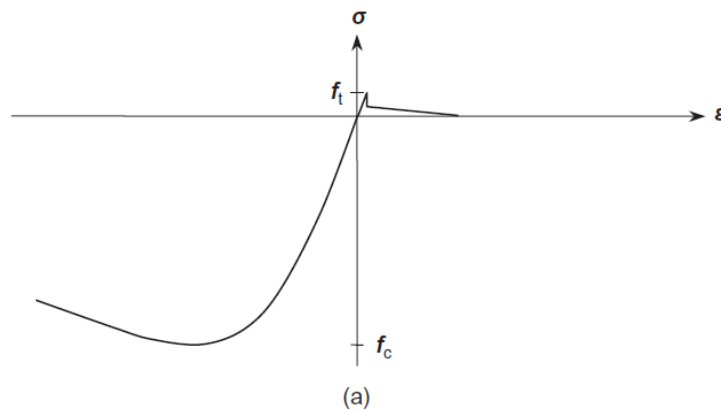


Figure 5.9: Stress-strain behaviour of concrete in tension and compression zones

This tension stiffening effect may be substantial in the performance of beams and slabs under service load. This phenomenon is also effective in long-term behaviour of concrete members due to the creep and shrinkage of the concrete.

Various methods have been proposed to implement the tension stiffening effect in the analysis of RC structures. To demonstrate that an area of concrete located at the level of tensile steel is effective in providing stiffening is an approach to modelling the tension stiffening.

There are various diagrams to show the tensile behaviour of concrete proposed by researchers. Damjanic and Owen (1984) [49] proposed an equivalent uniaxial constitutive

model for the tensioned concrete model depending only on dimensionless coefficients α_1 and α_2 as shown in Figure (5.10).

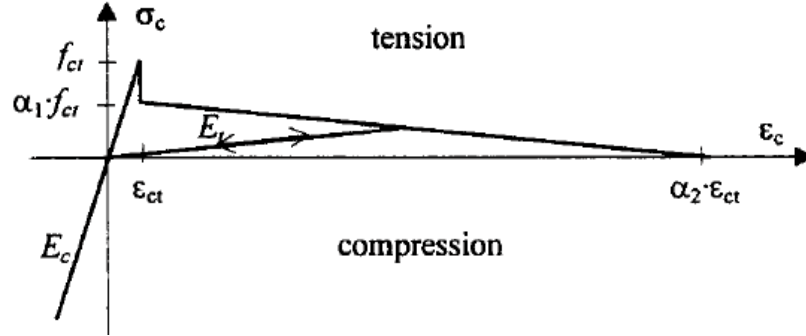


Figure 5.10: Equivalent stress-strain relationship for tensioned concrete [49]

Torres et al. (2004) [50] investigated the effect of these factors on the existing instantaneous moment-curvature ($M-\kappa$) models illustrated in Figures (5.11 a, b) and moment-strain diagram shown in Figure (5.11c) for a rectangular section. The reference fibre selected for the strain ϵ_0 is the one corresponding to tensioned steel. They concluded the significant effect of these factors on the shape of the $M-\kappa$ and moment-strain laws. In particular, the main effect of α_1 is on the part of the laws that immediately follow the cracking initiation and considerable influence of α_2 in the upper part of the diagram (where the applied service moment is considerably greater than the cracking moment, i.e., $M_a \gg M_{cr}$). Furthermore, the pairs of α_1 and α_2 values yielding best adjustments for the $M-\kappa$ and the moment-strain laws are different.

5.9.1. Existing Models of Tensile Stress Block of Concrete in Literature

Table (5.4) shows some existing models in the literature to predict the stress-strain relationship of concrete in tensile zone. The models are mainly proposed based on the experimental investigations of the sections under direct tension or flexural bending loading. Some models are developed based on numerical investigations and modification of the existing models for particular situations.

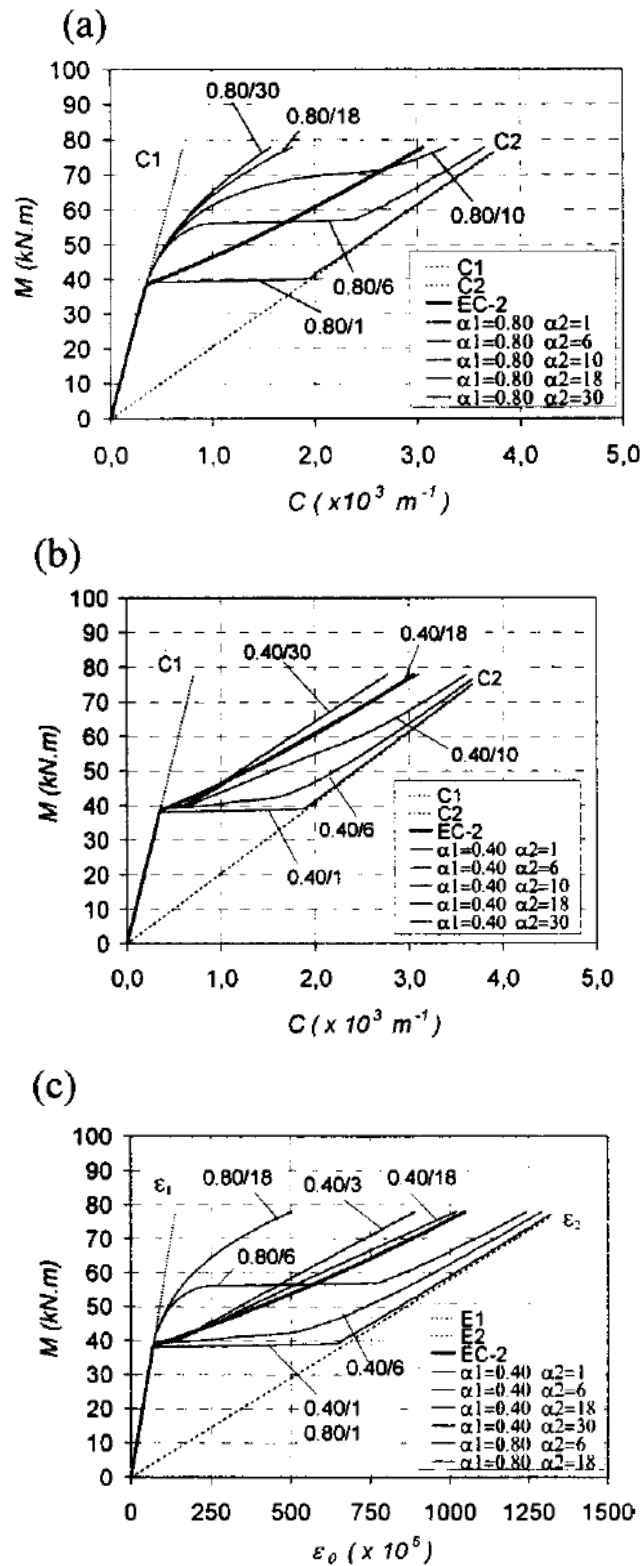


Figure 5.11: Influence of α_1 and α_2 in the moment-curvature (a, b) and moment-strain (c) laws [50]

Table 5.4: Existing models for tensile stress-strain diagrams of reinforced concrete considering tension stiffening

Reference	Tensile stress-strain model	Notes
Scanlon-Murray (1974) [51]	a)	Layered analysis of RC section f_t' : tensile strength of concrete ϵ_{cr} : Strain at peak stress f_t'
Lin and Scordelis (1975) [52]	b)	Ascending branch : linear Descending branch : Polynomial Perfect steel-concrete bond assumption
Vebo and Ghali model-1977 [53]	c)	For coefficients refer to Alih and Khelil (2012) [54]
Scott (1983) [55]		f_{bf} and e_{bf} tensile stress and strain at bottom face concrete f_t' : concrete ultimate tensile stress $(EI)_u$: EI for the uncracked section $(EI)_{cr}$: EI for the fully cracked section

Table 5.4: Existing models for tensile stress-strain diagrams of reinforced concrete considering tension stiffening (continueud)

Reference	Tensile stress-strain model	Notes																				
Gilbert and Warner (1978) [56]	a) Stepped Response After Cracking b) Gradually Unloading Response After Cracking c) Discontinuous Unloading Response After Cracking	For coefficients refer to Alih and Khelil (2012) [54] in this table <table><tr><th></th><th>Pt</th><th>Rt</th><th>St</th><th>Ft</th></tr><tr><td>Concrete layer adjacent to bar</td><td>0.8</td><td>0.4</td><td>4</td><td>10</td></tr><tr><td>one layer off the bar</td><td>0.5</td><td>0.2</td><td>4</td><td>10</td></tr><tr><td>two layers off the bar</td><td>0.2</td><td>0.1</td><td>4</td><td>10</td></tr></table>		Pt	Rt	St	Ft	Concrete layer adjacent to bar	0.8	0.4	4	10	one layer off the bar	0.5	0.2	4	10	two layers off the bar	0.2	0.1	4	10
	Pt	Rt	St	Ft																		
Concrete layer adjacent to bar	0.8	0.4	4	10																		
one layer off the bar	0.5	0.2	4	10																		
two layers off the bar	0.2	0.1	4	10																		
Damjanic and Owen- (1984) [49]	$\alpha_1 = 0.5$ $\alpha_2 = 5-10$	f_t: Tensile strength of concrete ϵ_{ct}: Strain at peak stress f_t $f_t = \begin{cases} E_{c0}\epsilon & \text{if } \frac{\epsilon}{\epsilon_{ct}} \leq 1 \\ \frac{\alpha_1 f_t (\alpha_2 \epsilon_{ct} - \epsilon)}{(\alpha_2 \epsilon_{ct} - \epsilon_{ct})} & \text{if } \epsilon_{ct} < \epsilon \leq \alpha_2 \epsilon_{ct} \\ 0 & \text{for } \alpha_2 \epsilon_{ct} < \epsilon \end{cases}$ $\alpha_1 = 0.5$, $5 \leq \alpha_2 \leq 10$																				

Table 5.4: Existing models for tensile stress-strain diagrams of reinforced concrete considering tension stiffening (continueud)

Reference	Tensile stress-strain model	Notes
Schnobrich (1985) [57]		f_t : tensile strength of concrete ϵ_{ct} : Strain at peak stress f_t $\epsilon_{max} = 20 \epsilon_{ct}$ ($\alpha_1=1$, $\alpha_2=20$) $f_t = \begin{cases} E_{c0}\epsilon & \text{if } \frac{\epsilon}{\epsilon_{ct}} \leq 1 \\ \frac{\alpha_1 f_t (\alpha_2 \epsilon_{ct} - \epsilon)}{(\alpha_2 \epsilon_{ct} - \epsilon_{ct})} & \text{if } \epsilon_{ct} < \epsilon \leq \alpha_2 \epsilon_{ct} \\ 0 & \text{for } \alpha_2 \epsilon_{ct} < \epsilon \end{cases}$
Carriera and Chu (1986) and Prakhya and Morely (1990) [58]		$f_t = \begin{cases} f'_t \left(\frac{\epsilon_t}{\epsilon'_t} \right) & \text{if } \frac{\epsilon_t}{\epsilon'_t} \leq 1 \\ \frac{\beta_t f'_t \left(\frac{\epsilon_t}{\epsilon'_t} \right)}{\beta_t - 1 + \left(\frac{\epsilon_t}{\epsilon'_t} \right)^{\beta_t}} & \text{if } \frac{\epsilon_t}{\epsilon'_t} > 1 \end{cases}$ $\epsilon_{max} = \alpha \epsilon'_t \quad \alpha = \frac{E_{ci}}{E_t} + 1$ $E_t = \frac{0.483 E_{ci}}{0.393 + f'_t}$ β_t : Empirical factor $\beta_t=1$ perfect plastic, $\beta_t = \infty$ sudden loss of strength ϵ'_t : strain at peak stress f'_t
Vecchio and Collins (1986) [59]		RC elements subjected to shear $f_{c1} = \frac{f_{cr}}{1 + \sqrt{200 \epsilon_1}}$ f_{c1}, ϵ_1 : principal tensile stress and strain in concrete f_{cr} : cracking stress

Table 5.4: Existing models for tensile stress-strain diagrams of reinforced concrete considering tension stiffening (contineud)

Reference	Tensile stress-strain model	Notes
Massicote et al. (1990) [60]		$E_c = 3320\sqrt{f'_c} + 6900$ (ACI - 363 - 84) $f'_t = 0.324 \sqrt[2/3]{f'_c}$ (Raphael (1984)) $\epsilon_{cr} = \frac{f'_t}{E_c}$ $E_1 = \frac{1}{6}$, $E_2 = \frac{1}{10} E_c$ $W_f = 5 \frac{f_t'^2}{E_c}$ cracking energy Total area = 10 × (ascending branch area)
BS 8110-1 (1997) [35]		
Kaklauskas (1999) [61]		$\sigma_t = a \cdot \sigma_{cr} \left(1 - \frac{\bar{\epsilon}_t}{\beta} - \frac{1 + \beta(1 - a)/a}{\beta(\bar{\epsilon}_t)^b} \right)$ $\bar{\epsilon}_t = \frac{\epsilon_t}{\epsilon_{cr}}$; $\epsilon_{cr} = \frac{\sigma_{cr}}{E_c}$ $a = 0.625$, $b = 1.0$ $\beta = 32.8 - 27.6\rho + 7.12\rho^2$; $\rho < 0.02$ $\beta = 5$; $\rho \geq 0.02$ $\sigma_{cr} = 0.23 \sqrt[3]{f_{cu-150}^2}$ (MPa) $E_c = \frac{5.5 \times f_{cu-150}}{27 + f_{cu-150}} \times 10^4$ (MPa)

Table 5.4: Existing models for tensile stress-strain diagrams of reinforced concrete considering tension stiffening (cntineud)

Reference	Tensile stress-strain model	Notes																														
Foster and Marti (2003) [62]		Applicable for deep beams and walls (plane stress elements) Based on model of Petersson (1981) [63] $\alpha_1 = \frac{1}{3}$, $\alpha_2 = \frac{2}{9} \alpha_3 + \alpha_1$, $\alpha_3 = \frac{18 E_c G_f}{5 l_{ch} f_{ct}^2}$ and Hillerborg et al. (1976) [64] $\alpha_1 = 0$, $\alpha_2 = \alpha_3 = \frac{2 E_c G_f}{l_{ch} f_{ct}^2}$ f_{ct} : tensile strength of the concrete G_f : fracture energy; l_{ch} :characteristic length of the finite element																														
Beeby et al. (2005) [65]	Bottom face concrete tensile stress Bottom face strain (microstrain)	Short-term loading Modification of ICE, TN-372 model (1983) or model proposed by Scott (1983) f_t : tensile strength of the concrete																														
Beeby et al. (2005) [65]	Bottom face concrete tensile stress Bottom face strain (microstrain)	Long-term loading Modification of ICE, TN-372 model (1983) or model proposed by Scott (1983)																														
Nayal and Rasheed (2006) [66]		General and fibre reinforced section Post-peak cyclic loading applied <table><tr><th></th><th>Cyc.1</th><th>Cyc.2</th><th>Cyc.3</th><th>Cyc.4</th><th>selected</th></tr><tr><td>Pt</td><td>0.6,0.8,1</td><td>0.8</td><td>0.8</td><td>0.8</td><td>0.8</td></tr><tr><td>Rt</td><td>0.45</td><td>0.3,0.45,0.6</td><td>0.45</td><td>0.45</td><td>0.45</td></tr><tr><td>St</td><td>4</td><td>4</td><td>4,6,8</td><td>4</td><td>4</td></tr><tr><td>Ft</td><td>10</td><td>10</td><td>10</td><td>5,10,40,100</td><td>10</td></tr></table>		Cyc.1	Cyc.2	Cyc.3	Cyc.4	selected	Pt	0.6,0.8,1	0.8	0.8	0.8	0.8	Rt	0.45	0.3,0.45,0.6	0.45	0.45	0.45	St	4	4	4,6,8	4	4	Ft	10	10	10	5,10,40,100	10
	Cyc.1	Cyc.2	Cyc.3	Cyc.4	selected																											
Pt	0.6,0.8,1	0.8	0.8	0.8	0.8																											
Rt	0.45	0.3,0.45,0.6	0.45	0.45	0.45																											
St	4	4	4,6,8	4	4																											
Ft	10	10	10	5,10,40,100	10																											

Table 5.4: Existing models for tensile stress-strain diagrams of reinforced concrete considering tension stiffening (continueud)

Reference	Tensile stress-strain model	Notes
Stramandinoli et al. (2008) [67]		f_{ct} : concrete tensile strength ϵ_{cr} : strain at f_{ct} $\sigma_{ct} = f_{ct} \cdot e^{-\alpha \left(\frac{\epsilon}{\epsilon_{cr}} \right)}$ $\alpha = 0.017 + 0.255(n\rho) - 0.106(n\rho)^2 + 0.016(n\rho)^3$ $\rho = \frac{A_{st}}{bd}$, $n = \frac{E_s}{E_c}$
Ng et al. (2010) [68]		General and fibre reinforced section $f'_t = \alpha_1 f_t$; $\alpha_1 < 1$ $\epsilon_{tu} = \alpha_2 \epsilon_{ct}$, $\alpha_2 > 1$, $\epsilon_{ct} = \frac{f_t}{E_{c0}}$ $\alpha_1, \alpha_2 = 0.4$ and 18 for point load $\alpha_1, \alpha_2 = 0.5, 14$ for distributed load $\sigma = E_{c0} \epsilon$ for $\epsilon \leq \epsilon_{ct}$ $\sigma = \frac{\alpha_1 f_t (\alpha_2 \epsilon_{ct} - \epsilon)}{(\alpha_2 \epsilon_{ct} - \epsilon_{ct})}$ for $\epsilon_{ct} < \epsilon \leq \alpha_2 \epsilon_{ct}$ $\sigma = 0$ for $\alpha_2 \epsilon_{ct} < \epsilon$
Alih and Khelil (2012) [54]		Inoxydable steel reinforcing bar $P_t = 0.8, R_t = 0.45, S_t = 4, F_t = 10$ $f'_t = 0.3 f_{ck}^{0.67}$, $\epsilon_{cr} = \frac{f'_t}{E_c}$ $\sigma_{c,TSE} = \rho_{eff} \sigma_{s,TSE}$, $\sigma_{s,TSE} = (\sigma_s - \sigma_{s,II})$ $\rho_{eff} = \frac{A_s}{A_{c,eff}}$ $A_{c,eff}$: effective zone of concrete around the rebar

Table 5.4: Existing models for tensile stress-strain diagrams of reinforced concrete considering tension stiffening (contineud)

Reference	Tensile stress-strain model	Notes
Bacinskas et al. (2012) [69]	Average stress $f_{ctm}(t_0)$ f_{ctr} $f_{ct}(\dots)$ $\alpha \cdot f_{ct}(t_0)$ $E_c(t_0)$ $\sigma_{ct,m}(t_0)$ $E'_c(t_0)$ Average strain $\epsilon_{el}(t_0)$ $\epsilon_{cr}(t_0)$ $\epsilon_{pl}(t_0)$ ϵ_{ult}	Short-term loading $\alpha = 0.45, \rho = A_{st}/bd$ $f_{ctm} = \text{tensile strength}$ From EC-2 $\epsilon_{ult} = \epsilon_{cr}(t_0) \times (32.8 - 27.6\rho + 7.12\rho^2); \rho \leq 0.02$ $\epsilon_{ult} = 5; \rho > 0.02$
Bacinskas et al. (2012) [69]	Average stress $f_{ctm}(t_0)$ $f_{ct}(t, t_0)$ $\alpha \cdot f_{ct}(t_0)$ $E_c(t, t_0)$ $E'_c(t, t_0)$ Average strain $\epsilon_{cr}(t, t_0)$ ϵ_{ult} age rain $t = t_0$ $t > t_0$	Long-term (sustained) loading $\alpha = 0.45, \rho = A_{st}/bd$ Reduced tensile strength under sustained loading: $f_{ct}(t, t_0) = f_{ctm}(t_0)[0.794 - 0.06 \times \log_{10}(t - t_0)]$ $f_{ct}(t, t_0) \geq \alpha f_{ct}(t_0)$

5.10. EFFECTIVE MOMENT OF INERTIA

When a reinforced concrete element cracks, its stiffness does not suddenly change to that of a section where the tension-concrete can be fully disregarded [70]. In fact, the sections where the cracks are localized are separated by regions where the concrete in tension is uncracked with an increase of the stiffness. Behaviour at the cracked section used to compute the cracking moment of inertia (I_{cr}) is assumed to be linear elastic, and nonlinearity of the member stiffness is taken into account with an effective moment of inertia (I_e) that models the transition from a gross (uncracked) moment of inertia (I_g) to the cracked transformed moment of inertia (I_{cr}). Figure (5.12) shows the cracked and uncracked sections of a flexural reinforced concrete member under sustained loading.

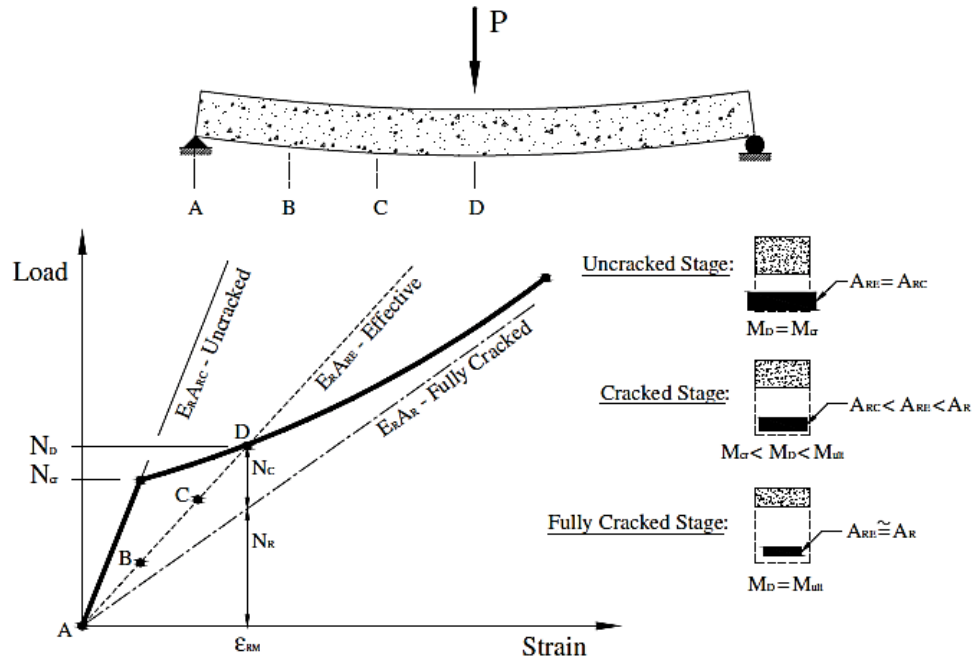


Figure 5.12: Deformation response of idealized reinforcement at critical section [70]

Figures (5.13 a, b) also show the load-deflection and moment-curvature diagrams of a simply supported RC flexural elements subjected to point-load at the mid-span.

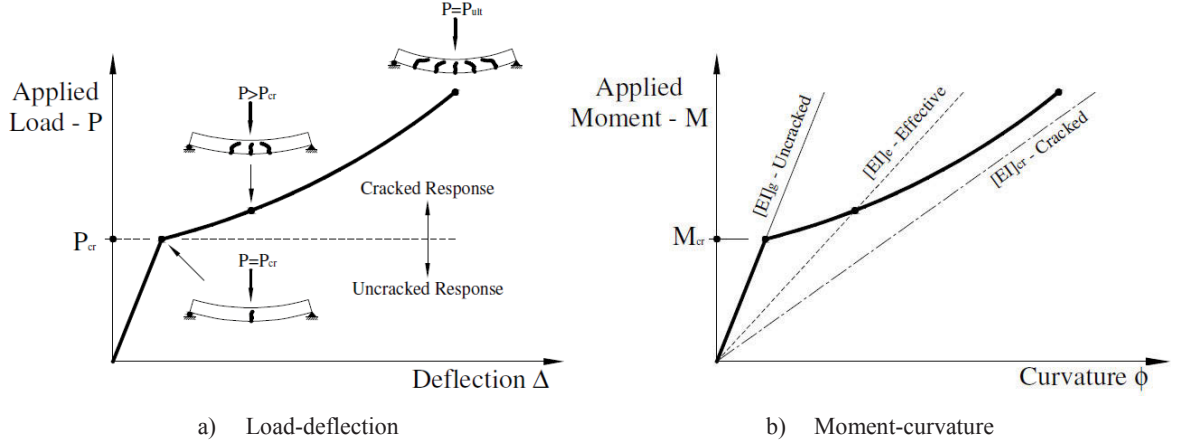


Figure 5.13: Flexural response of reinforced concrete members

Various empirical models have been used to model this behaviour, primarily to predict and check deflections of RC flexural members with code limits. Such models are normally compatible with moment-curvature ($M-\kappa$) space rather than stress-strain ($\sigma-\epsilon$) space, so do not model individual cracks [70].

Branson (1963, 1968) developed Equation (5.1) to predict the curvature and deflections by the effective moment of inertia directly from elasticity theory. This model was adopted by the deflection committee of the American Concrete Institute (ACI-435) in 1966 and then in ACI-318-77 (and recent revisions) building code and Australian code for concrete structures design (AS-3600-09) with some modifications [71].

$$I_e = (M_{cr}/M_a)^2 I_g + (1 - (M_{cr}/M_a)^2) I_{cr} \quad Eq. (5.1)$$

Where; I_g , I_e , I_{cr} , M_{cr} and M_a are representatives for gross moment of inertia, the effective moment of inertia, moment of inertia of cracked section, cracking moment and service moment of RC section respectively.

Flexural behaviour after cracking is more accurately reflected by taking a weighted average of flexibility instead of stiffness, and this leads to a subtle change in Branson's original expression. According to an investigation by Bischoff and Gross (2010) [72], in many cases there is not much difference between using the stiffness at the critical section to represent the member stiffness or taking into account the change in stiffness along the member length. This is particularly evident for steel reinforced concrete beams where stiffness of the cracked section ($E_c I_{cr}$) is typically one-third or more of the uncracked stiffness ($E_c I_g$), and the service load moment (M_a) is less than three times of the cracking moment of section ($M_a < 3M_{cr}$).

Deflection or curvature has an inverse relationship to the second moment of area, so a weighted average of flexibility and not stiffness should be used to compute deflection. Branson (1963-68) initially expected a squared term in his expression for I_e in Equation (5.1). However he found that the power in the (M_{cr}/M_a) term needed to be increased from 2 up to a value of 3 as presented in Equation (5.2) for average member behaviour or 4 for section behaviour [73]. This expression essentially represents a weighted average of the cracked ($E_c I_{cr}$) and uncracked ($E_c I_g$) member stiffness [71]. Equation (5.3) was adopted by various codes of practice such as ACI-435 (1966) and ACI-318 (1977) to compute for the effect of cracking in reduced stiffness of a reinforced concrete section.

$$\frac{1}{I_e} = \left(\frac{M_{cr}}{M_a}\right)^3 \frac{1}{I_g} + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) \frac{1}{I_{cr}} \geq \frac{1}{I_g} \quad Eq. (5.2)$$

$$I_e = \left(\frac{M_{cr}}{M}\right)^4 I_g + \left(1 - \left(\frac{M_{cr}}{M}\right)^4\right) I_{cr} \leq I_g \quad Eq. (5.3)$$

This method was intended to represent the sections at the working load and well below yield of the reinforcement. Considering the RC elements subjected to only pure bending, this can be regarded as a simple couple and neglecting the need to define a particular reference axis and distinction between the centroid and the neutral axis, limits the application of Branson's method to model the tension stiffening [70]. Tension stiffening eventually becomes ineffective with increasing the cracks, while in Branson's model, the stiffness becomes asymptotic to the fully cracked state but never reaches it [72].

Since some tension is transferred from steel to concrete through bond, so there are some tensile stresses in the concrete between the cracks. Distribution and magnitude of the bond stresses between the cracks represent the distribution of tensile stresses in the concrete and the reinforcing steel between the cracks. This sharing of the tensile force between the reinforcement and the concrete that represents difference between the actual and the zero tension response is referred to "tension stiffening" [26],[37].

The average instantaneous moment-curvature response of RC elements under uniform bending is shown as curve OAB in Figure (5.14). In reality, the flexural rigidity of the fully-cracked cross-section ($E_c I_{cr}$) underestimates stiffness after cracking because of tension stiffening that is represented by a reduction in average instantaneous curvature ($\delta\kappa_{0,ts}$) as shown in Figure (5.14) [74].

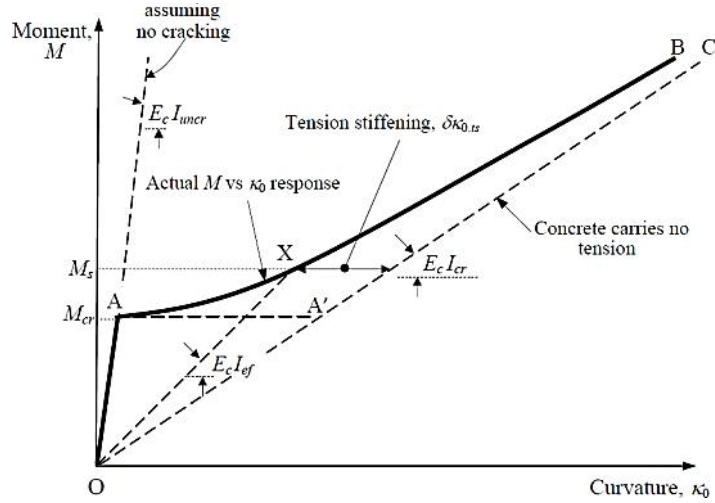


Figure 5.14: Average moment versus instantaneous curvature relationship [74]

Bischoff (2004) [75] proposed the Equation (5.4) to model the gradual transition from the gross (uncracked) moment of inertia (I_g) to the cracked transformed moment of inertia (I_{cr}) as loading progresses beyond the cracking moment M_{cr} .

$$I_e = \frac{I_{cr}}{1 - \eta \left(M_{cr}/M_a \right)^2} \leq I_g \quad \text{Eq. (5.4)}$$

Where; $\eta = 1 - \frac{I_{cr}}{I_g}$, M_a and M_{cr} are the service load moment and cracking moment of section, I_{cr} and I_g are cracking and gross moment of inertia of section.

Expression for I_e in Equation (5.4) was initially developed based on concepts of tension stiffening to axial tension members (Bischoff and Paixao, (2004)) and later extended to include flexural members (Bischoff, (2005)) resulting in the general model for I_e as shown in Equation (5.5). This equation models the flexural stiffness $E_c I_e$ corresponding to the M_a to define the moment curvature response by introducing the tension stiffening factor β , varying between 1 (just before cracking with full tension stiffening) and 0 (no bond after cracking and no tension stiffening) to account for tension carried by the concrete between cracks and stiffness of the member cross-section [76].

$$I_e = \frac{I_{cr}}{1 - \eta \beta \left(M_{cr}/M_a \right)} \leq I_g \quad \text{Eq. (5.5)}$$

As shown in Figure (5.15), setting $\beta=1$ representing the upper boundary on member stiffness, gives a bilinear response with constant tension stiffening, while $\beta=0$ gives a lower boundary with no tension stiffening that essentially gives the I_{cr} response. Approximating β with the ratio of M_{cr} / M_a gives a better model resulting to Equation (5.5) [75].

This tension-stiffening factor β , taken from the work of Rao (1966), and Leonhardt's (1977) group was instrumental in implementing the same tension-stiffening factor to compute crack widths and estimate deflection in CEB-FIP (1978) for concrete structures, which forms the basis of the later (CEB-1993) and Eurocode-2 (CEN-2004) [72].

As aptly shown in Figure (5.16), Branson's equation overestimates the stiffness for flexural members with $I_g / I_{cr} > 3$ or 4 and for $2 < I_g / I_{cr} < 3$, and the member response appears reasonable. On the other hand, as tension stiffening far exceeds the upper limit given by $\beta=1$, the computed member response is too stiff for a member with an I_g / I_{cr} ratio of 15.

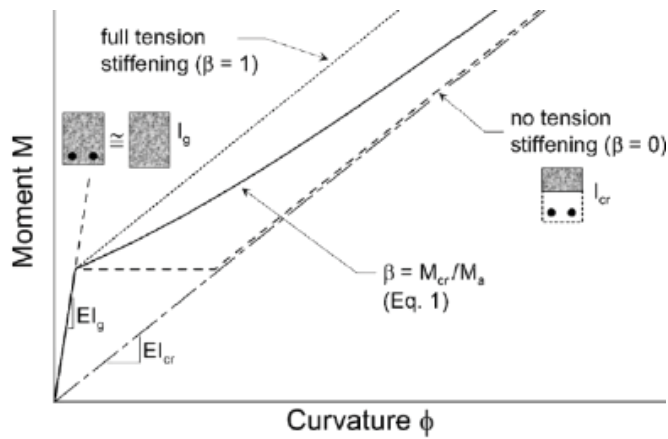


Figure 5.15: Theoretical moment-curvature response at the critical section using Bischoff's method (Eq. 5.1) with different tension stiffening values [76]

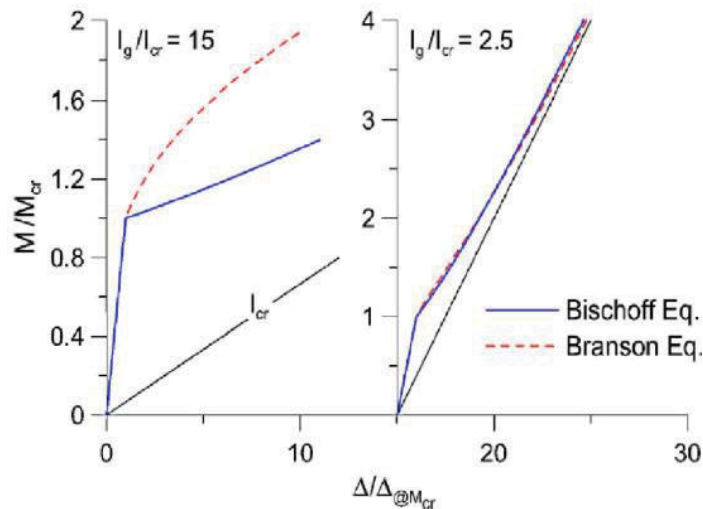


Figure 5.16: Deflection comparison using Branson's (Eq. 5.1) and Bischoff's expression for I_e [76]

Bischoff (2005, 2007) [75] and Bischoff and Scanlon (2007) [77] modified the more realistic and rational equation for I_e as a general expression in Equation (5.6), where β is used to account for both shrinkage-induced cracking and the reduction in tension stiffening that occurs with time. Early shrinkage in the days and weeks after casting will cause tension in the concrete and a reduction in the cracking moment.

Confirming Bischoff (2005, 2007), the assumed value of M_{cr} which depends on the concrete shrinkage prior to first loading, significantly affects the final predicted deflection [74]. The effect of the service load moment to cracking moment (M_a / M_{cr}) on accuracy of the predicted deflections is even more in lightly reinforced concrete and reinforce concrete members with Fibre Reinforced Polymer (FRP) that typically has a service load of less than twice the cracking value [72].

Scott and Beeby (2012) [78] showed that the tension stiffening and corresponding deflection of a flexure member after cracking, are strongly influenced by the assumed value of the cracking moment M_{cr} . The less assumed value of M_{cr} , the less stiffness and correspondingly more deflection (see Figures (5.17) and (5.18)).

Computed deflection values of beams reinforced with steel at reinforcing ratios greater than 1%, using either equation (Branson's or Bischoff's) are similar, but for flexural members with an I_g / I_{cr} ratio greater than 3, differences become evident [1].

Due to the high sensitivity of the I_e value to the calculated value of M_{cr} , especially for lightly loaded elements, failure to account for cracking due to unanticipated shrinkage restraint, temperature gradients or construction loads can significantly underestimate the deflection, so the upper limit of $0.6 I_g$ in Equation (5.6) is recommended by some researches. This limit has been applied in AS-3600-09 [9]. The correct response depends mainly on the level of cracking and lies somewhere between two extreme limits of the tension stiffening factor β as shown for Equation (5.6) in Figure (5.18) [1].

$$I_e = \frac{I_{cr}}{1 - \beta \left(\frac{M_{cr}}{M_a} \right) \left(1 - \frac{I_{cr}}{I_g} \right)} \leq I_g \quad Eq. (5.6)$$

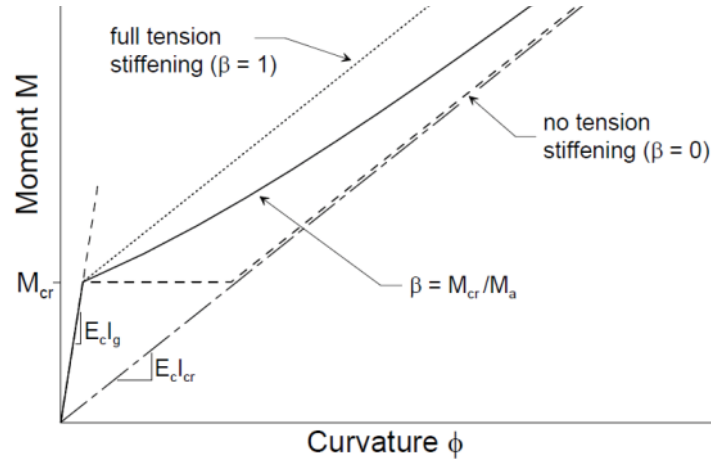


Figure 5.17: Effect of tension stiffening on member stiffness [75]

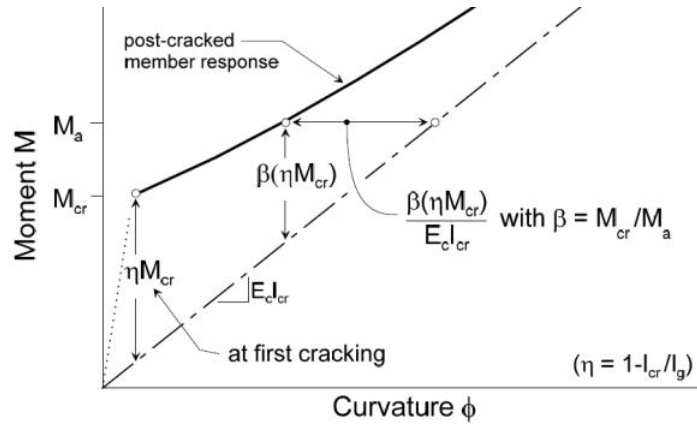


Figure 5.18: Effect of cracking moment and tension stiffening on the member response after cracking [1]

5.11. EXISTING MODELS OF EFFECTIVE MOMENT OF INERTIA

The model which was initially developed by Branson (1963) was widely investigated by other researchers worldwide. The basic concept of Branson's model (effective moment of inertia for a cracked section instead of gross moment of inertia) was modified by different researchers and organizations mainly by implementing the idea of tension stiffening to apply to different types of structures, loading rates, section reinforced with fibre and FRP sheets and FRP bars. Equations (5.7) to (5.25) shown in Table (5.5) represent some verified models of effective moment of inertia to enhance the prediction of short-term and long-term behaviour in RC members. These models are developed to adapt to different load types and conditions, reinforcing types, and supporting conditions. However, these models are generally applicable to all types of loading and support conditions by modifying some additional factors.

Table 5.5: Existing models of effective moment of inertia in RC section

Reference	Equation	Application	Expression
Bronson (1963) Bronson (1965) [71]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^2 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^2\right) I_{cr} \leq I_g$ Eq. (5.7) $I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \leq I_g$ Eq. (5.8) $I_e = \left(\frac{M_{cr}}{M_a}\right)^4 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^4\right) I_{cr} \leq I_g$ Eq. (5.9)	Normal RC beam	Power in expressions; 2: Initially expected 3: average member behavior 4: section behavior
Al Shaik and Al Zaid (1993) [79]	$I_e = I_{cr} + (I_g - I_{cr}) \left(\frac{M_{cr}}{M_a}\right)^{3-0.8\rho} \leq I_g$ Eq. (5.10)	Point load on beams	ρ : tensile reinforcement ratio
ACI-318-08 [34] ACI-435-68 [80] AS-3600-09 [9]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \leq I_g$ Eq.(5.11)	Normal concrete	$I_e \leq 0.6I_g$ in AS3600-09 proposed by Gilbert (2001) for lightly reinforced members
ACI 440 (2006) [81]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$ Eq. (5.12) $\beta_d = \alpha_b \left(\frac{E_f}{E_s} + 1\right), \quad \beta_d = 0.2 \frac{\rho}{\rho_b} < 1$	FRP members	Ef: Elasticity of FRP bars Es: Elasticity of steel bars
Bischoff and Paixao (2004) [82] Eurocode 2, (1994) [83]	$I_{ef} = \frac{I_{cr}}{1 - \eta \left(\frac{M_{cr}}{M_a}\right)^2} \leq I_g$ Eq. (5.13) $\eta = 1 - I_{cr}/I_g$	Originally for axial tension members	Applicable for flexural members
Bischoff (2005, 2007) [84] Bischoff and Scanlon (2007) [84]	$I_{ef} = \frac{I_{cr}}{1 - \eta \beta \left(\frac{M_{cr}}{M_a}\right)} \leq I_g$ Eq. (5.14) no tension stiffening $0 \leq \beta \leq 1$ full tension stiffening	flexural members	
Bischoff and Gross (2011) [72]	$I'_{ef} = \frac{I_{cr}}{1 - \eta \gamma \beta \left(\frac{M_{cr}}{M_a}\right)} \leq I_g$ Eq. (5.15) $\gamma = 1.72 - 0.72 M_{cr}/M_a, \quad \beta = M_{cr}/M_a$	Simply supported uniform load	Equivalent moment of inertia
Faza and GangaRao (1992) [85]	$I_m = \frac{23 I_{cr} I_e}{8 I_{cr} + 15 I_e}$ Eq. (5.16)	FRP beam	I_e from ACI-318
Benmokrane et al. (1996) [86]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \frac{I_g}{7} + 0.84 \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$ Eq. (5.17)	FRP beam	
Yost et al. (2003) [87] and ACI-440 (2006) [81]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$ $\beta_d = 0.2 \frac{\rho}{\rho_b} < 1$ Eq. (5.18)	FRP members	

Table 5.5: Existing models of effective moment of inertia in RC section (conineud)

Reference	Equation	Application	Expression
Rafi and Nadjai (2009) [88]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \frac{I_{cr}}{\gamma} \leq I_g$ Eq. (5.19) $\gamma = \left(\frac{0.0017\rho}{\rho_b} + 0.8541\right) \left(\frac{E_f}{2E_s} + 1\right)$	FRP members	
Alsayed et al. (2000) [89]	$I_e = I_{cr} \quad \text{for } \frac{M_{cr}}{M_a} > 3 \quad \text{Eq. (5.20)}$ $I_e = \left[1.4 - \frac{2}{15} \left(\frac{M_{cr}}{M_a}\right)\right] I_{cr} \leq I_g \quad \text{for } 1 < \frac{M_{cr}}{M_a} < 3$	FRP members	
ISIS Canada (2001) [90]	$I_e = \frac{I_g I_{cr}}{I_{cr} + [1 - 0.5 \left(\frac{M_{cr}}{M_a}\right)^2] (I_g - I_{cr})} \leq I_g$ Eq. (5.21)	FRP members	
Hall and Ghali (2000) [91]	$I_e = \frac{I_g I_{cr}}{I_{cr} + [1 - \beta_1 \beta_2 \left(\frac{M_{cr}}{M_a}\right)^2] (I_{cr} - I_g)} \leq I_g$ Eq. (5.22)	Normal concrete	$\beta_1=1$:ribbed and 0: smooth bars $\beta_2=1$:first and 0: sustained loading
ACI- 224.2R -92 [92]	$A_e = \left(\frac{P_{cr}}{P}\right)^3 A_g + \left(1 - \left(\frac{P_{cr}}{P}\right)^3\right) A_{cr} \leq I_g$ Eq. (5.23) A_g: Gross cross section area , A_{cr}: nA_s	Members under direct tension	Analogous approach for I_e in ACI-318
Fikry and Thomas (1998) [93]	$I_e = I_{cr} + (I_g - I_{cr})^{-\left(\frac{M_a}{M_{cr}}\right) \left(\frac{L_{cr}}{L}\right) \rho} \leq I_g$ for $\rho > 1\%$ Eq. (5.24) $I_e = I_{cr} + (I_g - I_{cr})^{-\left(\frac{M_a}{M_{cr}}\right) \left(\frac{L_{cr}}{L}\right)} \leq I_g$ Eq. (6.25)	FRP and Normal concrete	L_{cr} : Cracked length L : Length of member ρ : Reinforcement ratio

5.12. SUMMARY

The second phase of the experimental program, i.e. the instantaneous and long-term deflection under constantan loading and the load-deflection behaviour of slabs are presented in Chapters Seven, Eight and Nine of this study.

In this chapter, a brief description of the serviceability aspects of the concrete structures including the deflection limitations by the structure geometry is given. Additionally, some other limitations and uncertainties in the literature and the reported experimental investigations about inadequacy of the existing models to predict the flexural deflection of concrete members are discussed.

To calculate the long-term flexural deflection of reinforced concrete structures, majority of the codes of practice imply the implication factor to the instantaneous deflection. A comprehensive description of the ratio of long-term to instantaneous deflection as the multiplier to estimate the time-dependent deflection is given. In addition, some existing experimental results in the laboratory conditions and real in-situ data are analysed and discussed.

The effective parameters in the instantaneous and long-term deflection of reinforced concrete structures are summarized. The tensile behavior of concrete and the interaction between the concrete and reinforcement are among the most effective parameters in flexural deflection. Therefore, a wide range of the existing models in the literature to describe the tensile behaviour of concrete are presented and evaluated. The collected models developed since 1974 show advances in describing the concrete behaviour under tension and the included parameters to enhance the models.

The effective moment of inertia is another crucial parameter in studying the flexural deflection of reinforced concrete structures. A background on the development of this concept and the included parameters is given. Sensitivity of the effective moment of inertia to loading value, size and ratio of reinforcement and the moment capacity of the reinforced concrete section are also discussed. Existing models in the literature presented by the researchers and codes of practice since 1963 are collected and evaluated.

REFERENCES

1. Gilbert, R.I., *Creep and Shrinkage Induced Deflections in RC Beams and Slabs*. ACI , Special Publication, 2012. **284**(13): p. 1-16.
2. 209.2R-08, A., *Guide for Modeling and Calculating Shrinkage and Creep in hardened concrete*. American Concrete Institute (ACI), ACI Committee 209, Farmington Hills, MI 48331, U.S.A, 2008.
3. BS-5400-4, *Steel, concrete and composite bridges, Part 4: Code of practice for design of concrete bridges*. Committee reference CSB/59, Draft for comment 88/11190 DC , British Standard, 1999.
4. béton, C.e.-i.d., *CEB-FIP model code 1990: design code*. 1993: CEB Bulletin d'Information No.213/214, Comité EuroInternational du Béton, Lausanne, Switzerland, Telford.
5. CSA.A23.3-04, *Design of concrete structures*. Canadian Standards Association (CSA), 2010.
6. EC-2, E., *Design of Concrete Structures, Part 1: General Rules and Rules for buildings*. Commission of European Communities ENV, German version EN 1992-1-1:2004 + AC: 2010, The Concrete Centre: Blackwater, Camberley, UK, . 2004: p. 1-1.
7. JSCE, *Standard specification for concrete structures design*. Japan society of Civil Engineering, JSCE, Tokyo , Japan, 2007.
8. IS.456, *Plain and Reinforced concrete- code of practice*. Indian Standard, ICS, 91.100.30, New Delhi, 2000.
9. AS-3600-09, *Concrete structures*. Australian Standard, 2009.
10. 1170.0, A.N., *Structural design actions - General principles*. Standards Australia, Sydney, Australia, 2002.
11. Stewart, M.G., *Serviceability reliability analysis of reinforced concrete structures*. Journal of Structural Engineering, 1996. **122**(7): p. 794-803.
12. Branson, D.E., *Deformation of concrete structures*. 1977: McGraw-Hill Companies.
13. Jokinen, E.P. and A. Scanlon, *Field measured two-way slab deflections*. Canadian Journal of Civil Engineering, 1987. **14**(6): p. 807-819.
14. Unnikrishna Pillai, S. and D. Menon, *Reinforced Concrete Design* Tata Mc Graw Hill, New Delhi, India,, 2003.
15. Nejadi, S., *Time-dependent cracking and crack control in reinforced concrete structures*. 2005, PhD thesis, The University of New South Wales Sydney, Australia.
16. Taylor, P. and J. Heiman. *Long-term deflection of reinforced concrete flat slabs and plates*. in *ACI Journal Proceedings*. 1977. ACI.
17. Gilbert, R., *Shrinkage, cracking and deflection-the serviceability of concrete structures*. Electronic Journal of Structural Engineering, 2001. **1**(1): p. 2-14.
18. Taha, M.R. and M.A. Hassanain, *Estimating the error in calculated deflections of HPC slabs: a parametric study using the theory of error propagation*. ACI Special Publication, 2003. **210**.
19. Ghali, A. and A. Azarnejad, *Deflection prediction of members of any concrete strength*. ACI Structural Journal, 1999. **96**(5).
20. Gilbert, R.I., *Deflection Calculation for Reinforced Concrete Structures? Why We Sometimes Get It Wrong*. ACI Structural Journal, 1999. **96**(6).
21. Heiman, J., *A Comparison of Measured and Calculated Deflections of Flexural Memebers in Four Reinforced Concrete Buildings*. ACI Special Publication, 1974. **43**.
22. Kaminetzky, D. and P. Stivaros, *Construction Loads and Serviceability Requirements: Deflection Control, Span/Thickness Limitations*. ACI Special Publication, 2003. **210**.
23. Vollum, R., R. Moss, and T. Hossain, *Slab deflections in the Cardington in-situ concrete frame building*. Magazine of Concrete research, 2002. **54**(1): p. 23-34.

24. projec, E.c.b., *Case studie on applying best practice to in-situ concrete frame buildings, St George Wharf case study - slab deflection*. European concrete building project, 2004.
25. French, C.E., et al. *Field and Laboratory Study of Precast Composite Slab Span System (PCSSS)*. in *2010 Concrete Bridge Conference: Achieving Safe, Smart & Sustainable Bridges*. 2010.
26. Hirsch, J., et al., *Practical Deflection Prediction of Concrete Slabs*. ACI Special Publication, 2012. **284**.
27. Uy, B. and M.A. Bradford, *Ductility of profiled composite beams. Part I: Experimental study*. Journal of Structural Engineering, 1995. **121**(5): p. 876-882.
28. Uy, B., *Long-term service-load behaviour of simply supported profiled composite slabs*. Proceedings of the Institution of Civil Engineers: Structures and Buildings, 1997. **122**(2): p. 193-208.
29. BS-5950-82, *tructural use of steelwork in building, Part 4. Code of practice for design of floors with profiled steel sheeting*. British Standard Institution (BSI), London, BS 5950., 1982.
30. Gilbert, R. and X. Guo, *Time-dependent deflection and deformation of reinforced concrete flat slabs-an experimental study*. ACI structural journal, 2005. **102**(3).
31. Eigelaar, E.M., *Deflections of reinforced concrete flat slabs*. 2010, Stellenbosch: University of Stellenbosch.
32. Park, H.-G., et al., *Immediate and Long-Term Deflections of Reinforced Concrete Slabs Affected by Early-Age Loading and Low Temperature*. ACI Structural Journal, 2012. **109**(3).
33. Lee, Y.H. and A. Scanlon, *Comparison of one-and two-way slab minimum thickness provisions in building codes and standards*. ACI Structural Journal, 2010. **107**(02).
34. ACI-318-02. *Building code requirements for structural concrete (ACI 318-08) and commentary*. 2002. American Concrete Institute, International Organization for Standardization.
35. BS-8110-1, *Structural use of concrete, Part14: Code of practice for design and construction*. Committee reference B/525/2 Draft for comment 95/105430 DC, 1997.
36. Kaklauskas, G. and V. Gribniak, *Eliminating shrinkage effect from moment curvature and tension stiffening relationships of reinforced concrete members*. Journal of Structural Engineering, 2011. **137**(12): p. 1460-1469.
37. Gilbert, R., *The serviceability limit states in reinforced concrete design*. Procedia Engineering, 2011. **14**: p. 385-395.
38. Clarke, G., H. Scholz, and M. Alexander, *New method to predict the creep deflection of cracked reinforced concrete flexural members*. ACI Materials Journal, 1988. **85**(2).
39. Gilbert, R.I., *Time effects in concrete structures*. Developments in civil engineering, 1988. **23**.
40. Murcia, J., *Approximate time dependent analysis of reinforced concrete sections. Proposal of a new factor for the calculation of long term deflections*. Hormigón y Acero, 1991. **181**: p. 9-17.
41. Favre, R. and H. Charif, *Basic model and simplified calculations of deformations according to the CEB-FIP model code 1990*. ACI Structural Journal, 1994. **91**(2).
42. Nie, J. and C.S. Cai, *Deflection of cracked RC beams under sustained loading*. Journal of Structural Engineering, 2000. **126**(6): p. 708-716.
43. Chiorino, M.A., *A Rational Approach to the Analysis of Structural Effects due to Creep*. ACI Special Publication, 2005. **227**.
44. Ghali, A., R. Favre, and M. Elbadry, *Concrete structures: Stresses and deformations: Analysis and design for serviceability*. 2006: CRC Press.
45. Kang, Y.-J. and A.C. Scordelis, *Nonlinear analysis of prestressed concrete frames*. Journal of the structural division, 1980. **106**(2): p. 445-462.

46. Mari, A.R., *Nonlinear geometric, material and time dependent analysis of three dimensional reinforced and prestressed concrete frames*. 1984: Department of Civil Engineering, University of California.
47. El-Badry, M.M. and A. Ghali, *User's manual and computer program CPF: cracked plane frames in prestressed concrete*. 1985: University of Calgary. Department of Civil Engineering.
48. Ulm, F.-J., J.-L. Clément, and J. Guggenberger. *Recent advances in 3D non-linear FE-analysis of R/C and P/C Beam Structures*. in *Structures Congress XII*. 1994. ASCE.
49. Damjanic, F. and D. Owen. *Practical considerations for modelling of post-cracking concrete behaviour for finite element analysis of reinforced concrete structures*. in *Proceedings, International Conference on Computer Aided Analysis and Design of Concrete Structures, Split, Yugoslavia, edited by Damjanic, F., Hinton, E., Owen, DRJ, Bicanic, N. and Simovic*. 1984.
50. Torres, L., F. Lopez-Almansa, and L. Bozzo, *Tension-stiffening model for cracked flexural concrete members*. *Journal of Structural Engineering*, 2004. **130**(8): p. 1242-1251.
51. Scanlon, A. and D.W. Murray, *Time-dependent reinforced concrete slab deflections*. *Journal of the Structural Division*, 1974. **100**(9): p. 1911-1924.
52. Lin, C.-S. and A.C. Scordelis, *Nonlinear analysis of RC shells of general form*. *Journal of the Structural Division*, 1975. **101**(3): p. 523-538.
53. Vebo, A. and A. Ghali, *Moment-curvature relation of reinforced concrete slabs*. *Journal of the Structural Division*, 1977. **103**(3): p. 515-531.
54. Alih, S. and A. Khelil. *Tension Stiffening Parameter in Composite Concrete Reinforced with Inoxydable Steel: Laboratory and Finite Element Analysis*. in *Proceedings of World Academy of Science, Engineering and Technology*. 2012. World Academy of Science, Engineering and Technology.
55. Scott, R. *TECHNICAL NOTE. THE SHORT TERM MOMENT-CURVATURE RELATIONSHIP FOR REINFORCED CONCRETE BEAMS*. in *ICE Proceedings*. 1983. Thomas Telford.
56. Gilbert, R.I. and R.F. Warner, *Tension stiffening in reinforced concrete slabs*. *Journal of the structural division*, 1978. **104**(12): p. 1885-1900.
57. Schnobrich, W.C. *The role of finite element analysis of reinforced concrete structures*. in *Finite element analysis of reinforced concrete structures*. 1985. ASCE.
58. Prakhya, G. and C. Morley, *Tension-stiffening and moment-curvature relations of reinforced concrete elements*. *ACI Structural Journal*, 1990. **87**(5).
59. Vecchio, F.J. and M.P. Collins. *The modified compression-field theory for reinforced concrete elements subjected to shear*. in *ACI Journal Proceedings*. 1986. ACI.
60. Massicotte, B., A.E. Elwi, and J.G. MacGregor, *Tension-stiffening model for planar reinforced concrete members*. *Journal of Structural Engineering*, 1990. **116**(11): p. 3039-3058.
61. Kaklauskas, G., *A New Stress-Strain Relationship for Cracked Tensile Concrete in Flexure*. *Statyba*, 1999. **5**(6): p. 349-356.
62. Foster, S.J. and P. Marti, *Cracked membrane model: Finite element implementation*. *Journal of Structural Engineering*, 2003. **129**(9): p. 1155-1163.
63. Petersson, P.-E., *Crack growth and development of fracture zones in plain concrete and similar materials*. 1981, Division, Inst.
64. Hillerborg, A., M. Modéer, and P.-E. Petersson, *Analysis of crack formation and crack growth in concrete by means of fracture mechanics and finite elements*. *Cement and concrete research*, 1976. **6**(6): p. 773-781.
65. Beeby, A., R. Scott, and A. Jones, *Revised code provisions for long-term deflection calculations*. *Proceedings of the ICE-Structures and Buildings*, 2005. **158**(1): p. 71-75.
66. Nayal, R. and H.A. Rasheed, *Tension stiffening model for concrete beams reinforced with steel and FRP bars*. *Journal of Materials in Civil Engineering*, 2006. **18**(6): p. 831-841.

67. Stramandinoli, R.S. and H.L. La Rovere, *An efficient tension-stiffening model for nonlinear analysis of reinforced concrete members*. Engineering Structures, 2008. **30**(7): p. 2069-2080.
68. Ng, P.L., J.Y. Lam, and A.K. Kwan, *Tension stiffening in concrete beams. Part 1: FE analysis*. Proceedings of the Institution of Civil Engineers: Structures and Buildings, 2010. **163**(1): p. 19-28.
69. Bacinkas, D., et al., *Layer model for long-term deflection analysis of cracked reinforced concrete bending members*. Mechanics of Time-Dependent Materials, 2012. **16**(2): p. 117-127.
70. Achintha, P.M. and C.J. Burgoyne, *Moment-curvature and strain energy of beams with external fiber-reinforced polymer reinforcement*. ACI Structural Journal, 2009. **106**(1).
71. Vakhshouri, B. and S. Nejadi. *Limitations and Uncertainties in the Long Term Deflection Calculation of Concrete Structures*. in *Second International Conference on Vulnerability and Risk Analysis and Management (ICVRAM) and the Sixth International Symposium on Uncertainty, Modeling, and Analysis (ISUMA)*. 2014.
72. Bischoff, P.H. and S.P. Gross, *Equivalent moment of inertia based on integration of curvature*. Journal of Composites for Construction, 2010. **15**(3): p. 263-273.
73. ACI-363-R, *State of the art report on high-strength concrete*. armington Hills (Michigan): American Concrete Institute (ACI), 1992.
74. Bischoff, P.H. and M. Darabi, *Unified approach for computing deflection of steel and FRP reinforced concrete*. ACI Special Publication, 2012. **284**.
75. H, B.P., *Discussion of "Tension Stiffening in Lightly Reinforced Concrete Slabs" R. Ian Gilbert , June 2007, Vol. 133, No. 6, pp. 899-903*. Journal of structural engineering 2008. **134**(7): p. 1259-1260.
76. Vollum, R.L., *Influence of Construction Loading on Long-Term Slab Deflections*. ACI Special Publication, 2012. **284**.
77. Bischoff, P.H. and A. Scanlon, *Effective moment of inertia for calculating deflections of concrete members containing steel reinforcement and fiber-reinforced polymer reinforcement*. ACI Structural Journal, 2007. **104**(1).
78. Scott, R.H. and A.W. Beeby, *Evaluation and management of tension stiffening*. ACI Special Publication, 2012. **284**.
79. Alshaikh, A.H. and R. Al-Zaid, *Effect of reinforcement ratio on the effective moment of inertia of reinforced concrete beams*. ACI Structural Journal, 1993. **90**(2).
80. ACI-435, *Deflections of reinforced concrete flexural member*. American Concrete Institute (ACI), Concrete International, 1968.
81. ACI-440-06, *Guide for the design and construction of concrete reinforced with FRP bars*. American Concrete Institute (ACI) Committee 440, ACI 440.1R-06, ACI, Farmington Hills, MI, 44, 2006.
82. Bischoff, P.H. and R. Paixao, *Tension stiffening and cracking of concrete reinforced with glass fiber reinforced polymer (GFRP) bars*. Canadian Journal of Civil Engineering, 2004. **31**(4): p. 579-588.
83. du Béton, C.E.-I., *CEB-FIP model code (MC-90)*. 1993, Thomas Telford Ltd, London.
84. Bischoff, P.H., *Deflection calculation of FRP reinforced concrete beams based on modifications to the existing Branson equation*. Journal of Composites for Construction, 2007. **11**(1): p. 4-14.
85. Faza, S. and H. GangaRao. *Pre-and post-cracking deflection behaviour of concrete beams reinforced with fibre-reinforced plastic rebars*. in *Proceedings of the First International Conference on Advance Composite Materials in Bridges and Structures (ACMBS-I), Canadian Society of Civil Engineers, Sherbrooke, Canada*. 1992.
86. Benmokrane, B., O. Chaallal, and R. Masmoudi, *Flexural response of concrete beams reinforced with FRP reinforcing bars*. ACI Structural Journal, 1996. **93**(1): p. 46-55.
87. Yost, J.R., S.P. Gross, and D.W. Dinehart, *Effective moment of inertia for glass fiber-reinforced polymer-reinforced concrete beams*. ACI Structural Journal, 2003. **100**(6).

88. Rafi, M.M., et al., *Aspects of behaviour of CFRP reinforced concrete beams in bending*. Construction and Building Materials, 2008. **22**(3): p. 277-285.
89. Alsayed, S., Y. Al-Salloum, and T. Almusallam, *Performance of glass fiber reinforced plastic bars as a reinforcing material for concrete structures*. Composites Part B: Engineering, 2000. **31**(6): p. 555-567.
90. Newhook J., *Reinforcing concrete structures with fibre reinforced polymers*. ISIS Canada: Design Manual No.3, The Canadian Network of Centres of excellence on intelligent sensing for innovative structures, Winnipeg, Manitoba, Canada, 2001.
91. Hall, T. and A. Ghali, *Long-term deflection prediction of concrete members reinforced with glass fibre reinforced polymer bars*. Canadian Journal of Civil Engineering, 2000. **27**(5): p. 890-898.
92. ACI-224.2R-86, *Cracking of concrete members in direct tension* American Concrete Institute (ACI) Committee 224, Detroit, USA, 1986.
93. Fikry, A.M. and C. Thomas, *Development of a model for the effective moment of inertia of one-way reinforced concrete elements*. ACI Structural Journal, 1998. **95**(4).

CHAPTER 6

EXPERIMENTAL PROGRAM (PHASE I) – MATERIAL PROPERTIES OF LIGHTWEIGHT CONCRETE CONTAINING EXPANDED POLYSTYRENE BEADS

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Chapter 6: Material properties of lightweight concrete containing expanded polystyrene beads

6.1. INTRODUCTION

This chapter explains the experimental program investigating the mechanical characteristics of the Light-Weight Concrete (LWC) in fresh and hardened states. The experimental program has been conducted in the concrete laboratory of University of Technology Sydney (UTS).

The main objective of this study is to compare the short-term (instantaneous) and long-term deflection of one-way slabs made with different types of concrete, i.e. Conventional Concrete (CC) and Self-Compacting Concrete (SCC) from previously conducted researches and Light-Weight Concrete (LWC) in the present study. In this regard, the experimental results of the fresh and hardened properties of SCC conducted by Aslani (2014) [1] and CC carried out by Nejadi (2005) [2] have also been compared with the corresponding properties of LWC in this study. Comparative analysis of the material properties in CC, SCC and LWC and development of the mechanical properties with age are the major objectives of this section.

In terms of the fresh concrete properties, the fresh density and slump value from the mixture design of CC, SCC and LWC are investigated. In total, one mixture for LWC, one mixture for CC and four mixtures for SCC have been considered in evaluating the fresh and hardened properties of concrete. The mixtures for SCC include one plain mixture and three fibre reinforced mixtures of steel, polypropylene and hybrid (steel and polypropylene) fibres mixed with different proportions.

In a hardened state, the compared properties of CC, LWC and SCC include the Compressive Strength (CS), Modulus of Elasticity (MoE), and Modulus of Rupture (MoR), Splitting Tensile Strength (STS), and bond from pull-out test, and Compressive

Stress-Strain Curve (CSSC). Also, the time-dependent properties like creep and shrinkage strains from these types of concrete are investigated.

To determine the mechanical properties of CC SCC and LWC, different types of tests on the concrete specimens have been conducted. Table (6.1) shows the types of tests performed to obtain the concrete properties used in the experimental and numerical investigations of the slabs in Chapters Six, Seven, Eight and Nine of this study.

Table 6.1: Conducted tests on lightweight, self-compacting and conventional concrete specimens

Concrete type	Hardened concrete tests								Fresh concrete tests	
	Compressive strength	Elasticity Modulus	Modulus of rupture	Splitting tensile strength	Creep	Shrinkage	Stress-strain	Pull-out	Density	Slump
CC	*	*	*	*	*	*	*	*	*	*
SCC	*	*	*	*	*	*	*		*	*
LWC	*	*	*	*	*	*	*	*	*	*

Other tests by Aslani (2014) [1] such as slump flow, T50cm time, J-ring flow, V-funnel flow time, and L-box blocking ratio tests are applicable just for SCC in the fresh state and are not comparable with the fresh properties of LWC and CC in this study. These tests for SCC are performed to check the filling ability, passing ability and segregation resistance of SCC mixtures.

In addition to the comparable experimental results of CC and SCC properties, to better understand the post-cracking behaviour of LWC, flexural toughness characteristics analysis and energy absorption in compression has been performed in this study also.

The main parts of this chapter can be summarized as follows:

- Description of the mixture proportions of LWC along with the chemical, physical and mechanical properties of the components;
- Evaluation and analysis of the experimental results of the tests on fresh LWC mixture and the hardened LWC specimens;
- Proposing the models to predict the mechanical properties of LWC;
- Comparison of the corresponding material or mechanical characteristics of LWC in this study with SCC and CC from previously conducted studies.

6.2. LIGHTWEIGHT CONCRETE CONTAINING EXPANDED POLYSTYRENE BEADS

6.2.1. Mixture Components

The LWC used in this study contains Expanded Poly-Styrene (EPS) beads as the lightweight aggregates to reduce the mixture density. LWC containing EPS beads (EPS-LWC) is widely used in Australia. It is also a common construction material in floating decks worldwide; however, it has not been investigated as much as the other types of LWC and CC. In the mechanical properties and time-dependent behaviour of the test specimens and structures made with EPS-LWC reported in the literature have been collected and presented in Chapter Three.

Considering the limited investigations of the mechanical properties of EPS-LWC, the existing correlations between the experimental and predicted data are unclear; therefore regression analyses are conducted on existing experimental data to propose new models of predicting its mechanical properties in Chapter Three. In this Chapter, components of the utilized LWC and their proportions in the mixture are included. The mixture design of EPS-LWC in this study contains natural coarse and fine aggregates, artificial lightweight aggregate, cement, water and admixture. Each component is described as follows:

6.2.1.1. Natural aggregates

As discussed in Chapter Two, the main difference between the mixture design of CC and LWC is the replacement of some or whole part of the normal weight (natural) coarse aggregate with lightweight (natural or artificial) aggregates in LWC. The mixture design of LWC in this study contains both types of natural coarse aggregates and artificial lightweight aggregate. Moreover, a blend of two types of natural fine aggregates has been used in the EPS-LWC mixture.

6.2.1.2. Coarse aggregate

The crushed Latite aggregate with a maximum size of 10 mm from Dunmore quarry (NSW, Australia) is used as the natural coarse aggregate. The properties of Dunmore coarse aggregate, including the chemical components and physical and mechanical properties are presented in Table (6.2). Australian standard, AS-1141 (2011) [3] along with the Regional Transportation Authority (RTA) (2006) and Road and Maritime services Specifications (RMS) (2015) recommendations [4] were applied in sampling and testing of the aggregates. The applied standards and the minimum or maximum allowable

values in each case are also shown in the Table (6.2). The measured properties of Dunmore coarse aggregate are in the allowable range.

Table 6.2: Properties of Dunmore 10 mm coarse aggregate

Test method	Test	Allowable	Measured
AS-1141.11.1 (2009) [5]	Passing A.S. sieve (%)		
	13.2 mm	100	100
	9.5 mm	85-100	88
	6.7 mm	40-60	53
	4.75 mm	0-20	20
	2.36 mm	0-5	4
	1.18 mm		2
AS-1141.12 (2011) [6]	≤ 75 µm (%)		1
AS-1141.14 (2011) [7]	Mis-shaped particles (%)		
	Ratio 2:1	<25	23
	Ratio 3:1	<10	6
RMS-T235	Average least dimension (mm)		3.5
	Fraction (-9.5+6.7mm)		6.1
	Fraction (-6.7+4.75mm)		4.4
RMS-T278 [4]	Aggregate shape by the ratio of greatest to least		
	0-2 mm		5.6
	2-4 mm		2.5
	4-6 mm		2.3
	6-8 mm		2.0
	The ratio of AGD to ALD		2.49
AS-1141.18 (2011) [8]	Crushed particles derived from gravel		
	% of crushed particles	≥80	100
	% of uncrushed particles		Nil
AS-1141.4 (2011) [9]	Uncompacted bulk density (t/m3)		1.33
	Compacted bulk density (t/m3)	≥1.2	1.5
RMS-T262	Moisture content (%)		3.0
AS-1141.6.1 (2011) [10]	Particle density (dry) (t/m3)	≥2.1	2.63
	Particle density (SSD) (t/m3)		2.66
	Apparent Particle density (t/m3)		2.76
	Water absorption	<2.5	1.9
AS-1141.22 (2008) [11]	Average aggregate dry strength (KN)		366
	Average aggregate wet strength (KN)	≥100	246
	Wet/dry strength var. (%)	<35	33
	Fraction tested (mm)		-9.5+6.7
	The amount of significant breakdown (%)		<0.2
AS-1141.23 (2009)	Los Angeles value grading “K” (% loss)	< 30	15
AS-1141.24 (2013) [13]	Sodium sulphate soundness (Total weighted % loss)	< 6.0	1.0
	Fraction tested		
	-13.2mm + 9.5mm (% loss)		0.6
	-9.5mm + 4.75mm (% loss)		0.7
	-4.75mm + 2.36mm (% loss)		2.6
AS-1141.32 (2008) [14]	Weak particles (%)		
	% of original sample passing 2.36 mm (sieve 3.4)	< 0.5	Nil
As-1141.25.2 (2013) [15]	Degradation factor- coarse aggregate		66
	The washed water after using permitted 500 ml		Not clear
SSD : Saturated Surface-Dry			

6.2.1.3. Fine aggregates

In this type of LWC, a blend of 50% coarse sand and 50% fine sand provides optimum mix. An equal weight proportion of Peppertree-P sand with a maximum size of 5 mm, and washed Kurnell Natural River sand with a maximum size of 2 mm were used as fine aggregates in the mixture. Australian standard AS-1141(2011) [3] along with the Regional Transportation Authority (RTA) (2006) and RMS (2015) codes [4] were applied in sampling and testing of the aggregates. The properties of Peppertree-P coarse sand and washed Kurnell Natural River sand are presented in Tables (6.3) and (6.4) respectively. Testing and sampling standards are also illustrated in those Tables.

Table 6.3: Properties of Peppertree-P coarse sand

Test method	Test	Allowable	Measured
AS-1141.11.1(2009) [5]	Passing A.S. sieve (%)		
	6.7 mm	100	100
	4.75 mm	95-100	100
	2.36 mm	73-88	80
	1.18 mm	55-70	55
	600 µm	35-50	37
	425 µm		30
	300 µm	25-35	23
	150 µm	10-20	11
AS-1289.4.3.1.	PH value of soil		8.8
AS-1012.20	Chloride as Cl (%)		0.007
	Sulphate as SO ₃		0.03
AS-1141.12 (2011)	≤ 75 µm (%)	5-10	5
AS-1141.13 (2007)	≤ 2 µm (%)	1	0.7
AS-1141.4 (2011) [9]	Uncompacted bulk density (t/m ³)		1.69
	Compacted bulk density (t/m ³)	≥ 1.2	1.86
RMS-T262	Moisture content (%)		0.3
AS-1141.6.1(2011) [10]	Particle density (dry) (t/m ³)	≥ 2.1	2.69
	Particle density (SSD) (t/m ³)		2.71
	Apparent Particle density (t/m ³)		2.76
	Water absorption	< 2.0	0.9
AS-1141.24(2013) [13]	Sodium sulphate soundness (Total weighted % loss)	< 5.0	0.9
	Fraction tested		
	-4.75 mm + 2.36 mm (% loss)		1.1
	-2.36 mm + 1.18 mm (% loss)		0.8
	-1.18 mm + 600 µm (% loss)		0.7
	-600 µm + 300 µm (% loss)		0.9
AS-1141.32 (2008) [14]	Weak particles (%)		
	The % of original sample passing 2.36 mm (sieve 3.4)	< 0.5	0.9
As-1141.25.2(2013) [15]	Degradation factor- fine aggregate	≥ 60	85
	The washed water after using permitted 500 ml		clear
AS-1141.34(2007) [17]	Organic impurities other than sugar	Not darker than std.	0.2 % , Pass
	The colour assessment was made visually using coloured reference glass		
AS-1141.35(2007)	Sugar	Negative	Not detected
RMS-T279	Method of determining voids content		
	% of voids	38-44	40.7
	The mean flow time (sec.)	21-27	27.3
ASTM-D7428	Micro-deval abrasion test		
	% loss		15.3
	The % loss of the control aggregate tested closest to		19.5
AS-1289.3.7.1	Sand equivalent of the soil using a power-operated	≥ 60	76
RMS-T659	Average Methylene blue adsorption value (mg/g)	5	3

Table 6.4: Properties of washed Kurnell fine sand

Test method	Test	Allowable	Measured
AS-1141.11.1 [5]	Passing A.S. sieve (%)		
	6.7 mm	100	100
	4.75 mm	90-100	100
	2.36 mm	60-100	100
	1.18 mm	30-100	100
	600 μm	15-100	99
	425 μm		91
	300 μm	5-50	58
	150 μm	0-20	2
AS-1289.4.3.1.	PH value of soil		8.8
AS-1012.20	Chloride as Cl (%)		0.003
	Sulphate as SO_3		0.09
AS-1141.12	$\leq 75 \mu\text{m}$ (%)	0-5	Nil
AS-1141.13	$\leq 2 \mu\text{m}$ (%)		Not applicable
AS-1141.31	Light particles (%)		Nil
AS-1141.4	Uncompacted bulk density (t/m^3)		1.33
	Compacted bulk density (t/m^3)	≥ 1.2	1.47
RMS-T262	Moisture content (%)		
AS-1141.6.1 [10]	Particle density (dry) (t/m^3)	≥ 2.1	2.52
	Particle density (SSD) (t/m^3)		2.55
	Apparent Particle density (t/m^3)		2.59
	Water absorption		1.0
AS-1141.24 [13]	Sodium sulphate soundness	< 5.0	
	Total weighted (% loss)	C,B2<12	0.9
	Fraction tested (-600 μm +300 μm (% loss)	A1,A2,B1<15	0.9
AS-1141.34 [17]	Organic impurities other than sugar	Not darker than std.	Pass
	The colour assessment was made visually using coloured reference glass		
AS-1141.35 [18]	Sugar	Negative	Not detected
AS-1141.33	Silt content (%)		3
SSD : Saturated surface-dry			

6.2.1.4. Artificial lightweight aggregate

The low-density foam of Expanded Polystyrene (EPS) is used as lightweight aggregate to reduce the density of concrete. EPS is a closed-cell lightweight cellular plastic material produced from polystyrene. The material has been modified by the addition of flame

retardant additives. Polystyrene is literally translated as “polymerized styrene”. It is the single styrene molecules that are chemically joined to form a large molecule which is called the polymer. Styrene is produced from benzene and ethylene, and polymerization is accomplished in the presence of catalysts, usually organic peroxides. The expandable foam is produced as small beads.

“BST” aggregate is a commercial name for the used spherical-shaped polystyrene beads with hydrophilic type chemical coating. The percentage passing of the BST beads on 4.75 mm and 2.36 mm sieves are 100% and 90% respectively. Bulk and particle density of the beads are 35 and 67 kg/m³ respectively. Figure (6.1) shows the BST aggregates and the fresh LWC concrete mixture containing the BST beads utilized in this study. The mechanical and chemical properties of EPS beads are comprehensively discussed in Chapter Three.



Figure 6.1: BST aggregates in fresh lightweight concrete

6.2.1.5. Admixtures

The concrete containing BST aggregates (spherical-shaped polystyrene beads) is commercially known as BST concrete. Water-reducing admixtures are used with BST concrete in general. Plastiment-20 in agreement with AS-1478.1 (2000) [19] is used as water reducing agent in the LWC mixture in this study. It is based on lignosulfonates. This admixture helps to achieve increased workability without loss of strength and also to increase strength without loss of workability. The time period during which the concrete can be placed is also extended. Melamine based products do not perform well with LWC containing BST. Sika-Australia supplies Plastiment-20 as a water reducing admixture in the concrete. Table (6.5) presents general information about the applications, advantages and storage conditions of this product.

Table 6.5: General information about the used water reducing admixture (Plastiment-20)

Description	Water reducer admixture based on selected lignosulfonates
Application	- To provide a cost effective solution in enhancing the water reduction - To improve the efficiency of the water content in concrete mixes - To achieve increased workability without loss of strength - To increase strength without loss of workability
Advantages	- Improved Workability and Finish-ability - Higher slump can be achieved without further addition of water - Easier and quicker placing and compaction - Excellent Strength Performance - Enhances the strength gained at all ages - Homogeneous concrete with less shrinkage requiring lower pumping pressure - Improve plastic concrete properties - Improves cohesiveness with improved placement properties and minimises the segregation and bleeding problems. - Retardation without loss in strength
Storage and shelf-life	- Between 0°C and 35°C in unopened original containers - Protected from direct sunlight and frost - Shelf-life is at least one year.
How to use	- Added with the initial batching water. Never added to dry mix - Recommended dosage is 350 ml $\pm$ 100 ml per 100 kg of total cementitious material, Optimum dosage should be determined by site trials with the particular concrete mix design

6.2.1.6. Cement

BST concrete mix design is based on the Shrinkage Limited (SL) cement confirming AS-3972 (2010) [20]. This cement is a special and modified type of General Purpose (GP) cement. Other types of cement are suitable, but their effect on yield and performance of the mixture should be determined. The chemical, physical and mechanical properties of the used SL (GP) type cement are presented in Table (6.6). These properties are in agreement with the specifications defined in AS-2350 (2006) [21]. It is worth mentioning that the used SL type cement contains up to 7.5% additional limestone mineral powder.

Table 6.6: Properties of the Shrinkage Limited (SL) type cement

Test method		Allowable	Measured
Chemical components			
AS-2350.2 (2006) [22]	LOI (%)	4.3	4.1
	SO ₃ (%) (AS-2350.2)	< 3.5	3.1
	CI (%) (AS-2350.2)	< 0.10	0.02
	CaO (%)	64.3	62.6
	SiO ₂ (%)	19.4	19.26
	Al ₂ O ₃ (%)	5.15	5.15
	Fe ₂ O ₃ (%)	3.18	3.08
	MgO (%)	1.59	1.14
	K ₂ O (%)	0.51	0.53
	Na ₂ O (%)	0.18	0.08
	P ₂ O ₅ (%)	0.1	0.05
	Mn ₂ O ₃ (%)	0.08	0.06
	Cr ₂ O ₃ (%)	0.01	0.01
	SrO (%)	0.05	0.04
	NaEq (%)	0.4	0.3
Physical properties			
AS-2350.8(2006) [23]	Fineness index (m ² /kg)	441	395
AS-2350.3 (2006) [24]	Normal consistency (%)		27.7
Mechanical properties			
AS-2350.13 (2006) [25]	Residue (45 µm)	2.4	
	Drying shrinkage - 28 days mean (µε)	< 750	570
AS-2350.11 (2006) [26]	Compressive strength		
	f'_c - 3 days (MPa)		34
	f'_c - 7 days (MPa)	35	45.6
	f'_c - 28 days (MPa)	45	61.1
AS-2350.4 (2006) [27]	Setting time		
	Initial (minute)	≥ 45	105
	Final (minute)	< 600	150
AS-2350.5 (2006) [28]	Soundness	<5 mm	1mm

6.3. MIXTURE PROPORTIONS

The consistent materials and corresponding mixture proportions that used to achieve a structural concrete with low density in the range of 2000 kg/m³ is presented in Table (6.7). The quantities are given for each cubic metre of the LWC mixture.

Table 6.7: Mix design of lightweight concrete containing expanded polystyrene beads (SSD condition)

Component		Proportion
Cement (kg/m ³)		500
Water (litre/ m ³)		180
Water to cement ratio (w/c)		0.36
BST aggregate (Litre/ m ³)		300
Fine aggregate	Coarse sand (kg/ m ³)	310
	Fine sand (kg/ m ³)	310
Coarse aggregate (kg/ m ³)		800
Admixture (litre/ m ³)		2
Mixture volume (m ³)		1.3
Slump (mm)		130
Fresh density (kg/m ³)		2000

6.4. PREPARATION AND CURING CONDITION OF THE TEST SPECIMENS

All specimens were cast within 15 minutes after the end of the mixing process. The fresh concrete was poured into the moulds in 2 or 3 layers following the seconds of vibration by the vibrating table. Vibrating the moulds over 5 seconds causes the EPS beads to float up to the concrete surface. After casting, specimens were carefully moved and kept covered in a controlled chamber at 23 ± 2 °C conforming AS-1210.8.1 [29] up to the demoulding time after 24 hours. Then, all test specimens were stored in water pre-saturated with lime until the measurement time. Test ages are illustrated in Table (6.8). All tests were performed using different sizes of the cylinder, cube and prism specimens. For further investigation of the effects of curing conditions on shrinkage behaviour, various curing conditions were applied to the shrinkage prisms that will be explained in the following sections.

6.5. TEST METHODS OF THE HARDENED LIGHTWEIGHT CONCRETE

The test ages, shape and size of the specimens for hardened LWC are shown in Table (6.8). Three LWC specimens have been tested per each age and property.

Table 6.8: Shape and size of the LWC test specimens and test ages

Property	Cylinder (mm)	Cube (mm)	Prism (mm)	Test ages (days)
Compressive strength	75×150, 100×200,150×300	100, 150		3,7, 14, 21, 28, 56, 91
Modulus of elasticity	150×300			3,7, 14, 21, 28, 56, 91
Modulus of rupture			100×100×400	3,7, 14, 21, 28, 56, 91
Splitting tensile strength	100×200,150×300			3,7, 14, 21, 28, 56, 91
Steel-concrete Bond	150×300			3,7, 14, 21, 28, 56, 91
Pull-out test	150×300			7, 14, 28, 56, 91
Stress-strain				3,7, 14, 21, 28, 56, 91
Creep	150×300			Up to 210 days
Shrinkage			75×75×280	Up to 450 days

Australian codes and some international codes are applied to determine the mechanical properties of LWC in this study. Table (6.9) explains the applied Australian standards and international codes for each type of the test under laboratory conditions.

Table 6.9: Applied codes on the curing, sampling and testing the LWC specimens

Property	Test method	Property	Test method
Compressive strength	AS 1012.8.1 (2014) , ASTAM C39 (2000)	Modulus of rupture	AS 1012.8.2 (2014), ASTM C1018 (2002)
Splitting Tensile strength	AS 1012.8.1(2014), ASTM C496 (2000)	Steel-concrete Bond	RILEM –RC6 (2004)
Creep	AS 1012.16 (2014)	Pull-out test	RILEM –RC6 (2004)
Shrinkage	AS-1012.13 (2014)	Modulus of elasticity	AS 1012.17 (2014), ASTM C469 (2002)

6.6. PROPERTIES OF THE FRESH LIGHTWEIGHT CONCRETE

The required measurements of LWC in a fresh state are generally similar to those of CC, however it is almost entirely different from the fresh state of SCC. The specific gravity and slump values are the measured parameters of fresh LWC in this study. The experimental tests required for the LWC are implemented worldwide under laboratory conditions. These experimental investigations evaluate the density and workability of

fresh LWC. Table (6.10) explains the Australian standards AS-1012 (2014) [30] applied to curing, sampling, slump test and mass per volume measurements of the fresh LWC.

Table 6.10: Test methods to determine the fresh properties of LWC

	Curing	Sampling	Slump test	Mass per volume
Test method	AS-1012.8 (2014)	AS-1012.1 (2014)	AS1012.3.1 (2014)	AS1012.5 (2014)

6.7. EXPERIMENTAL TEST RESULTS OF FRESH AND HARDENED LIGHTWEIGHT CONCRETE

6.7.1. Slump and Fresh Density

According to test results, the slump value for the LWC was 130 mm. This slump value provided the required flow-ability and workability to pour fresh concrete in the test specimens and the reinforced concrete slabs. Density of the concrete in the fresh state was 2000 kg/m³. Additionally, to avoid any blocking problem, the maximum size of the coarse aggregate was restricted to 10 mm in the mixture.

6.7.2. Compressive Strength

Figures (6.2) and (6.3) show the testing set-up and the EPS-LWC specimens used to determine the compressive strength. Table (6.11) presents the recorded values of compressive strength in EPS-LWC at seven different ages. The compressive strength of different shape and size of the specimens, together with the effects of size and age are discussed in the following sections.

The Avery (UK, Birmingham) hydraulic testing machine with capacity of 50 tonnes was used to determine the compressive strength and modulus of elasticity of concrete. According to the instructions of the testing machine, determination of the compressive strength, the loading rates were different for the different shapes and sizes of the specimens. For example, the cylindrical specimens with dimensions of 150 mm × 300 mm were loaded in the testing machine under controlled loading at the rate of 0.3 MPa/sec until failure.

Table 6.11: Compressive strength of lightweight concrete mixture at different ages

Age (day)	3	7	14	21	28	56	91
Compressive strength (MPa)	14.49	25.38	27.93	29.62	30.96	32.33	33.3



Figure 6.2: Testing setup and different sizes and shapes of the specimens for compressive strength tests

The results show that the compressive strength at 3 and 7 days are 46.79% and 82.97% respectively of the compressive strength at the age of 28 days. Also the corresponding compressive strength values at ages 56 and 91 days are 4.4% and 7.64% higher than that of 28 days respectively. In other words, the main development of compressive strength up to 45% occurs at early age (3 days).

Equation (6.1) is proposed in this study to explain the relationship between the compressive strength of EPS-LWC at different ages with respect to the strength at 28 days.

$$f'_c(t) = (0.1633 \ln t + 0.417) f'_{c-28} \quad \text{Eq. (6. 1)}$$

Where; $f'_c(t)$ is the compressive strength (MPa) at age t (days) and f'_{c-28} is the compressive strength (MPa) at age 28 days.

Figure (6.3) compares the predicted values against experimental values of the compressive strength at various ages. Equation (6.1) underestimates the values for compressive strength under the age of 28 days, while it slightly overestimates the compressive strength for 56 and 91 days. However, the differences between the predicted and experimental value are not significant.

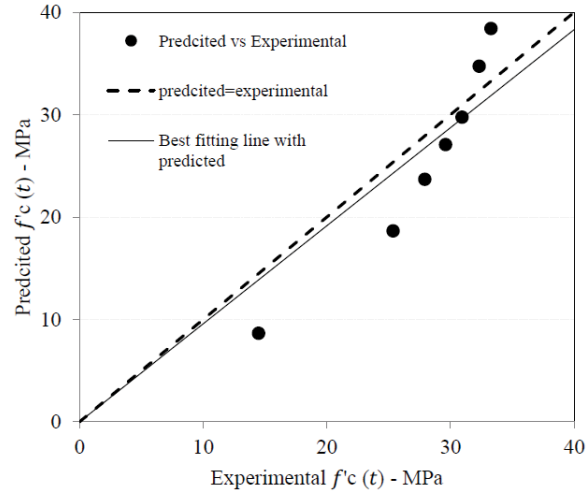


Figure 6.3: Experimental vs. predicted values of compressive strength by Eq. (5.1)

6.7.2.1. Effects of shape and size of test specimens on the compressive strength

Different size and shapes of concrete specimens were used to determine the compressive strength at different ages. Totally, 48 cylindrical specimens (18 cylinders with 100mm×200 mm, 18 cylinder specimens with 150mm×300 mm and 12 cylindrical specimens with 75×150 mm dimensions) and 36 cubic specimens (18 cubes with 100 mm and 18 cubes with 150 mm dimensions) were used to investigate the effect of size and shape of the specimen on the measured compressive strength of EPS-LWC. Figure (6.4) shows the schematic presentation of the shape and size of the test specimens.

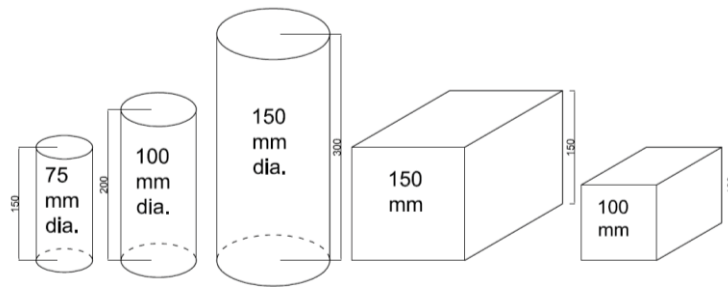


Figure 6.4: Different size and shapes of the compressive strength test specimens

The measured values of compressive strength at ages 3, 7, 14, 21, 28, 56 and 91 days for three sizes of cylindrical specimens and 2 sizes of cubic specimens are presented in Figure (6.5).

An overall comparison of the diagrams for the cubic and cylindrical specimens in Figure (6.5) reveals that the compressive strength values measured by the cube specimens

are higher than those for cylindrical specimens. However, the difference in compressive strength in the cylinder with 75mm×150 mm dimension and the cube with 100 mm dimension is not large. The cylinder with 150mm×300 mm dimension has the lowest compressive strength, while the cube with 100 mm dimension shows the highest compressive strength in the majority of the test ages. By comparing the average values of the compressive strength at all ages, the ratio of compressive strength of the 100 mm cube specimen to the compressive strength of the 150 mm cube specimen is 1.056. The ratio of the compressive strength in the 100 mm cube specimen to the compressive strength of the 75mm×150 mm, 100mm×200 mm and 150mm×300 mm cylindrical specimens are 1.007, 1.11 and 1.16 respectively.

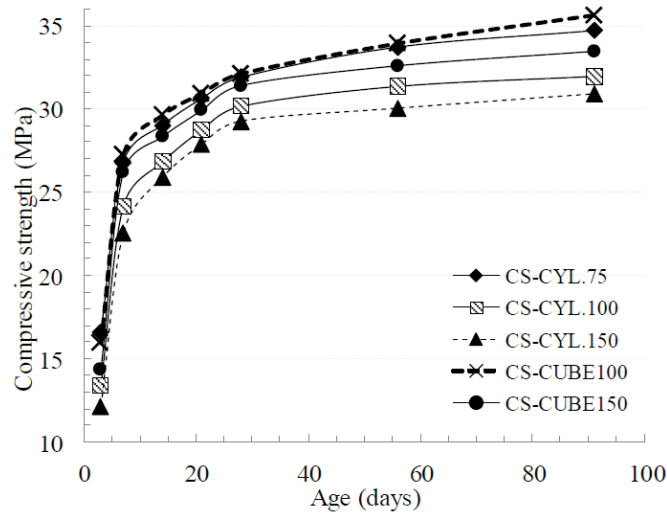


Figure 6.5: Compressive strength of LWC by different size and shape of test specimens

It is worth mentioning that the cylinder of 75mm×150 mm is not recommended for measuring of the compressive strength of concrete either in Australian or other international codes of practice. However, considering the identical mix design, testing methods and testing machines in all specimens, it can help evaluate the results by comparing other sizes and shapes of EPS-LWC specimens.

6.7.3. Splitting Tensile Strength

Figure (6.6) shows the specimen and testing equipment used to measure the splitting tensile strength of EPS-LWC in the laboratory conditions.



Figure 6.6: Specimen and test setup of splitting tensile strength

Table (6.12) presents the measured values of the splitting tensile strength at different ages. Development of the splitting tensile strength with age is represented by the proposed Equation (6.2) in this study.

Table 6.12: Splitting tensile strength of LWC mixture at different ages

Age (day)	3	7	14	21	28	56	91
Splitting tensile strength (MPa)	1.44	2.3	2.6	2.78	2.89	2.98	3.08

$$f_{ct}(t) = (0.156 \ln t + 0.4341) f_{ct-28} \quad \text{Eq. (6.2)}$$

Where; $f_{ct}(t)$ is the splitting tensile strength (MPa) at age t (days), and f_{ct-28} is the splitting tensile strength (MPa) at age 28 days.

Figure (6.7) compares the predicted and experimental values of the splitting tensile strength at various ages. Equation (6.2) gives underestimated values for the splitting tensile strength under age 28 days, while it slightly overestimates the splitting tensile strength for 56 and 91 days. However, the differences between the predicted and experimental value are not significant.

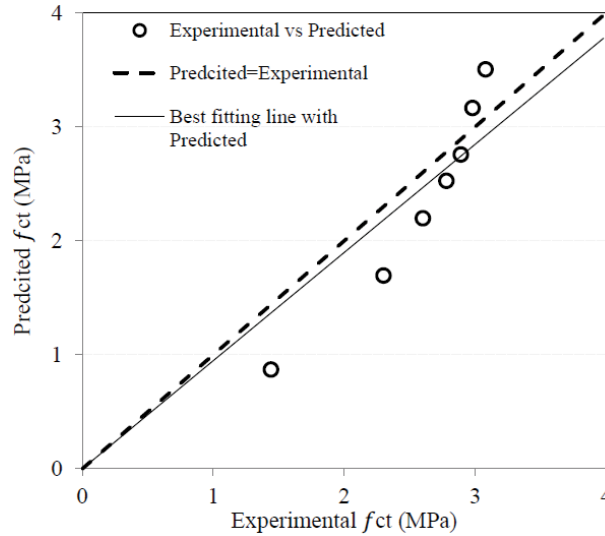


Figure 6.7: Experimental vs. predicted values of splitting tensile strength by Eq. (5.2)

6.7.3.1. Effect of shape and size of test specimens on splitting tensile strength

A total of 36 cylindrical specimens of two different sizes (18 cylinders with 100mm×200 mm and 18 cylinders with 150mm×300 mm dimensions) were used to measure the splitting tensile strength of EPS-LWC.

Figure (6.8) shows the tensile strength test results for cylindrical specimens with 100mm×200 mm and 150mm×300 mm dimensions. The cylinder with the smaller diameter gives about 7% higher tensile strength. Despite using the same concrete mixture the different sizes of cylinder specimens show different values of splitting tensile strength at the corresponding ages. Development rate of the splitting tensile strength with age is also influenced by the size of cylinder specimen.

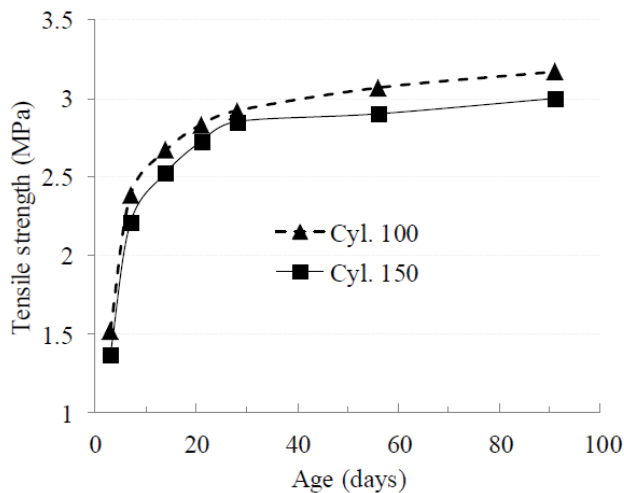


Figure 6.8: Splitting tensile strength of cylindrical specimens

6.7.4. Modulus of Elasticity

Cylindrical specimens with dimension of 150mm×300 mm are utilized to measure the modulus of elasticity of LWC containing EPS beads at different ages. Three specimens per age are tested in the concrete laboratory. Figure (6.9) shows the testing set-up and the specimens used to measure the modulus of elasticity of EPS-LWC.



Figure 6.9: Specimen and test setup of modulus of elasticity

Table (6.13) shows the measured values of the modulus of elasticity of EPS-LWC at different ages. The results show that the recorded values of the modulus of elasticity at 3, 7 and 91 days are 57%, 77% and 107% of the modulus of elasticity at age 28 days respectively.

Table 6.13: Modulus of elasticity of LWC mixture at different ages

Age (day)	3	7	14	21	28	56	91
Modulus of elasticity (GPa)	11.63	15.68	17.65	19.48	20.32	21.1	21.89

Figure (6.10) shows development of the modulus of elasticity of LWC by age. This development by age can be expressed by Equation (6.3). In fact, the equation represents the age factor in which the ratio of the modulus of elasticity at different ages to the modulus of elasticity at age 28 days is illustrated.

$$E_c(t) = (0.1467 \ln t + 0.4666) E_{c-28} \quad \text{Eq. (6.3)}$$

Where; $E_c(t)$ is the modulus of elasticity (GPa) at age t (days) and E_{c-28} is the modulus of elasticity (GPa) at age 28-days.

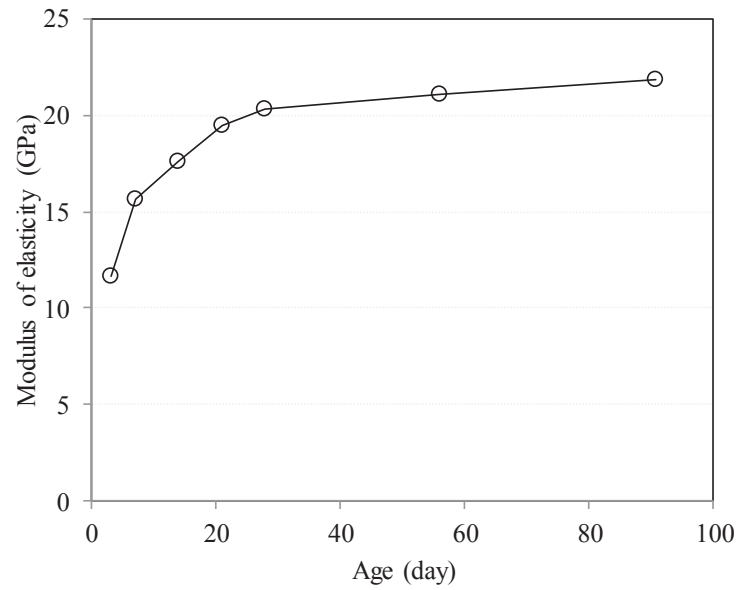


Figure 6.10: Development of modulus of elasticity of lightweight concrete with age

Figure (6.11) compares the predicted values against the experimental values of the modulus of elasticity at different ages. Equation (6.3) gives underestimated values for the modulus of elasticity at age 28 days, while it overestimates the modulus of elasticity at 56 and 91 days. However, the predicted and experimental values are very close.

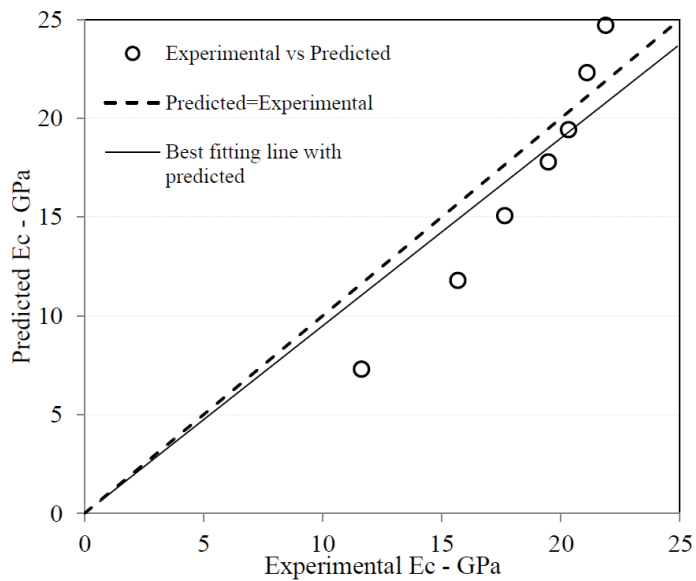


Figure 6.11: Experimental vs. predicted values of modulus of elasticity by Eq. (6.3)

6.7.5. Modulus of Rupture (Flexural Tensile Strength)

The modulus of rupture (flexural tensile strength) test of EPS-LWC was conducted by a prism with dimensions of 100 mm × 100 mm × 350 mm under third-point loading. The main objectives of this test were to determine the flexural toughness and first crack strength of LWC. Figure (6.12) shows the specimen and testing equipment used to measure the modulus of rupture (flexural tensile strength) of EPS-LWC.



Figure 6.12: Specimen and testing equipment of flexural tensile strength

Table (6.14) presents the measured modulus of rupture of EPS-LWC at different ages. The ultimate rupture load, applied from the testing machine, is also given in Table (6.14). According to the test results, the modulus of rupture at ages 3, 7 and 91 days are 69%, 81% and 105% of the modulus of rupture at age 28 days respectively. In other words, EPS-LWC gains its flexural tensile strength mainly at the early ages.

Table 6.14: Modulus of rupture of lightweight concrete at different ages

Age (day)	3	7	14	21	28	56	91
Ultimate rupture load (KN)	9.94	11.66	13.52	14.06	14.35	14.58	15.08
Modulus of rupture (MPa)	2.982	3.498	4.056	4.218	4.305	4.374	4.524

Figure (6.13) shows the general development of the modulus of rupture of EPS-LWC by age. This development by age can be expressed by the proposed Equation (6.4) in this study. In fact, the equation represents the age factor in which the ratio of the modulus of rupture at different ages in relation to the modulus of elasticity at the age of 28 days is explained.

$$f_r(t) = (0.1046 \ln t + 0.62) f_{r-28} \quad \text{Eq. (6. 4)}$$

Where $f_r(t)$ is the modulus of rupture (MPa) at age t (days) and f_{r-28} is the modulus of rupture (MPa) at age 28-days.

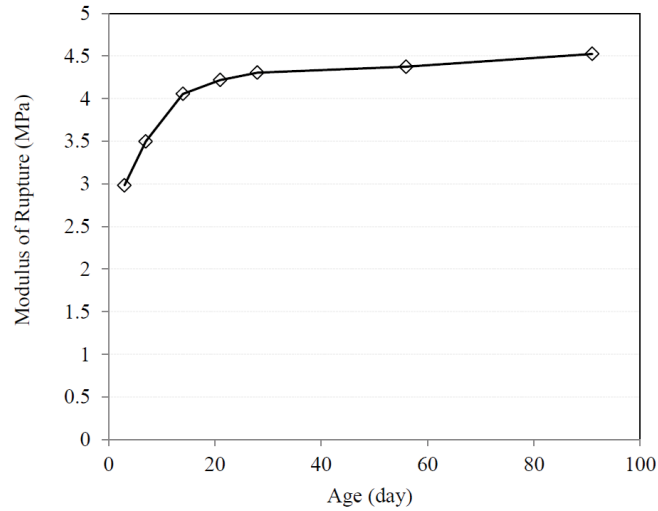


Figure 6.13: Development of modulus of rupture of lightweight concrete by age

Figure (6.14) compares the predicted and experimental values of the modulus of rupture at different ages. Equation (5.4) gives very close predicted values for the modulus of rupture, especially over the age of 28 days.

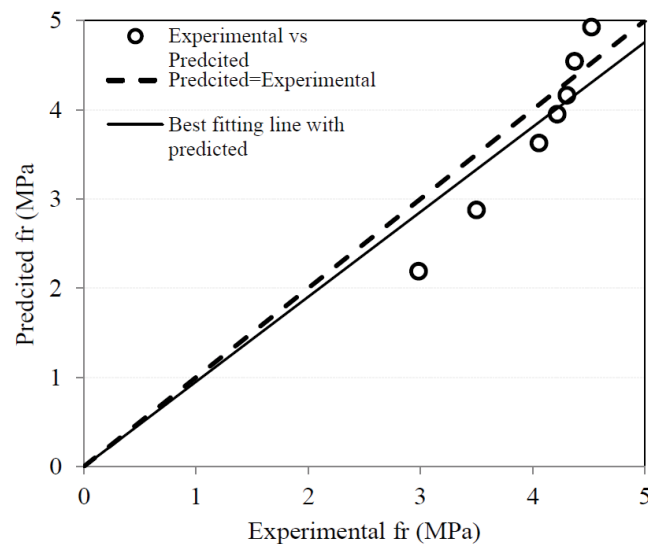


Figure 6.14: Experimental vs. predicted values of modulus of rupture by Eq. (5.4)

To investigate the mid-span deflection of the test specimens under the third-point loading, a transducer was attached to the prism specimen. The mid-span deflection was

the average of the ones detected by the transducers through the contact with brackets attached to the prism specimen. Figure (6.15) compares the load-deflection diagram at mid-span of the test prisms at ages 7, 28 and 56 days.

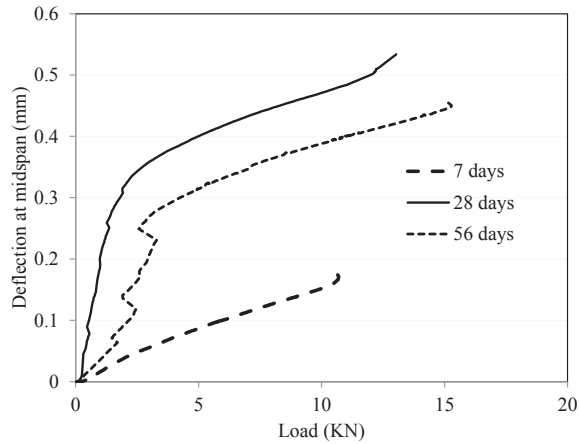


Figure 6.15: Load-deflection in mid-span of the flexural test specimens at different ages

The rupture load at 56 days is about 2% higher than that of 28 days; however, the mid-span deflection at 28 days is about 25% higher. This difference might be explained as result of the higher flexural stiffness of the specimen at 56 days. The rupture load and maximum deflection at age 7 days are considerably below those of 28 days.

6.7.6. Drying Shrinkage

The drying shrinkage test was conducted on the EPS-LWC prisms with 75 mm × 75 mm × 280 mm dimensions subjected to air-drying in the laboratory controlled conditions. The main objective of this test is to determine the shrinkage strain after a long period of drying which will be used in the time-dependent deflection calculation of EPS-LWC slabs. Also the interaction between the weight loss of specimens and the shrinkage strain at different ages is investigated.

Figure (6.16) shows the specimen and test setup used to measure the shrinkage strain of EPS-LWC in this study.



Figure 6.16: Shrinkage specimens and testing equipment

6.7.6.1. Curing and drying condition of shrinkage test at different ages

To study the drying shrinkage strain and weight loss of the shrinkage prisms, a total of nine specimens in 3 series of A, B and C are investigated. To explore the effect of curing duration on the shrinkage behaviour of specimens, each group containing 3 specimens was subjected to specific curing conditions. Different conditions of initial curing, drying and rewetting were applied to the specimens as illustrated in Figure (6.17). The relative humidity and temperature of the environment were kept at $50\% \pm 5$ and $23 \pm 2^\circ\text{C}$ respectively.

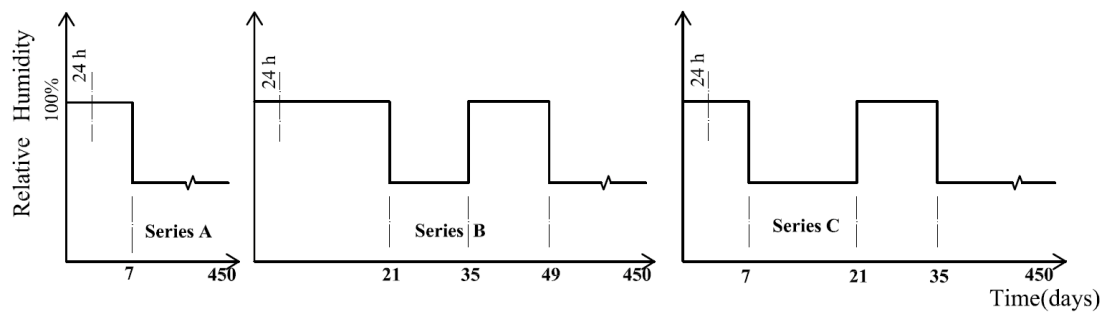


Figure 6.17: Initial curing, drying and rewetting conditions of shrinkage prisms

Table (6.15) shows the measured values of shrinkage strain and weight of the specimens at different ages after one-week of initial curing. According to Table (6.15), the shrinkage and weight measurement in the specimens of series A, B and C was initiated at 7, 21 and 35 days after pouring the concrete in the moulds. The specimens of series B were maintained in water between the days 35 and 49 after casting, so, their shrinkage and weight were not measured in that period. The presented values in Table (6.15) are the average of the 3 specimens in each series.

Figure (6.18) shows the development of shrinkage strain with time. Although the B and C series have been stored for a longer time in the water, there is a significant difference in the recorded strains, especially at early ages. It emphasizes the effect of the

longer curing time to reduce the drying shrinkage. Specimens of series C are subjected to drying, followed by two weeks of being stored in water, with the shrinkage and weight measurement beginning after the second stage of rewetting.

In the specimens of series B, which had a longer initial curing time than the other specimens, there is a reduction in drying shrinkage (swelling) after the second stage of rewetting. Swelling is defined as the expansion of concrete specimen due to water absorption in the cavities. It can be concluded that the better initial curing improves the capability of the specimen recovering the shrinkage strain, particularly at the early ages.

Table 6.15: Recorded shrinkage strain and weight of specimens at different ages

Time (day) after initial curing	Shrinkage strain ($\mu\epsilon$)			Weight of specimen (gr)		
	Specimen series			Specimen series		
	A	B	C	A	B	C
0	0	0	0	3399	3354.9	3321
1	140		168	3367		3289
15	658	536	685	3319	3301	3240
18	711	605	744	3316	3296	3237
19	741	642	780	3314	3293	3235
20	771	673	805	3312	3291	3234
21	793	696	825	3311	3289	3231
29	858		887	3307		3228
33	904		942	3305		3226
42	963	299	1002	3298	3353	3219
47	978	452	1006	3297	3318	3218
49	1001	546	1031	3296	3313	3217
57	995	735	1042	3295	3302	3216
71	1010	858	1061	3294	3293	3213
78	1029	902	1081	3293	3290	3212
88	1058	993	1111	3291	3286	3210
103	1039	964	1094	3291	3283	3210
112	1053	984	1111	3292	3282	3210
132	1100	1043	1167	3291	3278	3208
159	1121	1092	1188	3289	3274	3206
201	1173	1158	1235	3286	3268	3202
300	1212	1192	1268	3271	3249	3188
393	1239	1212	1296	3256	3234	3173
425	1253	1215	1311	3248	3227	3166
450	1264	1216	1322	3245	3224	3162

According to Figure (6.18), the recorded shrinkage strain in all specimens is in the range of 1200-1400 micro-strains. The average ultimate shrinkage strain in series B after 450 days is 96% of the average ultimate shrinkage strain of series A. The specimens of series C represent relatively a higher shrinkage strain among all series.

Based on the commencement age of the measurement series C, the average ultimate shrinkage strain after 450 days is 5% higher than that of series A specimens. The results confirm that the major part of the shrinkage strain up to 50% occurs at early ages of the concrete.

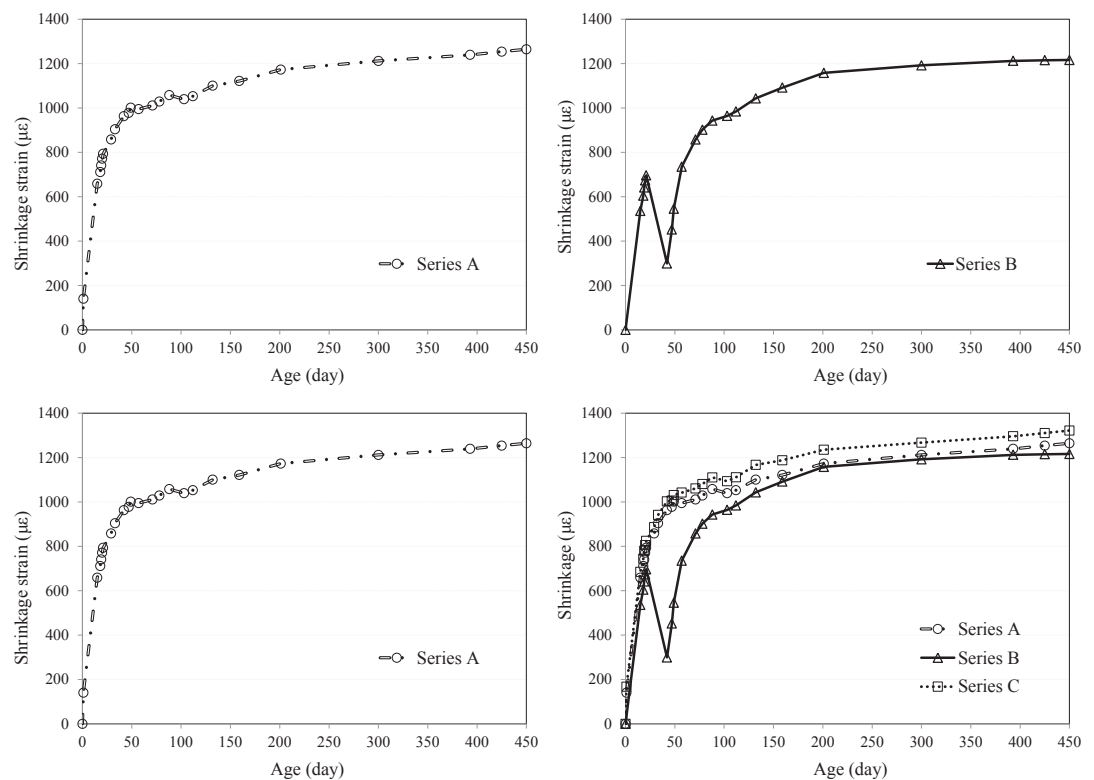


Figure 6.18: Shrinkage variation in series A and C and shrinkage and swelling in series B

Figure (6.19) shows changes in the weight of specimens at different ages. As a matter of interest, the rewetting in specimen B has caused an increase in the total weight due to the water absorption.

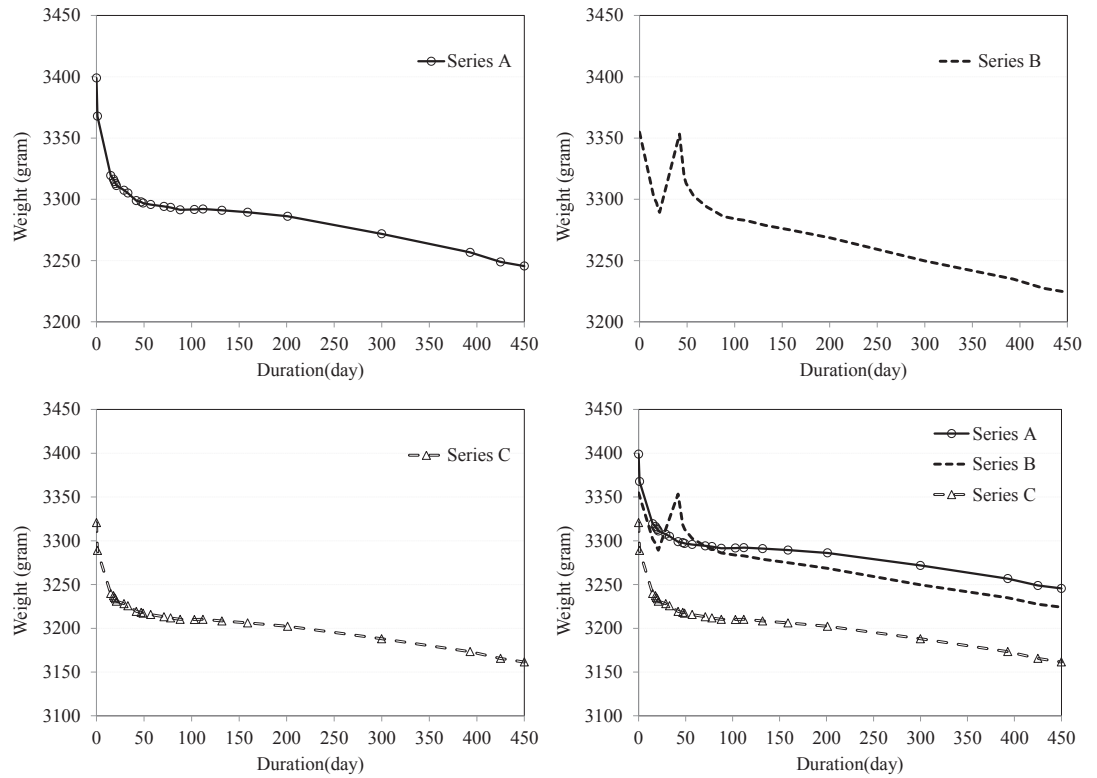


Figure 6.19: Weight of shrinkage test prisms at different ages

Weight loss (ΔW) and the ratio to the initial weight (W_0) of specimens ($\Delta W/W_0$) are two principal factors in studying the effect of specimen weight on the shrinkage behaviour. Considering the drying shrinkage, the main part of the weight loss of specimens is due to the loss of water. On the other hand, water loss is strongly related to the relative humidity of the environment which affects the shrinkage strain.

Figure (6.20) shows the weight loss of specimens at different ages. The weight loss follows a similar trend in series A and C after about 100 days from casting. However, due to the applied various conditions of drying and rewetting, the weight loss in series B is completely different, especially after rewetting. Following a significant weight recovery after rewetting, the rate of weight loss increases in specimen series B, but the final lost weight is still less than that of specimen series A. The final weight loss in specimens of series B and C are 50% and 43.5% below the weight loss of specimen A respectively.

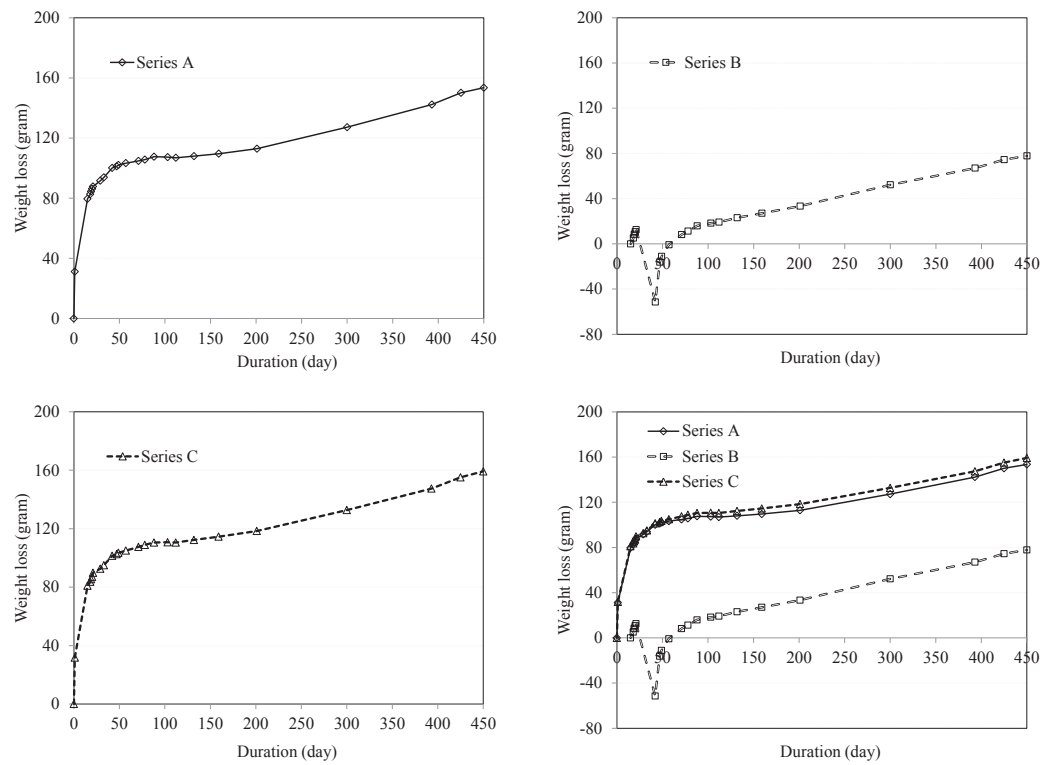


Figure 6.20: Rate of weight loss of shrinkage specimens

Figure (6.21) shows the rate of weight loss of specimens at different ages. Despite similar trends in specimens of series A and C, the rate of weight loss is entirely different in specimen B. The rate of weight loss in all specimens tends to be zero after about 100 days. However, there is a sharp drop in the rate of weight loss in specimens A and B at first two weeks. In the specimen B, which had a longer curing time, the difference between the initial and ending rate of the weight loss is considerably less than that of specimen A.

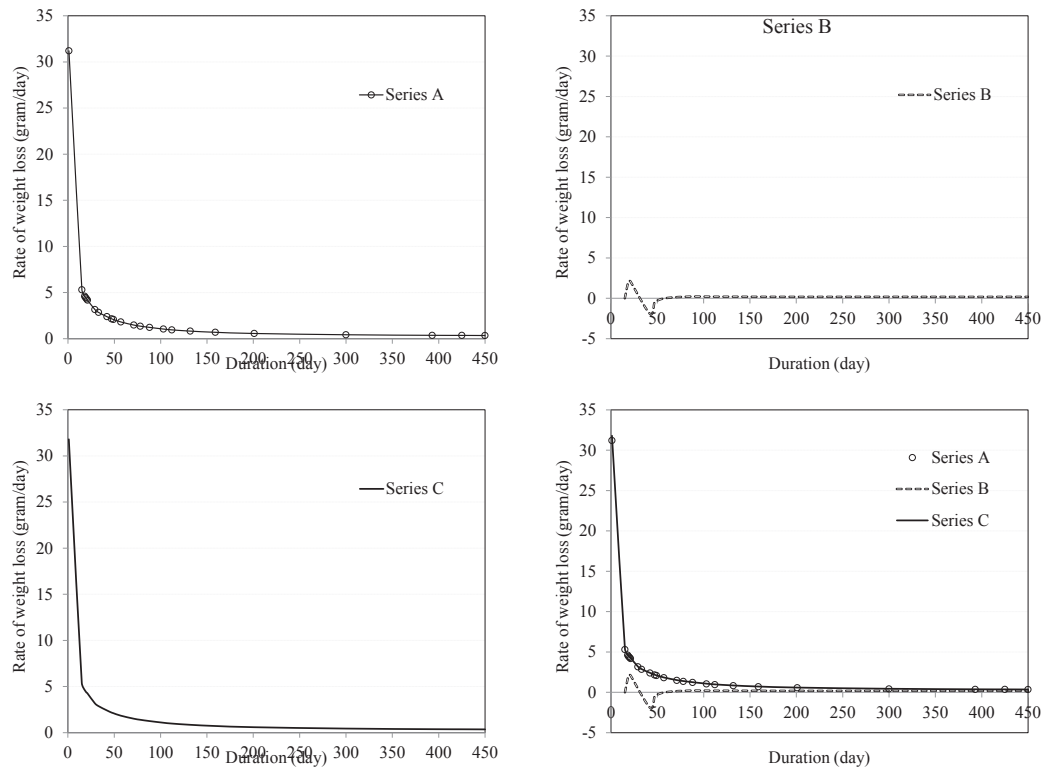


Figure 6.21: Variation of weight loss rate with age of specimens

Figure (6.22) shows the relationship between the shrinkage strain and weight loss of the specimens. In specimen of series A, there is a large increment of shrinkage, increasing the weight loss of specimens up to 120 grams. After that point, despite the increasing weight loss, the shrinkage strain variation changes to a steady state. Specimen of series B experiences a similar trend, however, the effect of the extended initial curing on the weight recovery and the stability point of the shrinkage is evident in Figure (6.22).

Since the specimens had different initial weights before the drying shrinkage test, the independent weight loss of specimens cannot be a good indicator of shrinkage behaviour. Therefore, the ratio of weight loss (ΔW) of specimens to their initial weight (W_0) is investigated. Figure (6.23) shows the relationship between $\Delta W/W_0$ ratio and the shrinkage strain of specimens at different ages.

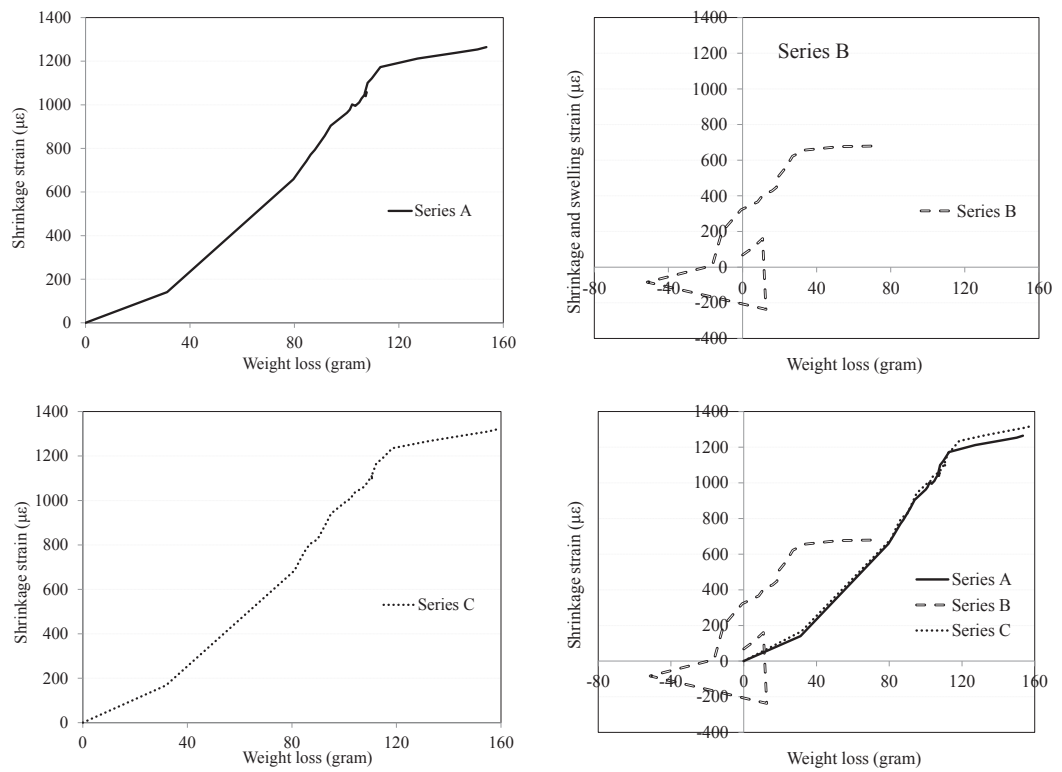


Figure 6.22: Weight loss of specimens versus shrinkage strain

The general trend of diagrams is similar in Figures (6.22) and (6.23). However, the effect of $\Delta W/W_0$ ratio on the shrinkage of series A is higher than other specimens.

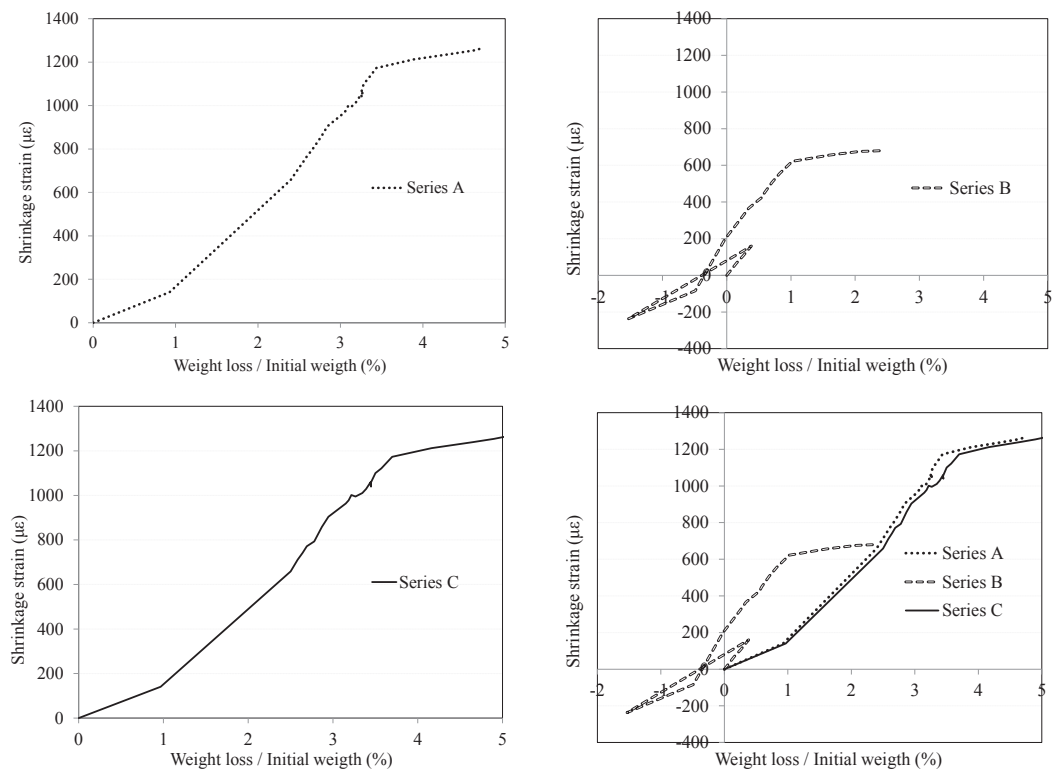


Figure 6.23: Lost weight per initial weight versus shrinkage strain

Apart from the ultimate shrinkage strain value, the increasing rate of shrinkage strain might have a significant effect on the performance of concrete structures, particularly at the early ages of construction. In another hand, the rate of water loss (weight loss of concrete member) can complicate the prediction of the shrinkage strain. Figure (6.24) shows the relationship between the shrinkage rate and the rate of water loss in the test specimens.

The trend in specimen A can be illustrated by a bilinear diagram, while the changes in the shrinkage rate with different rates of weight loss are almost linear. The maximum rates of weight loss and shrinkage rate in specimen B are about 10% and 25% of those for specimen A respectively.

Since the significant part of the weight loss of specimen C happened in the first drying period before the measurement, the weight loss and consequently, the rate of weight loss are not comparable with those parameters in specimens A and B.

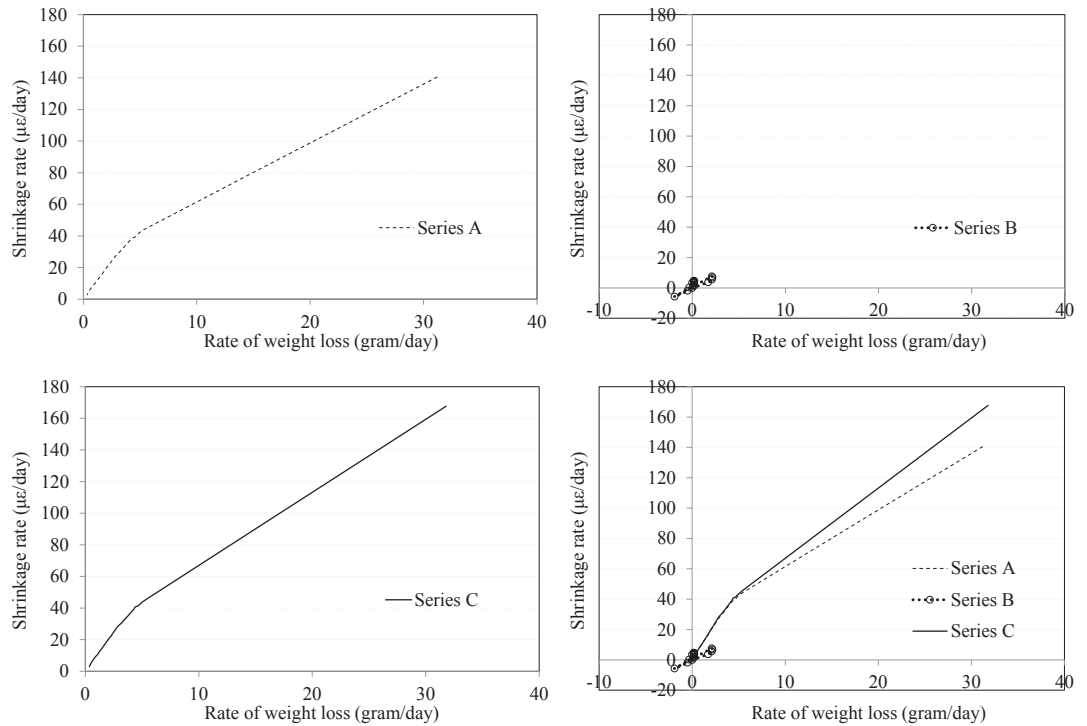


Figure 6.24: Rate of weight loss versus shrinkage rate

To study the effect of the increasing rate of the $\Delta W/W_0$ ratio (percentage of weight loss per day) on the shrinkage rate, Figure (6.25) compares the relationship between these two factors in all specimens. Similar to Figure (6.24), the relationship in specimens A and C can be illustrated by a bilinear diagram and the behaviour of specimen B by a multi-linear diagram. There is no particular trend in specimen C. The maximum rate of percentage of

weight loss in specimen B is about 10% and 25%, compared to specimens in series A. This rate is in agreement with the ratio of the maximum rate of weight loss in Figure (6.24).

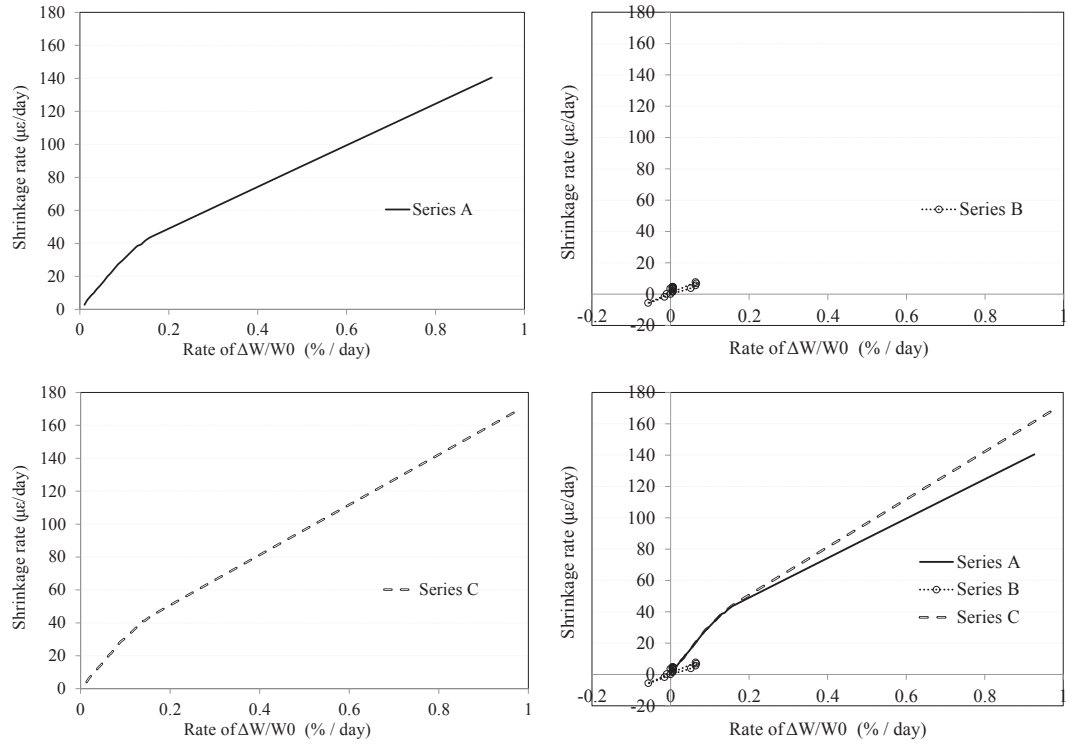


Figure 6.25: Rate of lost weight per initial weight versus shrinkage rate

6.7.6.2. Proposed analytical models

The mathematical models are effective tools used to express the properties of materials in absence of the accurate tests. These models are generally proposed on the basis of the most-compatible of the previously developed models in the literature and are verified by the experimental data. There are different models used in literature to predict the shrinkage strain of concrete at different ages. Tables (4.2) and (4.3) in Chapter 4 show some existing shrinkage models for LWC and conventional concrete.

Figure (6.26) compares the measured experimental data from the shrinkage specimens of this study versus the predicted values of the existing models presented in Table (4.2). There are obvious differences between the predictions of the models and measured values. According to Figure (6.26), all investigated models underestimate the shrinkage of EPS-LWC. The models in AS-3600(2009) [31] and Best and Polivka (1959) [32] give more reasonable predictions, however, differences between the measured experimental data and the prediction of AS-3600(2009) [31] is increasing by time. ACI-209R-92

(2008) [33] significantly underestimates the shrinkage strain of EPS-LWC at all ages, while ACI-435-R95 (2003) [34] gives shrinkage values between the predictions of AS-3600(2009) and ACI-209R-92 (2008).

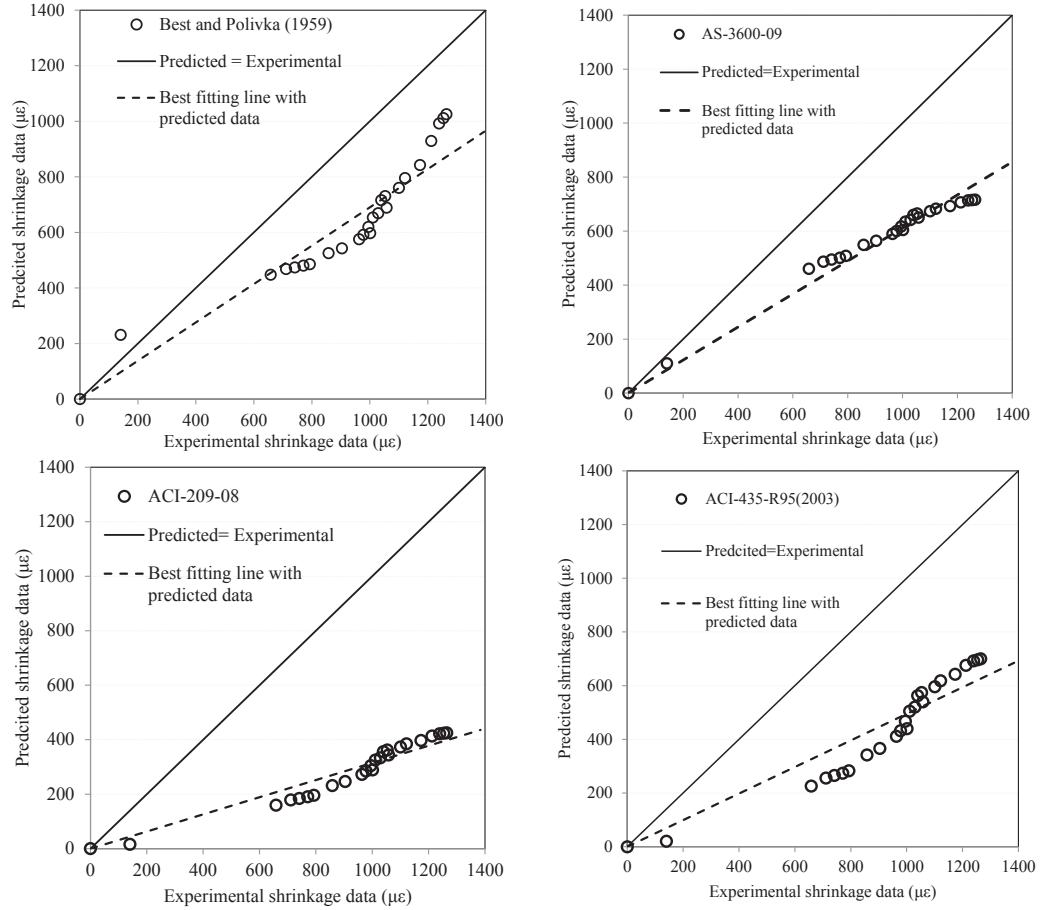


Figure 6.26: Experimental data of EPS-LWC versus predicted shrinkage strain by different models

However, the recorded shrinkage strains in this study are well-predicted by the model proposed by Best and Polivka (1959) [32]. Therefore, it is selected as the base of a new empirical model for describing the development of shrinkage strain of EPS-LWC as presented in Equation (6.5).

$$\varepsilon_{sh} = 188 \times t^{0.315} \quad \text{Eq. (6.5)}$$

Where; ε_{sh} is the shrinkage strain ($\mu\varepsilon$) and t is the age of shrinkage specimen (days) after one week moist curing.

Figure (6.27) compares the accuracy of the proposed model in this study with the experimental data and predictions of the presented models in the literature as presented in Table (4.2).

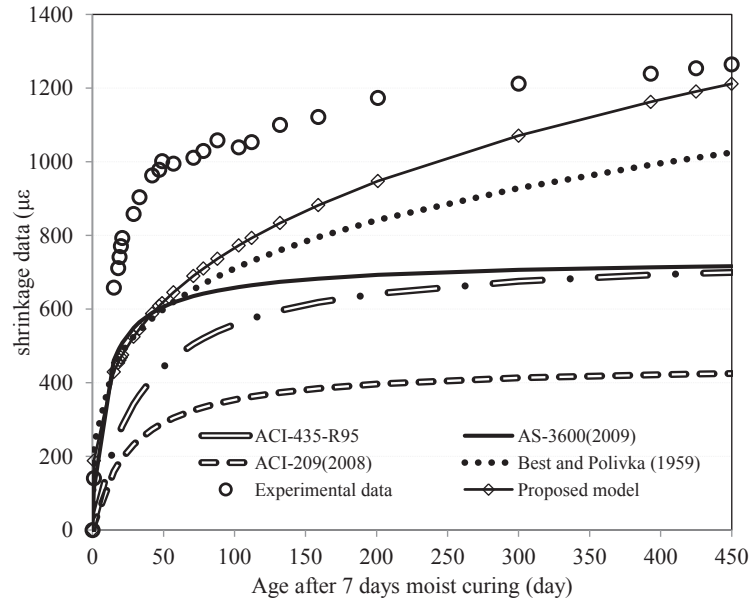


Figure 6.27: Experimental data of EPS-LWC versus predicted shrinkage strain by different models

6.7.6.3. Rate of weight loss with time in shrinkage specimens

According to Figure (6.21), there is a remarkable relationship between the rate of weight loss and age of the specimen. This relationship can be expressed by the developed Equation (6.6) in this study.

$$V_{\Delta W} = 41.937 \times t^{0.798} \quad \text{Eq. (6. 6)}$$

Where; $V_{\Delta W}$ is the rate of weight loss (gram/day) and t is the age of specimen (day) after 7 days of moist curing. The correlation factor between the proposed model and the measured data is $R^2 = 0.993$. The correlation factors for the models proposed by Best and Polivka, AS-3600(2009) and ACI-435-R95 are 0.79, 0.54 and 0.47 respectively.

6.7.7. Creep Coefficient of Lightweight Concrete

Standard concrete cylindrical specimens (150 mm × 300 mm) were used to measure the creep strain and coefficient of EPS-LWC in this study. The creep specimens and LWC slabs were loaded simultaneously at the age of 14 days. There were two levels of uniformly distributed sustained loading applied on the slabs. Magnitude of the sustained

loading at levels 1 and 2 were equal to 30% and 40% of the ultimate capacity of the slab section at mid-span, respectively. Consequently, the creep specimens were loaded up to 30% and 40% of the ultimate compressive strength of the used EPS-LWC mixture. Each creep rig consisted for 2 cylindrical specimens. Another companion cylindrical specimen with the same dimensions was monitored to measure the shrinkage strain after 14 days moist curing. The testing setup and measurement approach are presented in the codes presented in Table (6.9). The environmental condition in the concrete laboratory was kept at $23\pm 2^{\circ}\text{C}$, and the Relative Humidity (RH) was kept with the tolerance of $50\pm 5\%$.

Figure (6.28) shows the specimen and testing equipment used to measure the creep coefficient of LWC containing EPS beads.



Figure 6.28: Creep test rig at two different loading levels

Tables (6.16) and (6.17) present the measured strains in the cylindrical specimens up to 225 days under the two different loading levels. Also, the creep coefficients at different ages under two distinct levels of sustained loading are presented in the Tables (6.16) and (6.17).

Figure (6.29) graphically shows changes of the creep coefficients for EPS-LWC specimens under two levels of sustained loads.

The final creep coefficient after 225 days under loading levels 1 and 2 were 2.231 and 2.415 respectively. The maximum creep coefficient of specimens subjected to loading level 2 is 8.2% higher than that of loading level 1, which is smaller than loading level 2.

Table 6.16: Creep strain and creep coefficient of lightweight concrete cylindrical specimen (load level 1)

Time after loading		Creep strain ($\mu\epsilon$)	Creep coefficient
hr.	0		
	2	75	0.099
	4	95	0.125
day	1	156	0.206
	4	429	0.567
	5	462	0.61
	6	501	0.662
	14	767	1.012
	21	915	1.208
	26	957	1.265
	56	1238	1.635
	87	1419	1.874
	112	1495	1.975
	140	1565	2.067
	168	1611	2.128
	196	1651	2.181
	225	1689	2.231

Table 6.17: Creep strain and creep coefficient of lightweight concrete cylindrical specimen (load level 2)

Time after loading		Creep strain ($\mu\epsilon$)	Creep coefficient
hr.	0		
	2	115	0.119
	4	155	0.161
day	1	245	0.254
	4	609	0.631
	5	668	0.692
	6	729	0.754
	14	1015	1.052
	21	1180	1.222
	26	1320	1.368
	56	1665	1.725
	87	1945	2.016
	112	2053	2.127
	140	2154	2.232
	168	2225	2.306
	196	2281	2.364
	225	2330	2.415

The applied loading levels on the specimens were kept constant during the test. In other words, the creep rigs were under constant sustained loading of lower and higher levels for 225 days. The final creep coefficient in the creep rigs were in agreement with the loading level and the ratio of final creep of rig under higher load level to that of lower load level was compatible with the ratio of higher to lower loading magnitude. However, variation of the creep coefficient with time was completely different from the constant ratio of applied load. The ratio of creep coefficient in the rigs was increasing by time; however, the creep coefficient under higher loading level experienced a relatively higher rate of increment by time.

The general trend of creep coefficient of both creep rigs is descending by time. Despite the constant ratio of applied loads during the test, the ratio of creep coefficients follows a variable.

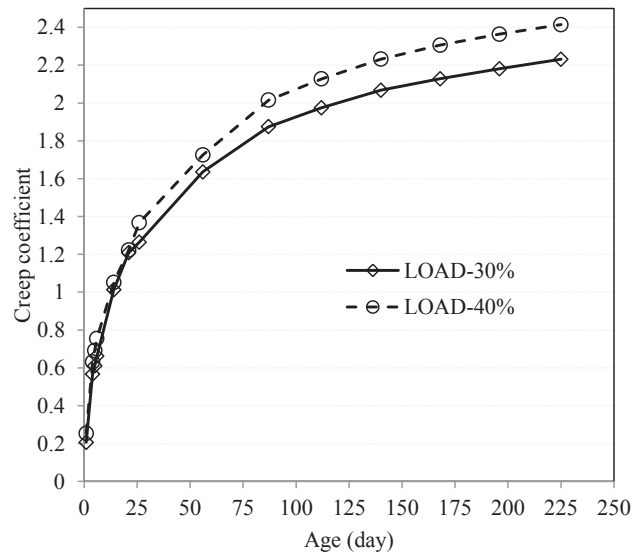


Figure 6.29: Creep coefficients of lightweight concrete under two different loading levels

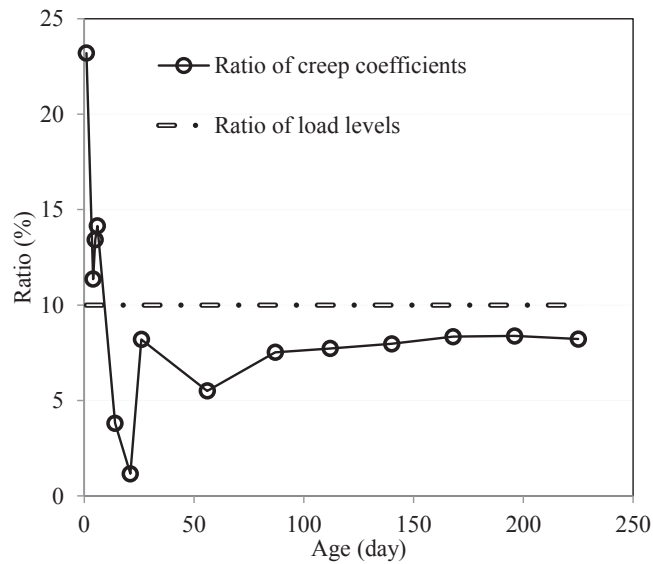


Figure 6.30: Comparing the ratio of creep coefficient at different ages

Figure (6.31) compares the changing ratio of creep strain at different ages in two creep rigs with the constant ratio of applied loads on the specimens.

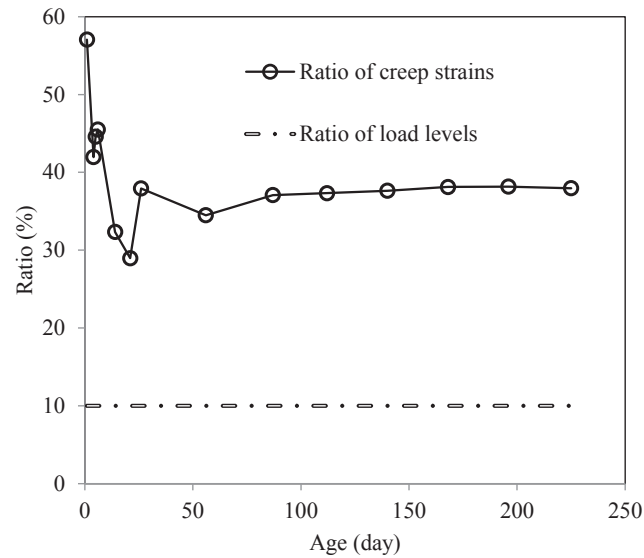


Figure 6.31: Comparing the ratio of creep strain at different ages

According to figure (6.31), despite the constant ratio of applied loads, the ratio of creep strain is following a descending rate from 53% during the first days to 37% at the age of 225 days. Comparing the decreasing ratio of creep strain and creep coefficient in Figures (6.30) and (6.31), they follow a similar trend throughout the testing duration from the loading date.

In addition, the final strain ratio is 27% higher than the ratio of applied loads, while, the final creep coefficient ratio is close to the ratio of applied loads. In other words, the ratio of the final creep coefficient after 225 days is more compatible with the ratio of the applied load.

Together with the creep measurement, a companion cylindrical specimen of the same dimensions was used to monitor the shrinkage behaviour. Table (6.18) shows the shrinkage strain of EPS-LWC after 14 days of moist-curing measured by cylindrical specimens at different ages.

Table 6.18: Shrinkage strain of cylindrical specimens (companion of creep specimens)

Age	hr.		Days													
	2	4	1	4	5	6	14	21	26	56	87	112	140	168	196	225
Shrinkage ($\mu\epsilon$)	4	5	14	25	29	31	48	60	67	103	115	122	129	134	137	140
	0	7	3	9	3	7	7	7	1	2	7	4	7	1	7	5

Figure (6.32) compares the measured shrinkage strain with the measured creep strains under the two different levels of sustained loading over a period of 225 days. The final

shrinkage strain is 87% and 60% of the final creep strain under the lower and higher levels of loading, respectively.

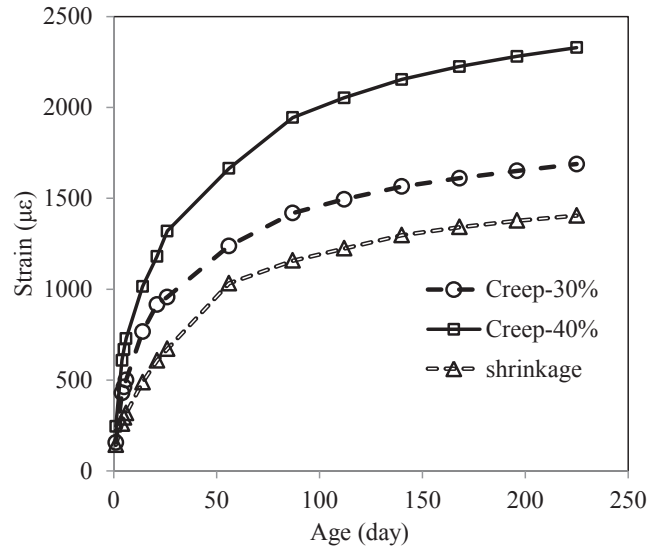


Figure 6.32: Development of creep and shrinkage strain of lightweight concrete by cylindrical specimens

6.7.7.1. Proposed analytical models

The mathematical models are effective tools used to express the properties of materials in absence of the accurate tests. These models are generally proposed on the basis of the most-compatible of the previously developed models in the literature and are verified by the experimental data. There are different models used in literature to predict the creep coefficient of concrete at different ages. Table (4.5) in Chapter 4 shows some existing creep models for lightweight concrete.

Equation (6.7) explains the relationship between the creep strain of EPS-LWC cylindrical specimen and age under two different levels of sustained stress. The coefficients of each loading level are indicated in Table (6.19).

$$\varepsilon_{cr} = 800\alpha t^{\beta} \quad \text{Eq. (6.7)}$$

Where; ε_{cr} is creep strain, α is the ratio of applied stress to the compressive strength of concrete, β is the coefficient of time.

The age of specimen and magnitude of the sustained stress (fraction of the compressive strength of concrete) are included in Equation (6.7). Considering the number of concrete

mixtures used in this study, inclusion of the other effective parameters of creep strain in Equation (6.7) will not be practical. Figures (6.33) and (6.34) confirm the efficiency of the proposed models in this study to predict the creep strain of EPS-LWC under different levels of sustained stress.

Table 6.19: Coefficients of creep strain of EPS-LWC in Eq. (6.7)

Sustained stress (MPa)	α	β
$0.3f'_c$	0.3	00.375
$0.4f'_c$	0.4	0.375

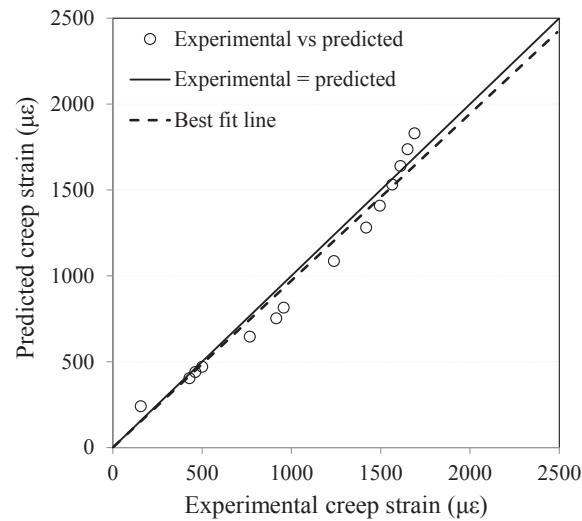


Figure 6.33: Comparison of the experimental and predicted creep strain at load level 1

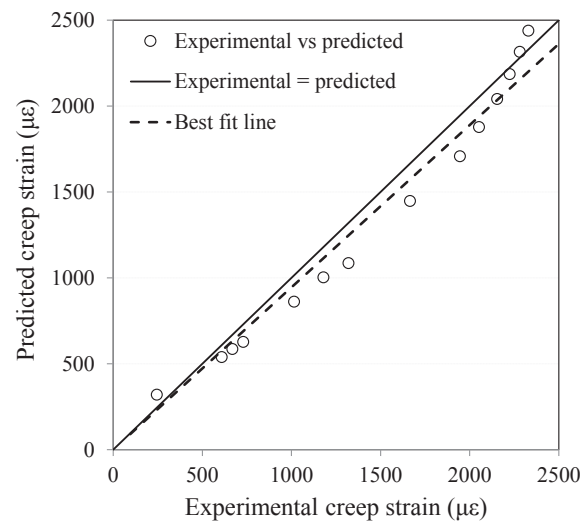


Figure 6.34: Comparison of the experimental and predicted creep strain at load level 2

Equation (6.8) is proposed in this study to predict the creep coefficient of EPS-LWC subjected to different levels of sustained stress. The coefficients of each loading level are indicated in Table (6.20).

$$\phi(t, t_0) = \frac{t^a}{b+t^c} \phi_u \quad \text{Eq. (6. 8)}$$

Where; $\phi(t, t_0)$ is creep coefficient, ϕ_u is ultimate creep coefficient, and a, b, c are experimental parameters.

Table 6.20: Coefficients of creep strain of EPS-LWC in Eq. (6.7)

Sustained stress (MPa)	a	b	c
$0.3f'_c$	0.65	8	0.6
$0.4f'_c$	0.75	10	0.72

Figures (6.35) and (6.36) confirm the efficiency of the proposed models in this study to predict the creep coefficient of EPS-LWC under different levels of sustained stress.

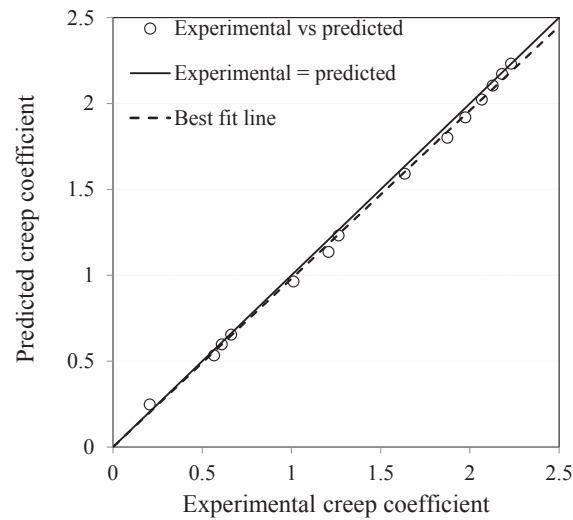


Figure 6.35: Comparison of the experimental and predicted creep coefficient at load level 1

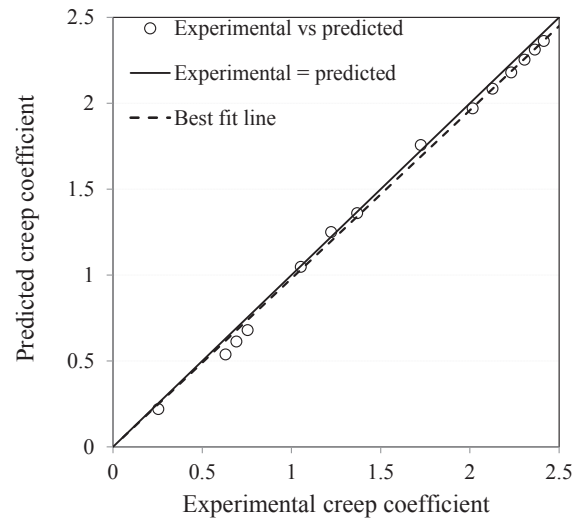


Figure 6.36: Comparison of the experimental and predicted creep coefficient at load level 2

6.7.7.2. Effect of shape and size of the test specimen on shrinkage strain

The shrinkage strain of EPS-LWC was measured by prism and cylindrical specimens for 225 days. A prism with dimensions of 285mm×75mm×75 mm was moist-cured for 7 days and a cylindrical specimen with 150 mm diameter and 30 mm height was moist-cured for 14 days. Figure (6.37) compares the shrinkage strain of prism and cylindrical specimens up to the age of 225 days.

According to Figure (6.37), the ultimate shrinkage after 225 days in the cylindrical specimen is 18.1% higher. Also, the rate of shrinkage strain in first two weeks is higher in the cylinder. In other words, the shrinkage of the prism specimen is growing slowly during the first two weeks, and then it experiences a sharp growth between the ages of 15 to 50 days. After 50 days, the shrinkage in the cylinder is increasing more rapidly with a higher incremental rate until the age of 225 days. The difference between the records of prisms and cylinders are increasing after 50 days remarkably.

The ratio of shrinkage at 225 days to 50 days is 1.19 in the prism and 1.40 in the cylindrical specimen, respectively. It means a faster growing of shrinkage in the cylindrical specimen comparing to the prism specimen.

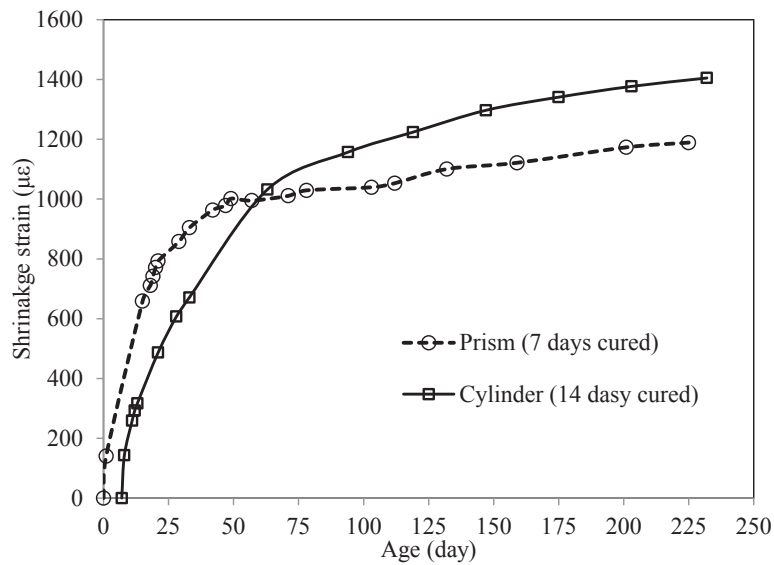


Figure 6.37: Shrinkage strain of EPS-LWC by prism and cylindrical specimen

6.7.7.3. Volume/surface ratio

The ratio of specimen volume to the drying surface (V/S) is an important parameter in studying the shrinkage behaviour of same mixture and similar curing conditions by different shape and sizes of concrete specimens. The V/S ratio in prism and cylinder specimens are 18.75 and 37.5 respectively. In other words, the hypothetical thickness in the cylinder is twice that of the prism. The increased V/S ratio decreases the ultimate shrinkage [35]. It is in good agreement with the recorded data up to the age 50 days. However, it is in contrast with the final shrinkage value after 225 days.

6.7.8. Pull-Out Test

The pull-out test is performed to measure the bond stress between reinforcement and concrete. The test specimens were a cylindrical specimen of 150mm × 300mm concrete with a steel bar of N12 in the middle. The specimens were cured in lime pre-saturated water until the test day. The still-concrete bond stress was recorded at ages 3, 7, 14, 21, 28, 56 and 91 days. Three specimens were tested for each age.

Since, corrosion of steel bars in the water, especially the tests at 56 and 91 days reduces the steel-concrete bond stress after casting, the steel bars and the concrete surface surrounding the bar were cleaned by a wipe and covered by ant-burst oil-based spray. Then all the specimens were kept in the water until the test age.

Figure (6.38) shows the general view of the universal pull-out test machine and the test specimens with embedded steel bars.



Figure 6.38: Pull-out test specimens and testing equipment

6.7.8.1. Testing procedure

The loading range and rate of the testing machine must be selected properly according to the expected failure load of the test specimen. The specimen is placed vertically on the bearing plate provided with a central $2d_b$ cavity in the traction device, where the d_b is steel bar diameter. The tension force (F) is applied to the lower extremity (the longer one) while the apparatus for measuring the displacement (Δ_0) is placed at the upper extremity of the bar.

6.7.8.2. Loading rate

According to the applied standard RILEM –RC6 (2004), the loading rate in Equations (6.9) and (6.10) is determined for each bar diameter so that the rate of increments of the bond stress is constant.

$$V_p = \frac{F}{t} \quad \text{Eq. (6. 9)}$$

$$V_p = 0.5d_b^2 \quad \text{Eq. (6. 10)}$$

Where; V_p is loading rate (N/sec), F is axial tension load (N), t is time ($sec.$) and d_b is bar diameter (mm).

In addition to the included parameters in Equations (6.9) and (6.10), the length of the steel bar inserted in the cylinder is another important factor in the pull-out test.

The inserted bar diameter at the centre of the cylinder was 12 mm (N12), hence, the loading rate was selected as 72 N/sec or 4.32 KN/min. The embedded steel bar depth in the cylinder specimens was 150 mm in the cylinder with total depth of 300 mm. The clear cover all around the steel bar was 69 mm.

The specimen was loaded progressively up to the bond failure or the splitting of the concrete cylinder, hence, the relation between tensile force (F) and the displacement (Δ_0) was determined.

The tension force F in the test were transformed by Equation (6.11) into bond stress τ . The bond stress has linear relationship with the ratio of the mean value of cylindrical strength to the characteristic strength of concrete ($\frac{f_{cm}}{f'_c}$).

$$\tau = \frac{1}{5\pi} \cdot \frac{F}{d_b^2} \cdot \frac{f_{cm}}{f'_c} \quad \text{Eq. (6. 11)}$$

Where; $f_{cm} = 30$ or 25.5 is the mean value of the cylindrical specimen (N/mm^2) and f'_c = characteristics compressive strength of test specimens (N/mm^2).

As presented in Equation (6.12), in the pull-out test, the bond stress is also dependent on the slip of reinforcement due to the tensile force F .

$$\tau = f(s) \quad \text{Eq. (6. 12)}$$

Where; s is the slip measured under the tension force F .

The mean curve of all the individual test results is thus obtained for assessing of the bond stress. Details of the specimen dimensions and the obtained test results are given in Table (6.20)

Figure (6.39) shows the general trend of load increment with time in the pull-out test specimens at different ages. The loading rate in all specimens is constant and the load-time diagram is linear up to the failure point in the specimens of all ages. The failure loading values increase by increasing the age of the specimen.

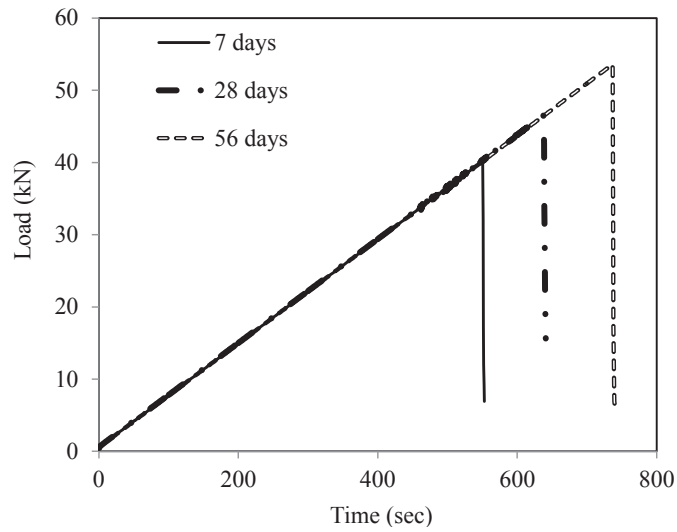


Figure 6.39: Failure load of specimens in pull-out test at different ages (loading rate=72N/sec)

Table 6.21: Specimen dimensions and variables of pull-out test

Age (day)	Cylinder	H_{cyl} (mm)	d_{cyl} (mm)	F_u (kN)	Slip (mm)	l_d (mm)	d_b (mm)	f'_c (MPa)	Presence of cracking
3	1	300	150	32.27	32	150	12	14.49	Un-cracked
	2	300	150	34.81	35	150	12		Un-cracked
	3	300	150	33.48	33	150	12		Un-cracked
7	1	300	150	43.94	26	150	12	25.38	Un-cracked
	2	300	150	35.06	25	150	12		Un-cracked
	3	300	150	41.52	25	150	12		Un-cracked
14	1	300	150	44.56	23	150	12	27.93	Un-cracked
	2	300	150	43.25	20	150	12		Un-cracked
	3	300	150	43.25	21	150	12		Un-cracked
21	1	300	150	45.15	20	150	12	29.62	Un-cracked
	2	300	150	47.25	14	150	12		Un-cracked
	3	300	150	44.32	19	150	12		Un-cracked
28	1	300	150	46.521	19	150	12	30.96	Un-cracked
	2	300	150	50.199	8	150	12		Cracked
	3	300	150	46.241	18	150	12		Un-cracked
56	1	300	150	44.67	17	150	12	32.33	Cracked
	2	300	150	56.925	8	150	12		Cracked
	3	300	150	53.525	9	150	12		Cracked
91	1	300	150	54.14	14	150	12	33.3	Cracked
	2	300	150	53.54	13	150	12		Cracked
	3	300	150	53.26	10	150	12		Cracked

In Table (6.20), l_d and d_b are the embedded length and diameter of the tension bar in middle of the cylindrical specimen.

6.7.8.3. Load-displacement behaviour of the test specimens

Figure (6.40) compares the load-displacement behaviour of pull-out test specimens at ages 28 and 56 days. The failure mode in 56 days is brittle, while the specimen at age 28 days shows a ductile failure. In the 28 days specimen, the steel bar is pulling out without any cracking in the concrete cylinder, while in the specimen tested at 56 days, slipping of the steel bar occurs with splitting cracking of the cylinder.

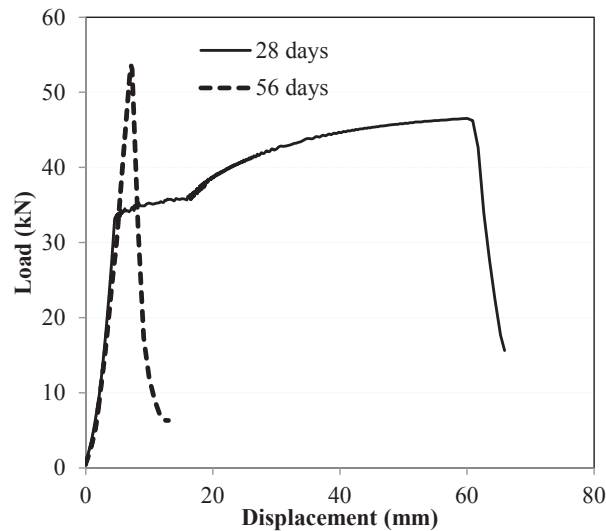


Figure 6.40: Brittle and ductile failure of pull-out test specimens at 28 and 56 days

6.7.8.4. Effect of loading rate on the failure load

To investigate the effect of loading rate on the ultimate pull-out load, two series of specimens at the age of 7 days were subjected to the different loading rates. First loading was very slow at the rate of 2.5 N/sec while second loading rate was standard rate of 72 N/sec according to RILEM-RC6 (2204). As shown in Figure (6.41) low rate application of load on the specimen increases the bond stresses between the steel bars and concrete. Also, gradual slipping of the bars changes the rate of applied load especially, at the first minutes of the loading.

The ratio of applied loads is 28.8 ($= 72/2.5$) however, the ultimate load at the low rate is about 10% higher than the maximum tensile load at a standard rate.

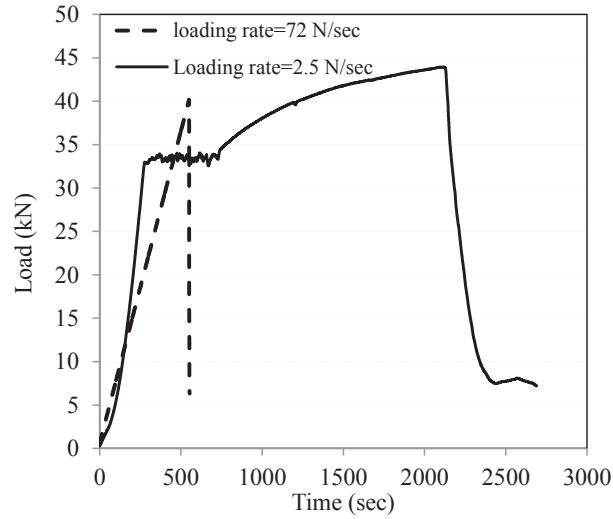


Figure 6.41: Comparison of failure load by different loading rates

6.7.8.5. Bond-slip relationship

Table (6.21) shows the final bond between the steel and concrete and the measured final slip values at different ages.

Table 6.22: Bond stress and corresponding slip values of test specimens at different ages

Age (days)	3	7	14	21	28	56	91
Bond stress τ (N/mm ²)	14.83	17.77	19.33	20.16	21.08	22.87	23.73
Slip s (mm)	33.3	25.3	21.3	17.7	15	11.3	12.3

As presented in Tables (4.10) to (4.12) in Chapter 4 of this study, there is a power type relationship between the bond stress and the bar slip. Accordingly, Equation (6.13) is proposed in this study to describe the relationship between the bond-stress and the slip in a reinforced lightweight concrete member under direct tension.

$$\tau = 62.928 s^{-0.399} \quad \text{Eq. (6.13)}$$

Where; “ τ ” is bond stress (MPa) and “ s ” is the slip of steel bar under tension (mm).

6.7.9. Compressive Stress-Strain Curve of Lightweight Concrete Containing Expanded Polystyrene Beads

To obtain the stress-strain curves, the cylindrical specimens were made of EPS-LWC were subjected to compression tests with a carefully controlled displacement rate. For each age three cylindrical specimens were tested. The final stress-strain curve for each age was taken as the average of three curves of tree cylindrical specimens. The compressive stress-strain curves at the ages of 14 and 28 days are presented in Figure (6.42).

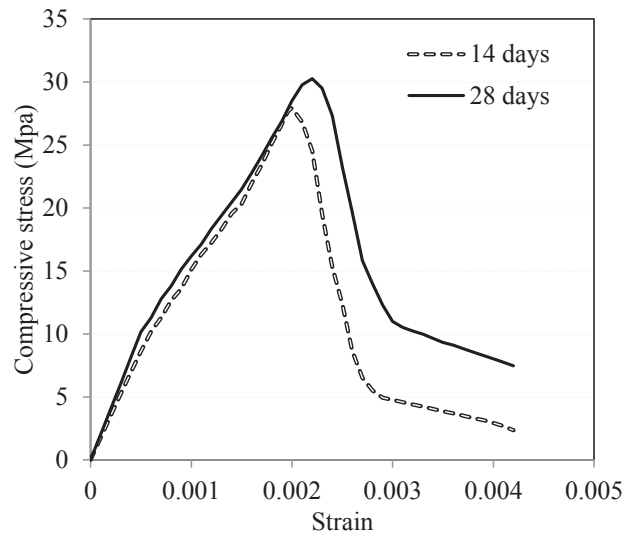


Figure 6.42: Stress-strain diagram of the LWC mixture at different ages

According to Figure (6.42), the general trend of compressive stress-strain variation in the EPS-LWC is similar at two different ages. However, due to a lower elasticity of hardened concrete at 14 days, the specimen shows higher strain values under the compressive stress compared to the specimen at 28 days. The ascending part of the Compressive Stress-Strain Curve (CSSC) at both ages are comparable, however, the descending part is different. Additionally, although the peak stress point at 14 days is lower than that of 28 days, however, the ultimate strain at both ages are tending to a similar value.

6.7.10. Energy Absorption under Compression

To calculate the absorbed energy per unit volume of the cylindrical specimens under compression, the Equation (6.14) is utilized. In fact, the absorbed energy is the area under the compressive stress-strain curve up to the failure load.

$$G_c = \int_0^{\varepsilon_u} \sigma \cdot d\varepsilon \quad \text{Eq. (6. 14)}$$

According to Figure (6.42), the ultimate strain in calculation of the stored energy at different ages was taken as 0.0042. Table (5.21) shows G_c values at different ages. Figure (6.43) also compares the absorbed energy in compressive cylinders at different ages. Both Table (6.22) and Figure (6.43) clearly show the increment of G_c by increasing the age of concrete specimens.

Table 6.23: The energy absorbed in cylindrical specimens under compression

	Age (days)						
	3	7	14	21	28	56	91
G_c (MPa)	0.02185	0.03892	0.04735	0.05477	0.0632	0.06856	0.07732

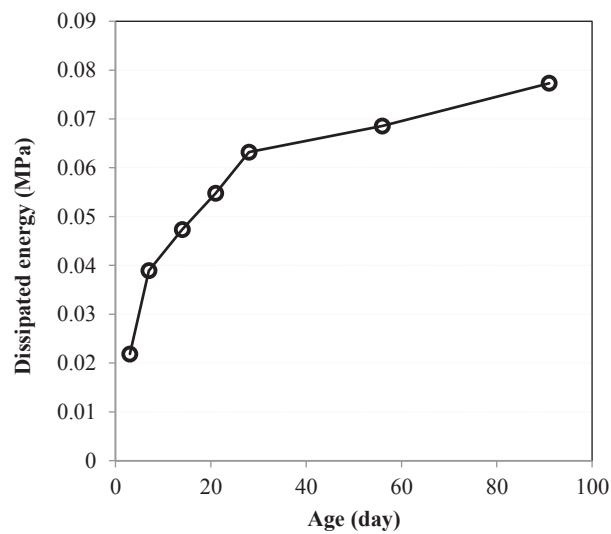


Figure 6.43: The energy stored under compression at different ages

This development of absorbed energy by age can be expressed by Equation (6.15). In fact, the equation represents the age factor at which the ratio of the modulus of elasticity at different ages to the modulus of elasticity at the age of 28 days is illustrated.

$$G_c(t) = (0.016 \ln t + 0.0061) G_{c-28} \quad \text{Eq. (6. 15)}$$

Where; $G_c(t)$ is the stored energy per unit volume of the specimen (MPa) at the age of t (days) and G_{c-28} is stored energy per unit volume of the specimen (MPa) at the age of 28 days.

Figure (6.44) shows the predicted versus experimental values of the energy absorptions at different ages. According to Figure (6.40), the proposed time-dependent relationship is in good agreement with the experimental results.

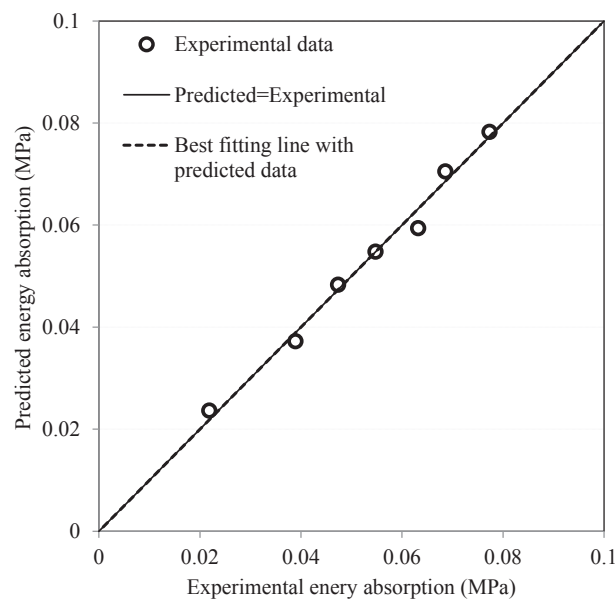


Figure 6.44: Predicted versus experimental energy absorption under compression

Variation of stored energy per unit volume under compression with the strain changes at different ages is shown in Figure (6.45). The $G_c-\epsilon$ diagrams in Figure (6.45) indicate the higher rate of G_c increment with strain by increasing the age of the specimen; i.e. older specimens demonstrate a quicker development of stored energy in the unit volume of the specimen.

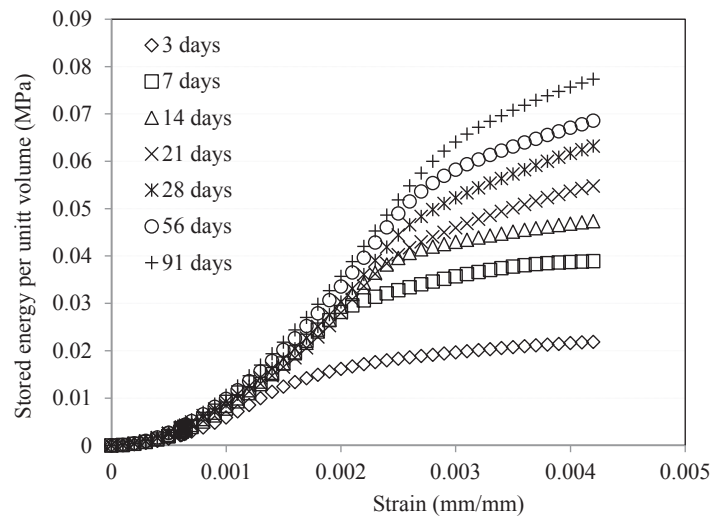


Figure 6.45: Stored energy under compression versus strain of EPS-LWC at different ages

Figure (6.46) shows the stress-strain relationship in the elastic range of the ascending part of the CSSC at different ages of EPS-LWC specimens. By increasing the compressive strength of the concrete, the corresponding elastic ranges develop with different slopes. In other words, the increased compressive strength of concrete with age increases the capacity of energy absorption (the area under the stress-strain curve) of the specimens. The slope of diagrams increases rapidly at higher ages, especially after 14 days. Consequently, the rate of the stored energy increment after 14 days are higher than the early ages of concrete.

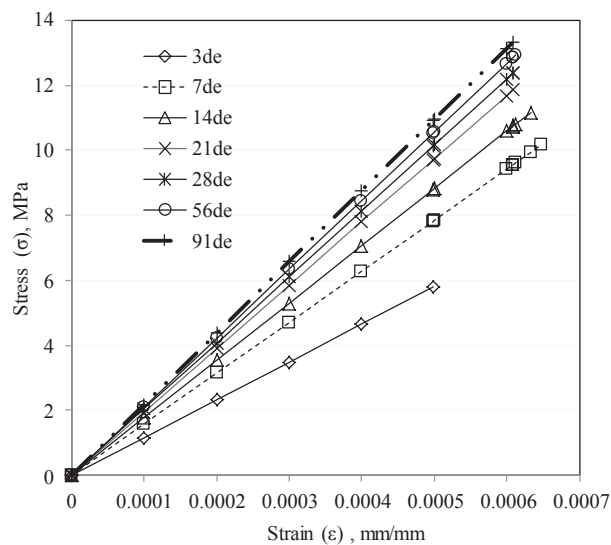


Figure 6.46: Compressive stress-strain variation of the LWC mixture in elastic range at different ages

6.7.11. Mechanical Properties of Steel Bars

The experimental part of this study is to investigate the time-dependent deflection of reinforced concrete slabs. Details of the experimental program will be described in the next chapters. To reinforce the concrete section, 4N12 steel bars have been embedded in each slab. To enhance the accuracy of the study, the steel bars have been tested, and exact values of mechanical properties of the steel bars have been determined. Mechanical properties of the steel bar, including the yield stress, modulus of elasticity and poisson ration are given in Table (6.23). Figure (6.47) also presents the tensile stress-strain curve of the N12 steel bars.

Table 6.24: Mechanical properties of N12 steel bar

Property	Yielding stress, f_y (MPa)	Modulus of elasticity E_s (kN/m ²)	Poisson ratio (ν)
Measured value	538	200553150	0.25

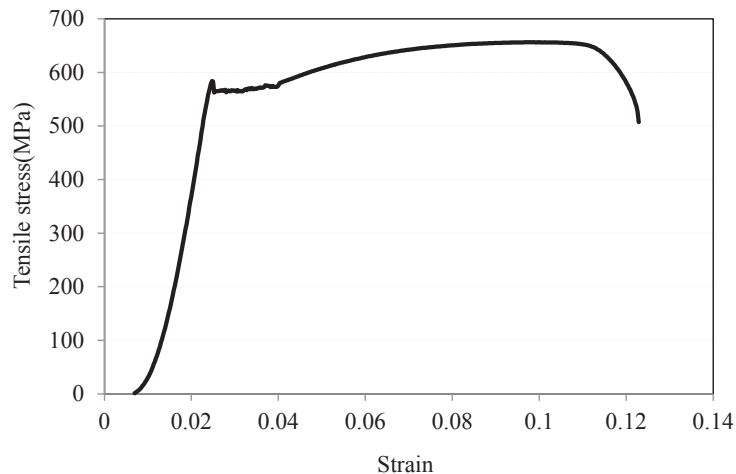


Figure 6.47: Tensile Stress- strain curve of the N12 steel bars

6.7.12. Relationships between Compressive Strength and Mechanical Properties

The compressive strength can illustrate mechanical properties of hardened concrete. There is a wide range of empirical relationships and models in literature to predict different mechanical properties from compressive strength. Chapter Three of this study presented some existing models for mechanical properties of LWC, CC, and SCC.

Table (6.24) summarizes the mechanical properties of EPS-LWC. Equations (6.14) to (6.15) are proposed based on regression analysis of the experimental data to estimate the splitting tensile strength, modulus of elasticity and modulus of rupture of EPS-LWC

based on compressive strength. The models presented in Chapter Three have been selected as the basis of these equations.

Figures (6.48) to (6.51) compare the predicted versus experimental values of splitting tensile strength, modulus of elasticity and modulus of rupture respectively.

Table 6.25: Mechanical properties of EPS-LWC at different ages

Age (day)	3	7	14	21	28	56	91
Compressive strength (MPa)	14.49	25.38	27.93	29.62	30.96	32.33	33.3
Splitting tensile strength (MPa)	1.44	2.3	2.6	2.78	2.89	2.98	3.08
Modulus of Elasticity (GPa)	11.63	15.68	17.65	19.48	20.32	21.1	21.89
Modulus of rupture (MPa)	2.982	3.498	4.056	4.218	4.305	4.374	4.524

6.7.12.1. Splitting tensile strength

Equation (6.16) is proposed to predict the splitting tensile strength of EPS-LWC at different ages. The obtained correlation factor (R^2) shows a good agreement between the measured values and predicted values of the splitting tensile strength by the proposed model.

$$f_{tc} = 0.1227 \times f_c'^{0.9176} \quad R^2 = 0.996 \quad \text{Eq. (6. 16)}$$

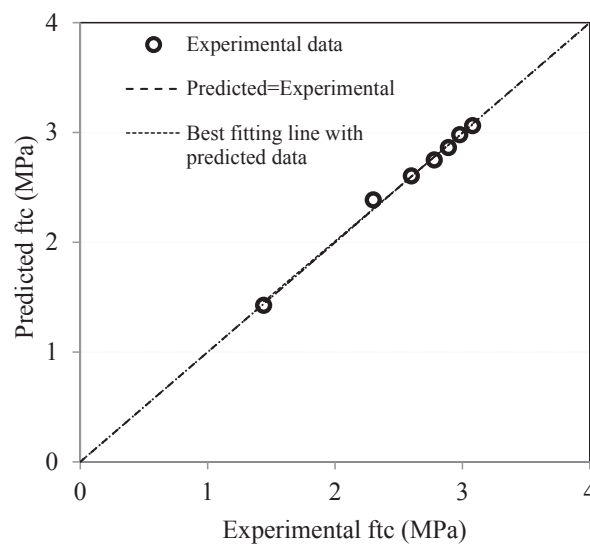


Figure 6.48: Predicted versus experimental splitting tensile strength of EPS-LWC

Together with the high correlation factor, according to Figure (6.48), the proposed model estimates the splitting tensile strength from compressive strength with high accuracy.

6.7.12.2. Modulus of elasticity

Equation (6.17) is proposed to predict the modulus of elasticity of EPS-LWC at different ages.

$$E_c = 1.5045 \times f_c'^{0.7523} \quad R^2 = 0.987 \quad \text{Eq. (6. 17)}$$

Along with the high correlation factor, according to Figure (6.49), the proposed model of predicting of the modulus of elasticity from the compressive strength is in good agreement with the experimental results.

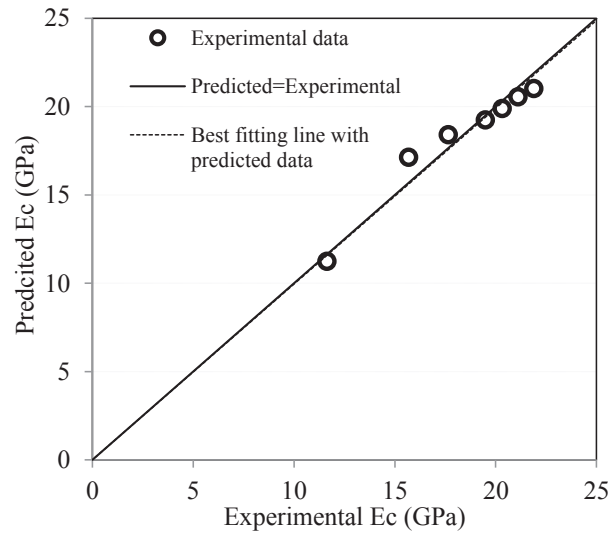


Figure 6.49: Predicted versus experimental modulus of elasticity of EPS-LWC

6.7.12.3. Modulus of rupture

Equation (6.18) is proposed to predict the modulus of rupture (flexural tensile strength) of EPS-LWC at different ages.

$$f_r = 0.7625 \times f_c'^{0.5003} \quad R^2 = 0.9196 \quad \text{Eq. (6. 18)}$$

According to Figure (6.50), the proposed model of predicting of the modulus of rupture from the compressive strength is in good agreement with the experimental results. Additionally, the high correlation factor clearly indicates efficiency of the proposed model.

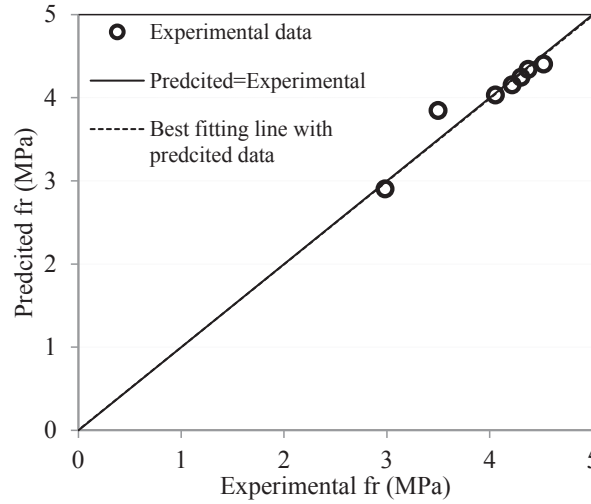


Figure 6.50: Predicted versus experimental modulus of rupture of EPS-LWC

The correlation factor (R^2) between the predicted and experimental values of splitting tensile strength, modulus of elasticity and modulus of rupture are presented in Table (6.25). According to R^2 values, the proposed models in Equation (6.14) to (6.16) for prediction of the splitting tensile strength, modulus of elasticity and modulus of rupture are precise and in good agreement with the experimental results.

Table 6.26: Correlation factor between the predicted and experimental mechanical properties

Property	Equation	R^2
Splitting tensile strength	5.7	0.99
Modulus of elasticity	5.8	0.95
Modulus of rupture	5.9	0.92

6.7.12.4. Ultimate pull-out load with respect to the compressive strength

Figure (6.51) shows the relationship between the compressive strength of the LWC mixture and the ultimate pull-out load on the cylindrical specimens. Equation (6.19) is developed to explain the relationship between the ultimate pull-out load and the compressive strength of EPS-LWC.

$$F = 0.9925f'_c + 17.481 \quad R^2 = 0.96 \quad \text{Eq. (6. 19)}$$

Where; F is ultimate pull-out load (KN) and f'_c is compressive strength (MPa).

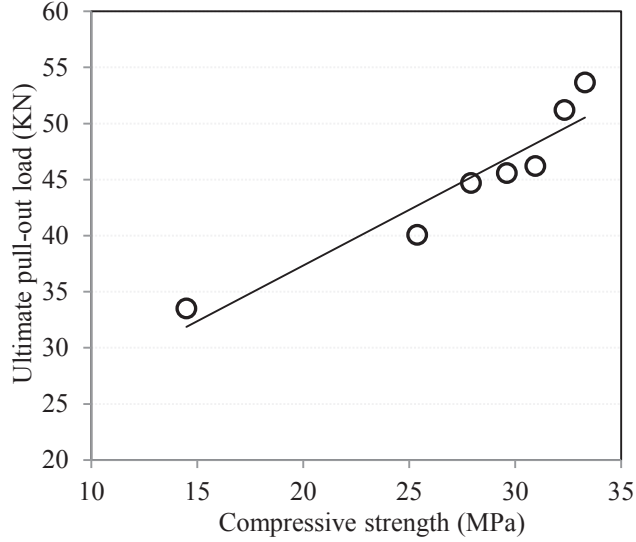


Figure 6.51: Ascending ultimate pull-out load by age development of compressive

6.7.12.5. Compressive stress-strain relationship

The existing models to estimate the stress-strain diagram of different types of concrete under compression are broadly discussed in Chapter Three of this study. Based on the existing models presented in Chapter Three, a new model in Equation (6.20) is developed to predict the compressive stress-strain curve of EPS-LWC. In the proposed model, the Equations (6.16) to (6.18) proposed for compressive strength and modulus of elasticity, are also utilized.

$$\sigma_c = \frac{f'_c \cdot \beta (\varepsilon / \varepsilon_0)}{[(\beta - 1 + (\varepsilon / \varepsilon_0)^\beta)]} \quad ; \quad 0 \leq \varepsilon \leq \varepsilon_{\max} \quad \text{Eq. (6. 20)}$$

$$\beta = \frac{1}{1 - \frac{f'_c}{E_0 \varepsilon_0}}$$

$$\sigma_c = \frac{f'_c \cdot n \beta (\varepsilon / \varepsilon_0)}{[(n \beta - 1 + (\varepsilon / \varepsilon_0)^{n \beta}]} \quad \text{if } \varepsilon > \varepsilon_0$$

$$n = 0.0012P^2 - 0.0846P + 1.551$$

$$P = \frac{E_{sec}}{E_0} \quad E_{sec} = \frac{f'_c}{\varepsilon_0}$$

$$\varepsilon_0 = 6.52f'_c \times 10^{-5} + 1.634 \times 10^{-4}$$

$$E_0 = 1.5045 \times f'_c{}^{0.7523}$$

Where;

σ_c, ε : Stress and strain of concrete

f'_c, ε_0 : Maximum stress and corresponding strain of concrete

β : Material parameter

E_0 : Initial tangent modulus of elasticity

Figure (6.53) compares the measured v stress-strain under the two different levels of compressive stress and the predicted values of creep strain by the proposed model in the Equation (6.20).

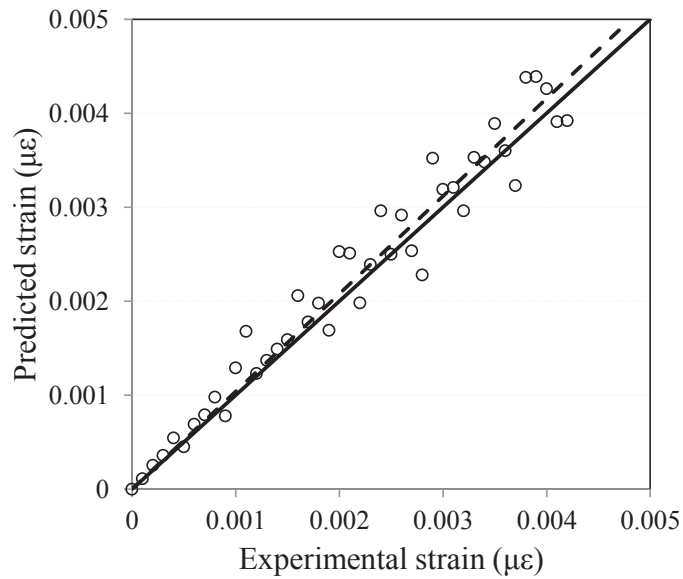


Figure 6.52: Comparison of the measured and predicted stress-strain variation under compression

6.8. COMPARING THE MECHANICAL PROPERTIES OF LIGHTWEIGHT, SELF-COMPACTING AND CONVENTIONAL CONCRETE

6.8.1. Mixture Design

The experimental results of this study are compared with the previously conducted experiments on self-compacting concrete slabs by Aslani (2014) [1] and conventional concrete slabs by Nejadi (2005) [2].

The mixture design and components are different in these types of concrete. In total, one mixture for LWC slabs and one mixture for CC slabs were used. For eight SCC slabs, two normal SCC mixtures (N-SCC), two steel fibre (D-SCC), two polypropylene fibre (S-SCC) and two hybrid fibre (DS-SCC) mixtures have been considered in the test program.

Table (6.27) explains the mixture proportions and density of LWC and SCC mixtures. Type, volume and aspect ratio of the fibres that influence the mechanical properties of SCC mixtures are also illustrated in Tables (6.28). The mixture design of CC with density of 2450 kg/m^3 is not reported in the reference.

Table 6.27: Mixture design of self-compacting concrete [1] and lightweight concrete

Constituents	N-SCC	D-SCC	S-SCC	DS-SCC	LWC	CC
Cement (kg/m ³)	160	160	160	160	250	
Fly Ash (kg/m ³)	130	130	130	130		
GGBFS * (kg/m ³)	110	110	110	110		
Cementitious content (kg/m ³)	400	400	400	400		
Water (kg)	208	208	208	208	150 lit/m ³	
Water Cementitious ratio	0.52	0.52	0.52	0.52	0.6	
Aggregate (kg/m ³)						
Coarse sand	660	660	660	660	620	
fine sand	221	221	221	221		
Coarse aggregate	820	820	820	820	800	
Light Aggregate (BST) (lit/m ³)					300	
Admixtures (lit/m ³)						
Super plasticizer	4	4.86	4.73	4.5		
Viscosity modifying agent	1.3	1.3	1.3	1.3		
High range water reducing agent	1.6	1.6	1.6	1.6	2	
Fibre content (kg/m ³)						
Steel	-	30	-	15		
Polypropylene	-	-	5	3		
Density	2340	2274	2330	2385	2000	2450

*Ground Granulated Blast Furnace Slag (GGBFS)

Table 6.28: Physical and mechanical properties of fibres in self-compacting concrete mixture [1]

Type	Trade name	Density (kg/m ³)	Length (l)	Diameter (d _b)	Aspect Ratio (l/d)	Tensile strength (MPa)	Elasticity modulus (GPa)
Steel	Drami x RC-80/60-BN	7850	60	0.75	80	1050	200
Polypropylene	Synmi x 65	905	65	0.85	76.5	250	3

6.8.2. Early-Age Mechanical Properties

In general, the existing models and empirical relationships in the literature provide relatively inaccurate values of the mechanical properties of concrete. Besides, the models used to predict the early age characteristics of concrete especially SCC and LWC are very rare in the literature. Therefore, compressive strength and modulus of elasticity of the mixtures were measured by the companion test specimens in the laboratory conditions at two ages of 14 and 28 days for all different types of concrete. Table (6.29) shows the measured compressive strength and modulus of elasticity of EPS-LWC, SCC and CC mixtures at 14 and 28 days.

According to test results, all mixtures are in the range of normal strength concrete.

Table 6.29: Mechanical properties of self-compacting concrete [1], lightweight concrete and conventional concrete [2] mixtures

Property	Age (day)	N-SCC	D-SCC	S-SCC	DS-SCC	LWC	CC
f'_c (MPa)	14	29.05	34.3	32.45	38.1	27.93	18.3
	28	33.3	38	38.1	45	30.96	24.8
E_c (GPa)	14	32.24	29.14	29.68	31.26	17.65	22.82
	28	35.39	35.76	35.76	36.1	20.32	24.95

Figures (6.53) and (6.54) compare the measured values of compressive strength and modulus of elasticity of the mixtures at ages 14 and 28 days respectively. Development rate of the compressive strength and modulus of elasticity with time in the mixtures are also compared in Figures (6.55) and (6.56) respectively.

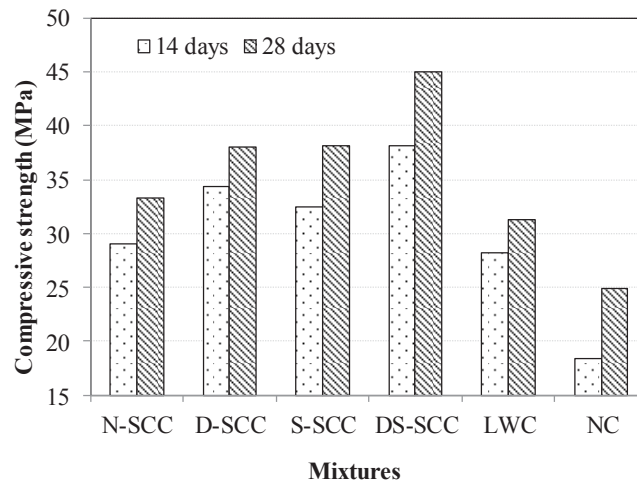


Figure 6.53: Compressive strength of EPS-LWC, SCC and CC mixtures at 14 and 28 days

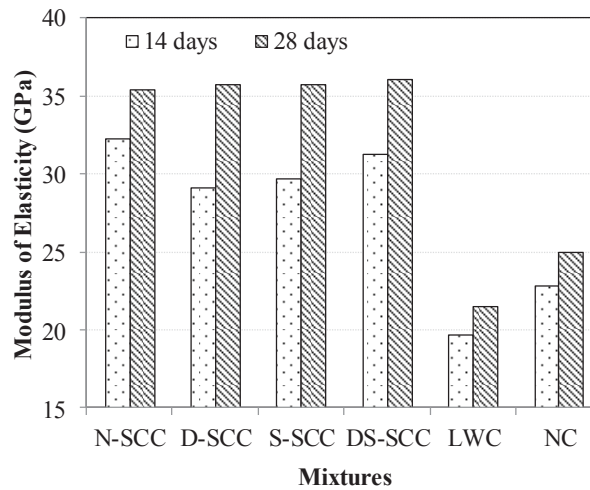


Figure 6.54: Modulus of elasticity of EPS-LWC, SCC and CC mixtures at 14 and 28 days

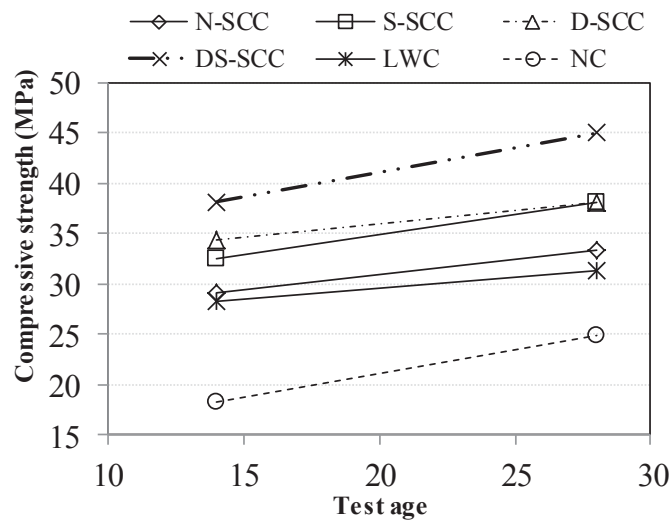


Figure 6.55: Increasing rate of compressive strength in EPS-LWC, SCC and CC mixtures

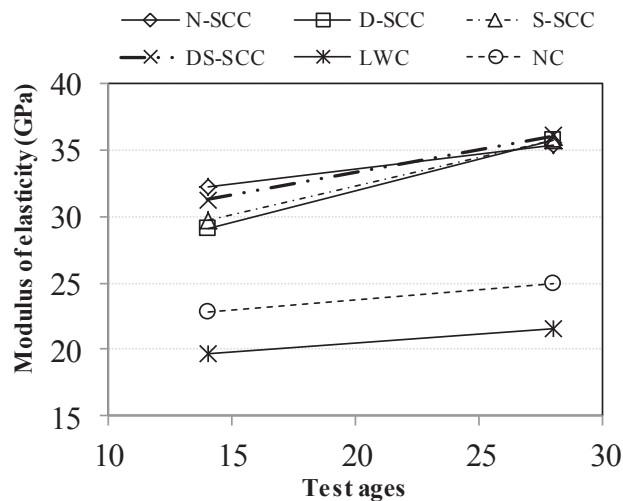


Figure 6.56: Increasing rate of modulus of elasticity in EPS-LWC, SCC and CC mixtures

In Figures (6.55) and (6.56), the presented lines between two points (14 and 28 days) are part of general development diagrams of the compressive strength and modulus of elasticity with time. The general relationship between the age and mechanical properties of concrete is not necessarily linear.

According to Figures (6.53) to (6.56), the average increasing ratio of the modulus of elasticity in CC, SCC and EPS-LWC are 1.10, 1.17 and 1.10 respectively. The development ratio of the compressive strength in CC, SCC and EPS-LWC is 1.35, 1.15 and 1.10 respectively. Overall, the early-age properties and increasing rate in SCC mixtures, particularly in presence of the fibres are greater than the corresponding properties of CC and LWC mixtures.

The compressive strength and modulus of elasticity are the main mechanical properties of concrete affecting the deflection behaviour of reinforced concrete slabs. However, other mechanical properties can influence the deflection behaviour of the slabs also. The tensile strength of concrete under flexural loading is an effective parameter in the cracking and deflection behaviour of EPS-LWC, SCC and CC slabs. Figure (6.57) compares the modulus of rupture (flexural tensile strength) of CC, LWC and SCC mixtures at different ages.

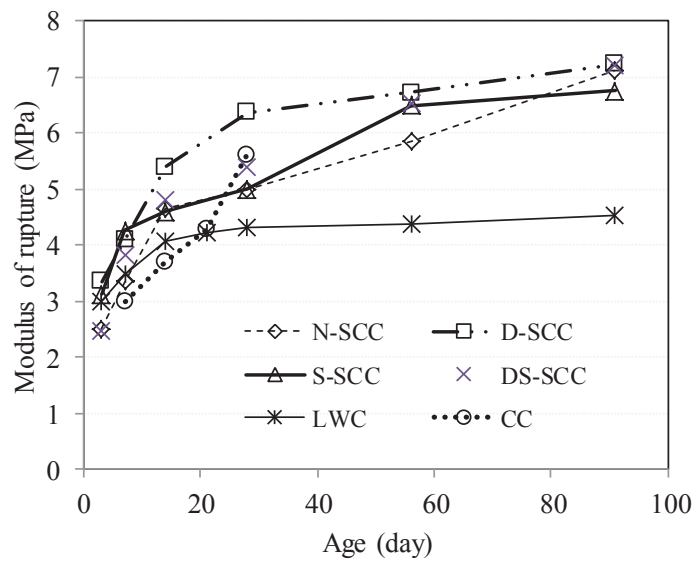


Figure 6.57: Comparison of the modulus of rupture in LWC, CC and SCC mixtures

According to Figure (6.58), EPS-LWC poses the lowest values of the modulus of rupture, especially after age 28 days. However, SCC mixture with and without fibre reinforcing are in the top range of modulus of rupture in almost all ages.

6.8.3. Time-Dependent Properties of Conventional, Self-Compacting and Lightweight Concrete

6.8.3.1. Shrinkage strain

Creep and shrinkage are the two main parameters which influence the time-dependent cracking and deflection behaviour of reinforced concrete slabs. The shrinkage strain is monitored for about 500, 250 and 400 days in EPS-LWC, SCC and CC specimens respectively. Additionally, the creep strain of EPS-LWC, SCC and CC specimens were measured for about 250 days under two distinct levels of compressive stress.

Figure (6.58) compares the shrinkage strain of EPS-LWC, SCC and CC mixtures up to 240 days. Table (6.30) summarizes the final free shrinkage strain of CC, LWC and CC specimens after 240 days from casting date.

Table 6.30: Free shrinkage strain of CC, LWC and SCC mixtures

	CC	LWC	N-SCC	D-SCC	S-SCC	DS-SCC
Shrinkage strain ($\mu\epsilon$)	784	1180	870	844	823	882

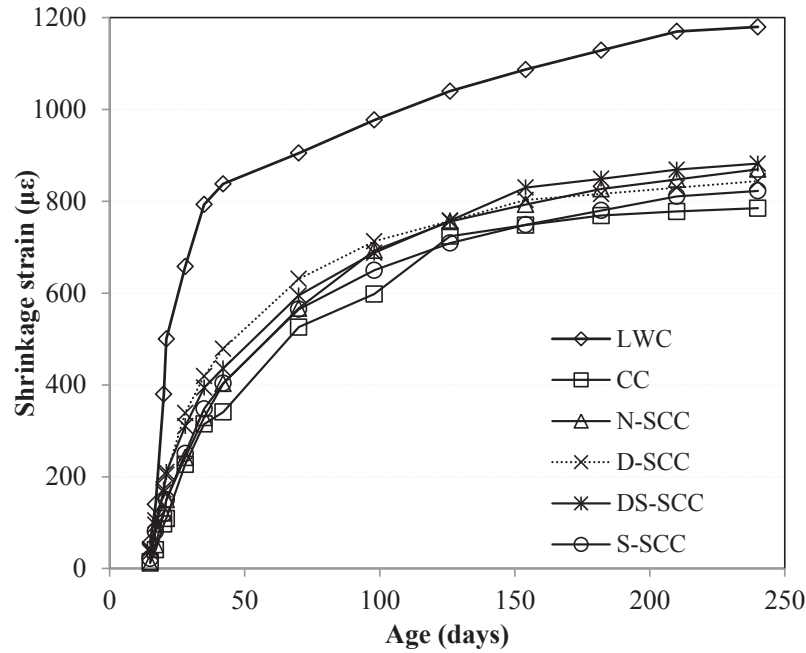


Figure 6.58: Comparison of shrinkage strain in EPS-LWC, SCC and CC mixtures

According to Figure (6.58) and Table (6.30), the shrinkage strain in LWC is considerably higher than that of the CC and SCC mixtures. The free shrinkage strain in LWC and N-SCC mixtures is 51% and 10% greater than that of the CC mixture. The porous (permeable) nature of the lightweight concrete, relatively higher ratio of water to cement and high rate of the water loss cause higher values of the drying shrinkage in lightweight concrete.

6.8.3.2. Creep strain

Figure (6.59) compares the creep coefficient in EPS-LWC, SCC and CC specimens under sustained compressive stress. The creep strain is measured by standard cylindrical specimens in all concrete types. The applied stress is equal to 40% of the compressive strength of the companion cylinders in each type of concrete. The creep strain under compressive stress equal to 30% of the compressive strength of cylinders is not reported for SCC and CC mixtures. However, the creep strain and creep coefficient of EPS-LWC under 30% and 40% of the compressive strength is comprehensively explained in section (6.7.8) of this chapter.

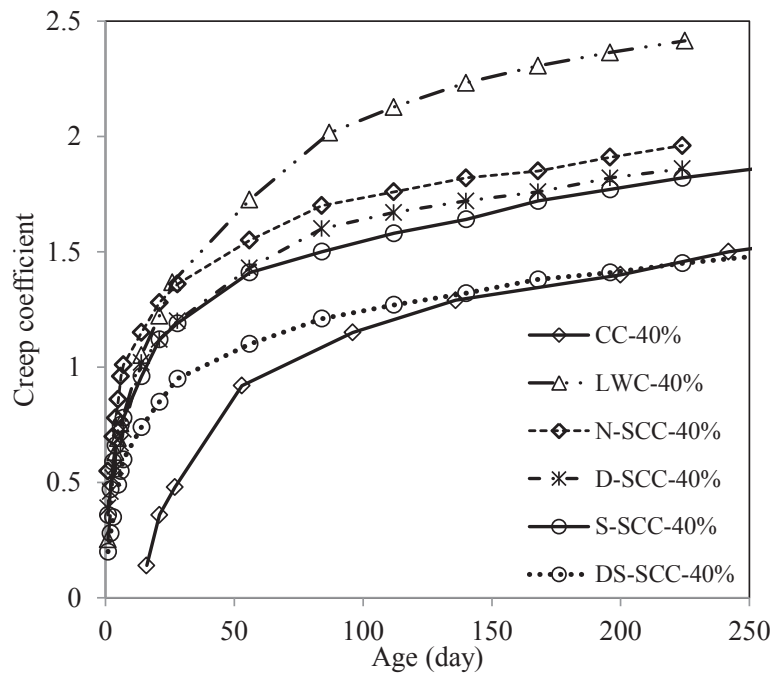


Figure 6.59: Creep coefficient of CC, SCC and EPS-LWC specimens loaded at age 14 days

According to Figure (6.59) the creep coefficient in EPS-LWC is considerably higher than that of SCC and CC specimens under the same level of sustained compressive stress.

Table (6.31) summarizes the creep coefficient of EPS-LWC, SCC and CC mixtures at 240 days after casting.

Table 6.31: Creep coefficient of CC, LWC and SCC mixtures after 240 days sustained loading

	CC	LWC	N-SCC	D-SCC	S-SCC	DS-SCC
Creep coefficient	1.5	2.415	1.96	1.86	1.82	1.45

According to Figure (6.59), the creep coefficient in EPS-LWC and CC reach the highest and lowest recorded values after 240 days. The creep coefficient at 240 days in LWC and N-SCC mixtures is 1.61 and 1.3 times higher than that of the CC mixture respectively. The creep coefficient in fibre reinforced SCC mixtures (D-SCC, S-SCC and DS-SCC) is lower than those of the EPS-LWC and N-SCC mixtures. However, the creep coefficient in D-SCC and S-SCC mixtures is higher than that of CC mixture. The final creep coefficient in DS-SCC is very close to that of CC mixture.

6.9. SUMMARY

The experimental program of the current study is divided into three major parts. The first part illustrated in this chapter, deals with the mechanical properties of fresh and hardened lightweight concrete made with EPS beads to manufacture the testing samples and slab specimens. Number and type of the required tests and the relevant Australian and international standards used to define the required properties of the concrete are given also.

The particular mixture design to achieve the required density and compressive strength is presented and the technical information of the cement, fine and coarse aggregates and admixture are also given in detail.

Preparation methods and curing conditions before testing the samples are illustrated. The hardened properties of concrete including compressive strength, splitting tensile strength, modulus of elasticity and modulus of rupture and pull out test are performed at the ages of 3, 7, 14, 21, 28, 56 and 91 days. Moreover, steel concrete bond properties, compressive stress-strain curve and creep and shrinkage of concrete are measured.

The test ages and the shape and size of the test specimens for hardened LWC are shown in Table (6.32). Three LWC specimens have been tested per age for each property.

Table 6.32: Shape and size of the LWC test specimens and test ages

Property	Cylinder (mm)	Cube (mm)	Prism (mm)	Test ages (days)
Compressive strength	75×150, 100×200, 150×300	100, 150		3, 7, 14, 21, 28, 56, 91
Modulus of elasticity	150×300			3, 7, 14, 21, 28, 56, 91
Modulus of rupture			100×100×400	3, 7, 14, 21, 28, 56, 91
Splitting tensile strength	100×200, 150×300			3, 7, 14, 21, 28, 56, 91
Steel-concrete Bond	150×300			3, 7, 14, 21, 28, 56, 91
Pull-out test	150×300			7, 14, 28, 56, 91
Stress-strain				3, 7, 14, 21, 28, 56, 91
Creep	150×300			Up to 210 days
Shrinkage			75×75×280	Up to 450 days

Table (6.33) explains the applied Australian standards and international codes for each test in the laboratory conditions.

Table 6.33:Applied codes on the curing, sampling and testing the LWC specimens

Property	Test method	Property	Test method
Compressive strength	AS 1012.8.1 (2014) , ASTAM C39 (2000)	Modulus of rupture	AS 1012.8.2 (2014), ASTM C1018 (2002)
Splitting Tensile strength	AS 1012.8.1(2014), ASTM C496 (2000)	Steel-concrete Bond	RILEM –RC6 (2004)
Creep	AS 1012.16 (2014)	Pull-out test	RILEM –RC6 (2004)
Shrinkage	AS-1012.13 (2014)	Modulus of elasticity	AS 1012.17 (2014), ASTM C469 (2002)

Table (6.34) explains the applied Australian standards AS-1012 (2014) to curing, sampling, slump test and mass per volume measurements of the fresh LWC.

Table 6.34:Test methods to determine the fresh properties of lightweight oconcrete

	Curing	Sampling	Slump test	Mass per volume
Test method	AS-1012.8 (2014)	AS-1012.1 (2014)	AS1012.3.1 (2014)	AS1012.5 (2014)

Table (6.35) shows the mixture design of EPS lightweight concrete in this study.

Table 6.35: Mix design of lightweight concrete containing expanded polystyrene beads (SSD condition)

Component		Proportion
Cement (kg/m ³)		500
Water (litre/ m ³)		180
Water to cement ratio (w/c)		0.36
BST aggregate (Litre/ m ³)		300
Fine aggregate	Coarse sand (kg/ m ³)	310
	Fine sand (kg/ m ³)	310
Coarse aggregate (kg/ m ³)		800
Admixture (litre/ m ³)		2
Mixture volume (m ³)		1.3
Slump (mm)		130
Fresh density (kg/m ³)		2000

Table (6.36) summarizes the mechanical properties of EPS-LWC.

Table 6.36: Mechanical properties of EPS-LWC at different ages

Age (day)	3	7	14	21	28	56	91
Compressive strength (MPa)	14.49	25.38	27.93	29.62	30.96	32.33	33.3
Splitting tensile strength (MPa)	1.44	2.3	2.6	2.78	2.89	2.98	3.08
Modulus of Elasticity (MPa)	11.63	15.68	17.65	19.48	20.32	21.1	21.89
Modulus of rupture (MPa)	2.982	3.498	4.056	4.218	4.305	4.374	4.524

- **Time-dependent mechanical properties of lightweight concrete**

The following equations are developed in this study to explain the relationship between each mechanical property of EPS-LWC at different ages with 28 days.

Parameter	Equation
Compressive strength (MPa, days)	$f'_c(t) = (0.1633 \ln t + 0.417) f'_{c-28}$
Splitting tensile strength (MPa, days)	$f_{ct}(t) = (0.156 \ln t + 0.4341) f_{ct-28}$
Modulus of elasticity (GPa, days)	$E_c(t) = (0.1467 \ln t + 0.4666) E_{c-28}$
Modulus of rupture (MPa, days)	$f_r(t) = (0.1046 \ln t + 0.62) f_{r-28}$

- **Shrinkage strain**

The shrinkage strain of EPS lightweight concrete is studied under different curing, drying and rewetting conditions. The ultimate shrinkage strain of EPS lightweight concrete after 450 days is considerably higher than that of the conventional and self-compacting concrete in the same grade of compressive strength.

Different conditions of initial curing, drying and rewetting were applied to the specimens as reported in Figure (6.60). The relative humidity and temperature of the environment were kept at $50\% \pm 5$ and $23 \pm 2^\circ\text{C}$ respectively.

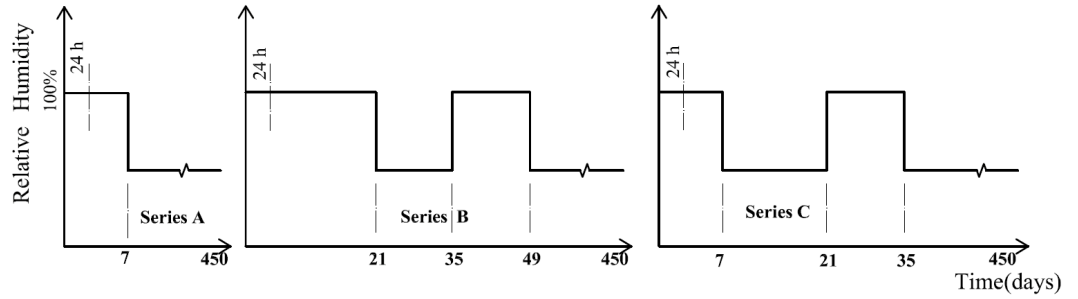


Figure 6.60: Initial curing, drying and rewetting conditions of shrinkage prisms

Among the existing models of predicting the shrinkage strain of lightweight concrete, the model proposed by Best and Polivka (1959), gives the most compatible results with the experimental data. Therefore, it is selected as the basis of the model proposed in this study. Figure (6.61) shows compatibility of the proposed model with the experimental data.

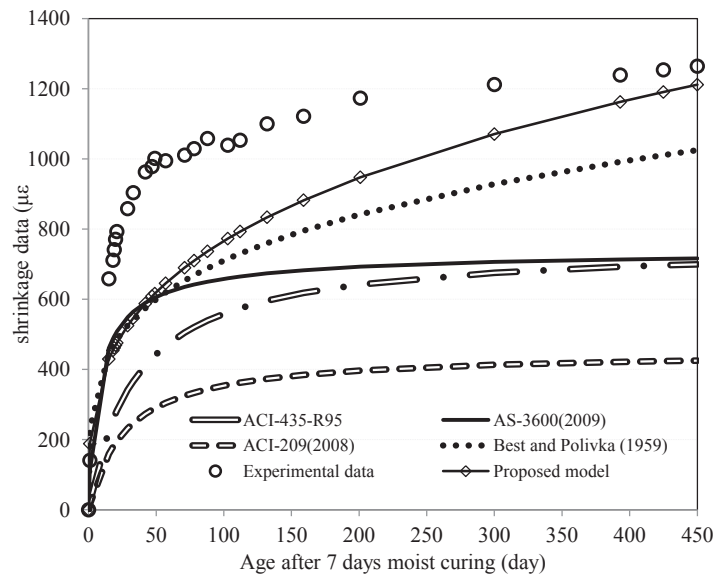


Figure 6.61: Experimental data of EPS-LWC versus predicted shrinkage strain by different models

The following models are proposed in this study to predict the shrinkage strain of lightweight concrete; also an accurate relationship is established between the rate of weight loss and the age of shrinkage specimens.

Parameter	Equation
Shrinkage strain ($\mu\epsilon$, days)	$\epsilon_{sh} = 188 \times t^{0.315}$
Rate of weight loss (gr/days, days)	$V_{\Delta W} = 41.937 \times t^{0.798}$

- Creep coefficient

Corresponding to two different levels of sustained loads on the slab specimens in the instantaneous and time-dependent deflection, two groups of creep specimens were loaded at the age of 14 days. The creep specimens were loaded to 30% and 40% of the ultimate compressive strength of the used EPS-LWC mixture. Each pair of creep rigs consisted of 2 cylinders.

The final creep coefficient after 225 days under loading levels 1 and 2 were 2.231 and 2.415 respectively. The maximum creep coefficient of specimens subjected to loading level 2 is 8.2% higher than that of loading level 1. Figure (6.62) evidently shows the effect of the loading level on the creep coefficients.

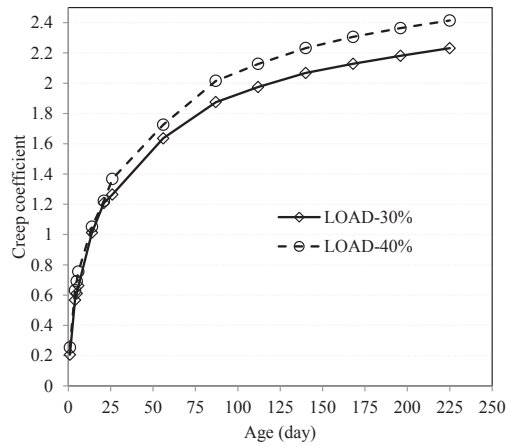


Figure 6.62: Creep coefficients of LWC specimens under sustained loads of 30% and 40% of the ultimate capacity

Figure (6.63) shows the ratio of creep coefficients at different ages in two creep rigs, compared to the constant ratio of loading levels. Despite the constant ratio of applied loads during the test in two creep rigs, the ratio of creep coefficients is following a descending rate from 25% at first days to 8% at age 225 days. Similar to descending trend of creep coefficients, the ratio of creep strain is following a descending rate from 53% at first days to 37% at the age of 225 days.

The final strain ratio is 27% higher than the ratio of applied loads in two rigs. However, the final creep coefficient ratio is close to the ratio of applied loads. In other words, the ratio of the final creep coefficient after 225 days is more compatible with the ration of applied load levels.

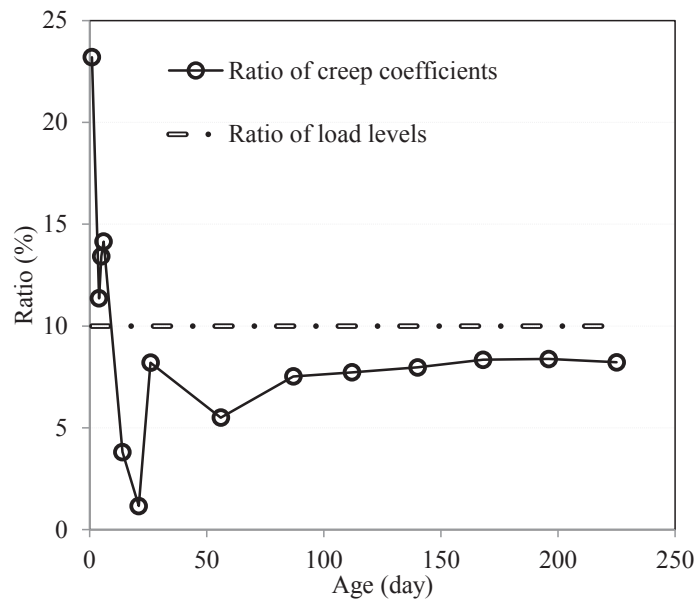


Figure 6.63: Comparing the decreasing ratio of creep coefficient at different ages

Equation (6.21) is proposed in this study to predict the creep strain of EPS-LWC cylindrical specimen respectively. Effect of the magnitude of the applied sustained stress on the creep specimen is also included in the models.

$$\varepsilon_{cr} = 800\alpha t^\beta \quad \text{Eq. (6. 21)}$$

Where; ε_{cr} is creep strain, α is the ratio of applied stress to the compressive strength of concrete, β is the coefficient of time.

Sustained stress (MPa)	α	β
$0.3f'_c$	0.3	0.375
$0.4f'_c$	0.4	0.375

Equations (6.22) is proposed in this study to predict the creep coefficient of EPS-LWC cylindrical specimen respectively

$$\phi(t, t_0) = \frac{t^a}{b+t^c} \phi_u \quad \text{Eq. (6. 22)}$$

Where; $\phi(t, t_0)$ is creep coefficient, ϕ_u is ultimate creep coefficient, and a , b , c are experimental parameters.

The predictions of these models are verified by the experimental data of creep strain and creep coefficient at two different levels of sustained compression.

Sustained stress (MPa)	a	b	c
$0.3f'_c$	0.65	8	0.6
$0.4f'_c$	0.75	10	0.72

- Pull-out test

The pull-out test is performed to measure the bond between reinforcement and concrete. Table (6.37) describes the measured values of bond and slip between the concrete and the reinforcement at different ages.

Table 6.37: Bond stress and corresponding slip values of test specimens at different

Age (days)	3	7	14	21	28	56	91
Bond (N/mm ²)	14.83	17.77	19.33	20.16	21.08	22.87	23.73
Slip (mm)	33.3	25.3	21.3	17.7	15	11.3	12.3

The following model in Equation (6.23) is developed in this study to describe the relationship between the bond-stress and the slip in a reinforced lightweight concrete member under direct tension.

$$\tau = 62.928s^{-0.399} \quad \text{Eq. (6. 23)}$$

Where; τ is bond stress (MPa) and s is the slip of steel bar under tension (mm).

- Compressive stress-strain curve

The stress-strain behaviour of the lightweight concrete cylinder specimens under compression at two ages of 14 and 28 days are given in Figure (6.64) as samples of all tests. For each age, three cylinder specimens were tested and considering the good reproduction of test results, the final stress-strain curves for each age were taken as the average of three curves.

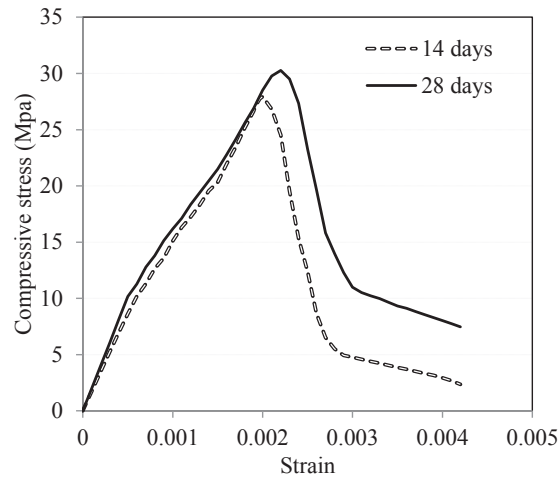


Figure 6.64: Stress-strain diagram of the LWC mixture at different ages

Table (6.38) shows absorbed energy per unit volume of cylinder specimens under compression (G_c) values at different ages.

Table 6.38: The energy absorbed in cylinder specimens under compression

	Age (days)						
	3	7	14	21	28	56	91
G_c (MPa)	0.02185	0.03892	0.04735	0.05477	0.0632	0.06856	0.07732

Equation (6.24) is developed to describe the development of absorbed energy by age.

$$G_c(t) = (0.016 \ln t + 0.0061) G_{c-28} \quad \text{Eq. (6. 24)}$$

Where; $G_c(t)$ is the stored energy per unit volume of the specimen (MPa) at the age of t (days) and G_{c-28} is stored energy per unit volume of the specimen (MPa) at the age 28 days.

Variation of stored energy per unit volume under compression with the strain changes at different ages is shown in Figure (6.65).

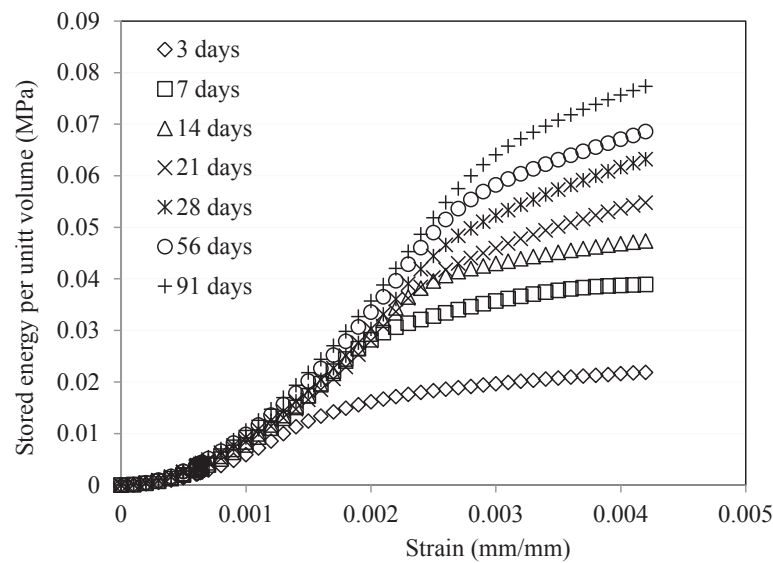


Figure 6.65: Stored energy under compression versus strain of EPS-LWC at different ages

- Relationships between compressive strength and mechanical properties

The compressive strength can illustrate mechanical properties of hardened concrete. The following equations have been developed in this study to describe the relationship between the compressive strength and other mechanical properties of concrete.

Parameter	Equation
Compressive strength (MPa)	$f'_c(t) = (0.1633 \ln t + 0.417) f'_{c-28}$
Splitting tensile strength (MPa)	$f_{ct} = 0.1227 \times f'_c{}^{0.9176}$
Modulus of elasticity (GPa, MPa)	$E_c = 1.5045 \times f'_c{}^{0.7523}$
Modulus of rupture (MPa)	$f_r = 0.7625 \times f'_c{}^{0.5003}$
Ultimate pull-out load (KN, MPa)	$F = 0.9925 f'_c + 17.481$

- Comparing the mechanical properties of lightweight, self-compacting and conventional concrete

The experimental results of this study are compared to the previously conducted experiments on self-compacting concrete slabs by Aslani (2014) and conventional concrete slabs by Nejadi (2005).

Table (6.39) explains the mixture proportions and density of LWC and SCC mixtures.

Table 6.39: Mixture design of self-compacting and lightweight concrete

Constituents	N-SCC	D-SCC	S-SCC	DS-SCC	LWC	CC
Cement (kg/m ³)	160	160	160	160	250	
Fly Ash (kg/m ³)	130	130	130	130		
GGBFS (kg/m ³)	110	110	110	110		
Cementitious content (kg/m ³)	400	400	400	400		
Water (kg)	208	208	208	208	150 lit/m ³	
Water Cementitious ratio	0.52	0.52	0.52	0.52	0.6	
Aggregate (kg/m ³)						
Coarse sand	660	660	660	660	620	
fine sand	221	221	221	221		
Coarse aggregate	820	820	820	820	800	
Light Aggregate (BST) (lit/m ³)					300	
Admixtures (lit/m ³)						
Super plasticizer	4	4.86	4.73	4.5		
Viscosity modifying agent	1.3	1.3	1.3	1.3		
High range water reducing agent	1.6	1.6	1.6	1.6	2	
Fibre content (kg/m ³)						
Steel	-	30	-	15		
Polypropylene	-	-	5	3		
Density	2340	2274	2330	2385	2000	2450

Table (6.40) shows the measured compressive strength and modulus of elasticity of EPS-LWC, SCC and CC mixtures at 14 and 28 days.

Table 6.40: Mechanical properties of SCC, LWC and CC mixtures

Property	Age (day)	N-SCC	D-SCC	S-SCC	DS-SCC	LWC	CC
f'_c (MPa)	14	29.05	34.3	32.45	38.1	27.93	18.3
	28	33.3	38	38.1	45	30.96	24.8
E_c (GPa)	14	32.24	29.14	29.68	31.26	17.65	22.82
	28	35.39	35.76	35.76	36.1	20.32	24.95

Figures (6.66) and (6.67) compare the measured values of compressive strength and modulus of elasticity of the mixtures at ages 14 and 28 days respectively.

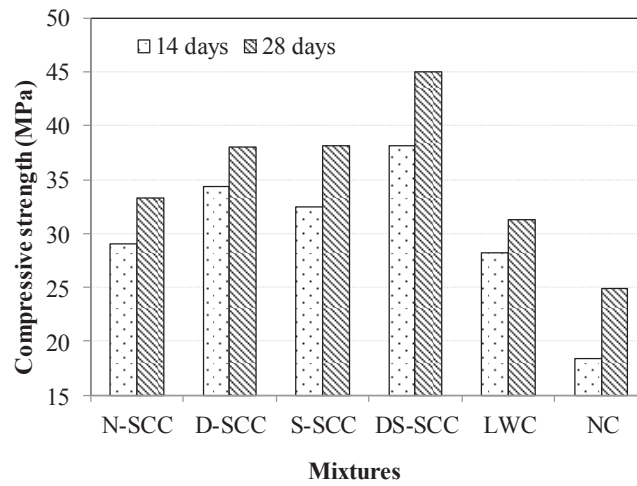


Figure 6.66: Compressive strength of EPS-LWC, SCC and CC mixtures at 14 and 28 days

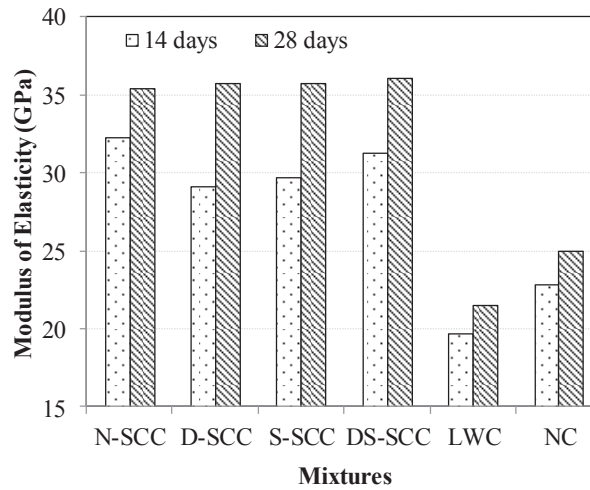


Figure 6.67: Modulus of elasticity of EPS-LWC, SCC and CC mixtures at 14 and 28 days

Figure (6.68) compares the modulus of rupture (flexural tensile strength) of CC, LWC and SCC mixtures at different ages.

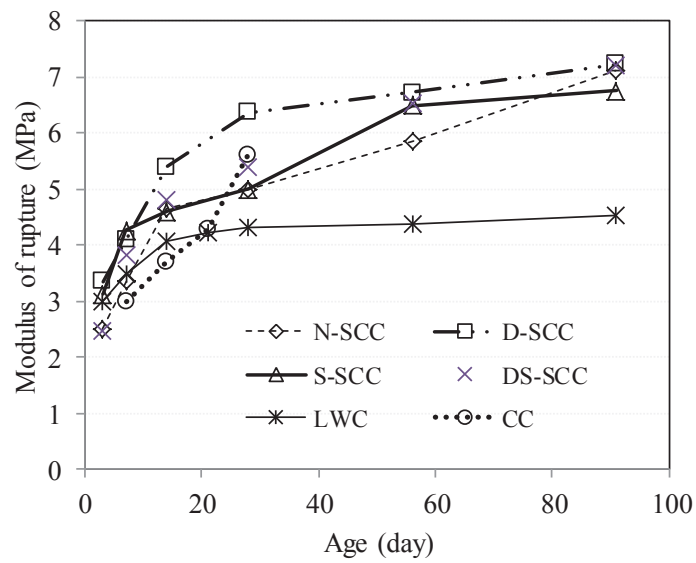


Figure 6.68: Comparison of the modulus of rupture in LWC, CC and SCC mixtures

According to Figure (6.68), EPS-LWC poses the lowest values of the modulus of rupture, especially after age 28 days. However, SCC mixture with and without fibre reinforcing are in the top range of the modulus of rupture at almost all ages.

- Shrinkage strain

Figure (6.69) compares the shrinkage strain of EPS-LWC, SCC and CC mixtures up to 240 days.

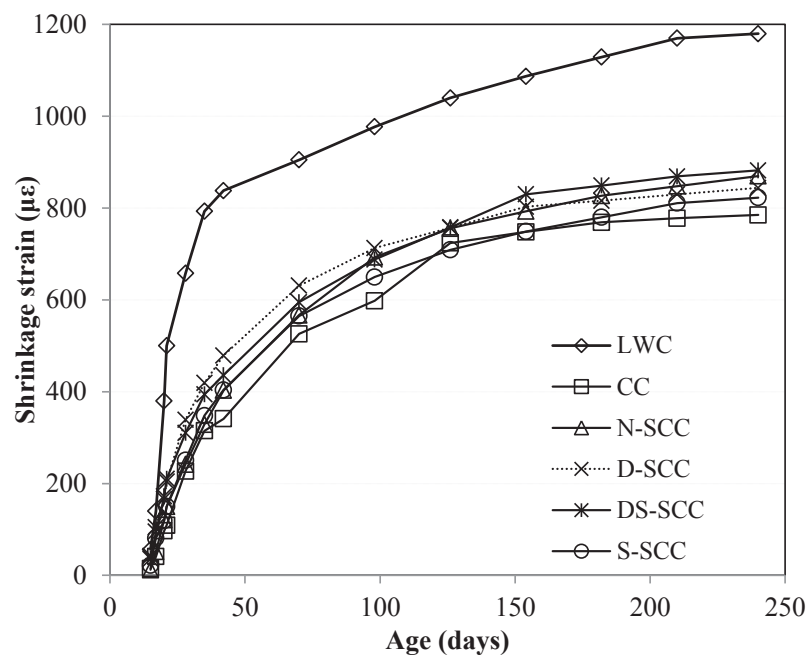


Figure 6.69: Comparison of shrinkage strain in EPS-LWC, SCC and CC mixtures

According to the Figure (6.69), the shrinkage strain in LWC is considerably higher than that of the CC and SCC mixtures. The free shrinkage strain in LWC and N-SCC mixtures is 51% and 10% greater than that of the CC mixture. The porous (permeable) nature of the lightweight concrete, relatively higher ratio of water to cement and high rate of the water loss cause higher values of the drying shrinkage in lightweight concrete.

Figure (6.70) compares the creep coefficient in EPS-LWC, SCC and CC specimens under sustained compressive stress. The specimens were loaded at age 14 days. The applied stress is equal to 40% of the compressive strength of the companion cylinders in each type of concrete.

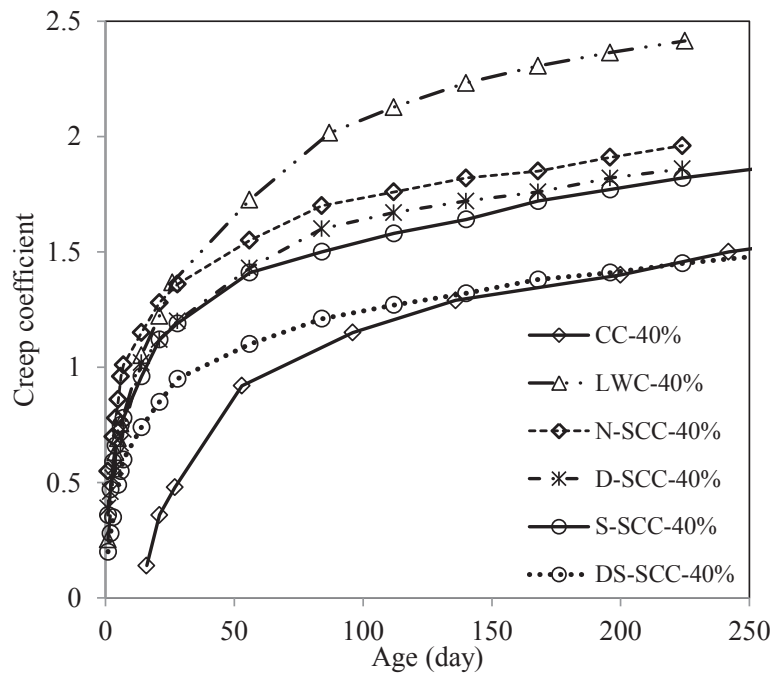


Figure 6.70: Creep coefficient of CC, SCC and EPS-LWC specimens loaded at age 14 days

According to Figure (6.70) the creep coefficient in EPS-LWC is considerably higher than that of SCC and CC specimens under the same level of sustained compressive stress.

The creep coefficient in EPS-LWC and CC are the highest and lowest recorded values after 240 days. The creep coefficient at 240 days in LWC and N-SCC mixtures is 1.61 and 1.3 times higher than that of the CC mixture respectively. The creep coefficient in fibre-reinforced SCC mixtures (D-SCC, S-SCC and DS-SCC) is lower than those of the EPS-LWC and N-SCC mixtures. However, the creep coefficient in the D-SCC and S-SCC mixtures is higher than that of the CC mixture. The final creep coefficient in DS-SCC is very close to that of the CC mixture.

REFERENCES

1. Aslani, F., *Experimental and numerical study of time-dependent behaviour of reinforced self-compacting concrete slabs*. PhD dissertation, University of Technology Sydney (UTS), 2014.
2. Nejadi, S., *Time-dependent cracking and crack control in reinforced concrete structures*. 2005, PhD thesis, The University of New South Wales Sydney, Australia.
3. AS-1411, *Methods for sampling and testing aggregates, particle size distribution-sieving method*. Standards Australia, Sydney, Australia, 2011.
4. RTA-06, *Materials test methods*. Regional transportation authority, Chicago, IL, USA., 2006. 1.
5. AS-1141.11.1, *Methods for sampling and testing aggregates - Particle size distribution - Sieving method*. Standards Australia, Sydney, Australia, 2009.
6. AS-1141.12, *Methods for sampling and testing aggregates - Materials finer than 75 μm in aggregates (by washing)*. Standards Australia, Sydney, Australia, 2011.
7. AS-1141.14, *Methods for sampling and testing aggregates - Particle shape, by proportional caliper*. Standards Australia, Sydney, Australia, 2011.
8. AS-1141, *Methods for sampling and testing aggregates*. Standards Australia, Sydney, Australia, 2011.
9. AS-1141.4, *Methods for sampling and testing aggregates - Bulk density of aggregate*. Standards Australia, Sydney, Australia, 2011.
10. AS-1141.6.1, *Methods for sampling and testing aggregates - Particle density and water absorption of coarse aggregate - Weighing-in-water method*. Standards Australia, Sydney, Australia, 2011.
11. AS-1141.22, *Methods for sampling and testing aggregates - Wet/dry strength variation*. Standards Australia, Sydney, Australia, 2008.
12. AS-1141.23, *Methods for sampling and testing aggregates - Los Angeles value*. Standards Australia, Sydney, Australia, 2009.
13. AS-1141-24, *Methods for sampling and testing aggregates - Aggregate soundness - Evaluation by exposure to sodium sulphate solution*. Standards Australia, Sydney, Australia, 2013.
14. AS-1141.32, *Methods for sampling and testing aggregates - Weak particles (including clay lumps, soft and friable particles) in coarse aggregates*. Standards Australia, Sydney, Australia, 2008.
15. AS-1141.25.2, *Methods for sampling and testing aggregates - Degradation factor - Coarse aggregate*. Standards Australia, Sydney, Australia, 2013.
16. AS-1141.13, *Methods for sampling and testing aggregates - Material finer than 2 micrometer*. Standards Australia, Sydney, Australia, 2007.
17. AS-1141.34, *Methods for sampling and testing aggregates - Organic impurities other than sugar*. Standards Australia, Sydney, Australia, 2007.
18. AS-1141.35, *Methods for sampling and testing aggregates - Sugar*. Standards Australia, Sydney, Australia, 2007.
19. AS-1478.1, *Methods for sampling and testing aggregates, particle size distribution-sieving method*. Standards Australia, Sydney, Australia, 2000.
20. AS-3972, *General purpose and blended cements*. Standards Australia, Sydney, Australia, 2010.
21. AS-2350, *Methods of testing Portland and blended cements*. Standards Australia, Sydney, Australia, 2006.

22. AS-2350.2, *Methods of testing portland, blended and masonry cements - Chemical composition*. Australian Standard, 2006.
23. AS-2350.8, *Methods of testing portland, blended and masonry cements - Fineness index by air permeability method*. Australian standard, 2006.
24. AS-2350.3, *Methods of testing portland, blended and masonry cements - Normal consistency*. Standards Australia, Sydney, Australia, 2006.
25. AS-2350.13, *Methods of testing portland, blended and masonry cements - Determination of drying shrinkage of cement mortars*. Standards Australia, Sydney, Australia, 2006.
26. AS-2350.11, *Methods of testing portland, blended and masonry cements - Compressive strength*. Standards Australia, Sydney, Australia, 2006.
27. AS-2350.4, *Methods of testing portland, blended and masonry cements - Setting time*. Standards Australia, Sydney, Australia, 2006.
28. AS-2350-5, *Methods of testing portland, blended and masonry cements - Determination of soundness*. Australian Standard, 2006.
29. AS-1210.8.1, *Methods of testing concrete- Method 8.1: method for making and curing concrete, compression and indirect tensile test specimens*. Standards Australia, Sydney, Australia, 2014.
30. AS-1012, *Methods of testing concrete*. Standards Australia, Sydney, Australia, 2014.
31. AS-3600-09, *Concrete structures*. Australian Standard, 2009.
32. Best, C.H. and M. Polivka *Creep of lightweight concrete*. Magazine of Concrete Research, 1959. **11**, 129-134.
33. ACI-209.2R-08, *Guide for Modeling and Calculating Shrinkage and Creep in hardened concrete*. American Concrete Institute (ACI), ACI Committee 209, Farmington Hills, MI 48331, U.S.A, 2008.
34. ACI-435-R95, *Control of Deflection in Concrete Structures*. American Concrete Institute (ACI) Committee 435, 2003.
35. Neville, A.M., *Properties of Concrete*. 4th ed , Birmingham, UK, 2010.

CHAPTER 7

EXPERIMENTAL PROGRAM (PHASE II) – FLEXURAL LOAD-DEFLECTION BEHAVIOUR

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Chapter 7: Flexural load-deflection behaviour

7.1. SHORT-TERM EXPERIMENTAL PROGRAM

The experimental part of this study consists of the laboratory investigation of Light-Weight Concrete (LWC) slabs to investigate the short-term load-deflection of slabs up to the failure point, instantaneous deflection under tow different levels of service loads and long-term defection under two levels of sustained service loads and the effects of creep and shrinkage.

Following are the main investigated parts in the experimental phase of this study:

- a) Load-deflection behaviour of slab under short-term loading until failure;
- b) Instantaneous flexural deflection of slabs under service loads;
- c) Time-dependent flexural deflection of slabs under service load and effects of creep and shrinkage;
- d) Comparing the experimental results with the previously conducted laboratory experiments on idem slabs made with Self-Compacting Concrete (SCC) and Conventional Concrete (CC).

For instantaneous and time-dependent deflection study, different points and regions were monitored to record the changes of deflection and strain in the slabs. Deflection was monitored at mid-span and the point on the border of high moment region of the span length. Variation of the strain in the tensile reinforcing bars embedded in the concrete was monitored within the high moment region of the slab length. Strain of the concrete under service sustained loading and effects of creep and shrinkage were recorded in the concrete surface at the level of

reinforcement steel. Additionally, the compressive strain of concrete on top surface at the mid-span of slab was measured under the sustained loading and effects of creep and shrinkage.

For short-term load-deflection investigation, the strain and deflection were monitored at the same positions under the load increment until the failure of the slabs.

Properties of concrete including the compressive strength, modulus of elasticity and modulus of rupture and splitting tensile strength at different ages were measured on companion specimens (as presented in Chapter Five).

The main objectives of the laboratory tests in the experimental phase are as follows:

- Better understanding of the mechanisms associated with LWC flexural deflection;
- To obtain benchmark, laboratory-controlled data to assist in the development of rational design-oriented procedures for the control of deflection in slabs;
- To study the difference between elastic and instantaneous deflection of LWC slabs;
- To modify the ratio of long-term to short-term deflection of LWC slabs;
- Investigating the effect of loading history on deflection and strain changes on identical specimens made of LWC;
- Proposing a new model for effective moment of inertia for deflection calculations;
- To verify the accuracy of the existing models of LWC properties to simulate the nonlinear behaviour of RC structures in Finite Element (FE) analysis in the ATENA program;
- Proposing a new model for tensile stress-strain behaviour of the LWC and verifying it by test results and FE analysis;
- To investigate the accuracy of the commonly used coefficient of creep and shrinkage (k_{cs}) and other existing models in the calculations by applying test results in FE analysis.

7.2. TEST PARAMETERS AND REINFORCEMENT LAYOUT

In addition to the above-mentioned objectives, instantaneous and long-term deflection and short-term load-deflection behaviour are compared with the previously conducted experiments on similar section and length of slabs made in other types of concrete. In this regard, the experimental investigation by Nejadi (2005) [1] on the Conventional Concrete (CC) slabs and the experimental investigation by Aslani (2014) [2] on the Self-Compacting Concrete (SCC) slabs with and without fibre reinforcing have been considered for comparison and verification purposes. Nejadi (2005) [1] investigated the time-dependent cracking of two slabs made of the

same batch of concrete, while Aslani (2014) [2] studied four pairs of one-way slabs made of 4 different batches of normal and fibre-reinforced SCC mixtures.

All LWC, CC and SCC slabs were singly reinforced by 4N12 steel bars at the tensile zone. There was no shear reinforcement applied in the slabs. All slabs were simply supported with the same span length. The applied sustained loads on the CC slabs were uniformly distributed loads of about 20%, 30% of the ultimate bending capacity of the slab section. While, the sustained loading level on the SCC and LWC slabs were about 30% and 40% of the ultimate bending capacity of the slab section.

Overall, the changing parameters in the slabs of LWC, SCC and CC types are of mechanical properties of concrete, shrinkage, creep and loading level. These parameters are investigated and compared with the obtained results from the previously conducted experimental results on idem slabs made with conventional and self-compacting concrete. Considering the LWC, CC and SCC slabs, the parameters of tests varied, including one LWC mixture, one CC mixture and four SCC mixtures.

This chapter explains the testing setup, investigated parameters and results of the short-term load-deflection of slabs. The instantaneous and long-term deflections of slabs are comprehensively discussed in Chapters 7 and 8 of this study, respectively.

7.3. CONSTRUCTION OF THE SPECIMENS AND TEST PROCEDURE IN SHORT-TERM LOAD-DEFLECTION TEST

To investigate the short-term load-deflection of slabs made of Light-Weight Concrete Containing Expanded Poly-Styrene beads (EPS-LWC), four simply-supported one-way slabs were subjected to increasing loading up to the failure of the span.

All specimens were constructed in 3800 mm long formwork and were simply supported over a 3500 mm span. Variations of the mechanical properties of concrete were measured on companion cylinders and prisms at ages 3, 7, 14, 21, 28, 56 and 91 days.

Before casting each specimen, the inside surface of the mould was cleaned and coated with a thin layer of concrete release agent to prevent adhesion of the concrete. The LWC was placed into the mould in equal layers and compacted by internal vibration. For each layer, the concrete was vibrated until the surface became smooth in appearance. Floating of the BST beads to the concrete surface due to vibration also controlled the vibration duration of each layer. Sufficient concrete was placed into the top layer to overfill the mould, after which the surface was stripped off and finished with a steel trowel. The companion specimens were also cast at the same time

as the test specimens. Within two hours of casting, the slab specimens were covered with wet hessian and plastic sheets and left in their moulds for three days. After three days they were removed and kept moist continuously using a thick covering of wet hessian until the age of testing. Figure (7.1) shows the slab moulds before and after pouring of LWC.

The slab specimens were each nominally 3500 mm long by 400 mm wide. In all slabs, the nominal distance from the soffit of the centroid of the main reinforcement was 25 mm. Each slab was reinforced with 4N12 steel bars. Details of the cross section for all the LWC, SCC and CC slabs are presented in Figure (7.2). Also, Table (7.1) shows the reinforcement details and the ratio of tensile bars in the effective section area.



Figure 7.1: Slab moulds before and after pouring of lightweight concrete

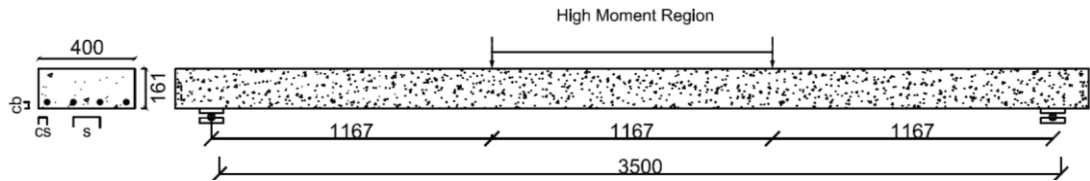


Figure 7.2: Dimensions and reinforcement details for LWC slab specimens

Table 7.1: Details of slabs for short-term load-deflection testin of slabs

Specimen	No. of bars	c_b (mm)	c_s (mm)	s (mm)	Bar dia. (mm)	Steel area (mm ²)	$\rho=A_{st}/bd$
Slab - a	4N12	25	40	103	12	452	0.0083
Slab - b	4N12	25	40	103	12	452	0.0083

7.4. LOADING BLOCKS

To study the effect of loading history on the deflection behaviour of the specimens, two different levels of service loads at the age of 14 days have been applied to the top surface of the slabs. To distribute the applied loads on the slabs uniformly, timber pads with 100mm width

×30mm thickness and 450 mm length were used at equal intervals under each loading block. The dimension of loading blocks and centre to centre spacing of loading pads (timber pad) under loading blocks are shown in Figure (7.3). All dimensions are in millimetres.

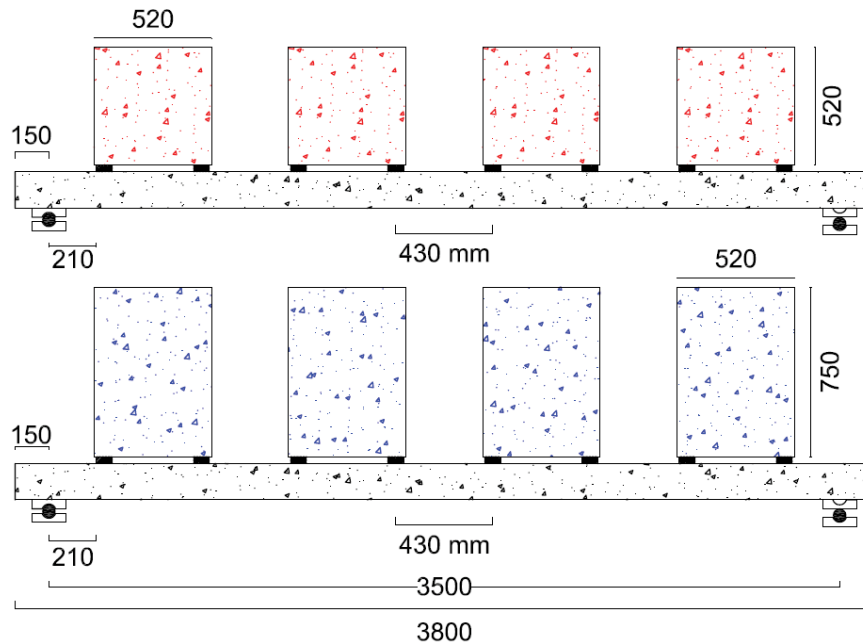


Figure 7.3: Loading blocks and arrangement to supply two loading levels on slabs

7.5. INSTRUMENTATION OF THE SLABS AND TEST SETUP

7.5.1. Strain Gauges

Since the critical area for the flexural behaviour of the simply supported slab is in the middle one-third of span length, strain changes and deflection variation were monitored in this region. According to Figure (7.4) different strain gauges were attached to the tensile reinforcement to record the strain variation in steel bars under short-term loading, sustained service loads and the effects of creep and shrinkage.

To monitor the strain changes in tensile concrete, a different type of strain gauges were attached to the concrete surface at the same level and direction of the monitored strain in steel bars. It helps to compare the changes of strain in concrete and reinforcement at the same level in the tensile zone. Furthermore, the bond characteristics of the steel bars and concrete could be investigated by these strain gauges. To record the compression strain on concrete surface, two strain gauges in the same direction of middle steel bars were attached to the top surface of the concrete slab at mid-span. Figure (7.5) shows the arrangement and positioning of strain gauges attached to steel bars and concrete surface in LWC slabs. The strain gauges arrangement in SCC and CC slabs are similar except for the top surface strain gauges.

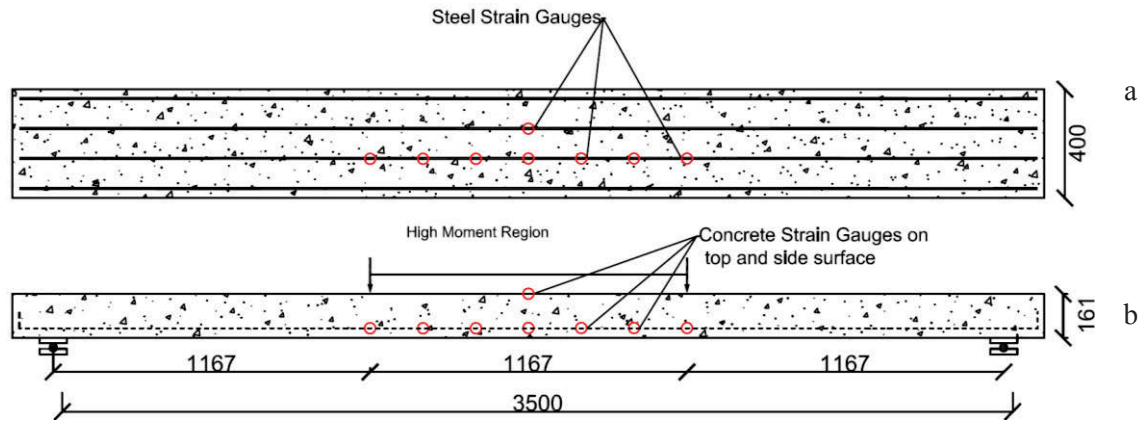


Figure 7.4: a) arrangement of strain gauges attached to steel bars and b) Position and number of the strain gauges on concrete surface (side and top)

WFLA6-11 and PL60-11/3L are two different types of strain gauges used to measure the strain in steel bars and concrete surfaces respectively. The WFLA-6-11 strain gauge is a type of waterproof strain gauge having a pre-attached vinyl lead wire and a continuous coating with epoxy resin. The coating is transparent and flexible, which ensures easy installation of the gauge. PL-60-11 is a wire strain gauge having polyester resin backing. It is mainly used for measurement of strain in concrete, mortar or rock. This gauge has a pre-attached vinyl lead wire to a P series. Works for lead wire connection such as strain gauge terminal installation and lead wire soldering are not required, which saves much time and labour. Figure (7.5) shows these types of strain gauges applied in the current study.

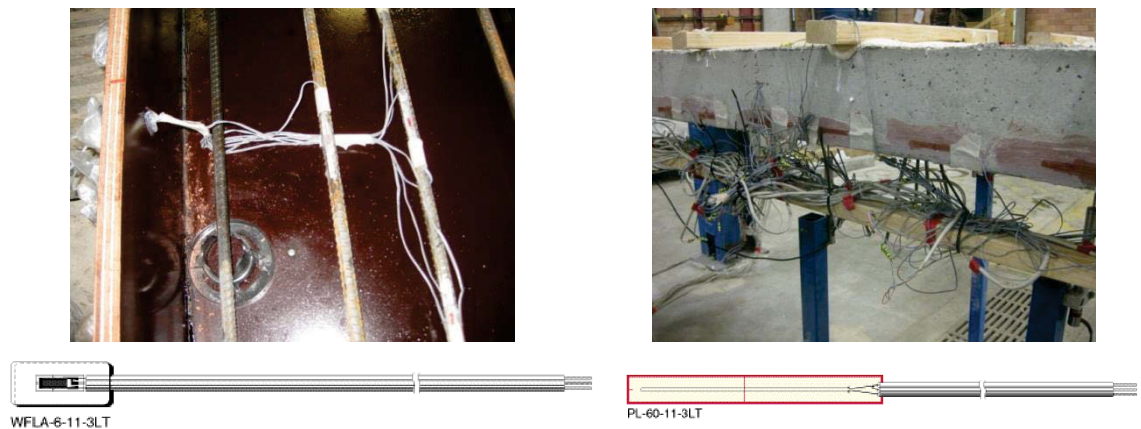


Figure 7.5: Installed strain gauges on steel bars and surface of concrete

7.5.2. DEFLECTION RECORDING DEVICE

The deflection was recorded in the mid-span of slabs under short-term loading, instantaneous and time-dependent sustained loading by a MTS-Temposonics R-Series position sensor.

Furthermore, to compare the deflection behaviour in high flexural moment regions, another point on the border of the critical zone i.e. the point in one-third of span length was monitored. The Linear Variable Differential Transformer (LVDT) (also called just a differential transformer, linear variable displacement transformer, or linear variable displacement transducer) was used to monitor the deflection. It is a type of electrical transformer used to measure the linear displacement (position). Another version of this device that is used to measure rotary displacement is called a Rotary Variable Differential Transformer (RVDT). The LVDT transforms a position or linear displacement from a mechanical reference (zero, or null position) into a relative electrical signal containing phase (for direction) and amplitude (for distance) information. Figure (7.6) shows the installed LVDT at two points of each slab to record the deflection variation.



Figure 7.6: Installation of LVDT at two points of slab span

7.6. LOAD-DEFLECTION BEHAVIOUR

For short-term investigation of load-deflection relationship, four singly reinforced LWC slab specimens were cast and moist cured for 28 days. After 28 days the wet hessian was removed and the specimens were identified and tested at different ages. Each specimen was slowly and gradually loaded to failure over a period of approximately four hours. All the specimens were simply supported on a 3.5 m span and tested to failure to investigate the mid-span deflection, changes of strain in the critical bending region and distribution and extent of primary and secondary cracking under short-term loading using two equal point loads applied at the third points of the span, at ages greater than 28 days. Deflection at the mid-span was monitored. The schematic diagram of the test set-up is shown in Figure (7.7).

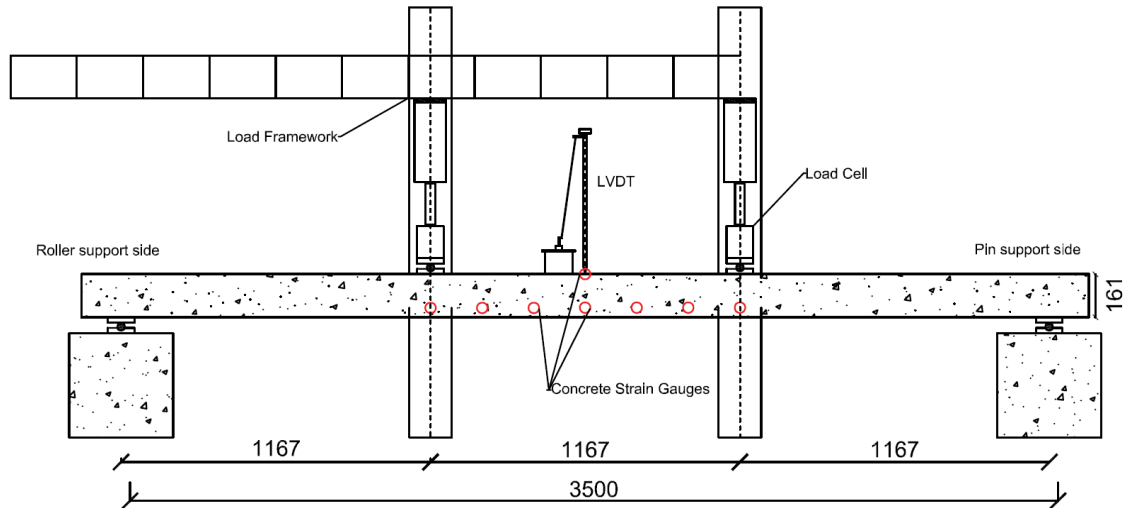


Figure 7.7: Load- deflection test arrangements for all specimens

The position sensor and resistance strain gauge measurements were retrieved from a data acquisition system connected to a computer while the load was supplied by a hydraulic jack, attached to an electrically powered pressure pump. Figure (7.8) shows views of the experimental set-up. Each specimen was simply supported at each end before testing, and then one Temposonics R-Series position sensor was attached at the mid-span and linked to a computer. Details of supports and the sensor are shown in Figure (7.9) also. Along with the measured strain variation in steel bars, the changes of strain in the concrete surface at the level of reinforcement and the top surface at the mid-span were recorded by different type of strain gauges.

7.6.1. Test Results of Lightweight Concrete Slabs

To set the basis of the monitored data, and to calculate the exact values of deflection at mid-span and the strain changes by increasing the loading value, the initial readings were taken at zero load condition. To have an accurate estimation, the ultimate load capacity of the section was calculated before the test. The load was then applied by 3 KN increments in each step on the slab specimens. Although this study emphasises the deflection behaviour, every visible crack on the tensile concrete surface was monitored at each load increment. This rate of loading was maintained until about 60% of the expected ultimate load capacity. Then, the load was gradually increased monotonically to failure and crack patterns were recorded at each load step.



Figure 7.8: Load-deflection test setup and slab failure

The experimental results from four lightweight concrete slabs made from the same concrete mixture are presented. The results contains the measured material properties, variation of the strain in eight points of the steel bars, and the strain in the concrete surface at the level of steel bars and two point on top surface of the concrete. Also the load-deflection diagrams at the mid-span of the slabs are presented. The results from each slab are discussed in detail and the results from all slabs are also compared. The relevant diagrams for concrete strain at top and side surfaces, steel strain, and some extra view of the test results are presented in Appendix-C.

7.6.2. Material Properties:

Table (7.2) gives a summary of the mechanical properties of the lightweight concrete used in the slab specimens. Comprehensive information of the testing procedure and the detailed results are given in Chapter 5 of this study. The mechanical properties are performed by standard test methods and specimens to determine the compressive strength, splitting tensile strength, modulus of rupture and modulus of elasticity at the ages of 3,7,14,21,28,56 and 91 days.

Table 7.2: Mechanical properties of EPS-lightweight concrete used in the slab specimens

Property	Specimen age (days)						
	3	7	14	21	28	56	91
Compressive strength (MPa)	14.49	25.38	27.93	29.62	30.96	32.33	33.3
Modulus of rupture (MPa)	1.44	2.3	2.6	2.78	2.89	2.98	3.08
Modulus of elasticity (GPa)	11.63	15.68	17.65	19.48	20.32	21.1	21.89
Splitting tensile strength (MPa)	2.982	3.498	4.056	4.218	4.305	4.374	4.524

7.6.3. Load-deflection behaviour of slab LWC-1

The first cracking at the mid-span took place approximately at load of 6 kN (3 kN from each loading cell). Then, the increasing load increased the number and width of the cracks, and caused the gradual growth in the mid-span deflection of up to 60% of the expected ultimate load. The rate of deflection increment was higher at higher loading levels by increasing the crack patterns and width. The majority of the crack pattern happened at the high moment region due to tensile stress.

Figure (7.9) shows the crack pattern and deflection of slab LWC-1 at the failure point.

The measured deflection at mid-span versus applied load in the slab LWC-1 is shown in Figure (7.10). The amount of tensile reinforcement in the slab provides an under-reinforced section with a probability of ductile failure. The test showed a good ductile failure of slab LWC-1 with an extended flat plateau in the load-deflection curve. The ultimate capacity was reached when $P=46.5$ kN (total load of 2 load cells), when crushing of the top compressive surface occurred. The maximum deflection corresponding to the failure point was 95.9 mm. The slab LWC-1 failed in flexure in the pure moment zone, however, the first failure point with the compressive concrete crushing above the crack was not exactly in the mid-span of slab.



Figure 7.9: Deflection and cracking pattern of LWC-1 at failure point

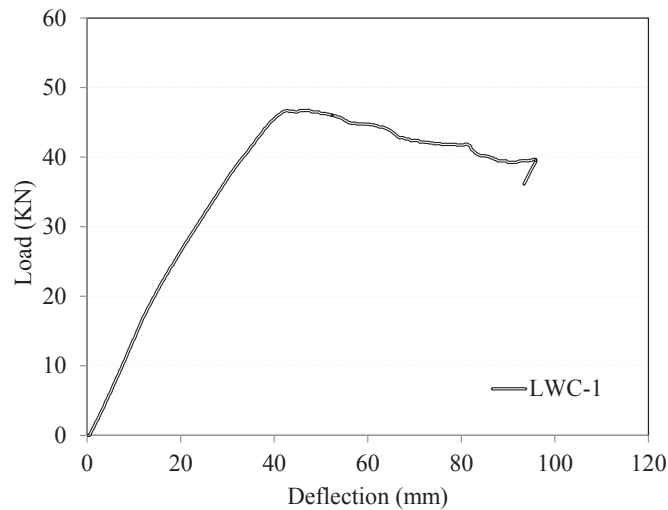


Figure 7.10: Load-deflection curve of slab LWC-1 at mid-span

7.6.4. Load-deflection behaviour of slab LWC-2

The first cracking at the mid-span took place approximately at load of 6 kN (3 kN from each loading cell). Then, the increasing load increased the number and width of the cracks, and caused the gradually growth in the mid-span deflection of up to 60% of the expected ultimate load. The rate of deflection increment was higher at higher loading levels by increasing the crack patterns and width.

Figure (7.11) shows the crack pattern and deflection of slab LWC-2 at the failure point.



Figure 7.11: Deflection and cracking pattern of LWC-2 at failure point

The measured deflection at mid-span versus applied load is shown in Figure (7.12). The test showed a good ductile failure of the slab LWC-2 in the load-deflection curve. The ultimate strength was recorded as $P=48$ KN, when the top compressive surface was crushed. The maximum deflection of the slab at the failure point was 88 mm. The slab LWC-2 failed in flexure in the pure moment zone, however, the first failure point with the compressive concrete crushing above a crack was close to but not exactly at the mid-span of slab.

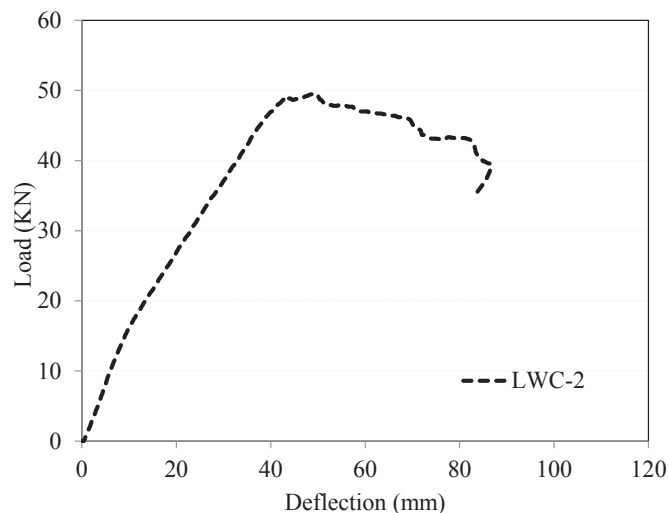


Figure 7.12: Load-deflection curve of slab LWC-2 at mid-span

7.6.5. Load-deflection behaviour of slab LWC-3

The first cracking at the mid-span took place approximately at load of 6 KN (3 KN from each loading cell). Then, the increasing load increased the number and width of the cracks, and

caused the gradual growth in the mid-span deflection of up to 60% of the expected ultimate load. The rate of the deflection increment was higher at higher loading levels by increasing the crack patterns and width.

Figure (7.13) shows the crack pattern and deflection of slab LWC-3 at the failure point.

The measured deflection at mid-span versus applied load is shown in Figure (7.14). The test showed a good ductile failure of the slab LWC-3 in the load-deflection curve. The ultimate strength was recorded at $P=41.53$ KN, when the top compressive surface was crushed. The maximum deflection of slab at the failure point was 114.7 mm. The slab LWC-3 failed in flexure in the pure moment zone, however, the first failure point with the compressive concrete crushing above a crack was close to but not exactly at the mid-span of slab.



Figure 7.13: Deflection and cracking pattern of LWC-3 at failure point

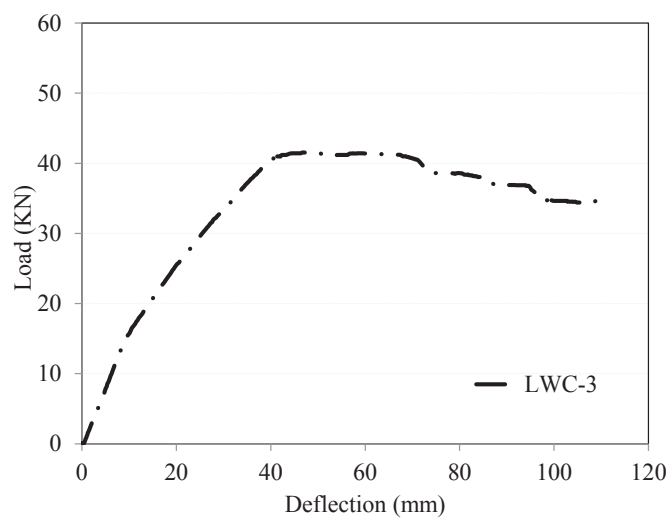


Figure 7.14: Load-deflection curve of slab LWC-3 at mid-span

7.6.6. Load-deflection behaviour of slab LWC-4

The first cracking at the mid-span took place at approximately load of 6 kN (3 kN from each loading cell). Then, the increasing load increased the number and width of the cracks, and caused the gradual growth in the mid-span deflection of up to 60% of the expected ultimate load. The rate of deflection increment was higher at higher loading levels by increasing the crack patterns and width.

Figure (7.15) shows the crack pattern and deflection of slab LWC-4 at the failure point.



Figure 7.15: Deflection and cracking pattern of LWC-4 at failure point

The measured deflection at mid-span versus applied load is plotted in Figure (7.16). The ductile failure of slab LWC-4 was in agreement with the expected behaviour of slab with the applied reinforcement ratio. The ultimate capacity was reached at $P=49.52$ kN, when crushing of the top compressive surface occurred. The maximum deflection corresponding to the failure point was 86.65 mm. The failure of the slab LWC-4 occurred in the pure moment zone, however, the first failure point with the compressive concrete crushing above a crack was not exactly in the mid-span of the slab.

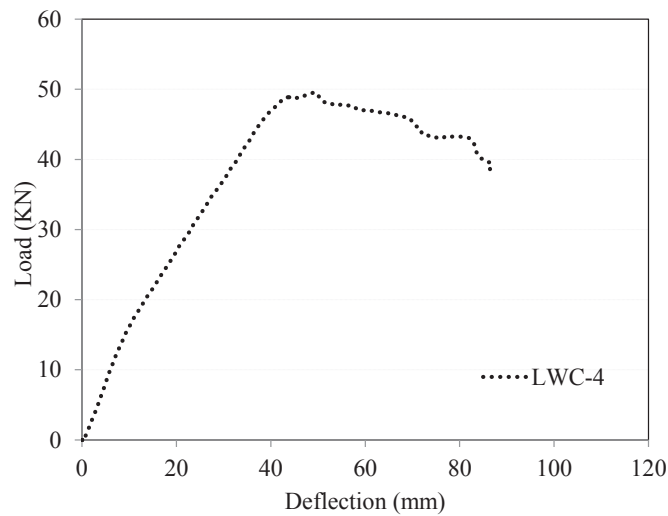


Figure 7.16: Load-deflection curve of slab LWC-4 at mid-span

7.6.7. Comparison of load-deflection curves in all LWC slabs

Figure (7.17) compares the load-deflection behaviour of four lightweight concrete slabs. All slabs experience a similar trend of ductile failure and there is no significant difference between the load-deflection curves of slabs.

The average values of the ultimate load-capacity and the maximum deflection are 46.4 kN and 96 mm respectively. Slabs LWC-2 and LWC-4 are compatible in the recorded data, while the recorded data of ultimate strength and the maximum deflection in LWC-3 is different compared to the other slabs. The load-deflection diagram in slab LWC-1 is approximately the average of all specimens.

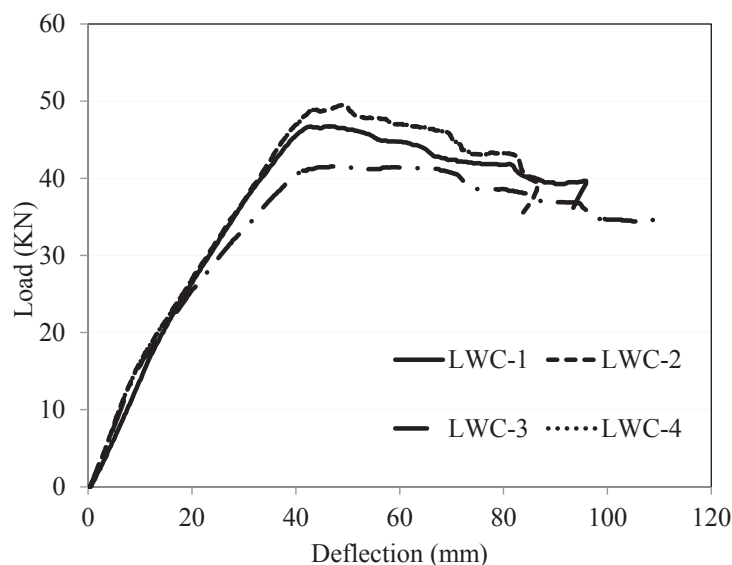


Figure 7.17: Comparing the load-deflection curves in four LWC slabs

According to Figure (7.17), the elastic load-deflection curve is seen up to 15% of the ultimate load capacity point. The yielding point of steel bar is about 50% of the ultimate load-capacity of slab section in almost all slabs.

7.7. PREVIOUS EXPERIMENTS ON LOAD-DEFLECTION BEHAVIOUR OF SLABS

Nejadi (2005) [1] and Aslani (2014) [2] conducted experimental programs to investigate the time-dependent behaviour of Conventional Concrete (CC) and Self-Compacting Concrete (SCC) respectively. The main objective in their study was to monitor the cracking of flexural elements over long period of time under the effects of both sustained service load and of creep and shrinkage. They have investigated the load-deflection relationship by applying an increasing short-term loading up to failure on the slabs. They also have recorded the instantaneous and long-term deflection of test specimens under service loads and creep and shrinkage effects.

The current study emphasises the time-dependent deflection of flexural elements. To extend the test results in a wider range of comparable data of previously conducted experiments, this study used idem dimensions and support conditions of slabs with a different type of concrete. In other words, to provide a basis of comparison for the deflection variation of the slabs under different levels of loading, three types of concrete have been used in the slab members with the same dimensions, bar arrangements and support conditions. By choosing the same number, diameter, arrangement and cover of steel bars, it is obvious that the other effective parameters such as the reinforcement ratio bar diameter and bar orientation has no effect on the test results

7.7.1. Short-term loading of conventional concrete slabs (Nejadi, 2005)

Nejadi (2005) [1] investigated the flexural cracking of beams and slabs in short-term and long-term periods. Twelve prismatic singly-reinforced concrete specimens including six beams and 6 slabs were cast and moist-cured for 28 days. All specimens were simply supported on a 3.5 m span and tested to failure to investigate the distribution and extent of primary and secondary cracking under short-term loading.

All specimens were fabricated in 3800 mm long formwork and were simply supported over a 3500 mm span. Eleven Demec targets were glued on the side face of each specimen at the steel level to measure the concrete surface strains and two targets were fixed onto the top fibre of the side face at midspan with HBB-X60 non-shrink adhesive. Also, ten electric resistance strain gauges were attached to one of the main reinforcement bars. Deflection at mid-span was

measured by LVDT. The measured data from LVDT and strain gauges were retrieved through a data acquisition system connected to a computer, while the load was supplied by a hydraulic jack connected to an electrically powered pressure pump.

Figure (7.18) shows the schematic diagram of the test set-up for short-term loading of beams and slabs.

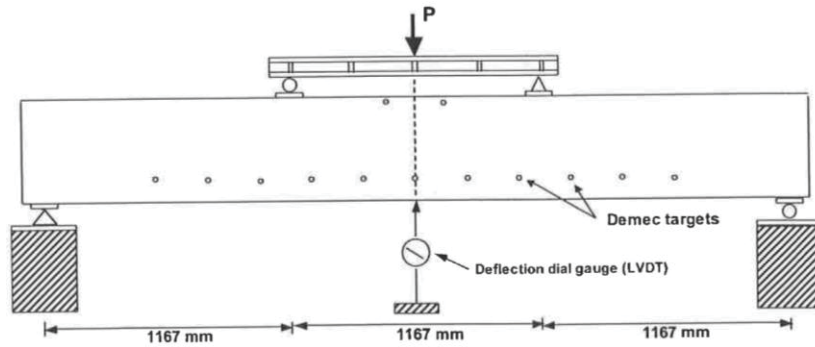


Figure 7.18: Test arrangement for all the beams and slab in short-term loading [1]

Table (7.3) presents the width (b) and height (h) of the sections, number and diameter of tensile bars for the beams and slabs. All specimens were simply supported along the 3.5 m span length. There were no compressive bars and stirrups in the sections.

Deflection at mid-span, crack widths, crack patterns, steel strains within the high moment region and concrete surface strains at the steel level were recorded at each load increment in the post-cracking range. The development of a primary crack pattern throughout the test was monitored also.

Table 7.3: Detail of test specimens in the experiment by Nejadi (2005) [1]

Beams	Tensile bar	L(m)	$b \times h$ (mm ²)	Slabs	Tensile bar	L(m)	$b \times h$ (mm ²)
B1-a	2N16	3.5	250×348	S1-a	2N12	3.5	400×161
B1-b	2N16	3.5	250×348	S1-b	2N12	3.5	400×161
B2-a	2N16	3.5	250×333	S2-a	3N12	3.5	400×161
B2-b	2N16	3.5	250×333	S2-b	3N12	3.5	400×161
B3-a	3N16	3.5	250×333	S3-a	4N12	3.5	400×161
B3-b	3N16	3.5	250×333	S3-b	4N12	3.5	400×161

Figure (7.19) shows the failure of Beam B1-a and slab S2-b as samples of the conventional concrete beams and slabs failure in the tests conducted by Nejadi (2005).



Figure 7.19: Failure of a) slab S2-b and b) beam B1-a [1]

The failure loads and the corresponding deflection for beams and slabs are given in Table (7.8). Accordingly, the results for slabs are graphically presented in Figure (7.38). The data for slab S1-B is not available to include in the graph presented in Figure (7.38).

Table 7.4: Summary of the results from short-term flexural test [1]

Beams	Failure load (KN)	Deflection at failure load (mm)	Slabs	Failure load (KN)	Deflection at failure load (mm)
B1-a	109	66	S1-a	22	210
B1-b	103	72	S1-b	21	211
B2-a	112	68	S2-a	37	178
B2-b	104	77	S2-b	36	204
B3-a	138	10	S3-a	50	136
B3-b	127	9	S3-b	47	156

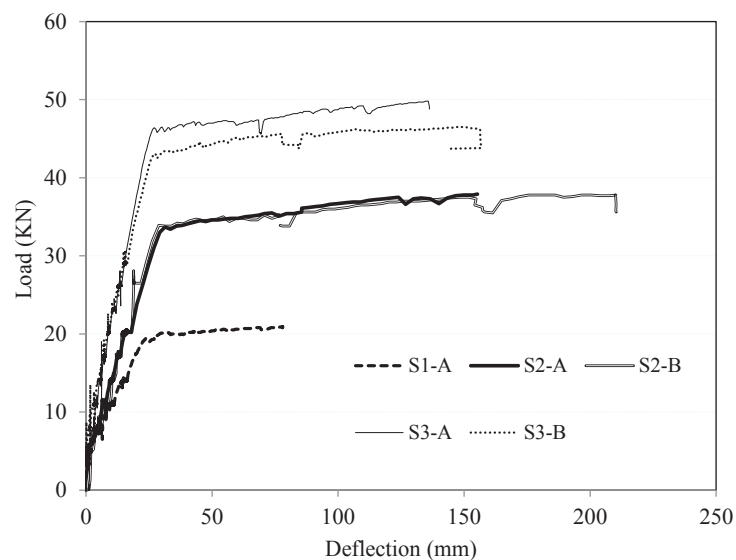


Figure 7.20: Short-term load-deflection curves for conventional concrete slabs

Except for the ratio of tensile reinforcement in the section, all slabs are similar. However, according to Figure (7.20), there are considerable differences in the load-deflection trend, the failure load, and the corresponding deflection to the failure load. In slabs S3-a and S3-b with the highest ratio of tensile reinforcement, the failure load and the corresponding deflection values are the highest and lowest values of all slabs respectively.

The reinforcement ratio in slabs S_2 and S_3 are 50% and 100% higher than the reinforcement in slabs S_1 , however, the average failure load in slabs S_2 and S_3 are 69% and 125% higher than that of slab S_1 .

7.7.2. Short-term loading of self-compacting concrete slabs (Aslani, 2014)

The experiment of Nejadi (2005) [1] for short-term and long-term monitoring of cracking in conventional concrete beam and slab was followed by Aslani (2014) [2] with similar specimens of Self-Compacting Concrete (SCC) slabs.

In the experimental investigation by Aslani (2014) [2], eight singly-reinforced SCC and Fibre Reinforced Self-Compacting Concrete (FRSCC) slab specimens were cast and moist-cured for 28 days. All the specimens were simply supported on a 3.5 m span and tested to failure to investigate the distribution and extent of primary and secondary cracking under short-term loading. The objectives are similar to the objectives of Nejadi (2005) [1] and the same dimension slabs were investigated under short-term loading.

Deflections at mid-span, crack widths, crack patterns, steel strains within the high moment region, and concrete surface strains at the steel level were recorded at each load increment in the post-cracking range and development of the primary crack pattern was monitored throughout the test duration. Reinforcement and cover details and the SCC slab dimensions are explained in Table (7.9).

Table 7.5: Detail of test specimens in the experiment by Aslani (2014) [2]

Slabs	Tens. bar	Cover(mm)	L(m)	b×h (mm ²)
N-SCC-A	4N16	19	3.5	400×161
N-SCC-B	4N16	19	3.5	400×161
D-SCC-A	4N16	19	3.5	400×161
D-SCC-B	4N16	19	3.5	400×161
S-SCC-A	4N16	19	3.5	400×161
S-SCC-B	4N16	19	3.5	400×161
DS-SCC-A	4N16	19	3.5	400×161
DS-SCC-B	4N16	19	3.5	400×161

Figure (7.21) shows a general view of test set-up for short-term loading of SCC and FRSCC slabs. The instrumentations process and measurement method of the strain and midspan deflection are similar to those in the experiment by Nejadi (2005) [1].



Figure 7.21: General view of the loading cells, concrete strain gauges and LVDT set-up [2]

Table (7.6) shows a summary of the load-deflection result of SCC slabs under short-term loading. Figure (7.22) shows the load-deflection curves of the slab at mid-span. Except for the inclusion or type of the applied fibre in the concrete mixture, all SCC and FRSCC slabs are similar. In general, the failure load in the FRSCC slab is higher than the normal SCC slab series. However, the average value of deflection at failure point is high in the FRSCC slabs.

Table 7.6: Summary of the results from short-term flexural test

Slab	N-SCC-A	N-SCC-B	D-SCC-A	D-SCC-B	S-SCC-A	S-SCC-B	DS-SCC-A	DS-SCC-B
Failure load (KN)	49	48.5	53	52	50	48	56	54
Deflection at failure load (mm)	180	163	205	177	185	167	220	182

In Table (7.6), N-SCC, D-SCC, S-SCC and DS-SCC refer to SCC mixtures with no fibre, steel fibre, polypropylene, and hybrid fibre, respectively.

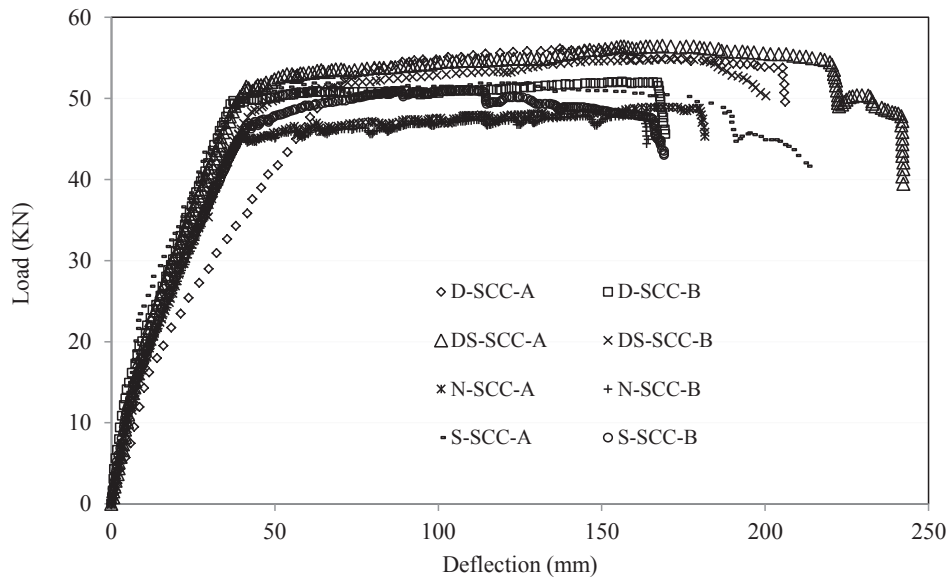


Figure 7.22: Load-deflection curve for slab SCC and FRSCC slabs at mid-span

7.8. COMPARISON OF LOAD-DEFLECTION CURVES IN LIGHTWEIGHT, CONVENTIOANL AND SELF-COMAPCTING CONCRETE SLABS

Figure (7.23) compares the load-deflection curves in LWC, SCC and CC slabs. The figure includes all slabs with different mix design and various ratios of reinforcement.

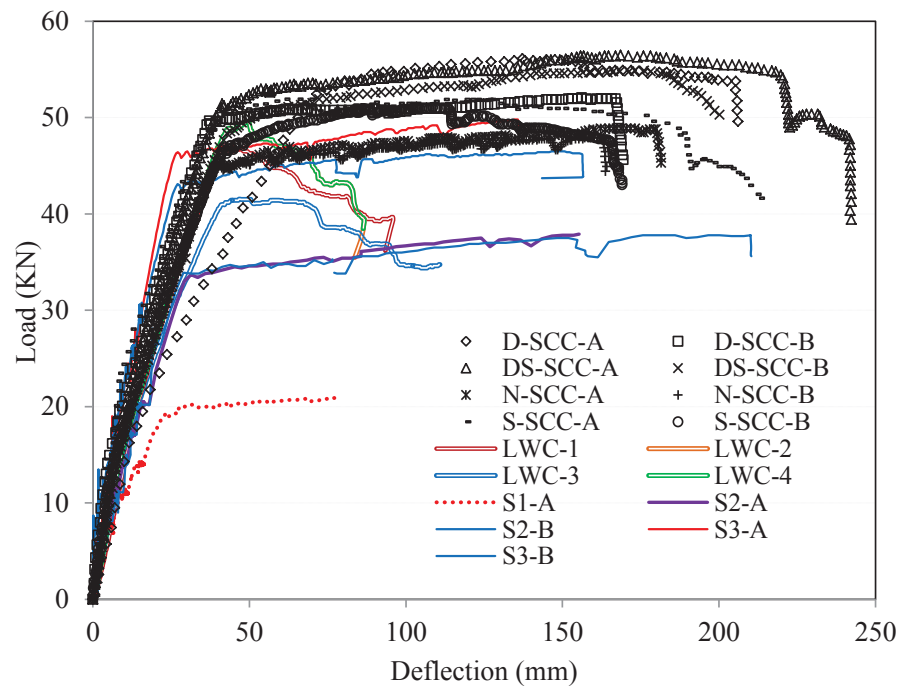


Figure 7.23: Comparison of the load-deflection curves in all LWC, SCC and CC slabs

To have a better comparison, the SCC slabs with fibre reinforcing and the CC slabs with 2N12 and 3N12 reinforcement are eliminated from the diagrams in Figure (7.23). Figure (7.24) shows the comparison between four LWC slabs (LWC, A, B, C, D), two SCC slabs (N-SCC A, B) and two CC slabs (S3-A, B) in which the slab dimensions and steel bar details are the same.

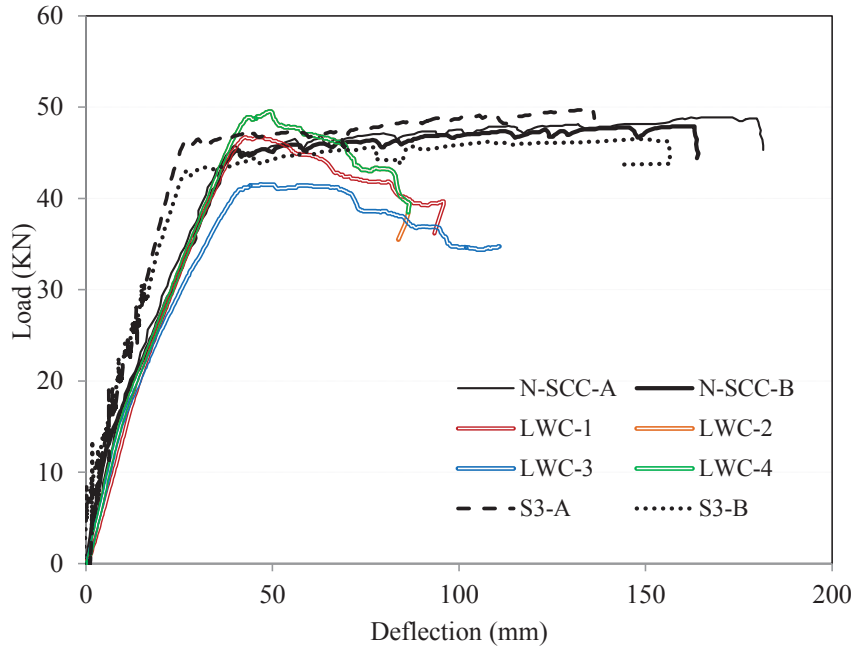


Figure 7.24: Load-deflection curves in identical LWC, SCC and CC slabs

Figure (7.25) compares the average load-deflection curves of the LWC, SCC and CC slabs. The failure load in all slabs are in the same range, however, the failure trend is completely different in LWC slabs. The failure in CC and SCC slabs are extensively ductile. The LWC slabs fail with less ductility, compared to the ductile failure of CC and SCC slabs.

The elastic range of the load-deflection curve in all slabs is very close. In other words, they experience a similar trend of load-deflection up in the elastic range.

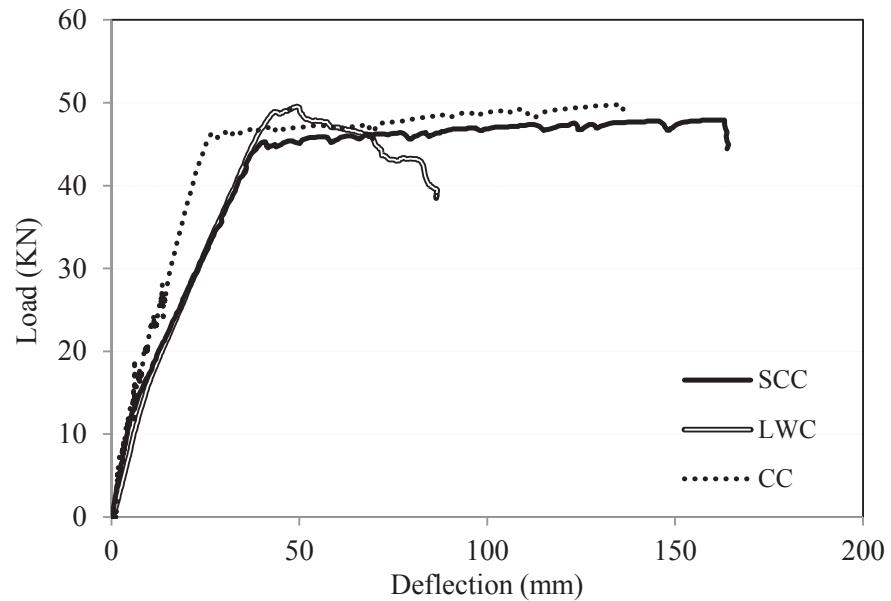


Figure 7.25: Comparison of the load-deflection curves in LWC, SCC and CC slabs

According to Figures (7.9), (7.11), (7.13), (7.15), (7.19) and (7.21), there is noticeable difference in cracking pattern of the slabs under increasing load in terms of the crack spacing and crack width. The average crack spacing in SCC and CC slabs are less than in EPS-LWC slabs. However, the crack width and height in the SCC and CC slabs are higher than those of the EPS-LWC slabs.

7.9. SUMMARY

The second phase of the experimental program, i.e. short-term flexural deflection of the slabs is discussed in this chapter. The experimental program for short-term load-deflection tests are explained in detail. The results were compared with the corresponding previously conducted experiments by Nejadi (2005) on the conventional concrete slabs and Aslani (2014) on the self-compacting concrete slabs.

Testing setup and slab details

The total number of four one-way lightweight concrete slabs was cast and moist-cured for 28 days. The slab specimens were each nominally 3500 mm long by 400 mm wide and 161 mm deep. Each slab was reinforced with 4N12 steel bars in tensile side of the section and the concrete cover was 19 mm. Then each slab was slowly and gradually loaded to failure over a period of approximately four hours.

All specimens were tested to failure to investigate the mid-span deflection, changes of strain in critical bending region and distribution and extent of the primary and secondary cracking under short-term loading using two equal point loads applied at the third points of the span, at ages greater than 28 days. To make an accurate estimation, the ultimate load capacity of the section was calculated before the test. The load was then applied by 3 KN increments in each step on the slab specimens. This rate of loading was kept constant until about 60% of the expected ultimate load capacity. Then, the load was gradually increased monotonically to failure and crack patterns were recorded at each load step.

Load-deflection behaviour of lightweight concrete slabs

Table (7.7) describes the ultimate failure load and the maximum deflection of lightweight concrete slabs under short-term loading. Since all slabs were similar, the average values of the failure load and maximum deflection at midspan are given also.

Table 7.7: Failure load and maximum deflection of lightweight concrete slabs

Slab	Ultimate load (KN)	Maximum deflection (mm)
Slab LWC-1	46.5	95.9
Slab LWC-2	48	88
Slab LWC-3	41.53	114.7
Slab LWC-4	49.52	86.65
Average	46.4	96.3

Figure (7.26) compares the load-deflection diagrams in the lightweight concrete slabs. Variation of the deflection at mid-span by increasing the applied load is similar in all slabs. Slabs LWC-2 and LWC-4 are compatible in the recorded data, while the recorded data of ultimate strength and the maximum deflection in LWC-3 is different compared to the other slabs. The load-deflection diagram in slab LWC-1 is approximately the average of all specimens.

According to Figure (7.26), the elastic load-deflection curve is seen up to 15% of the ultimate load capacity point. The yielding point of steel bar is about 50% of the ultimate load-capacity of the slab section in almost all slabs.

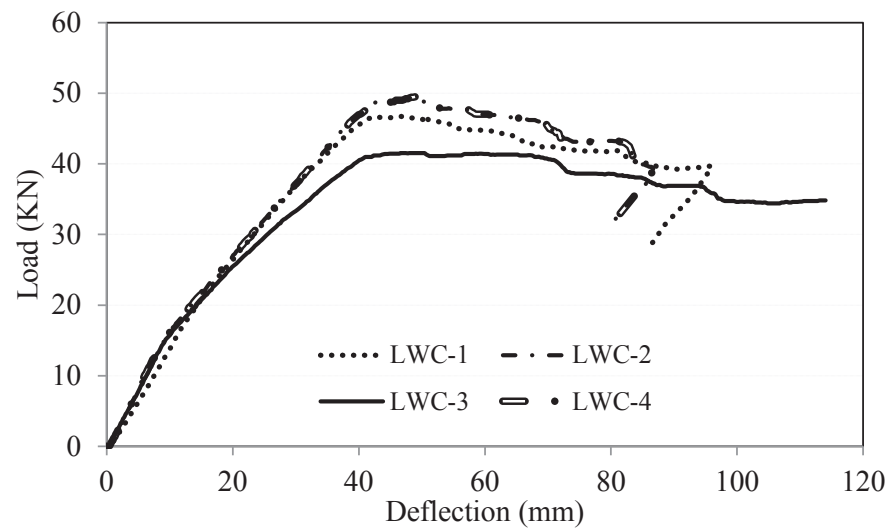


Figure 10.26: Comparing the load-deflection curves in four LWC slabs

Comparing the results with the previous experiments

Figure (10.27) compares the average load-deflection diagram of the light-weight concrete slab of this study with the previously performed tests on 8 self-compacting concrete slabs by Aslani (2014) and 2 conventional nonconcrete slabs by Nejadi (2005). All slabs with three types of concrete were similar in terms of the geometry and support conditions.

Table (7.8) shows a summary of the load-deflection result of all slabs in three different experiments under short-term loading.

Table 7.8: Summary of the results from short-term flexural test

	Beams	Failure load (KN)	Deflection at failure load (mm)		Slabs	Failure load (KN)	Deflection at Failure load (mm)
Self-compacting concrete slabs	NSCC-A	49	180	Lightweight concrete slabs	LWC-1	46.5	95.9
	NSCC-B	48.5	163		LWC-2	48	88
	D-SCC-A	53	205		LWC-3	41.53	114.7
	D-SCC-B	52	177		LWC-4	49.52	86.65
	S-SCC-A	50	185	Conventional concrete slabs			
	S-SCC-B	48	167		S3-a	50	136
	DS-SCC-A	56	220		S3-b	47	156
	DS-SCC-B	54	182				

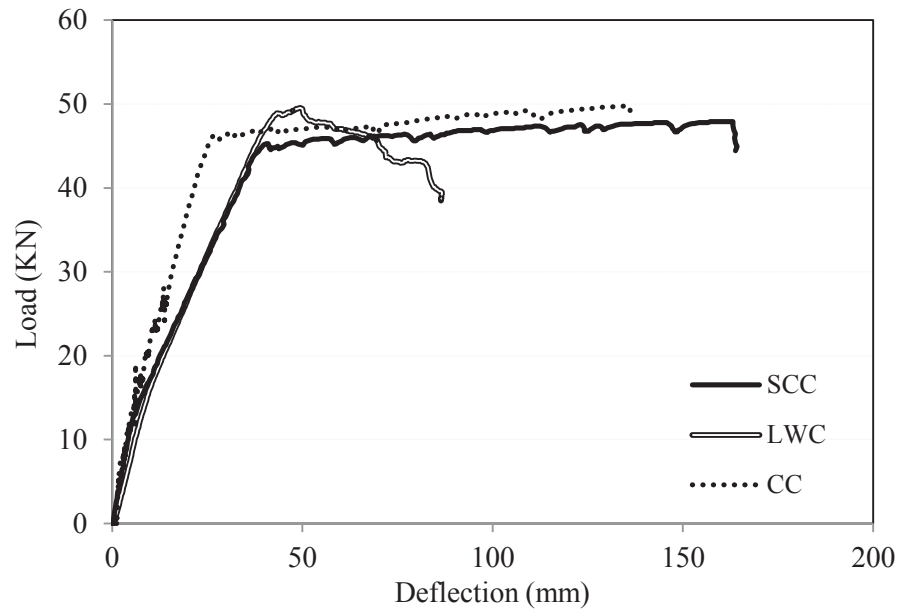


Figure 7.27: Comparison of the load-deflection curves in EPS-LWC, SCC and CC slabs

The failure load in all slabs is in the same range; however, the failure trend is completely different in LWC slabs. The failure in the CC and SCC slabs are extensively ductile, while, LWC slabs fail with less ductility, compared to the ductile failure of CC and SCC slabs. The elastic range of the load-deflection curve in all slabs is very close. In other words, they experience a similar trend of load-deflection up in the elastic range.

Considering the ratio of tensile reinforcement in the section, all slabs experienced ductile failure under the increasing flexural loading. In other words, the ultimate bending capacity of the slab section was more sensitive to the yield strength of the steel bars rather than the compressive strength of the concrete. According to Table (7.8), there is no considerable difference in the failure loads of SCC, CC and EPS-LWC slabs and all failure loads are in the range of 50 KN. While

Despite similar range of the failure load in all slabs, there is a substantial difference in the maximum deflection of slabs at mid-span at failure load.

Fibre reinforcing of the SCC mixture improves the ductility of the slabs. The ultimate flexural deflection of these slabs is considerably higher than the other slabs.

Compared to SCC and CC slabs, the lower modulus of elasticity in EPS-LWC slabs resulted in smaller amount of the maximum deflection at failure load. The relatively brittle failure of the EPS-LWC slabs may be considered as the reason of the difference in the recorded ultimate deflection. It emphasises the simultaneous effect of failure load and maximum deflection in design of one-way slabs.

REFERENCES

1. Nejadi, S., *Time-dependent cracking and crack control in reinforced concrete structures*. 2005, PhD thesis, The University of New South Wales Sydney, Australia.
2. Aslani, F., *Experimental and numerical study of time-dependent behaviour of reinforced self-compacting concrete slabs*. PhD dissertation, University of Technology Sydney (UTS), 2014.

CHAPTER 8

EXPERIMENTAL PROGRAM (PHASE II) – FLEXURAL INSTANTANEOUS DEFLECTION

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Chapter 8: Flexural instantaneous deflection

8.1. INTRODUCTION

Performance of reinforced concrete structures under sustained loads particularly in the post-cracking range is a key design concern. If sections are designed according to strength requirements only, the behaviour of the structure under sustained service loads may be unsatisfactory. For example, at service loads, deflections of the member may be excessively large or the width of the cracks may be improper; nevertheless the degree of the safety against collapse is acceptable.

For a concrete structure to be serviceable; cracking must be controlled and deflection must not be excessive. Service load behaviour depends primarily on the properties of the concrete and these are often not known reliably at the design stage, so it is necessary to check the serviceability of the structure in the post-cracking range. The behaviour in this range is complicated by the effects of several factors, which are difficult to assess from analytical considerations only. Of prime importance are the effects of tension stiffening, random development of primary and secondary cracks, and the degree of bond breakdown [1].

According to the existing literature, the increase in crack width occurs at a decreasing rate with time due to long-term or cyclic loading and this increase can be up to twice the initial value within a few years.

Instantaneous deflection (Δ_{ins}) is the fundamental parameter of the time-dependent and total deflection calculation of the Reinforced Concrete (RC) structures. The existing models predict the Δ_{ins} for Conventional Concrete (CC) beams considering the concrete properties at the age of

28 days [2] and do not cover other types of concrete and loading ages that are inevitable in most construction projects.

In recent decades, utilizing the chemical and mineral admixtures in concrete technology has introduced Light-Weight Concrete (LWC) as a reliable and economic construction material. Using LWC may result in the lighter elements and consequently, the smaller dimension that both decrease the total weight of the structure and the lateral load effects [3, 4]. Therefore, the construction cost can be protected when applied to structures such as long span bridge and high-rise buildings [5, 6]. Also, the better steel-concrete bond, better thermal insulation, durability performance, tensile strain capacity, and fatigue resistance make it superior to normal weight concrete [4, 5].

Generally, two types of natural (pumice, diatomite, volcanic cinders, etc.) and artificial (perlite, clay, sintered fly ash, expanded shale, etc.) lightweight aggregates are used to supply LWC [7]. Access to lightweight aggregates such as expanded clay, shale, and slate is limited in some locations and Expanded Polystyrene (EPS) is commercially available worldwide [8]. EPS is a type of artificial lightweight aggregate with a very low density. It is a thermoplastic foam involving a gas phase in a polymer matrix and has high compressibility, therefore, it may provide little restraint to volume changes of the cement paste due to the applied load as well as the changes in the moisture content [9].

Deflection of flexural elements with low reinforcement ratios is highly sensitive to the age of concrete at loading and the shrinkage-restraint stress [10, 11]. Modulus of Elasticity (MoE), loading magnitude and load distribution and support conditions are the principle factors in Δ_{ins} of one-way slabs. Also, continuity of the sections along the span; i.e. variable cross-sections and hence variable moment of inertia depending on the cracking pattern, strongly influence Δ_{ins} .

Considering the effect of mechanical properties on deflection behaviour of the RC structures, application of the existing models for LWC should be handled with care. Partial replacement of normal-weight fine or coarse aggregate with EPS aggregate decreases the Compressive Strength (CS), MoE and density of LWC depending on the level of replacement [9]. However, the CS in this type of concrete is more sensitive to density changes than the variation of MoE [7-9].

Mechanical properties of concrete change from an approximately liquid state to a viscoplastic material within a few hours, which is followed by further development into a hardened material with almost elastic properties up to about 40% of the compressive strength value. The cracking moment that has a considerable effect on time-dependent deflection can be reduced by shrinkage caused during the curing period before 28 days. The loading of young concretes structures require using the distinct properties at the age of loading; in other words, either the model or the early age properties should be modified to get an accurate prediction. In LWC

containing EPS aggregate, the CS and MoE at seven days is about 75% to 83% and 85% of those for 28 days, respectively [7-9, 12].

Flexural stiffness in the instantaneous deflection investigation changes with the cracking due to shrinkage strains of concrete, and the loading magnitude changes the predictable shrinkage values. The higher the EPS aggregate content in the mix, the greater the drying shrinkage and cracking development [9], [12].

The main objectives of this Chapter are:

- (a) To investigate the measured experimental values of instantaneous deflection of LWC slabs at the age of 14 days (Δ_{ins-14}). The LWC contains EPS beads as light-weight aggregate in the mix;
- (b) To compare the development of MoE and CS with time in LWC mix containing EPS aggregate;
- (c) To evaluate and compare the recorded deflections with the predictions of the existing models of effective moment of inertia (I_e) in codes of practice and empirical equations in the literature;
- (d) To investigate the effect of loading level, ratio of applied service moment (M_a) to cracking moment (M_{cr}) and ultimate bending capacity of section (M_u) on Δ_{ins-14} ;
- (e) To determine the best matching models of I_e to propose a new model for LWC.

8.2. EXPERIMENTAL PROGRAM

8.2.1. Construction of the specimens and curing method

Despite the detailed and comprehensive explanation of the testing program, construction of the slabs and companion specimens, curing and loading conditions in Chapter Seven, a brief summary of the testing program to monitor and record the instantaneous deflection of LWC slabs is given in this chapter.

The experimental program consists of four LWC one-way slabs to record the deflection and companion test specimens to measure the mechanical properties. The slabs were simply supported with identical span length and section dimensions (3.5m length $\times$ 0.4m width $\times$ 0.161m thickness). Each slab specimen was moist cured for 14 days and then subjected to loading. Six 150mm $\times$ 300mm cylindrical specimens to measure CS and three 150mm $\times$ 300mm cylindrical specimens were used to measure MoE at the different ages. The specimens for CS and MoE tests were kept covered in a controlled chamber at $20 \pm 2^\circ\text{C}$ for 24 hours until

demoulding. The relative humidity of the environment was maintained at 50% during the tests. Then, the specimens were placed in water pre-saturated with lime at 20°C. The compressive strength test was performed according to AS 1012.14 (1991) [13] and ASTM C39 (2000) [14] instructions, and the loading rate of the machine on the cylinders was 0.3 MPa/s until failure. MoE tests confirmed the instructions in AS 1012.17 (2014) [15] and ASTM C469 (2000) [16].

8.2.2. Load configuration and arrangement

In total, four LWC one-way slabs with idem mix design in the concrete laboratory of University of Technology Sydney (UTS) were investigated. All slabs were internally reinforced with 4 N12 deformed steel bars. As shown in Figure (8.1), the slabs are similarly simply supported having length 3500 mm, depth 161 mm and width 400 mm. The side and bottom cover of concrete from the bar centroid are 40 and 25 mm respectively.

Each pair of slab specimens were subjected to different gravity loads, consisting of self-weight plus superimposed sustained loads via carefully constructed and arranged concrete blocks supported on the top surface of the specimens. The uniformly distributed loads are about 30 and 40 percent of the ultimate bending capacity of section in each pair of slabs constructed with the same mix design of LWC.

To provide the required loading, rectangular concrete blocks of predetermined size and weights were cast and weighed before the commencement of the test. Slab specimens were uniformly loaded by the concrete blocks using wooden timbers as loading pads. The wooden pads, having length 450 mm, depth 30 mm and width 100 mm were placed in equal intervals to provide uniformly distributed loading on the slabs. Figure (8.2) shows the arrangement of concrete blocks on the top surface of each slab to achieve the desired sustained load level and the position of LVDTs under the slabs.

8.2.3. Monitoring points and measurement

The instantaneous deflection of slabs is recorded in two different points along the span at the age of 14 days. As shown in Figure (8.2), the deflection at mid-span as the maximum flexural deflection was recorded by Linear Variable Differential Transformer (LVDT). Another LVDT installed on the border of the high-moment region of the slab recorded the instantaneous deflection of the slab at the boundary of critical bending moment area along the span length. In this chapter, Δ_1 and Δ_2 denote the flexural deflection of slab at mid-span and the border of high-moment region respectively. Comparing the recorded deflection at these two points provides an

overview of the changes in the curvature of slab under the uniformly distributed flexural deflection. It can also give an insight into the variation of section stiffness along the span.

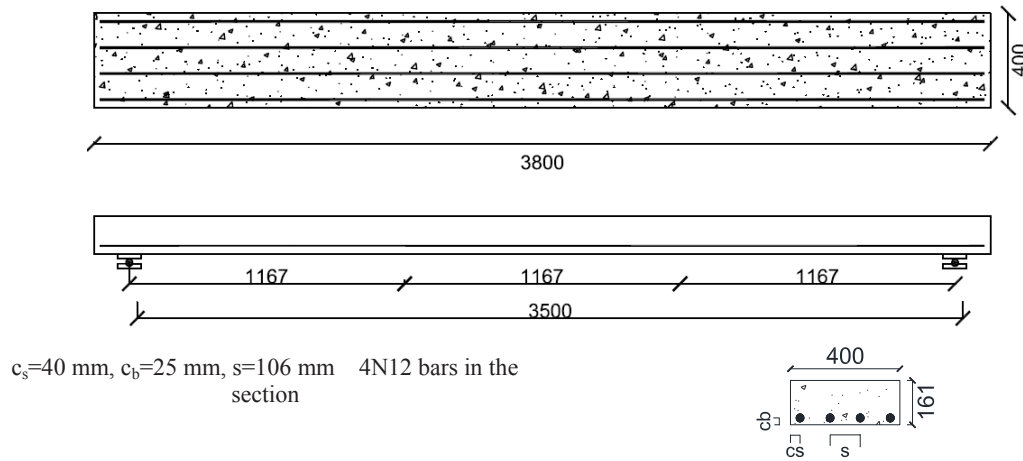


Figure 8.1: Bar arrangement, cover details and dimension of slab specimens



Figure 8.2: Loading blocks and LVDT positioning for flexural test of LWC slabs

Figure (8.3) shows the monitoring points along the span length of the slab.

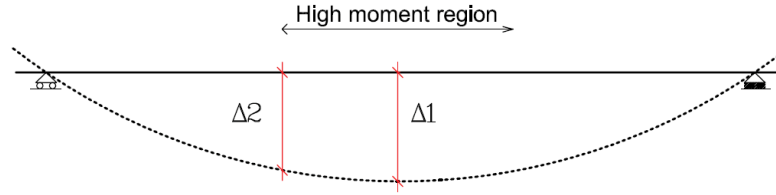


Figure 8.3 : Monitoring points of flexural instantaneous deflection in LWC slabs

8.2.4. Ultimate capacity and the applied load

The instantaneous deflection was recorded by LVDT installed at mid-span (Δ_1), immediately after loading of the slabs at the age of 14 days. The ultimate bending capacity (M_u) of a reinforced concrete section is calculated based on the mechanical properties of concrete at the age of 28 days. However, the slabs in this study are subjected to the service load for instantaneous and long-term deflection investigations at the age of 14 days. Mechanical properties of concrete such as CS and MoE are developing by increasing the age of concrete; especially up to age of 28 days. Therefore, calculation of the ultimate bending capacity, M_u at loading age (14 days) by properties of the concrete at the age of 28 days will bring overestimated capacity and consequently, apply some design errors in deflection calculations.

The existing models give under or overestimated values of the mechanical properties of concrete. Besides, the models of predicting the properties of LWC at the ages other than 28 days are very rare in the literature. Therefore, exact values of CS and MoE of the LWC mix have been measured by test specimens in laboratory conditions at two ages of 14 and 28 days. Figure (8.4) compares the measured values and development of CS and MoE with time. There are about 11% and 9% increment of CS and MoE of LWC with time, respectively.

The graph in Figure (8.4) shows the development of CS and MoE between 14 and 28 days age of concrete only. These graphs are part of nonlinear development trend comprehensively explained in Chapter Five of this study.

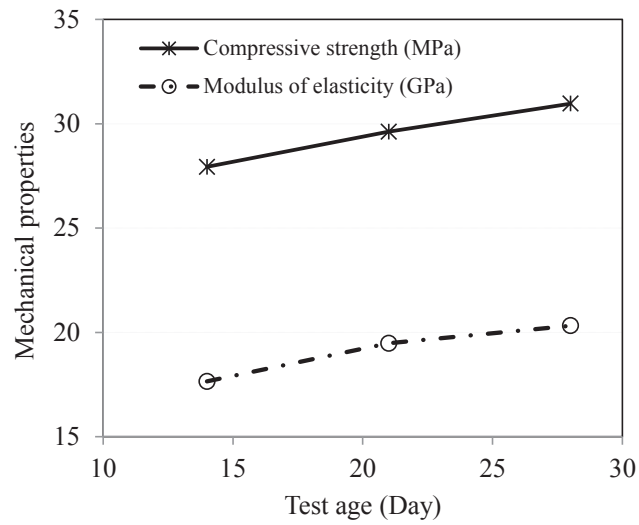


Figure 8.4: Development of compressive strength and modulus of elasticity with time

Table (8.1) presents the applied service moment (M_a) of slabs at mid-span due to two different load conditions of service loads (w_a). Also, to consider the exact capacity of slab section at loading age, the M_u of slab section is presented by applying the mechanical properties of concrete at two ages of 14 and 28 days. The applied load in each pair of slabs is about 30% and 40% of the ultimate bending capacity of slabs. In other words, two levels of uniformly distributed loads are applied to the slabs. The loading levels equal to 30% and 40% of the M_u is presented by LL_1 and LL_2 in this Chapter, respectively.

Table 8.1: Loading values, moment ratios and elastic deflection of slabs

Slab	w_a kN/m	M_a kN.m	f'_{c-28} (MPa)	M_u (kN.m)		M_a/M_u (%)	
				14 days	28 days	14 days	28 days
LWC-A	7.0	10.72	30.96	27.4	27.65	39.13	38.78
LWC-B	7.0	10.72	30.96	27.4	27.65	39.13	38.78
LWC-C	5.29	8.095	30.96	27.4	27.65	29.55	29.29
LWC-D	5.29	8.095	30.96	27.4	27.65	29.55	29.28

8.3. EXPERIMENTAL INSTANTANEOUS DEFLECTION

To attain the required loading levels, concrete blocks of predetermined size and weight were put on top of the slabs in equal distances. Details of these loading blocks are explained in Chapter 7 of this study. Figure (8.5) shows the placement order of the concrete blocks in all slabs.

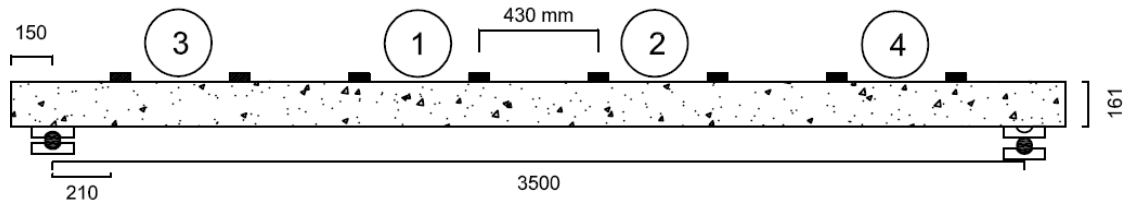


Figure 8.5 : Placement order of the loading blocks on slabs

Figures (8.6) to (8.9) show the instantaneous deflection of slabs under the applied loads. Since the loading was applied by four concrete blocks on the top surface of the concrete slabs, there is a sharp increment of deflection after putting each block on the slabs. After all loading blocks were applied and the changes in the deflection were not considerable, it was considered as the instantaneous deflection point.

Figure (8.6) shows the gradually increment of the deflection of Slab LWC-1 until reaching the establishment point that is considered as the instantaneous deflection at two monitoring points. According to Table (8.1), the applied loading level in this slab is about $0.4M_u$ (LL_2).

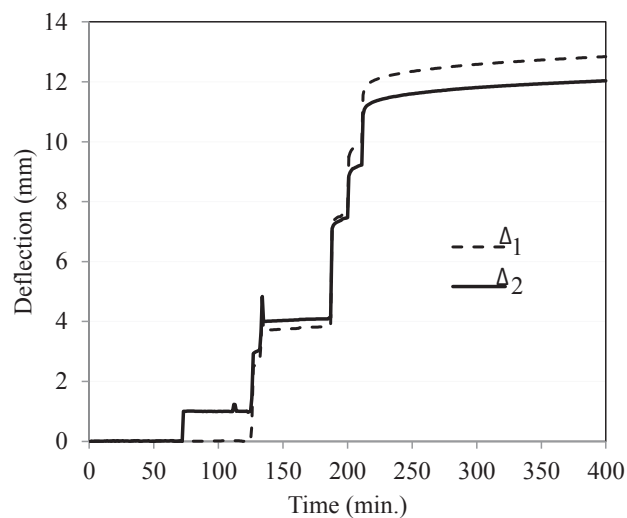


Figure 8.6: Gradually increment of deflection under each loading block and the instantaneous deflection of slab LWC-1 after applying the loading blocks

According to Figure (8.6), the loading block have similar effect on both monitoring points to suddenly increase the deflection before establishment point. After the last block placement, the instantaneous deflection at mid-span grows more than the other point.

Figure (8.7) shows the gradually increment of the deflection of Slab LWC-2 until reaching the establishment point that is considered as the instantaneous deflection at two monitoring points. According to Table (8.1), the applied loading level in this slab is nearly 0.4Mu (LL_2).

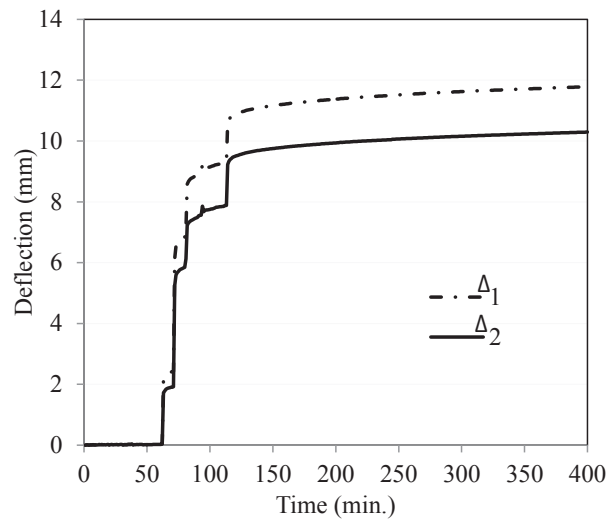


Figure 8.7: Gradually increment of deflection under each loading block and the instantaneous deflection of slab LWC-2 after applying the loading blocks

In Slab LWC-2, the effect of loading blocks on the sharp increment of the flexural deflection is changing considerably after putting the third block on top of the slab.

Figure (8.8) shows the gradually increment of the deflection of Slab LWC-3 until reaching the establishment point that is considered as the instantaneous deflection at two monitoring points. According to Table (8.1), the applied loading level in this slab is about 0.3Mu (LL_1).

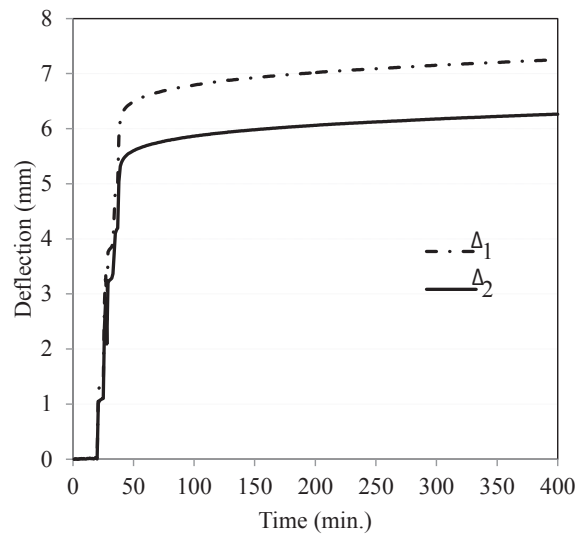


Figure 8.8: Gradually increment of deflection under each loading block and the instantaneous deflection of slab LWC-3 after applying the loading blocks

Despite different weight of the loading blocks in LL_1 in this slab, the increment of the deflection at two monitoring points is similar to that of slab LWC-2 under LL_2 .

Figure (8.9) shows the gradually increment of the deflection of Slab LWC-4 until reaching the establishment point that is considered as the instantaneous deflection at two monitoring points. According to Table (8.1), the applied loading level in this slab is about $0.3Mu$ (LL_1).

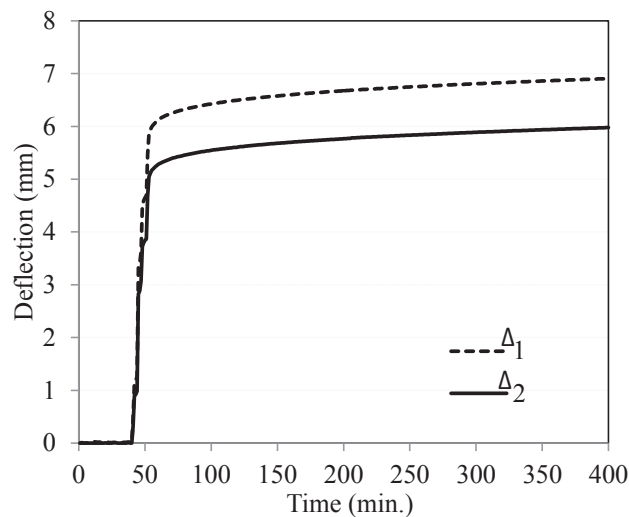


Figure 8.9: Gradually increment of deflection under each loading block and the instantaneous deflection of slab LWC-4 after applying the loading blocks

Considering the same loading level on the slabs LWC-3 and LWC-4, the initial deflection growth and the instantaneous deflection after establishment of changes is similar in these slabs.

8.4. DIFFERENCE BETWEEN THE ELASTIC AND INSTANTANEOUS DEFLECTION

Considering the instantaneous deflection (Δ_{ins}) as the elastic deflection (Δ_e) is a source of errors in the design of concrete structures, this study compares the experimental data with the calculated elastic deflection (Δ_e) of slabs by 14 and 28 days and MoE at two loading levels.

Figure (8.10) shows the ratio of measured instantaneous deflection to the predicted elastic deflection at the ages of 14 days ($\Delta_{ins-14} / \Delta_{e-14}$) and the ratio of recorded instantaneous to the elastic deflection at the age of 28 days ($\Delta_{ins-14} / \Delta_{e-28}$) in the LWC slabs. The Δ_e is calculated on the basis of the gross moment of inertia in the uncracked section at mid-span.

Equations (8.1) and (8.2) are utilized to determine the elastic deflection of slabs under two levels of loading values and M_{cr} respectively.

$$\Delta_e = \frac{5}{384} \times \frac{W_a \cdot l^4}{E_c \cdot I_g} \quad Eq. (8.1)$$

$$M_{cr} = I_{cr} \times f_r \quad Eq. (8.2)$$

Where; W_a , l , E_c , I_g , I_{cr} and f_r are uniformly distributed load (KN/m), span length (m), MoE of concrete (KN/m²), gross moment of inertia (m⁴), cracking moment of inertia (m⁴) and modulus of rupture (KN/m²), respectively.

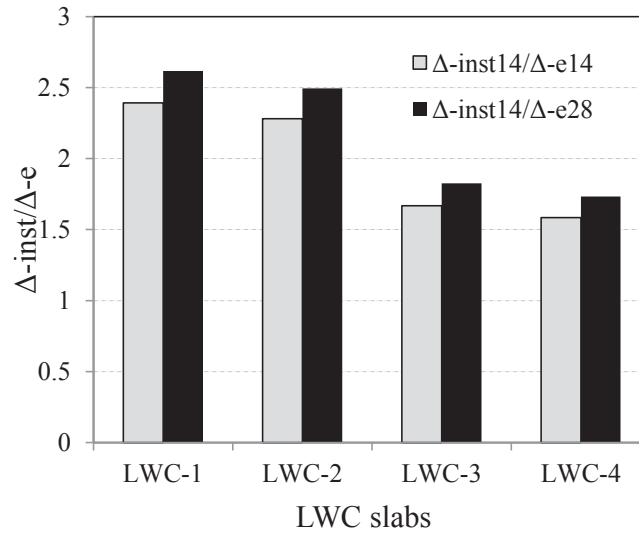


Figure 8.10: Ratio of the recorded deflection to the estimated elastic deflection at 14 and 28 days

The ratio of $(\Delta_{\text{ins}} / \Delta_e)_{14}$ in the mid-span of slabs varies between 1.58 and 2.39. Comparing the recorded deflection at 14 days ($\Delta_{\text{ins-14}}$) with the estimated elastic deflection at 28 days ($\Delta_{\text{e-28}}$), the ratio of $\Delta_{\text{ins-14}} / \Delta_{\text{e-28}}$ shows a discrepancy between 1.73 and 2.62. In fact, the increased flexural stiffness at the age of 28 days decreases the elastic deflection and, consequently, enhances the ratio of $\Delta_{\text{ins-14}} / \Delta_{\text{e-28}}$. The average ratio of $(\Delta_{\text{ins}} / \Delta_e)_{14}$ and $\Delta_{\text{ins-14}} / \Delta_{\text{e-28}}$ are 1.98 and 2.17 respectively.

Table (8.2) shows the measured instantaneous and calculated elastic deflection of LWC slabs at the two monitoring points of slab. The Δ_1 and Δ_2 are the measured instantaneous deflections respectively at two monitoring points, while Δ_{e1} and Δ_{e2} are the calculated elastic deflection of slabs at the corresponding points.

Table 8.2: Instantaneous and elastic deflection of LWC slabs at monitoring points

Slab	Loading level 0.3Mu		Loading level 0.4Mu		Δ_{e-1} (mm)		Δ_{e-2} (mm)	
	Δ_1 (mm)	Δ_2 (mm)	Δ_1 (mm)	Δ_2 (mm)	14days	28 day	14days	28days
LWC-1	11.84	11.19			4.99	4.57	4.37	4.11
LWC-2	10.76	9.22			4.99	4.57	4.37	4.11
LWC-3			6.33	5.43	3.77	3.45	3.22	2.85
LWC-4			5.98	5.16	3.77	3.45	3.22	2.85

Figure (8.11) shows a schematic diagram to compare the calculated elastic and measured instantaneous deflection of slabs under uniformly distributed loading.

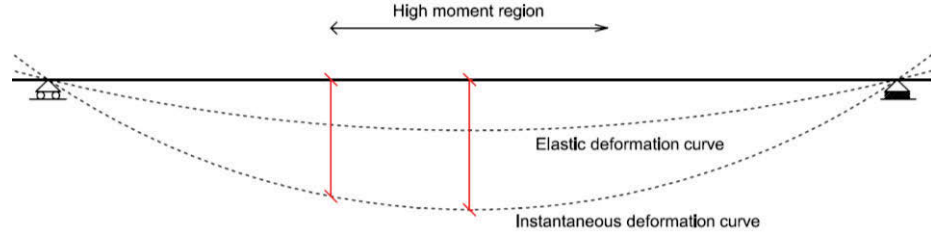


Figure 8.11: Schematic diagram of instantaneous and elastic deflection of LWC slabs in bending

8.5. EFFECT OF LOADING LEVEL

Since the slabs are identical concerning the dimensions, reinforcement and support conditions, the relative ratio of the applied load to the flexural capacity of section (M_a/M_u) could be a critical factor in Δ_{ins} of slabs that are generally lightly reinforced concrete structures. This ratio becomes more important for the loading of the slabs before 28 days age. Figure (8.12) compares the effect of M_a/M_u on the ratio of $(\Delta_{ins} / \Delta_e)_{14}$ in slabs. The applied service load is about 30 and 40 percent of the ultimate section capacity in each pair of slabs with the same mix design.

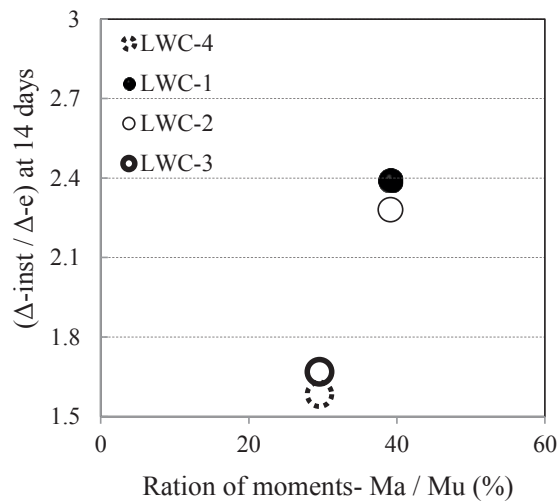


Figure 8.12: $(\Delta_{ins} / \Delta_e)_{14}$ under different ratios of service moment and ultimate moment

The increasing rate of Δ_{ins-14} under two levels of M_a/M_u ratio is different from the variation of $(\Delta_{ins}/\Delta_e)_{14}$ ratio. Unexpectedly, the effect of amplifying M_a/M_u ratio by 10% on the Δ_{ins-14} is considerably different in these identical slabs. Increasing the M_a/M_u ratio by 10% has resulted about 80% to 100% higher Δ_{ins-14} in the LWC slabs. In other words, the early-state deflection of LWC slabs (before the age of 28 days) is significantly sensitive to the loading levels and variation. Figure (8.13) compares the Δ_{ins-14} of slabs under two levels of the M_a/M_u ratio.

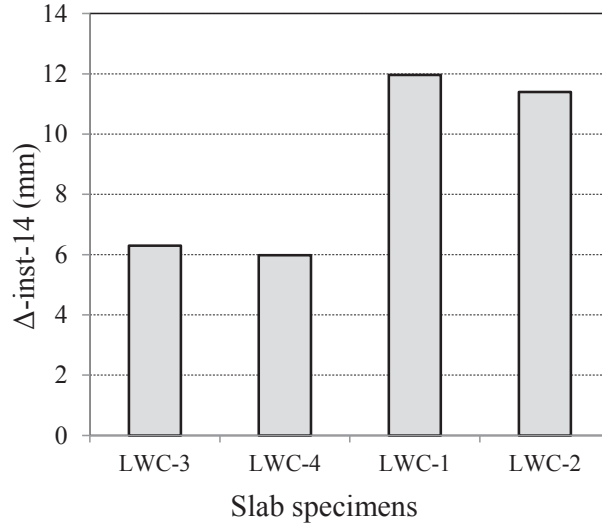
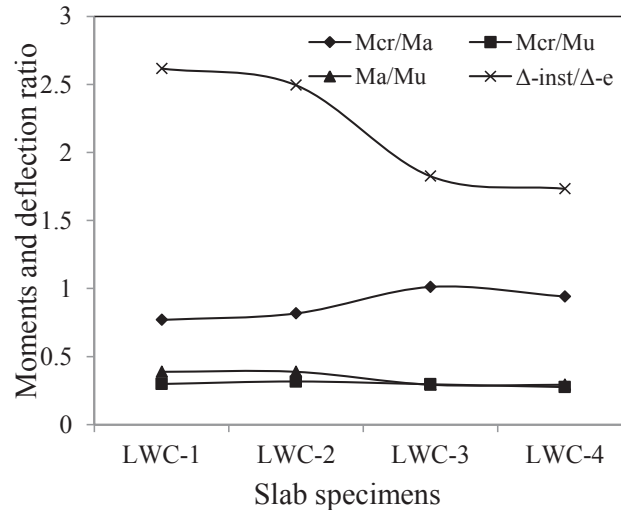


Figure 8.13: Growing rate of Δ_{ins-14} due to 10 % increment of M_a/M_u

Together with the effects of M_a and M_u , cracking moment (M_{cr}) can be another critical parameter in deflection calculations of slabs. Notwithstanding the effects of shrinkage, M_{cr} causes the first cracking in the tensile zone and changes the section behavior to non-elastic. Since M_{cr} and M_u are dependent characteristics of the reinforced concrete section, they may have similar influences on the Δ_{ins}/Δ_e ratio. Besides, the ratio of M_a/M_u is limited to 30 to 40% in each pair of slabs in this study. In fact, it looks like the application of the predefined relationship between M_a and M_u in this study. According to Figure (8.14) the $(\Delta_{ins}/\Delta_e)_{14}$ ratio is changing by variation of M_{cr}/M_u and M_a/M_u ratios; however, it is noticeably more sensitive to the ratio of M_{cr}/M_a compared to the M_a/M_u ratio. It is evident from Figure (8.12) that the M_a/M_u and M_{cr}/M_a ratios have increasing and decreasing effects on $(\Delta_{ins}/\Delta_e)_{14}$ respectively.

The value of the applied M_a is very close to M_{cr} in LWC-3 and LWC-4, in which the elastic behavior of the section is expected. However due to the shrinkage effects and initiation of cracking in the tension zone, Δ_{ins} is about 60% higher than the estimated elastic deflection.

Figure 8.14: Effect of moment ratios on $(\Delta_{ins} / \Delta_e)_{14}$

8.6. PREVIOUS EXPERIMENTS ON THE INSTANTANEOUS DEFLECTION OF SLABS

8.6.1. Conventional Concrete Slabs by Nejadi (2005)

Similar to the short-term program, a total of twelve specimens including six beams and six slabs, with the same cross section and details for short-term tests, were monitored for up to 400 days. One batch of commercially pre-mixed concrete was used to manufacture all the test and companion specimens. The steel stain within high moment regions, the concrete surface strains at the tensile steel level, deflection at the mid-span, crack widths and crack spacing, were recorded throughout the testing period. All beam and slabs were subjected to different gravity sustained loads at the age of 14 days. For both the beams and slab specimens, two identical specimens were constructed for each combination of variables, with one loaded to 40% of its ultimate capacity (type a) and one loaded to 30% of its ultimate capacity (type b). Figure (8.15) shows the general view of test arrangements for time-dependent deflection monitoring.



Figure 8.15: Loading of convention concrete slabs for time-dependent deflection monitoring (Nejadi, 2005)

The instantaneous deflection of conventional concrete slabs at mid-span was recorded immediately after the loading blocks were applied on the top surface of slabs. Table (8.3) shows the instantaneous deflection of conventional concrete slabs under two different levels of service loading.

Table 8.3: Experimental instantaneous deflection of CC slabs (Nejadi, 2005)

Slab	Instantaneous deflection	
	Load level $0.26 M_u$	Load level $0.2 M_u$
CC-a	11.8	
CC-b		5.04

8.6.2. Self-Compacting Concrete Slabs by Aslani (2014)

A total of eight numbers of Self-Compacting Concrete (SCC) and Fibre Reinforced Self-Compacting Concrete (FRSCC) slabs with the same cross-section and details as for the short-term tests were monitored for up to 240 days to measure the time-dependent development of cracking and deformations under service loads. Deflections at mid-span, crack widths, crack patterns, steel strains within the high moment region, and concrete surface strains at the steel level were recorded under the service load and shrinkage effects. For this purpose, four SCC mixes, two plain SCC, two steel, two polypropylene, and two hybrid FRSCC slab specimens were considered in the test program. The SCC mixture without fibre, with steel fibre and polypropylene fibre are called N-SCC, D-SCC and S-SCC respectively. The term DS-SCC refers to the SCC mixture with both the steel and polypropylene fibres.

Two sustained load levels were considered, namely 40% and 30% of the ultimate design load, and designated load conditions ‘a’ and ‘b’, respectively. The applied load consisting of self-weight plus superimposed sustained loads at the age of 14 days via carefully constructed and arranged concrete blocks supported off the top (of the specimens). Figure (8.16) shows the general view of test arrangement for time-dependent deflection monitoring.



Figure 8.16: General view of the test specimens of SCC and FRSCC slabs (Aslani, 2013)

The instantaneous deflection of SCC and FRSCC slabs at mid-span was recorded immediately after the loading blocks were applied on the top surface of the slabs. Table (8.4) shows the instantaneous deflection of SCC and FRSCC slabs under two different levels of service loading.

Table 8.4: Experimental instantaneous deflection of SCC and FRSCC slabs (Aslani, 2014)

Slab	Instantaneous deflection	
	Load $0.4 M_u$	Load $0.3 M_u$
N-SCC-a	12.1	
N-SCC-b		5.89
D-SCC-a	7.65	
D-SCC-b		7.59
S-SCC-a	6.41	
S-SCC-b		2.91
DS-SCC-a	8.98	
DS-SCC-b		5.14

8.7. COMPARISON OF LOAD-DEFLECTION DIAGRAMS IN LIGHTWEIGHT, SELF-COMPACTING AND CONVENTIONAL CONCRETE SLABS

Table (8.5) shows the values of the experimental instantaneous deflection and the calculated elastic deflection of LWC, SCC and CC slabs under two levels of sustained loading. The values of M_u at 14 and 28 days, M_a , and the ratio of M_a/M_u are also given in the table.

Table 8.5: Loading values, recorded and elastic deflection

Slab	w_a (KN/m)	M_u (KN.m)		M_a (KN.m)	M_a / M_u (%)		Δ_e (mm)		Δ_{ins} (mm)
		14days	28days		14 day	28 day	14days	28days	
N-SCC-a	7.31	27.47	27.78	11.189	40.73	40.27	3.18	2.9	12.1
N-SCC-b	5.41	27.47	27.78	8.28	30.14	29.8	2.36	2.15	5.89
D-SCC-a	7.26	27.84	28.05	11.12	39.94	39.65	3.5	2.85	7.65
D-SCC-b	5.36	27.84	28.05	8.21	29.48	29.27	2.58	2.11	7.59
S-SCC-a	7.3	27.73	28.05	11.18	40.32	39.85	3.45	2.87	6.41
S-SCC-b	5.4	27.73	28.05	8.27	29.83	29.48	2.55	2.12	2.91
DS-SCC-a	7.33	28.05	28.34	11.23	40.03	39.63	3.29	2.85	8.98
DS-SCC-b	5.43	28.05	28.34	8.32	29.66	29.36	2.44	2.11	5.14
LWC-1	7	27.4	27.65	10.72	39.13	38.78	4.99	4.57	11.96
LWC-2	7	27.4	27.65	10.72	39.13	38.78	4.99	4.57	11.4
LWC-3	5.29	27.4	27.65	8.095	29.55	29.28	3.77	3.45	6.3
LWC-4	5.29	27.4	27.65	8.095	29.55	29.28	3.77	3.45	5.98
CC-a	4.44	26.03	27.05	6.8	26.12	25.14	2.73	2.5	11.8
CC-b	3.46	26.03	27.05	5.3	20.36	19.59	2.13	1.95	5.04

Figures (8.17) and (8.18) compare the measured values and increasing rate of the Compressive Strength (CS) and Modulus of Elasticity (MoE) of the mixtures in LWC, SCC and CC slabs at the ages of 14 and 28 days.

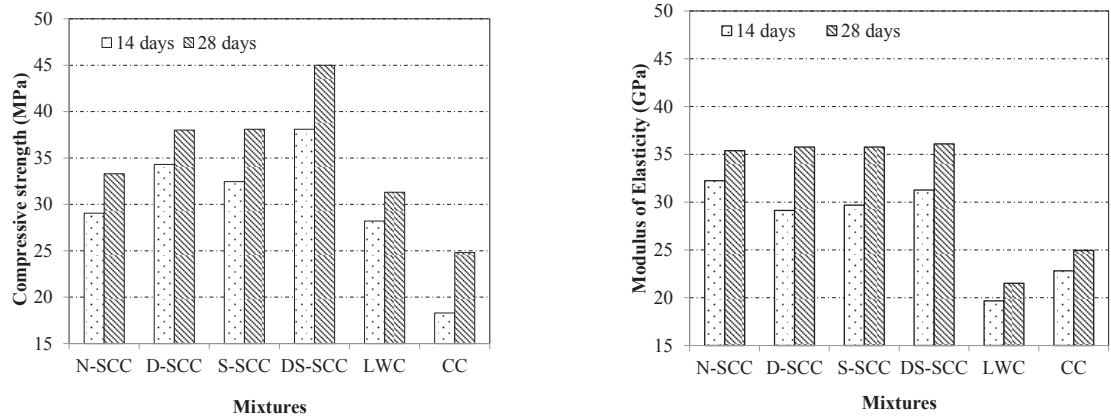


Figure 8.17: Compressive strength and modulus of elasticity of mixtures at 14 and 28 days

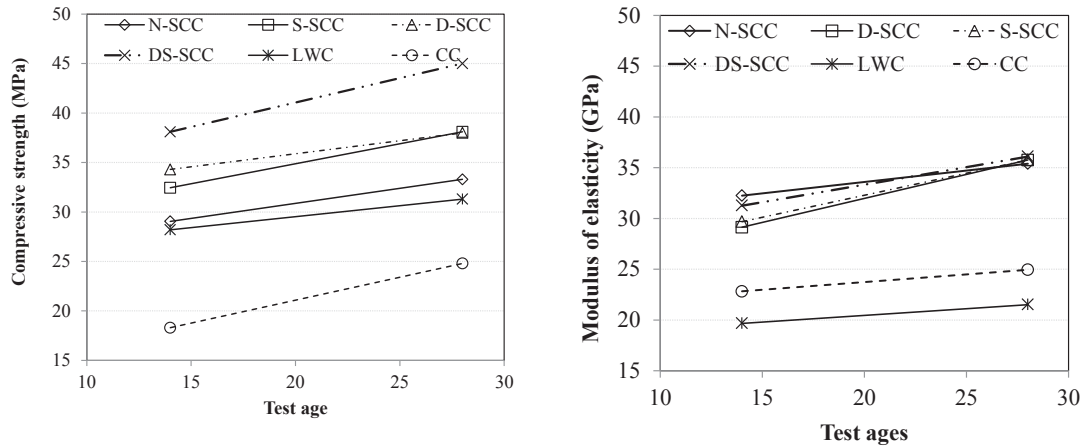


Figure 8.18: Increasing rate of compressive strength and modulus of elasticity of mixtures

8.7.1. Instantaneous and Elastic Deflection

According to Figures (8.17) and (8.18), the average increasing ratio of MoE in CC, SCC and LWC is 1.10, 1.17 and 1.10 respectively. The increasing ratio of compressive strength in NC, SCC and LWC is 1.35, 1.15 and 1.10 respectively. Overall the early age properties and increasing rate in SCC mixtures particularly in presence of fibres are better than the corresponding properties of CC and LWC.

Figure (8.19) shows the ratio of the experimental instantaneous deflection (Δ_{ins}) to the elastic deflection (Δ_e) at the ages of 14 and 28 days in concrete slabs with different mixture designs. The ratio of $(\Delta_{ins}/\Delta_e)_{14}$ in CC, LWC and SCC slabs vary between 1.15 in S-SCC-b to 4.3 in NC-a. However the majority of ratios are in the range of 1.5-2.5. Increased elasticity modulus at the age of 28 days decreases the elastic deflection at the age of 28 days and hence increases the ratio of $\Delta_{ins-14}/\Delta_{e-28}$.

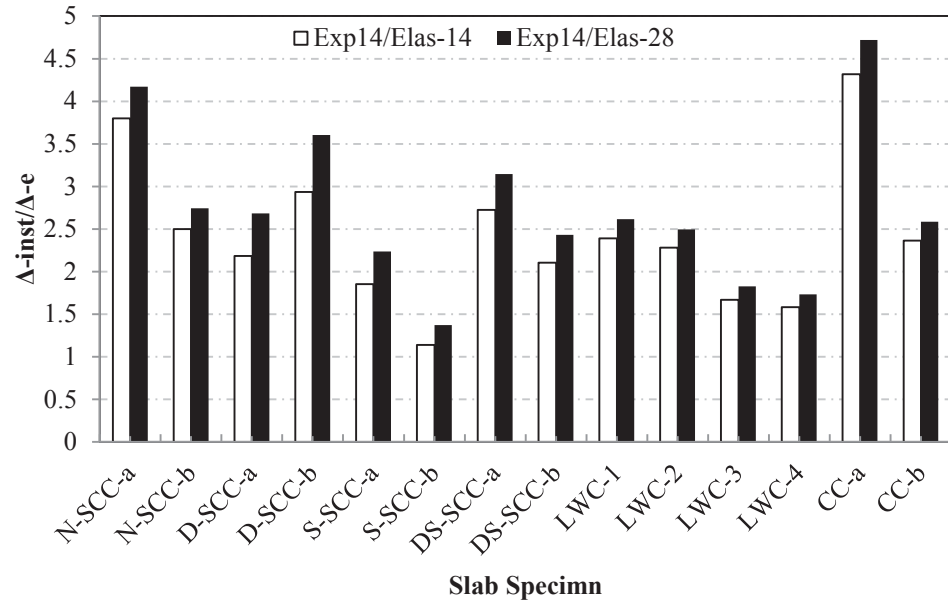


Figure 8.19: Ratio of recorded instantaneous deflection to elastic deflection at the ages of 14 and 28 days

8.7.2. Effect of Loading Level on the Flexural Deflection

All slab specimens are identical with the same reinforcement arrangement. Among all effective parameters in instantaneous deflection of lightly reinforced concrete structures, the loading rate and M_d/M_u ratio play major role. Figure (8.20) compares the effect of M_d/M_u on the ratio of $(\Delta_{ins}/\Delta_e)_{14}$ in the slabs. The CC slabs are under 20% and 26% of their ultimate capacity, while SCC and LWC slabs are under about 30% and 40% of the ultimate bending capacity loading. Except D-SCC (a, b) slabs, a 10% load increase in the slabs, causes about 50 % to 70% growth in the $(\Delta_{ins}/\Delta_e)_{14}$ ratio. Considering the uncertainties and unexpected loads during the construction, this ratio of $(\Delta_{ins}/\Delta_e)_{14}$ becomes more crucial in the accurate design of concrete slabs. The low strength CC slabs are more sensitive to the loading values, while a 6% load increment causes about an 80% rise in the $(\Delta_{ins}/\Delta_e)_{14}$ ratio.

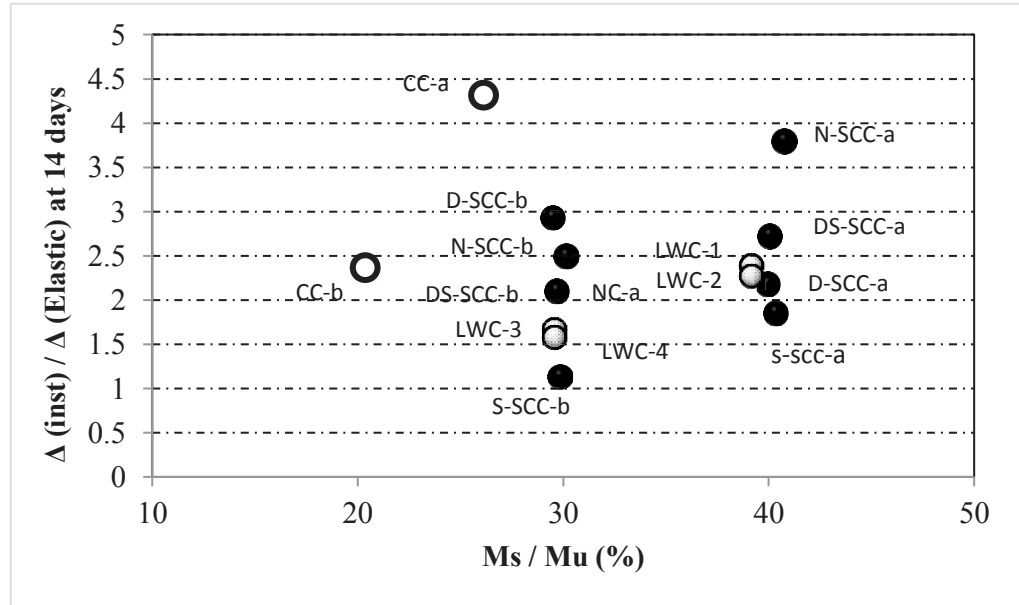


Figure 8.20: Instantaneous / elastic deflection ratio at 14 days under different M_a / M_u ratios

Along with the magnitude of the service moment, the effect of cracking moment (M_{cr}) that causes the first cracking is considerable in deflection calculations of slabs. Since M_{cr} and M_u are dependent characteristics of section, consequently they may change the Δ_{ins} / Δ_e ratio in a similar manner. The ratio of M_a / M_u is limited to 20% to 26% in CC slabs and 30% to 40% in LWC and SCC slabs; in other words M_a is related to M_u in this experiment. According to Figure (8.21), the ratio of $(\Delta_{ins} / \Delta_e)_{14}$ changes very similarly in various ratios of M_{cr} / M_u and M_a / M_u , however the ratio of M_{cr} / M_u is more effective in instantaneous deflection calculations especially in low strength concrete specimens.

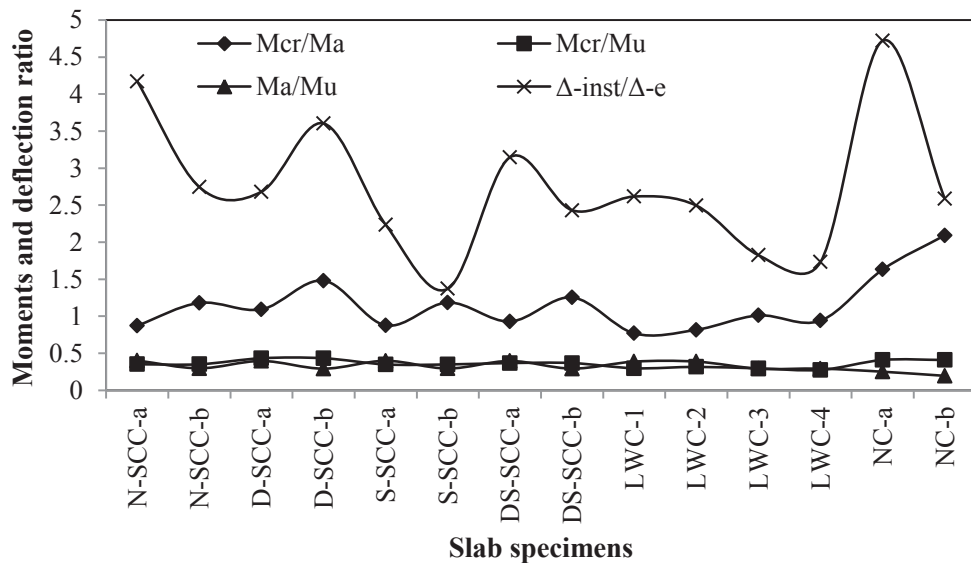


Figure 8.21: Effect of moment ratios on Δ_{ins} / Δ_e at 14 days

In all slabs the applied M_a is smaller than M_{cr} and the section is supposed to be in elastic range, however due to the shrinkage-induced cracking, the measured instantaneous deflection is higher than calculated elastic deflection. As presented in Figure (8.22), the instantaneous deflection in one way slabs slightly increases by decreasing the ratio of M_{cr}/M_a , while it increases by increasing the ratios of M_{cr}/M_u and M_a/M_u in general.

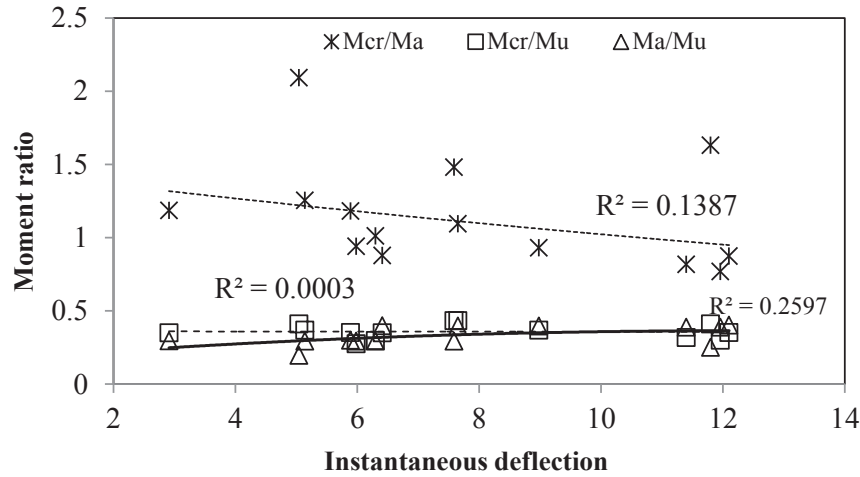


Figure 8.22: Effect of moment ratios on instantaneous deflection of slabs at 14 days

8.8. PROPOSED ANALYTICAL MODEL

Existing models of the effective moment of inertia (I_e) have been utilized to predict the Δ_{ins-14} of LWC, SCC and CC slabs. Among the existing models, some equations are developed for particular conditions like point loading and the concrete members with FRP bars and near surface mounted FRP strips; however, their predictions have been compared with Δ_{ins-14} values in this study to observe any compatibility in between.

Estimated values of I_e from Equations (8.1) to (8.20) (see Table (8.6)) have been utilized to predict the Δ_{ins-14} of slabs. The recorded Δ_{ins-14} in slabs have been compared and evaluated with these predicted deflections in Figure (8.24). Surprisingly, there are considerable differences between the predicted and experimental data in two levels of the applied M_a/M_u ratio. Existing models give a better prediction at the higher loading level ($M_a/M_u = 0.4$), and overestimate the Δ_{ins-14} by about 100% in the lower loading level ($M_a/M_u = 0.3$).

The effective moment of inertia in the measured instantaneous deflection of slabs is calculated and compared with the existing models. Considering the identical conditions and known parameters such as support condition, loading magnitude and mechanical properties of the section in each slab, the actual value of I_e relevant to the recorded deflection has been compared with the I_e determined by Equations (8.3) to (8.21) in Figures (8.23) and (8.24).

Table 8.6: Existing models of effective moment of inertia in RC section

Reference	Equation	Equation
Bronson (1963)	$I_e = (M_{cr}/M_a)^2 I_g + (1 - (M_{cr}/M_a)^2) I_{cr} \leq I_g$	Eq. (8.3)
Bronson (1965) [17]	$I_e = (M_{cr}/M_a)^3 I_g + (1 - (M_{cr}/M_a)^3) I_{cr} \leq I_g$	Eq. (8.4)
	$I_e = (M_{cr}/M_a)^4 I_g + (1 - (M_{cr}/M_a)^4) I_{cr} \leq I_g$	Eq. (8.5)
Al Shaik and Al Zaid (1993) [18]	$I_e = I_{cr} + (I_g - I_{cr}) (M_{cr}/M_a)^{3-0.8\rho} \leq I_g$	Eq. (8.6)
ACI-318-08 [19]		
ACI-435-68 [20]	$I_e = (M_{cr}/M_a)^3 I_g + (1 - (M_{cr}/M_a)^3) I_{cr} \leq I_g$	Eq. (8.7)
AS-3600-09 [21]		
ACI 440 (2006) [22]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$ $\beta_d = \alpha_b \left(\frac{E_f}{E_s} + 1\right), \quad \beta_d = 0.2 \frac{\rho}{\rho_b} < 1$	Eq. (8.8)
Bischoff and Paixao (2004) [23]	$I_{ef} = \frac{I_{cr}}{1 - \eta (M_{cr}/M_a)^2} \leq I_g$	Eq. (8.9)
Eurocode 2, (1994) [24]	$\eta = 1 - I_{cr}/I_g$	
Bischoff (2005, 2007) [25]	$I_e = \frac{I_{cr}}{1 - \eta \beta (M_{cr}/M_a)} \leq I_g$	Eq. (8.10)
Bischoff and Scanlon (2007) [25]	no tension stiffening $0 \leq \beta \leq 1$ full tension stiffening	
Bischoff and Gross (2011) [26]	$I'_e = \frac{I_{cr}}{1 - \eta \gamma \beta (M_{cr}/M_a)} \leq I_g$ $\gamma = 1.72 - 0.72 M_{cr}/M_a, \quad \beta = M_{cr}/M_a$	Eq. (8.11)
Faza and GangaRao (1992) [27]	$I_m = \frac{23 I_{cr} I_e}{8 I_{cr} + 15 I_e}$	Eq. (8.12)
Benmokrane et al. (1996) [28]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \frac{I_g}{7} + 0.84 \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$	Eq. (8.13)
Yost et al. (2003) [29] and ACI-440 (2006) [22]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$ $\beta_d = 0.2 \frac{\rho}{\rho_b} < 1$	Eq. (8.13)
Rafi and Nadjai (2009) [30]	$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \frac{I_{cr}}{\gamma} \leq I_g$ $\gamma = \left(\frac{0.0017\rho}{\rho_b} + 0.8541\right) \left(\frac{E_f}{2E_s} + 1\right)$	Eq. (8.14)
Alsayed et al. (2000) [31]	$I_e = I_{cr} \quad \text{for } \frac{M_{cr}}{M_a} > 3$ $I_e = \left[1.4 - \frac{2}{15} \left(\frac{M_{cr}}{M_a}\right)\right] I_{cr} \leq I_g \quad \text{for } 1 < \frac{M_{cr}}{M_a} < 3$	Eq. (8.15) Eq. (8.16)
ISIS Canada (2001) [32]	$I_e = \frac{I_g I_{cr}}{I_{cr} + [1 - 0.5 \left(\frac{M_{cr}}{M_a}\right)^2] (I_g - I_{cr})} \leq I_g$	Eq. (8.17)
Hall and Ghali (2000) [33]	$I_e = \frac{I_g I_{cr}}{I_{cr} + [1 - \beta_1 \beta_2 \left(\frac{M_{cr}}{M_a}\right)^2] (I_{cr} - I_g)} \leq I_g$	Eq. (8.18)
ACI- 224.2R-92 [34]	$A_e = \left(\frac{P_{cr}}{P}\right)^3 A_g + (1 - (P_{cr}/P)^3) A_{cr} \leq I_g$ Ag: Gross cross section area, Acr: nAs	Eq. (8.19)
Fikry and Thomas (1998) [35]	$I_e = I_{cr} + (I_g - I_{cr})^{-\left(M_a/M_{cr}\right) \left(L_{cr}/L\right)^\rho} \leq I_g \quad \text{for } \rho > 1\%$ $I_e = I_{cr} + (I_g - I_{cr})^{-\left(M_a/M_{cr}\right) \left(L_{cr}/L\right)} \leq I_g$	Eq. (8.20) Eq. (8.21)

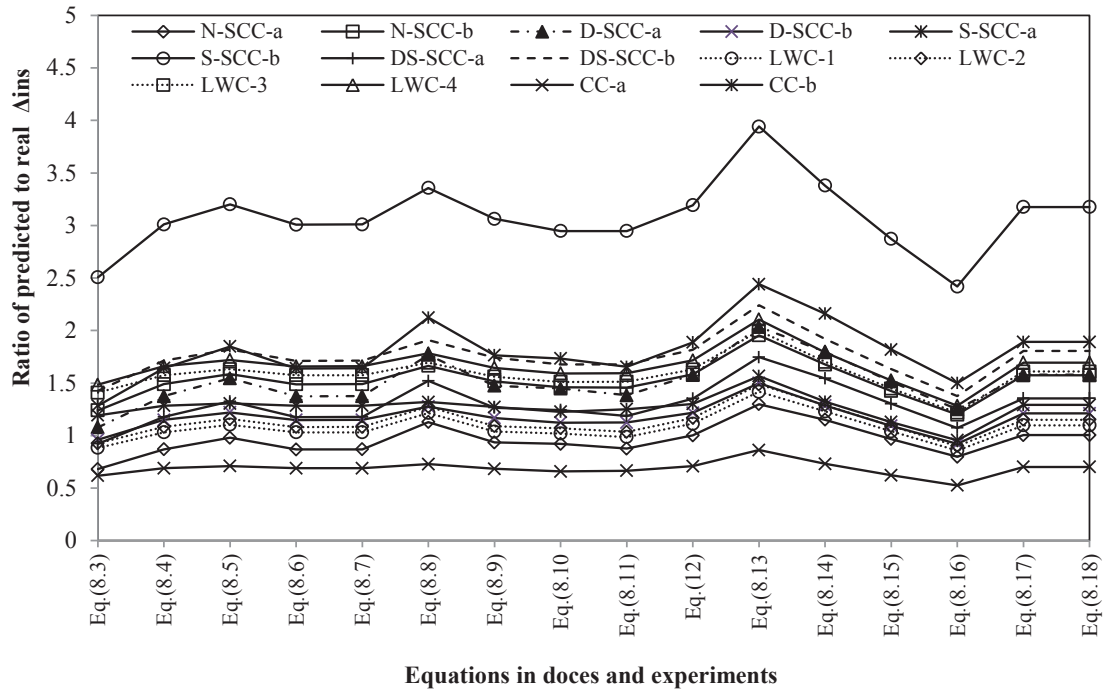


Figure 8.23: Comparing the experimental and calculated data of instantaneous deflection of slabs

In Figure (8.23), the measured experimental data of instantaneous deflection in the LWC slabs are compared with the instantaneous deflection of the same slabs by existing I_e models in Table (8.6).

In Figure (8.24), the effective moment of inertia for the slabs are calculated with the existing models in Table (8.6) and compared with the effective moment of inertia for the measured deflection. The effective moment of inertia for the measured deflections in the slabs is calculated form the Equation (8.22).

$$I_e = k \cdot \frac{w_a \cdot l^4}{E_c \cdot \Delta_{ins}} \quad Eq. (8.22)$$

Where; k is the coefficient of support condition, w_a is the applied loading, l is the span length and Δ_{ins} is the measured instantaneous deflection of slab.

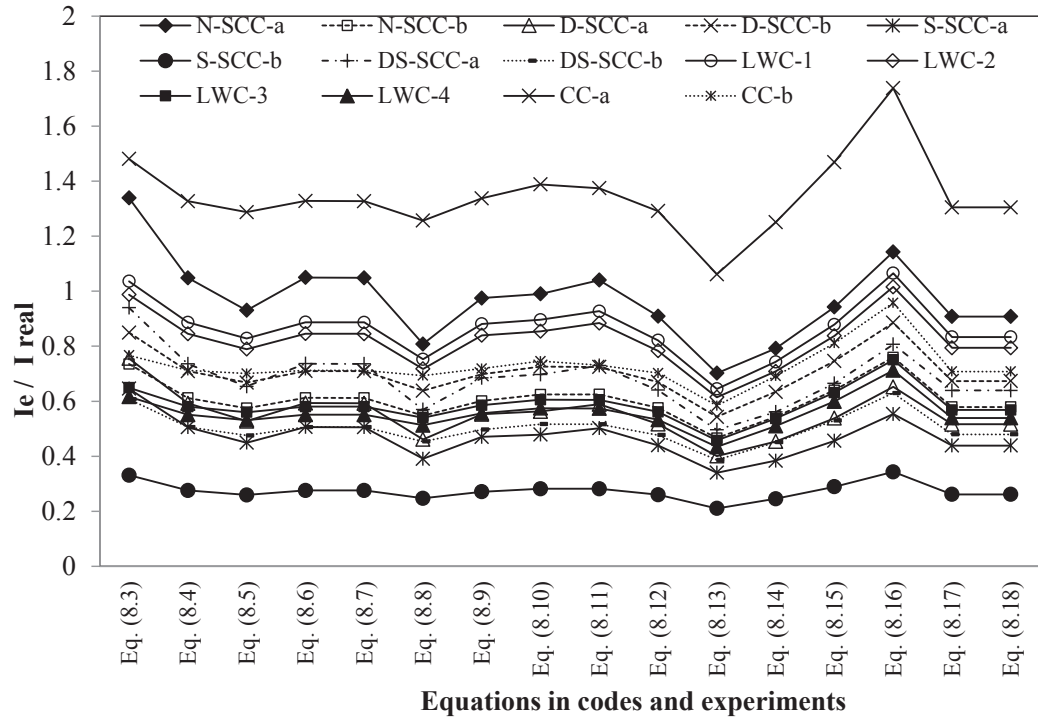


Figure 7.24: Comparing the experimental and predicted I_e of slabs

Comparative investigation of the presented models and experimental data show that the models in Equations (8.13) and (8.14) give conservative Δ_{ins-14} predictions in SCC and FRSCC slabs. In this study, certain required intrinsic and/or extrinsic variables (i.e., properties of concrete, support conditions, loading type and age) in I_e prediction are well covered by the presented equations. Beside, both Equation (8.3) as a basis of all other related studies and Equation (8.18) give the closest prediction to the recorded experimental data. Therefore, with the models in Equations (8.3) and (8.16) as the basis, the effect of time-dependent elasticity, fibre volume fraction in the mixture, ultimate and cracking moment capacity along with the applied service moment have been included in the equation in order to obtain a better estimate of the instantaneous deflection.

Fibre type, fibre volume fraction and the aspect ratio of fibber are the main effective parameters in comparison of fibre reinforced sections especially in the tensile and flexural behaviour. Regarding the negligible difference in aspect ratio of the applied fibres in the slabs, the effect of type and volume fraction of the fibre on the instantaneous deflection is investigated. Additionally, according to considerable effect of the time-dependent M_u on the moment ratios and deflection in Figures (8.20) and (8.21), the M_{cr}/M_u ratio is also included in the developed relationship. To include the effect of the loading on the immature concrete (before 28 days), the ratio of modulus of elasticity of concrete at loading age to that of the 28 days is applied in the equation.

Equations (8.3), (8.16) are basically developed for beams. Equation (8.16) predicts I_e in two distinct ranges of $1 < M_{cr}/M_a < 3$ and $M_{cr}/M_a > 3$ and there is no limit for M_{cr}/M_a in Equation (8.3). The ratios of M_{cr}/M_a and M_{cr}/M_u for slabs in this study are between 0.87-1.48 and 0.35-0.43 respectively. Accordingly, the proposed equation is recommended to apply for $M_{cr}/M_a < 3$.

8.9. PROPOSED MODEL OF I_e IN SELF-COMPACTING CONCRETE SLABS

Equation (8.23) is proposed to predict the I_e in SCC and FRSCC slabs. The developed model is based on the most-compatible results in Figure (8.23) and (8.24). The required coefficients and the limitations of Equation (8.23) application are illustrated in Table (8.7).

$$I_e = \alpha \times I_{cr} \left(\frac{M_{cr}}{M_a} \right)^{(1-0.1V_f)} + \left[(1 - \beta) + \frac{M_{cr}^2}{(M_a \times M_u)^{(2+\beta)}} \right] I_g \quad \text{Eq. (8.23)}$$

Table 8.7: Coefficients and limitations of proposed equation for I_e estimation

I_e, ρ, M_a	Coefficient of fibre type (α)		Ratio of elasticity β	Fibre content V_f
$I_e \leq 0.6I_g$ $\rho \geq 0.005$ $M_{cr}/M_a < 3$	$\alpha=0.9$	for DS-SCC	$\beta=E_{c-14}/E_{c-28}$	Fibre volume fraction in the mixture (Kg/m ³) presented in Table 1
	$\alpha=1.0$	for SCC	E_{c-14} and E_{c-28}	
	$\alpha=1.15$	for D-SCC	Modulus of elasticity of concrete at	
	$\alpha=1.95$	for S-SCC	14 and 28 days	

Figure (8.25) shows the comparison of the recorded data of instantaneous deflection for SCC and FRSCC slabs with the prediction of the proposed equation by application corresponding coefficients. Obviously the predicted values in all slabs are close to the recorded values.

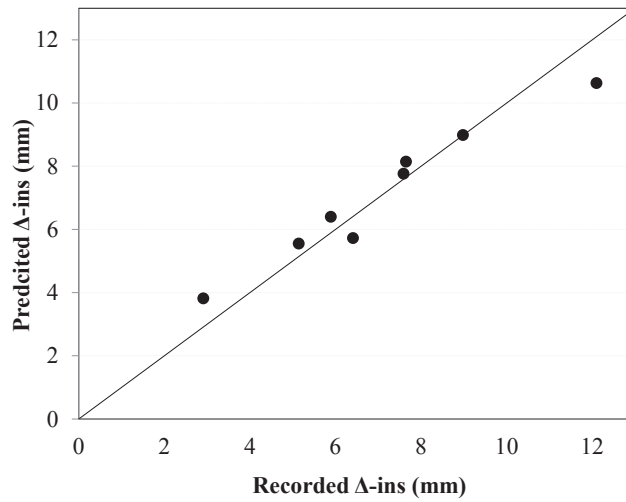


Figure 8.25: Comparison of the recorded and predicted deflection of slabs at the age of 14 days

8.10. ROPOSED MODEL OF I_e IN LIGHTWEIGHT CONCRETE SLABS

Equation (8.24) is proposed to predict I_e in LWC slabs. The developed model is based on the most-compatible results in Figure (8.23) and (8.24). The required coefficients and limitations in application of the model are illustrated in Table (8.8). This type of LWC is widely used in Australia; hence, the maximum I_e is recommended to agree with the limitations of AS-3600-09 [36].

$$I_e = I_{cr} \times \left(\frac{M_{cr}}{M_a} \right)^2 + \left[(1 - \beta) + \frac{(M_{cr})^{1.7}}{(M_a \times M_u)} \right] \times I_g \quad Eq. (8.24)$$

Table 8.8: Coefficients and limitations of proposed model for I_e in LWC slabs

Coefficient	β	ρ	M_{cr}/M_a	I_e
Limit	E_{c-14}/E_{c-28}	≥ 0.005	≤ 3	$\leq 0.6I_g$

Where; ρ is the ratio of the tensile reinforcement in the section, and E_{c-14} and E_{c-28} are the modulus of elasticity of LWC at 14 and 28 days respectively.

Excluding the area of steel bars from the section, underestimates the I_g by 17%. Therefore, the transformed section method is utilized to minimize the error in the calculation of I_g of the uncracked section in the slabs.

Figure (8.26) compares the recorded Δ_{ins-14} of LWC slabs with the prediction of the proposed model in Equation (8.24). The proposed model accurately predicts the I_e values; consequently, the predicted Δ_{ins-14} of LWC slabs is in good agreement with the measured experimental data.

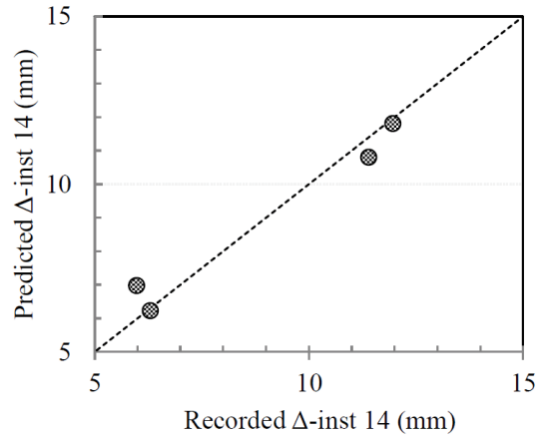


Figure 8.26: Comparison of the recorded and predicted deflection of slabs at 14 days by proposed model

8.11. SUMMARY

The instantaneous deflection of lightweight concrete slabs subjected to service load in the experimental program is described in detail. Effect of the loading level and the ratio of applied moment to the cracking moment of slab on the recorded instantaneous deflection are also investigated.

The experimental program consists of four LWC slabs used to record the deflection immediately after applying the uniformly distributed loading on the slabs. The slabs were simply supported with identical span length and section dimensions (3.5m length $\times$ 0.4m width $\times$ 0.161m height). Each slab specimen was moist cured for 14 days and then subjected to loading. All slabs were internally reinforced with 4 N12 deformed steel bars.

The applied load on the slabs consisted of self-weight plus superimposed sustained loads by the concrete blocks on top of the slabs. Each pair of slabs was subjected to the total loading equal to 30% and 40% of the ultimate bending capacity of the slab section.

The instantaneous deflection of slabs was recorded in two different points along the span at the age of 14 days. The first point was the mid-span of the span and second point was the border of high moment region in the span length.

The measured instantaneous deflection was compared with the calculated elastic deflection of slabs at age 14 days. Table (8.9) shows the instantaneous and elastic deflection of the LWC slabs at the monitoring points in mid-span and the third point of slab.

Table 8.9: Instantaneous and elastic deflection of LWC slabs at monitoring points

Slab	Loading level (1)		Loading level (2)		Δ_{e-1} (mm)		Δ_{e-2} (mm)	
	Δ_1 (mm)	Δ_2 (mm)	Δ_1 (mm)	Δ_2 (mm)	14d	28d	14d	28d
LWC-1	11.84	11.19			4.99	4.57	4.37	4.11
LWC-2	10.76	9.22			4.99	4.57	4.37	4.11
LWC-3			6.33	5.43	3.77	3.45	3.22	2.85
LWC-4			5.98	5.16	3.77	3.45	3.22	2.85

According to Figure (8.27) the $(\Delta_{ins} / \Delta_e)_{14}$ ratio is changing by variation of the M_{cr} / M_u and M_a / M_u ratios. However, the $(\Delta_{ins} / \Delta_e)_{14}$ ratio is noticeably more sensitive to the ratio of M_{cr} / M_a compared to M_a / M_u ratio. It is evident from Figure (8.27) that the M_a / M_u and M_{cr} / M_a ratios have increasing and decreasing effects on $(\Delta_{ins} / \Delta_e)_{14}$ respectively.

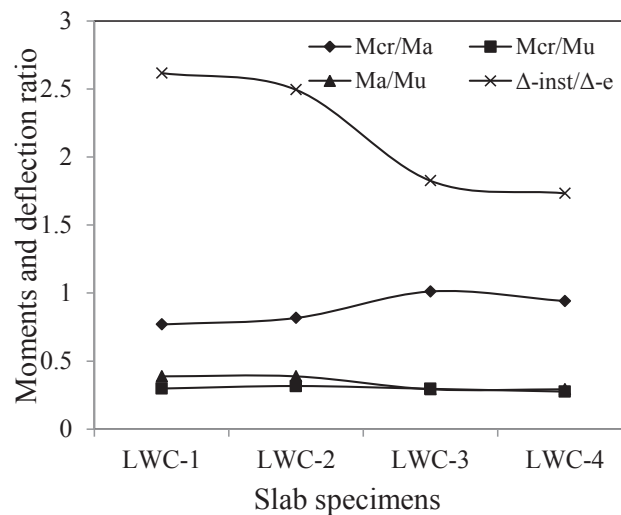


Figure 8.27: Effect of moment ratios on the ratio of instantaneous to elastic deflection of slabs at 14 days.

Comparing the results with previously conducted experiments

The experimental results of instantaneous deflection of lightweight concrete slabs are analysed and discussed broadly and compared with the previously conducted experimental results of conventional concrete slabs by Nejadi (2005) and self-compacting concrete slabs by Aslani (2014).

Table (8.10) shows the recorded instantaneous deflection in the lightweight, self-compacting and conventional concrete slabs. The applied service load and service moment and the calculated elastic deflection at two ages of 14 and 28 days are given in the table.

Table 8.10: Load values, measured instantaneous and predicted elastic deflection of slabs

Slab	w_a (KN/m)	M_a (KN.m)	Δ_e (mm)		Δ_{ins} (mm)
			14 day	28 day	
N-SCC-a	7.31	11.189	3.18	2.9	12.1
N-SCC-b	5.41	8.28	2.36	2.15	5.89
D-SCC-a	7.26	11.12	3.5	2.85	7.65
D-SCC-b	5.36	8.21	2.58	2.11	7.59
S-SCC-a	7.3	11.18	3.45	2.87	6.41
S-SCC-b	5.4	8.27	2.55	2.12	2.91
DS-SCC-a	7.33	11.23	3.29	2.85	8.98
DS-SCC-b	5.43	8.32	2.44	2.11	5.14
LWC-1	7	10.72	4.99	4.57	11.96
LWC-2	7	10.72	4.99	4.57	11.4
LWC-3	5.29	8.095	3.77	3.45	6.3
LWC-4	5.29	8.095	3.77	3.45	5.98
CC-a	4.44	6.8	2.73	2.5	11.8
CC-b	3.46	5.3	2.13	1.95	5.04

Comparison of the results presented in Table (8.10) gives the following conclusions:

- Except the D-SCC (a, b) slabs, 10% *increase* of the service load causes about 100% increment of the instantaneous deflection in the slabs of all concrete types. In D-SCC (a, b) slabs, the increased load shows almost no effect on the instantaneous deflection at the age of 14 days;
- The instantaneous deflection at the first load level ($0.3M_u$ in LWC and SCC slabs, $0.2M_u$ in CC slabs) in all slab specimens except S-SCC is in a similar range;
- The instantaneous deflection at the higher load level ($0.4M_u$ in LWC and SCC slabs, $0.26M_u$ in CC slabs) in N-SCC, LWC and CC slabs is in a similar range, however, the instantaneous deflection in S-SCC is about half that of the other slabs;
- There is considerable difference between the predicted elastic deflection and measured instantaneous deflection of all slabs. The ratio of instantaneous to elastic deflection varies from 1.14 in S-SSC-b to 3.8 in N-SCC-a.

Equations (10.1) and (10.2) are proposed and verified to predict I_e in lightweight and conventional concrete and self-compacting concrete slabs with and without fibre reinforcement respectively. The required coefficients and the limitations of the Equations (8.25) and (8.26) application are illustrated in Tables (9.11) and (9.12) respectively.

$$I_e = I_{cr} \times \left(\frac{M_{cr}}{M_a} \right)^2 + \left[(1 - \beta) + \frac{(M_{cr})^{1.7}}{(M_a \times M_u)} \right] \times I_g \quad Eq. (8.25)$$

Table 8.11: Coefficients and limitations of proposed model for I_e in LWC slabs

Coefficient	β	ρ	M_{cr}/M_a	I_e
Limit	E_{c-14}/E_{c-28}	≥ 0.005	≤ 3	$\leq 0.6I_g$

$$I_e = \alpha \times I_{cr} \left(\frac{M_{cr}}{M_a} \right)^{(1-0.1V_f)} + \left[(1 - \beta) + \frac{M_{cr}^2}{(M_a \times M_u)^{(2+\beta)}} \right] I_g \quad \text{Eq. (8.26)}$$

Table 8.12: Coefficients and limitations of proposed equation for I_e estimation

I_e, ρ, M_a	Coefficient of fiber type (α)	Ratio of elasticity β	Fiber content V_f
$I_e \leq 0.6I_g$	$\alpha=0.9$ for DS-SCC	$\beta=E_{c-14}/E_{c-28}$	Fiber volume
$\rho \geq 0.005$	$\alpha=1.0$ for SCC	E_{c-14} and E_{c-28}	fraction in the mixture
$M_{cr}/M_a < 3$	$\alpha=1.15$ for D-SCC	Modulus of elasticity of	(Kg/m ³) presented in
	$\alpha=1.95$ for S-SCC	concrete at	Table 1
		14 and 28 days	

According to the instantaneous deflection of three different types of reinforced concrete slabs made with normal, lightweight and self-compacting concrete slabs, the following conclusion can be drawn from the current study:

- Flexural deflection behavior of slabs is strongly influenced by concrete type with similar compressive strength;
- In the existing models in the literature, there is an almost linear relationship between the load magnitude and the instantaneous deflection of slabs. The results of this study indicated a considerable nonlinear effect of the load level on the instantaneous deflection, especially in the low-strength concrete slabs;
- 10% increment of the load value causes about 100% and 105% higher instantaneous deflection in the lightweight concrete and self-compacting concrete slabs, respectively. In low-strength normal concrete slabs, the instantaneous deflection is raise about 135% by 6% higher loading;
- The instantaneous deflection of fiber reinforced self-compacting concrete slabs experienced less variations with the changes of loading level;
- Considering the elastic deflection equal to instantaneous deflection even for $M_a < M_{cr}$ in slabs is completely incorrect and causes unsafe design of reinforced concrete slabs.
- Elastic deflection is not a trustworthy indicator of the instantaneous, time-dependent and total deflection of slabs;
- Development of the mechanical properties of normal, lightweight and self-compacting concrete with age is different before the age of 28 days. Therefore, for the early age

loading, calculations of the instantaneous deflection of the slabs based on the mechanical properties at the age of 28 days may result in inaccurate values;

- The ratio of service moment to the ultimate moment capacity of slabs strongly affects the instantaneous deflection;
- A new model is proposed and verified to predict the effective moment of inertia in the self-compacting concrete slabs with and without fiber reinforcing. The predicted instantaneous deflections by this model are in agreement with the experimental results;
- Another model of the effective moment of inertia is proposed and verified for the normal and lightweight concrete slabs. The experimentally recorded instantaneous deflections are well-predicted by this model.

REFERENCES

1. Nejadi, S., *Time-dependent cracking and crack control in reinforced concrete structures*. 2005, PhD thesis, The University of New South Wales Sydney, Australia.
2. Mazzotti, C. and M. Savoia, *Long-term deflection of reinforced self-consolidating concrete beams*. ACI Structural Journal, 2009. **106**(06).
3. Yoğurtcu, E. and K. Ramyar, *Self-compacting lightweight aggregate concrete: design and experimental study*. Magazine of Concrete Research, 2009. **61**(7): p. 519-527.
4. Güneyisi, E., M. Gesoğlu, and E. Booya, *Fresh properties of self-compacting cold bonded fly ash lightweight aggregate concrete with different mineral admixtures*. Materials and structures, 2012. **45**(12): p. 1849-1859.
5. Mazaheripour, H., et al., *The effect of polypropylene fibers on the properties of fresh and hardened lightweight self-compacting concrete*. Construction and Building Materials, 2011. **25**(1): p. 351-358.
6. Vakhshouri, B. and S. Nejadi, *Mix design of light-weight self-compacting concrete*. Case Studies in Construction Materials, 2016. **4**: p. 1-14.
7. Xu, Y., et al., *Mechanical properties of expanded polystyrene lightweight aggregate concrete and brick*. Construction and Building Materials, 2012. **27**(1): p. 32-38.
8. Trussoni, M., C.D. Hays, and R.F. Zollo, *Fracture Properties of Concrete Containing Expanded Polystyrene Aggregate Replacement*. ACI Materials Journal, 2013. **110**(5).
9. Sabaa, B. and R.S. Ravindrarajah. *Engineering properties of lightweight concrete containing crushed expanded polystyrene waste*. in *Materials Research Society, 1997, Fall Meeting, Symposium MM, Advances in Materials for Cementitious Composites December*. 1997.
10. Scanlon, A. and P.H. Bischoff, *Shrinkage restraint and loading history effects on deflections of flexural members*. ACI Structural Journal, 2008. **105**(4).
11. Vakhshouri, B. and S. Nejadi. *Experimental study of time dependent bond transfer length under pure tension in slabs*. in *International Conference on Construction Materials and Structures*. 2014. IOS Press BV.
12. Li, Y., N. Liu, and B. Chen, *Properties of lightweight concrete composed of magnesia phosphate cement and expanded polystyrene aggregates*. Materials and Structures, 2015. **48**(1-2): p. 269-276.
13. AS-1012.14, *Methods of testing concrete - Method for securing and testing cores from hardened concrete for compressive strength*. Standards Australia, 1991.
14. ASTM-C39, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. American Society of Testing and Materials, 2000.
15. AS-1012.17-97, *Methods of testing concrete - Determination of the static chord modulus of elasticity and Poisson's ratio of concrete specimens*. Standards Australia, 2014.
16. ASTM-C469/C469M-14, *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*. American Society of Testing and Materials, 2000.
17. Vakhshouri, B. and S. Nejadi. *Limitations and Uncertainties in the Long Term Deflection Calculation of Concrete Structures*. in *Second International Conference on Vulnerability and Risk Analysis and Management (ICVRAM) and the Sixth International Symposium on Uncertainty, Modeling, and Analysis (ISUMA)*. 2014.
18. Alshaikh, A.H. and R. Al-Zaid, *Effect of reinforcement ratio on the effective moment of inertia of reinforced concrete beams*. ACI Structural Journal, 1993. **90**(2).
19. ACI-318-02. *Building code requirements for structural concrete (ACI 318-08) and commentary*. 2002. American Concrete Institute, International Organization for Standardization.
20. ACI-435, *Deflections of reinforced concrete flexural member*. American Concrete Institute (ACI), Concrete International, 1968.

21. AS-3600-09, *Concrete structures*. Australian Standard, 2009.
22. ACI-440-06, *Guide for the design and construction of concrete reinforced with FRP bars*. American Concrete Institute (ACI) Committee 440, ACI 440.1R-06, ACI, Farmington Hills, MI, 44, 2006.
23. Bischoff, P.H. and R. Paixao, *Tension stiffening and cracking of concrete reinforced with glass fiber reinforced polymer (GFRP) bars*. Canadian Journal of Civil Engineering, 2004. **31**(4): p. 579-588.
24. du Béton, C.E.-I., *CEB-FIP model code (MC-90)*. 1993, Thomas Telford Ltd, London.
25. Bischoff, P.H., *Deflection calculation of FRP reinforced concrete beams based on modifications to the existing Branson equation*. Journal of Composites for Construction, 2007. **11**(1): p. 4-14.
26. Bischoff, P.H. and S.P. Gross, *Equivalent moment of inertia based on integration of curvature*. Journal of Composites for Construction, 2010. **15**(3): p. 263-273.
27. Faza, S. and H. GangaRao. *Pre-and post-cracking deflection behaviour of concrete beams reinforced with fibre-reinforced plastic rebars*. in *Proceedings of the First International Conference on Advance Composite Materials in Bridges and Structures (ACMBS-I)*, Canadian Society of Civil Engineers, Sherbrooke, Canada. 1992.
28. Benmokrane, B., O. Chaallal, and R. Masmoudi, *Flexural response of concrete beams reinforced with FRP reinforcing bars*. ACI Structural Journal, 1996. **93**(1): p. 46-55.
29. Yost, J.R., S.P. Gross, and D.W. Dinehart, *Effective moment of inertia for glass fiber-reinforced polymer-reinforced concrete beams*. ACI Structural Journal, 2003. **100**(6).
30. Rafi, M.M., et al., *Aspects of behaviour of CFRP reinforced concrete beams in bending*. Construction and Building Materials, 2008. **22**(3): p. 277-285.
31. Alsayed, S., Y. Al-Salloum, and T. Almusallam, *Performance of glass fiber reinforced plastic bars as a reinforcing material for concrete structures*. Composites Part B: Engineering, 2000. **31**(6): p. 555-567.
32. Newhook J., *Reinforcing concrete structures with fibre reinforced polymers*. ISIS Canada: Design Manual No.3, The Canadian Network of Centres of excellence on intelligent sensing for innovative structures, Winnipeg, Manitoba, Canada, 2001.
33. Hall, T. and A. Ghali, *Long-term deflection prediction of concrete members reinforced with glass fibre reinforced polymer bars*. Canadian Journal of Civil Engineering, 2000. **27**(5): p. 890-898.
34. ACI-224.2R-86, *Cracking of concrete members in direct tension* American Concrete Institute (ACI) Committee 224, Detroit, USA, 1986.
35. Fikry, A.M. and C. Thomas, *Development of a model for the effective moment of inertia of one-way reinforced concrete elements*. ACI Structural Journal, 1998. **95**(4).
36. AS-3600-09, *Concrete structures*. Standards Australia, 2009.

CHAPTER 9

EXPERIMENTAL PROGRAM (PHASE II) – FLEXURAL LONG-TERM DEFLECTION

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Chapter 9: Flexural long-term deflection

9.1. INTRODUCTION

There have been a considerable number of investigations about the time-dependent behaviour of concrete structures in the literature since it was first experienced and reported almost a century ago. However, there is no distinctive boundary in Reinforced Concrete (RC) members as to when the instantaneous (short-term) deformation ends and the long-term deformation begins. Even, there is no end for time-dependent deflection of reinforced concrete structures during the service life which in turn, makes the prediction of the final total deflection inaccurate. To determine the long-term deflection of a concrete member, the time-dependent effects of shrinkage, creep and the consequence reduction in tension stiffening need to be accurately considered. Additional factors which can contribute to the increased long-term deflection include the development of new cracks, widening of secondary cracks due to creep and shrinkage effects, and effects of the repeated load cycles [1]. The initial deflection in the flexural reinforced concrete elements can be considerably smaller than the long-term values, thus, the time-dependent deflection is fundamentally significant in design and assessment of reinforced concrete structures.

Instantaneous deflection (Δ_{ins}) is the basis of time-dependent and total deflection calculation of RC structures. As by definition, long-term deflection is the increase in instantaneous deflection by time due to inelastic deformations as a result of creep and shrinkage effects. The ratio of long-term to the instantaneous deflection is a crucial parameter in serviceability considerations of the reinforced concrete structures. Although it is not verified by all experimental investigations, the existing design codes predict the long-term deflection of flexural elements as a multiple of the initial or instantaneous deflection. It might generally lead

to an underestimated time-dependence and consequently, total deflection of the RC member. Therefore, it may reduce the safety margin in the design process dramatically.

Along with the uncertainties associated with determination of creep, shrinkage and material properties of the concrete, accurate prediction of Δ_{ins} is a major problem in the calculation of long-term and total deflection of slabs [2]. Existing models in ACI-209-08 [3] and ACI-318-08 [4] overestimate the modulus of elasticity and the effective stiffness and therefore, underestimate the instantaneous and long-term deflection in the concrete slabs subjected to early age loading [5-7].

Long-term deflections are calculated either by modifying the material parameters such as modulus of elasticity or solely by multiplying the initial deflection by a factor. Deflection multipliers depend on the increase in curvature due to cracking, shrinkage and creep which in turn depend on (a) structural layout; (b) the ratio of applied load to cracking load; (c) reinforcement; and (d) material properties [8].

Rahman et al. (1998) [9] proposed a simplified method of predicting the long-term deflection of cracked beams by the multiplication of an amplification factor in the initial deflection. The multiplier factor was a function of the long-term strain and the initial neutral axis.

In the finite element analysis, some researchers apply the reduced modulus of elasticity to predict the time-dependent deflection of slabs to account for the deflection increment by time due to cracking, creep and shrinkage. Vollum (2003) [8] verified the previously suggested equivalent elastic modulus model for the flat slabs and found a factor of approximately “2” for the increased long-term deflection due to the effects of cracking and shrinkage.

Neville et al. (1977) [10] discussed the limitation of deflection control by span to depth ratio in some codes of practice and compared the results with predictions of British Standard CP110.1972. They also investigated the accuracy of the long-term deflection multiplier in some design codes and found that the creep and shrinkage do not increase the calculated instantaneous deflections as much as where multipliers are used. The calculated long-term deflections by CP110.1972 [11] were of the order of 1.5 to 2 times the instantaneous values and the span to depth ratio gave underestimated long-term deflection due to the effects of creep and shrinkage.

Deflection of flexural elements with low-reinforcement ratio e.g. slabs is highly sensitive to the loading before the age of 28 days [12]. Modulus of elasticity, loading value, load distribution and support conditions are the principle factors in the deflection calculation of slabs. Also, continuity of sections along the span; i.e. variable cross-sections and hence variable moment of inertia depending on the degree of cracking, strongly influence the instantaneous and time-dependent deflections.

Mechanical properties of concrete are changing continuously from an approximately liquid state to a visco-plastic material within a few hours, which is followed by further development into a hardened material with almost elastic properties at two loading values up to about 40% of the compressive strength of concrete. In different types of concrete, the compressive strength and modulus of elasticity at seven days is changing from about 50% to 85% of those for 28 days respectively [13-16]. The early age loading requires using the distinct properties at the age of loading; in other words, either the model or the early age properties should be modified to get an accurate prediction. The flexural stiffness is also changing with the cracking due to creep and shrinkage strains of concrete, and the early age loading changes the predictable creep and shrinkage behaviour.

Gilbert and Guo (2005) [17] performed a three year experimental program by testing of seven specimens of large-scale reinforced concrete flat slabs. They recorded the ratio of long-term to instantaneous deflection in the range of 4 to 7 after 240 days and 5 to 9 after 1100 days, while, this ratio may be as large as two to three in the predictions of design codes [18]. The loading history and the environmental effects were a source of considerable differences in the slabs.

In the absence of a reliable reference, the unknown effects of early age loading and type of concrete make the deflection estimation of reinforced concrete slabs more complicated. Determination and limitation of the time-dependent deflection by Δ_{ins} play actively a critical role in RC slab design. Hence accurate estimation of Δ_{ins} and Δ_{long} enriches the total deflection estimation and consequently makes an improved structural and performance-based design.

The validity of the constitutive laws for the instantaneous and long-term behaviour of CC, LWC and SCC slabs should be verified, and the effect of mixture proportions must be carefully checked. This study also provides empirical evidence of the principal impact of the creep coefficient and shrinkage strain on the long-term deflection of slabs made by different types of concrete.

The main objectives of this chapter are:

- Presenting the time-dependent deflection of Light-Weight Concrete (LWC) slabs;
- Comparing the experimental values of instantaneous and time-dependent deflection of conventional Concrete (CC), Self-Compacting Concrete (SCC) and LWC slabs subjected to the loading at the age of 14 days (Δ_{ins-14} , Δ_{long});
- Investigating the effect of the loading level, creep and shrinkage strains on Δ_{ins-14} and the ratio of $\Delta_{long} / \Delta_{ins-14}$;

- Comparing the experimental results of the instantaneous and long-term deflection with the predictions of ACI-318-08 [4], AS-3600-09 [19], and CSA.A23.04 [20].

9.2. EXPERIMENTAL PROGRAM

9.2.1. Construction of the Specimens and Curing

The lightweight concrete slabs described in Chapter (7) were maintained under the sustained loading to monitor the time-dependent deflection at different points. In other words, the slabs for the instantaneous study were kept under the applied loads to monitor and record the long-term deflection due to effects of service loading, creep and shrinkage.

Figure (9.1) shows the reinforcing detail and slab dimensions for the long-term deflection study. In total, four LWC slabs with the same mix design were internally reinforced with 4 N12 deformed steel bars. The slabs were similarly simply supported having span length of 3500 mm, depth 161 mm and width 400 mm. The side and bottom cover of concrete are 34 and 19 mm respectively. The steel bars are set in equal spacing through the slab width.

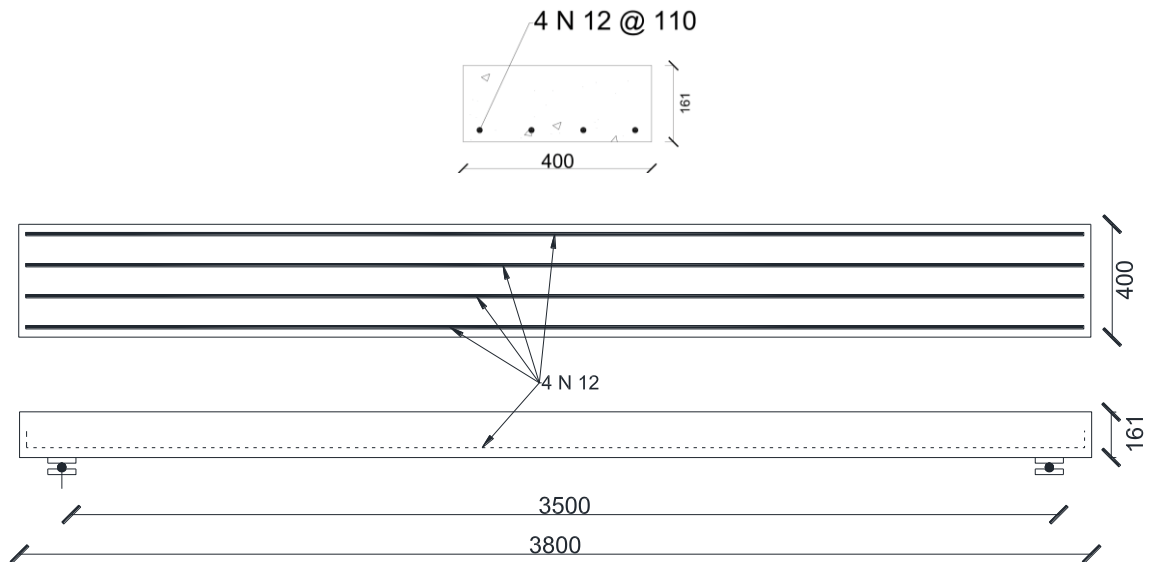


Figure 9.1: Cross section and reinforcing detail of LWC slab specimens in long-term deflection test

Figure (9.2) shows a general view of the test setup to monitor the time-dependent behaviour of the four LWC slabs. The figure also shows the support condition at two edges of each slab, the pads under the concrete loading blocks, the cables connected to the embedded strain gauges

and the data taker to record the strain variation of the steel bars and concrete surface. The strain data and flexural deflection were recorded in 60 minutes intervals during the test period.



Figure 9.2: General view of the long-term deflection test set up of LWC slabs

The slabs were divided into two groups to apply the service loads. The gravity load on each pair of slab specimens was different. The sustained loads were consisted of the self-weight plus superimposed sustained loads via carefully constructed and arranged concrete blocks supported on the top surface of the specimens.

9.2.2. Monitoring points of time-dependent flexural deflection

The time-dependent deflection of slabs was recorded in two different points along the span length. As shown in Figure (9.2), the deflection at mid-span as the maximum flexural deflection was recorded by Linear Variable Differential Transformer (LVDT). Another LVDT at the third point of the slab recorded the deflection of the slab at the boundary of the middle third high moment region. Comparing the recorded deflection at these two points provide an overview of the changes in the curvature of slab under the uniformly distributed flexural loading. It can also give an insight to the time-dependent variation of section stiffness along the span.

9.2.3. Time-dependent Deflection of Lightweight Concrete Slabs

The time-dependent deflection was monitored under the service load and the effects of creep and shrinkage. For laboratory investigation of long-term deflection of slabs, a total of four singly reinforced LWC one-way slab specimens with identical length, cross-section and reinforcing details shown in Figure (9.2), were monitored for up to 550 days to measure the time-dependent development of strain and deformations under service loads.

The slabs were cast and moist for 14 days and then subjected to uniformly distributed loads at the age of 14 days. The steel strains within the high-moment regions in all slabs, the concrete surface strains at the tensile steel level, the concrete compressive strain at the top surface, deflection at the mid-span and third point of span length and crack spacing were recorded all over the testing period. The required properties of LWC including the compressive and tensile strength were measured on companion specimens in forms of the cylinder, cube and prism and unreinforced blocks at various ages. Also the elastic modulus, creep coefficient and free shrinkage in the concrete specimens were recorded. The measured mechanical properties are presented and discussed in Chapter 5 of this study.

The applied service load in the LWC slab specimens were different gravity loads, consisting of self-weight plus sustained superimposed loads via carefully constructed and arranged concrete blocks supported from the top (of the specimens). To study the influence of loading history on the time-dependent deflection of slabs, two different levels of service loads at the age of 14 days were applied to the slabs. To provide the sustained loading, rectangular concrete blocks of predetermined size and weight were cast and weighed prior to the commencement of the test. As shown in Figure (9.3), the loading blocks were arranged in equal distances on the top surface of each specimen to achieve the desired uniformly distributed sustained load level.



Figure 9.3: Loading pads and concrete block on the slabs for time-dependent testing

Two sustained load levels were considered, namely 40% and 30% of the ultimate design load, and designated as load conditions ‘a’ and ‘b’, respectively. The slab specimens were subjected to uniformly distributed sustained loads, UDL + self-weight. All measurements were taken within the high moment region, i.e. the middle third of the span where $M \geq 90\% M_{\max}$. For long-term tests the loading arrangement and high moment regions are shown in Figure (9.4).

9.2.4. Testing Duration

The time-dependent deflection of LWC slabs was monitored up to 550 days after loading at the age of 14 days. In this period, the changes of strain in the steel bars and side and top surface of the concrete were recorded also. The deflection and strain data from the first instance of loading were recorded every minute for up to 24 hours. Then the data taker recorded data every hour until the end of the test program.

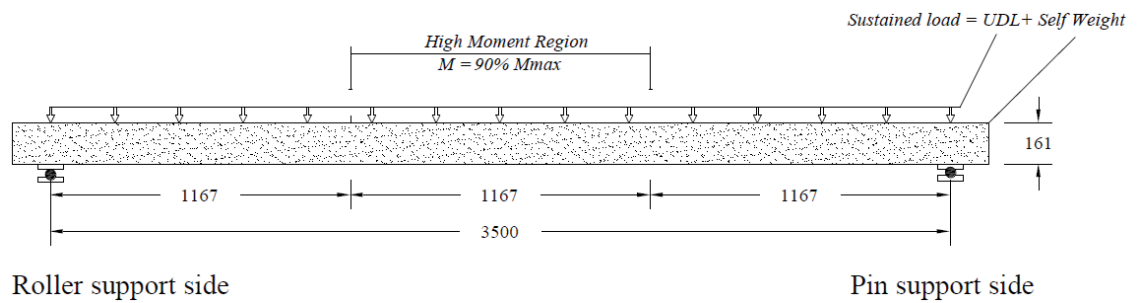


Figure 9.4: Illustrative sustained loads slab specimens

9.2.5. Experimental Data

The flexural deflection of LWC slabs was monitored and recorded immediately after loading and through the test duration by two LVDTs installed at the monitoring points. Measuring the flexural deflection at mid-span and the boundary point of the high moment region moment in Figure (9.4) provides a comparable data of deflection changes along the span length. Considering different levels of cracking along the span length due to the bending moment and effects of creep and shrinkage over a long period of time, the recorded deflection can give an insight to the varying moment of inertia along the span.

Table (9.1) presents the detailed information of the loading level, service and ultimate moments of the LWC slabs.

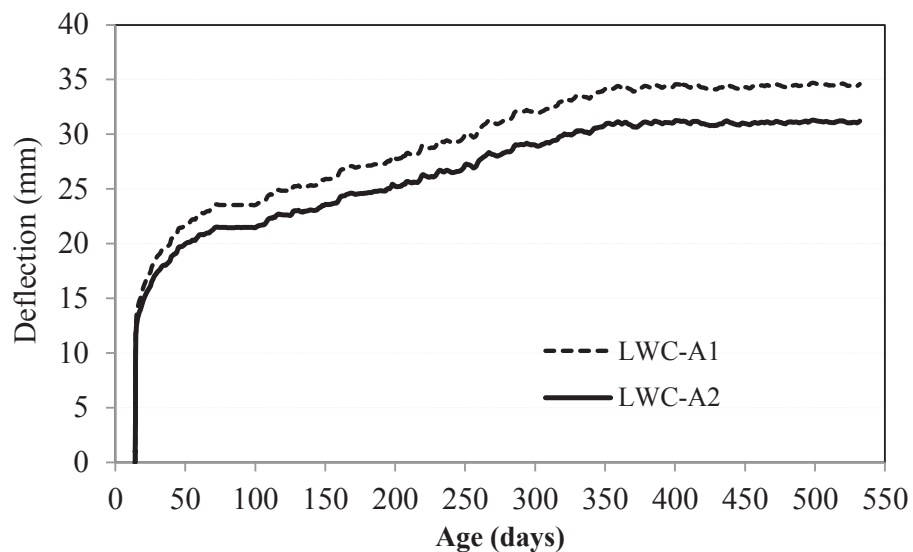
Table 9.1: loading level and bending capacity of the LWC slabs in the long-term test

Slab	W_a (KN/m)	M_a (KN.m)	M_u (KN.m)		M_a / M_u (%)	
			14 day	28 day	14 day	28 day
LWC-A	7	10.72	27.4	27.65	39.13	38.78
LWC-B	7	10.72	27.4	27.65	39.13	38.78
LWC-C	5.29	8.095	27.4	27.65	29.55	29.28
LWC-D	5.29	8.095	27.4	27.65	29.55	29.28

9.2.6. Time-Dependent Deflection at Monitoring Points

Figures (9.5) to (9.8) show the measured time-dependent deflection of LWC slabs under the sustained service loads and effects of creep and shrinkage for 550 days. The Slabs LWC-A and LWC-B are subjected to loading equal to 40% of their ultimate bending capacity, while slabs LWC-C and LWC-D are loaded by 30% of the ultimate bending capacity of the section at the critical point.

The loadings equal to 30% and 40% of the ultimate bending moment are considered as LL_1 and LL_2 in this chapter respectively. The index “1” and “2” after the slab identity indicates the deflection at mid-span and the point on the border the high moment region respectively. For example, LWC-A1 denotes the deflection at mid-span of slab LWC-A. The diagrams in Figures (9.5) to (9.8) consist of the initial deflection of slabs at the time of loading (instantaneous deflection)

Figure 9.5: Time-dependent deflection of slab LWC-A under LL_2 at monitoring points

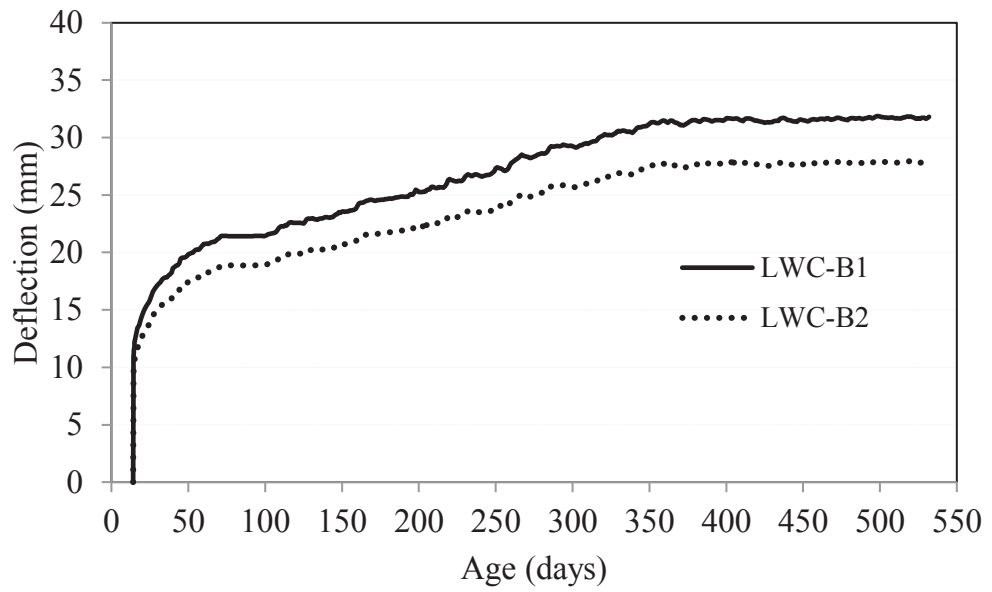


Figure 9.6: Time-dependent deflection of slab LWC-B under LL_2 at monitoring points

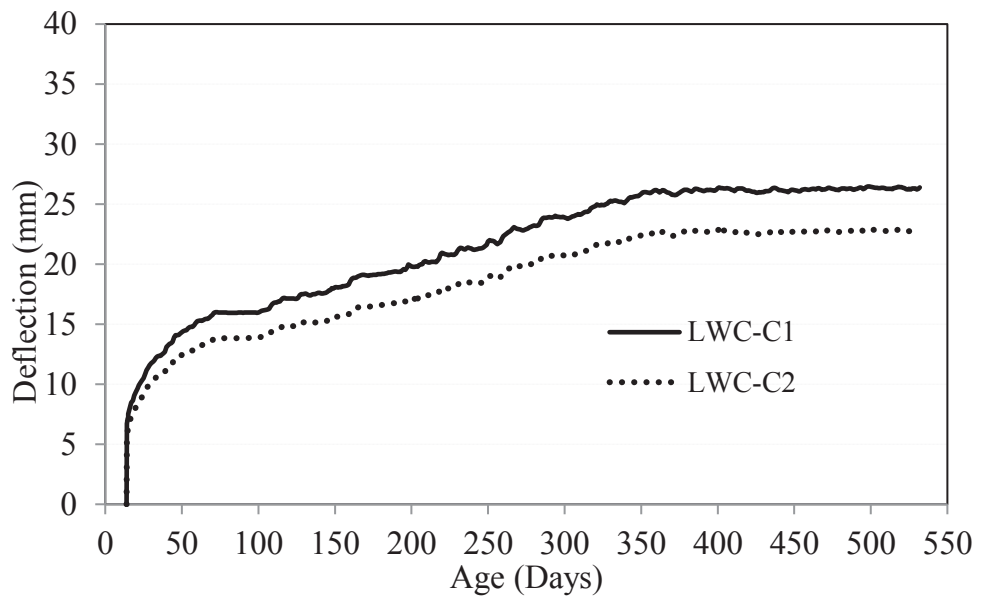


Figure 9.7: time-dependent deflection variation of slab LWC-C under LL_1 at monitoring points

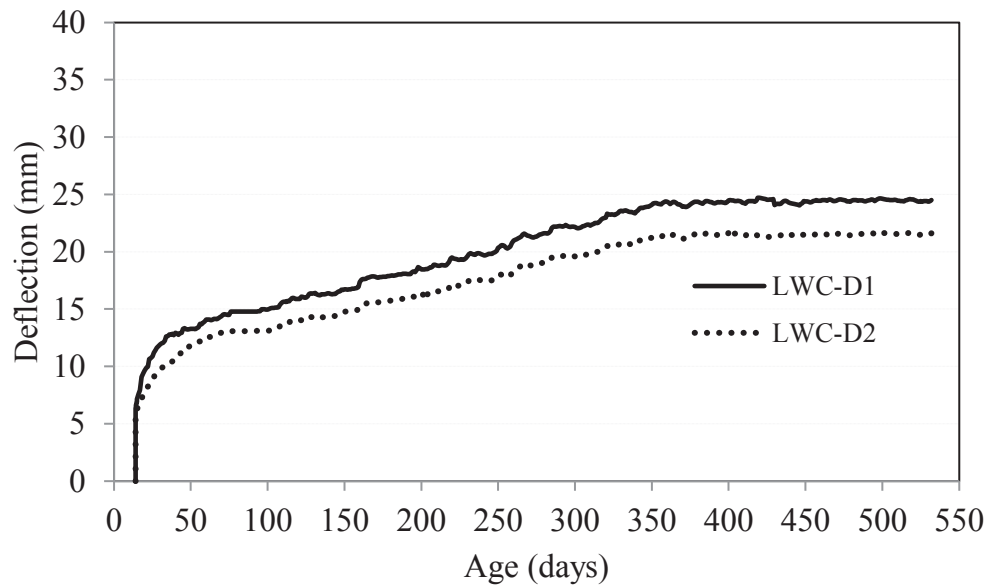


Figure 9.8: Time-dependent deflection variation of slab LWC-D at monitoring points

Figure (9.9) compares the corresponding time-dependent deflection of slabs at mid-span subjected to two distinct loading levels of LL_1 and LL_2 . Figure (9.10) compares the corresponding time-dependent deflection of slabs at the third point of slabs under two different loading levels. The diagrams in Figure (9.9) consist of the initial deflection of slab at the time of loading (instantaneous deflection)

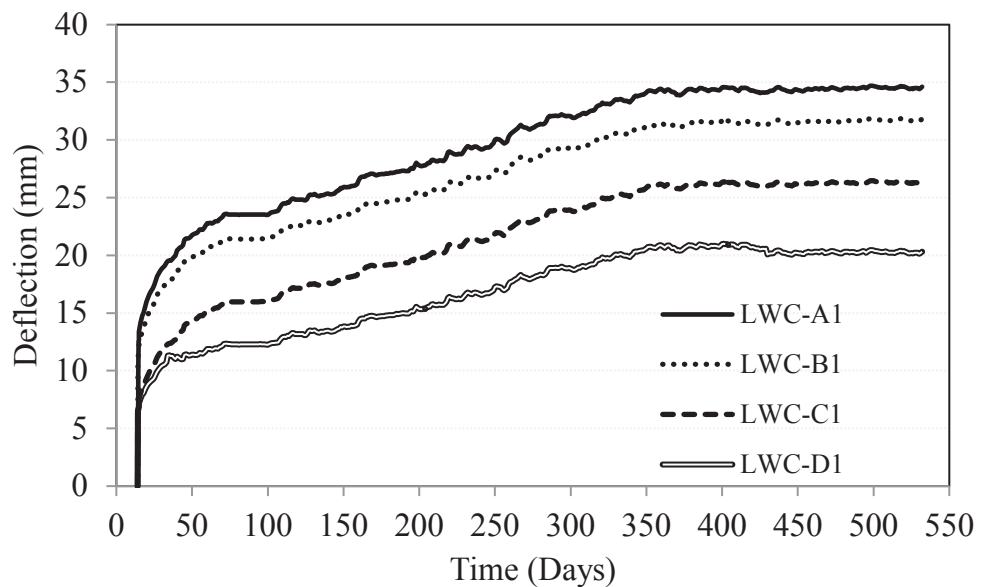


Figure 9.9: Time-dependent deflection of LWC-A and LWC-B slabs under LL_2 and LWC-C and LWC-D under LL_1 at mid-span

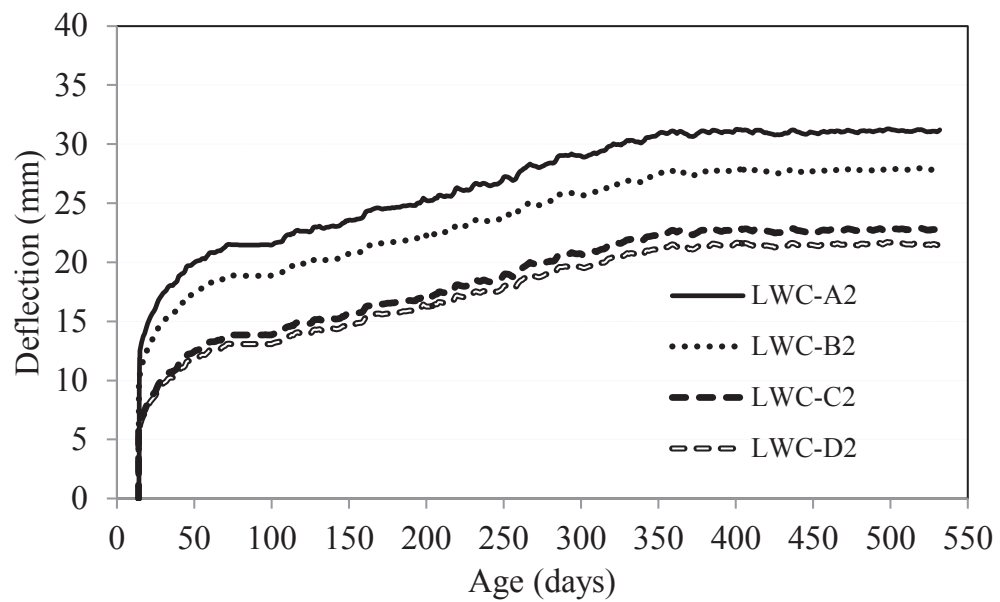


Figure 9.10: Time-dependent deflection of LWC-A and LWC-B slabs under LL_2 and LWC-C and LWC-D under LL_1 at the border of high moment region

The general trend of the time-dependent deflection of slabs at two monitoring points during the monitoring time is similar. However, the ratio of deflection at mid-span to the deflection at the border point of high moment region is considerably difference in the slabs.

Figure (9.11) compares the above-mentioned deflection ratio in the monitoring points of LWC slabs. It can be concluded that the higher the loading level, the higher the deflection ratio in all slabs. In other words, the higher loading level causes more primary cracking in the critical section of the slab that in turn, increases the time-dependent deflection. While, due to change in the primary cracking pattern of the section at the point on the border of high moment region, variation of the deflection ratio with time is not considerable.

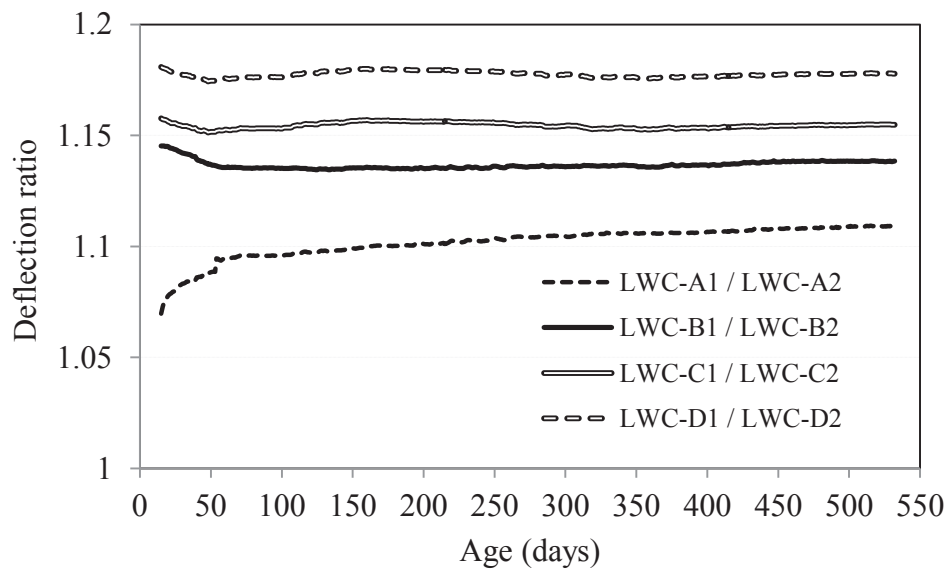


Figure 9.11: Ratio of deflection at mid-span to deflection at border point of high moment region under LL_1 (slabs LWC-C and LWC-D) and LL_2 (slabs LWC-A and LWC0B)

Figure (9.12) compares the ratio of deflection of corresponding points in the slabs under two distinct loading levels. Slabs LWC-A and LWC-C are considered as the representative of slabs under higher and lower loading levels respectively. Despite differences at the initial stage of flexural deformation, the general trend of deflection variation with time is similar in the corresponding points of slabs under different loading levels.

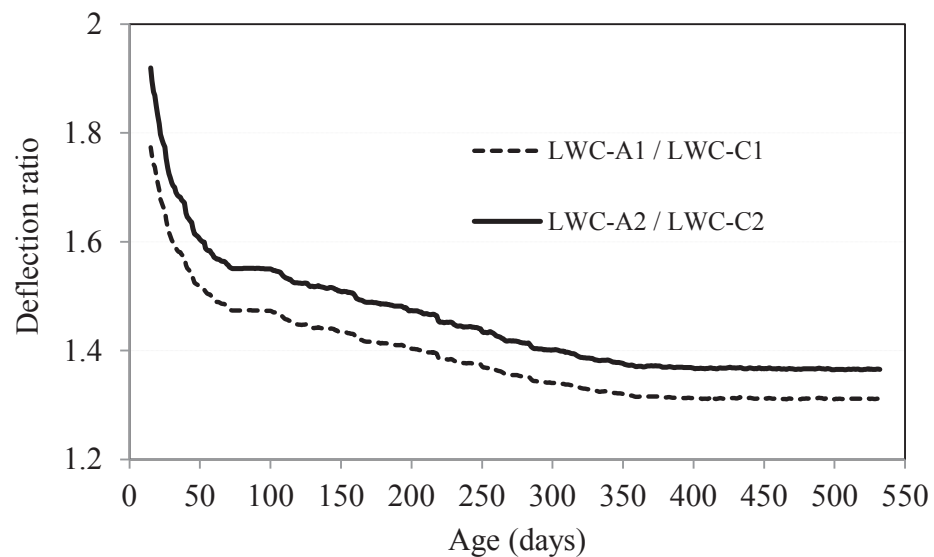


Figure 9.12: Ratios of deflection at corresponding points of LWC-A under LL_2 and LWC-C under LL_1

9.3. RATIO OF LONG-TERM TO INSTANTANEOUS DEFLECTION

The long-term deflection is considered as multiplier of the instantaneous deflection in majority of the design codes. Table (9.2) presents the ratio of long-term deflection to the instantaneous deflection of LWC slabs at different ages.

Table 9.2: Ratio of total to instantaneous deflection of LWC slabs at different ages

Slab	Age (days)															
	14	21	28	56	63	100	128	156	200	249	300	349	400	449	500	532
LWC-A	1	1.37	1.54	1.86	1.90	1.97	2.11	2.17	2.32	2.48	2.68	2.84	2.89	2.87	2.90	2.89
LWC-B	1	1.31	1.47	1.78	1.82	1.88	2.01	2.07	2.22	2.37	2.57	2.73	2.78	2.77	2.79	2.79
LWC-C	1	1.53	1.81	2.35	2.43	2.53	2.78	2.88	3.14	3.43	3.79	4.09	4.18	4.16	4.20	4.19
LWC-D	1	1.65	1.94	2.29	2.34	2.50	2.74	2.80	3.08	3.36	3.70	4.02	4.09	4.08	4.12	4.10

Regardless of the loading level, the deflection multiplier in design codes varies in the range of 2 to 3. According to Table (9.2), this ratio is in agreement with the long-term deflection of slabs at higher loading level. By 10% increase in the loading level, the deflection multiplier increases about 50% above the range recommended in most design codes.

Figures (9.13) and (9.14) graphically compare the increment of the ratio of long-term to instantaneous deflection of LWC slabs. In Figure (9.13), LWC-A1 and LWC-B1 are representatives for the ratio of long-term to instantaneous deflection of slabs LWC-A and LWC-B at mid-span of slab subjected to $LL_2 (0.4M_u)$, respectively.

In Figure (9.14), LWC-C1 and LWC-D1 are representatives for the ratio of long-term to instantaneous deflection of slabs LWC-C and LWC-D at mid-span of slab subjected to $LL_1 (0.3M_u)$, respectively.

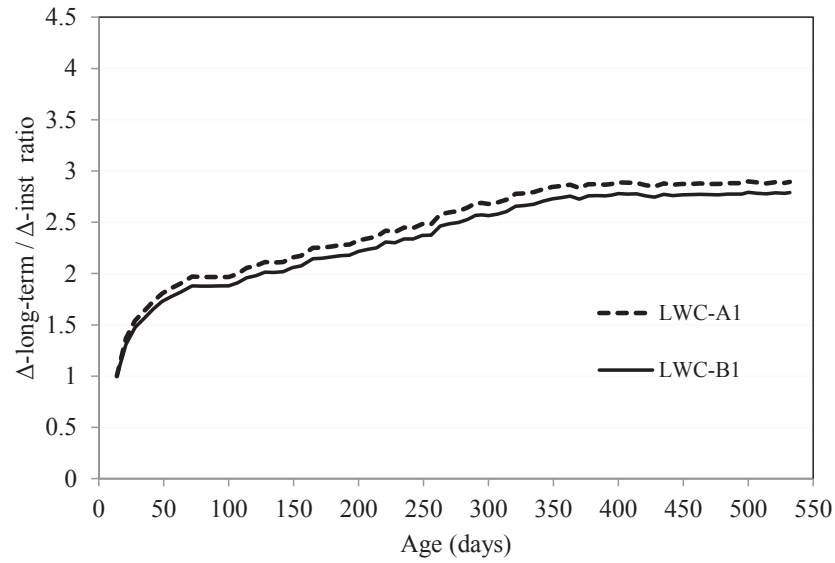


Figure 9.13: Ratio of long-term to instantaneous deflection in LWC slabs under LL_2 ($0.4 M_u$)

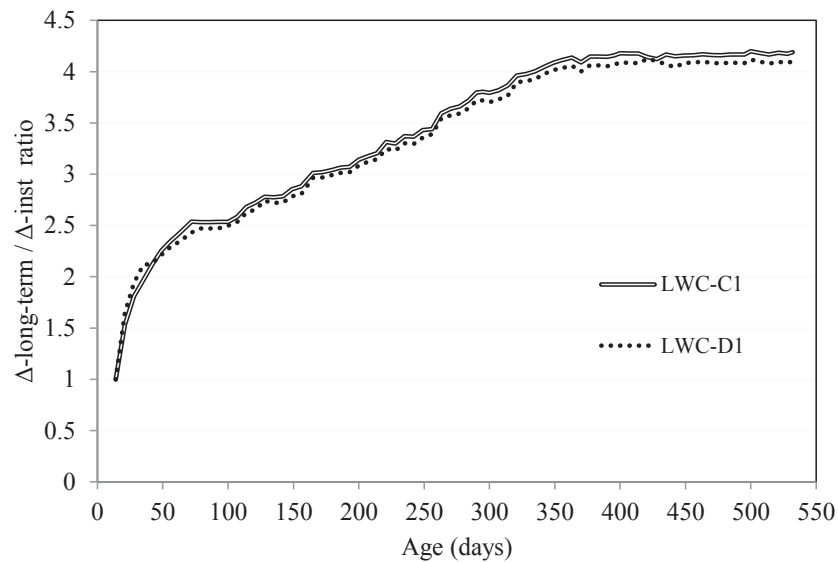


Figure 9.14: Ratio of long-term to instantaneous deflection in LWC slabs under LL_1 ($0.3 M_u$)

The ratio of long-term to instantaneous deflection of LWC slabs changes by time. The ratio changes from 1 in the first loading time to 2.89 after 525 days sustained loading in the slabs subjected to LL_2 . For the slabs subjected to LL_1 , this ratio changes from 1 at the loading day to 4.19 after 525 sustained d loading.

The instantaneous and final deflection after 525 days in both monitoring points of the slabs LWC-A and LWC-B are greater the corresponding deflections in LWC-C and LWC-D. Despite the greater values of the instantaneous deflection in the slabs under LL_2 , the relative increment of the long-term deflection in the slabs subjected to lower sustained loading is considerably

high. Considering the constant value of the sustained load of slabs in the monitoring period, it shows the significant effect of the creep and shrinkage on the long-term deflection of LWC slabs.

9.4. RATE OF TIME DEPENDENT DEFLECTION IN LIGHTWEIGHT CONCRETE SLABS

Not only the total deflection after a long time is a crucial parameter in design of the reinforced concrete structures, but also the rate of deflection increment with time is important. A relatively rapid deflection rate limits the service life and consequently applies more conservative design considerations. However, in a concrete structure with slow increment of the time-dependent deflection, service life is not significantly affected by the deflection rate.

Figures (9.15) and (9.16) show the rate of time-dependent deflection of LWC slabs. In Figure (8.15), the rate of deflection increment in the slabs LWC-C and LWC-D at the mid-span is presented. Both slabs are subjected to the lower loading level, LL_1 . Time-dependent deflection of these slabs is sharply growing after the first loading until about 50 days. The deflection rate is changing from 8mm per day in the first days to less than 1 mm per day after 50 days. This rate of time-dependent deflections tends to a constant rate at about after 150 days until 525 days. Despite the tendency of deflection rate to zero after 525, the time-dependent deflection never stops in the lightweight concrete slabs.

In Figure (9.16), the rate of deflection increment in the slabs LWC-A and LWC-B at the mid-span is presented. Both slabs are subjected to the higher loading level, LL_2 . Time-dependent deflection of these slabs is abruptly increasing after the first loading until about 50 days. The deflection rate is changing from 8mm per day in the first days to less than 1 mm per day after 50 days. This rate of time-dependent deflections tends to a constant rate at about after 150 days until 525 days. Despite the tendency of deflection rate to zero after 525, the time-dependent deflection continues even by very low rate.

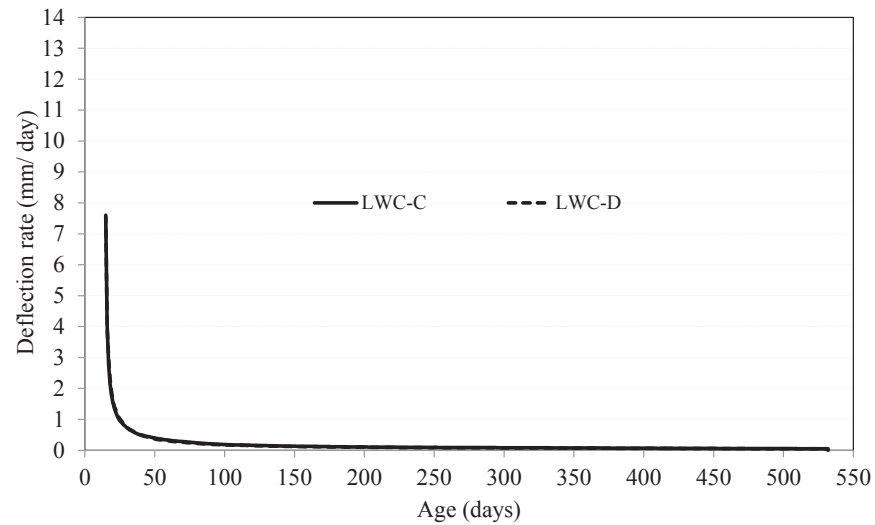


Figure 9.15: Deflection rate of slabs C and D under LL_1 ($0.3M_u$)

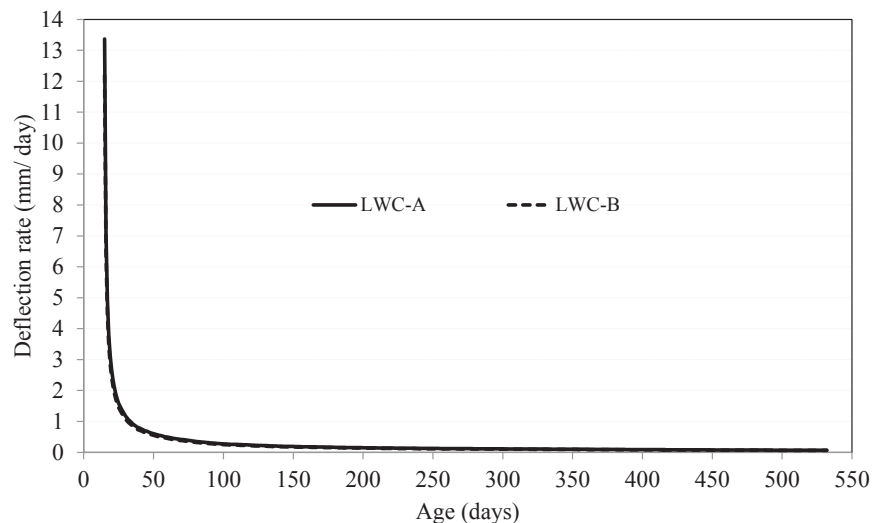


Figure 9.16: Deflection rate of slabs A and B under LL_2 ($0.4M_u$)

9.5. CRACKING HISTORY OF THE LIGHTWEIGHT CONCRETE SLABS AT EARLY AGES OF LOADING

According to Figures (9.15) and (9.16), the highest rate of the time-dependent deflection in the monitored LWC slabs happened before the age of 50 days of loading. In other words, majority of the time-dependent deflection of slabs under two different loading levels happened before 50 days of sustained loading.

The load increment changes the cracking pattern in terms of the crack width and crack spacing. The more cracks, the higher the effects of creep and shrinkage on the time-dependent deflection of slabs and vice versa. Figures (9.17) to (9.20) show the cracking distribution of

LWC slabs at two ages, one week after loading and 6 weeks (about 50 days) after loading, respectively.

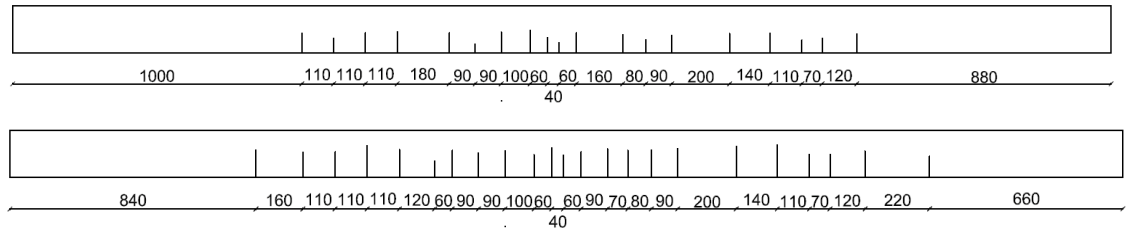


Figure 9.17: Cracking pattern of slab LWC-A at 7 and 48 days after sustained loading

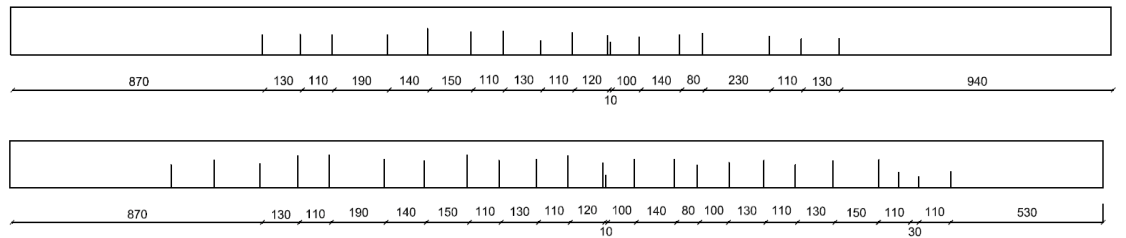


Figure 9.18: Cracking pattern of slab LWC-B at 7 and 48 days after sustained loading

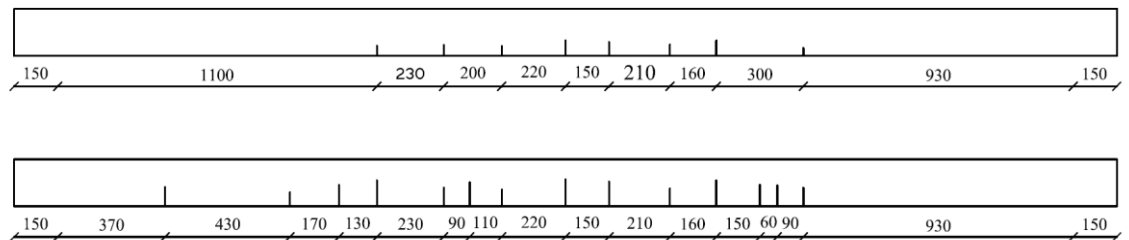


Figure 9.19: Cracking history of slab C at 7 and 48 days after sustained loading

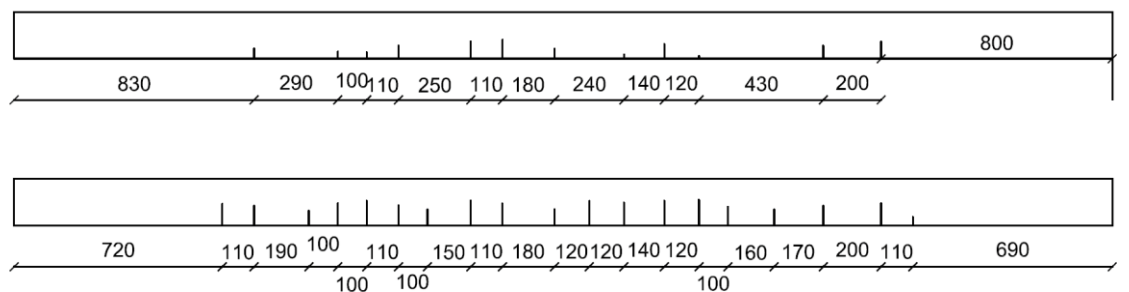


Figure 9.20: Cracking history of slab D at 7 and 48 days after sustained loading

9.6. SIMILAR STUDIES ON THE CONVENTIONAL AND SELF-COMPACTING CONCRETE SLABS

This study compares the experimental results of the instantaneous and time-dependent deflection of three different types of concrete slabs. The experimental program consists of six one-way slabs including three pairs of SCC slabs conducted by Aslani (2014) [21], LWC slabs in this research program, and CC slabs conducted by Nejadi (2005) [22] with different mixture designs.

The slabs were simply supported with identical span length, section dimensions and reinforcement arrangement. The standard size of the cylindrical specimen (150×300 mm) was used to measure the compressive strength, modulus of elasticity and creep coefficient of the mixtures. Also, the shrinkage strain was measured by the standard prisms in laboratory controlled conditions. The test specimens were kept covered in a controlled chamber at $23 \pm 2^\circ\text{C}$ for 24 hours until demoulding. The relative humidity of the environment was maintained at $50 \pm 5\%$ during the tests. Then, the specimens were placed in water pre-saturated with lime at 20°C . The compressive strength test was performed according to AS-1012.14 (1991) [23] and ASTM-C39 (2000) [24] instructions, and the loading rate of the machine on the cylinders was 0.3 MPa/s until failure. The modulus of elasticity test was performed according to the instructions in AS-1012.17 (2014) [25] and ASTM-C469 (2000) [26].

9.6.1. Conventional Concrete Slabs

Figure (9.21) shows the experiment conducted by Nejadi (2005) on the conventional concrete slabs. The time-dependent deflection of slabs at mid-span was monitored for 400 days in the laboratory conditions.

One batch of commercially pre-mixed concrete was used to manufacture all the test and companion specimens. Two sustained load levels were considered, being 30% and 20% of the ultimate design load, and designated load conditions 'a' and 'b', respectively. The applied load consisted of self-weight plus superimposed sustained loads at the age of 14 days via carefully constructed and arranged concrete blocks supported at the top (of the specimens).



Figure 9.21: The loading of convention concrete slabs for time-dependent deflection monitoring

The summary of the long-term deflection results for the CC slabs is shown in Figure (9.22).

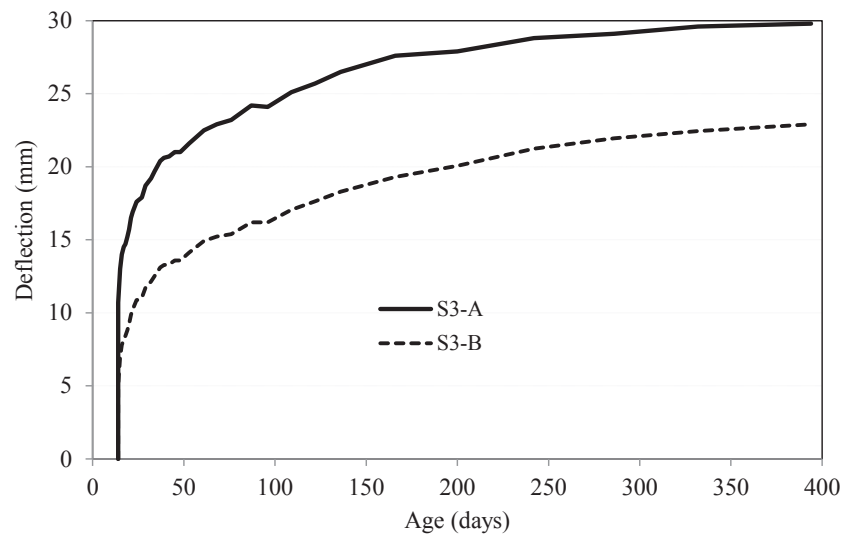


Figure 9.22: Time-dependent deflection of slabs at mid-span (Nejadi, 2005)

9.6.2. Self-Compacting Concrete Slabs

In an experiment by Aslani (2014), a total of eight SCC and Fibre Reinforced Self-Compacting Concrete (FRSCC) slabs with the same cross-section and details as for the short-

term tests were monitored for up to 240 days to measure the time-dependent development of cracking and deformations under service loads. Deflections at mid-span, crack widths, crack patterns, steel strains within the high moment region, and concrete surface strains at the steel level were recorded under the sustained service load and effects of creep and shrinkage. For this purpose, four SCC mixes – two plain SCC, two steel, two polypropylene, and two hybrid FRSCC slab specimens – are considered in the test program.

Two sustained load levels were considered, being 40% and 30% of the ultimate design load, and designated load conditions ‘a’ and ‘b’, respectively. Figure (9.23) shows the general view of the test arrangement for time-dependent deflection monitoring of self-compacting concrete slabs.



Figure 9.23: The loading of Self-compacting concrete slabs for time-dependent deflection monitoring (Aslani, 2014)

Summary of the long-term deflection results for SCC and FRSCC slabs under higher and lower loading levels are shown in Figures (9.24) and (9.25) respectively.

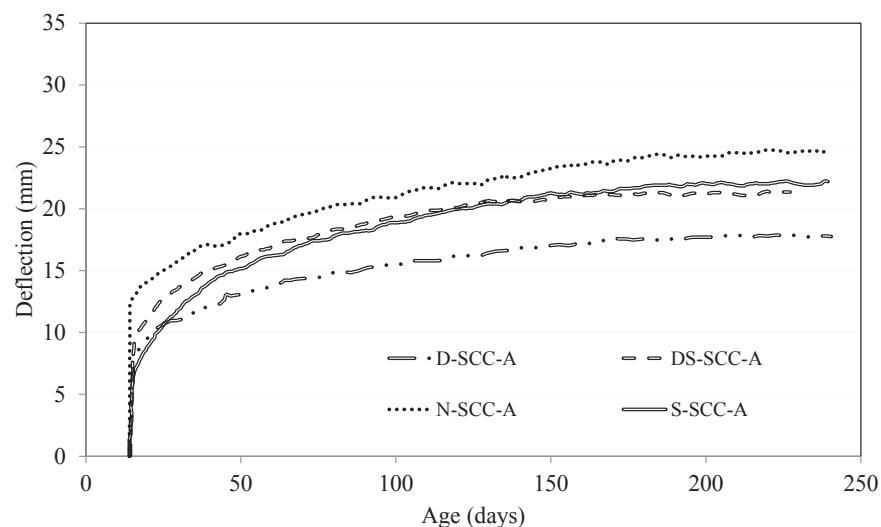


Figure 9.24: Time-dependent deflection of SCC and FRSCC slabs under 0.4Mu loading (Aslani, 2014)

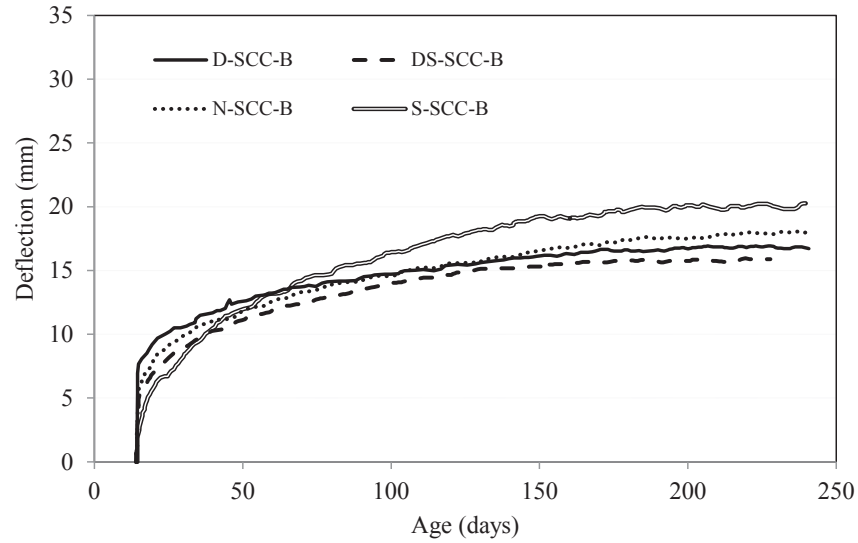


Figure 9.25: Time-dependent deflection of SCC and FRSCC slabs under $0.3M_u$ loading (Aslani, 2014)

9.7. COMPARISON OF TIME-DEPENDENT DEFLECTION IN LIGHTWEIGHT, SELF-COMPACTING AND CONVENTIONAL CONCRETE SLABS

Table (9.3) summarizes the ultimate moment capacity (M_u), applied service moment at the critical section (M_a), predicted elastic deflection (Δ_e), and the recorded instantaneous deflection (Δ_{ins}) immediately after the load application on the top surface of simply supported slabs.

The monitoring period of long-term deflection for LWC, SCC and LWC slabs are 550, 250 and 400 days respectively. Therefore, the long-term deflection of the LWC, SCC and CC slabs are compared for 250 days. However, the LWC and CC slabs are compared for 400 days.

Figure (9.26) compares the mid-span deflection versus time in the CC, LWC and SCC slabs under the higher loading level. As expected from the measured mechanical properties (see chapter 5), the CC slab shows the leading ratio of the total to instantaneous deflection, while the SCC slab is placed in the lower limit of the diagram. The variation of the ratio in LWC and CC slab is similar up to 60 days. Then, the deflection of the CC slab increases with a higher rate until the end of the monitoring period. The time-deflection diagram in the SCC slab follows a relatively constant increasing rate from the first loading at the age of 14 days.

Table 9.3: Loading level, predicted elastic deflection and recorded instantaneous deflection of slabs

Slab	W_a (KN/m)	M_u (KN.m)		M_a (KN.m)	M_a / M_u (%)		Δ_e (mm)		Δ_{ins} (mm)
		14 day	28 day		14 day	28 day	14 day	28 day	
N-SCC-a	7.31	27.47	27.78	11.189	40.73	40.27	3.18	2.9	12.1
N-SCC-b	5.41	27.47	27.78	8.28	30.14	29.8	2.36	2.15	5.89
D-SCC-a	7.26	27.84	28.05	11.12	39.94	39.65	3.5	2.85	7.65
D-SCC-b	5.36	27.84	28.05	8.21	29.48	29.27	2.58	2.11	7.59
S-SCC-a	7.3	27.73	28.05	11.18	40.32	39.85	3.45	2.87	6.41
S-SCC-b	5.4	27.73	28.05	8.27	29.83	29.48	2.55	2.12	2.91
DS-SCC-a	7.33	28.05	28.34	11.23	40.03	39.63	3.29	2.85	8.98
DS-SCC-b	5.43	28.05	28.34	8.32	29.66	29.36	2.44	2.11	5.14
LWC-1	7	27.4	27.65	10.72	39.13	38.78	4.99	4.57	11.96
LWC-2	7	27.4	27.65	10.72	39.13	38.78	4.99	4.57	11.4
LWC-3	5.29	27.4	27.65	8.095	29.55	29.28	3.77	3.45	6.3
LWC-4	5.29	27.4	27.65	8.095	29.55	29.28	3.77	3.45	5.98
CC-a	4.44	26.03	27.05	6.8	26.12	25.14	2.73	2.5	11.8
CC-b	3.46	26.03	27.05	5.3	20.36	19.59	2.13	1.95	5.04

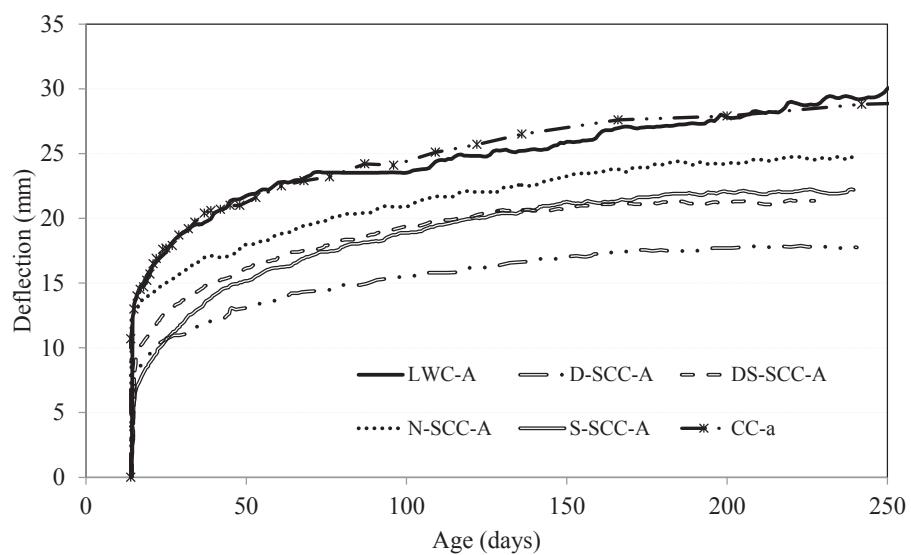


Figure 9.26: Comparison of deflection vs. time in SCC, LWC and CC slabs under higher loading level

Figure (9.27) also compares the mid-span deflection versus time in the CC, LWC and SCC slabs under the lower loading level.

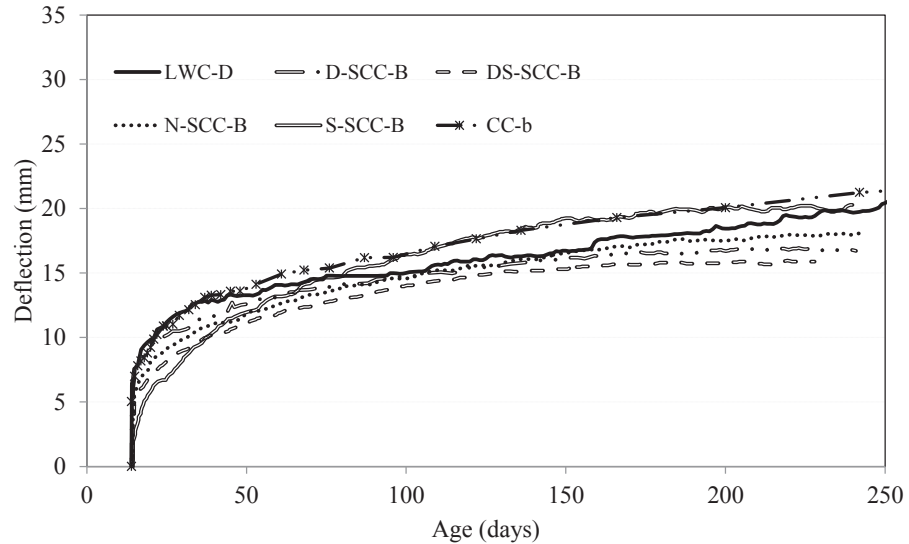


Figure 9.27: Comparison of deflection vs. time in SCC, LWC and CC slabs under lower loading level

To evaluate the total and time-dependent deflection over a longer period, the variation of mid-span deflection versus time in LWC and CC slabs under higher loading level is given in Figure (9.28) for the sustained loading period of 400 days.

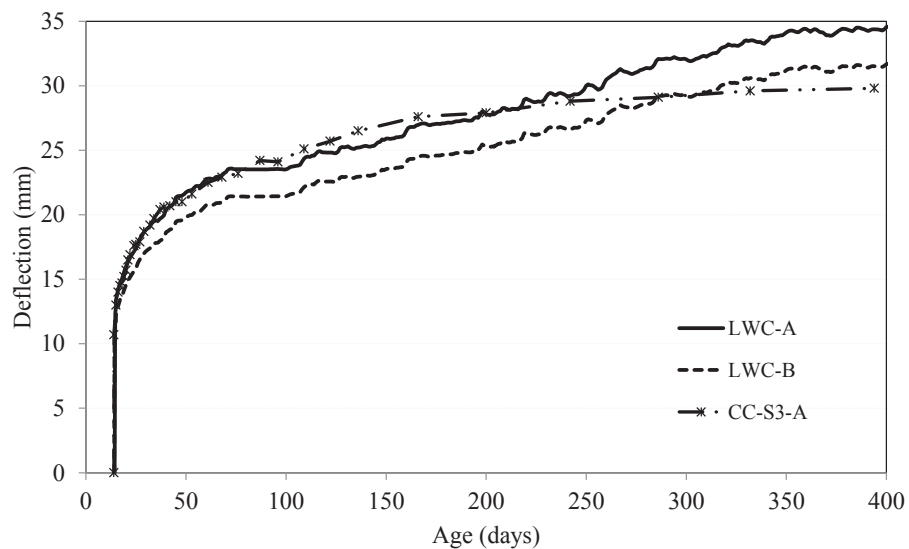


Figure 9.28: Deflection vs. time in LWC and CC slabs under higher loading level

The time-dependent deflection increases at a decreasing rate, with much of the long-term deflection taking place in the first 100 days after first loading in all slabs of different concrete types. However, the rate of deflection increase before and after that time is not entirely similar.

9.8. TIME-DEPENDENT DEFLECTION IN CODES OF PRACTICE

Design codes recommend simplified methods of calculating the time-dependent and long-term deflection of concrete structures. Despite the uncertainties, it is necessary for the calculated deflection to match the real deflection precisely. In some universally applied codes of practice, the long-term deflection is predicted as a multiple of the instantaneous or short-term deflection. To calculate the long-term deflection, ACI318-08 [27] and CSA.A23.04[20] recommend a simplified procedure by multiplying the instantaneous deflection by a multiplier. AS3600-09[19] uses a similar procedure with some modifications to calculate the time-dependent deflection. Table (8.4) presents the equations in the above mentioned codes to calculate the long-term deflection from instantaneous deflection.

Table 9.4: Calculation of long-term deflection in codes of practice

Code	Total deflection	Coefficients
ACI-318-08[27]	$\Delta_{ins}(1 + \frac{\delta}{1 + 50 \rho'})$	δ : 1.2, 1.25, 1.4 for 180, 225, 365 days
CSA.A23-04[20]	$\Delta_{ins}(1 + \frac{\xi}{1 + 50 \rho'})$	ξ : 1.2, 1.25, 1.4 for 180, 225, 365 days
AS-3600-09[19]	$\Delta_{ins} + k_{cs}\Delta_{sus}$	k_{cs} : creep and shrinkage coefficient Δ_{sus} : time-dependent deflection under sustained load

Figures (9.29 a, b) show the predictions of AS-3600 (2009)[19] , ACI-318 (2008)[27] and CSA.A23 (2004)[20] for the instantaneous and total (long-term) deflection of SCC, LWC and CC slabs under two different levels of sustained loading respectively.

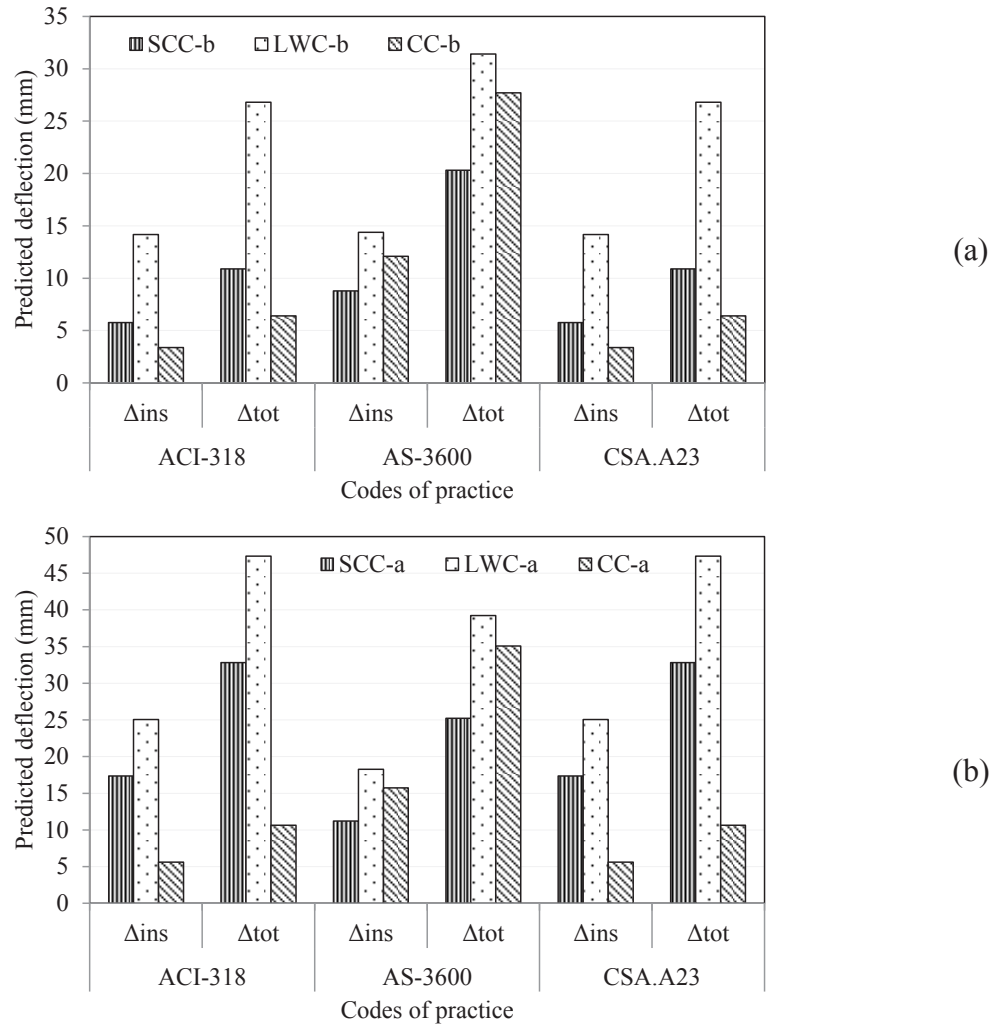


Figure 9.29: Predicted Δ_{ins} and Δ_{tot} of SCC, LWC and CC slabs under a) LL₁, b) LL₂

According to Figures (8.29 a, b), there is a considerable difference between the predicted Δ_{ins-14} and Δ_{long} of slabs under different load values. The instantaneous and total deflection in the slabs under higher loading level is significantly greater than those of the slabs under lower loading levels, especially by ACI-318[27] and CSA.A23[20] predictions. However, in predicted values of AS-3600[19], the difference between the Δ_{ins-14} and Δ_{long} is less, compared to the other design codes.

All listed codes in Figures (9.29 a, b) predict the highest values of Δ_{ins-14} and Δ_{long} for LWC slabs under both loading levels, however, AS-3600[19] predicts the lowest Δ_{ins-14} and Δ_{long} values for SCC slabs under both loading levels.

9.9. RATIO OF LONG-TERM TO INSTANTANEOUS DEFLECTION IN LIGHTWEIGHT, SELF-COMPACTING AND CONVENTIONAL CONCRETE SLABS

Since the total monitoring period for three different types of concrete slabs are not the same, the individual recording of each concrete type is presented. Then the comparative records for the compatible duration are illustrated. The ratio of long-term deflection of CC slabs to the short-term deflection under sustained loading in different ages is presented in Table (9.5) respectively.

Table 9.5: Long-term to short-term deflection ratio in slab series tested by Nejadi and Gilbert (2005)

Age (day)	14	21	28	56	60	95	122	200	285	330	394
Slab S3-a	1	1.54	1.67	1.96	2.1	2.25	2.4	2.61	2.72	2.77	2.79
Slab S3-b	1	1.96	2.19	2.69	2.96	3.21	3.5	3.98	4.35	4.45	4.54

The ratio of long-term deflection of SCC and FRSCC slabs to the short-term deflection under sustained loading in different ages is presented in Table (9.6) respectively.

Table 9.6: Long-term to short-term deflection ratio in SCC and FRSCC slabs

Slab	Age (days)								
	14	21	28	45	60	95	122	200	240
N-SCC-a	1	1.18	1.28	1.42	1.55	1.73	1.72	2	2.04
N-SCC-b	1	1.42	1.63	1.91	2.14	2.48	2.65	2.98	3.07
D-SCC-a	1	1.31	1.43	1.71	1.82	2.03	2.13	2.31	2.32
D-SCC-b	1	1.28	1.4	1.67	1.75	1.94	2.03	2.2	2.21
S-SCC-a	1	1.44	1.79	2.31	2.53	2.91	3.12	3.44	3.47
S-SCC-b	1	2.15	2.66	3.94	4.53	5.52	6.06	6.91	9.65
DS-SCC-a	1	1.3	1.48	1.72	1.89	2.13	2.26	2.36	2.37
DS-SCC-b	1	1.42	1.7	2.06	2.31	2.67	2.88	3.06	3.08

The comparative presentation of the deflection ratios in Tables (9.2), (9.5) and (9.6) in 225 days duration is presented in Table (9.7).

Table 9.7: Ratio of long-term to instantaneous deflection in LWC, CC and SCC slabs

Slab	Age (days)										
	14	28	49	72	100	121	149	172	200	214	225
N-SCC-A	1	1.27	1.48	1.62	1.73	1.82	1.92	1.97	2.01	2.02	2.03
N-SCC-B	1	1.61	1.96	2.26	2.47	2.64	2.8	2.9	2.98	3.01	3.02
D-SCC-A	1	1.43	1.71	1.88	2.02	2.12	2.13	2.18	2.2	2.21	2.22
D-SCC-B	1	1.38	1.65	1.82	1.94	2.01	2.12	2.17	2.2	2.22	2.23
S-SCC-A	1	1.76	2.34	2.69	2.94	3.12	3.31	3.34	3.43	3.44	3.45
S-SCC-B	1	2.59	4.08	4.88	5.64	6.11	6.59	6.67	6.86	6.87	6.89
DS-SCC-A	1	1.47	1.78	1.97	2.15	2.24	2.3	2.35	2.36	2.37	2.38
DS-SCC-B	1	1.68	2.15	2.43	2.72	1.82	2.98	3.04	3.07	3.08	3.09

Figure (9.30) compares the mid-span deflection versus time in the CC, LWC and SCC slabs under higher loading level.

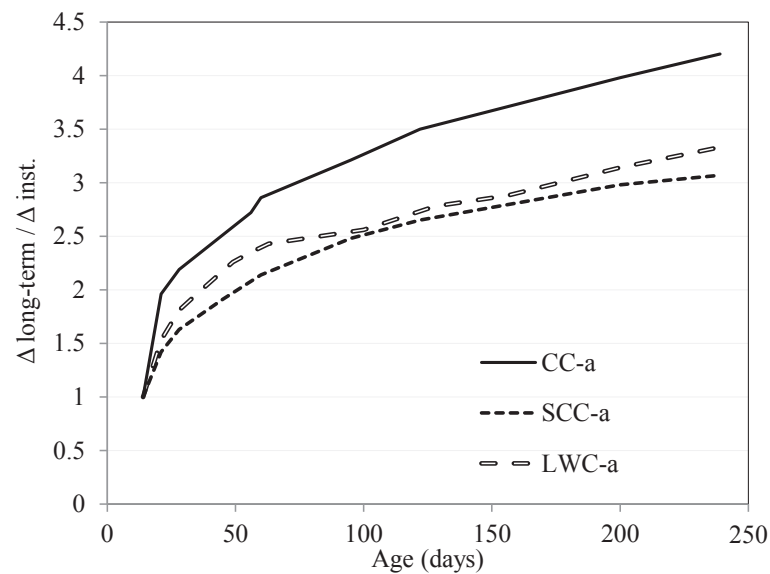


Figure 9.30: The ratio of long-term to instantaneous deflection under higher loading level

Figure (9.31) also compares the mid-span deflection versus time in the CC, LWC and SCC slabs under lower loading level. Similar to the Figure (9.30), the CC slab poses the largest ratio of the long-term to instantaneous deflection, while the SCC slab experiences the lowest ratio over all the testing period.

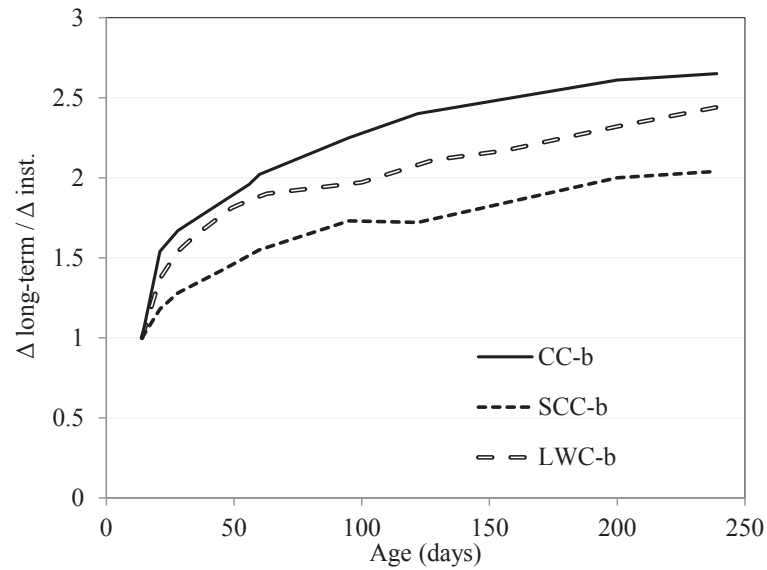


Figure 9.31: The ratio of long-term to instantaneous deflection under lower loading level

The ratio of the long-term to instantaneous deflection is the result of the combined effects of the creep and shrinkage on the member. However, only the time-dependent deflection due to creep may be correctly expressed as a multiple of the instantaneous deflection. The shrinkage induced deflection is dependent on the free shrinkage that is independent of the instantaneous deflection [28]. Table (9.8) shows the free shrinkage strain of the CC, LWC and SCC specimens at 240 days after casting. The creep coefficients of the companion specimens of the concrete slabs since the loading age are presented in Table (9.8) also. The creep coefficients are measures in the cylinder specimens under the corresponding sustained compression stress at slab specimens.

Table 9.8: Free shrinkage strain and creep coefficient of CC, LWC and SCC mixtures

	CC	LWC	SCC
Shrinkage strain ($\mu\epsilon$)	784	1180	870
Creep coefficient	1.5	2.415	1.96

According to the Table (8.8), the shrinkage strain in LWC is considerably higher than that of the CC and SCC mixtures. The free shrinkage strain in LWC and SCC mixtures is 51% and 10% greater than that of the CC mixture respectively. However, the ratio of long-term to instantaneous deflection in LWC and SCC slabs is 90% and 75% of the corresponding deflection ratio in CC slab under lower loading levels respectively. In the slabs subjected to

higher loading levels, the deflection ratio in LWC and SCC slabs reaches to 79% and 73% of the corresponding deflection ratio in CC slab respectively.

Comparing the deflection ratios in Figures (9.30), (9.31) and the free shrinkage values in Table (9.8), the ratio of long-term to instantaneous deflection cannot be concluded from the free shrinkage trend over the time period. Also, the rate of deflection and the free shrinkage rate in different types of concrete are not compatible.

The creep coefficient in LWC and SCC mixtures is 1.61 and 1.3 times higher than that of the CC mixture respectively. Although the ratio of long-term to instantaneous deflection is frequently used as the deflection multiplier in the codes of practice, the diagrams and data presented in Figures (9.30), (9.31) and Table (9.8) are not thorough enough to establish any relationship. In other words, the creep coefficient is not a good indicator of the time-dependent deflection increment in different types of concrete, without another mechanism of assessment. The loading value, type of concrete and mixture components, loading age, environmental effects and testing equipment strongly affect the time-dependent and long-term deflection of slabs.

The long-term deflection after 240 days in LWC slabs is 0.99 and 0.92 of the corresponding deflection in CC slabs under the lower and higher loading levels respectively. Also, the deflection of SCC slabs under the lower and upper loading levels is 0.85 and 0.78 of the deflection under corresponding loading levels in CC slabs.

Figure (9.32) compares the predicted and experimental ratio of $\Delta_{long} / \Delta_{ins-14}$ of slabs under two loading levels. The experimental ratio ranges from 2.04 to 4.35, however, it changes in the range of 1.89 to 2.31 in the predictions of codes of practice.

According to Figure (8.32), AS-3600[19] gives the lowest ratio of $\Delta_{long} / \Delta_{ins-14}$ of all slabs. Except for the SCC-b slab, the experimental ratio in all slabs is higher than the predicted values of design codes. The difference between the experimental and predicted ratio of the deflection in slabs increases by increasing the loading value.

The CC slabs, particularly CC-a, show the highest difference between the experimental and predicted deflection ratio. In other words, the codes of practice give the farthest prediction of the total to instantaneous deflection ratio in CC slabs under both loading levels.

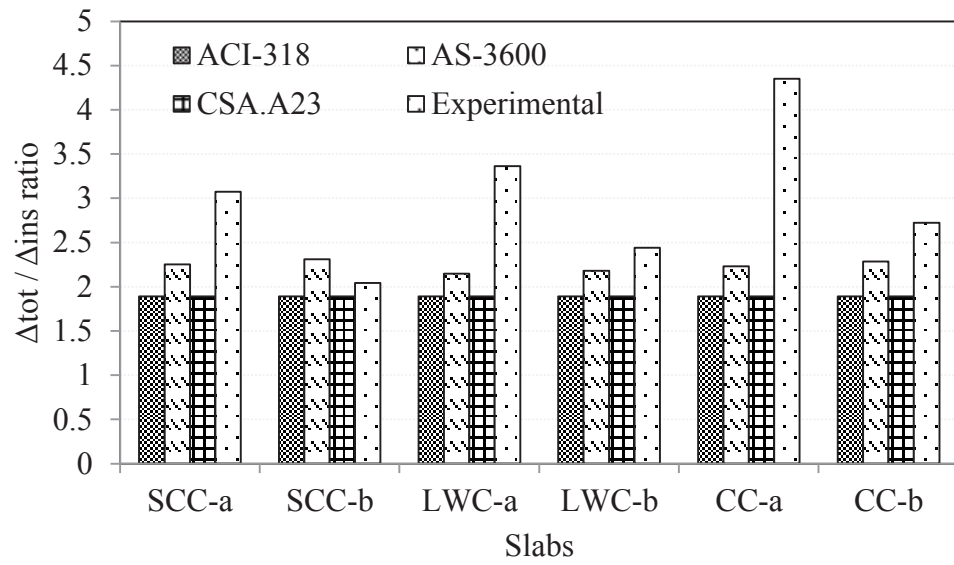


Figure 9.32: Comparing the experimental and predicted ratio of the total to instantaneous deflection

Figures (9.33) and (9.34) compare the deflection ratio in SCC and FRSCC slabs under two different loading levels.

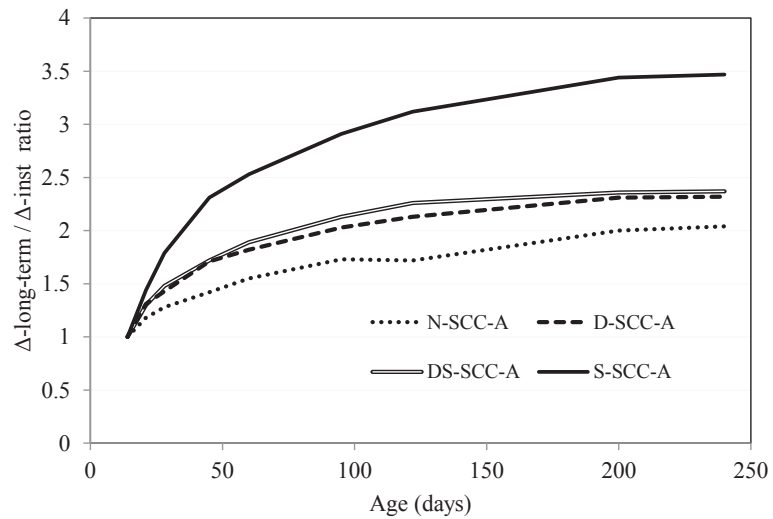


Figure 9.33: The ratio of long-term to instantaneous deflection of SCC and FRSCC slabs (higher loading level)

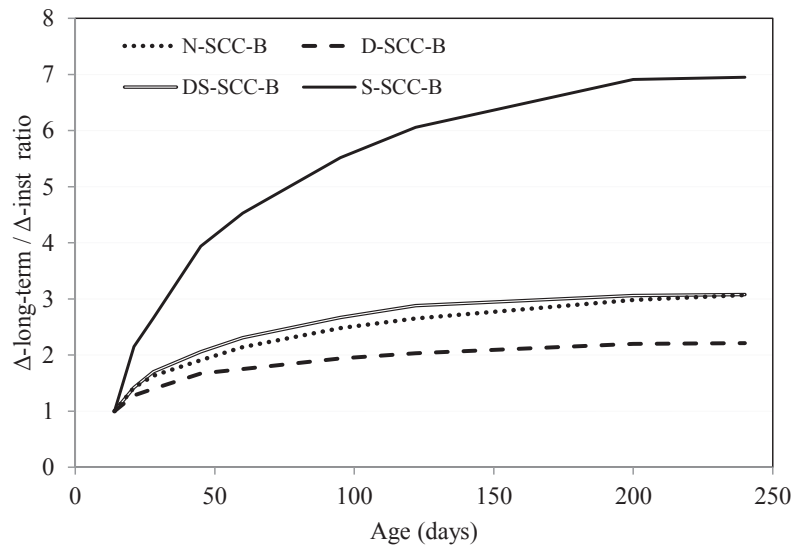


Figure 9.34: The ratio of long-term to instantaneous deflection of SCC and FRSCC slabs (lower loading level)

Figure (9.35) compares the LWC, SCC and CC slabs in their real monitored period under the higher loading level.

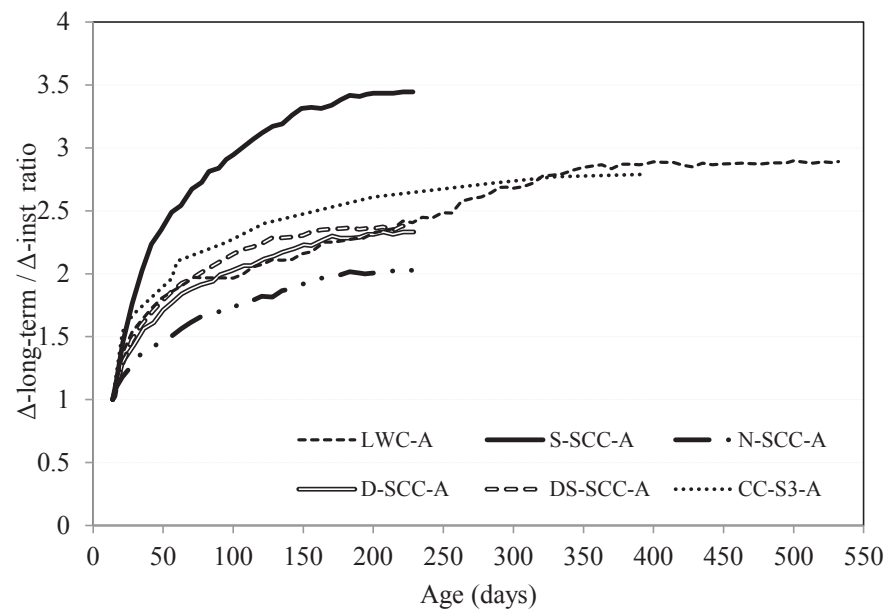


Figure 9.35: The ratio of long-term to instantaneous deflection of all slabs (higher loading level)

9.10. DEFLECTION RATE IN LIGHTWEIGHT, SELF-COMPACTING AND CONVENTIONAL CONCRETE SLABS

Figures (9.36) and (9.37) compare the rate of deflection in the LWC, SCC, FRSCC and CC slabs under two different loading levels respectively.

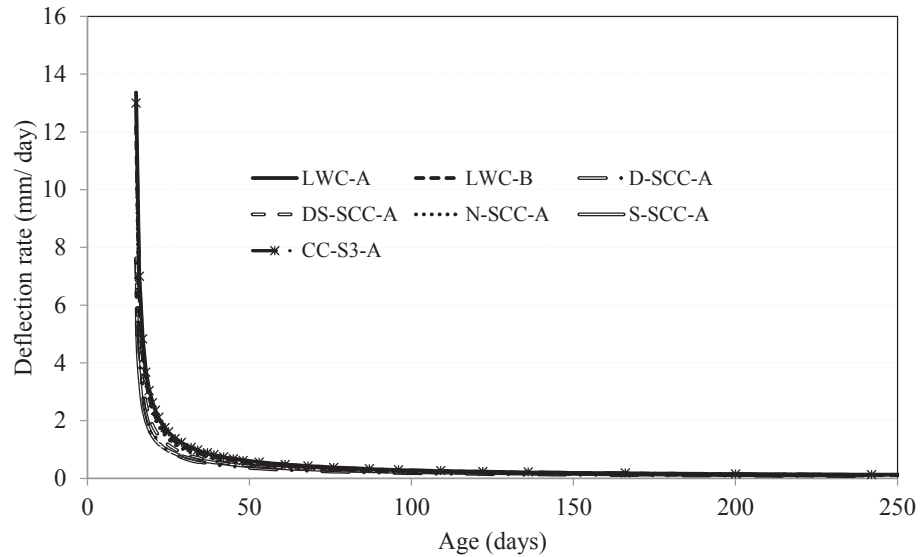


Figure 9.36: Deflection rate of all slabs under higher loading level

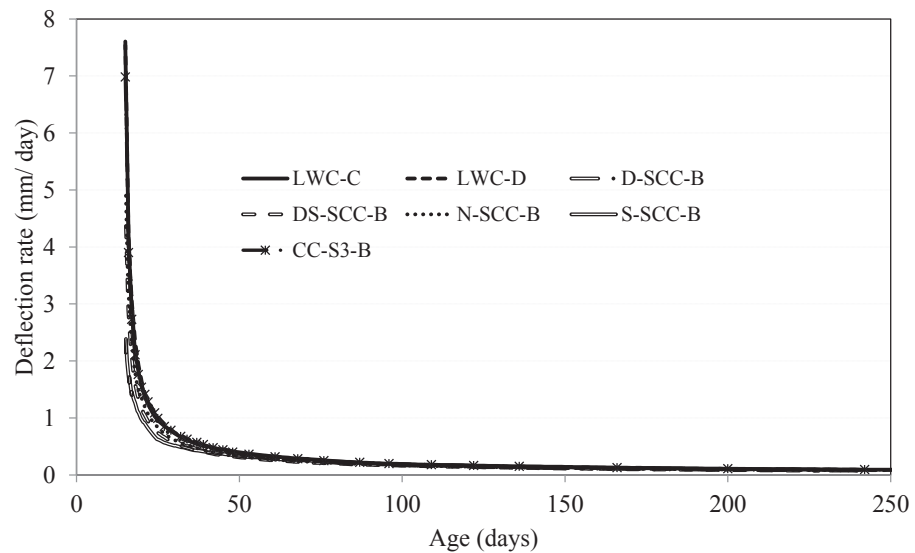


Figure 9.37: Deflection rate of all slabs under lower loading level

9.11. SUMMARY

Time-dependent deflection of lightweight concrete slabs under two different levels of sustained service loads in the experimental program is described in details. The obtained results are analysed and discussed broadly and compared with the previously performed experimental results by Nejadi (2005) and Aslani (2014).

Figure (9.38) shows the section details, span length and support conditions of the lightweight concrete slabs subjected to the sustained loading for 550 days. The slabs were loaded at age 14 days after casting. Deflection of the slabs was measured at two points; the mid-span and the point on border of the high moment region.

The loadings equal to 30% and 40% of the ultimate bending moment are considered as LL_1 and LL_2 in this chapter respectively. Table (9.9) presents the detailed information of the loading level, service and ultimate moments of the LWC slabs.

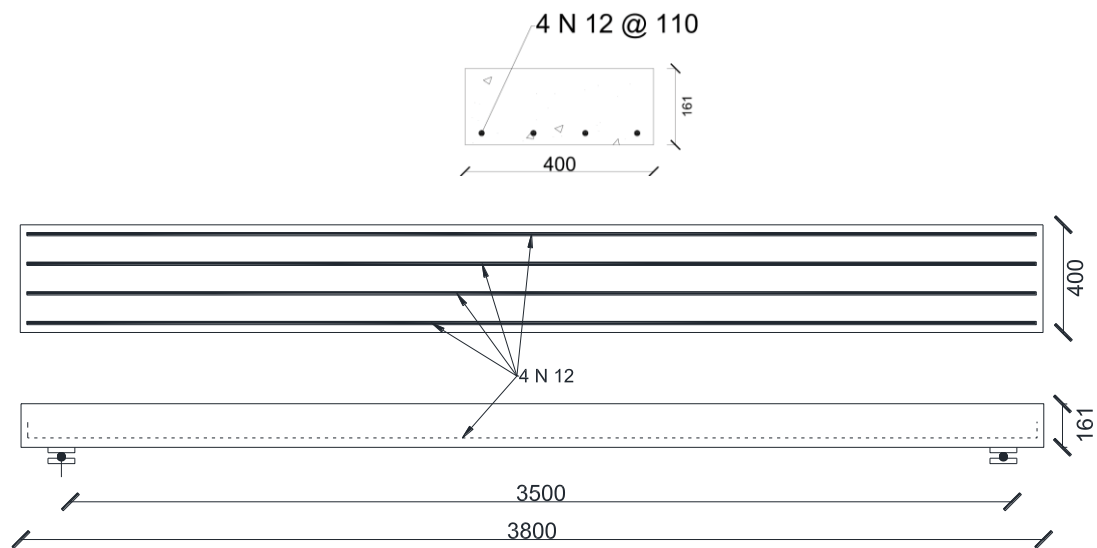


Figure 9.38: Cross section and reinforcing detail of LWC slab specimens in long-term deflection test

Table 9.9: loading level and bending capacity of the LWC slabs in the long-term test

Slab	W_a (KN/m)	M_a (KN.m)	M_u (KN.m)		M_a / M_u (%)	
			14 day	28 day	14 day	28 day
LWC-A	7	10.72	27.4	27.65	39.13	38.78
LWC-B	7	10.72	27.4	27.65	39.13	38.78
LWC-C	5.29	8.095	27.4	27.65	29.55	29.28
LWC-D	5.29	8.095	27.4	27.65	29.55	29.28

Figure (9.39) compares the corresponding time-dependent deflection of slabs at mid-span subjected to two distinct loading levels of LL_1 and LL_2 . Figure (9.40) compares the corresponding time-dependent deflection of slabs at the point on the border of high moment region of slabs under two different loading levels. The diagrams in Figures (9.39) and (9.40) consist of the initial deflection of slab at the time of loading (instantaneous deflection)

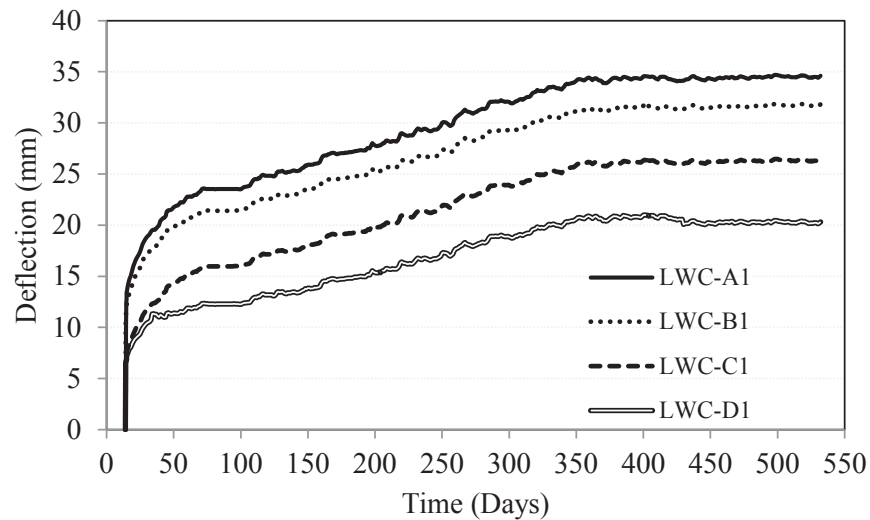


Figure 9.39: Time-dependent deflection of LWC-A and LWC-B slabs under LL_2 and LWC-C and LWC-D under LL_1 at mid-span

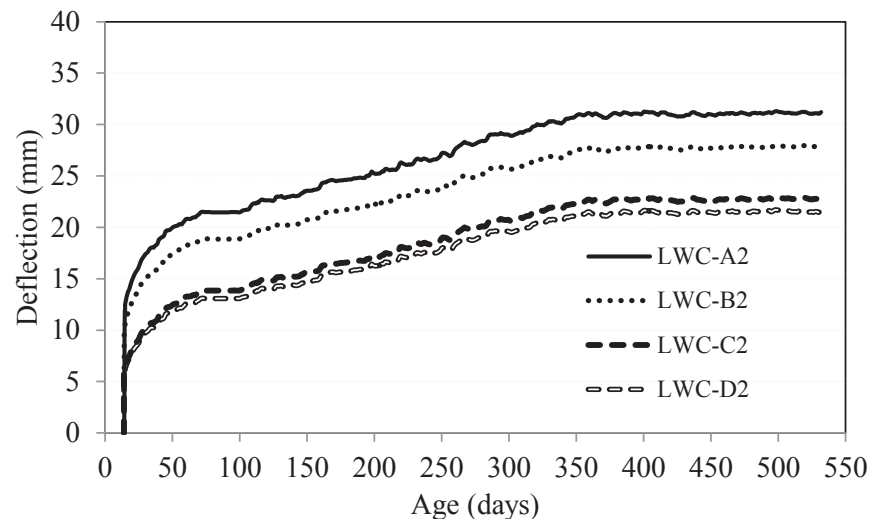


Figure 9.40: Time-dependent deflection of LWC-A and LWC-B slabs under LL_2 and LWC-C and LWC-D under LL_1 at the border of high moment region

The general trend of the time-dependent deflection of slabs at two monitoring points during the monitoring time is similar. However, the ratio of deflection at mid-span to the deflection at the border point of high moment region is considerably difference in the slabs.

The long-term deflection is considered as multiplier of the instantaneous deflection in majority of the design codes. Table (9.10) shows the ratio of long-term to instantaneous deflection in the LWC slabs under two different loading levels.

Table 9.10: Ratio of total to instantaneous deflection of LWC slabs at different ages

Slab	Age (days)															
	14	21	28	56	63	100	128	156	200	249	300	349	400	449	500	532
LWC-A	1	1.37	1.54	1.86	1.90	1.97	2.11	2.17	2.32	2.48	2.68	2.84	2.89	2.87	2.90	2.89
LWC-B	1	1.31	1.47	1.78	1.82	1.88	2.01	2.07	2.22	2.37	2.57	2.73	2.78	2.77	2.79	2.79
LWC-C	1	1.53	1.81	2.35	2.43	2.53	2.78	2.88	3.14	3.43	3.79	4.09	4.18	4.16	4.20	4.19
LWC-D	1	1.65	1.94	2.29	2.34	2.50	2.74	2.80	3.08	3.36	3.70	4.02	4.09	4.08	4.12	4.10

Regardless of the loading level, the deflection multiplier in design codes varies in the range of 2 to 3. According to Table (9.10), this ratio is in agreement with the long-term deflection of slabs at lower loading level. By 10% increase in the loading level, the deflection multiplier increase about 50% above the range of design codes.

- Comparing the results with previously conducted experiments on conventional and self-compacting slabs

Table (9.11) shows the measured deflections of lightweight, self-compacting and conventional concrete slabs at the time 0 after loading (instantaneous deflection) and the final deflection after long-term sustained loading. As presented in Table (9.11) the test duration is not equal in the slabs with different concrete types. The ratio of long-term to instantaneous deflection is a crucial parameter in design of the reinforced concrete flexural elements.

Figure (9.41) compares the mid-span deflection versus time in the CC, LWC and SCC slabs under the higher loading level. As expected from the measured mechanical properties (see Chapter 6), the CC slab shows the leading ratio of the total to instantaneous deflection, while the SCC slab is placed in the lower limit of the diagram. The variation of the ratio in LWC and CC slab is similar up to 60 days. Then, the deflection of the CC slab increases with a higher rate until the end of the monitoring period. The time-deflection diagram in the SCC slab follows a relatively constant increasing rate from the first loading at the age of 14 days.

Table 9.11: Ratio of the long-term to instantaneous deflection of LWC, SCC and CC slabs

Slab	M_a / M_u (%)		Test duration (days)	Δ_{tot} (mm)	Δ_{ins} (mm)	$\Delta_{tot} / \Delta_{ins}$
	14 day	28 day				
N-SCC-a	40.73	40.27	240	24.76	12.1	2.04
N-SCC-b	30.14	29.8	240	18.08	5.89	3.07
D-SCC-a	39.94	39.65	240	17.76	7.65	2.32
D-SCC-b	29.48	29.27	240	16.78	7.59	2.21
S-SCC-a	40.32	39.85	240	22.27	6.41	3.47
S-SCC-b	29.83	29.48	240	20.25	2.91	6.95
DS-SCC-a	40.03	39.63	240	21.31	8.98	2.37
DS-SCC-b	29.66	29.36	240	15.83	5.14	3.08
LWC-1	39.13	38.78	525	34.56	11.96	2.89
LWC-2	39.13	38.78	525	31.81	11.4	2.79
LWC-3	29.55	29.28	525	26.4	6.3	4.19
LWC-4	29.55	29.28	525	24.52	5.98	4.1
CC-a	26.12	25.14	400	32.1	11.8	2.72
CC-b	20.36	19.59	400	21.92	5.04	4.35

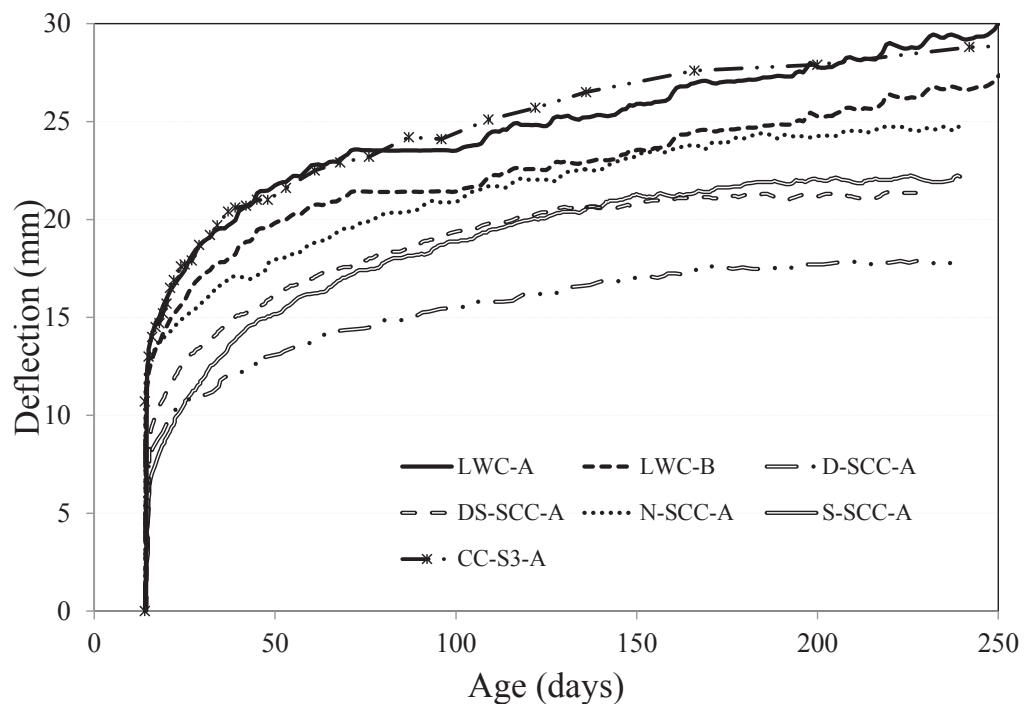


Figure 9.41: Comparison of deflection vs. time in SCC, LWC and CC slabs under higher loading level

Figure (9.42) also compares the mid-span deflection versus time in the CC, LWC and SCC slabs under the lower loading level.

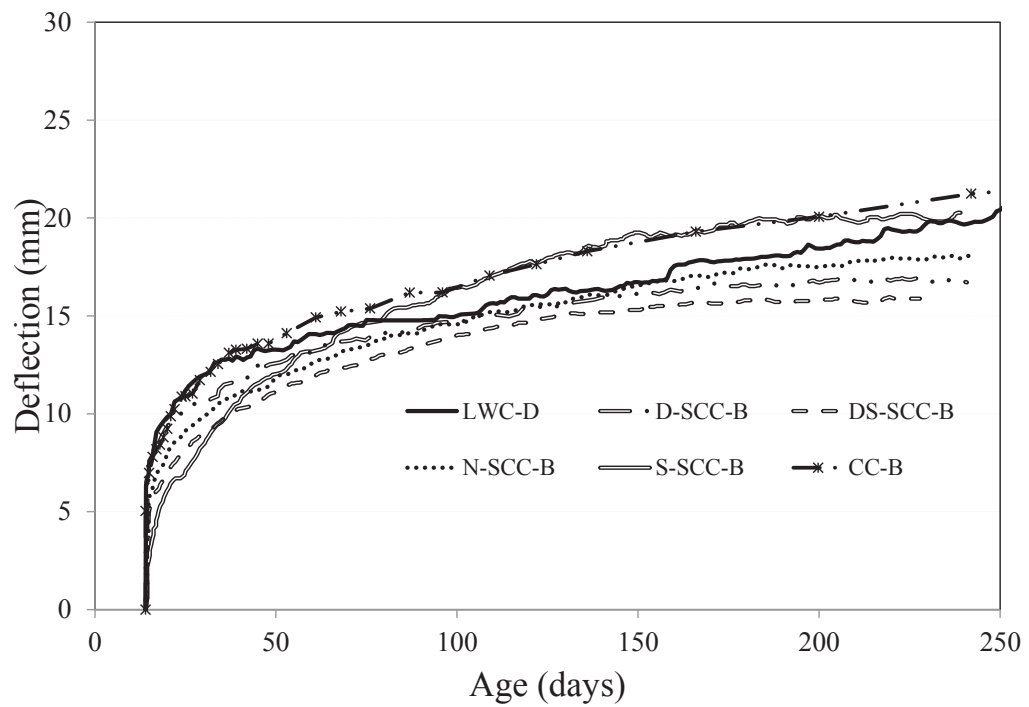


Figure 9.42: Comparison of deflection vs. time in SCC, LWC and CC slabs under lower loading level

Figure (9.43) compares the mid-span deflection versus time in the CC, LWC and SCC slabs under higher loading level.

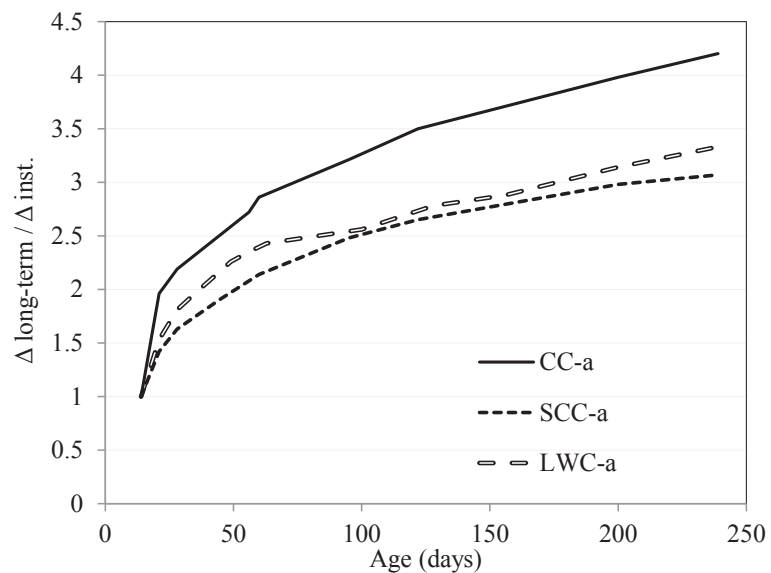


Figure 9.43 The ratio of long-term to instantaneous deflection under higher loading level

Figure (9.44) also compares the mid-span deflection versus time in the CC, LWC and SCC slabs under lower loading level. Similar to the Figure (9.43), the CC slab poses the largest ratio

of the long-term to instantaneous deflection, while the SCC slab experiences the lowest ratio over all the testing period.

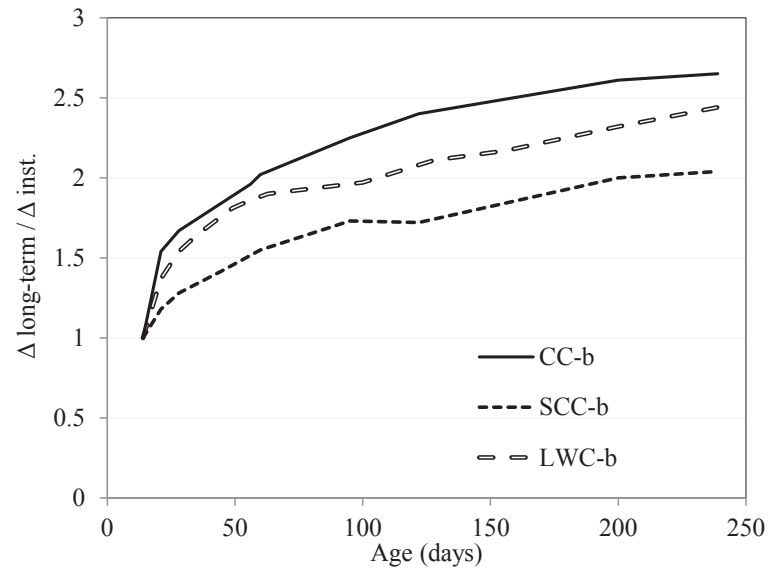


Figure 9.44: The ratio of long-term to instantaneous deflection under lower loading level

Table (9.12) shows the free shrinkage strain of the CC, LWC and SCC specimens at 240 days after casting. The creep coefficients of the companion specimens of the concrete slabs since the loading age are presented in Table (8.8) also.

Table 9.12: Free shrinkage strain and creep coefficient of CC, LWC and SCC mixtures

	CC	LWC	SCC
Shrinkage strain ($\mu\epsilon$)	784	1180	870
Creep coefficient	1.5	2.415	1.96

Comparing the deflection ratios in Figures (9.43), (9.44) and the free shrinkage values in Table (9.12), the ratio of long-term to instantaneous deflection cannot be concluded from the free shrinkage trend over the time period. Also, the rate of deflection and the free shrinkage rate in different types of concrete are not compatible.

Figure (9.45) compares the predicted and experimental ratio of $\Delta_{long} / \Delta_{ins-14}$ of slabs under two loading levels. The experimental ratio ranges from 2.04 to 4.35, however, it changes in the range of 1.89 to 2.31 in the predictions of codes of practice.

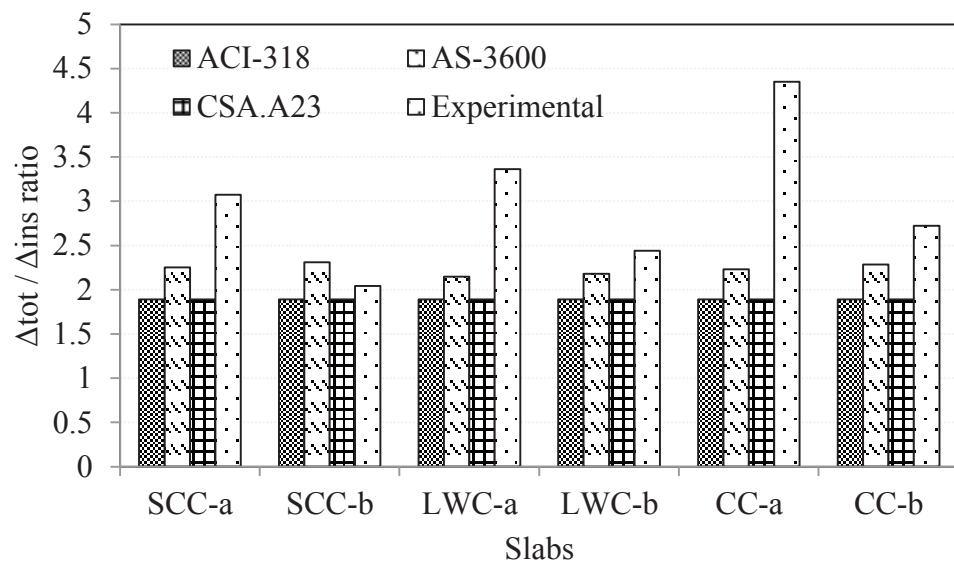


Figure 9.45: Comparing the experimental and predicted ratio of the total to instantaneous deflection

Comparison of the long-term to instantaneous deflection ratio with multipliers in codes of practice

Figures (9.46 a, b) show the predictions of AS-3600 (2009)[19] , ACI-318 (2008)[27] and CSA.A23 (2004)[20] for the instantaneous and total (long-term) deflection of SCC, LWC and CC slabs under two different levels of sustained loading respectively.

According to Figures (9.46 a, b), there is a considerable difference between the predicted Δ_{ins-14} and Δ_{long} of slabs under different load values. The instantaneous and total deflection in the slabs under higher loading level is significantly greater than those of the slabs under lower loading levels, especially by ACI-318[27] and CSA.A23[20] predictions. However, in predicted values of AS-3600[19], the difference between the Δ_{ins-14} and Δ_{long} is less, compared to the other design codes.

All listed codes in Figures (8.29 a, b) predict the highest values of Δ_{ins-14} and Δ_{long} for LWC slabs under both loading levels, however, AS-3600[19] predicts the lowest Δ_{ins-14} and Δ_{long} values for SCC slabs under both loading levels.

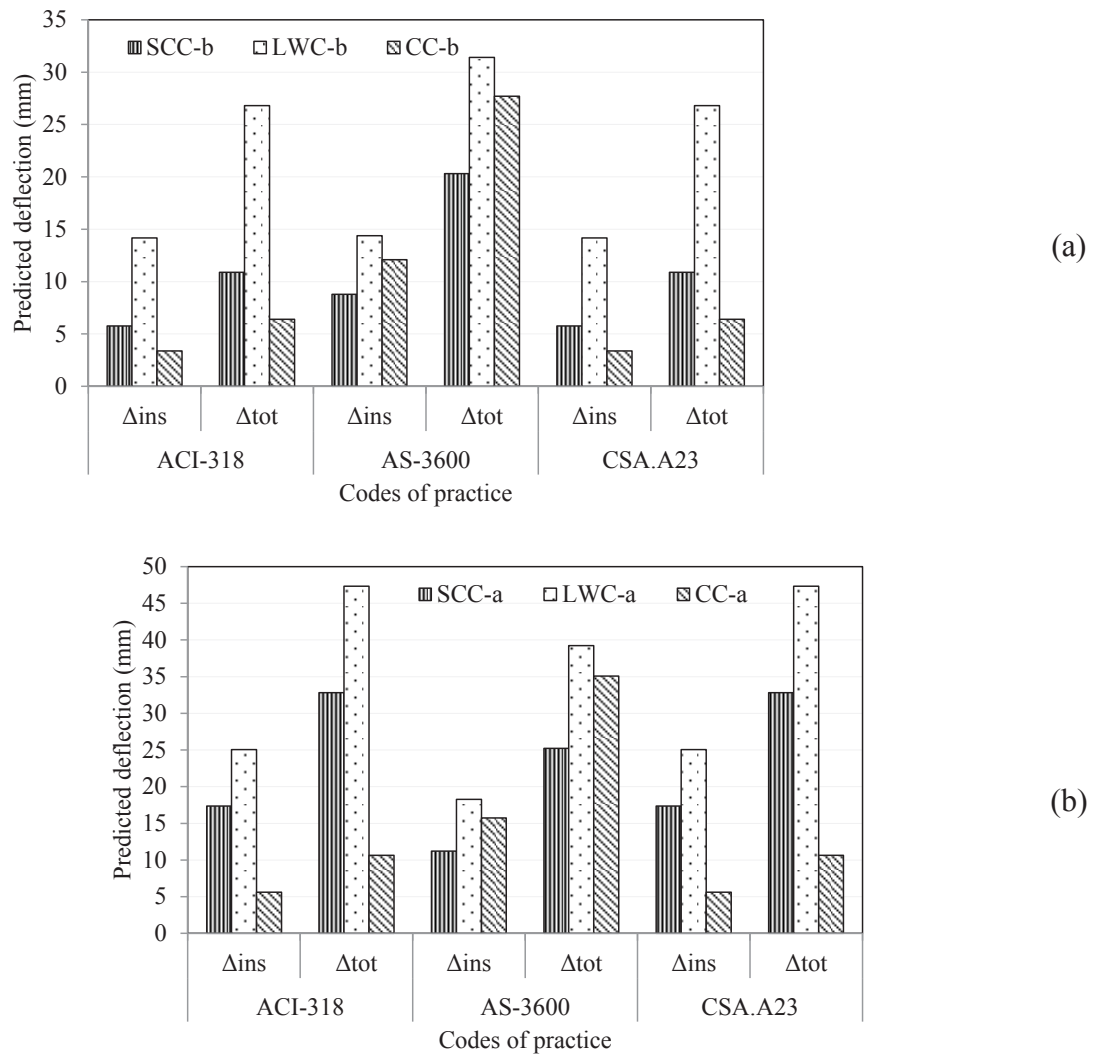


Figure 9.46: Predicted Δ_{ins} and Δ_{tot} of SCC, LWC and CC slabs under a) LL₁, b) LL₂

REFERENCES

1. Unnikrishna Pillai, S. and D. Menon, *Reinforced Concrete Design* Tata Mc Graw Hill, New Delhi, India,, 2003.
2. Vakhshouri, B. and S. Nejadi. *Limitations and Uncertainties in the Long Term Deflection Calculation of Concrete Structures*. in *Second International Conference on Vulnerability and Risk Analysis and Management (ICVRAM) and the Sixth International Symposium on Uncertainty, Modeling, and Analysis (ISUMA)*. 2014.
3. ACI-209.2R-08, *Guide for Modeling and Calculating Shrinkage and Creep in hardened concrete*. American Concrete Institute (ACI), ACI Committee 209, Farmington Hills, MI 48331, U.S.A, 2008.
4. ACI-318-05, *Building code requirements for structural concrete* ACI Committee, Standard ACI 318. American Concrete Institute, Farmington Hills, Mich, 2005.
5. Park, H., et al., *Creep and effective stiffness of early age concrete slabs*. 2010.
6. Park, H.-G., et al., *Immediate and Long-Term Deflections of Reinforced Concrete Slabs Affected by Early-Age Loading and Low Temperature*. ACI Structural Journal, 2012. **109**(3).
7. Lee, J., A. Scanlon, and M. Scanlon, *Effect of Early Age Loading on Time-Dependent Deflection and Shrinkage Restraint Cracking of Slabs with Low Reinforcement Ratios*. ACI Special Publication, 2007. **246**.
8. Vollum, R., *Multipliers for deflections in reinforced concrete flat slabs*. Magazine of Concrete Research, 2003. **55**(2): p. 95-104.
9. Rahman, M., T. Ayano, and K. Sakata, *The use of physical phenomena to predict time effects in cracked reinforced concrete beams*. Magazine of Concrete Research, 1998. **50**(3): p. 219-227.
10. Neville, A., W. Houghton-Evans, and C. Clarke, *Deflection control by span/depth ratio*. Magazine of Concrete Research, 1977. **29**(98): p. 31-41.
11. Bate, S.C.C., *Handbook on the Unified Code for structural concrete (CP110: 1972)*. 1972: Cement and Concrete Association.
12. Scanlon, A. and P.H. Bischoff, *Shrinkage restraint and loading history effects on deflections of flexural members*. ACI Structural Journal, 2008. **105**(4).
13. Xu, Y., et al., *Mechanical properties of expanded polystyrene lightweight aggregate concrete and brick*. Construction and Building Materials, 2012. **27**(1): p. 32-38.
14. Trussoni, M., C.D. Hays, and R.F. Zollo, *Fracture Properties of Concrete Containing Expanded Polystyrene Aggregate Replacement*. ACI Materials Journal, 2013. **110**(5).
15. Sabaa, B. and R.S. Ravindrarajah. *Engineering properties of lightweight concrete containing crushed expanded polystyrene waste*. in *Materials Research Society, 1997, Fall Meeting, Symposium MM, Advances in Materials for Cementitious Composites December*. 1997.
16. Li, Y., N. Liu, and B. Chen, *Properties of lightweight concrete composed of magnesia phosphate cement and expanded polystyrene aggregates*. Materials and Structures, 2015. **48**(1-2): p. 269-276.
17. Gilbert, R. and X. Guo, *Time-dependent deflection and deformation of reinforced concrete flat slabs-an experimental study*. ACI structural journal, 2005. **102**(3).
18. Ian Gilbert, R., *Tension Stiffening in Lightly Reinforced Concrete Slabs*. Journal of Structural Engineering, 2007. **133**(6): p. 899-903.
19. AS-3600-09, *Concrete structures*. Australian Standard, 2009.
20. CSA.A23.3-04, *Design of concrete structures*. Canadian Standards Association (CSA), 2010.
21. Aslani, F., *Experimental and numerical study of time-dependent behaviour of reinforced self-compacting concrete slabs*. PhD dissertation, University of Technology Sydney (UTS), 2014.

22. Nejadi, S., *Time-dependent cracking and crack control in reinforced concrete structures*. 2005, PhD thesis, The University of New South Wales Sydney, Australia.
23. 1012.14, A., *Methods of testing concrete - Method for securing and testing cores from hardened concrete for compressive strength*. Standards Australia, 1991.
24. C39, A., *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. american Society of Testing and Materials, 2000.
25. 1012.17-97, A., *Methods of testing concrete - Determination of the static chord modulus of elasticity and Poisson's ratio of concrete specimens*. Standards Australia, 2014.
26. 14, A.C.C.M.-. *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*. American Society of Testing and Materials, 2000.
27. 318-08, A., *Building code requirements for structural concrete (ACI 318-11) and commentary*. ACI Committee, Standard ACI 318. American Concrete Institute, Farmington Hills, Mich, 2008.
28. Ghali, A., *Deflection of reinforced concrete members: a critical review*. ACI Structural Journal, 1993. **90**(4): p. 364-373.

CHAPTER 10

FINITE ELEMENT MODELLING OF INSTANTANEOUS AND LONG-TERM DEFLECTION OF LIGHTWEIGHT CONCRETE SLABS

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Chapter 10: Numerical analysis

10.1. INTRODUCTION

To develop advanced design and analysis methods, the need for experimental research is crucial. Experimental results provide a secure basis for verification of proposed design equations. Results of the empirical study also supply the required basic data and information for Finite Element (FE) models and programming for analysing the structure. Furthermore, the results of FE models have to be utilized to assess the structure and proposed models should be calibrated with experimental results.

Since the first FE study in reinforced concrete members conducted by Ngo and Scordelis (1967) [1], analyses of reinforced concrete structures using Finite Element Method (FEM), have experienced a remarkable improvement. Development of appropriate analytical models can decrease the number of required empirical activities carried out upon the test specimens or even the real structures for the solution of a given problem, also; identifying that these tasks are time-consuming, very expensive and often do not simulate exactly the loading and support conditions of the actual structure.

FEM has thus become a powerful computational tool, allowing complex analyses of the nonlinear response of Reinforced Concrete (RC) structures to be carried out in a routine fashion. However, the complex behaviour of concrete sets some limitations in implementing FEM. The complexity is mainly due to[2]:

- Non-linear stress-strain relation of the concrete under multi-axial stress conditions;
- Strain softening and anisotropic stiffness reduction;
- Progressive cracking caused by tensile stresses and strains;

- Bond between concrete and reinforcement;
- Aggregate interlocks and dowel action of reinforcement;
- Time-dependent behaviour as creep and shrinkage.

The nonlinear Finite Element Analysis (FEA) can define crack formation and growth, the appearance and process of structure breakage, and additionally assess the ultimate bearing capacity and reliability of the structure, which is important in analysis and design of the structure [3].

Constitutive relation of the reinforced concrete material has considerable influence on the nonlinear analysis. An appropriate material model in the FE model should certainly be able to represent both elastic and plastic behaviour of concrete in compression and tension.

Chen and Saleeb (1982); Parton and El-Barbary (1983); Vidoso et al. (1988); Marzouk and Chen (1993); Marzouk and Jiang (1996); Al-Nasra (1997); Jiang and Mirza (1997); Reitman and Yankelevsky (1997); Polak (1998); Huang et al. (1999); Enochsson and Dufvenberg (2001); Shanmugam et al. (2002); Vainiunas et al. (2004); Polak (2005); Murray et al. (2005) and Deaton (2005) are just some researchers who have applied FE models to investigate the interaction of different non-linear effects on the response of reinforced concrete structures[4].

The aim of this study is to simulate and compare the effective modelling of reinforced concrete one-way slabs made with Light-Weight Concrete (LWC) containing treated Expanded Poly-Styrene (EPS) beads. FEM has been used to model the slabs with the aid of a computer program to process the FE model. The models were assessed on their capability of replicating the real cracking and deflection behaviour in the real models properly. There are multiple methods for determining the deflection of beams as well as calculating crack spacing and crack width concerning the instantaneous (short-term) loading or after being subjected to the sustained load for a long time. This study utilizes the FE based computer program, ATENA, developed by Cervenka et al., (2015) [5] to simulate the reinforced concrete slabs. Mechanical properties of concrete including the time-dependent properties have been simulated by the ATENA program also. The main objectives of this chapter are:

- A brief presentation and discussion of the basic concepts, materials modelling and fracture models in general FEMs and particularly, ATENA;
- Non-linear FE modelling of the reinforced concrete structures subjected to sustained service loads;
- Parametric study of the hardened concrete properties models, bond-slip models, and creep and shrinkage models based on the experimental results and the default ATENA models, to simulate and determination of instantaneous and time-dependent deflection of lightweight concrete slabs.

In addition, the numerical analysis and parametric study of the slabs in this research contains the following items:

- 1- Comparing the instantaneous deflection of LWC slabs under two different levels of loading;
- 2- Comparing the time-dependent deflection of LWC slabs under the same two distinct levels of loading as for the instantaneous deflection;
- 3- Sensitivity of the instantaneous and time-dependent deflection to the implemented material models in ATENA;
- 4- Investigating the ratio of time-dependent deflection to the instantaneous deflection under two different levels of loading and comparing these with the experimental results to verify the corresponding multipliers in codes of practice.

10.2. FINITE ELEMENT MODELS FOR REINFORCED CONCRETE STRUCTURES

Constitutive concrete models are implemented in several software applications. These analysis programs contain possible approaches to the behaviour of concrete elements and concrete models. To choose the proper model of concrete, considering the purpose and the area where the concrete should be used is mandatory; because the concrete models of different purposes require different data and accuracies of the models. The required details of the computational model are an example of this matter. In the majority of the advanced models, the more accurate the model, the higher the demands regarding the extent and quality of the complex data, which makes, in turn, the computational process more demanding. A non-linear analysis of reinforced and plain concrete is often combined with other types of the task. Such analysis tasks could, for instance, simulate dynamic or earthquakes loads.

Developing a constitutive model which can provide reliable predictions of short-term and long-term deflection behaviour of reinforced LWC to facilitate a parametric study of deflection behaviour is the primary scope of this study. In this regard, the effective parameters of the deflection variation with time may be identified and quantified.

10.3. DAMAGE MODELLING IN CONCRETE

There are three different models used to simulate the damage in reinforced concrete elements: (1) Smeared crack concrete model, (2) Brittle crack concrete model, and (3) Concrete damaged

plasticity model [6]. Smeared crack and damage plasticity models are two main methods of the non-linear analysis of concrete in FE simulations.

10.3.1. Smeared crack concrete model

In this model, the cracking process begins at any location where the concrete stresses reach one of the failure surfaces either in the biaxial tension region or a combined tension-compression region and it does not track individual “macro” cracks. Developed cracks affect the stress and material stiffness associated with the integration point. Cracking is assumed to be the most important aspect of the concrete behaviour and the representation of cracking and post-cracking anisotropic behaviour dominates the modelling [2]. This model is proper for the concrete which is subjected to essentially monotonic straining at low-confining pressures. It uses oriented damaged elasticity concepts (smeared cracking) to describe the reversible part of the response of material after cracking failure [7].

The strain is decomposed into elastic strain and plastic strain as illustrated in Equation (10.1).

$$\varepsilon = \varepsilon_e + \varepsilon_p \quad \text{Eq. (10.1)}$$

Where; $\varepsilon_e = \sigma/E_c$ is elastic strain, and ε_p is plastic strain.

The uniaxial compressive stress-strain relationship proposed by Saenz (1964) is now widely accepted as the mathematical description of the uniaxial stress-strain curve for concrete [8]. The cracking and compressive responses included in the concrete model are demonstrated by the uniaxial response of a specimen as shown in Figure (10.1). When concrete is subjected to compressive stresses, the non-linear deformations of concrete are basically inelastic, because upon unloading, only a portion of those strains can be recovered from the total strains. Therefore, the stress-strain behaviour of the concrete material may be separated into recoverable and non-recoverable components. The recoverable part can be treated within the field of elasticity theory while the theory of plasticity can treat the irrecoverable part. Plasticity-based models have been used extensively in recent years to describe the behaviour of concrete [8].

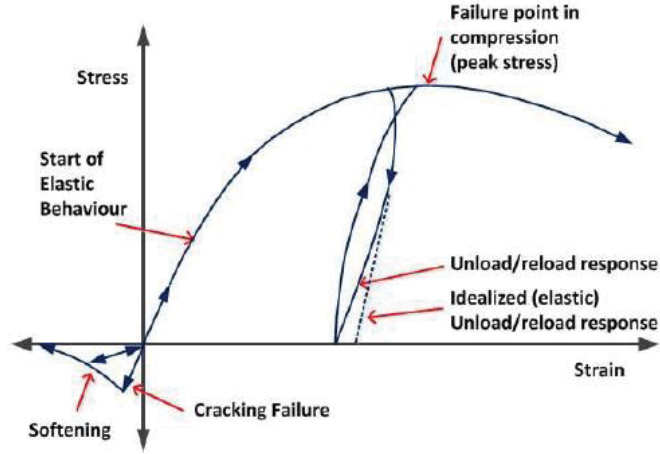


Figure 10.1: Uniaxial stress-strain response and cracking of concrete [2]

A “compression” failure surface forms the basis of the model for the non-linear response when the principal stresses are mainly compressive. When the stress reaches a failure surface which is called the “crack detection” surface in tension (including tension-compression zone), cracking is expected to happen.

10.3.1.1. Tension before cracking

The behaviour of concrete in tension without cracks is assumed to be linear elastic. The equivalent uniaxial law is used to describe the non-linear behaviour of concrete in the biaxial state regarding the equivalent uniaxial strain, ε^{eq} and the effective stress σ_c^{ef} in Equation (10.2).

$$\sigma_c^{ef} = E_c \varepsilon^{eq} \quad ; \quad 0 \leq \sigma_c \leq f_t'^{ef} \quad Eq. (10.2)$$

where; $f_t'^{ef}$ is the effective tensile strength derived from the biaxial failure function of concrete and E_c is the initial elastic modulus of concrete.

As shown in Figure (10.2), the biaxial failure of concrete contains distinct tension and compression failure criteria. In the compression-compression stress state, the failure is a function of the principal stresses in concrete (σ_{c1} , σ_{c2}) and the uniaxial cylinder strength of concrete (f'_c). In the tension-compression state, the failure function continues linearly from the point $\sigma_{c1} = 0$, $\sigma_{c2} = f'_c$ into the tension-compression region with the linearly decreasing strength as described in Equation (10.3).

$$f_c'^{ef} = r_{ec} \cdot f_c' \quad ; \quad 0.9 \leq r_{ec} \leq 1.0 \quad Eq. (10.3)$$

Where; r_{ec} the reduction factor of the compressive strength is a function of σ_{e2} and f_c' .

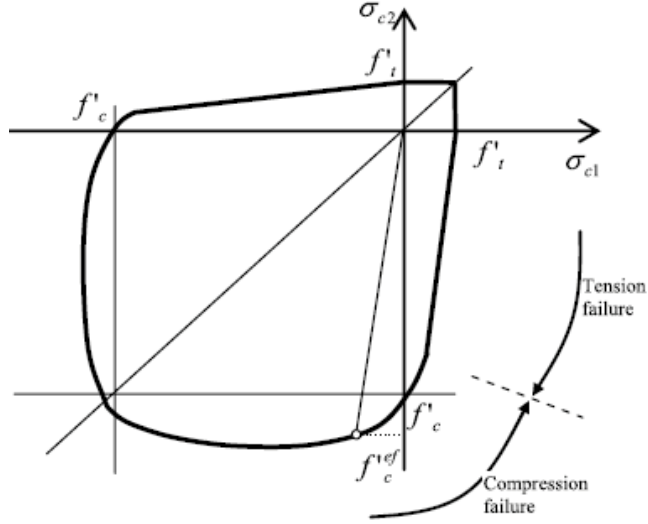


Figure 10.2: Biaxial Failure criteria of concrete [9]

In the tension-tension area, the tensile strength is constant and equal to the uniaxial tensile strength (f_t'). In the tension-compression state, the tensile strength is reduced as described in Equation (10.4).

$$f_t'^{ef} = r_{et} \cdot f_t' \quad Eq. (10.4)$$

Where; r_{et} the reduction factor of the tensile strength is a function of σ_{e2} and f_c' .

10.3.1.2. Tension after cracking

After cracking, a model based on a crack opening law and fracture energy is used in combination with the crack band theory. Gilbert et al. (2010) [9] used the exponential model in Equation (10.5) to describe the post-cracking behaviour of concrete. Other existing models in literature have been discussed in Chapter 4 of this study.

$$\frac{\sigma}{f_t'^{ef}} = \left\{ 1 + \left(c_1 \frac{w}{w_c} \right)^3 \right\} e^{-c_2 \frac{w}{w_c}} - \frac{w}{w_c} (1 + c_1^3) e^{-c_2} ;$$

$$c_1 = 3 ; c_2 = 6.93 ; w_c = 5.14 \frac{G_f}{f_t'^{ef}} \quad Eq. (10.5)$$

Where; w is the crack width; w_c is the crack width at the complete release of stress; σ is the normal stress on the crack (crack cohesion); G_f is the fracture energy needed to create a unit area of stress-free crack and was assigned a value of 25 N/m in the numerical model; and $f_t'^{ef}$ is the effective tensile strength derived from the failure function.

The crack opening displacement w is derived from strains according to the crack band theory. Figure (10.3) shows the exponential crack opening law in concrete. This law is just one of the five alternatives available in the utilized program, ATENA.

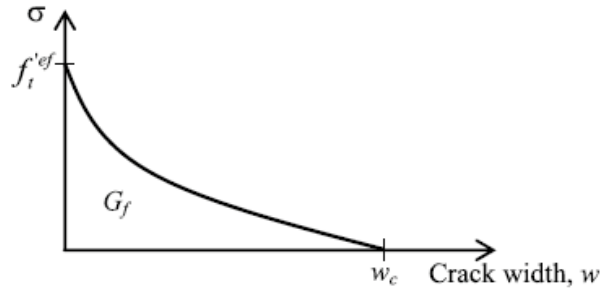


Figure 10.3: Exponential crack opening model of concrete in ATENA [9]

Once the crack occurs, the FE program uses a smeared crack approach in which constitutive calculations are performed independently at each integration point of the FE model. The initiation of cracks enters into these calculations by affecting the stress and material stiffness associated with integration points [2].

The smeared crack concrete model applies tension stiffening to specify the post-failure stress-strain relationship. Tension stiffening also can be modelled by applying a fractured energy cracking criterion. A typical tension stiffening model is shown in Figure (10.4).

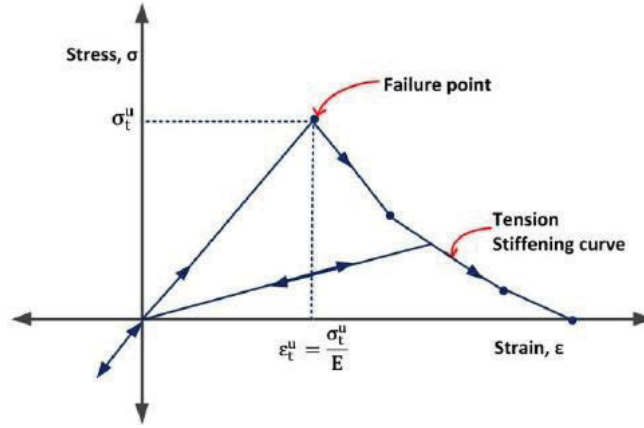


Figure 10.4: Typical tension stiffening model in FEA simulation [2]

Proper application of tension stiffening parameters is essential, as tension stiffening values which are too small cause the local cracking failure in the concrete to introduce temporarily unstable behaviour in the overall response of the model while high tension stiffening makes it easier to obtain numerical solutions.

10.3.2. Damage plasticity method:

The damage in quasi-brittle materials in which the confining pressure is sufficiently large to prevent the brittle fracture can be defined by evaluating the dissipated fracture energy required to create micro cracks [2]. Strain rate decomposition is assumed for the rate-independent model as presented in Equation (10.1).

The stress-strain relations are governed by scalar damaged elasticity in Equation (10.6):

$$\sigma = (1 - \xi)D_0^{el} : (\varepsilon - \varepsilon_p) = D^{el} : (\varepsilon - \varepsilon_p) \quad Eq. (10.6)$$

Where; D_0^{el} is the initial (undamaged) elastic stiffness of the material; σ is the degraded elastic stiffness; ξ is the scalar stiffness degradation variable, which can take variation in the range from zero (undamaged material) to one (fully damaged material).

The associated damage with the failure mechanisms of the concrete (cracking and crushing) results in a reduction in the elastic stiffness. Within the context of the scalar-damage theory, the stiffness degradation is isotropic and described by a single degradation variable, d . Following

the usual concepts of continuum damage mechanics, the effective stress is defined as Equation (10.7):

$$\sigma_e \stackrel{\text{def}}{=} D_0^{\text{el}} : (\varepsilon - \varepsilon_p) \quad \text{Eq. (10.7)}$$

The Cauchy stress is associated with the effective stress through the scalar degradation relationship as explained in Equation (10.8):

$$\sigma = (1 - \xi)\sigma_e \quad \text{Eq. (10.8)}$$

10.4. REINFORCEMENT MODELLING

There are three approaches to how to model the steel bars in FE modelling; distributed steel approach, embedded steel approach and discrete steel approach. The general view of these models is presented in Figure (10.5). The classic model [10] is generally used for the elastic and plastic behaviour of concrete under tension and compression. A multi-linear stress-strain relationship is used to model the reinforcement in the numerical model and allows all stages of steel behaviour to be readily modelled, including the elastic state, yield plateau, strain hardening, and fracture [9, 10].

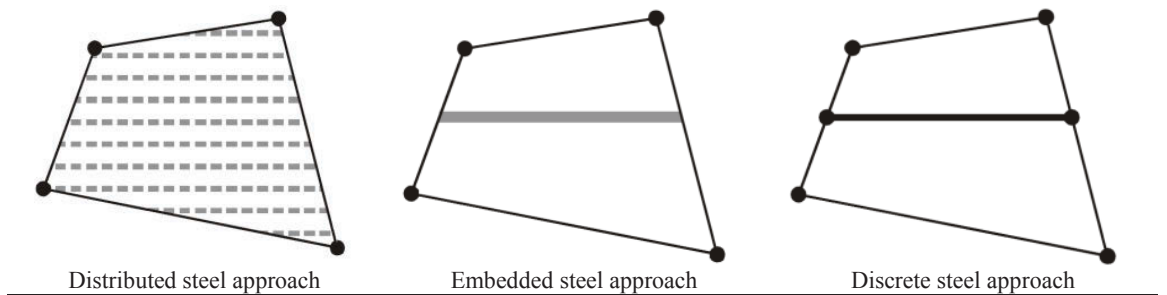


Figure 10.5: Different methods of reinforcing modelling in FEM [11]

Figure (10.6) shows the typical bilinear and multi-linear stress-strain relationship of steel bars in FEM.

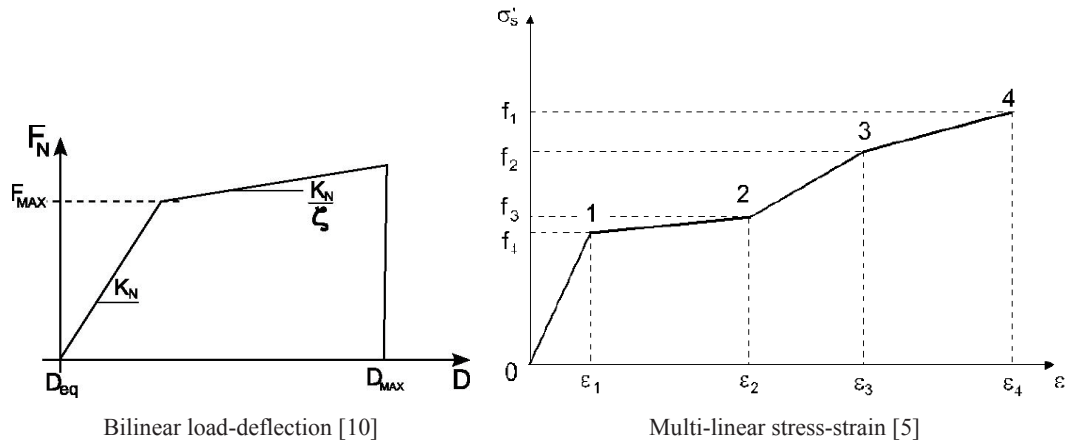


Figure 10.6: Interaction law for reinforcing steel in FEA

10.5. EFFECT OF ANALYSIS STEPS

Finite element simulations enable designers and researchers to consider problems that exhibit some form of non-linear behaviour. Whether it is large-scale deformation, plasticity, or contact between bodies, we generally classify these non-linear problems as containing some form of varying stiffness in the model. Sometimes, this change in flexibility can be abrupt-like bodies coming into contact with each other, or smooth-like creep or low levels of plasticity. With all of these problems, a representative force-deflection curve is not linear and the stiffness changes with deformation.

In general, geometric non-linearity, material non-linearity, force-boundary condition nonlinearity, and displacement boundary condition non-linearity are the common sources of non-linearity. Non-linear FEA applies to the following conditions:

- Stability analysis: To find critical points (limit points and bifurcation points) closest to operational range;
- Service configuration analysis: To find the ‘operational’ equilibrium configuration of certain slender structures when the fabrication and service configurations are quite different (e.g. cable and inflatable structures);
- Reserve strength analysis: To find the load-carrying capacity beyond critical points to assess safety under abnormal conditions;
- Progressive failure analysis: A combined strength and stability analysis in which progressive deterioration (e.g. cracking) is considered.

For static FEA, the equation representing equilibrium can be written in the matrix form of Equation (10.9):

$$K. \Delta U = N - F \quad Eq. (10.9)$$

Where N represents the vector of applied loads, F is the vector of internal forces, and ΔU is the vector of current nodal displacement. In non-linear analysis, the load N is used in a series of load steps $N_1, N_2, N_3, \dots, N_n$.

Finite Element Analysis (FEA) involves solving the Equation (10.9) for ΔU . For the n -th load step, the equation is often solved by iterations of the form presented in Equation (10.10)

$$K. \Delta U_{(i+1)} = N_{(n)} - F_{(i)} \quad Eq. (10.10)$$

There are different methods of solving the non-linear equations in FEA. By increasing the load value in higher load steps until the cracking load, the concrete structures often show a plastic flow and even softening behaviour, while deformation is increased quickly with slight load change. Since the load is not sensitive, if the force is used as a convergent norm, it is often difficult to achieve convergence. So the displacement is used as the convergent norm during the loading process [12].

FE programs have taken on the task of solving non-linear static structural problems in a number of ways. In all cases, the algorithms have to iterate to achieve equilibrium for static structural problems. The solver will apply some portion of the external forces to the model, and then calculate the resultant internal forces to balance the applied load. There will initially be some out of balance residual force so the solver will have to improve the stiffness to narrow down the imbalance difference toward zero. Obtaining final convergence and equilibrium, when running these iterative problems, can be very challenging.

10.6. CONVERGENCE METHODS IN NON-LINERA ANALYSIS

A convergence criterion measures how well the obtained solution satisfies the non-linear equilibrium. In FEA, the convergence criteria are usually based on some norms of the displacement, residuals and energy as the product of residual and displacement.

The convergence criteria and tolerances must be carefully chosen so as to provide accurate yet economical solutions. If the convergence criterion is too broad, inaccurate results are obtained. If the convergence criterion is too tight, too much effort will be spent in obtaining unnecessary accuracy.

10.7. ATENA SOFTWARE

Among the existing FE software examining the accurate simulation of reinforced concrete structures, this study utilizes the ATENA program for analysis of the short-term and long-term behaviour of the reinforced LWC one-way slabs.

Application of ATENA for instantaneous, time-dependent and fracture analysis of reinforced concrete structures has been increasing, especially in recent years. Cervenka et al. (2002) [13], Johnson (2006) [14], Yang et al. (2007) [15], Yu (2007) [16], Malm and Holmgren (2008)[17], Talley (2009)[18], Gribniak et al. (2010) [19], Gilbert and Sakka (2010)[19], Dobromil et al. (2010)[20], Olteanu et al. (2011) [21], Lampropoulos et al. (2012) [22], Sucharda and Brozovsky (2014) [23], Singh ET AL. (2014) [24], Meskenas et al. (2014)[25] and Namy (2015) [26], are just some researchers who have used ATENA in their investigations.

The main reasons for choosing ATENA in the current study can be justified as follows:

- a)** Almost all of the creep models in the widely used codes of practice and international standards are included in ATENA and can be used during the analysis;
- b)** By utilizing the ATENA program it is possible to simulate the shrinkage behaviour of reinforced concrete members precisely;
- c)** In addition to the available models for different properties of concrete, there is a facility to define extra properties or modify the available models by the user. Hardened mechanical properties of concrete such as compressive strength, tensile strength, modulus of elasticity, stress-strain relationship, bond stress, tension stiffening factor and crack width are some user-defined parameters in this program;
- d)** A wide range of structural, non-structural, fibre reinforced and special concrete can be used by this program;
- e)** The ATENA program package includes ATENA-GiD and the ATENA-FE program. The main ATENA-FE program is used for analysis of the simulated model, while the software GiD is parallel to ATENA and can be utilized for interfacing with ATENA for data preparation and mesh generation.

In comparison to the multiple choices of material models, ATENA provides a much more appropriate selection of models for simulating requirements. Moreover, in most cases, ATENA also offers the option of using either a rotating crack model or a fixed crack model [7].

Other key features in ATENA are as follows:

- Three-dimensional (3D) environment with User friendly modelling of reinforcement;
- Unique visualization of crack propagation;

- Real-time display of results even during the non-linear analysis based on FE method and fracture mechanics;
- Advanced material models for concrete, reinforcement, steel, rock, soil and masonry;
- Support for the analysis of modern fibre reinforced concrete materials: Strain Hardening Cementitious Composites (SHCC), Engineered Cementitious Composite (ECC), High-Performance Fibre Reinforced Concrete (HPRFC), Ultra High-Performance Fibre Reinforced Concrete (UHPFRC);
- Dynamics, statics, creep, thermal and moisture analyses.

10.8. TECHNICAL SPECIFICATIONS OF ATENA-V5

10.8.1. Materials modelling

This program offers a wide variety of different material models for various materials and purposes. The following are general definitions of the existing material models. Detailed specifications are discussed in next sections:

- Three-dimensional fracture-plastic concrete model based on Menetrey-Willam law: smeared cracks, fracture-energy based softening, compression field theory, aggregate interlock in shear, non-associated plasticity, unloading direction, and user defined functions, variable parameters;
- Two-dimensional (2D) “SBETA” concrete model, also for high-strength and Steel Fibre Reinforced Concrete (SFRC): smeared cracks, crack-band, fracture energy based softening, Kupfer’s compressive failure, variable shear retention, strength reduction of cracked concrete;
- Reinforcement bi-linear and multi-linear, reinforcement with bond, cyclic reinforcement (Menegotto-Pinto / Bauschinger effect) and bond;
- Von-Mises plasticity for metals such as steel reinforcement;
- Drucker-Prager plasticity with associated / non-associated flow rule for rock and soil;
- Bazant M4/M7 micro-plane concrete;
- Interface with Mohr-Coulomb material law;
- Isotropic elastic;
- Non-linear springs;
- Temperature-dependent material properties (Fire loading);
- Creep and shrinkage (Bazant B3, Euro code, ACI and others);
- Heat and moisture transport, hydration heat model CERHYD;

- Fatigue of concrete in tension;
- User-defined material model (user-compiled DLL in C/C++ or FORTRAN).

10.8.2. Loading models

ATENA enables loading of the structure with various actions. These loading cases are combined into load steps, which are solved utilizing advanced solution methods like Newton–Raphson, modified Newton–Raphson or arc-length. The following are the main parts of loading models in ATENA:

- Load cases: body forces, nodal or linear forces, supports, prescribed deformations, temperature, shrinkage, prestressing;
- Load steps: the combination of load cases, with solution methods;
- Arbitrary load history in steps, non-proportional, cyclic, dynamic;
- Construction process.

10.8.3. Finite Elements

ATENA includes a wide variety of finite elements of linear and non-linear analysis of reinforced concrete structures. The following are the main types of finite element in ATENA:

- 2D iso-parametric elements, quadrilateral, triangular, axisymmetric elements
- 3D solid elements: tetrahedron, brick, wedge; low and high order
- 3D shell elements (layered) and beams (fibre)
- Truss elements for reinforcement, external cable elements
- Spring supports – point, line, surface
- Interface elements, 2D and 3D

10.8.4. Solution methods

There are different methods of solving the non-linear equations in FEA of ATENA program. The following are the main solution methods in ATENA:

- Direct band (skyline) and sparse iterative equation system solvers; eigenvalues;
- Newton-Raphson, modified Newton-Raphson, arc-length, line-search;
- Tangential and elastic stiffness predictors;

- Newmark and Hughes Alpha for dynamics;
- 32-bit and 64-bit solution core.

10.8.5. Graphical user environment

ATENA offers a user-friendly graphical interface, which enables efficient solving of engineering problems including anchoring technology and reinforcement of concrete:

- 2D Graphical User Environment (GUE) including: pre-processing (geometrical modelling, reinforcement (bars, smeared), automatic meshing, material properties, loading and supports, solution methods, monitoring), solution (non-linear FE solution, graphical monitoring, restart), post-processing (iso-lines, iso-areas, rendering, vectors, tensors, cracks, response diagrams, cuts/sections, internal forces, user-defined numerical output);

- 3D GUE including: pre-processing (geometrical modelling, reinforcement (bars, smeared), copy and move, automatic meshing, material properties, loading and supports, solution methods, monitoring), solution (direct or sparse iterative, graphical monitoring, restart), post-processing (iso-lines, iso-areas, rendering, vectors, tensors, cracks, response diagrams, cuts/sections, user-defined numerical output);

- ATENA Studio: Graphical user interface for ATENA Science for solution and post-processing of 2D and 3D models;

- GiD interface (GiD - general FE pre- and post-processor from CIMNE, Spain) with interface with ATENA.

10.9. CONCRETE CONSTITUTIVE MODELS IN ATENA

10.9.1. Concrete Stress-Strain Relations

There are different models in ATENA used to simulate the material behaviour of the elastic, elastic-plastic and plastic range in linear and non-linear based analysis. Also, the time-dependent variation of materials such as creep and shrinkage behaviour, the elasticity-based material model (SBETA), the fracture-plastic material model, and creep and shrinkage analysis algorithms are implemented in ATENA.

The constitutive model of materials used to explain the non-linear behaviour of concrete in a two-dimensional plane stress state is known as SBETA in ATENA. The fracture of plastic material as one of the advanced material models is a predecessor program using the SBETA

model [23]. This model is founded on the smeared cracks, damage and fracture mechanics concepts [17].

The material SBETA includes the following effects of concrete behaviour also.

- (i) Non-linear behaviour in compression including hardening and softening;
- (ii) Fracture of concrete in tension, based on the non-linear fracture mechanics;
- (iii) Biaxial strength failure criteria;
- (iv) Reduction of compressive strength after cracking;
- (v) Tension stiffening effect;
- (vi) Cracking models (fixed crack direction and rotated crack direction).

The equivalent uniaxial law is used to describe the non-linear behaviour of concrete in the biaxial state regarding the equivalent uniaxial strain, and the effective stress [8].

10.9.2. Compressive Strength of Cracked Concrete

The reduced compressive strength of concrete due to cracking can affect the results of analysis. In ATENA, the reduction of the compressive strength after cracking in the direction parallel to the cracks is included. This decrease is performed by a Gaussian function as shown in Figure (10.7). The constant 'c' represents the maximal strength reduction under the large transverse strain.

In Figure (10.7), $f_c^{'ef}$ is the effective tensile strength derived from the biaxial failure function of concrete, f_c' is the uniaxial cylinder strength of concrete and r_c is the reduction factor.

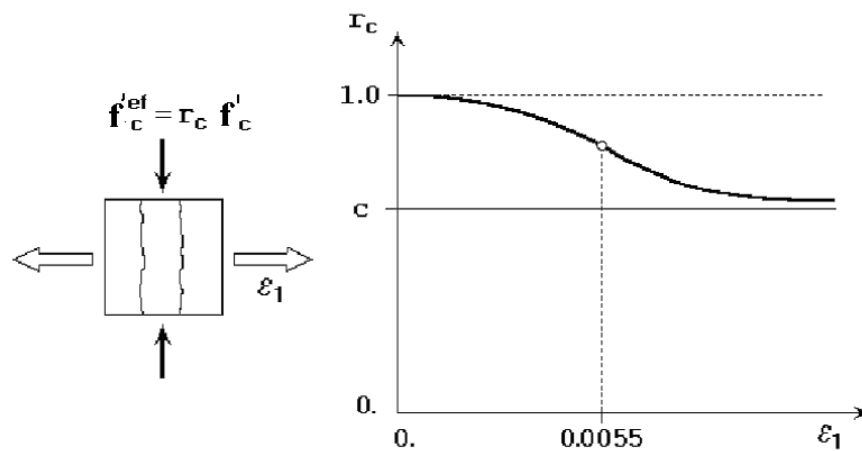


Figure 10.7: Compressive strength reduction of cracked concrete [5]

Reinforcing bars are the main effective parameters on the value of the reduction factor in reinforced concrete. However, this factor covers the reduction of compressive strength in bridges in plain concrete; when the strain localizes in one main crack, the compressive concrete struts can cross this crack (bridging effect).

Kolleger et al. (1988) [27] proposed r_c equal to 0.45 for reinforced concrete with fine mesh. In the investigation by Dyngeland (1989)[28], the reduction factor did not experience values below 0.8. Despite different values of r_c , it should be adjusted for the experimental data and type of reinforcing.

10.9.3. Bond-slip models in ATENA

ATENA contains three models to simulate the bond and slip at the concrete-reinforcement interface: the CEB-FIP model code 1990 [29], the bond-slip law proposed by Bigaj [30], and the user defined bond model. The perfect bond (no slip) model can be defined in the user-defined part also.

In the first two models (CEB-FIP and Bigaj models), the laws are created based on the compressive strength of concrete, diameter and type of steel bar. The confinement conditions and the quality of the concrete casting are other effective parameters in the definition of bond behaviour in these models. In this study, of the three available bond models, the one best able to simulate the observed behaviour of the test slabs was the user defined model. Therefore, the experimental bond-slip model for LWC in Chapter 5 is used for this purpose. Also, the proposed bond-slip models for LWC in Chapter 4 are used for comparison purposes. Figures (10.8) and (10.9) show the table of input data to define the bond-slip model in ATENA 2D and 3D analysis environments.

In the analysis of the slab in this study, the mechanical anchorage at both ends of the steel bars, coupled with the deformed shape of the reinforcing bars proves to provide a good bond between the cracked concrete and the longitudinal reinforcing bars.

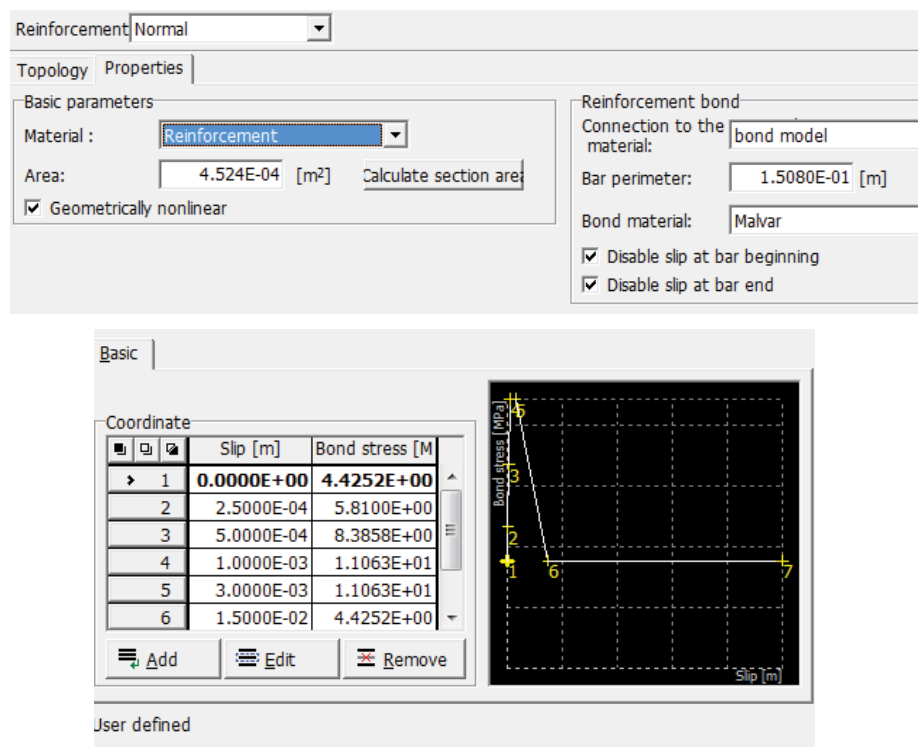


Figure 10.8: Reinforcement and bond details in ATENA (2014) 2D analysis [5]

10.9.4. Crack Models in ATENA

In the broad sense, the constitutive models of concrete may be classified into two categories: the macroscopic phenomenological models and the micromechanical models. Two major approaches are available in the literature for the modelling of cracking in concrete structures, namely the discrete crack approach and the smeared crack approach.

The development of constitutive models that are capable of describing the behaviour of concrete is a significant aspect in smeared crack modelling of concrete. ATENA includes two smeared crack approaches to simulate the effects of cracking. The fixed crack and rotated crack models are two types of smeared crack approach. Concrete without cracks is considered isotropic, while concrete with cracks is orthotropic. The material axes of cracked concrete on the orthotropic axes can be defined by two models; rotated or fixed cracks. Figure (10.10) shows the fixed and rotated crack models in ATENA.

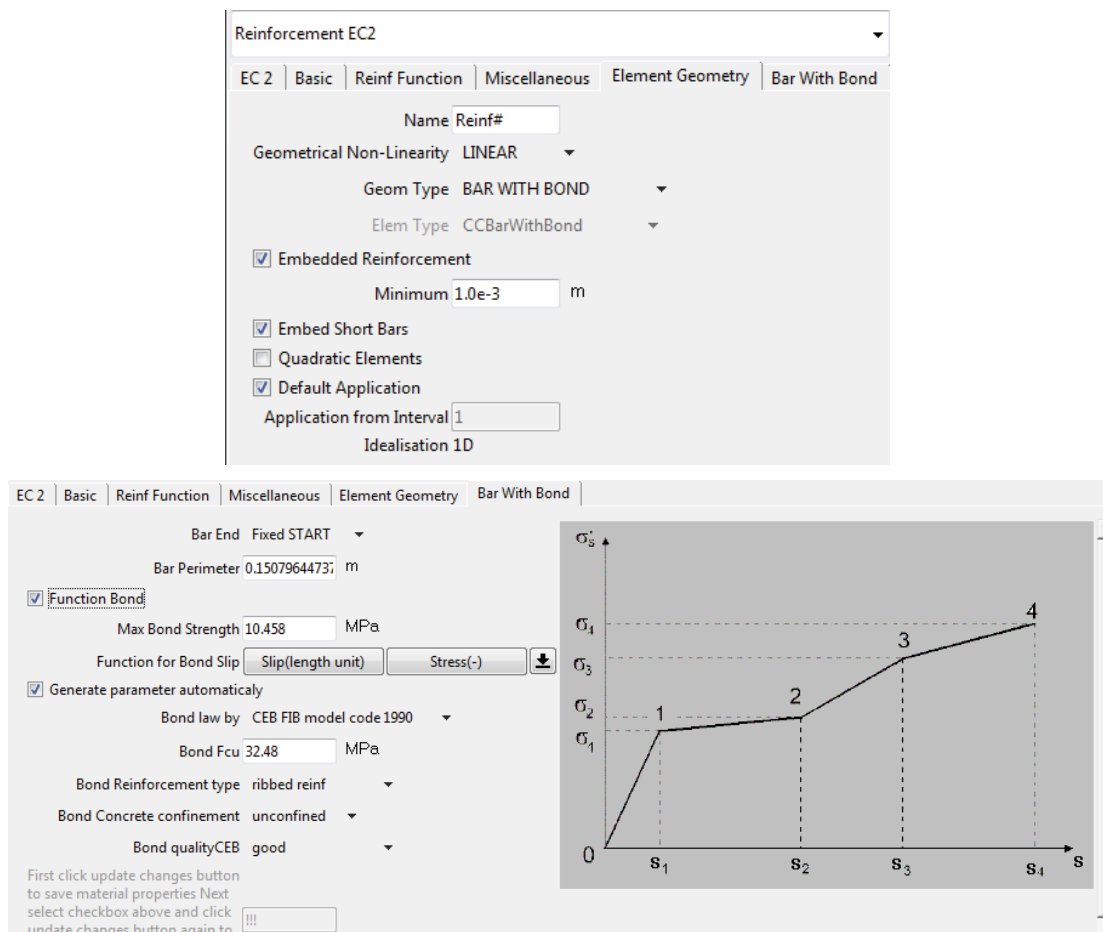


Figure 10.9: Reinforcement and bond details in ATENA (2014) 3D analysis [5]

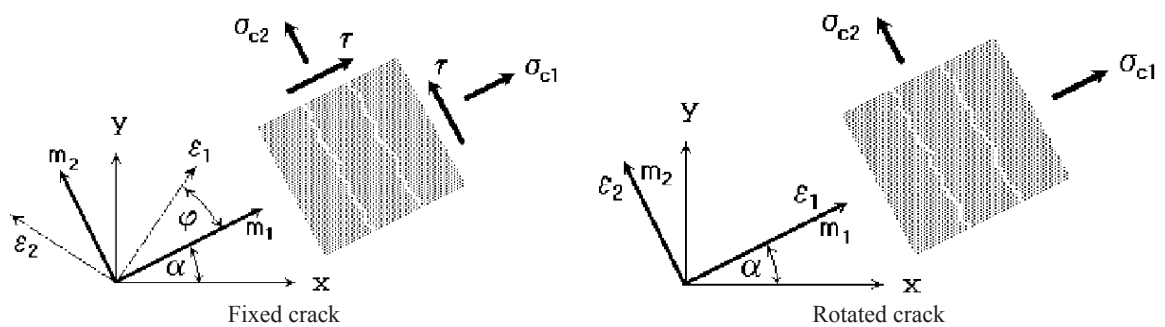


Figure 10.10: Smeared crack models in ATENA [5]

Due to the assumption of isotropy in the concrete component, the directions of principal strain and stress coincide in the uncracked concrete. After cracking the orthotropy is presented. The weak axis of the material (m_1) is normal to the crack direction and the strong axis of the material (m_2) is parallel with the cracks.

In the rotated crack model, the crack direction of the principal stress always coincides with the principal strain direction. In the fixed crack model, the crack direction and the material axes are defined by the principal stress direction at the initiation of cracking.

A significant difference in the above approaches is in the shear model on the crack plane. In the fixed crack model, a strain field rotation generates shear stress on the crack plane. Consequently, the modelling of shear becomes necessary. In case of the rotated cracks, a shear on the crack plane never appears and the shear model need not be employed [13].

In this study, both the fixed and rotated crack models are used and compared. The crack direction is determined from the direction of principal stress at the instant of crack initiation. This direction is then fixed and applied as the axis of the material orthotropy.

10.10. MODELLING OF REINFORCEMENT IN ATENA

Reinforcement can be modelled in two ways in ATENA. The first way is a discrete bar, which is defined as a geometrical multi-linear object. It is embedded in solid objects. Its geometry is defined independently of macro-elements. Thus, one reinforcing bar can be embedded in any number of macro elements. The second way is a smeared reinforcement, defined as a composite material, which has no effect on the geometrical model [5]. Gilbert and Sakka (2010) [9] used discrete truss elements to model the embedded reinforcement in concrete to study the strength and ductility of slabs.

Gribniak et al. (2010) [19] applied the reinforcement in the layers with the isotropic quadrilateral FE with four integration points to model the reinforcement and concrete. They studied the effect of mesh size on deformation of bridge girders.

10.11. CRITERIA TO SELECT PROPER ELEMENTS IN ATENA

The implemented elements in ATENA can be divided into three main groups:

- Plane elements for 2D, 3D and axisymmetric analysis;
- 3d solid elements;
- Special elements, which comprise elements for modelling external cable, springs, gaps, etc.

Almost all existing elements in ATENA are constructed using the iso-parametric formulation with linear and/ or quadratic interpolation functions. The interpolation function in a hierarchical

manner is a considerable advantage of iso-parametric elements in ATENA. Despite the application of this manner for plane quadrilateral elements in general, ATENA applies it for plane triangular elements, 3D bricks, tetrahedral and wedge elements too. Together with the interpolation function, the numerical integration scheme is another effective parameter in FEA. It strongly affects the element properties by integrating the stiffness matrix and nodal forces. ATENA applies the Gauss integration scheme to incorporate the majority of the elements.

10.11.1.Plane elements

10.11.1.1. Truss 2D and 3D Element

This group of elements is coded as CCIsoTruss <xx> for 2D analysis and CCIsoTruss <xxx> for 3D analysis. Figure (10.11) shows the geometry of CCIsoTruss elements. This type of element has constant strain along its length.

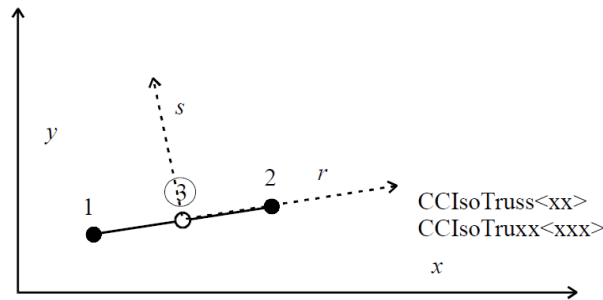


Figure 10.11: Geometry of CCIsoTruss elements [5]

10.11.1.2. Plane Quadrilateral Elements

This group of elements is coded as CCIsoQuad <xxxx> for 4 element nodes, to CCIsoQuad <xxxxxxxx> for 9 element nodes. Plane 2D, axisymmetric and 3D problems can apply this type of element. Figure (10.12) shows the geometry of CCIsoQuad elements.

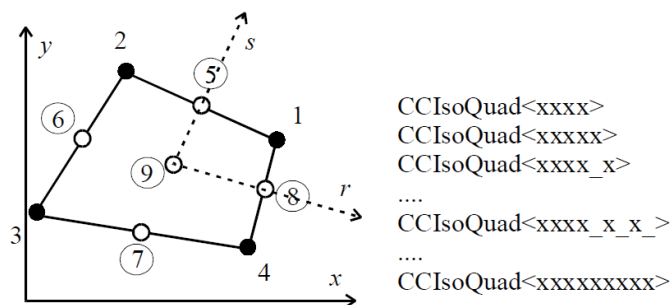


Figure 10.12: Geometry of CCIsoQuad elements (by 4 to 9 nodes) [5]

10.11.1.3. Plane Triangular Elements

This group of elements is coded as CCIsoTriangle<xxx> for 3 element nodes, to CCIsoTriangle<xxxxxx> for 6 element nodes. Plane 2D, axisymmetric and 3D problems can apply this type of element. Figure (10.13) shows the geometry of CCIsoTriangle elements.

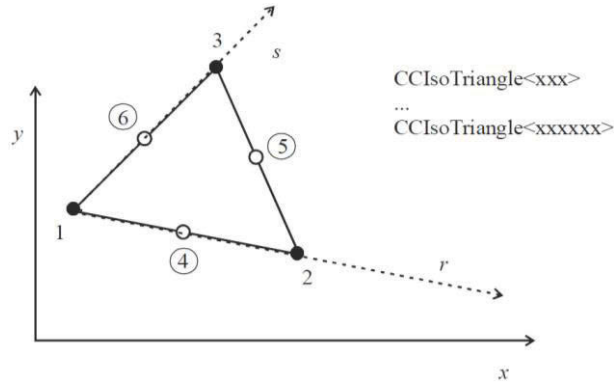


Figure 10.13: Geometry of CCIsoTriangle elements (by 3 to 6 nodes) [5]

10.11.2.3D Solid Elements

10.11.2.1. Tetrahedral elements

This group of elements is coded as CCIsoTetra<xxxx> for 4 node element, to CCIsoTetra<xxxxxxxxxx> for 10 node element. Figure (10.14) shows the geometry of CCIsoTetra elements.

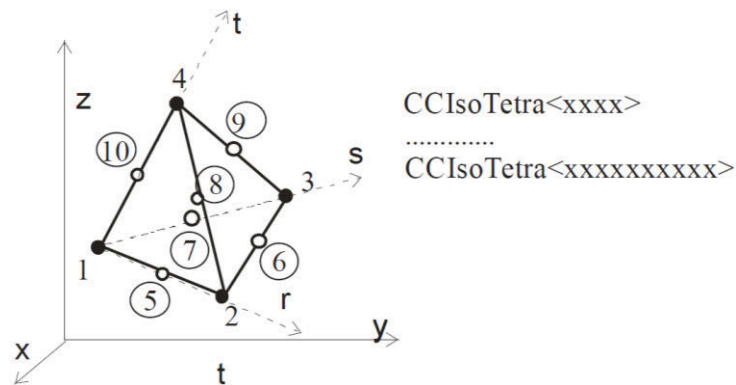


Figure 10.14: Geometry of CCIsoTetra elements (by 4 to 10 nodes) [5]

10.11.2.2. Brick elements

This group of elements is coded as CCIsoBrick<xxxxxxxx...> for 8 up to 20 node element. Figure (10.15) shows the geometry of CCIsoBrick elements.

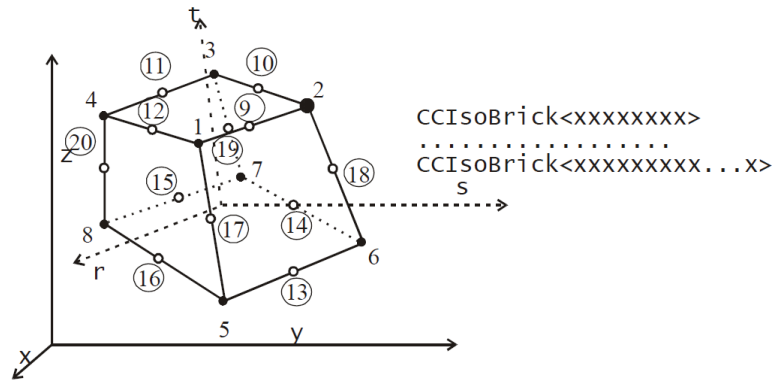


Figure 10.15: Geometry of CCIsoBrick elements (by 8 to 20 nodes) [5]

10.11.2.3. Wedge elements

This group of elements is coded as CCIsoWedge<xxxxxx...> for 6 up to 15 node element. Figure (10.16) shows the geometry of CCIsoWedge elements.

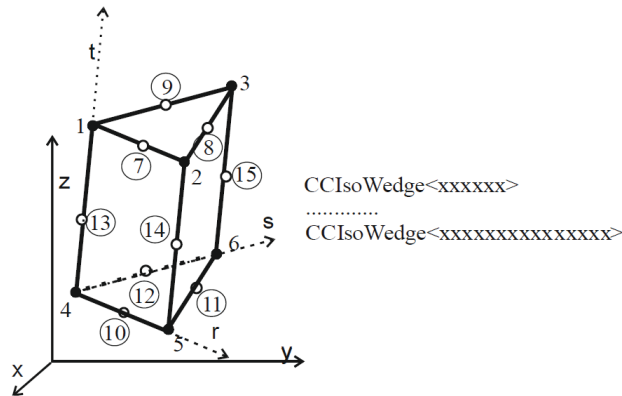


Figure 10.16: Geometry of CCIsoWedge elements (by 8 to 20 nodes) [5]

10.11.3.Spring Element

This type of elements is used to model the spring-type boundary conditions such as the linear relationship of the external forces on the boundary of structure with the associated displacement. CCSpring, CCLineSpring and CCPlaneSpring are three figures of this element. All these elements are derived from 2D or 3D formulation of the CCIsoTruss<xx> element.

CCSpring-2D and 3D elements model the spring-related condition at a point.

CCLineSpring- 2D elements model the spring-like boundary conditions along a line.

CCPlaneSpring-3D element models the spring-like boundary conditions along a triangular area.

Figure (10.17) shows the geometry of spring elements.

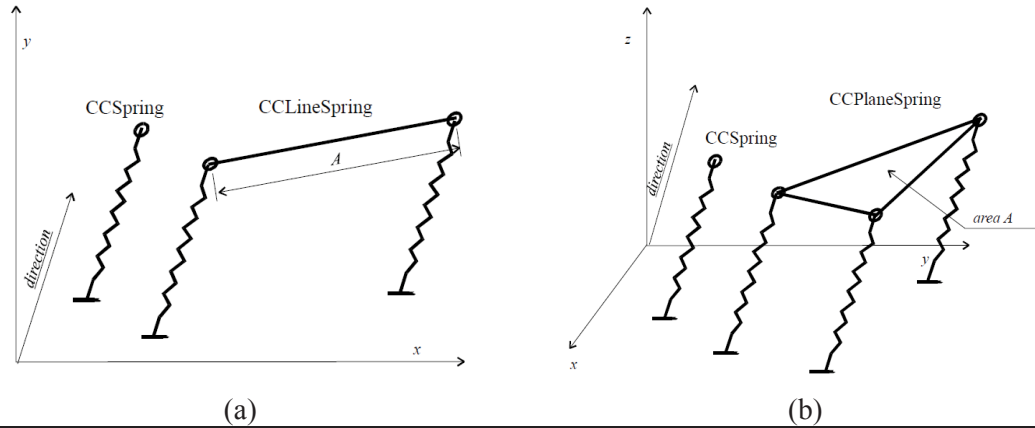


Figure 10.17: Geometry of a) 2D CCSPRING and CCLINESPRING and b) 3D CCSPRING and CCPLANE SPRING [5]

Reinforcement can be modelled in two distinct forms: discrete and smeared. Discrete reinforcement involves using a form of reinforcing bars and is modelled by truss elements. The smeared reinforcement is a component of composite material and can be considered either as a single (only one constituent) material in the element under consideration or as one of most such constituents. The former case can be a special mesh element (layer); while, the latter can be an element with concrete containing one or more reinforcements. In both cases the state of uniaxial stress is assumed and the same formulation of stress-strain law is used in all types of reinforcement.

Table (10.1) summarizes the available types and codes of elements in ATENA.

Table 10.1: Available element types in ATENA [5]

Type	Name	Description
3d solid	CCIsoBrick	Isoperimetric brick element (hexahedron)
	CCIsoWedge	Isoparametric wedge element E.g.: CCIsoWedge<xxxxxx>
	CCIsoTetra	Isoparametric tetrahedral element E.g.: CCIsoTetra<xxxx>
Plane	CCIsoTriangle	Isoparametric triangular element E.g.: CCIsoTriangle<xxx>
	CCIsoQuad	Isoparametric quadrilateral E.g.: CCIsoQuad<xxxx>
	CCQ10	Four nodes quadrilateral element composed of two triangle isoparametric elements. This element must be defined by at least four corner nodes. E.g.: CCQ10<xxxx>
	CCQ10Sbeta	4 nodes quadrilateral element composed of two triangles. Four corner nodes must define this element. The material model at this element is evaluated at the element centre. The constitutive secant matrix evaluated at the element centre is used throughout the whole element to calculate internal forces of element. E.g.: CCQ10Sbeta<xxxx>
	CCSpring	Spring element defined by a single node. This element type should be used to define a spring support at given node.
	CCLineSpring	Line spring element defined by two nodes. This element type should be used for spring supports along solid element edges.
	CCPlaneSpring	Planar spring element defined by three nodes. This element type should be used for spring supports along faces of solid elements.
	CCIsoTruss	Isoparametric truss element. E.g.: CCIsoTruss<xx>
Special elements	CCIsoASymTruss	Isoparametric truss element for axisymmetric problems. The element contributes stiffness in the direction of its axis. Also, for adding radial stiffness, combine this element with the CCCircumferentialTruss or CCCircumferentialTruss2 element. E.g.: CCIsoASymTruss<xx>
	CCIsoGap	Gap/Interface element E.g.: CCIsoGap<xxxx>
	CCCircumferentialTruss	Circumferential truss element. This element is defined by only one node and is used in axisymmetric analysis to model circumferential reinforcement. It also contributes radial stiffness. E.g.: CCCircumferentialTruss
	CCCircumferentialTruss2	Circumferential truss element. This element is defined by two nodes and is used in axisymmetric analysis to model circumferential reinforcement. It is similar to the CCCircumferentialTruss element, however, it is “cross-sectional area” is equal to its length multiplied by its thickness. For adding stiffness also in the element’s axial direction combine this element with the CCIsoASymTruss element. E.g.: CCCircumferentialTruss2
	CCExternalCable	2D or 3D truss element to model the external prestress cables. The bar is anchored at one end and prestressed at the other. The intermediate nodes are deviators, where frictional force is defined, see external geometry definition. The whole bar must consist of one or more elements. All the elements must compose the same element group.
	CCBarWithBond	2D or 3D truss element for modelling reinforcement bars with specified cohesion with concrete. If exceeded, the bar will slip. The element type uses external cable geometry definitions to specify the appropriate solution parameters. The whole bar must consist of one or more elements. All the elements must compose the same element group.

Table 10.1: Available element types in ATENA [5] (Continued)

Type	Name	Description
	CCAhmadElement33L CCAhmadElement32L CCAhmadElement33H CCAhmadElement32H CCAhmadElement22S	3D shell elements. The first and the second digits in the element name specify the number of integration points for element bending and shear energy. The digit three notes that the element is integrated in 3 IPs in X-dir * 3 IPs in Y-dir * number of layers. The last letter L, H and S stands for 9-nodes Lagrangian element, for 9 nodes Heterosis element and 8 nodes Serendipity element. See theoretical manual for more details. All elements must use a 3D material and LayeredShell geometry! They specified by 16 nodes, 8 for top and 8 for bottom surface similar to brick elements. The top and bottom middle points for Lagrangian and Heterosis elements (for the bubble functions) are generated automatically. At each node, the elements have 3 degrees of freedom. As top and bottom node have altogether, 6 DOFs and shell theory uses only 5 DOFs per shell node, the z-displacement of the bottom node is automatically constrained during the execution.
	CCBeamNL	3D non-linear beam element. The element uses quadratic interpolation along its axis so that it can have a curvilinear shape. Similar to the implemented CCAhmad elements it is also input to a 3D hexahedral box. Nevertheless, the usual axial nodal points are available (e.g. for checking resulting deformations and rotations. They are generated automatically.

Table (10.2) explains the available material types in ATENA.

Table 10.2: Material types in ATENA [5]

Material and sub-command	Description
LINEAR_ELASTIC_ISOTROPIC	Linear elastic isotropic materials for 1D, Plane Stress, Plane Strain, Axisymmetric and 3D analyses
3DCEMENTITIOUS	Material suitable for rock or concrete like materials.
3DNONLINCEMENTITIOUS	Materials suitable for rock or concrete-like materials. Enhanced 3DCEMENTITIOUS material.
3DNONLINCEMENTITIOUS2	Materials suitable for rock or concrete like materials. This material is identical to 3DNONLINCEMENTITIOUS except that this model is fully incremental.
3DNONLINCEMENTITIOUS2VARIABLE	Materials suitable for rock or concrete-like materials. This material is identical to 3DNONLINCEMENTITIOUS2 except that selected material parameters can be defined using time or load step function.
3DNONLINCEMENTITIOUS2USER	Materials suitable for rock or concrete-like materials. This material is identical to 3DNONLINCEMENTITIOUS2 except that the selected material laws can be defined by user curves.
3DNONLINCEMENTITIOUS2SHCC	Strain Hardening Cementitious Composite material. Material suitable for fibre reinforced concrete, such as SHCC and HPFRCC materials.
3DNONLINCEMENTITIOUS2FATIGUE	Based on the 3DNONLINCEMENTITIOUS2 material, suitable for fatigue analysis of rock or concrete like materials.
3DNONLINCEMENTITIOUS3	Materials suitable for rock or concrete-like materials. This material is an advanced version of 3DNONLINCEMENTITIOUS2 material that can handle the increased deformation capacity of concrete under triaxial compression. Suitable for problems including confinement effects.
VON_MISES_PLASTICITY	Plastic materials with Von-Mises yield condition, e.g. suitable for steel.
DRUCKER_PRAGER_PLASTICITY	Plastic materials with Drucker-Prager yield condition.
USER_MATERIAL	User-defined material (derived from elastic isotropic). The user provides a dynamic link library.
INTERFACE_MATERIAL	Interface material for 2D and 3D analysis.
REINFORCEMENT	Material for discrete reinforcement.
REINFORCEMENT_WITH_CYCLING_BEHAVIOR	Material for discrete reinforcement subject to cycling loading.
SMEARED_REINFORCEMENT	Material for smeared reinforcement.
MICROPLANE	Bazant Micro-plane material models for concrete
COMBINED_MATERIAL	This material can be used to create a composite material consisting of various components, such as for instance concrete with smeared reinforcement in various directions. The unlimited number of components can be specified. Output data for each component are then indicated by the label # <i>i</i> ; where <i>i</i> indicates a value of the <i>i</i> -th component.
BEAM_RC_MATERIAL	Material for (reinforced) structures modelled by CCBeal material
try_reduce_MyMz_keep_NBEAM_REINF_BAR_MATERIAL	Material for reinforcement bar used in solids modelled by either BEAM_RC_MATERIAL or BEAM_MASONRY_MATERIAL material.

10.12. CREEP AND SHRINKAGE ANALYSIS

The time-dependent analysis of the reinforced concrete structures implements creep and shrinkage data. The main difference between the usual static and creep analysis is the inclusion of the structural response integration in time, in the creep analysis. Time factor also appears in

the constitutive equations such as material models by implementing the models to predict the creep and shrinkage behaviour of concrete.

There are different models of creep used in ATENA to analyses the time-dependent cracking and deflection of reinforced concrete member. Table (10.3) shows the existing creep models in ATENA used to assign to the concrete base material in finite element analysis.

Table 9.3: Material models for creep in TANEa [5]

Material	Description
CCModelB3	Bazant-Baweja B3 model
CCB3Improved	Model same as the above with support for specified time and humidity history
CCModelBP_KX	Creep model developed by Bazant-Kim, 1991
CCModelCEB-FIP	Creep model advocated by CEB-FIP 1978
CCModelACI_78	Creep model by ACI Committee in 1978
CCModelCSN731202	Model recommended by CSN731202
CCModelBP1	Full version of the creep model developed by Bazant-Panulla
CCModelBP2	Simplified version of the above model
CCModelGeneral	Creep model for direct input of material compliance, strength and shrinkage at times typically measured in a laboratory
CCModelFIB_MC2010	Model by CEB-fib bulletin 65 from the year 2010
CCModelEN1992	Creep model by Eurocode EN1992.1.1-2006

Completeness and continuity are two aspects of the so-called consistency condition between the discrete and mathematical models, while, consistency and stability imply convergence in FEA.

In FEA, it is crucial to select an element that is compatible with the assigned element type. Element types that are incompatible with the material type of the concrete member may cause some convergence problems in solving the non-linear equations. For a structural FE, the stiffness matrix includes the material behaviour and geometric information that points to the resistance of the element to deformation when subjected to loading. The shape functions of the element should provide displacement continuity between elements to ensure that no material gaps appear as the elements deform.

Table (10.4) explains the compatibility between the type of elements and the chosen materials in FEA in ATENA.

Table 10.4: Element type and material compatibility [5]

Material	CCIso-Brick	CCIso-Quad	CCIso-Triangle	CCQ10	CCQ10Sbeta	CCSpring	CCLine-Spring	CCPlane-Spring	CCIso-Truss	CCIso-Gap	CCCircumferentialTruss
CC1DElastIsotropic						✓	✓	✓	✓		✓
CCPlaneStressElastIsotropic		✓	✓	✓	✓	✓	✓	✓	✓		✓
CCPlaneStrainElastIsotropic		✓	✓	✓	✓	✓	✓	✓	✓		✓
CC3DElastIsotropic	✓	✓	✓	✓		✓	✓	✓	✓		✓
CCASymElastIsotropic		✓	✓	✓		✓	✓	✓	✓		✓
CC3DBiLinearSteelVonMises		✓	✓	✓	✓	✓	✓	✓	✓		✓
CC3DCementitious	✓	✓	✓	✓		✓	✓	✓	✓		✓
CC3DNonLinCementitious	✓	✓	✓	✓		✓	✓	✓	✓		✓
CC3DNonLinCementitious2	✓	✓	✓	✓		✓	✓	✓	✓		✓
CC3DNonLinCementitious2User	✓	✓	✓	✓		✓	✓	✓	✓		✓
CC3DNonLinCementitious2Variable	✓	✓	✓	✓		✓	✓	✓	✓		✓
CCSBETAMaterial		✓	✓	✓		✓	✓	✓	✓		✓
CC3DDruckerPragerPlasticity	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓
CC2DInterface										✓	
CC3DInterface										✓	
CCReinforcement	✓	✓	✓	✓	✓						
CCCyclingReinforcement						✓	✓	✓	✓		✓
CCSmearedReinf	✓	✓	✓	✓	✓						
CCCircumferentialSmearedReinf	✓	✓	✓	✓		✓	✓	✓	✓		✓
CCMaterialWithVariableProperties	✓	✓	✓	✓		✓	✓	✓	✓		✓
CCMaterialWithTempDepProperties	✓	✓	✓	✓		✓	✓	✓	✓		✓
CCMaterialWithRandomFields	✓	✓	✓	✓		✓	✓	✓	✓		✓
CCCombinedMaterial	✓	✓	✓	✓		✓	✓	✓	✓		✓

10.13. MODELING OF SHRINKAGE BEHAVIOUR IN ATENA

There are three different methods in TAENA to implement the shrinkage effect in FE models:

- Reducing the tensile strength of concrete and/ or reducing the fracture energy. This option takes the reduction of tensile strength due to cracking of concrete;

- Introducing the shrinkage as initial strain due to constant volume reduction throughout the whole structure volume. This type of initial constant strain is typically in the range of 150 to 450 micro-strains;
- Applying the real shrinkage data of concrete in creep analysis module. Since this option takes into account the exact shrinkage data, it gives the most accurate FEA results;

Figure (10.18) shows a typical shrinkage data input box in ATENA-CREEP analyses.

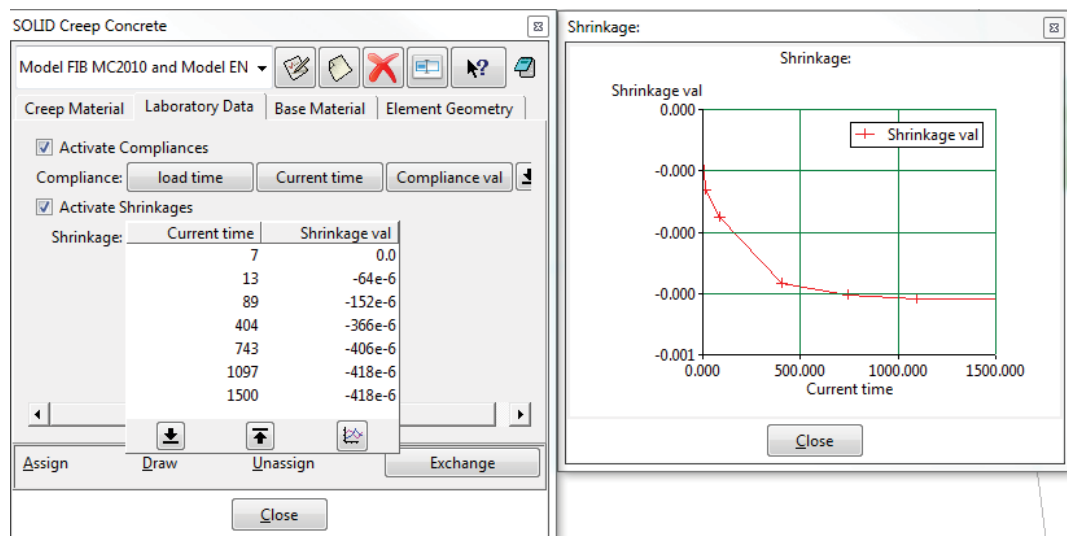


Figure 10.18: Shrinkage input data implementation in ATENA-CREEP analysis

10.14. SELECTION OF COMPATIBLE ELEMENTS WITH EXPERIMENTAL DATA

The obtained results from experimental studies have been verified by 2-D and 3-D FE analysis in the GID-ATENA program.

The monitored parameters of analysis of one-way slabs in 2D and 3D environments were deflection and cracking behaviour under instantaneous loading conditions. Additionally, the time-dependent effects of creep and shrinkage on the long-term deflection over 500 days were investigated. The cracking pattern along the slab span and the time-dependent deflection variation were also investigated in both 2D and 3D models. According to the experimental program, two different levels of instantaneous and sustained loading were applied on the slabs in 2D and 3D models to monitor the cracking and deflection behaviour when subjected to instantaneous and time-dependent behaviour.

Based on the previously explained properties of materials and element types implement in ATENA, the quadratic element type in Table (10.4) was selected in the 2-D analysis. Compared to the analysis of other available element types, the selected element showed the best compatibility with the experimental data of instantaneous deflection and behaviour.

For 3D analysis, CCIsoBrick element in Figure (10.15) was the most appropriate element to cover the analysis objectives and applied material types in this study. To decrease the computation time of the model processing, the CCIsoBrick element was configured in 8 nodes. Considering the higher accuracy of the 20-nodded CCIsoBrick element compared to 8-nodded element in the model, to compensate for the reduced efficiency of 8-nodded element, a finer structural mesh size together with the quadratic type option have been utilized in meshing the model before processing. The finer structural mesh size and 8-nodded CCIsoBrick element provide an optimized analysis in terms of the processing time and required efficiency.

The effect of mesh size on the FE analysis results is described in the next section of this study.

10.15. EFFECT OF MAXIMUM AGGREGATE SIZE AND AGGREGATE INTERLOCK

The aggregate interlock transfers the shear loads across the crack faces. It is reliant upon the frictional property of the aggregates along the crack faces and the crack opening. Type and maximum size of the aggregate, crack tortuosity, stiffness and strength of concrete, boundary conditions and loading characteristics affect the transferred shear throughout the crack surface due to the aggregate interlock [31, 32].

According to Figure (10.19), in GID-ATENA, it is possible to include the effects of aggregate interlock and maximum size of coarse aggregate on the shear and compressive strength of the reinforced concrete structure in FE analysis. For realistic analysis and the most accurate simulation of the experimental conditions, the aggregate interlock is activated during the analyses. Also, the maximum size of aggregate is set to 10 mm. The aggregate size is in agreement with the experimental phase of this study.

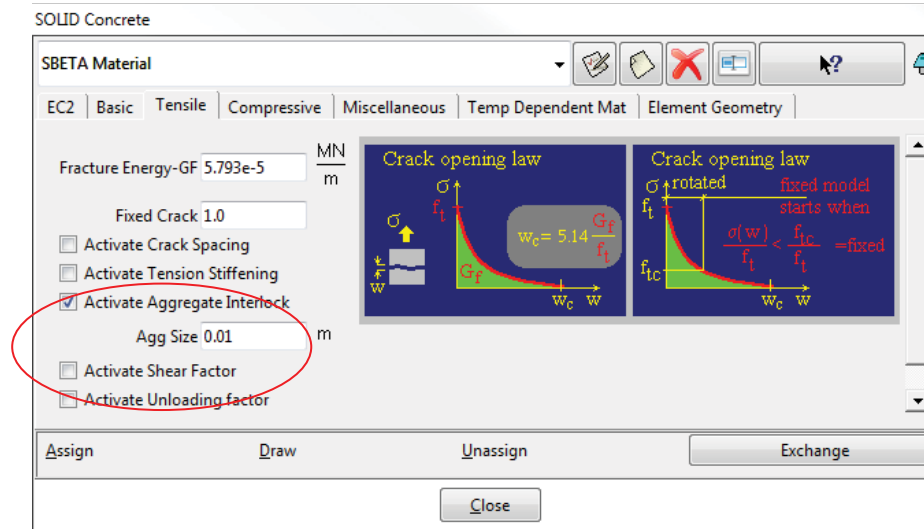


Figure 10.19: Implementation of aggregate interlock and maximum size of aggregate in FEA in ATENA

10.16. EFFECT OF MESH SIZE

The mesh quality of FE has a very significant influence on the quality of the analysis results. The size and type of the mesh affect the FEA in the following ways:

- Considering the accuracy of the FEA, proper selection of size and type of mesh is an effective way to make substantial savings in computational time and computer memory;
- The pattern of crack propagations depends on the main directions of the mesh. However the problem is less important in discrete elements [10];
- Strain localization in the post-peak softening of concrete is greatly influenced by the mesh size and type;
- In a FEA, the overall non-linear response of a reinforced concrete member subjected to bending can significantly be affected by the mesh size and type. This effect is more critical in using the smeared crack modelling [33];
- Compatibility of the mesh size and type in contact surfaces like the simulation of the steel-concrete bond is inevitable in convergence and analysis of the model.

The effect of FE mesh sizes on the analysis of the instantaneous and time-dependent behaviour of slabs under two different load levels is investigated. Displacements at midspan and third point and computational time are the two main categories in this regard.

10.17. WIDTH AND HEIGHT OF THE SELECTED ELEMENTS

Utilization of each type of element in GID-ATENA requires the specified quantity or width and height of element in each direction of the model to get the most efficient models of structures by FE analysis.

In FE analysis, the faces of 2-D elements build the 3-D elements. The CCIsoBrick element was utilized in the 3-D analysis of the slabs and CCIsoQuad in Table (10.4) is its plane version. Considering the thickness of slabs and the position of the reinforcement layer, the minimum number of 8 elements through the thickness of the slab is selected. The element type was CCIsoQuad. In other words, the minimum dimension of CCIsoQuad element throughout the slab thickness (161 mm) was selected as 20mm. The quantity of 6 to 8 finite elements per height demonstrates sufficient accuracy models of reinforced concrete slabs.

10.18. EFFECT OF LOADING PADS IN PROGRAMMING

Loading pads are interface elements between the heavy loading blocks and the main structure to be loaded. The loading pads ensure that the load transfer is correct through the width of the pad. Therefore, the loading pads eliminate any chance of fretting at the edge due to the rotation of the slab.

The interface between the loading pads and the slab is a type of contact element. Generally, in the iterative scheme, the contact conditions in FEA model are classified into three groups: [34]

- Open condition; gap remains open;
- Stick condition; gap remains closed, and no sliding motion occurs in the tangential direction;
- Sliding contact; gap remains closed, and the sliding takes place in the tangential directions.

The loading pads transfer the applied loading to the slab, however the stiffness of the pad is transmitted into the slab as displacement of the mesh within each element is linked to the contact surface. This stiffness transition causes the localized stress and strain concentration, due to the increased stiffness in some areas.

To simulate the real behaviour of the experimental program, the pads under the loading block are also implemented in the slab models. Figure (10.20) shows the loading pads in real test

specimens and the simulated pads in GID-ATENA program. These pads are modelled in both 2D and 3D models.

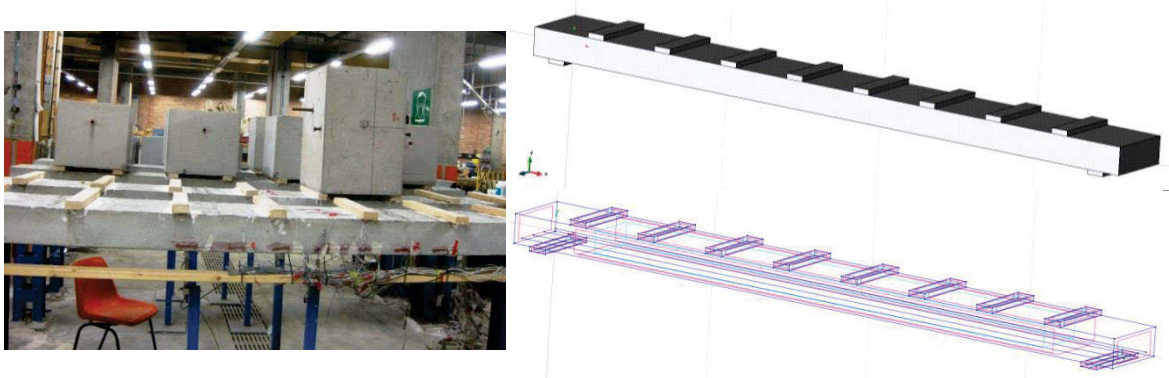


Figure 10.20: Simulated and real loading pads

10.19. ADVANTAGES OF SLABS SYMMETRY IN NUMERICAL ANALYSIS

The investigated slabs are in the shape of a long prism, and the position and condition of supports, reinforcement arrangement and loading pads distribution are symmetric all over the slabs. Therefore, it is possible to analyse half of the model to save the processing time, keeping the mesh quality high and achievable efficiency of the model.

To simulate the whole slab behaviour by a half slab, the boundary conditions in the FEA should be set properly. According to the slab geometry and support conditions, there is no rotation at the mid-span cut section. Also, the lateral transition in “z” direction and longitudinal movement along the span length in “x” direction in that section are supposed to be zero.

The tensile stress at the extreme point of the half-slab (mid-span of whole slab) is a crucial monitoring point in this study. Hence, the CCIsoBrick element in Table (10.2) can be used with high accuracy to mesh the half-slab model. The optimum width and height of the element are defined based on the sensitivity of FEA to the mesh size in 3D analysis.

Figure (10.21) shows the general view of the half-slab model in GID environment. This model is the same for all four slabs in this study.

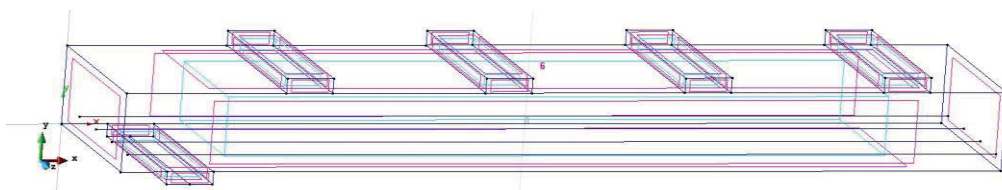


Figure 10.21: Half-slab model of slab in GID environment

GID-ATENA is a FE-based software system explicitly developed for the non-linear analysis of reinforced concrete structures. ATENA is used for the analysis of the model, while the program GiD is used for the data preparation, modelling of the structure and the mesh generation.

GID is a universal, adaptive and user-friendly pre and post-processor for numerical simulations in science and engineering. It has been designed to cover all the common needs in the numerical simulations field from pre to post-processing including: geometrical modelling, effective definition of analysis data, meshing, and data transfer to analysis software, as well as the visualization of numerical results. The main advantage of using GID, together with ATENA is the reduced analysis time with higher quality of meshing for the simulated reinforced concrete structure.

In the pre-processing stage, GID is used to define the geometry of the structure, material properties, boundary conditions and analysis data including the loading steps, solving method of non-linear equation and the convergence method.

Three-dimensional regions are modelled by volumes in GiD. The embedded reinforcement is modelled as a line and is not usually connected to any surface or volume.

10.20. ANALYSIS OF SLABS UNDER INSTANTANEOUS LOADING

10.20.1.Process of the modelling

In the two-dimensional environment of GID-ATENA, two types of analysis have been applied on the simulated models of LWC slabs. The first type is non-linear static analysis, to study the cracking behaviour and deformation of slabs under the instantaneous load on day zero. The day zero, is the first day that the slabs were loaded. In other words, this type of analysis doesn't take into the account the time-dependent properties of concrete.

The study of cracking behaviour study in this research includes crack propagation and direction under instantaneous loading. Deflection study includes the maximum flexural deflection at the mid-span and, the deflection in the third point under instantaneous loading.

The version of the GID program used to model the structure in this study is 12.0.6 and the version of ATENA program linked to this GID program to analysis the models is V5.

10.20.2.Slab dimensions

Mesh size and number of elements are two crucial parameters in the FEA of reinforced concrete structures. Finer mesh size and higher number of elements provide a more accurate analysis of structure. Considering the proper mesh size, whole structure with real boundary conditions, loading pads and supporting pads is modelled in FE and analysed subjected to two different loading levels. Figure (10.22) shows the details and geometry of the slab model built in the GID program and the applied mesh size. Two points in the mid-span and the third point are monitored for instantaneous deflection.

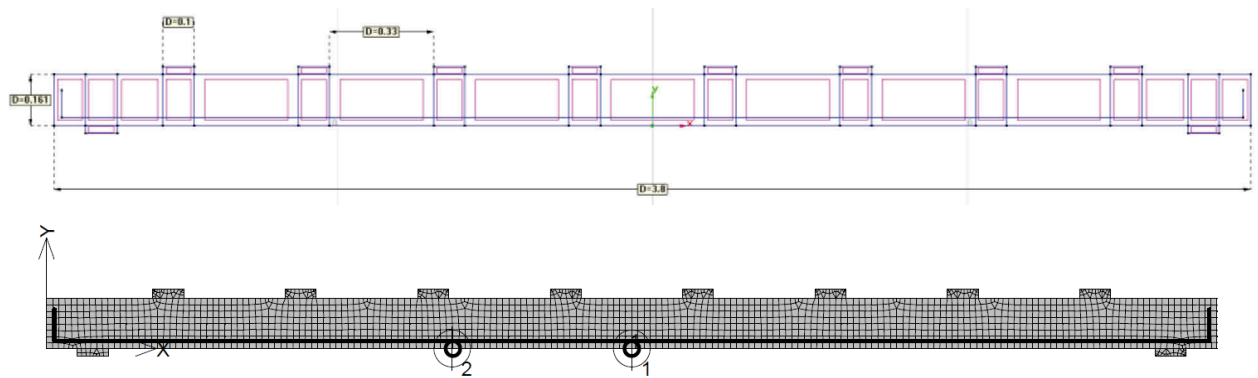


Figure 10.22: Details, geometry and meshing of the LWC slabs in GID program

10.20.3.Basic material properties

According to the experimental results presented in Chapter 5, Tables (10.5) and (10.6) explain the basic material properties of concrete and reinforcing steel bars implemented in the FE models.

Table 10.5: Mechanical properties of lightweight concrete

Property	Age	
	14 days	28 days
Strength class in ATENA	25/30	30/37
Elastic modulus (GPa)	17.25	19.48
Compressive strength (MPa)	27.93	30.89
Tensile strength (MPa)	2.6	2.89
Strain at peak stress	0.000098	0.00011
Type of tension stiffening	Exponential	Exponential
Crack model	Fixed	Fixed

Table 10.6: Mechanical properties of steel bars

Property	Value or type
Elastic modulus (GPa)	200.553
Yield strength (MPa)	538
Hardening	Perfectly plastic
Density (kg/m ³)	7850

Additionally, the proposed models of material properties in Chapter 6 are utilized in ATENA models and the results are compared.

According to the existing material models in ATENA in Tables (10.1) and (10.2), the following models can be used to simulate the concrete properties in this study.

- **CC3DCementitious material:**

This material model assumes a linear response of compressive and tensile concrete, up to the point when the failure envelope is reached. This material model doesn't consider the hardening regime.

- **CC3DNonLinCementitious material:**

In contrast with CC3DCementitious, this material model assumes a hardening regime before the compressive strength is reached.

- **CC3DNonLinCementitious2 material:**

Except the purely incremental formulation, this material model is equivalent to CC3DNonLinCementitious. Therefore, it is proper to apply for the creep calculation, or when the properties of material require some modification during the analysis.

- **CC3DNonLinCementitious2Variable material:**

In addition to the included properties in CC3DNonLinCementitious2, this model contains the history evaluation law's definition of the material parameters.

- **CC3DNonLinCementitious2User material:**

In this model, the user can define the compression, tensile, shear and softening diagrams and all other required properties of the concrete.

Considering the existing models in GID-ATENA, CC3DNonLinCementitious2 model of concrete is used for a general analysis. It allows changing the material properties during analysis

and can consider the creep effect also. Figure (10.23) shows the table of implemented properties of this material model in ATENA. Some implemented values of the tables in the figure are the default values of the GID-ATENA program.

To model the reinforcement material in the FE model, a multi-linear law consisting of four lines is utilized to adopt to the experimental stress-strain test result of steel bars in Chapter 6. The implemented multi-linear law contains the elastic state, yield plateau, hardening state and the fracture point.

Considering two types of smeared and discrete reinforcement models in GID-ATENA, the stress-strain law is used to describe both the smeared and discrete models. However, to model the smeared reinforcement model, the reinforcement ratio in the section and angle of direction is required to implement in ATENA input data.

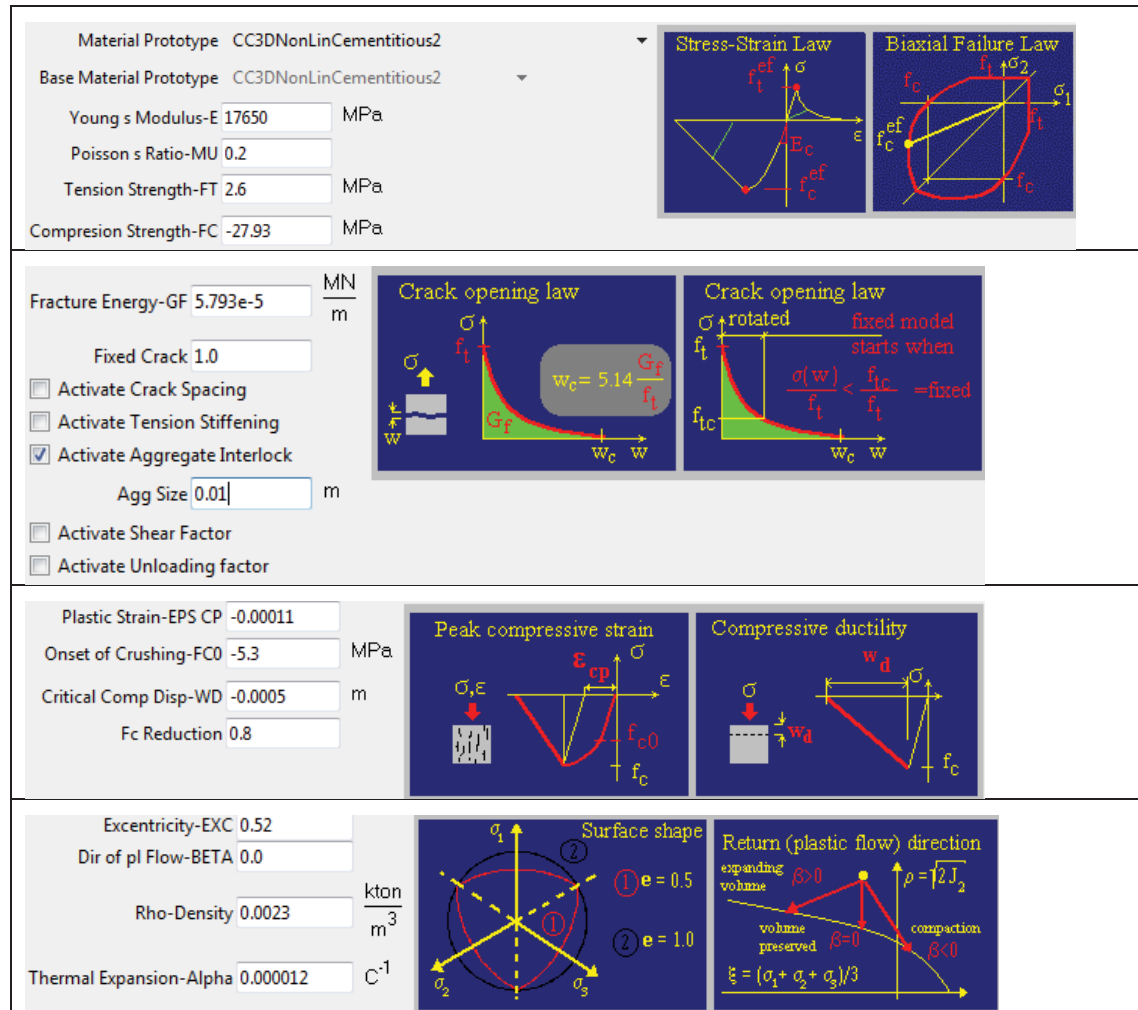


Figure 10.23: Material properties for CC3DNonLinCementitious2

Figure (10.24) shows the typical input data table to define the reinforcement in GID-ATENA. The table includes the stress-strain curve definition, and the bond-slip behaviour between the reinforcement and surrounded concrete.

Basic | Reinf Function | Miscellaneous | Element Geometry | Bar With Bond

Material Prototype CCReinforcement

☒ Calculator

Profile 12 mm

Number of Profiles 4

R01 To recalculate click 2x Update changes next to material box

Area 0.00045238934 m²

Reinf 01 Young's Modulus-E 2.0E+5 MPa

Figure 10.24: Parameters to define the reinforcement in GID-ATENA

Despite constant values of the basic parameters of concrete and reinforcement, there is wide range of parameters to define their other mechanical properties in GID-ATENA.

In the concrete material definition part of the ATENA program, the variable parameters in tensile, compressive and shear section are presented in Table (10.7). To find out about the most compatible adjustment of the parameters with the experimental results, different arrangements of the effective parameters in tensile, compressive and shear partition in the input data have been examined. In addition, the sensitivity of the model to various values of other constant parameters in tension, compression and shear has been investigated.

Table 10.7: Variable parameters in definition of typical concrete properties in GID-ATENA [5]

Property	Type of Tension Softening (TTS)					Crack model	
Tensile	Exponential	Linear	Local strain	SFRC	SFRC local strain	Fixed	Rotated
compressive	Type of Compression Softening (TCS)						
	Crush band			Softening modulus			
	Shear Retention Factor (SRF)			Tension-compression interaction (TCI)			
Shear	Fixed	Variable		Linear	Hyperbola A	Hyperbola B	

10.20.4. Solution parameters in instantaneous loading analysis

Table (10.8) describes the applied solution method and parameters of converging the analysis of the models subjected to instantaneous loading. The applied parameters provide the required convergence and the most accurate results with a reasonable processing time of the models.

Table 10.8: Solution parameters in instantaneous deflection analysis of LWC slabs

Solution method	Newton-Raphson
Stiffness/update	Tangent/each iteration
Number of irritations	80
Error tolerance	0.01
Line search	On, with iteration
Optimization	Sloan
Element shape smoothing	On

Figure (10.25) shows a typical convergence diagram of the slab analysis in GID-ATENA program.

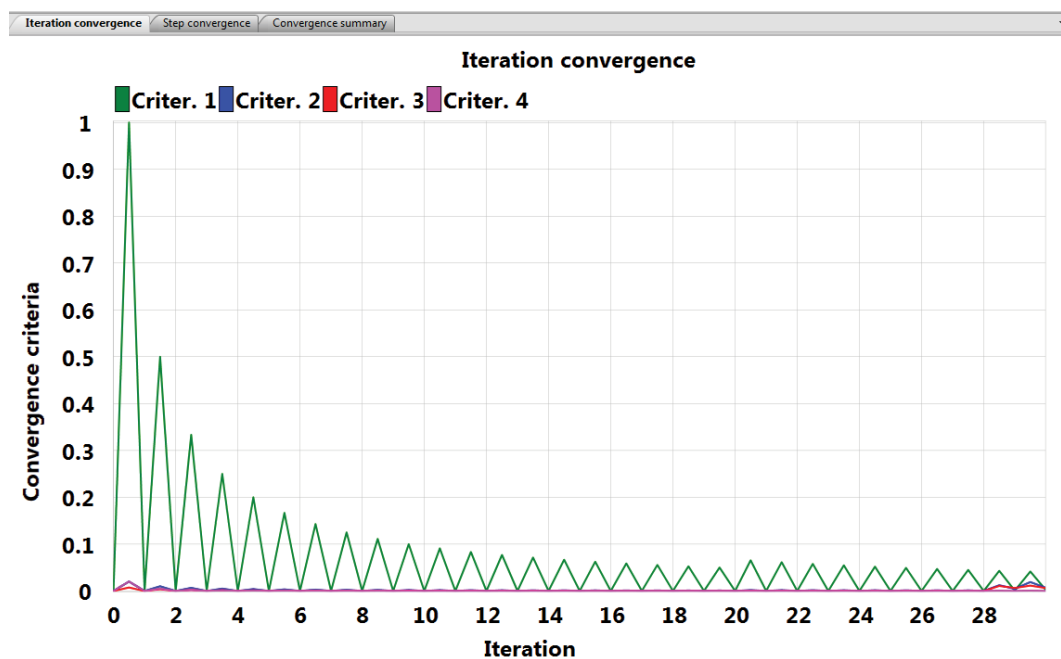


Figure 10.25: Typical convergence of the model analysis in ATENA program

According to Figure (10.25), there are four convergence criteria to solve the non-linear equations of the model.

Criteria 1: relative error in displacement

Criteria 2: relative error in residual forces

Criteria 3: absolute error in residual forces

Criteria 4: relative error in energy, i.e. basically a product of relative residual error and displacement error

10.20.5. Monitoring points of instantaneous deflection

The maximum deflection at mid-span and the deflection at the third point of the slab length were the main investigated parameters in the instantaneous loading of the slabs. Furthermore, the cracking behaviour of the slabs under two different levels of uniformly distributed loading has been studied. The ratio of the deflections at monitoring points is a matter of investigation as well.

There are two different levels of uniformly distributed loading on the simply supported slabs. The loading values of 30% and 40% of the ultimate loading capacity of the slab section were applied to each pair of slabs. In the experimental program, the loading levels of $0.3M_u$ and $0.4M_u$ are described as loading level 1 (LL1) and loading level 2 (LL2) respectively in all model analyses.

According to the experimental data in Chapter 6, the recorded instantaneous deflection at the monitoring points in two loading levels is presented in Table (10.9).

Table 10.9: Experimental instantaneous deflection values under LL1 and LL2 in monitoring points

Slab	Loading level (1)		Loading level (2)	
	$\Delta_1(\text{mm})$	$\Delta_2(\text{mm})$	$\Delta_1(\text{mm})$	$\Delta_2(\text{mm})$
LWC-1	11.84	11.19		
LWC-2	10.76	9.22		
LWC-3			6.33	5.43
LWC-4			5.98	5.16

As part of calibration of the ATENA program to analyse the LWC slabs, Table (10.10) describes different arrangements of the variables in the definition of SBETA material type.

Also, the obtained deflections in the monitoring points of the slab under loading level (1) are presented in the table.

Table 10.10: Variable arrangement of parameters in SBETA material and the obtained deflections in LL1

Case	Tensile		Compressive		Shear		Deflection (mm)	
	TTS	Crack model	f'_c reduction due to cracks	TCS	SRF	TCI	Point 1	Point 2
1	SFRC	Fixed	0.8	0.2	Fixed	Linear	5.98	5.2
2	exponential	Fixed	0.8	0.2	Fixed	Linear	11.8	10.3
3	linear	Fixed	0.8	0.2	Fixed	Linear	*	*
4	local strain	Fixed	0.8	0.2	Fixed	Linear	11.9	10.4
5	SFRC local strain	Fixed	0.8	0.2	Fixed	Linear	11.9	10.4
6	SFRC	Rotated	0.8	0.2	Fixed	Linear	5.98	5.19
7	exponential	Rotated	0.8	0.2	Fixed	Linear	*	*
8	linear	Rotated	0.8	0.2	Fixed	Linear	11.5	10
9	local strain	Rotated	0.8	0.2	Fixed	Linear	11.8	10.3
10	SFRC local strain	Rotated	0.8	0.2	Fixed	Linear	11.8	10.3
11	SFRC local strain	Rotated	0.7	0.2	Fixed	Linear	11.8	10.3
12	SFRC local strain	Rotated	0.6	0.2	Fixed	Linear	11.8	10.3
13	SFRC local strain	Rotated	0.5	0.2	Fixed	Linear	*	*
14	SFRC local strain	Rotated	0.4	0.2	Fixed	Linear	11.8	10.3
15	SFRC local strain	Rotated	0.2	0.2	Fixed	Linear	11.8	10.3
16	SFRC local strain	Rotated	0	0.2	Fixed	Linear	*	*
17	SFRC local strain	Rotated	0.3	0.2	Fixed	Linear	*	*
18	SFRC local strain	Rotated	0.1	0.2	Fixed	Linear	11.8	10.3
19	SFRC local strain	Rotated	0.8	0.2	Fixed	Linear	11.8	10.3
20	SFRC local strain	Rotated	0.8	0.2	Fixed	Linear	11.8	10.3

* Model processing failed

After calibrating the model with acceptable instantaneous deflection in agreement with the experimental results, other parametric studies have been performed, and the results are presented.

10.20.6.Effect of mesh size on deflection and cracking of slabs

After selecting the most compatible element type to allow for study of the effect of mesh size on the cracking pattern, the maximum size of cracks and the deflection of slabs, six distinct sizes of elements by meshing have been implemented in the finite element model. Except the changing mesh size, all other parameters such as material properties, loading values and support conditions were kept constant during the analysis. Table (10.11) explains the applied type of element and other preferences in the model analysis with different mesh sizes.

Table 10.11: Preferences in studying the effect of mesh size on instantaneous deflection

Parameter	Mesh type	Element type	Loading steps	Iteration number	Bond-slip model
Preference	Quadrilaterals	CCIsoQuad	5	40	Perfect connection

Figure (10.26) shows the deflection and crack propagation at the critical flexural area of mid-span by processing the model with different mesh sizes in 2D environment.

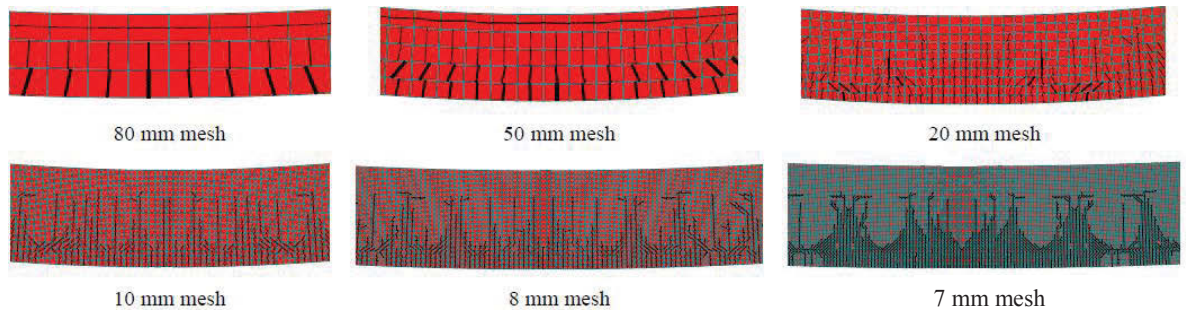


Figure 10.26: Crack propagation and deflection in the mid third area of the slab span by different mesh sizes

Figure (10.27) shows the crack propagation along the span length by applying different mesh sizes.

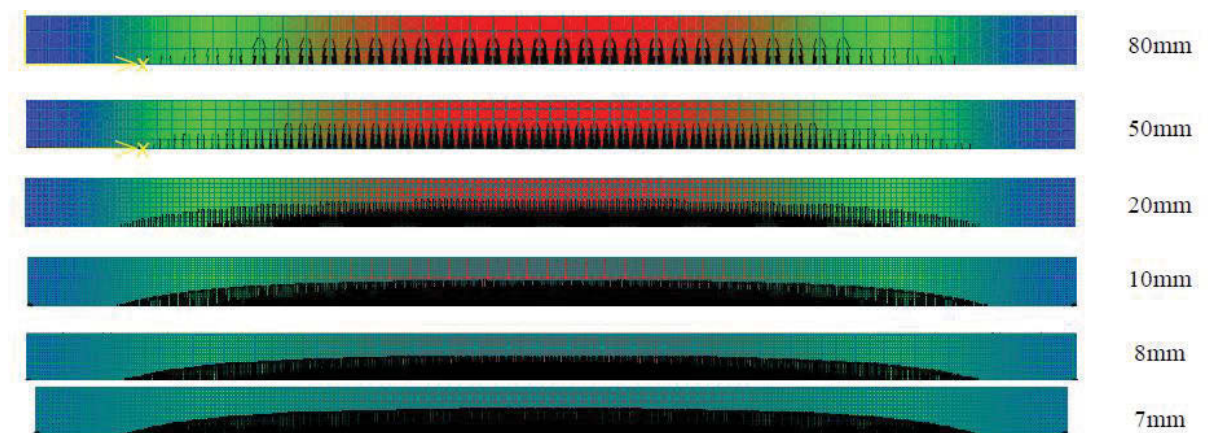


Figure 10.27: Cracking pattern and propagation in slab spans by different mesh sizes

According to Figures (10.26) and (10.27), the mesh size changes the propagation and direction of the cracking in the high moment region (middle third of span length) critical flexural range of the slab and whole slab length. It also changes the deflection curve of the slab under uniformly distributed loading and the maximum deflection of the slab at mid-span.

10.20.7.Effect of bond-slip model

To study the effect of bond-slip effect on the instantaneous deflection of slabs, the perfect bond between the steel bars and concrete is compared with the existing bond-slip models in

ATENA. Figure (10.28) describes the recorded maximum deflection of models with different mesh size to the maximum deflection of the model by 7 mm mesh. There were no considerable changes in the values of deflection and crack width in the models by finer mesh size, therefore, the 7 mm was selected as the basis for evaluation of the effect of bond-slip model. According to Figure (10.28) the impact of different bond-slip relationships on the models by different mesh sizes is evident.

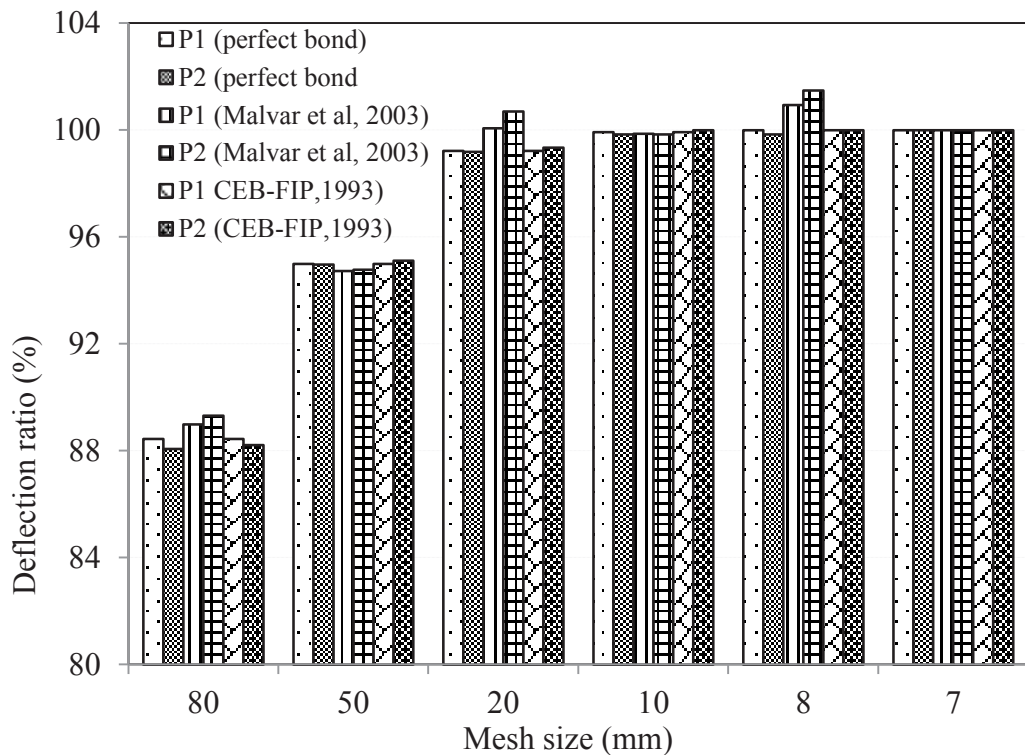


Figure 10.28: Ratio of deflection by different mesh sizes to deflection by 7 mm mesh

P1 and P2 in Figure (10.28) are deflections of slab at mid-span and the third point of span length respectively.

To study the effect of bond-slip behaviour on the maximum crack width under the instantaneous loading, the model with perfect bond is compared with the models with implemented bond-slip models in ATENA. Figure (10.29) shows the ratio of the maximum crack width in models with different mesh sizes to the maximum crack width in a model with 7 mm mesh size. There is no considerable difference in the maximum crack width of the models at midspan by perfect bond or bond-slip relationship between the steel bars and concrete.

Similar results were obtained for the monitoring point (P2) at the border of third middle of slab length; i.e. the inclusion of perfect bond or bond-slip between the steel bars and concrete did not show considerable changes in the crack width at the third point of slab length under

instantaneous loading. However, the maximum crack width in mid-span was about 6% smaller than the maximum crack width in the third point by analysis of models with different mesh sizes.

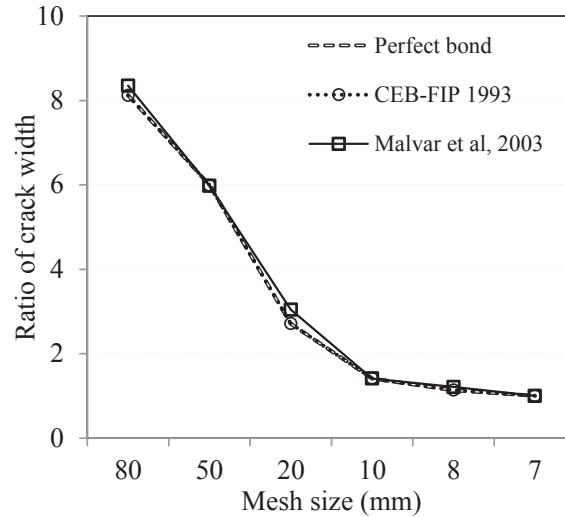


Figure 10.29: Ratio of the maximum crack with at different mesh sizes to crack with by 7 mm mesh

10.20.8. Ratio of deflection at monitoring points

Despite different values of deflection at mid-span (Δ_1) and deflection at the third point of slab length (Δ_2) in the models with perfect bond and bond-slip model, the ratio of Δ_1/Δ_2 is approximately constant.

The maximum crack widths at the monitoring points experience a similar behaviour. Despite different values of the maximum crack width at mid-span ($W_{\max-1}$) and the maximum crack width at the third point of slab length ($W_{\max-12}$) in the models with perfect bond and bond-slip model, the ratio of $w_{\max-1}/w_{\max-2}$ is approximately constant.

Inclusion of the bond-slip behaviour between the steel bars and concrete in the model changes the maximum deflection value at the mid-span and the maximum crack width. However, the changes are not considerable. In terms of deflection, in the model with perfect bond (no slip) between the steel bars and concrete, the maximum deflection at mid-span is about 4% less than the corresponding deflection in a model with bond-slip behaviour. However, this ratio may change by applying different bond-slip relationships in the model.

10.20.9. Effect of support condition on the instantaneous deflection

Different support conditions are investigated to monitor the ratio of Δ_1/Δ_2 . The ratio of deflection in the corresponding monitoring points of Simply Supported (SS) slab to the Fixed Ends (FE) slab is given in Table (10.12).

Table 10.12: Ratio of deflection in corresponding points of simply-supported and fixed ends slab

Parameter	$\Delta_{1-SS}/\Delta_{2-SS}$	$\Delta_{1-FE}/\Delta_{2-FE}$	$\Delta_{1-SS}/\Delta_{1-FE}$	$\Delta_{2-SS}/\Delta_{2-FE}$
Non-linear	1.17	1.22	2.88	3.02

According to Table (10.12), fixing the slab ends reduces the maximum instantaneous displacement of mid-span to 0.33 of the maximum deflection in a simply supported slab in the range of service load. However, the maximum crack width in the negative moment (fixed support) zone, especially in the slab with one-layered steel bars in tension zone, is considerably higher, compared to the maximum crack width in the simply-supported beam.

10.20.10. Sensitivity of deflection to early age properties of concrete

Conventionally, the mechanical properties at age 28 days are considered as the representatives of the concrete material. The slabs in the experimental part of this study have been subjected to loading at age 14 days. To investigate the effect of concrete properties at different ages on the instantaneous cracking and deflection behaviour, the slab model is analysed by implementing the concrete properties at 14 and 28 days.

The measured values of compressive strength and modulus of elasticity at the age of 14 days are 90% and 87% of those values at age 28 days respectively. However, in the range of service load, deflection of the monitoring points at 14 days is 10% higher than those at 28 days. Surprisingly, the non-linear static analysis of the slab showed about 30% higher value of the maximum crack width at 14 days under the same support conditions and applied load values.

Deflection (Δ_{i-t}) and the maximum crack width (w_{max-t}) ratios of the simply supported LWC slab in the monitoring points, subjected to instantaneous loading by different properties of concrete at 14 and 28 days, is presented in Table (10.13).

Table 10.13: deflection ratio at the corresponding point by concrete properties at 14 and 28 days

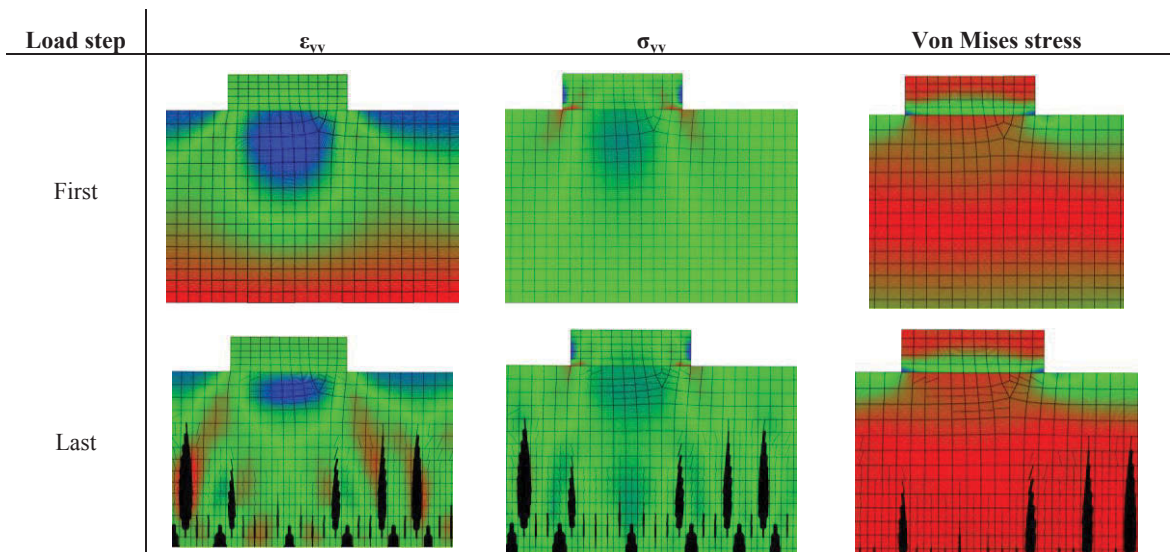
	$\Delta_{1-14}^* / \Delta_{1-28}$	$\Delta_{2-14} / \Delta_{2-28}$	$W_{\max-14} / W_{\max-28}$
Deflection ratio	1.1	1.1	0.73

In Table (10.13), Δ_{i-d} is deflection at monitoring point “i” with properties of concrete at the age of “t” days. $W_{\max-d}$ is the maximum crack width in the model with properties of concrete at age “t” days.

10.20.11. Effect of loading pads

In the FE models in this study, the loading pads have been implemented to simulate the real experimental conditions. According to the experimental conditions, the applied concrete blocks on the loading pads are heavy and huge, compared to the dimension of pads. Furthermore, the contact surface between the pads and the rough top surface of the slab was filled and prepared by plaster. Therefore, the contact condition between the loading pads and the slab surface is assumed to behave together; i.e. the loading pads are perfectly bonded to the slab.

Figure (10.30) shows the non-uniform distribution of the stress and strain around the contact area in different loading steps of the static non-linear analysis. The pads are modelled as flexible plates. There is no crack initiation and crack propagation in the first load step. However, the displacement value and the maximum crack width are increasing at higher loading values in higher load steps.

**Figure 10.30: Localized stress and strain distribution around the loading plate (flexible plates)**

Since the pads under the loading blocks were made of timber, the corresponding modulus of elasticity and mechanical properties of timber were assigned to the pads in the analysis of the slabs. However, some studies consider these types of plates as rigid plates in contact with the slab. Therefore, to monitor the effect of the elastic or rigid pad on the instantaneous deflection of slabs, different values of rigidity were assigned to the pads during the analysis of the model.

Figure (10.31) shows the corresponding parameters (localized stress and strain distribution) presented in Figure (10.30) for a rigid loading pads.

Comparison of the flexible and rigid loading pads in Figures (10.30) and (10.31) shows a considerable difference in the cracking pattern and distribution of the stress and strain around the pads in the tensile zone of the concrete. In case of the rigid pad, there are more and wider cracks around the pads due to higher levels of strain and stress transmitted from the rigid plate to the slab.

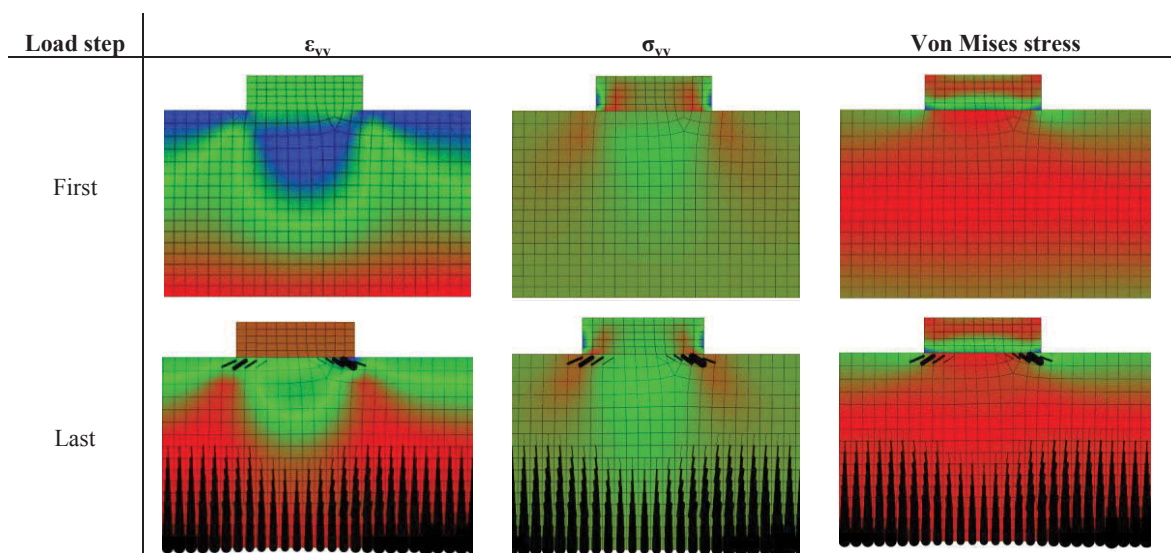


Figure 9.31: Localized stress and strain distribution around the loading plate (Rigid plates)

10.21. TIME-DEPENDENT ANALYSIS OF SLABS

The second type of analysis in GID-ATENA is the creep analysis to include the effects of creep and shrinkage on the time-dependent cracking and deflection behaviour of slabs. Since the experimental program has lasted for 500 days, the creep test was performed to monitor the slab behaviour from the first day of loading up to 500 days monitoring the effects of creep and shrinkage under the same sustained loading.

10.21.1. Modelling of the slab

Modelling of the structure for any type of problem is performed in the graphical environment of GID. However, there are different features and environments of ATENA to analyse the different models for various problems types. Figure (10.32) shows the pre-processing environment in GID and the implemented problem types in ATENA to analyse the model. In switching between the methods (problem types), it is possible to convert the data information (material properties, support and loading conditions and other analysis data) of the existing model to the new problem type.

According to Figure (10.32), there are different post-processing environments to analyse the model. To perform the creep analysis, the modelling process is similar to the process of static analysis. To process the model for time-dependent analysis, the problem type should be changed to ATENA-Creep.

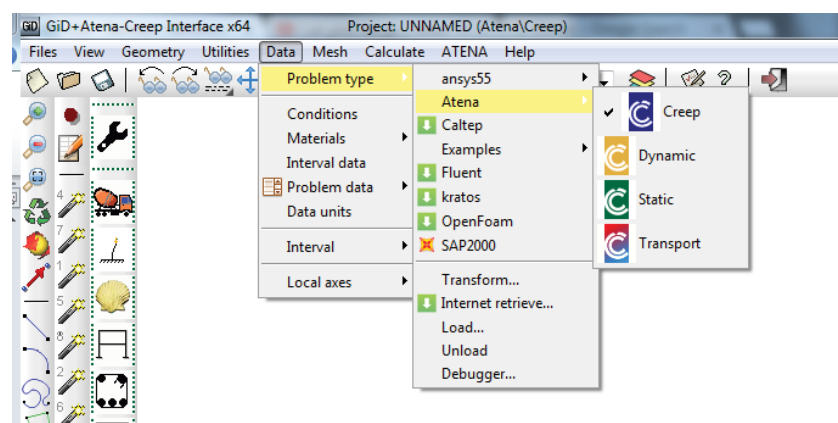


Figure 10.32: Different problem types in ATENA to analyse the model

Since the modelling of slab structures for both ATENA-Static (instantaneous loading analysis) and ATENA-Creep (time-dependent analysis) problem types are accomplished in

GID, the major part of the modelling process and preferences in 2D and 3D environment are similar. The main differences between the pre-processing stage in static and creep analysis in GID-ATENA are the implementation of the creep behaviour in the material properties and the addition of the interval data (loading step) for creep application.

10.21.2. Basic material properties

Based on the calibration process of selecting the most compatible material in the model, CC3DNonLinCementitious2 material model (Cementitious2) is chosen as the base material for the solid concrete in the creep analysis of the slabs under different loading conditions.

The creep behaviour of concrete in the model can be implemented in direct or indirect ways. In the direct method, the user can develop the creep model of concrete and specify all other related details manually. However, there are some previously developed creep models implemented in the ATENA program to facilitate the creep application in the model. Figure (10.33) shows the existing creep models in ATENA.

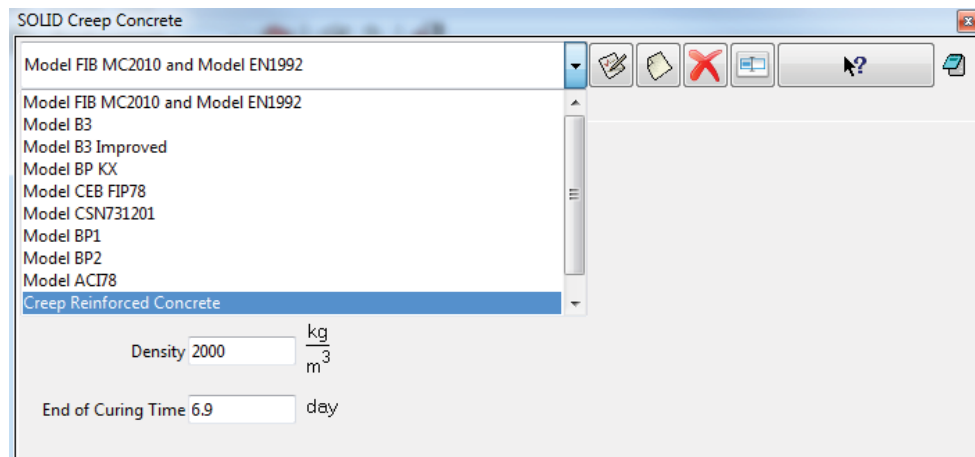


Figure 10.33: Implemented creep models in ATENA-creep problem type

The creep models in Figure (10.33) have been developed for different conditions of physical and mechanical properties of concrete, mixture proportioning, environment conditions and loading history. Therefore, they may vary in accurately predicting the creep behaviour and the required parameters of the model.

Table (10.14) describes the required parameters for each type of the creep models in ATENA.

Table 10.14: Input parameters in the creep and shrinkage models implemented in ATENA

Model name	B3	B3 improved	BP-KX	CEB	ACI	CSN	BPI	BP2	General
Concrete type	✓	✓	✓		✓	✓	✓	✓	
Thickness (Volume/Surface)	✓	✓	✓	✓	✓	✓	✓	✓	
Strength f_{cyl28}	✓	✓	✓	✓	✓	✓	✓	✓	
Strength f_{cyl028}		✓							
Fracture energy G_{f28}		✓							
Tensile strength f_{t28}		✓							
Young modulus E_{c28}	✓	✓	✓	✓		✓			
Ambient humidity (h)	✓	✓	✓	✓	✓	✓	✓	✓	
Aggregate / cement ratio	✓	✓	✓		✓		✓	✓	
Water / cement ratio	✓	✓	✓		✓		✓	✓	
Fine/total aggregate ratio (s_a)							✓	✓	
Coarse/fine aggregate ratio (g_s)							✓	✓	
Fine aggregate/cement ratio (s_c)							✓	✓	
Shape factor	✓	✓	✓				✓	✓	
slump					✓				
Air content					✓				
Cement mass							✓		
Concrete density	✓	✓	✓		✓				
Curing type	✓	✓	✓		✓		✓	✓	
End of curing	✓	✓	✓	✓	✓	✓	✓	✓	
Thermal expansion coefficient (α_t)		✓							
Autogeneous shrinkage at infinity time ($\varepsilon_{a,\infty}$)		✓							
Half-time of autogeneous shrinkage (τ_a)		✓							
Time of final set of cement (t_s)		✓							
Final self-desiccation relative humidity ($h_{a,\infty}$)		✓							
I/D	Current time t	✓	✓	✓	✓	✓	✓	✓	✓
	Load time t'	✓	✓	✓	✓	✓	✓	✓	✓
Total water loss w	✓	✓			✓				
Improvement	Water loss w(t)	✓	✓						
	Shrinkage $\overline{\varepsilon^0}(t)$	✓	✓	✓	✓	✓	✓	✓	
	Compliance $\phi(t, t')$	✓	✓	✓					
History	Humidity h(t)		✓	✓		✓			
	Temperature T(t)		✓	✓		✓			
Direct	Compliance $\phi(t, t')$								✓
	Shrinkage $\overline{\varepsilon^0}(t)$								✓
	Strength $f_{cyl}(t)$								✓

10.21.3.Slab dimensions

All details and geometry of the slab specimens in the ATENA-Creep analysis are similar to the models in ATENA-Static. In addition, the applied element type for concrete, reinforcement and loading pads in creep analysis are similar to those in static analysis. Based on the sensitivity analysis of the time-dependent deflection to the mesh size, a minimum of 8 elements per

thickness of the slab are selected to mesh the slab structure. Mesh size of the pads is in agreement with the mesh size of the main concrete structure.

10.21.4. Monitoring points for long-term deflection

The point at mid-span of the slab is selected as the monitoring point in creep analysis. The time-dependent deflection variation under the effects of creep and shrinkage over 500 days is verified by the experimental results. Moreover, development of the cracking pattern along the slab span is monitored under the loading levels and simultaneous effects of creep and shrinkage.

10.21.5. Loading steps

The creep analysis in ATENA contains two types of applied loads. The first type is the sustained service load, and the second type is the creep implementation in the model. To avoid any loading shock and the unexpected effects of sudden loading on the time-dependent behaviour of slabs, application of the loads on the model was accomplished in multiple steps. Creep or other non-linear effects of concrete will cause a redistribution of stresses in the structure. For an accurate processing, a sufficient number of time steps is required. The time spacing is created by the number of the time steps. Determination of the time steps and time spacing depends on the type of the analysed structures as well as on the choice of time units.

Figure (10.34) shows the input table in the ATENA-Creep environment to define the interval data for loading types and to assign the proper number of the loading steps for each interval.

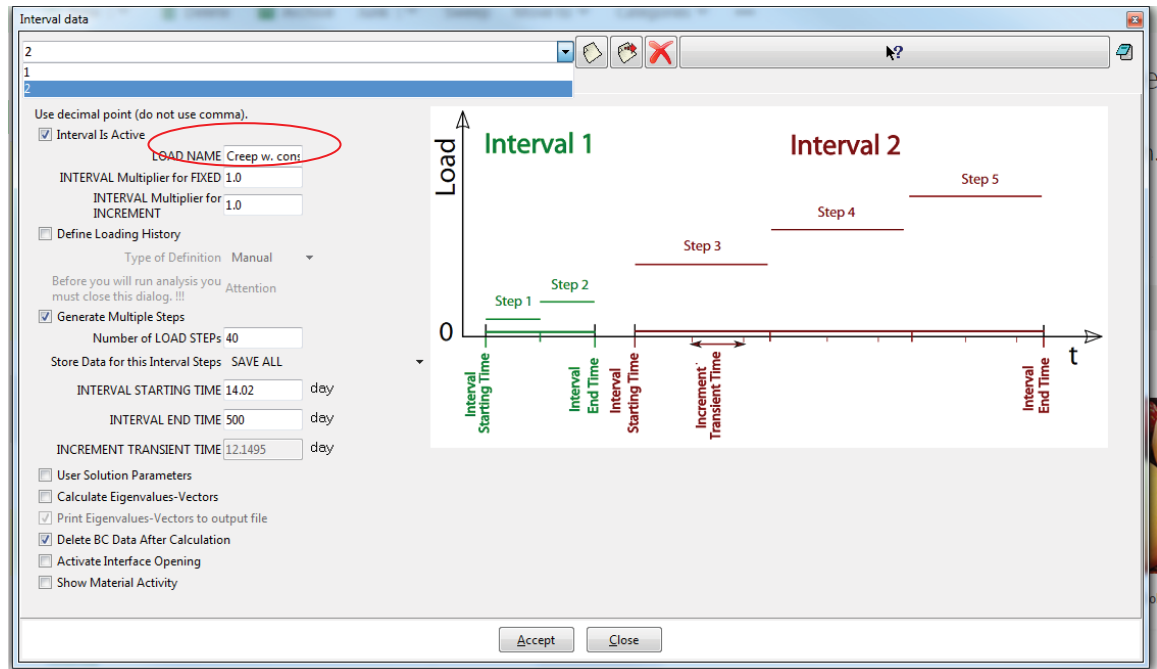


Figure 10.34: Input data table to define the interval data in creep analysis

The results of creep analysis can be plotted by the maximum deflection of the slab at mid-span versus the age of concrete structure after loading. Figure (10.35) shows a typical deflection-time diagram in the ATENA-Creep environment.

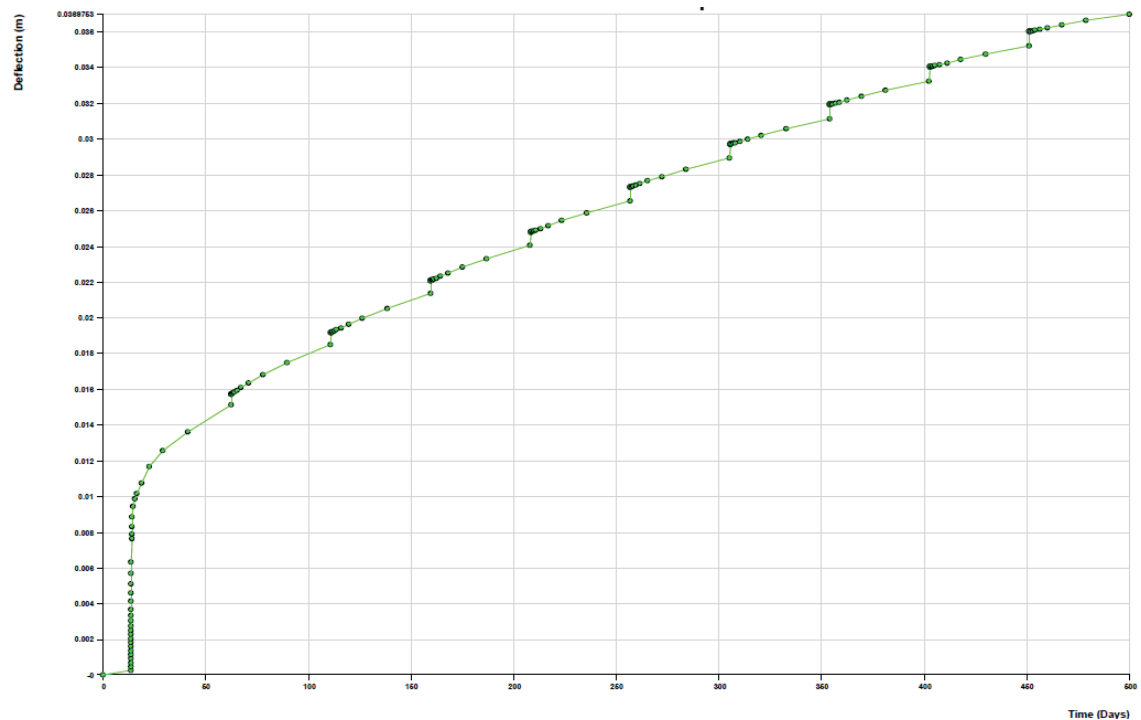


Figure 10.35: Typical deflection-time diagram of LWC slab in ATENA environment

10.21.6. Verification of the proposed models

Based on the experimental results, the models presented in Equations (6.5), (6.7) and (6.8) of Chapter 6 have been developed in this study to predict the shrinkage and creep of lightweight concrete containing expanded polystyrene beads respectively. Details of the utilized data and verification of the developed models have been explained in Chapter 6 of this study.

The proposed models for creep and shrinkage behaviour are verified by comparing the experimental and numerical analysis of deflection-time diagrams in the FE models.

Figure (10.36) compares the experimental deflection-time diagram in loading level 1 (LL_1) with predictions of the model, determined by utilizing the proposed creep and shrinkage models.

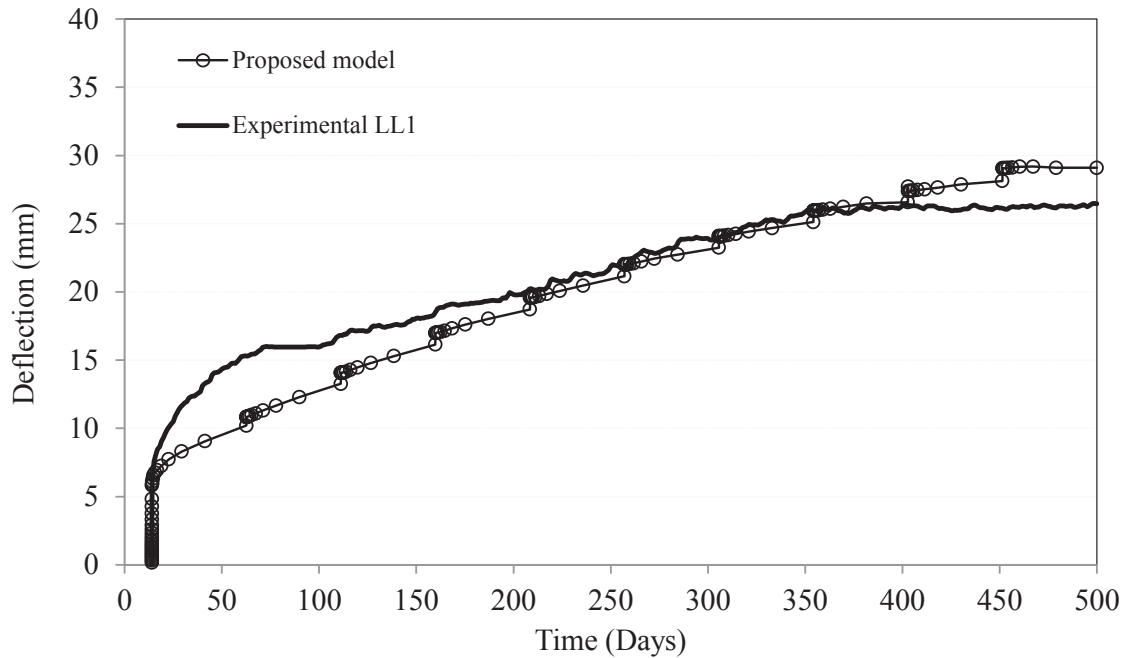


Figure 10.36: comparison of the proposed model with the experimental data in loading level 1

Figure (10.37) compares the experimental deflection-time diagram in loading level 2 (LL_2) with predictions of the model, determined by utilizing the proposed creep and shrinkage models.

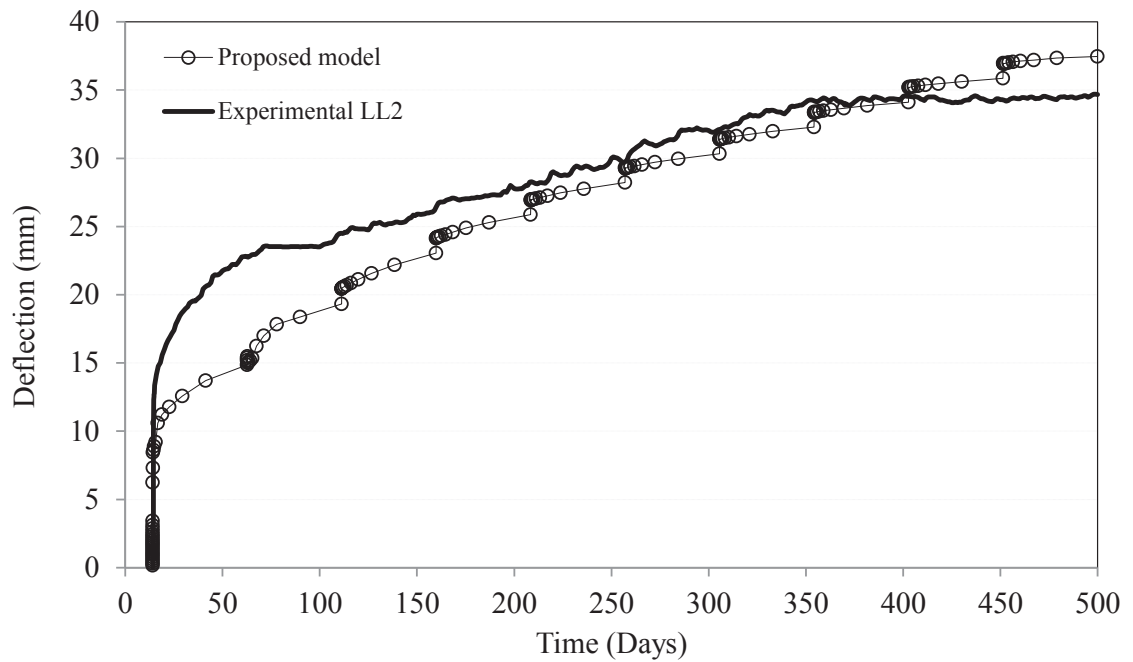


Figure 10.37: Comparison of the proposed model with experimental data in loading level 2

The diagrams in Figures (10.36) and (10.37) confirm that the proposed creep and shrinkage models are in a good agreement with the measured experimental data in two loading levels. The predicted deflections at early ages in both figures are less than the recorded experimental values. This trend of lower predicted values continues up to the age of 350 days. After 350 days, the experimental deflection goes down to the predicted values. However, the long-term deflections after 500 days in both loading levels are very close to the experimental values.

10.21.7. Plane stress and plane strain analysis

Some problems in elasticity may be treated reasonably by a two-dimensional or plane theory of elasticity. Plane stress and plane strain are two general types of problems involved in the plane analysis and are defined by setting down specific restrictions and assumptions on the stress and displacement fields and their physical prototypes.

As shown in Figure (10.38a), the plane stress is defined to be a state of stress in which the normal stress, σ_z , and the shear stresses, τ_{xz} and τ_{yz} , directed perpendicularly to the x-y plane are assumed to be zero. The geometry of the body is essentially that of a plate with one dimension much smaller than the others. The loads are applied uniformly over the thickness of the plate and act in the plane of the plate. The plane stress condition is the simplest form of behaviour of continuum structures and represents situations frequently encountered in practice.

As shown in Figure (10.38b), the plane strain is defined to be a state of strain in which the strain normal to the x-y plane, ϵ_z , and the shear strain γ_{xz} and γ_{yz} are assumed to be zero. In the plane strain, one deals with a situation in which the dimension of the structure in one direction is enormous in comparison to the dimensions of the structure in the other two directions. The geometry of the body is fundamentally that of a prismatic cylinder with one dimension much larger than the others. The applied forces act in the x-y plane and do not vary in the z direction, i.e. the loads are uniformly distributed with respect to the large dimension and act perpendicularly to it. Some important practical applications of plane strain condition occur in the analysis of dams, tunnels, and other geometrical works. Also, many small-scale problems such as bars and rollers compressed by forces normal to their cross section are amenable to analysis under plane strain condition.

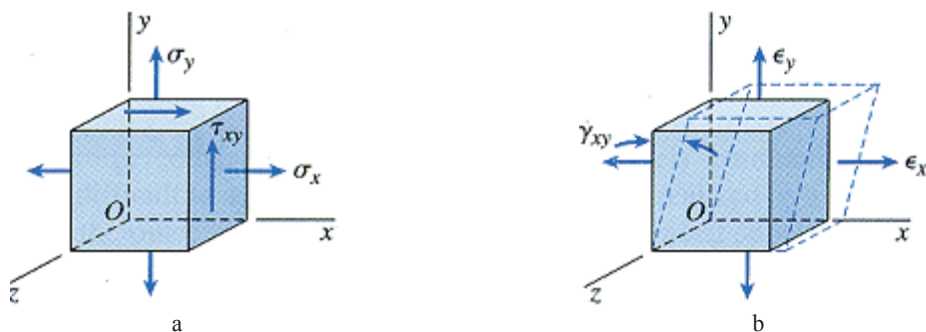


Figure 10.38: Plane stress (a) and plane strain (b) presentation of a general element in the structure

Table (10.15) illustrates the general assumptions in plane stress and plane strain conditions of the structure.

Table 10.15: Stress and strain assumptions in plane stress and plane strain conditions

	Plane stress	Plane strain
Stress “ σ ”	$\sigma_z = 0$ $\tau_{xz} = 0$ $\tau_{yz} = 0$ σ_x, σ_y and τ_{xy} may have nonzero values	$\tau_{xz} = 0$ $\tau_{yz} = 0$ $\sigma_x, \sigma_y, \sigma_z$ and τ_{xy} may have nonzero values
Strain “ ε ”	$\gamma_{xz} = 0$ $\gamma_{yz} = 0$ $\varepsilon_x, \varepsilon_y, \varepsilon_z, \gamma_{xy}$ may have nonzero values	$\varepsilon_z = 0$ $\gamma_{xz} = 0$ $\gamma_{yz} = 0$ $\varepsilon_x, \varepsilon_y, \gamma_{xy}$ may have nonzero values

This study investigates the long-term deflection analysis of LWC slabs in 2D and 3D environments in the ATENA program. Considering the symmetry of the slab geometry, support conditions, reinforcement arrangement and loading, it is reasonable to analyse the slab in the 2D environment to get acceptable results. However, the proper selection of plane stress or plane strain elements may affect the accuracy of the analysis.

Generally, 2D analysis takes one meter thickness of the structure in third dimension by default, while the width of the investigated slabs is 400 mm in the third dimension. This study investigates the effect of plane stress or plane strain elements in the third dimension to find out the most effective depth of structure in 2D creep analysis of the structure.

Comparing the plane strain and plane stress elements in analysing the slab specimens, and considering the geometry of the slabs and direction of the applied load on top surface of the slabs, plane stress elements give a better simulation of the flexural deflection of slabs under two different loading levels.

10.21.7.1. Effect of depth of structure in 2D analysis

To study the sensitivity of the time-dependent deflection to the thickness of plane stress element, different values of thickness have been implemented in the input data, and the model has been analysed.

The creep and shrinkage values were maintained unchanged during the analyses. Also the compliance function of the LWC mixture in consideration of the relaxation of concrete is included in the models.

Figure (10.39) shows a typical input data table in the GID-ATENA program for creep analysis. As mentioned earlier, the default value for element thickness in the third dimension (z direction) is 1.0 metre. The thickness of elements in x and y directions are defined by mesh size in 2-D analysis.

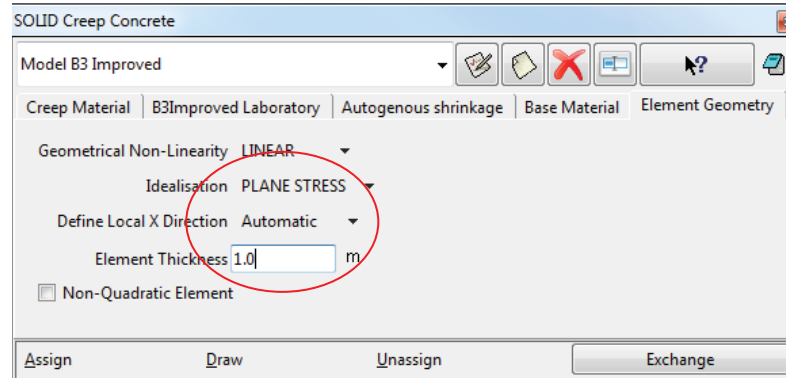


Figure 10.39: Typical input data table for creep analysis in GID-ATENA

- Loading level 1

This section contains the results of analysis of the models by different thickness values of plane stress or plane strain. The models are subjected to the first level of the uniformly distributed sustained loading equal to 30% of the ultimate moment capacity.

Although the plane strain element is not compatible with the slabs behaviour under the flexural loading in the 2-D environment, to have the comparison base with the model with plane stress element, some analyses have been accomplished by plane strain element.

Figure (10.40) compares the time-dependent deflection versus time in the model with plane strain elements of various thickness values. The experimental data is well-predicted by the models with 0.6 to 0.8 mm thickness.

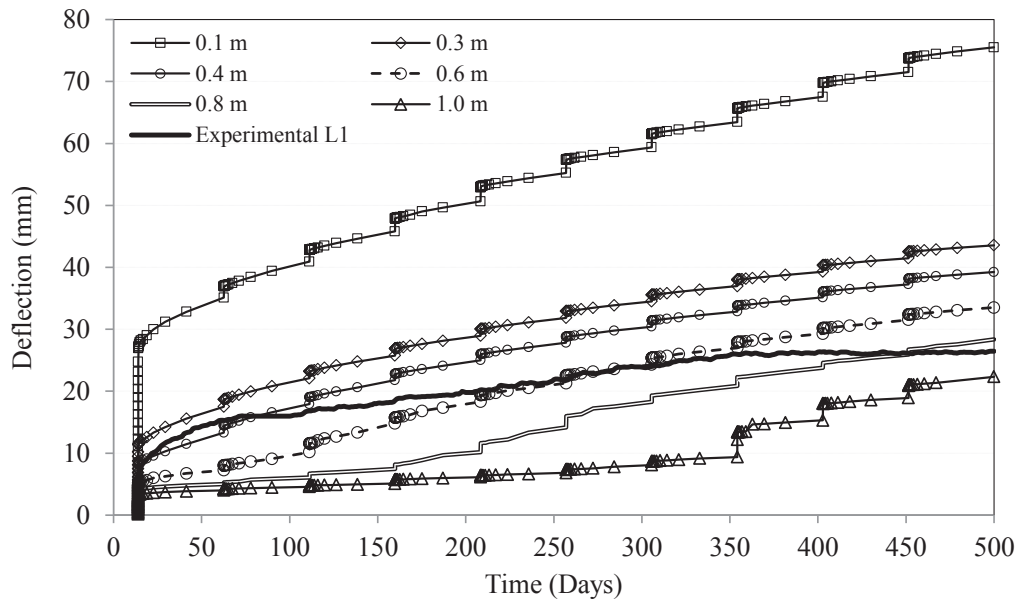


Figure 10.40: Deflection versus time in the model with different thickness values of plane strain element

Figure (10.41) shows time-dependent deflection of the slabs versus time in 2D analysis by applying the plane stress elements with different thickness values. The applied load level is equivalent to 30% of the ultimate bending capacity of the slabs.

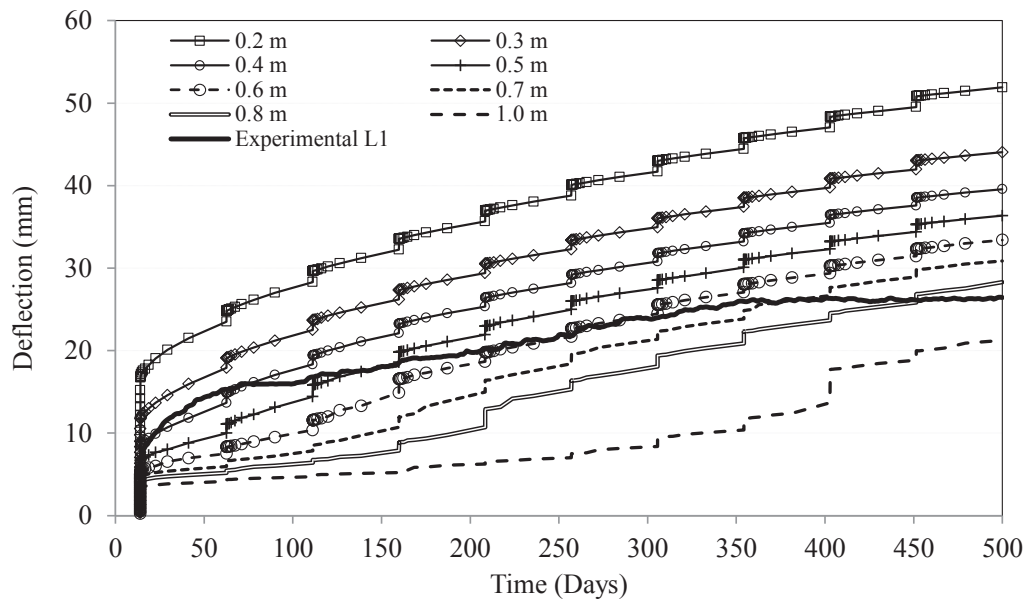


Figure 10.41: Deflection versus time in the model with different thickness values of plane stress element

The experimental result of long-term deflection of slab subjected to first loading level lies between the predicted curves by thicknesses of 0.4 to 0.8m, while the exact depth of structure in the z direction is 0.4 m. In other words, the thickness of 2-D models strongly affects the

predicted trend of deflection and the maximum value of the time-dependent deflection after 500 days. Also, the 2-D model gives higher values of the time-dependent deflection for exact thickness (0.4 m) of the structure in the z direction.

Figure (10.42) compares the ratio of the maximum deflection after 500 days in the plane stress models to the experimental values. The higher thickness of the plane stress element gives fewer values of the deflection after 500 days in the model.

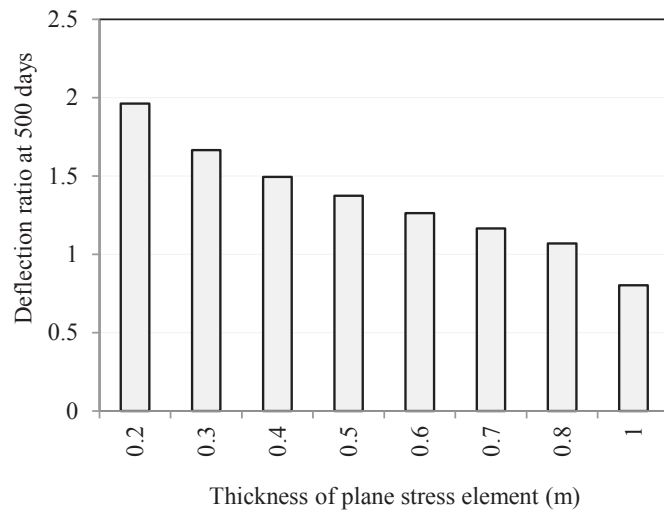


Figure 10.42: Ratio of deflection of plane stress element models to the experimental deflection at 500 days

- Loading level 2

This section contains the results of analysis of the models by different thickness values of plane stress or plane strain subjected to the second level of the uniformly distributed sustained loading equal to 40% of the ultimate moment capacity of slab section.

Figure (10.43) shows the time-dependent deflection of the slabs in 2-D analysis by applying the plane stress element with different thickness values.

The experimental result of long-term deflection of the slab subjected to second loading level lies between the predicted curves with thickness values of 0.3 to 0.75 m. In other words, the thickness of 2D models strongly affects the predicted trend and maximum value of the time-dependent deflection after 500 days. Also, the 2-D model gives higher values of the time-dependent deflection for exact thickness (0.4m) of the structure in z direction.

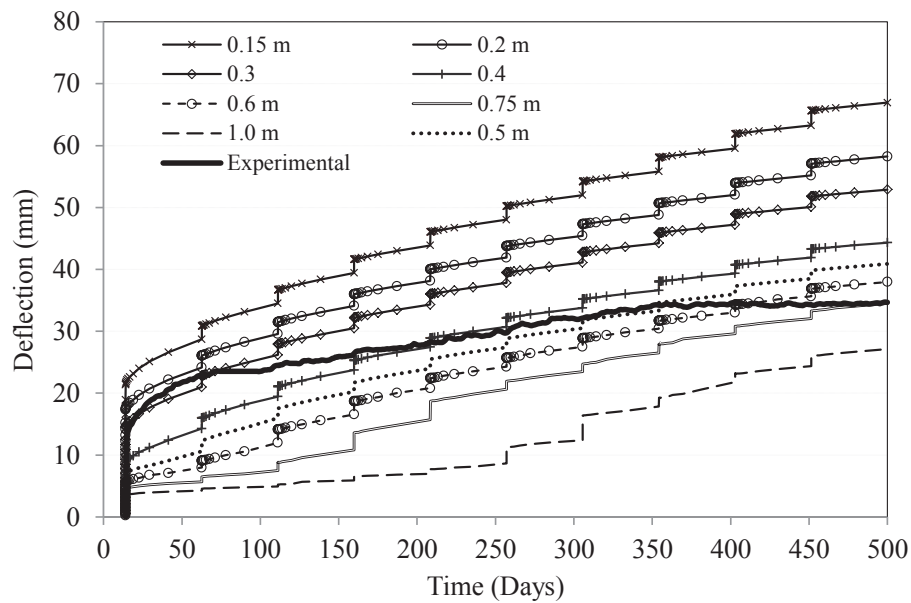


Figure 10.43: Deflection versus time in the model with different thickness values of plane stress element

Figure (10.44) compares the ratio of the maximum deflection after 500 days in the plane stress models to the experimental values. The higher thickness of the plane stress element gives fewer values of the deflection after 500 days in the model.

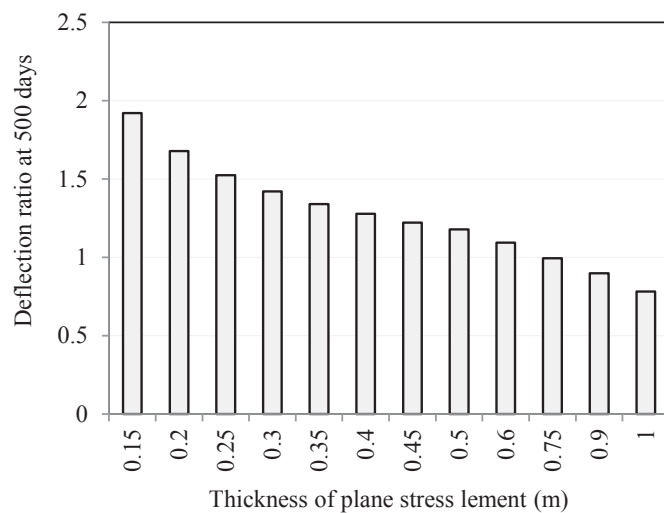


Figure 10.44: Ratio of deflection of plane stress element models to the experimental deflection at 500 days

According to Figures (10.43) and (10.44), increasing the thickness of the plane stress element in 2D analysis, gives lower values of the maximum deflection of LWC slab at mid-span. In other words, the lesser depth of structure in the third dimension in 2D analysis enlarges the resultant time-dependent deflection of slabs for 500 days.

Figure (10.45) compares the ratio of the maximum deflection at 500 days for different thickness values of the plane stress elements under two loading levels. The figure shows the importance of the loading level in creep analysis of the plane stress elements.

The ratio of the corresponding deflection in each thickness varies in the range of 1.12 to 1.27. On average, the deflection at loading level 2 is about 16% higher than the corresponding value in loading level 1.

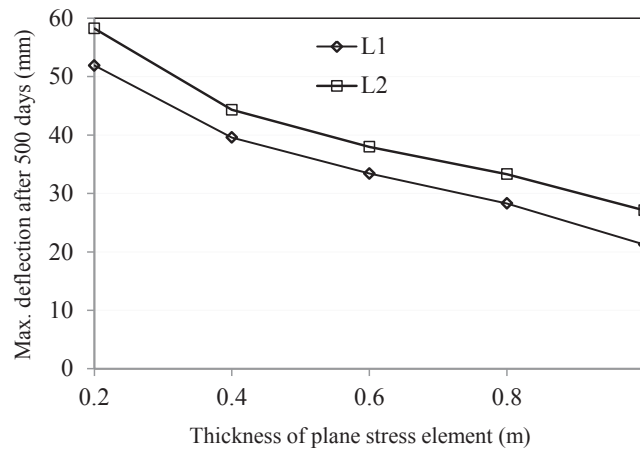


Figure 10.45: Effect of loading level and thickness of plane stress element on maximum deflection

10.21.8. Comparing plane stress, plane strain and 3D elements

There are different types of elements used to model LWC slabs in ATENA. Figure (10.46) compares the time-dependent trend of deflection over 500 days by implementing plane stress, plane strain and 3D elements. The thickness of elements in plane stress and plane strain elements in 2D analysis is taken as 0.4 m. Initial values of the deflection of the model with plane stress and plane strain elements are very close to the recorded experimental values, however, the trend and the maximum deflection after 500 days are higher. On the other hand, despite lower initial values of deflection by implementing 3D elements, the time-dependent deflection after 500 days is very close to the experimental value. However, the increasing rate of deflection under combined effects of creep and shrinkage is entirely different from the experimental results.

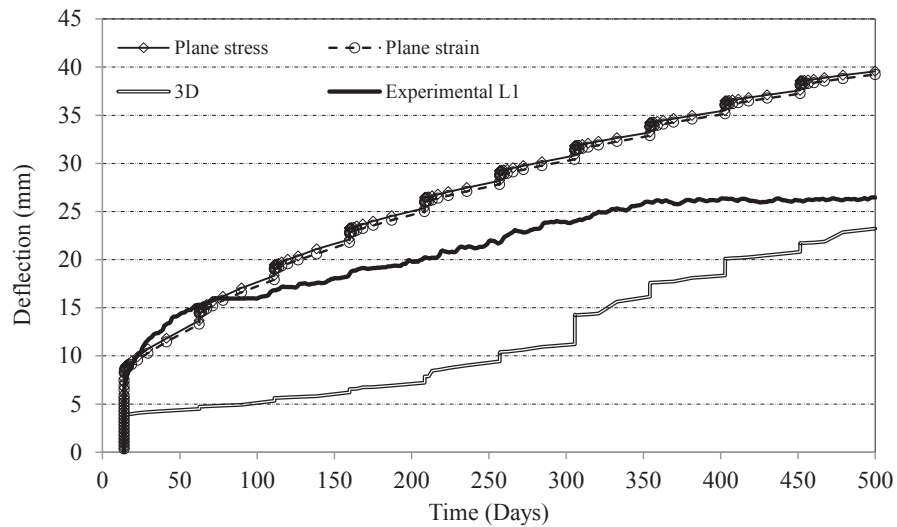


Figure 10.46: Comparing the plane stress, plane strain and 3D elements

10.22. EFFECT OF THE IMPLEMENTED CREEP MODELS ON THE LONG-TERM DEFLECTION OF SLABS

This section compares the implemented creep models in the ATENA program to predict the time-dependent deflection of a LWC slab under the creep and shrinkage effects. The existing models in the ATENA program are the commonly used models in the engineering applications of concrete and reinforced concrete structures. However, they have been developed originally for the conventional concrete other than LWC containing expanded polystyrene beads. The environmental conditions, physical and mechanical properties of the concrete and the mixture proportioning are other crucial parameters in developing those creep models.

This study aims to discover the most compatible creep model with the experimental results of LWC slabs containing expanded polystyrene beads. The selected model(s) can be modified for different conditions, properties and aggregate types to develop a more specific model for LWC. Also, the compatible models can be applied to verify the proposed creep and shrinkage models and the models for mechanical properties of LWC, presented in Chapters 5 and 6. Utilized models in the ATENA-creep problem type are presented in Table (10.3)

The ATENA-Creep problem type enables the user to include or exclude the effects of the compliance (see section 4.2 in Chapter 4) and shrinkage of the concrete in or from the creep analysis. It makes feasible studying of the individual and combined effects of the shrinkage and compliance on the time-dependent behaviour of the concrete structure feasible.

Figures (10.47 a, b) show the options of activating the compliance and shrinkage effects in the implemented creep models in the ATENA programs. Both the compliance and shrinkage

activation option (Figure 10.47 a) is included in the FIBMC2010, B3, B3 improved and the BP-KX creep models of ATENA. However, in CEB-FIP78, CSN-731201, BP1 and BP2 creep models, only the shrinkage effect activation is included (Figure 10.47 b), and the compliance effect is included in the creep data. A comprehensive list of the implemented creep models in ATENA and the effective parameters in each model are previously presented in in section (10.21) and Table (10.14) of this Chapter.

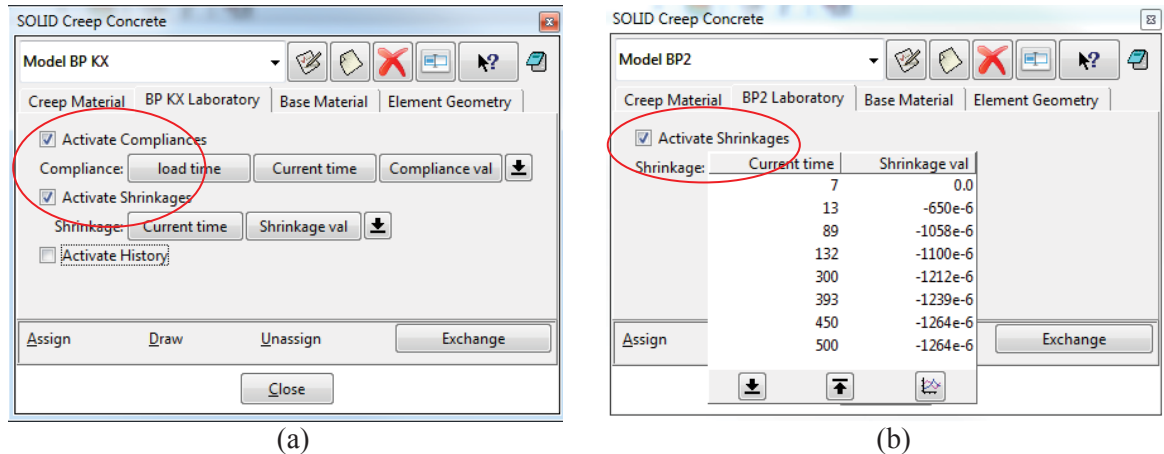


Figure 10.47: Options to activate the shrinkage and compliance effects in creep analysis in ATENA

10.22.1.B3 creep model of ATENA

Figures (10.48) and (10.49) compare the deflection-time diagrams by implementing the B3 creep model of ATENA and the experimental data in the slabs subjected to the loading levels 1 (LL_1) and loading level 2 (LL_2) respectively. Furthermore, the individual effects of the shrinkage and compliance of concrete are analysed in the models.

By comparing the diagrams in Figure (10.48), the shrinkage has a stronger effect than the compliance on the time-dependent deflection of LWC slab. Also, the simultaneous effect of the creep and shrinkage on B3 creep model gives a lower prediction of the time-dependent deflection in the slab subjected to LL_1 . The porous structure of the lightweight concrete gives higher values of the shrinkage strain, while the B3 creep model in Table (10.14) is not completely compatible with the lightweight concrete in terms of including the air content, slump and aggregate ratio. In other words, the shrinkage data are the actual experienced data, while the implemented creep data in the model is not compatible with the utilized.

Regarding the shrinkage and compliance effects, diagrams in Figure (10.49) for LL_2 are in good agreement with the diagrams of LL_1 shown in Figure (10.48); i.e. the effect of shrinkage in

the time-dependent deflection of slab is higher than the effect of compliance of the concrete in both LL_1 , LL_2 .

In B3 creep model without shrinkage and compliance effects in Figure (10.49), the ultimate deflection at 500 days is approximately equal to the instantaneous deflection under the effect of the applied loading on the slab.

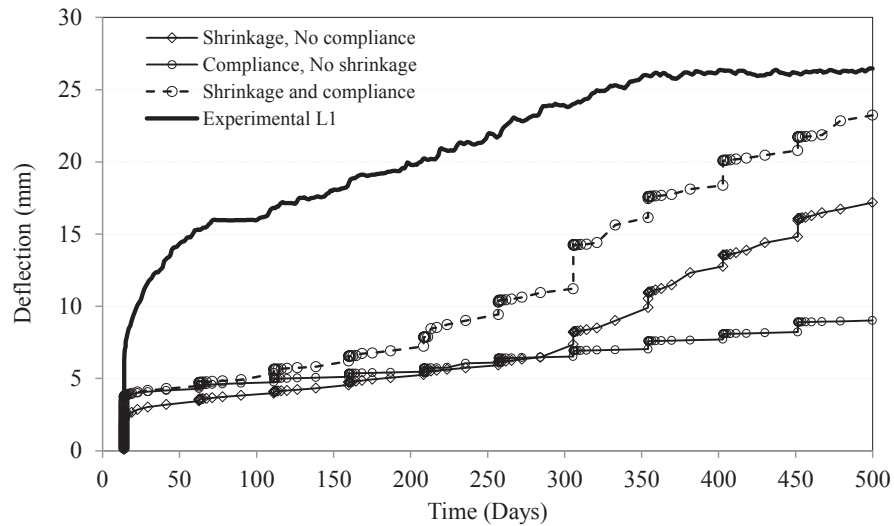


Figure 10.48: B3 creep model with and without shrinkage and compliance effects in loading level 1

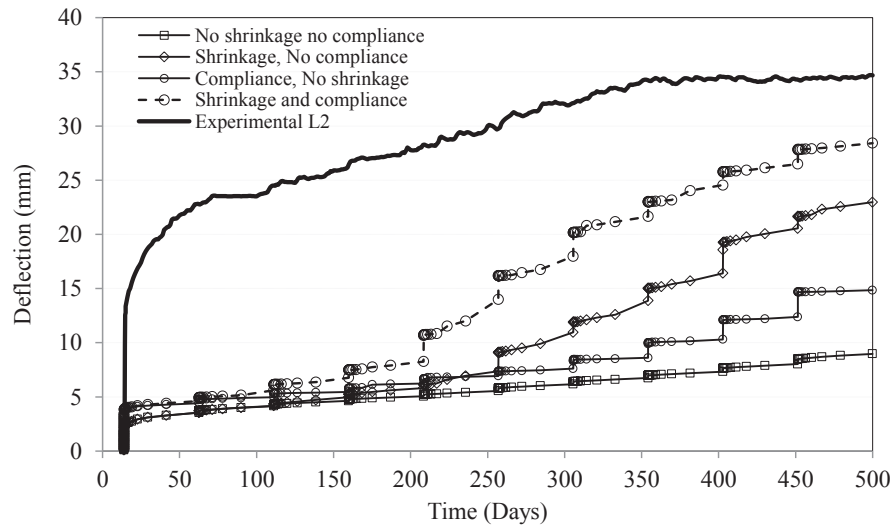


Figure 10.49: B3 creep model with and without shrinkage and compliance effects in loading level 2

10.22.2.FIBMC-2010 creep model of ATENA

Figures (10.50) and (10.51) compare the prediction of deflection-time diagrams by the FIBMC-2010 creep model of ATENA and the experimental data of the slabs subjected to LL_1 and LL_2 respectively. Moreover, the individual effects of the shrinkage and compliance of concrete are compared in the models. The less compatibility of the creep model with the utilized concrete type, and the application of real shrinkage data is the main difference in the resulted diagram.

From the comparison of the diagrams shown in Figure (10.50), the shrinkage poses a greater effect than the compliance on the time-dependent deflection of LWC slab. Also, the simultaneous effect of the creep and shrinkage in the FIBMC-2010 model gives greater prediction of the time-dependent deflection in the slab under LL_1 .

Concerning the shrinkage and compliance effects, diagrams in Figure (10.51) for LL_2 are in reasonable agreement with the diagrams in Figure (10.50) for LL_1 ; i.e. the effect of shrinkage on the time-dependent deflection of the slab is higher than the effect of compliance of the concrete in both LL_1 and LL_2 .

Also, the simultaneous effect of creep and shrinkage in the FIBMC-2010 model gives higher prediction of the time-dependent deflection for the slabs under LL_1 and LL_2 .

The FIBMC-2010 creep model without shrinkage and compliance effects in Figures (10.50) and (10.51) gives greater deflection than the instantaneous deflection of slabs under both LL_1 and LL_2 .

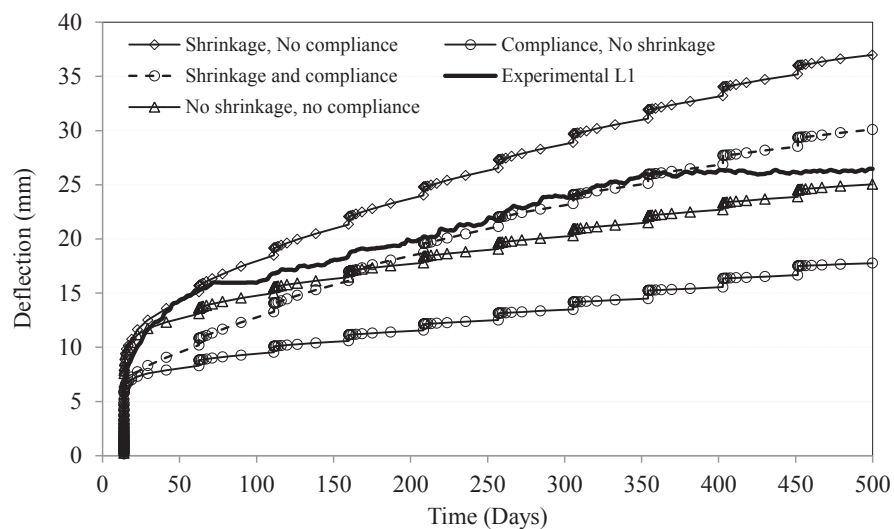


Figure 10.50: FIBMC-2010 creep model with and without shrinkage and compliance effects in loading level 1

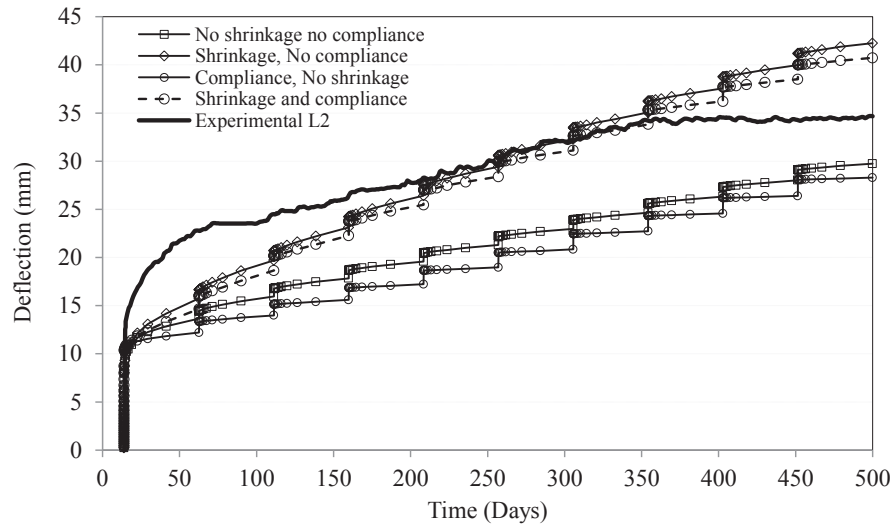


Figure 10.51: FIBMC-2010 creep model with and without shrinkage and compliance effects in loading level 2

10.22.3.BP-KX creep model of ATENA

Figures (10.52) and (10.53) compare the prediction of deflection-time diagrams by the BP-KX creep model of ATENA and the experimental data of slabs subjected to LL_1 and LL_2 respectively. In addition, the individual effects of the shrinkage and compliance are compared in the model.

Regarding the shrinkage and compliance effects, the diagrams shown in Figure (10.53) for LL_2 are in good agreement with the diagrams shown in Figure (10.52) for LL_1 ; i.e. the effect of shrinkage on the time-dependent deflection of the slab is higher than the effect of compliance of the concrete in both LL_1 and LL_2 . Also, the simultaneous effect of the creep and shrinkage in the this model gives a higher prediction of the time-dependent deflection for the slabs under LL_1 and LL_2 .

The BP-KX creep model of ATENA without shrinkage and compliance effects in Figures (10.52) and (10.53) gives higher deflection compared to the instantaneous deflection of slabs under both LL_1 and LL_2 . Furthermore, effect of the compliance of concrete on the deflection-time diagram is not constant with time.

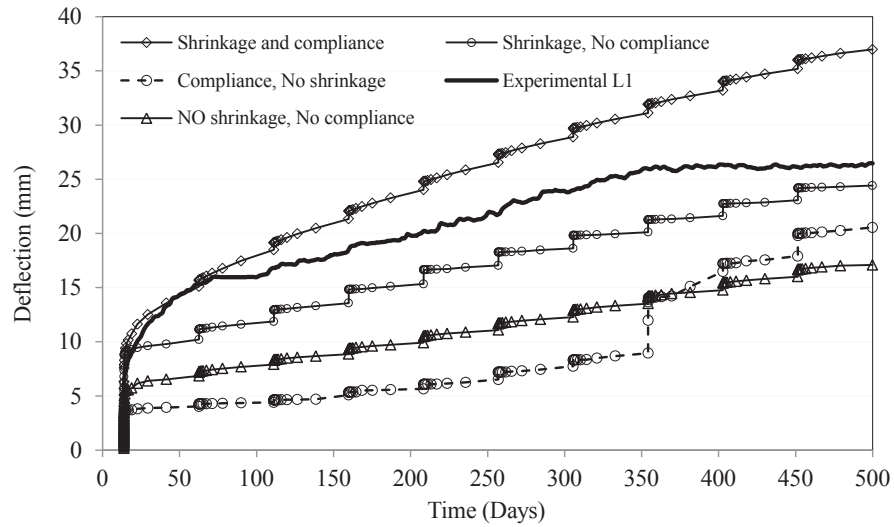


Figure 10.52: BP-KX creep model with and without shrinkage and compliance effects in loading level 1

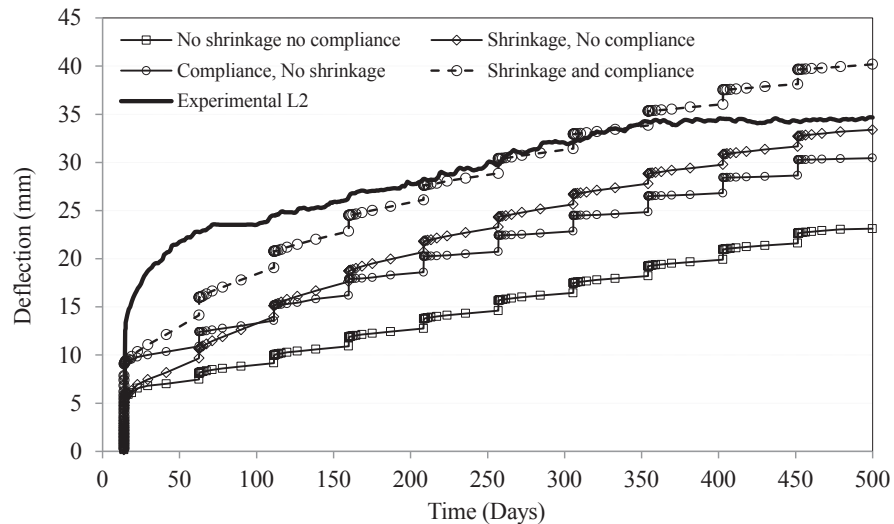


Figure 10.53: BP-KX creep model with and without shrinkage and compliance effects in loading level 2

10.22.4.CEB-FIP78 creep model of ATENA

Figures (10.54) and (10.55) compare the predicted deflection-time diagrams by the CEB-FIP78 creep model with the experimental data of the slabs subjected to LL_1 and LL_2 respectively. In addition, the individual effect of the shrinkage is evaluated in the model.

From the comparison of the diagrams shown in Figure (10.54), inclusion of the shrinkage effect in the analyses changes the deflection-time diagrams for LL_1 and LL_2 considerably. However, the effect of shrinkage in LL_1 and LL_2 is different.

The deflection-time diagram of the slab by the CEB-FIP78 creep model, including the shrinkage effect, is in moderate agreement with the experimental results, especially for LL_1 .

Despite a similar trend of time-deflection, the increasing rate of deflection after 250 days is considerably higher than the experimental data for LL₂.

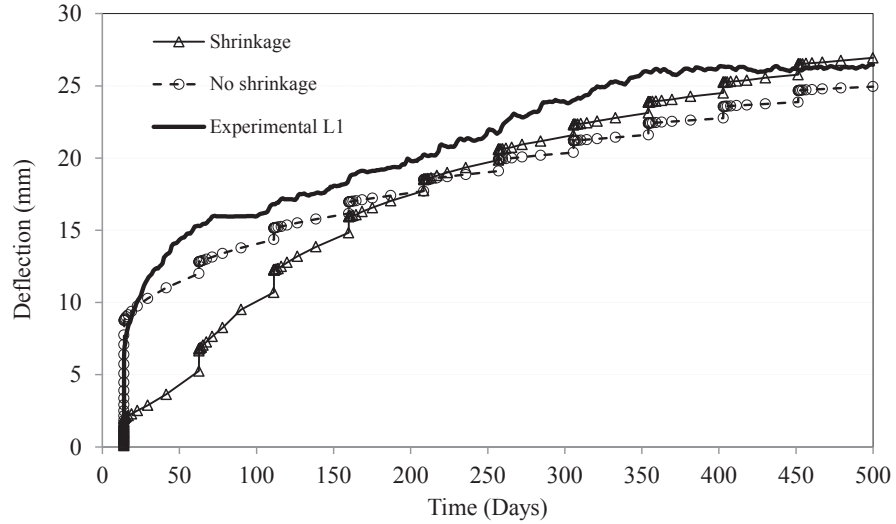


Figure 10.54: CEB-FIP78 creep model with and without shrinkage effect in loading level 1

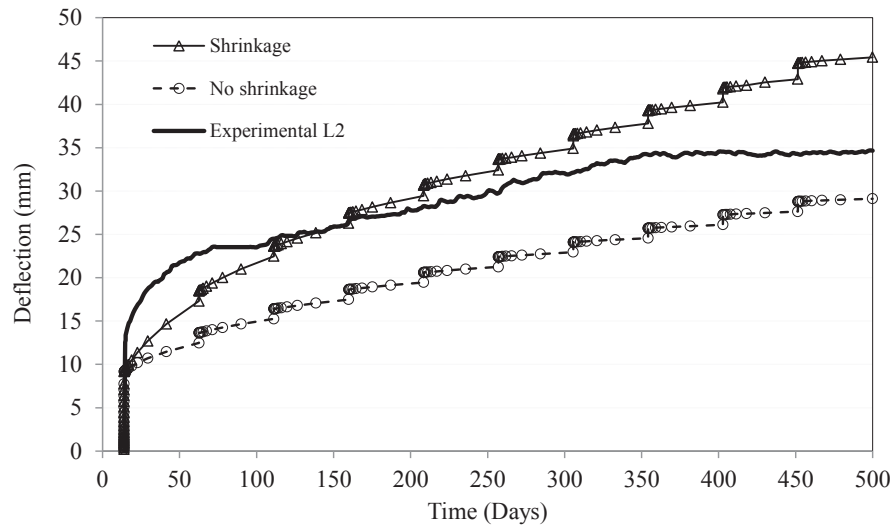


Figure 10.55: CEB-FIP78 creep model with and without shrinkage effect in loading level 2

10.22.5.CSN-731201 creep model of ATENA

Figures (10.56) and (10.57) compare the predicted deflection-time diagrams by the CSN-731201 creep model of ATENA with the experimental data of the slabs subjected to LL₁, LL₂ respectively. Also, the individual effect of the shrinkage is studied in the model.

The comparison of the diagrams shown in Figure (10.56) and (10.57), including the shrinkage effect in the FEA, significantly changes the deflection-time diagrams in LL_1 and LL_2 respectively. However, the effect of shrinkage in LL_1 and LL_2 is different.

The deflection-time diagram shown in the slab by CSN-731201 creep model inclusion of the shrinkage effect is approximately in agreement with the experimental results in LL_2 . However, the predicted deflection in LL_1 is noticeably below the experimental data.

In the first loading level shown in Figure (10.56), despite a similar trend of time-deflection diagrams up to 350 days, the model with the shrinkage effect changes the deflection behaviour. However, the ultimate deflation is substantially below the exact experimental value.

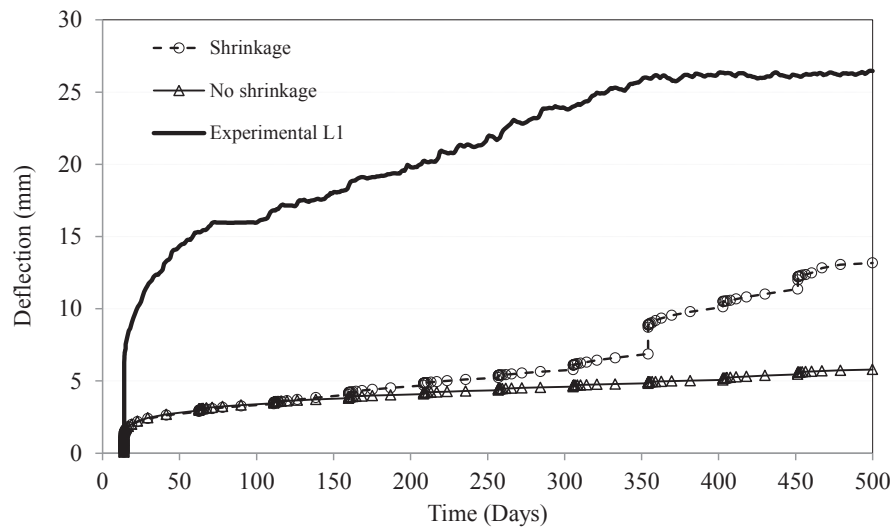


Figure 10.56: CSN-731201 creep model with and without shrinkage effect in loading level 1

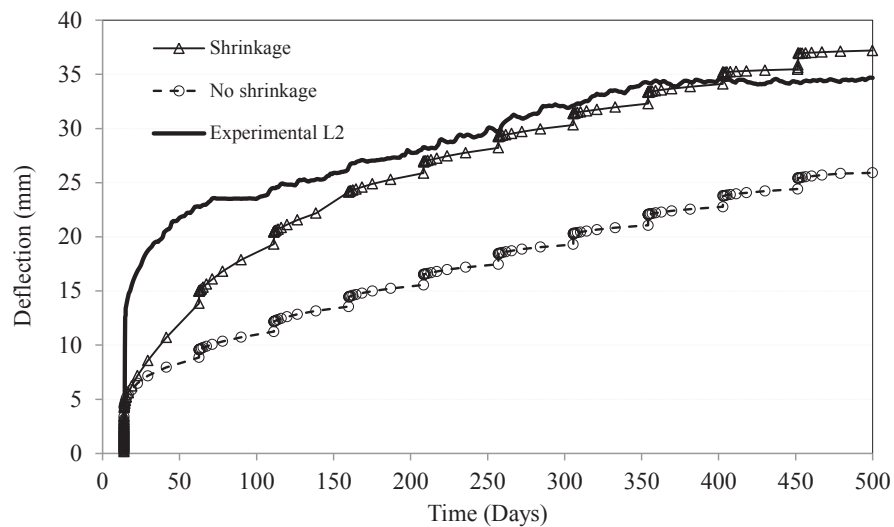


Figure 10.57: CSN-731201 creep model with and without shrinkage effect in loading level 2

10.22.6.BP1 creep model of ATENA

Figures (10.58) and (10.59) compare the predicted deflection-time diagrams by the BP1 creep model of ATENA with the experimental data of slabs subjected to LL_1 , LL_2 respectively. In addition, the individual effect of the shrinkage is considered in the model.

From the comparison of the diagrams shown in Figure (10.58) and (10.59), inclusion of the shrinkage effect in the analysis changes the deflection-age diagrams, especially in LL_2 significantly. However, the effect of shrinkage in LL_1 and LL_2 is totally different.

The deflection-time diagram of the slab by the BP1 creep model, including the shrinkage effect, is in high agreement with the experimental data in LL_1 . This creep model gives reasonable predictions in LL_2 .

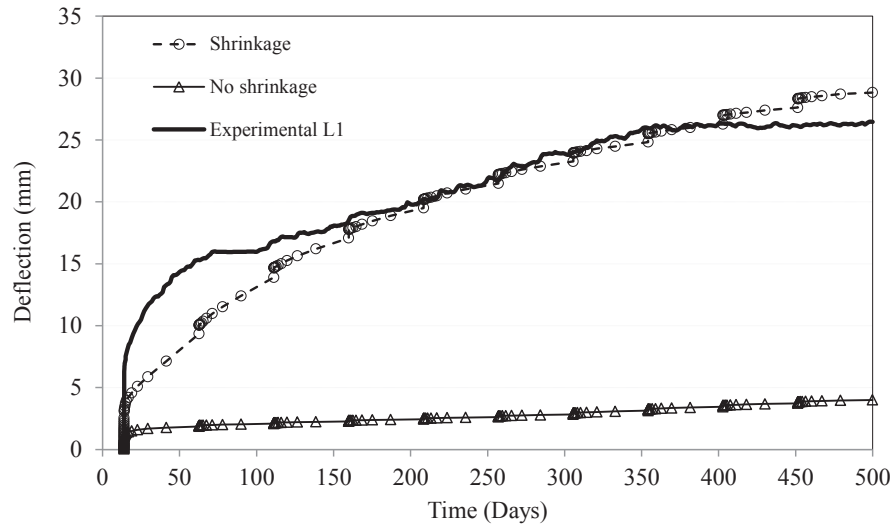


Figure 10.58: BP1 creep model with and without shrinkage effect in loading level 1

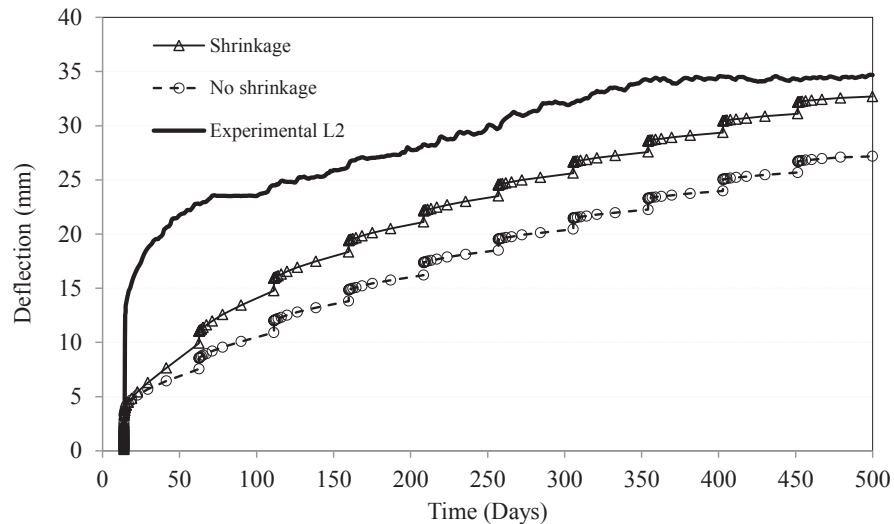


Figure 10.59: BP1 creep model with and without shrinkage effect in loading level 2

10.22.7.BP2 creep model of ATENA

Figures (10.60) and (10.61) compare the predicted deflection-time diagrams of the BP2 creep model of ATENA with the experimental data of slabs subjected to LL_1 and LL_2 respectively. Additionally, the individual effect of the shrinkage is analysed in the model.

From the comparison of the diagrams shown in Figures (10.60) and (10.61), inclusion of the shrinkage effect in the FEA, strong changes in the deflection-time diagrams in both LL_1 and LL_2 are clear. However, the effect of the shrinkage in LL_1 and LL_2 is different.

The deflection-time diagram in the slab of the BP2 creep model including the shrinkage effect is approximately in agreement with the experimental results in LL_2 . However, the predicted deflection in LL_1 is considerably below the experimental data.

In the first loading level shown in Figure (10.60), despite a similar trend of the time-deflection diagrams up to 420 days, the model with shrinkage effect changes the deflection behaviour. However, the ultimate deflections are significantly below the exact experimental values.

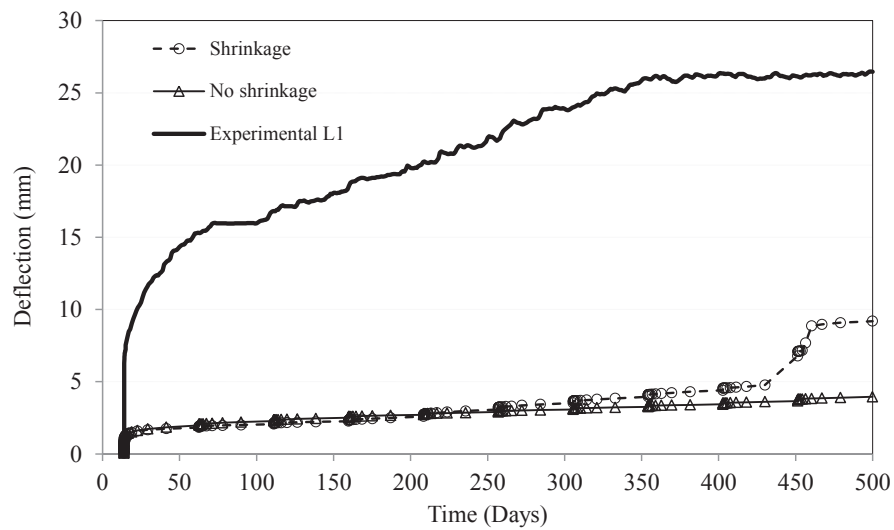


Figure 10.60: BP2 creep model with and without shrinkage effect in loading level 1

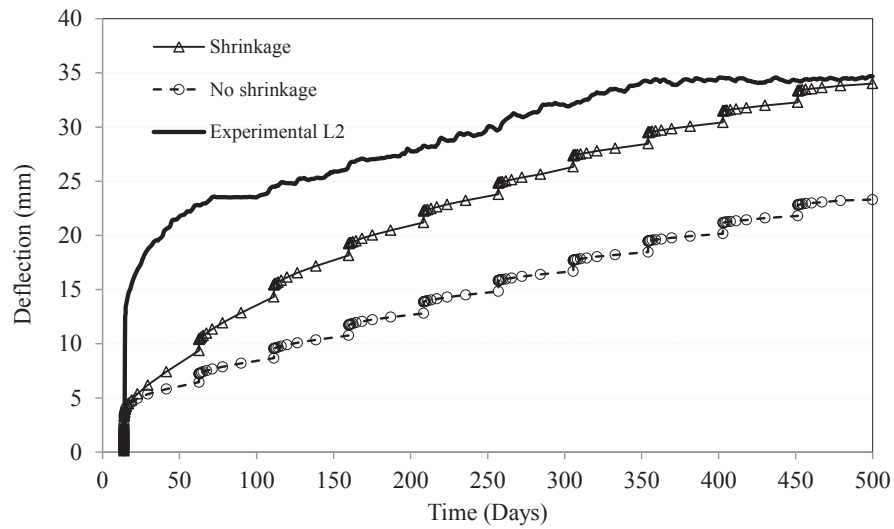


Figure 10.61: BP2 creep model with and without shrinkage effect in loading level 2

10.22.8. Comparing the creep models of ATENA

Figures (10.62) and (10.63) compare different creep models used to predict the deflection-time diagrams under the two loading levels of LL1 and LL2 respectively. Unpredictably, the loading level influences the predictions of these creep models. In other words, the models are highly sensitive to the applied sustained loading. However, in the ordinary engineering projects, to determine the creep coefficient, only specified stress level is applied on the test specimens.

As shown in Figure (10.62), the FE models of the slabs on the basis of the FIBMC-2010, CEB-FIP78 and BP1 creep models, give the best estimation of time-dependent deflection of the slabs subjected to LL₁. However, the predicted values and diagrams by the BP2 and CSN-731201 creep model significantly underestimate the experimental deflection of the LWC slabs under LL₁.

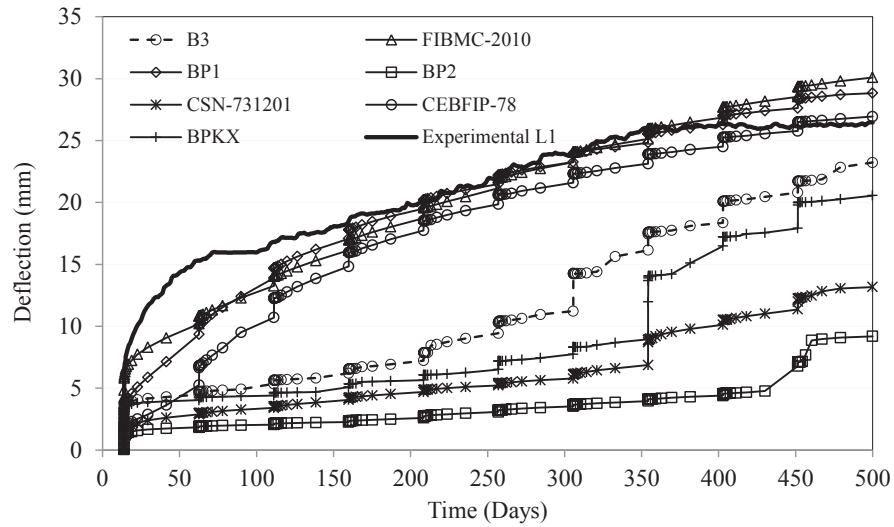


Figure 10.62: Comparing the implemented creep models in loading level 1

The FEA with all the creep models except CEBFIP-78, predict the long-term deflection values close to the experimental values under LL₂ in Figure (10.63). However, the CSN-731201 creep model reveals the best agreement with the experimental data. Furthermore, the BP1 and BP2 models in the simulated slab models are in good agreement with the experimental data.

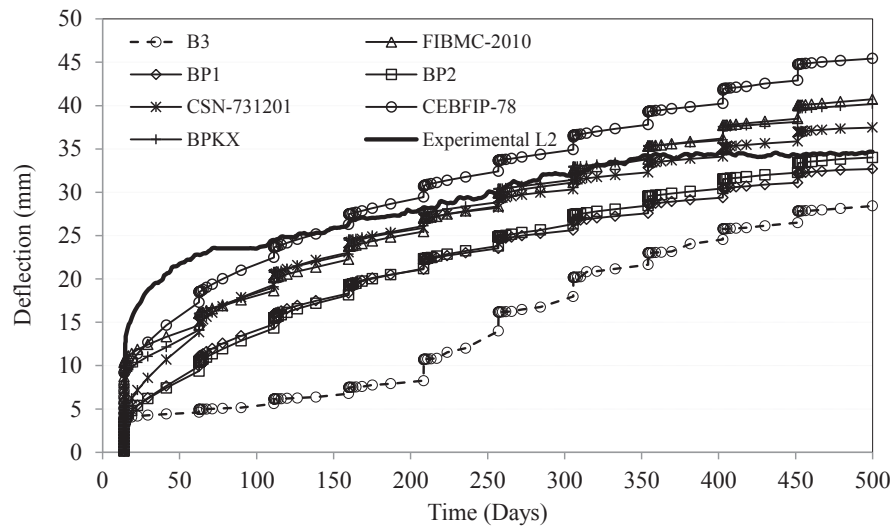


Figure 10.63: Comparing the implemented creep models in loading level 2

Comparing the results in Figures (10.62), (10.63) confirms the sensitivity of the creep models to the sustained loading level in the prediction of the time-dependent deflection of slabs. Among the implemented creep models in ATENA, BP2 is strongly influenced by the load level, while FIBMC-2010 shows the least variation from the experimental results under two loading levels.

Variation of the deflection by other creep models is between BP2 and FIBMC-2010. They are between the extreme points of the compatibility with the experimental data.

As general conclusion, FIBMC-2010 is the most compatible model for the time-dependent behaviour of LWC slab, therefore it is recommended to apply these particular creep and shrinkage effects in study of the structures with this type of concrete.

10.23. SUMMARY

The proposed models for the mechanical and time-dependent properties of lightweight concrete and the deflection-age behaviour of slabs need to be verified by finite element analysis. Among the existing finite element programs, the ATENA software was selected to model and analyses of the slabs. ATENA has the capability to implement the non-linear and time-dependent behaviours that are crucial in this study. A brief description of the existing capabilities of ATENA in relation to the accurate modelling of the reinforced concrete structures, together with the material models for concrete, reinforcement and content elements were given. The element types and compatibility between the material and element types were also given to assign the most appropriate element and meshing to the selected materials.

Two types of problems in ATENA, namely ATENA-Static and AATENA-Creep were utilized to study the instantaneous and long-term deflection of the lightweight concrete slabs respectively. In each problem type, a comprehensive parametric study was performed to calibrate the simulated models to show the most compatible results with the experimental data. The following results can be drawn from the finite element analysis of the calibrated models in ATENA in the static and creep problem types.

- The finite element models verify the proposed models of mechanical properties of lightweight concrete in Chapter Five;
- The finite element models verify the proposed creep and shrinkage models of lightweight concrete in Chapter Six;
- 3D analysis is the most accurate method, however, regarding the symmetric geometry of slabs and loading and supporting conditions, 2D analysis can be used with high accuracy of the results both in static and creep analysis;
- In 2D analysis, the plane stress analysis of the models shows better agreement with the experimental data;
- The smaller mesh size and higher number of the loading steps increase the accuracy of the e results both in static and creep analysis;

- The loading level has a considerable effect on the obtained results, particularly on the creep analysis. In other words, loading level can change the creep effect on the structure;
- The slab model were analysed by applying different creep models and compared with the experimental data. The loading level changed the effect of each creep model;
- Time-dependent analysis of the models shows the higher effect of the shrinkage strain than compliance of the concrete, on the long-term deflection of concrete slabs;
- In the first loading level ($0.3M_u$), FIBMC-2010, CEB-FIP78 and BP1 creep models give the best estimation of the time-dependent deflection of lightweight concrete slabs;
- In the second loading level ($0.4M_u$), CSN-731201 creep model gives the best estimation of the time-dependent deflection of lightweight concrete slabs. Overall, FIBMC-2010 is the most compatible model for the time-dependent behaviour of LWC slab;
- Among the creep models, BP2 is strongly influenced by the load level, while FIBMC-2010 shows the least variation with the experimental results under two loading levels.

Figures (10.64) and (10.65) show the compatibility between the experimental data and predictions of the implement creep models.

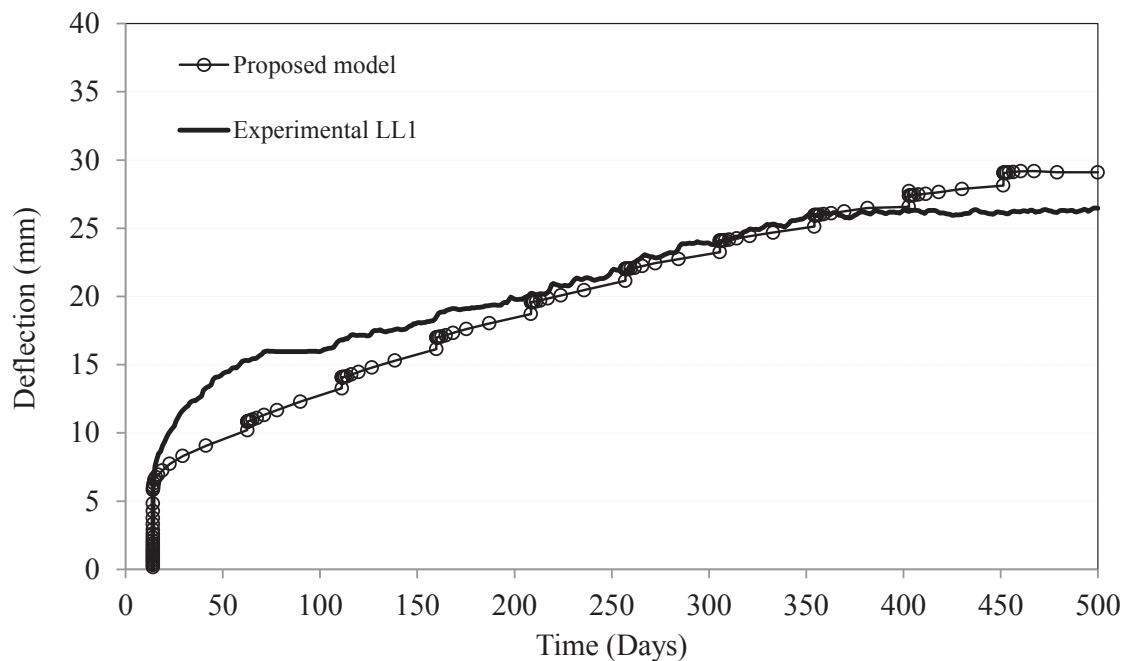


Figure 10.64: comparison of the proposed model with the experimental data in loading level 1

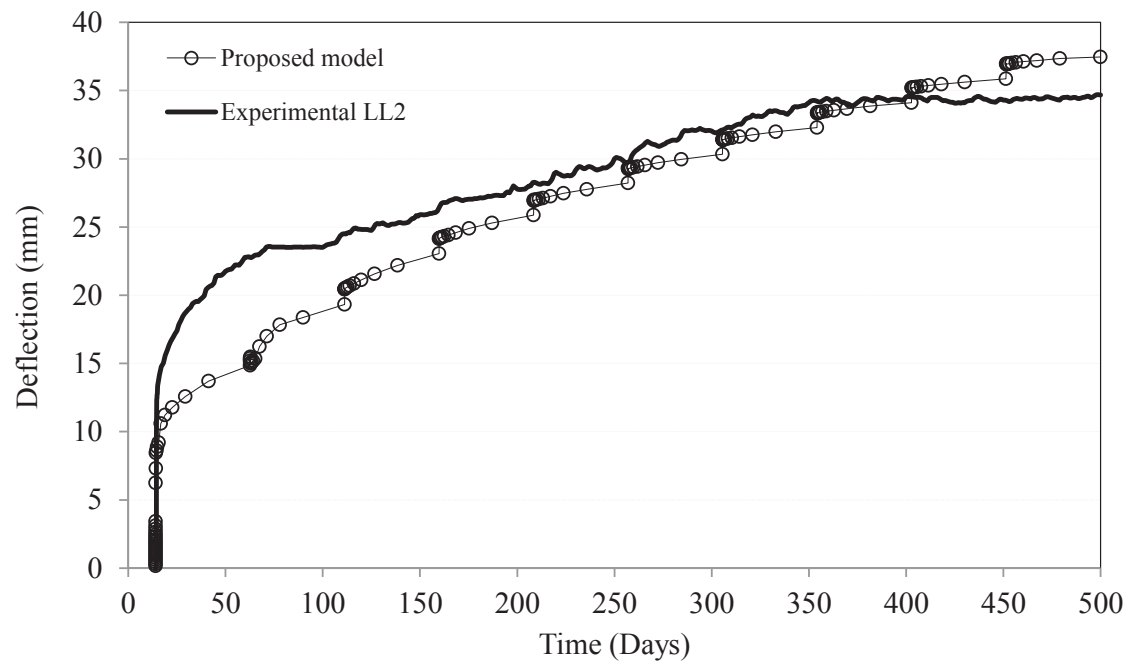


Figure 10.65: Comparison of the proposed model with experimental data in loading level 2

REFERENCES

1. Ngo, D. and A. Scordelis, *Finite element analysis of reinforced concrete beams*. in *ACI Journal Proceedings*. 1967. ACI.
2. Chaudhari, S.V. and M.A. Chakrabarti, *Modeling of concrete for nonlinear analysis Using Finite Element Code ABAQUS*. International Journal of Computer Applications, 2012. **44**(7): p. 14-18.
3. Yuanfeng, L.X.W., *Three-dimensional nonlinear finite element analysis of reinforced concrete structures based on ANSYS program*. 2nd International Conference on Computer Engineering and Technology, 2010. **6**: p. 42-46.
4. Smadi, M. and K. Belakhdar, *Development of finite element code for analysis of reinforced concrete slabs*. Jordan Journal of Civil Engineering, 2007. **1**(2).
5. Červenka, V. and L. Jendele, *ATENA Program Documentation Part 2-2 User's Manual for ATENA 3D*. 2014.
6. ABAQUS, *ABAQUS MANUAL* 2010.
7. Lee, H.H., *Estimation of Uncertainties and Validation of Computational Models for Structural Concrete*. 2009: ProQuest.
8. Hu, H.-T. and W.C. Schnobrich, *Constitutive modeling of concrete by using nonassociated plasticity*. Journal of Materials in Civil Engineering, 1989. **1**(4): p. 199-216.
9. Gilbert, R.I. and Z.I. Sakka, *Strength and ductility of reinforced concrete slabs containing welded wire fabric and subjected to support settlement*. Engineering Structures, 2010. **32**(6): p. 1509-1521.
10. Rousseau, J., P. Marin, and L. Daudeville, *Discrete element modeling of reinforced concrete with a particular steel-concrete interface*. Discrete Element Group for Hazard Mitigation, 2007.
11. Chong, K.T., *numerical modelling of time-dependent cracking and deformation of reinforced concrete structures*. PhD dissertation, University of New South Wales, Sydney, Australia, 2004.
12. Jing, H. and Y.Q. Li. *Nonlinear Finite Element Analysis of Layered Steel Fiber Reinforced Concrete Beam*. in *Applied Mechanics and Materials*. 2012. Trans Tech Publ.
13. ČERVENKA, V., J. ČERVENKA, and R. PUKL, *ATENA – A tool for engineering analysis of fracture in concrete*. Sādhanā, 2002. **27**(4): p. 485-492.
14. Johnson, S., *Comparison of Nonlinear Finite Element Modeling Tools for Structural Concrete*. 2006.
15. Yang, K.-H., H.-S. Chung, and A.F. Ashour, *Influence of shear reinforcement on reinforced concrete continuous deep beams*. ACI structural journal, 2007. **104**(4).
16. Yu, Q., *Size Effect and Design Safety in Concrete Structures under Shear*. PhD dissertation, NORTHWESTERN UNIVERSITY, ILLINOIS, USA, 2007.
17. Malm, R. and J. Holmgren, *Cracking in deep beams owing to shear loading. Part 2: Non-linear analysis*. Magazine of Concrete Research, 2008. **60**(5): p. 381-388.
18. Talley, K.G., *Assessment and strengthening of ASR and DEF affected concrete bridge columns*. 2009.
19. Gribniak, V., et al., *Finite element mesh size effect on deformation predictions of reinforced concrete bridge girder*. The Baltic Journal of Road and Bridge Engineering, 2010. **5**(1): p. 19-27.
20. Dobromil, P., C. Jan, and P. Radomir, *Material model for finite element modelling of fatigue crack growth in concrete*. Procedia engineering, 2010. **2**(1): p. 203-212.
21. Olteanu, I., A. Alistar, and M. Budescu, *Non-linear Analysis of Reinforced Concrete Frames with ATENA 3-D Program*. Buletinul Institutului Politehnic din Iasi. Sectia Constructii, Arhitectura, 2011. **57**(2): p. 93.

22. Lampropoulos, A.P., O.T. Tsioulou, and S.E. Dritsos, *Biaxial stress due to shrinkage in concrete jackets of strengthened columns*. ACI Materials Journal, 2012. **109**(3).
23. Sucharda, O. and J. Brozovsky, *Numerical modelling of reinforced concrete beams with fracture-plastic material*. Frattura ed Integrità Strutturale, 2014. **30**: p. 375-382.
24. Singh, V., et al., *Finite Element Modeling of CFRP Retrofitted RC Beam-Column Joints*. International Journal on Emerging Technologies, 2014. **5**(2): p. 31.
25. Meskenas, A., et al., *Simplified technique for constitutive analysis of SFRC*. Journal of Civil Engineering and Management, 2014. **20**(3): p. 446-453.
26. Namy, M., J.-P. Charron, and B. Massicotte, *Structural behavior of cast-in-place and precast concrete barriers subjected to transverse static loading and anchored to bridge deck overhangs*. Canadian Journal of Civil Engineering, 2015. **42**(2): p. 120-129.
27. Kollegger, J. and G. Mehlhorn, *Experimentelle und Analytische Untersuchungen zur Aufstellung eines Materialmodells fuer Gerissene Stahbetonscheiben*. Nr. 6 Forschungsbericht, Massivbau, Gesamthochschule Kassel, 1988.
28. Dyngeland, T., *Behaviour of Reinforced Concrete Panels*. Doktor Ingenierafhandling (Ph. D. thesis, Institut for Betongkonstruksjoner, Trondheim, 1989.
29. béton, C.e.-i.d., *CEB-FIP model code 1990: design code*. 1993: CEB Bulletin d'Information No.213/214, Comité EuroInternational du Béton, Lausanne, Switzerland, Telford.
30. Bigaj, A.J., *Structural dependence of rotation capacity of plastic hinges in RC beams and slabs*. 1999.
31. Jensen, E. and W. Hansen, *Nonlinear aggregate interlock model for concrete pavements*. International Journal of Pavement Engineering, 2006. **7**(4): p. 261-273.
32. Yang, K.-H. and A.F. Ashour, *Aggregate interlock in lightweight concrete continuous deep beams*. Engineering Structures, 2011. **33**(1): p. 136-145.
33. Choi, C.-K. and H.-G. Kwak, *The effect of finite element mesh size in nonlinear analysis of reinforced concrete structures*. Computers & Structures, 1990. **36**(5): p. 807-815.
34. Zheng, Y., *Finite element analysis for fixture stiffness*. 2005.

CHAPTER 11

SUMMARY AND CONCLUSIONS

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Chapter 11: Summary and conclusion

11.1. SUMMARY

This study investigated the deflection behaviour of lightweight reinforced concrete simply-supported one-way slabs containing treated expanded polystyrene beads. In addition, the deflection behaviour of slabs was monitored in three distinct categories as a) incremental deflection under increasing load until the flexural failure of slab, b) instantaneous deflection at the time of loading and c) time-dependent deflection under sustained loading.

Regarding the time-dependent deflection, a particular emphasis was on the development of the flexural deflection attributed to the combined effects of the constant service loads and the time-dependent properties of concrete such as creep and shrinkage. The investigation included both the experimental and analytical parts to verify the results precisely. For the analytical part, a finite element analysis using the ATENA program and statistical methods were utilized.

The overall objective of this study was to compare the deflection behaviour of the slabs made of different types of concrete. Nejadi (2005) conducted an experimental study on the cracking and deflection behaviour of conventional concrete slabs for 400 days. Aslani (2014) investigated the time-dependent cracking and deflection behaviour of self-compacting concrete slabs for 250 days.

Details of the specimen geometry (length, thickness, and depth), support conditions, and reinforcement details in the experiments are the same for all three series of conventional, self-compacting and lightweight concrete slabs. The applied loading on the lightweight and self-compacting concrete slabs were 30% and 40% of the ultimate bending capacity of the slabs

section. However, for conventional concrete slabs, the uniformly distributed loading was about 20% and 30% of the ultimate bending capacity of the slabs section.

The deflection history and development with age were the main parameters in monitoring the deflection behaviour. The key drivers of the deflection development were also investigated. The fresh properties of concrete, bond characteristics, the time-dependent effects of creep and shrinkage on the lightweight concrete members thereof have been also monitored and studied.

The properties of concrete in the hardened state, including the mechanical properties, shrinkage and creep and the bond-slip between the reinforcing bars and the surrounded concrete were also studied. The hardened properties of lightweight concrete have been compared to the corresponding properties of the conventional concrete and self-compacting concrete. Furthermore, new models have been proposed as a means of predicting the mechanical properties of concrete in comparison with the existing models in the literature.

To study the effect of short-term loading on the deflection behaviour of slabs, a series of tests was performed on the simply-supported one-way lightweight concrete slabs. The slabs were subjected to monotonically increasing loads up to the failure point to construct the load-deflection diagram and determine the ultimate loading capacity. The deflection of each slab at mid-span, crack patterns, steel tensile strains, and concrete surface strains at the level of the reinforcing bars were carefully monitored under the increasing load up to the failure point.

Mechanical properties of the light-weight concrete including the compressive strength, the modulus of rupture, the modulus of elasticity, the bond strength between the concrete and steel bars and the splitting tensile strength of lightweight concrete were measured at different ages. Also, the creep and shrinkage strain of the lightweight concrete were measured by the companion specimens.

Another series of tests were conducted to evaluate the long-term deflection of lightweight concrete slabs. In this regard, four simply supported one-way slabs were subjected to two distinct levels of sustained loads for 525 days. Development of the flexural deflection under the combined effects of service load and creep and shrinkage effects were also studied. Deflection of slabs at mid-span and the point on the border of the high-moment region, crack patterns, steel tensile strains and the concrete surface strains at the level of reinforcing bars were recorded immediately after loading for up to 525 days under two constant values of the sustained load on each pair of slabs. Moreover, the compression strain of concrete at the top surface on mid-span of the slab was monitored under the loadings.

The finite element program was utilized to model the slab structures to simulate the instantaneous and time-dependent flexural cracking and deflection. New analytical models were

proposed to predict the mechanical properties of lightweight concrete. These models were verified by the obtained results from the numerical models of the slabs in the ATENA program.

The results of the experimental tests were compared with predictions of the proposed models and the existing models in the literature. The comparison included both instantaneous and long-term deflections.

A comprehensive parametric study for both types of the experimental study (instantaneous and time-dependent deflection) of the slabs by finite element analysis has been performed. The parametric analysis of the models in finite element analysis included the proposed models for hardened properties, bond characteristics, creep, and shrinkage models.

11.2. CONCLUSIONS

This study is presented in eleven chapters. Chapters One to Five contain the literature review of the used material, structural concepts of the topic and relevant investigations. In Chapters One the research methodology and layout of the thesis are described. Chapter Two includes a brief history of the invention, development and mechanical properties of lightweight concrete. In Chapter Three, experimental results of almost all investigations about the lightweight concrete containing expanded polystyrene beads and its hardened properties are collected and analysed. Chapter Four describes the time-dependent properties of concrete and existing models of creep, shrinkage, and steel-concrete bond characteristic. In chapter Five, mechanism of the flexural deflection and effective parameters and existing models are broadly discussed. In addition, in the literature review, main differences and existing models of the mechanical properties of self-compacting concrete, conventional concrete and lightweight concrete are comprehensively evaluated.

Experimental and numerical phase of the study are explained in Chapters Six to Ten. Chapter Five includes the hardened properties of lightweight concrete. Chapter Seven explain the flexural load-deflection of slabs. In Chapters Eight and Nine, the instantaneous and Long-term flexural deflections of slabs are presented. Chapter Ten explains the verification of experimental investigation by finite element analysis. Summary and conclusions of the experimental and numerical phase of the study are presented in Chapter Eleven in the following main categories:

- Hardened concrete properties;
- Flexural load-deflection of one-way slabs;
- Flexural Instantaneous deflection of one-way slabs;
- Flexural long-term deflection of one-way slabs;
- Numerical analysis of one-way slabs.

11.2.1. Experimental program; Hardened Concrete Properties

11.2.1.1. Material properties

New models are proposed in this study to estimate the mechanical properties of lightweight concrete including the Compressive Strength (CS), Modulus of Elasticity (MoE), Modulus of Rupture (MoR), Splitting Tensile Strength (STS), Compressive Stress-Strain Curve (CSSC) and energy absorption capacity. In addition, CS, MoE, MoR and STS of Self-Compacting Concrete (SCC), Conventional Concrete (CC) and Light-Weight Concrete (LWC) and their development with time are compared. Form the proposed models and comparison of the results, the following conclusions can be made:

- Generally, the achievable compressive strength is considered as the main difference between SCC, CC and LWC. However, despite different nature of mix design and proportions of SCC, CC and LWC, it is possible to reach a relatively high compressive strength in these types of concrete.
- The existing models of MoE, MoR, STS and CSSC for LWC in the literature are based on the specific conditions and particular type of lightweight aggregates in different researches. The proposed models for MoE, MoR, STS and CSSC for LWC containing Expanded Poly-Styrene (EPS) beads in this study attempt to solve this problem.
- Despite similar range of the compressive strength of the investigated SCC, CC and EPS-LWC, there is considerable difference between the other mechanical properties.
- The early-age properties and development rate in SCC, especially, the SCC mixture reinforced with different types of fibre are higher than CC and LWC. Development rate of CS, MoR and MoE in EPS-LWC with time is the least among all concrete types.
- The presence of PES beads in the used LWC, makes the CSSC different form the SCC and CC. The relatively brittle failure of EPS-LWC gives sharper descending branch of CSSR compared to that of SCC and CC. However, the ascending branch of CSSC in EPS-LWC is similar to that of the investigated CC and SCC.
- Due to brittle failure EPS-LWC compared to CC and SCC, the area under the stress-strain curve is smaller; therefore, the absorbed energy is less than that of SCC and SCC in the same grade compressive strength.
- The proposed models for MoE, MoR, STS and CSSR in this study that are on the base of the most compatible models presented in Chapter Three, are in good agreement with the experimental data. Some modifications have been applied to the best-matching existing models to get the best results of the proposed models.

11.2.1.2. Bond-slip between the embedded bar and surrounding concrete

The following conclusions can be made from the pull-out test in this study and comparison of the results with other types of concrete and literature.

- The bond failure surface through the lightweight aggregates particularly in low-strength LWC, while in CC specimens most aggregates are not broken. For higher strength levels over 40 MPa, the cracking surface is similar in both types of concrete;
- Considering SCC, CC and LWC, the ultimate bond strength and mean bond strength are the highest and lowest values in SCC and LWC respectively.
- Corresponding bond strength in LWC is relatively lower than that for CC. However, in high-strength LWC, the bond strength is greater than that of CC, due to higher mortar strength.
- Embedded length, type and diameter of steel bar in concrete, compressive strength of concrete, concrete cover, shape and size of the concrete specimen and aggregate type are the most effective parameters in the bond-slip behaviour of SCC, CC and LWC. However, due to lower values of compressive strength, effects of other parameters are more obvious in LWC.
- The maximum bond stress and bond-slip behaviour are presented in many empirical models and codes of practice. However, they are not accurate enough to predict the maximum bond stress and bond-slip relationship in EPS-LWC.
- The most compatible existing models with the experimental data of EPS-LWC are modified to propose new models to get the best compatibility with the experimental data. The proposed models of the maximum bond stress and bond-slip accurately is in agreement with the experimental data.
- Due to lower bond stress between the embedded bar in EPS-LWC cylindrical specimen in this study, there is considerable slip seen between the ribbed embedded bars and surrounding EPS-LWC. In most cases, the brittle failure of EPS-LWC specimen due to tension force ended the pull-out test. However, in some cases, there were no cracking in the cylinder even after pulling out the steel bar.
- The ultimate failure force in pull-out test is increasing by age of specimen. However, due to the improvement of the bond-strength by time, there is less slip of steel bar seen by increasing the age of the specimen.

11.2.2. Time-dependent properties of hardened concrete

11.2.2.1. Shrinkage of EPS-LWC

According to the test results and discussions presented in Chapter Six regarding the shrinkage behaviour of EPS-LWC, and comparison with the shrinkage strain of SCC and CC, the following conclusions can be made:

- The ultimate shrinkage strain in EPS-LWC is considerably higher than that of SCC and CC. The shrinkage strain in SCC and CC are similarly in the range of 800-900 $\mu\epsilon$, while it reaches to about 1200 $\mu\epsilon$.
- The Shrinkage strain of EPS-LWC is strongly influenced by the shape and size of the specimen. The cylinder specimen showed about 15% higher ultimate shrinkage strain than prism specimens for the same mixture and similar curing and laboratory conditions.
- Based on the results of different curing, drying and rewetting conditions of the shrinkage specimens of EPS-LWC, the duration of initial curing is the most effective parameter in shrinkage of EPS-LWC. The longer the initial curing time, the smaller the ultimate shrinkage strain.
- Rewetting of specimens followed by a period of drying after initial curing time shows almost no improving effect on the shrinkage of EPS-LWC.
- The existing models in ACI-435-R95, AS-3600-09 and ACI-209-08 provide underestimated shrinkage strain of EPS-LWC. The predicted ultimate shrinkage strain in ACI-209-08, AS-3600-09 and ACI-435-R95 is about 33%, 50% and 50% of the measured experimental shrinkage strain of EPS-LWC respectively.
- The empirical shrinkage model proposed by Best and Polivka (1959) gives the best prediction of the shrinkage strain of EPS-LWC compared to the other existing models.
- The proposed model in this study, accurately predicts the shrinkage strain of EPS-LWC.
- There is a remarkable relationship between the rate of weight loss (water loss) of shrinkage specimen and time. The model proposed in this study to accurately predict the weight loss of EPS-LWC specimen at different ages.

11.2.2.2. Creep of EPS-LWC

According to the test results and comparison of the creep strain and creep coefficient in EPS-LWC, SCC and CC, the following conclusions can be made:

- The creep behaviour of all types of concrete is highly sensitive to the level of sustained stress. In average, 10% higher sustained stress caused about 25% higher creep strain in the concrete specimen;

- Since, in most design codes and standards, the design creep is determined from a specified magnitude of sustained stress, application of the determined creep for other stress levels should be verified;
- The creep cylindrical specimens in SCC, CC and EPS-LWC were loaded at age 14 days. The creep coefficient in EPS-LWC is considerably higher than that of SCC and CC cylindrical specimens. While, the CC showed the least creep coefficient. Table (11.2 shows the creep coefficient of SCC, CC and EPS-LWC specimens;

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Table 11.1: Creep coefficient of CC, LWC and SCC mixtures after 240 days sustained loading

	CC	LWC	N-SCC	D-SCC	S-SCC	DS-SCC
Creep coefficient	1.5	2.415	1.96	1.86	1.82	1.45

- Despite the similar trend of creep increment at first days in all concrete types, the growing rate of creep strain in EPS-LWC after about 50 days is noticeably higher than the creep increment of SCC and CC specimens;
- The existing creep models provide underestimated creep behaviour of EPS-LWC. However, the most compatible models have been selected and modified to propose new accurate models;
- The models proposed in this study, precisely predict the creep strain and creep coefficient of EPS-LWC. In the proposed models to predict the creep strain and creep coefficient of EPS-LWC, the effect of the sustained stress level is also included.

11.2.3. Experimental program; Flexural Load-Deflection Behaviour

The short-term load increment until the failure and corresponding deflection values at mid-span of slabs were monitored. From the experimental results and comparison of the load-deflection behaviour in SCC, CC and EPS-LWC slabs, the following conclusions can be made:

Table (11.2) shows the failure load and the maximum deflection of the slab at mid-span in the short-term failure loading.

- The slab geometry (span length and section dimensions) and reinforcement details in all SCC, CC and EPS-LWC slabs were similar. In addition, despite the similar range of the compressive strength of the slabs, there was a significant difference in the modulus of elasticity of applied SCC, CC and EPS-LWC mixtures in the slabs.
- Considering the ratio of tensile reinforcement in the section, all slabs experienced ductile failure under the increasing flexural loading. In other words, the ultimate

bending capacity of the slab section was more sensitive to the yield strength of the steel bars rather than the compressive strength of the concrete.

- As presented in Table (11.2), there is no considerable difference in the failure loads of SCC, CC and EPS-LWC slabs and all failure loads are in the range of 50 KN. While
- Despite the similar range of the failure load in all slabs, there is a substantial difference in the maximum deflection of slabs at mid-span at failure load.
- Fibre reinforcing of the SCC mixture improves the ductility of the slabs. The ultimate flexural deflection of these slabs is considerably higher than the other slabs.
- Compared to SCC and CC slabs, the lower value of the modulus of elasticity in EPS-LWC slabs resulted in a smaller amount of the maximum deflection at failure load. The relatively brittle failure of the EPS-LWC slabs may be considered as the reason of the difference in the recorded ultimate deflection.

Table 11.2: Summary of the results from short-term flexural test

	Beams	Failure load (KN)	Deflection at failure load (mm)		Slabs	Failure load (KN)	Deflection at Failure load (mm)
Self-compacting concrete slabs	N-SCC-A	49	180	Lightweight concrete slabs	LWC-1	46.5	95.9
	N-SCC-B	48.5	163		LWC-2	48	88
	D-SCC-A	53	205		LWC-3	41.53	114.7
	D-SCC-B	52	177		LWC-4	49.52	86.65
	S-SCC-A	50	185	Conventional concrete slabs	S3-a	50	136
	S-SCC-B	48	167		S3-b	47	156
	DS-SCC-A	56	220				
	DS-SCC-B	54	182				

- There is a noticeable difference in cracking pattern of the slabs under increasing load in terms of the crack spacing and crack width. The average crack spacing in SCC and CC slabs are less than in EPS-LWC slabs. However, the crack height and crack width in SCC and CC slabs are higher than those in EPS-LWC slabs.
- As a general conclusion of the load-deflection of slabs, the failure load, and corresponding maximum deflection should be considered simultaneously in the design of one-way slabs.

11.2.4. Experimental program; Flexural Instantaneous deflection

The flexural deflection of slabs immediately after loading at age 14 days was recorded. In this regard, three different types of reinforced concrete slabs made with normal, lightweight and self-compacting concrete slabs was investigated under two distinct levels of service loads. From the experimental results and comparison of the flexural instantaneous deflection of SCC, CC and EPS-LWC slabs, the following conclusions can be made:

- The concrete type strongly affects the deflection behavior of the reinforced concrete slabs;
- Elastic deflection is not a trustworthy indicator of the instantaneous, time-dependent and total deflection of slabs;
- Considering the elastic deflection (Δ_e) equal to instantaneous deflection (Δ_{ins}) even for service load under the cracking load in slabs is completely wrong and causes unsafe design of reinforced concrete slabs to calculate the time-dependent and long-term deflection.
- The ratio of measured instantaneous deflection to calculated elastic deflection at 14 days varies from 1.14 to 3.8 in one-way slabs.

Table (11.3) shows the applied uniformly distributed service load (w_a), service moment (M_a), calculated elastic deflection at 14 and 28 days and the measured instantaneous deflection of slabs at 14 days.

Table 11.3: Loading values, recorded and elastic deflection of SCC, CC and EPA-LWC slabs

Slab	w_a (KN/m)	M_a (KN.m)	Δ_e (mm)		Δ_{ins} (mm)
			14 day	28 day	
N-SCC-a	7.31	11.189	3.18	2.9	12.1
N-SCC-b	5.41	8.28	2.36	2.15	5.89
D-SCC-a	7.26	11.12	3.5	2.85	7.65
D-SCC-b	5.36	8.21	2.58	2.11	7.59
S-SCC-a	7.3	11.18	3.45	2.87	6.41
S-SCC-b	5.4	8.27	2.55	2.12	2.91
DS-SCC-a	7.33	11.23	3.29	2.85	8.98
DS-SCC-b	5.43	8.32	2.44	2.11	5.14
LWC-1	7	10.72	4.99	4.57	11.96
LWC-2	7	10.72	4.99	4.57	11.4
LWC-3	5.29	8.095	3.77	3.45	6.3
LWC-4	5.29	8.095	3.77	3.45	5.98
CC-a	4.44	6.8	2.73	2.5	11.8
CC-b	3.46	5.3	2.13	1.95	5.04

- Effective moment of inertia (I_e) is the most critical parameter in calculating the flexural instantaneous deflection. In majority of the existing models to predict the effective

moment of inertia, effects of applied service moment (M_a), cracking moment (M_{cr}), the gross moment of inertia (I_g) and cracking moment of inertia (I_{cr}) are included. This study confirms the implementation of the loading age factor and ultimate moment capacity of the reinforced concrete section. In fibre reinforced sections, the effect of fibre volume in the mixture should be considered in the I_e model.

- The ratio of service moment to the ultimate moment capacity of slabs strongly affects the instantaneous deflection;
- In the existing models in the literature, there is an almost linear relation between the load value and the instantaneous deflection of slabs. The results of this study showed a considerable effect of the load level, especially in the low strength concrete slabs;
- The instantaneous deflection due to 10% of load increment increased about 100%, and 105% in the lightweight concrete and self-compacting concrete slabs. In low-strength normal concrete slabs, only 6% higher load resulted in 135% higher instantaneous deflection;
- The instantaneous deflection of fiber reinforced self-compacting concrete slabs experienced fewer variations with the changes of loading level;
- Development of the mechanical properties of normal, lightweight and self-compacting concrete is different before the age of 28 days. Therefore, for the early age loading, the calculations based on the mechanical properties at the age of 28 days may result in the wrong values for the instantaneous deflection of the slabs;
- Almost all existing I_e models in the literature estimate higher values of the instantaneous deflection for the investigated slabs. The most compatible models in the literature are selected as the basis of new proposed models in this study. Implementation of the loading age, ratio of Modification on the age
- A new model is proposed and verified to predict the effective moment of inertia in the self-compacting concrete slabs with and without fiber reinforcing. The predicted instantaneous deflections by this model are in agreement with the experimental results;
- Another model of the effective moment of inertia is proposed and verified for the normal and lightweight concrete slabs. The experimentally recorded instantaneous deflections are well-predicted by this model.

11.2.5. Experimental program; flexural long-term deflection

The flexural deflection of slabs under sustained service loads and effects of creep and shrinkage was recorded. In this regard, three different types of reinforced concrete slabs made

with SCC, CC and EPS-LWC slabs were investigated under two distinct levels of service loads. From the experimental results and comparison of the flexural long-term deflection of SCC, CC and EPS-LWC slabs, the following conclusions can be made:

- The concrete type strongly affects the time-dependent deflection behavior of reinforced concrete slabs;
- In EPS-LWC slabs, the time-dependent deflection was monitored at mid-span and the point on the border of the high moment region. The general trend of the time-dependent deflection of at these points was similar. However, the ratio of deflection at mid-span to the deflection at the border point of high moment region is considerably different in the slabs under different loading levels.
- The long-term deflection is considered as the multiplier of the instantaneous deflection in the majority of design codes. Table (11.4) shows the measured deflections of SCC, CC and EPS-LWC slabs at the time zero after loading (instantaneous deflection) and the final deflection after long-term sustained loading.
- Regardless of the loading level, the deflection multiplier in design codes varies in the range of 2 to 3. According to Table (11.4), this ratio is in agreement with the long-term deflection of slabs at lower loading level. By 10% increase in the loading level, the deflection multiplier increases about 50% above the range of design codes.
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Table 11.4: Ratio of the long-term to instantaneous deflection of LWC, SCC and CC slabs

Slab	M_a / M_u (%)		Test duration (days)	Δ_{tot} (mm)	Δ_{ins} (mm)	$\Delta_{tot} / \Delta_{ins}$
	14 day	28 day				
N-SCC-a	40.73	40.27	240	24.76	12.1	2.04
N-SCC-b	30.14	29.8	240	18.08	5.89	3.07
D-SCC-a	39.94	39.65	240	17.76	7.65	2.32
D-SCC-b	29.48	29.27	240	16.78	7.59	2.21
S-SCC-a	40.32	39.85	240	22.27	6.41	3.47
S-SCC-b	29.83	29.48	240	20.25	2.91	6.95
DS-SCC-a	40.03	39.63	240	21.31	8.98	2.37
DS-SCC-b	29.66	29.36	240	15.83	5.14	3.08
LWC-1	39.13	38.78	525	34.56	11.96	2.89
LWC-2	39.13	38.78	525	31.81	11.4	2.79
LWC-3	29.55	29.28	525	26.4	6.3	4.19
LWC-4	29.55	29.28	525	24.52	5.98	4.1
CC-a	26.12	25.14	400	32.1	11.8	2.72
CC-b	20.36	19.59	400	21.92	5.04	4.35

Figures (11.1) and (11.2) compare the mid-span deflection versus time in the SCC, CC and EPS-LWC slabs under the lower and higher loading levels, respectively. The loading level of each slab is shown in Table (11.4).

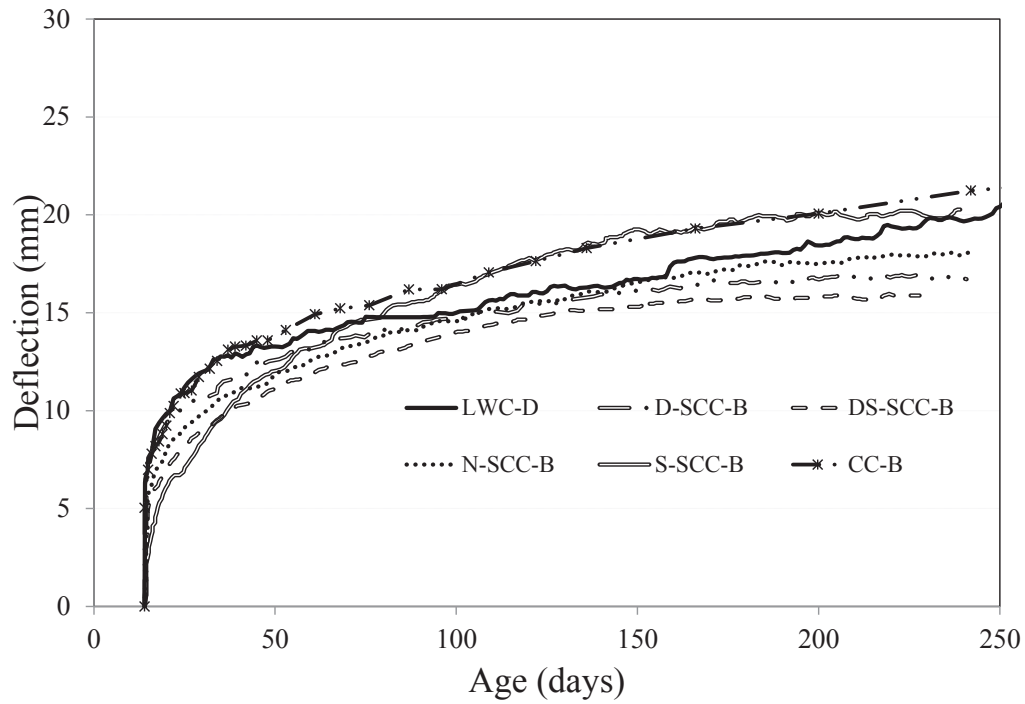


Figure 11.1: Comparison of deflection vs. time in SCC, LWC and CC slabs under lower loading level

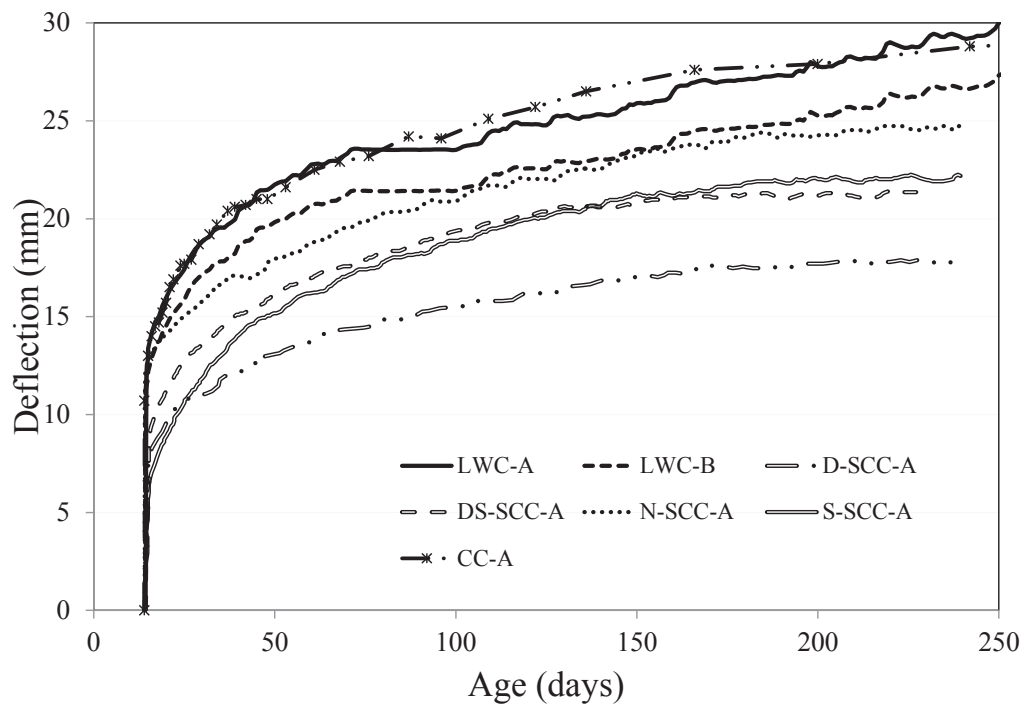
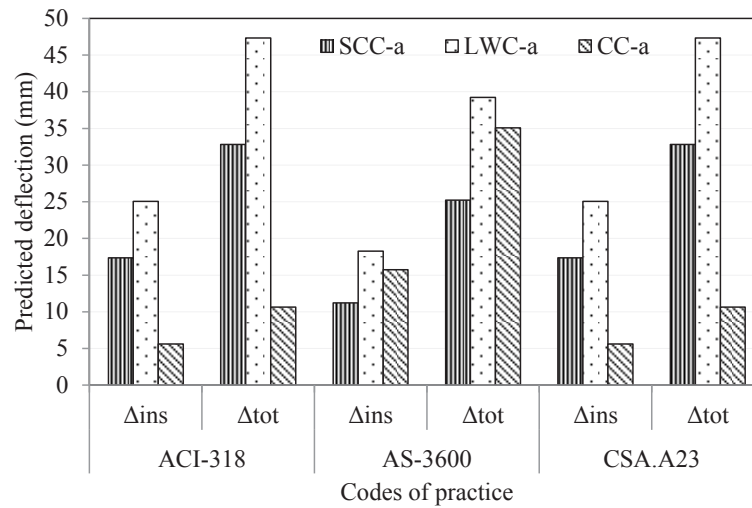


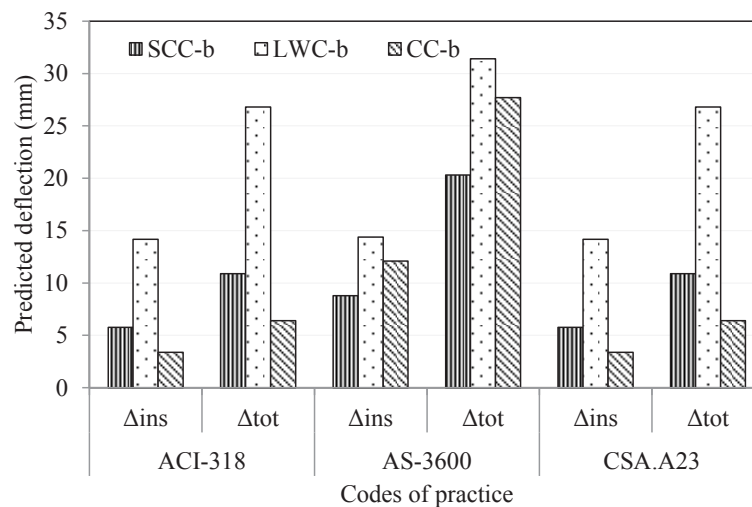
Figure 11.2: Comparison of deflection vs. time in SCC, LWC and CC slabs under higher loading level

Figures (11.3 a, b) compare the predictions of AS-3600 (2009), ACI-318 (2008) and CSA.A23 (2004) for the instantaneous and total (long-term) deflection of SCC, LWC and CC slabs under two different levels of sustained loading respectively.

- The instantaneous and total deflection in the slabs under higher loading level is significantly greater than those of the slabs under lower loading levels, especially by ACI-318 and CSA.A23 predictions. However, in predicted values of AS-3600, the difference between the Δ_{ins-14} and Δ_{long} is less, compared to the other design codes.
- According to Table (11.4), the experimental ratio of $\Delta_{tot} / \Delta_{ins-14}$ varies between 2.04 and 4.35; however, the investigated codes of practice predict the ratio in the range of 1.89 to 2.31.



(a)



(b)

Figure 11.3: Predicted Δ_{ins} and Δ_{tot} of SCC, LWC and CC slabs under two different loading levels

- There is a considerable difference between the predicted Δ_{ins-14} and Δ_{long} of slabs under different load values. The instantaneous and total deflection in the slabs under higher loading level is significantly greater than those of the slabs under lower loading levels, especially by ACI-318 and CSA.A23 predictions. However, in predicted values of AS-3600, the difference between the Δ_{ins-14} and Δ_{tot} is less, compared to the other design codes.
- All listed codes in Figures (11.3 a, b) predict the highest values of Δ_{ins-14} and Δ_{tot} for EPS-LWC slabs under both loading levels. However, AS-3600 predicts the lowest Δ_{ins-14} and Δ_{tot} values for SCC slabs under both loading levels.
- By comparing the ratio of $\Delta_{tot} / \Delta_{ins-14}$ in Table (11.4) and the drying shrinkage strain of SCC-CC and EPS-LWC, there is no obvious relationship between the ratio of $\Delta_{tot} / \Delta_{ins-14}$ and the shrinkage behaviour of slabs.
- The rate of deflection (mm/day) and the free shrinkage rate ($\mu\epsilon$ /day) in SCC, CC and EPS-LWC are not compatible.

11.2.6. Numerical investigations

Two types of problems in ATENA, namely ATENA-Static and ATENA-Creep were utilized to study the instantaneous and long-term deflection of the EPS-LWC slabs respectively. In each problem type, a comprehensive parametric study was performed to calibrate the simulated models to show the most compatible results with the experimental data. The following results can be drawn from the finite element analysis of the calibrated models in ATENA in the static and creep problem type:

- The proposed models of mechanical properties of EPS-LWC in Chapter Six are verified;
- The finite element models verify the proposed creep and shrinkage models of lightweight concrete in Chapter Six;
- 3D analysis is the most accurate method, however, regarding the symmetric geometry of slabs and loading and supporting conditions, 2D analysis can be used with high accuracy of the results both in static and creep analysis;
- In 2D analysis, the plane stress analysis of the models shows better agreement with the experimental data;
- The smaller mesh size and higher number of the loading steps increase the accuracy of the results both in static and creep analysis;
- The loading level has a considerable effect on the obtained results, particularly on the creep analysis. In other words, loading level can change the creep effect on the structure;

- The slab models were analysed by applying different creep models and compared with the experimental data. The loading level changed the effect of each creep on the slab deflection;
- Time-dependent analysis of the models shows the higher effect of the shrinkage strain than compliance of the concrete, on the long-term deflection of concrete slabs;
- In the first loading level ($0.3M_u$), FIBMC-2010, CEB-FIP78 and BP1 creep models give the best estimation of the time-dependent deflection of lightweight concrete slabs;
- In the second loading level ($0.4M_u$), CSN-731201 creep model gives the best estimation of the time-dependent deflection of lightweight concrete slabs. Overall, FIBMC-2010 is the most compatible model for the time-dependent behaviour of LWC slab;
- Among the creep models, BP2 is strongly influenced by the load level, while FIBMC-2010 shows the least variation with the experimental results under two loading levels.

Figures (11.4) and (11.5) show the compatibility between the experimental data and predictions of the proposed creep and shrinkage models in this study.

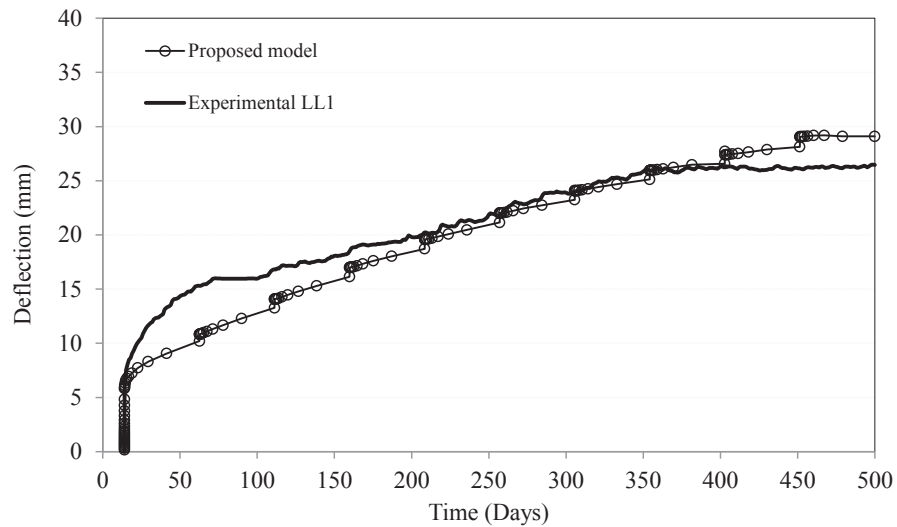


Figure 11.4: comparison of the proposed model with the experimental data in loading level 1

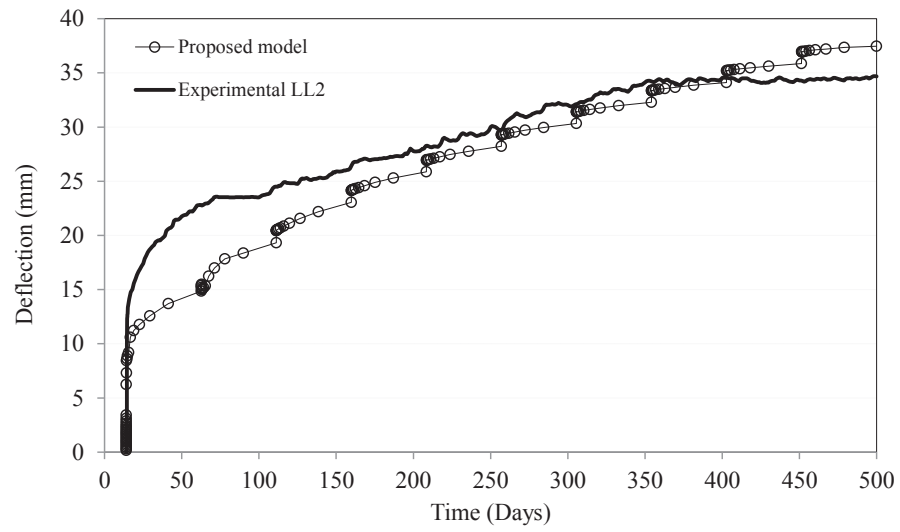


Figure 10.5: Comparison of the proposed model with experimental data in loading level 2

11.3. RECOMMENDATIONS FOR FUTURE RESEARCH

The following areas of research regarding the lightweight concrete members remain relatively unexplored and could form the basis for future studies:

11.3.1. Mechanical properties:

- The structural lightweight concrete members are excellent solutions for lighter structures with higher performance. However, fibre reinforcing can increase the performance and mechanical properties of lightweight structures while the gravity loads and the resultant lateral effects are almost unchanged. A wider investigation is required to develop the flexural failure mechanism of fibre reinforced lightweight concrete to use in beams and slabs.
- There are many sources of natural and artificial lightweight aggregates to make the lightweight concrete. Comparing the mechanical properties of lightweight concrete with different types of lightweight aggregates to achieve a similar structural concrete (density and compressive strength) might be an interesting topic in choosing the most economic and accessible aggregates to make the lightweight concrete;
- Considering the higher thermal and sound insulation, and better energy absorption of lightweight concrete, an experimental program to compare the lightweight and the new types of construction materials might be interesting in the construction industry.

11.3.2. Structural behaviour

- The current study emphasized the flexural capacity of the lightweight concrete structures, while other failure mechanisms such as shear, axial and torsional failures are not taken into account. Considering the shear failure importance in most engineering structures, more investigations are needed to develop an accurate failure mechanism to describe the collapse of lightweight concrete slabs with and without fibre reinforcement.
- Surprisingly, experimental studies about the time-dependent properties of lightweight concrete are very rare in the literature. The long-term experimental program can help to discover the uncertain parameters in the serviceability of lightweight concrete members with and without fibre reinforcement.
- Load-bearing capacity, fatigue and the effects of cyclic and repeating loads in short or long-term durations in the structural lightweight concrete still require more studies.

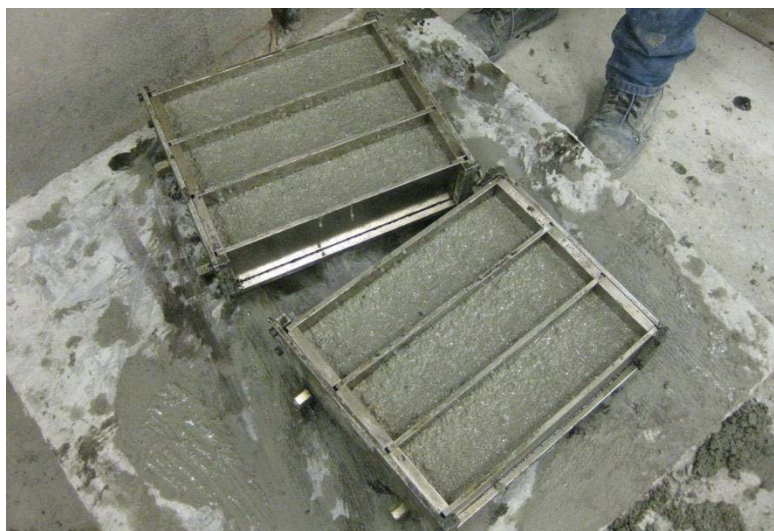
11.3.3. Especial applications

- An extensive experimental and numerical program is required to investigate the long-term behaviour of precast prestressed lightweight concrete members;
- Lightweight concrete containing expanded polystyrene beads is frequently used in marine structures and decks;
- Application of nanotechnology to modify the mixture design and structural performance of the lightweight concrete containing expanded polystyrene beads is almost new and requires more investigation;
- Lightweight concrete is a type of green materials in regard to the lower weight and consequently reduced gas emission, reduced pollution and fewer mining and logistics problems. The comparison of the energy saving and environmental considerations in building with conventional and different types of lightweight concrete requires further studies;
- Sustainability and durability are the main factors in the application of the concrete in most of the structures. Studying the freeze-thaw resistance, chloride penetration and other chemical attacks particularly in the absence of harsh conditions experienced by marine structures requires empirical and numerical investigation.

APPENDIX A

MATERIAL TESTS

PREPARATION OF TEST SPECIMENS









PULL OUT TEST









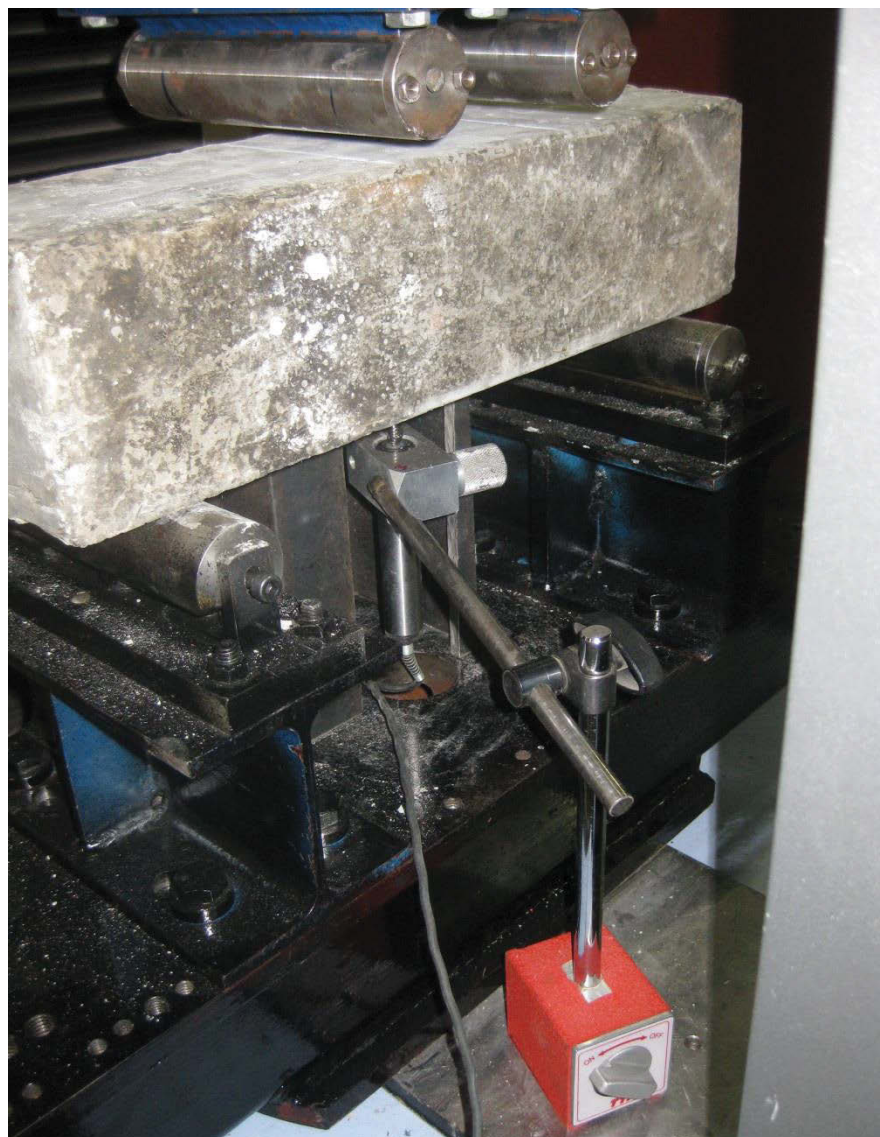
SPLITTING TENSILE STRENGTH





MODULUS OF RUPTURE





APPENDIX B

MANUFACTURING THE SLABS AND TEST SETUP FOR LOAD-DEFLECTION TEST, INSTANTANEOUS DEFLECTION MONITORING AND LONG-TERM DEFLECTION MONITORING

