



DEVELOPMENT OF HIGH THERMAL CRACK RESISTING MASS CONCRETE

BY

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**SIRINDHORN INTERNATIONAL INSTITUTE OF TECHNOLOGY
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THESIS

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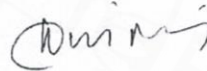
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DEVELOPMENT OF HIGH THERMAL CRACK RESISTING MASS CONCRETE

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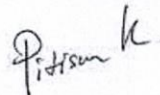
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ABSTRACT

According to the large dimension of mass concrete, it will generally have a problem with the heat generation from cement hydration and the temperature difference between inside and near surface of the concrete that may lead to thermal cracking. Nowadays, many efforts are implemented to minimize thermal cracking potential in mass concrete structures. Existing materials and construction methods have limitations and cannot cope with the requirements, so new solutions with new binder and aggregate systems need to be explored. Supplementary cementitious materials or SCMs provided in concrete i.e., fly ash, limestone powder, etc., have been used widely to reduce the heat generation. SCMs have been reported to improve properties of concrete such as slump and slump loss, compressive strength, and temperature rise.

To develop thermal crack resisting mass concrete, the investigation on the binder system and aggregate system should be done. In the case of binder system, the SCMs which are fly ash and limestone powder are used as cement replacing materials. To control the temperature difference in mass concrete, the reliable temperature prediction model is required. There are some existing temperature rise prediction models that have been developed to simulate the temperature rise for mass concrete. However, those proposed models were still not considered for ternary binder i.e.,

cement containing fly ash and limestone powder. In the case of the aggregate system, the temperature rise of concrete with the maximum size of coarse aggregate (G_{max}) equal to 1 inch instead of $\frac{3}{4}$ inch is investigated. Moreover, the experiments on slump and slump loss, compressive strength, and tensile strain capacity were done for both binder and aggregate system investigations.

The study reveals for the investigation on the binder system that higher cement content mixtures show higher compressive strength, tensile strain capacity (TSC), and semi-adiabatic temperature rise. The adiabatic temperature rise prediction model for ternary binder system was done by considering degree of hydration, degree of pozzolanic reaction, total heat generation, free water content, and thermal properties. The proposed adiabatic temperature rise prediction model was then verified and showed reasonable accuracy with laboratory results. For the investigation on the aggregate system, the experimental results show that the mechanical properties which are compressive strength and tensile strain capacity of concrete are improved with the concrete containing maximum size of coarse aggregate equal to 1 inch instead of $\frac{3}{4}$ inch. Moreover, the maximum temperature rise for concrete was also reduced when G_{max} 1 inch was used in the concrete.

Keywords: Fly ash, Limestone powder, Maximum size of coarse aggregate, Mass concrete, Adiabatic temperature rise, Thermal properties, Simulation model

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Mr. Kanin Pinitoppapun

TABLE OF CONTENTS

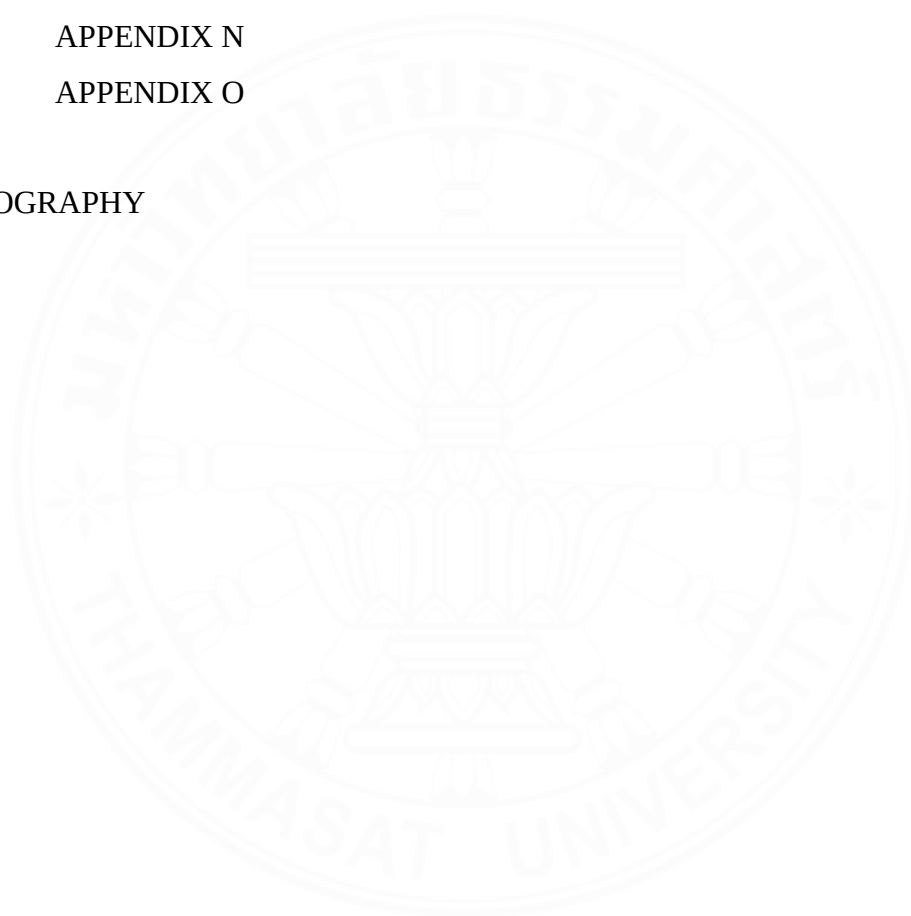
	Page
ABSTRACT	(1)
ACKNOWLEDGEMENTS	(3)
LIST OF TABLES	(9)
LIST OF FIGURES	(10)
LIST OF SYMBOLS/ABBREVIATIONS	(16)
CHAPTER 1 INTRODUCTION	1
1.1 Background of the study	1
1.2 Statement of problems and significance of the study	2
1.3 Objectives of the study	3
1.4 Scopes of the study	3
CHAPTER 2 REVIEW OF LITERATURES	7
2.1 Introduction	7
2.2 Introduction to fly ash and limestone powder	8
2.2.1 Fly ash	8
2.2.2 Limestone powder	8
2.3 Properties of fly ash concrete	8
2.4 Properties of limestone powder concrete	11
2.5 Properties of ternary binder concrete by using the combination of fly ash and limestone powder	13
2.6 Properties of the maximum size of aggregate (G _{max}) equal to 1-inch concrete	19
2.7 Parameters affecting heat generation, and thermal properties of concrete	22

CHAPTER 3 THEORETICAL BACKGROUND	29
3.1 Adiabatic temperature rise prediction model introduction	29
3.2 Semi-adiabatic temperature rise prediction model introduction	30
CHAPTER 4 FREE WATER CONTENT	32
4.1 Free water content prediction model	32
4.2 Materials	34
4.3 Specimen preparation	35
4.4 Experimental procedures	37
4.5 Experimental results	37
4.5.1 Effects of water to binder ratio	38
4.5.2 Effects of binders	38
4.6 Verification of free water content prediction model	41
CHAPTER 5 SPECIFIC HEAT	43
5.1 Specific heat prediction model	43
5.2 Materials	44
5.3 Specimen preparation	45
5.4 Experimental procedures	47
5.5 Experimental results	49
5.5.1 Effects of water to binder ratio	49
5.5.2 Effects of binders	49
5.6 Verification of specific heat prediction model	53
CHAPTER 6 DEGREE OF HYDRATION REACTION	55
6.1 Degree of hydration reaction prediction model	56
6.1.1 Effects of fly ash and limestone powder on degree of hydration reaction	57
6.1.2 Effects of fly ash on retardation of C_3S reaction	60
6.2 Average degree of hydration reaction	61

CHAPTER 7 TOTAL HEAT GENERATION	63
7.1 Total heat prediction model	63
7.2 Cumulative heat generation	64
7.2.1 Cumulative heat generation of each cement compound	64
7.2.2 Cumulative heat generation of fly ash	67
7.2.3 Cumulative heat generation of ettringite and monosulphate	68
7.3 Calculation of adiabatic temperature rise	68
CHAPTER 8 THERMAL CONDUCTIVITY	70
8.1 Thermal conductivity prediction model	70
CHAPTER 9 EFFECTS OF FLY ASH AND LIMESTONE POWDER ON CONCRETE PROPERTIES OF TERNARY BINDER CONCRETE AND VERIFICATION WITH THE PROPOSED MODELS	74
9.1 Materials and mix proportions	74
9.2 Experimental programs	77
9.2.1 Slump and slump loss	77
9.2.2 Compressive strength	77
9.2.3 Tensile strain capacity	79
9.2.4 Semi-adiabatic temperature rise	81
9.2.5 Verification of the proposed model by semi-adiabatic temperature rise	82
9.3 Experimental results	83
9.3.1 Slump and slump loss	83
9.3.2 Compressive strength	84
9.3.3 Tensile strain capacity	84
9.3.4 Relationship between Compressive strength and Tensile strain capacity	85
9.3.5 Semi-adiabatic temperature rise	86
9.3.6 Verification of the proposed model by semi-adiabatic temperature rise	87

CHAPTER 10 EFFECTS OF THE MAXIMUM SIZE OF COARSE AGGREGATE ON TERNARY BINDER CONCRETE	91
10.1 Materials and mix proportions	91
10. 2 Experimental programs	96
10. 3 Experimental results	96
10.3.1 Slump and slump loss	96
10.3.2 Compressive strength	97
10.3.3 Tensile strain capacity	98
10.3.4 Relationship between Compressive strength and Tensile strain capacity	99
10.3.5 Semi-Adiabatic temperature rise	100
10.3.6 Verification of the proposed model by semi-adiabatic temperature rise with FA50 and FA50LP10 with Gmax equals to $\frac{3}{4}$ and 1 inch concrete	100
CHAPTER 11 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY	103
11.1 Conclusions	103
11.1.1 The study of binder system	103
11.1.2 The study of aggregate system	104
11.2 Recommendations for future study	105
11.2.1 The study of binder system	105
11.2.2 The study of aggregate system	106
REFERENCES	107
APPENDICES	115
APPENDIX A	116
APPENDIX B	117
APPENDIX C	118
APPENDIX D	119
APPENDIX E	120
APPENDIX F	121

APPENDIX G	122
APPENDIX H	123
APPENDIX I	124
APPENDIX J	126
APPENDIX K	127
APPENDIX L	128
APPENDIX M	129
APPENDIX N	130
APPENDIX O	131
BIOGRAPHY	132



LIST OF TABLES

Tables	Page
4.1 Chemical compositions and physical properties of binders for free water content tests	35
4.2 Mix proportions for free water content tests	37
5.1 Specific heat values of concrete ingredients by Choktaweeakarn (2008 and 2010)	44
5.2 Chemical compositions and physical properties of binders for specific heat tests	45
5.3 Mix proportions for specific heat tests	47
8.1 The thermal conductivity values of concrete ingredients by Choktaweeakarn (2008 and 2010)	71
8.2 Mix proportions for studying the proposed thermal conductivity prediction model for ternary binder concrete	72
9.1 Chemical compositions and physical properties of tested materials	75
9.2 Mix proportions of the tested concrete	77
10.1 Chemical compositions and physical properties of tested materials	92
10.2 Mix proportions of the tested concrete	96

LIST OF FIGURES

Figures	Page
1.1 Illustration chart for the investigation on ternary binder system	5
1.2 Illustration chart for the investigation on different maximum sizes of coarse aggregate system	6
2.1 Temperature rise of concrete comparing between non-fly ash replacement, and fly ash replacement at 0.15, 0.3, and 0.5 (Sheikh, 2001)	9
2.2 Temperature rise result of concrete comparing between control specimen, and fly ash replacing at 0.2 and 0.4 (Sheikh, 2001)	9
2.3 Adiabatic temperature rise result of concrete comparing between control specimen and fly ash replacing at 0.2 and 0.4 (Ng et al., 2011)	10
2.4 Adiabatic temperature rise result and rate of adiabatic temperature rise of concrete comparing between normal concrete and fly ash replacing at 0.4, 0.5, 0.6, and 0.7 (Wang et al., 2011)	11
2.5 Comparison between temperature rise result with model verification of normal concrete and concrete with limestone powder (Kishi et al., 1999)	12
2.6 Comparison between temperature rise result with model verification of limestone powder concrete (Kishi et al., 1999)	12
2.7 Experimental result for heat development of pastes with fly ash replacing at 0, 0.1, 0.2, and 0.3 (Thongsanitgarn et al., 2014)	14
2.8 Experimental result for heat development of pastes with limestone powder replacing at 0, 0.05, 0.1, 0.15, and 0.3 (Thongsanitgarn et al., 2014)	14
2.9 Experimental result of heat development of ternary binder system of pastes with fly ash and limestone powder as cement replacing materials (Thongsanitgarn et al., 2014)	15
2.10 Adiabatic temperature rise results of normal concrete comparing with fly ash and/or limestone powder used as cement replacing material (Si-Jun et al., 2015)	15
2.11 Comparison of experimental result and calculation model result reaction degree of cement and reaction degree of fly ash (Wang, 2018)	16

2.12 Comparison of slump and slump loss of normal concrete, LP5, LP10, FA30, FA25LP5, and FA20LP10 (Kaewmanee et al., 2014)	17
2.13 Compressive strength of concrete at the testing age of 1, 28, 90, 140 days (De Weerd et al., 2010)	18
2.14 Compressive strength of concrete at various fly ash and limestone powder replacing levels at the testing age of 1, 28, 90, 140 days (De Weerd et al., 2010)	18
2.15 Compressive strength of mortars at various fly ash and limestone powder replacement levels (Mounanga et al., 2010)	19
2.16 Temperature envelopment of concrete (Lagundžija et al., 2017)	20
2.17 Maximum temperature development of concrete (Lagundžija et al., 2017)	20
2.18 Compressive strength of concrete at 7, 14, and 28 days (Meddah et al., 2009)	21
2.19 Compressive strength of concrete for the coarse aggregate maximum size of 9.5, 12.7, and 19.1 and non-uniform gradation. (Ekwulo et al., 2017)	22
2.20 Hydration heat rate of cement pastes containing limestone powder at 0, 10, and 30 with Blaine's fineness 3500, 4700, and 8900. (Kishi et al., 1999)	23
2.21 Rate of heat development for cement pastes containing fly ash and limestone powder. (De Weerd 2007)	24
2.22 Rate of the heat of hydration for cement pastes containing fly ash and limestone powder. (De Weerd 2010)	24
2.23 Hydration heat flow of cement pastes containing fly ash or limestone filler (Mounanga et al., 2010)	25
2.24 Hydration heat flow of cement pastes containing fly ash and limestone filler (Mounanga et al., 2010)	25
2.25 Heat flow of cement pastes containing fly ash and limestone powder (Pailyn et al., 2014)	26
3.1 Adiabatic temperature rise chart based on Saengsoy (2002)	29
3.2 Semi-adiabatic temperature rise chart based on Choktaweekarn (2008)	31
4.1 Plastic molds completely sealed for the free water content specimen	36
4.2 Container for keeping the free water content testing molds in the control room	36
4.3 Free water content results of cement pastes with w/b ratios of 0.25 and 0.4	38

4.4 Free water content results of binary binder pastes with OPC, FA30, FA50, and LP10 at w/b ratio of 0.25	39
4.5 Free water content results of binary binder pastes with OPC, FA30, FA50, and LP10 at w/b ratio of 0.4	39
4.6 Free water content results for cement pastes with OPC, FA30, LP10, and FA30LP10 at w/b ratio of 0.25	40
4.7 Free water content results for cement pastes with OPC, FA50, LP10, and FA50LP10 at w/b ratio of 0.25	40
4.8 Free water content results for cement pastes with OPC, FA30, LP10, and FA30LP10 at w/b ratio of 0.4	41
4.9 Free water content results for cement pastes with OPC, FA50, LP10, and FA50LP10 at w/b ratio of 0.4	41
4.10 Comparison between free water content experimental result and model of pastes with FA30LP10 and FA50LP10 at w/b ratio of 0.25	42
4.11 Comparison between free water content experimental result and model of pastes with FA30LP10 and FA50LP10 at w/b ratio of 0.4	42
5.1 The specimen preparation for specific heat testing	46
5.2 The insulated container for specific heat testing	47
5.3 The apparatus for specific heat testing	48
5.4 Experimental setup for specific heat testing	48
5.5 Specific heat results of cement pastes with FA30LP10 and FA50LP10 at w/b ratios of 0.25 and 0.4	49
5.6 Specific heat results of cement pastes with OPC, FA30, FA50, and LP10 at w/b ratio of 0.25	50
5.7 Specific heat results of cement pastes with OPC, FA30, FA50, and LP10 at w/b ratio of 0.4	51
5.8 Specific heat results of cement pastes with OPC, FA30, LP10, and FA30LP10 at w/b ratio of 0.25	51
5.9 Specific heat results of cement pastes with OPC, FA30, LP10, and FA30LP10 at w/b ratio of 0.4	52
5.10 Specific heat results of cement pastes with OPC, FA50, LP10, and FA50LP10 at w/b ratio of 0.25	52

5.11 Specific heat results for cement pastes with OPC, FA50, LP10, and FA50LP10 at w/b ratio of 0.4	53
5.12 Comparison between specific heat experimental result and model of pastes with FA30LP10 and FA50LP10 at w/b ratio of 0.25	54
5.13 Comparison between specific heat experimental result and model of pastes with FA30LP10 and FA50LP10 at w/b ratio of 0.4	54
6.1 Physical acceleration factor of C ₃ A with FA30, LP10, and FA30LP10	59
6.2 Physical acceleration factor of C ₃ A with FA50, LP10, and FA50LP10	59
6.3 Physical acceleration factor of C ₃ S with FA30, LP10, and FA30LP10	60
6.4 Physical acceleration factor of C ₃ S with FA50, LP10, and FA50LP10	60
6.5 Average degree of hydration of cement paste with FA30, FA50, LP10, FA30LP10 and FA50LP10	62
7.1 Total heat generation simulation results for OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10	64
7.2 Dispersion factor for concrete with FA30, FA30LP10, and FA30LP20	66
7.3 Dispersion factor for concrete with FA50, FA50LP10, and FA50LP20	67
8.1 Thermal conductivity prediction model results for concrete with OPC, FA30, LP10, and FA30LP10	72
8.2 Thermal conductivity prediction model results for concrete with OPC, FA50, LP10, and FA50LP10	73
9.1 Particle size distribution of fine aggregate based on ASTM C33	75
9.2 Particle size distribution of coarse aggregate based on ASTM C33	76
9.3 The specimen's preparation in the plastic cube covered with the plastic wrap	78
9.4 The experimental setup for the compressive strength testing	79
9.5 Preparation for concrete beam casting in the steel molds	80
9.6 Apparatus for tensile strain capacity testing	80
9.7 Experimental setup for tensile strain capacity testing	81
9.8 The inside of the prepared boxes for the semi-adiabatic temperature rise testing	81
9.9 Experimental setup for the semi-adiabatic temperature rise testing	82
9.10 Specimens with the temperature rise history recording system	82

9.11 Initial slump and slump loss for concrete with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4	83
9.12 Compressive strength results for concrete with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4	84
9.13 The tensile strain capacity results for concrete OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4	85
9.14 Relationship between compressive strength and tensile strain capacity with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4	85
9.15 Semi-adiabatic temperature rise profile for concrete with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4	86
9.16 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with OPC	87
9.17 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with LP10	88
9.18 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA30	88
9.19 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50	89
9.20 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA30LP10	89
9.21 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50LP10	90
10.1 Particle size distribution of fine aggregate based on ASTM C33	92
10.2 The particle size distribution of coarse aggregate with Gmax equals to 1 inch based on ASTM C33 grade 67	93
10.3 The particle size distribution of coarse aggregate with Gmax equals to $\frac{3}{4}$ inch based on ASTM C33 grade 7	93
10.4 The comparison of upper and lower limit of coarse aggregate grade 67 and 7 based on ASTM C33	95
10.5 The comparison of void content between coarse aggregate with Gmax equal to $\frac{3}{4}$ inch and 1 inch	95

10.6 Initial slump and slump loss of concrete with FA50 and FA50LP10 with G _{max} ¾ inch and 1 inch at w/b ratio of 0.4	97
10.7 Compressive strength of concrete with FA50 and FA50LP10 with G _{max} ¾ inch and 1 inch at w/b ratio of 0.4	98
10.8 Tensile strain capacity of concrete with FA50 and FA50LP10 with G _{max} ¾ inch and 1 inch at w/b ratio of 0.4	99
10.9 Relationship between compressive strength and tensile strain capacity with FA50 and FA50LP10 with G _{max} ¾ inch and 1 inch at w/b 0.4	99
10.10 Semi-adiabatic temperature rise profiles of concrete with FA50 and FA50LP10 with G _{max} ¾ inch and 1 inch at w/b 0.4	100
10.11 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50 with G _{max} ¾ inch at w/b 0.4	101
10.12 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50LP10 with G _{max} ¾ inch at w/b 0.4	101
10.13 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50 with G _{max} 1 inch at w/b 0.4	102
10.14 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50LP10 with G _{max} 1 inch at w/b 0.4	102

LIST OF SYMBOLS/ABBREVIATIONS

Symbols/Abbreviations	Terms
ACI	American concrete institute
G _{max}	Maximum size of coarse aggregate
SCMs	Supplementary cementitious materials
ASTM	American society for testing and materials
EIT	Engineering institute of Thailand
SiO ₂	Silicon dioxide
Al ₂ O ₃	Aluminium oxide
Fe ₂ O ₃	Ferric oxide
SO ₃	Sulfur trioxide
CaO	Calcium oxide
OPC	Ordinary Portland Cement
C ₃ S	Tricalcium silicate
C ₂ S	Dicalcium silicate
C ₃ A	Tricalcium aluminate
C ₄ AF	Tetracalcium alumino ferrite
Ca(OH) ₂	Calcium hydroxide
CTE	Thermal expansion coefficient
TSC	Tensile strain capacity
FEM	Finite Element Method
FA	Fly ash
LP	Limestone powder

CHAPTER 1

INTRODUCTION

1.1 Background of the study

Nowadays, concrete is the most worldwide material to be used for construction because of many advantages such as it is easily available in most of place, it can be designed for the determined strength, it can be cast in the designed shape, and it is considered to save cost more than other materials. Moreover, there are many types of concrete to be selected in construction depending on the purposes or the specifications of structures. Mass concrete is one of the kinds of concrete to be used and needed for large structure construction such as dam and foundation. According to ACI116 R, the definition of mass concrete has been given as “concrete of any volume with the large dimensions that large enough to be required the measurement to be taken as dealing with the generation of heat from the cementitious reaction and sometimes including fly ash reaction and minimize crack due to the volume changing. Any concrete element having at least dimension greater than 3 ft or 0.9 m.” By its large dimension and generation of heat from the reaction, this will lead to the thermal crack due to heat generation. Thermal crack is caused by heat generation which comes from cement (hydration reaction), and pozzolan materials (pozzolanic reaction). The thermal crack occurs due to the temperature gradient between heat generation from the reaction at the center or inside of the mass concrete which can lead to higher temperature at the center and heat-releasing at the surface of the mass concrete which leads to lower temperature, causing restrained strain which can lead the concrete to crack. Therefore, in the massive concrete structure, the temperature rise due to heat of hydration can induce the temperature gradient which can cause thermal cracking after that, especially at the early age of concrete. There are many available methods to prevent thermal cracking. An example is to control the cementitious material content which is the way of selecting cement type and amount for the concrete mix proportion. This method can reduce the generation of heat of concrete due to the cement hydration reaction. Next method is the long-term strength design. Typically, mass concrete structures do not have to carry the total service load

at 28 days but at much longer ages, so the compressive strength can be designed to be achieved at a longer age such as 56, 91 or even 365 days, in case of dams for the latter. Pre-cooling is also one of the methods to prevent thermal cracking. It can be done by reducing ingredients' temperature to achieve a lower concrete temperature. Post cooling can be done by the installation of pipes for cooling the mass concrete to limit the temperature rise. Insulated curing can also be done by surface insulation which can reduce thermal differentials or temperature gradient between the inside and near surface of the structure. Proper construction plans to prevent structure from having excessive temperature rise can also be considered such as layer casting, etc. Thermal cracks can reduce the load capacity, performance, and service life of structures. Cracking which does follow the criteria of design can also cause problems. Effective methods have to be implemented to prevent severe thermal cracking in mass concrete structures. However, those mentioned efforts also have limitations and are not compatible with all kinds of cementitious materials, so reliable simulating prediction models for temperature rise prediction of mass concrete are needed to minimize and control thermal cracking. As a result, this study aims to minimize and control thermal cracking by investigating two main compositions of concrete which are binder system and aggregate system. The investigation and development of temperature rise model prediction are done by model development according to ternary binder consideration. The investigation on the effects of maximum size of coarse aggregate or G_{max} on mass concrete is done for the aggregate system. Moreover, the experimental tests which are slump and slump loss, compressive strength, tensile strain capacity, and semi-adiabatic temperature rise are also conducted to investigate the effects of fly ash with limestone powder and the effects of G_{max} equals 1 inch on ternary binder concrete.

1.2 Statement of problems and significance of the study

High-rise buildings have been constructed in many areas, so mass concrete casting is required. In the past, the specification for compressive strength and slump are only two factors of design criteria which are not enough for construction of mass concrete due to thermal cracking which can lead to many problems, as stated in the background of study. The methods to prevent cracking are very important. The

existing methods and materials for construction have limitations and not be able to cope with the requirements. New reliable methods to prevent thermal cracking are very important. Therefore, the selected materials are used to study and investigate to control and minimize thermal cracking potential. Models for predicting semi-adiabatic temperature rise of multi-binder concrete are also beneficial for effective analysis and design of mass concrete. Finally, the performance enhancement of thermal crack resisting concrete will be achieved.

1.3 Objectives of the study

There are 5 main objectives of the study which focus on the development of temperature rise prediction model and experimental investigation on both binder system and aggregate system in fly ash - limestone powder concrete. These objectives are as follows:

- To conduct experiments on free water content, and specific heat for ternary binder pastes.
- To develop the prediction models for specific heat, free water content, degree of hydration reaction, total heat generation, and thermal conductivity for ternary binder concrete.
- To develop the adiabatic temperature rise prediction model for ternary binder concrete
- To develop thermal crack resisting mass concrete by using fly ash and limestone powder as cement replacing material for binder system and investigating the effects of maximum size of coarse aggregate (G_{max}) for aggregate system.
- To investigate slump and slump loss, compressive strength, tensile strain capacity, and semi-adiabatic temperature rise in concrete for the tested binder and aggregate systems.

1.4 Scopes of the study

This study focuses on the investigation on binder system, cement with fly ash and limestone powder as cement replacing materials, and aggregate system which is the study of using coarse aggregate maximum size or G_{max} equal to 1 inch instead of $\frac{3}{4}$ inch to enhance the performance of thermal resisting mass concrete. The adiabatic

temperature rise model prediction for ternary binder, cement containing fly ash and limestone powder, is proposed by considering the effects of fly ash and limestone powder in mixtures and verify them with laboratory test results of semi-adiabatic temperature rise. The ternary binder assumption was done to propose simulation of models on the degree of hydration, degree of pozzolanic reaction, free water content, cumulative heat generation, total heat generation, specific heat, and thermal conductivity to be able to cope with the ternary binder concrete. Then, the proposed prediction model on free water content and specific heat are verified with the experimental results on cement pastes. Moreover, the experiments on slump and slump loss, compressive strength, tensile strain capacity (TSC), and semi-adiabatic temperature rise are conducted for the ternary binder concrete. Finally, the verification of adiabatic temperature rise prediction model for ternary binder is done by the comparison between computed results and experimental results of semi-adiabatic temperature rise specimens. Figure 1.1 shows the illustration chart of the investigation on ternary binder system.

In case of the aggregate system, the concrete with G_{max} equal to 1 inch is investigated and the different behavior compared with the normal concrete with G_{max} equals to $\frac{3}{4}$ inch is studied. Void contents of aggregates having both maximum sizes of coarse aggregate are measured for mix proportion design. The binder mix proportions for studying effects of aggregate system are selected by considering the capacity of maximum temperature rise reduction from the study in the previous part. Finally, the ternary binder concrete together with two different G_{max} are investigated on the slump and slump loss, compressive strength, tensile strain capacity, and semi-adiabatic temperature rise as shown in Figure 1.2.

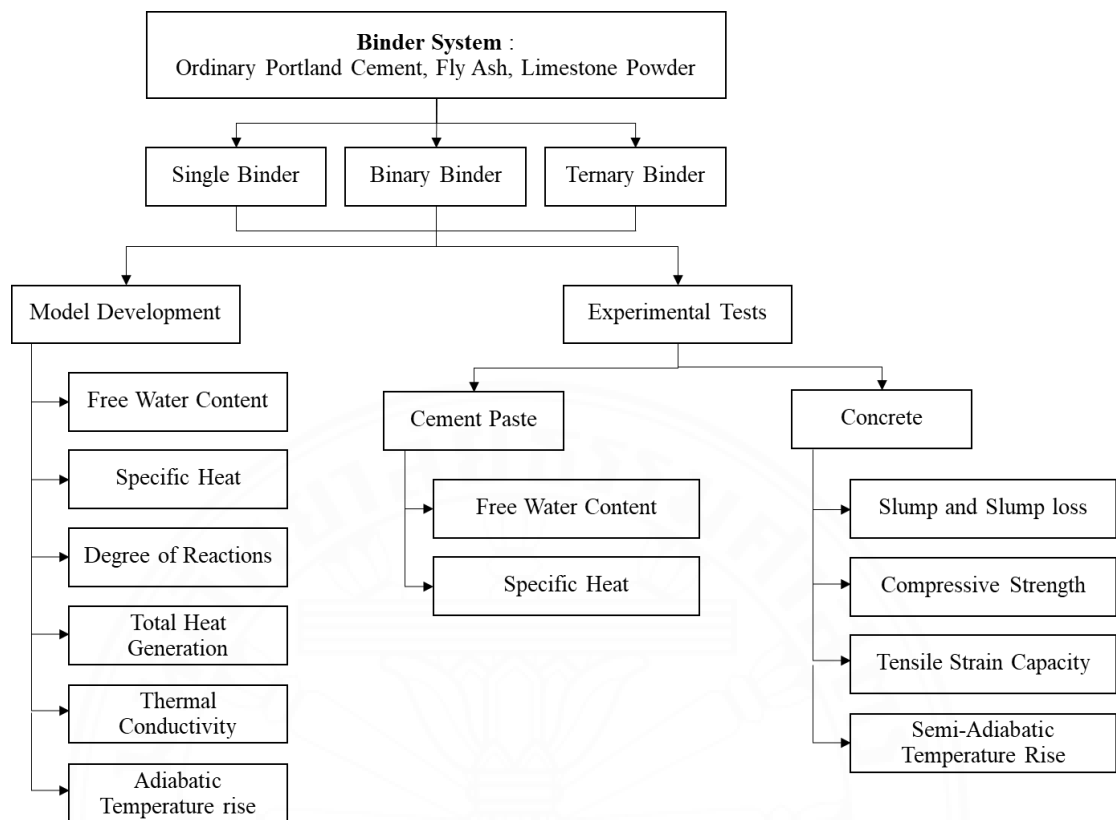


Figure 1.1 Illustration chart for the investigation on ternary binder system

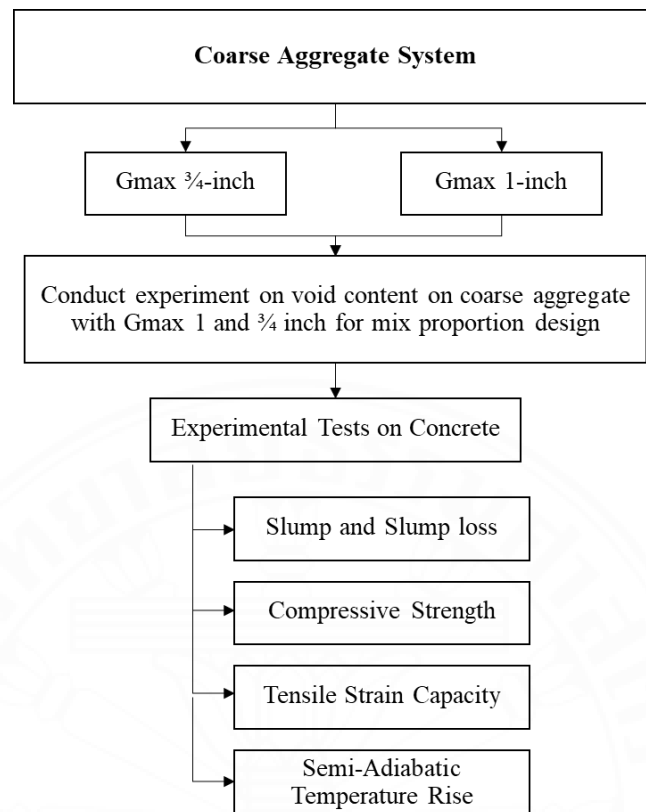


Figure 1.2 Illustration chart for the investigation on different maximum sizes of coarse aggregate system

CHAPTER 2

REVIEW OF LITERATURES

2.1 Introduction

As the demand for concrete is higher, the production of cement which is the base composite of concrete is also higher. Carbon dioxide emission which is generated by cement manufacturing and leads to an environmental problem is also higher. Moreover, high cement content used in mix proportions creates a disadvantage for mass concrete. Mass concrete is also one kind of concrete structure used to carry a heavy load to suit the design criteria. ACI also stated that any concrete that has a thickness more than 3 feet can be considered as mass concrete. When the concrete structure is getting larger or thicker, the significance of thermal cracking caused by heat generated from the reaction of cement with water have to be considered since this kind of problem can deteriorate the performance of concrete in a way that the structure may not be able to perform according to the design criteria and may accelerate the deterioration of the structure. According to this, the enhancement of mass concrete performance is necessary to achieve the good performance of the structure so the supplementary cementitious materials (SCMs) are introduced to be the optional materials and many studies have stated the improvement of concrete properties by using SCMs. The use of SCMs is one method to minimize thermal cracking because it can reduce cement content in concrete which can reduce the heat generation from cement. The history of using cementitious materials (SCMs) to reduce heat generation in mass concrete is long. The aggregate content is also considered because it affects the concrete mix proportion which can be one parameter considered to reduce temperature rise in mass concrete. According to the literature reviews, there are many studies related to the use of SCMs and Gmax cooperating with the compressive strength and tensile strain capacity development, but only few reports related to the temperature rise in mass concrete.

2.2 Introduction to fly ash and limestone powder

2.2.1 Fly ash

Fly ash is the result of the combustion process of pulverized coal and has been one of the widely used supplementary cementitious materials in construction. As the pozzolan material which contains siliceous and aluminous component, fly ash, in the presence of water, react with calcium hydroxide to produce cementitious substances. Many researches have provided and confirmed the information on the proper use of fly ash. Chemical compositions of fly ash can be different based on sources and coal combustion process and these are used to categorize fly ash type by standards.

There are 3 types of fly ash as stated by ASTM C618 and divided by physical properties and chemical compositions. Class N is natural pozzolan material as pumicites. Class C is high lime content fly ash, and providing pozzolanic properties and some cementitious properties. Class F is common fly ash and also has only pozzolanic properties.

Based on EIT 1014-40, there are 3 classes of fly ash classified by chemical and physical properties. Class 1 is appropriate for extremely high-quality concrete. Class 2 which is divided into two types, type a and type b, is suitable for conventional concrete. For type a, it has low Cao content, meanwhile, type b has high CaO content. Class 3 has lower quality than the first two classes and is allowed for low-quality concrete.

2.2.2 Limestone powder

Limestone powder (LP) is the outcome of finely ground limestone, the sedimentary rock which mainly containing calcium carbonate. It has been studied and recommended to use LP to partially replace cement in mix design for several years. By consideration to be an inert material, limestone powder is normally used as a filler among cement particles, contributing particle packing effects and also be the physical accelerator of the hydration process at the early age of the reaction.

2.3 Properties of fly ash concrete

Sheikh (2001) investigated the temperature rise of mass concrete by using fly ash as cement replacing material. The study found that the concrete with the binder

content 400 kg/m^3 incorporating fly ash at 30% and 50% reduce the adiabatic temperature rise up to 30% but at the 15% replacement level, the final temperature of fly ash concrete is still higher than the normal mixture as shown in Figure 2.1.

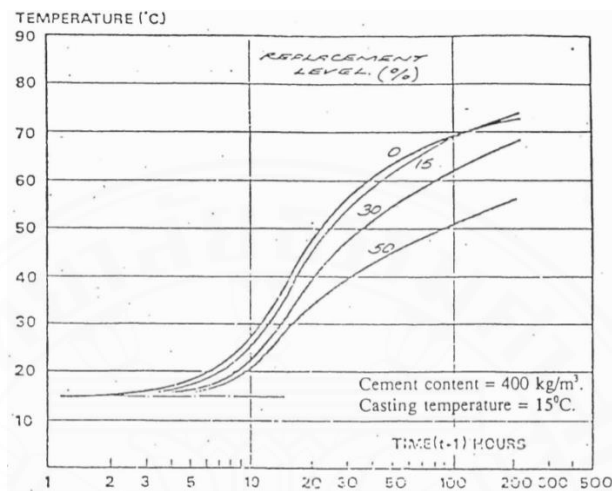


Figure 2.1 Temperature rise of concrete comparing between non-fly ash replacement, and fly ash replacement at 0.15, 0.3, and 0.5 (Sheikh, 2001)

Sheikh (2001) conducted the temperature rise tests for the binary binder. The fly ash replacing level at 20% and 40% were. The results for both replacement levels show the lower temperature rise compared to neat cement mixture and delayed the peak temperature rise of the concrete specimens as shown in Figure 2.2.

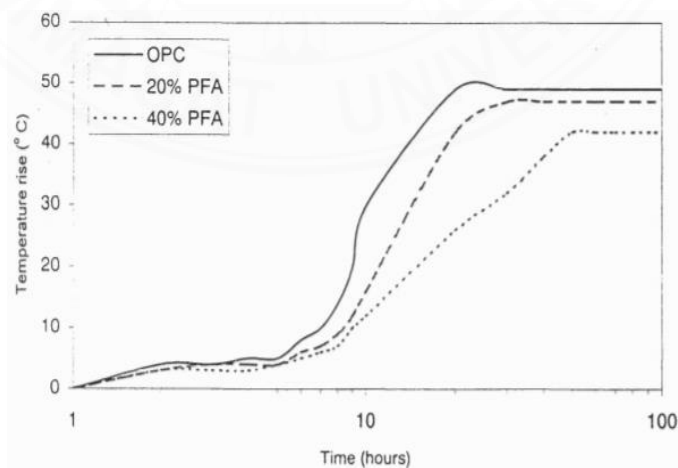


Figure 2.2 Temperature rise result of concrete comparing between control specimen, and fly ash replacing at 0.2 and 0.4 (Sheikh, 2001)

Oh, Lee and Wang (2010) revealed experimental results that the replacement of fly ash gives a lower temperature rise than that of the control mixture. Moreover, the proposed model agrees well with the various combination mixtures of experimental results.

Ng et al. (2011) conducted the semi-adiabatic temperature rise and programmed the adiabatic temperature rise for fly ash concrete. The mix proportions were designed by varying water to binder ratio and percent replacing cement by fly ash at 20% and 40%. The results for adiabatic temperature rise show that at any water to binder ratios, fly ash concrete gives the lower temperature rise than conventional concrete and continue decreasing at higher percent replacement as shown in Figure 2.3.

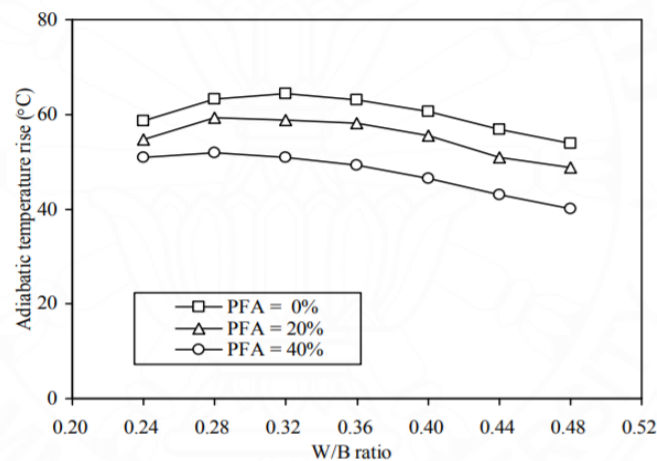


Figure 2.3 Adiabatic temperature rise result of concrete comparing between control specimen and fly ash replacing at 0.2 and 0.4 (Ng et al., 2011)

D. Wang, Tian, and C. Wang (2011) report about the characteristic and comparison of adiabatic temperature rise. They stated that the lower temperature rise can be found with the fly ash presence in the mixture and the maximum temperature rise rate decreases together with fly ash replacement level increase as shown in Figure 2.4.

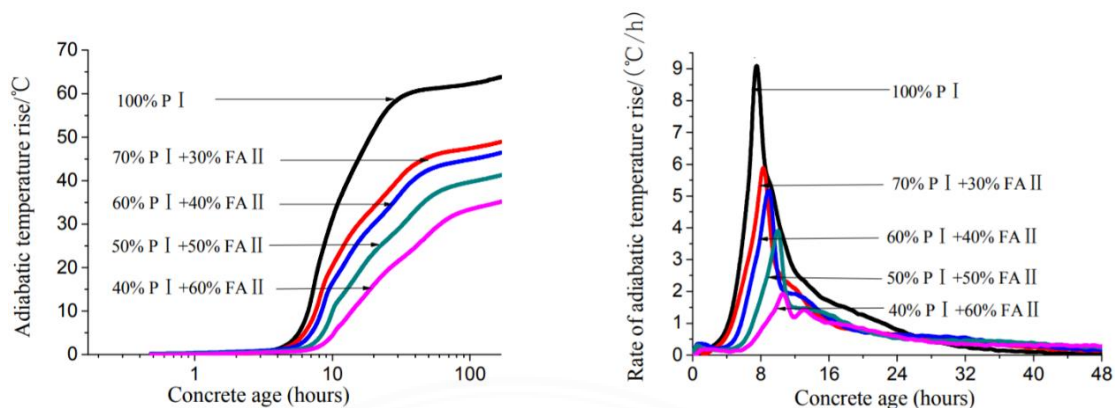


Figure 2.4 Adiabatic temperature rise result and rate of adiabatic temperature rise of concrete comparing between normal concrete and fly ash replacing at 0.4, 0.5, 0.6, and 0.7 (Wang et al., 2011)

Lagundžija and Thiam (2017) found that fly ash concrete gives a lower temperature rise and higher reduction percentage than those with control mixture and the introducing of large aggregates, cold aggregate, and ice used in concrete can also reduce the temperature rise.

Tongaroonsri and Tangtermsirikul (2008) studied tensile strain capacity for the binary binder of fly ash as cement replacing material and reported that fly ash concrete shows the lower cracking strain up to 28 days compared to control mixture due to the slower strength development of paste.

2.4 Properties of limestone powder concrete

Limestone powder is one of the common uses as cement replacing material and also well known as the inert filler for concrete. Many properties of limestone powder concrete are studied and reported to be improved.

Kishi and Dorjparamyn (1999) implemented the physical effect of limestone powder to improve the computerized model to predict hydration heat and temperature rise in concrete by introducing the total surface area effect which is calculated based on weight fraction and Blaine's fineness. The proposed model is well met with the experimental results and more accuracy can be found when compared to the previous model without consideration of these physical properties of limestone powder. The verification was conducted by the temperature measurement at the center of the concrete specimen with various percent replacement of limestone powder and

compared with those with the model as shown in Figures 2.5 and 2.6. The result also presented that limestone powder concrete leads to lower temperature rise than conventional concrete and the increase of limestone powder replacement level reduces the maximum temperature rise of concrete.

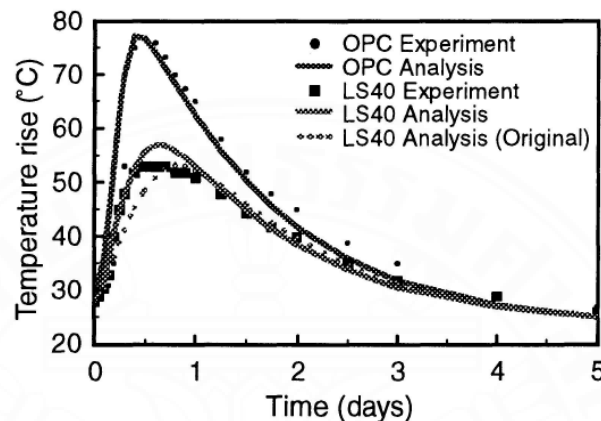


Figure 2.5 Comparison between temperature rise result with model verification of normal concrete and concrete with limestone powder (Kishi et al., 1999)

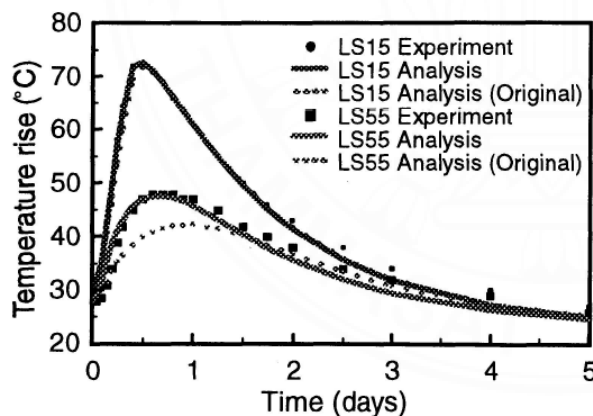


Figure 2.6a Temperature rise of LS15 and LS55

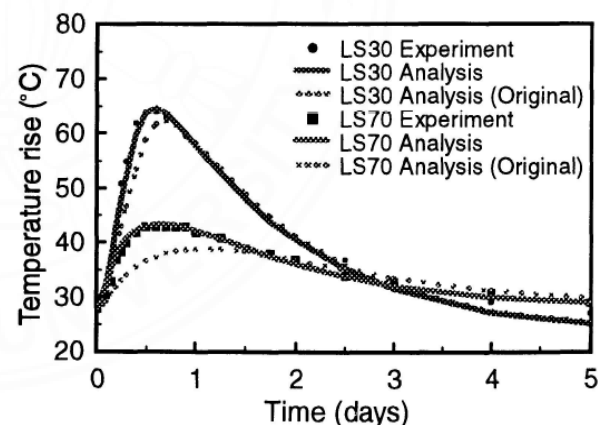


Figure 2.6b Temperature rise of LS30 and LS70

Figure 2.6 Comparison between temperature rise result with model verification of limestone powder concrete (Kishi et al., 1999)

Wei et. al (2017) conducted an adiabatic temperature rise of various replacement levels of limestone powder concrete and recorded the maximum temperature rise of each mixture. They found that positive effects can be noticed due to the lower maximum temperature rise in concrete but did not show the important

effect in terms of temperature rise rate reducing and retardation of the highest temperature period of occurrence.

The disadvantages of using limestone powder in concrete be found on compressive strength according to Wei et. al (2017). The various replacement levels of limestone powder are used in the investigating. The result showed that as the replacement levels increase, the compressive strength is decreased for all testing ages.

Tongaroonsri and Tangtermsirikul (2008) studied tensile strain capacity for the binary binder of limestone powder as cement replacing material and reported that the replacement of limestone powder lowers the cracking strain when compared to cement-only concrete for both fine and coarse limestone powder due to the non-available cementitious material property of the limestone powder.

2.5 Properties of ternary binder concrete by using the combination of fly ash and limestone powder

There have been many studies about using fly ash incorporating limestone powder as cement replacing materials because of each benefit to improve various properties of concrete such as hydration reaction, slump and slump loss, compressive strength, and temperature rise of concrete. Fly ash can improve the workability of concrete and provide a longer working time according to the setting time retardation but the contrasting phenomena can be found for limestone powder which reduces flowability and working time. However, the improvement of early compressive strength can be found from concrete with limestone powder. Fly ash concrete is well known for the outstanding later age compressive strength. The compensation can be found by using these two cementitious materials together. The addition of both fly ash and limestone powder can generate lower heat development which will lead to lower temperature rise due to the cement hydration because of the lower cement content in mix proportions so this also shows the good trend to use with mass concrete which normally has to face with the thermal stress problem. As a result, many researches have been focusing on the combination of using both fly ash and limestone powder to enhance the properties of concrete.

Thongsanitgarn et al. (2014) reported that the heat evolution profile of fly ash pastes was the same as that of no-fly ash paste and also delay the heat development as

shown in Figures 2.7 and 2.8. However, the mixture containing this limestone powder at 15% result in the same total heat release as plain cement, and when using the 20 μm of limestone powder in ternary binder paste, the heat evolution and total heat evolved up to 48 hours was found to be lower than that of plain OPC as shown in Figure 2.9.

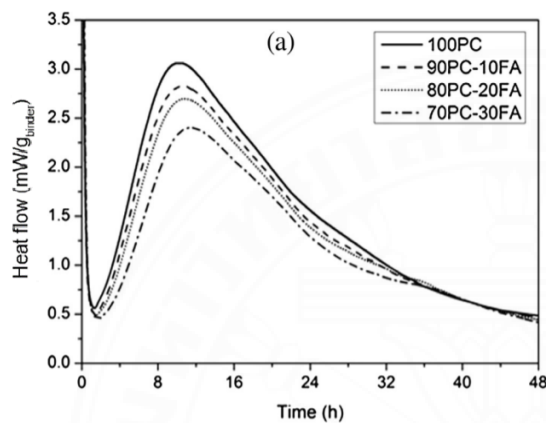


Figure 2.7a Heat flow

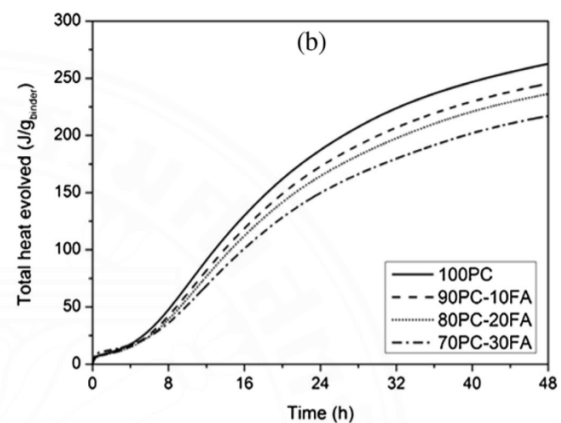


Figure 2.7b Total heat evolved

Figure 2.7 Experimental result for heat development of pastes with fly ash replacing at 0, 0.1, 0.2, and 0.3 (Thongsanitgarn et al., 2014)

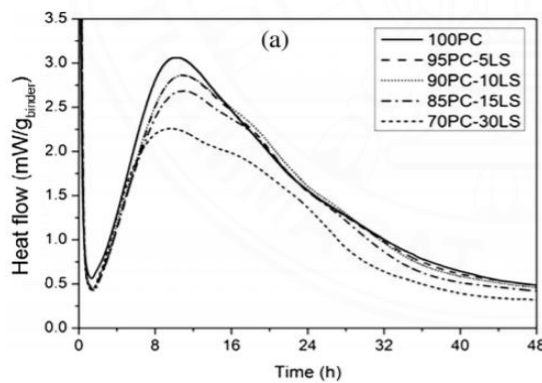


Figure 2.8a Heat flow

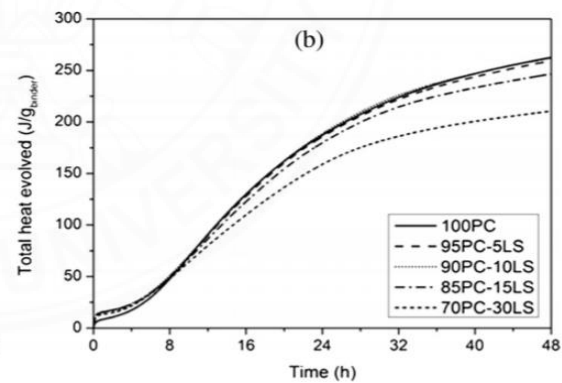


Figure 2.8b Total heat evolved

Figure 2.8 Experimental result for heat development of pastes with limestone powder replacing at 0, 0.05, 0.1, 0.15, and 0.3 (Thongsanitgarn et al., 2014)

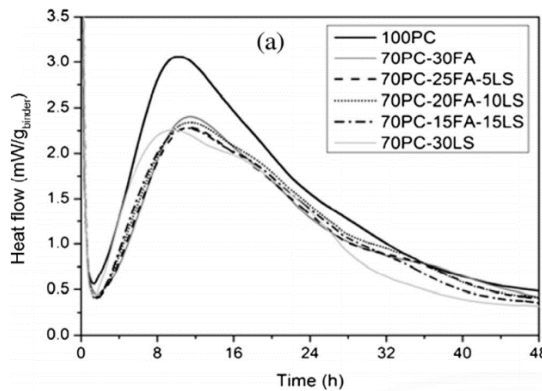


Figure 2.9a Heat flow

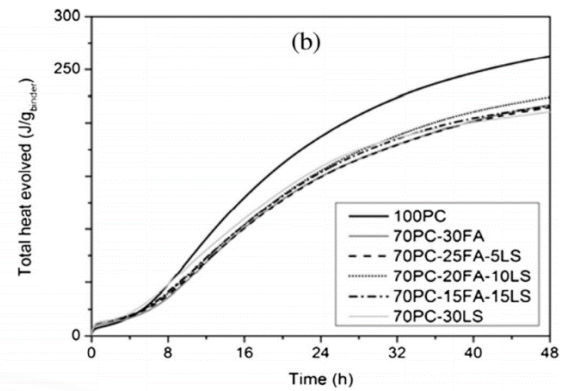


Figure 2.9 Total heat evolved

Figure 2.9 Experimental results of heat development of ternary binder system of pastes with fly ash and limestone powder as cement replacing materials (Thongsanitgarn et al., 2014)

Si-Jun K. et. al (2015) investigated the adiabatic temperature rise of mass concrete and found the ternary binder, denoted as W32.5B540, gave the lowest maximum temperature compared to other mixtures which are control specimen, binary mixture, and different water to binder mixtures as shown in Figure 2.10.

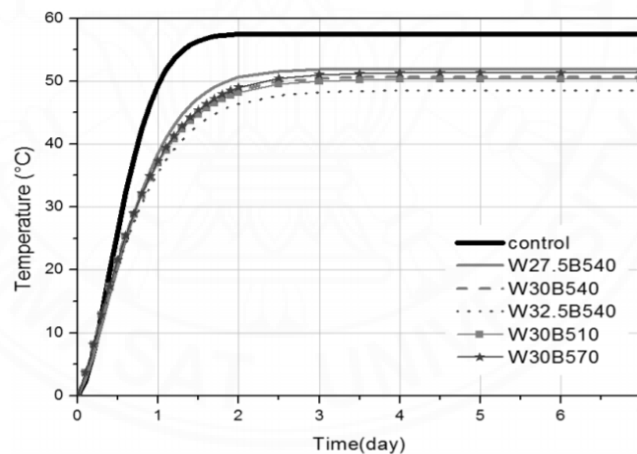


Figure 2.10 Adiabatic temperature rise results of normal concrete comparing with fly ash and/or limestone powder used as cement replacing material (Si-Jun et al., 2015)

Wang (2018) stated that in the case of binary blends, 5% limestone powder replacement, the degree of hydration reaction is a bit higher than that of the control mixture. Cement with 35% replacement of fly ash hydration degree is also higher than that of cement only and the proposed hydration model is shown in Figures 2.11.

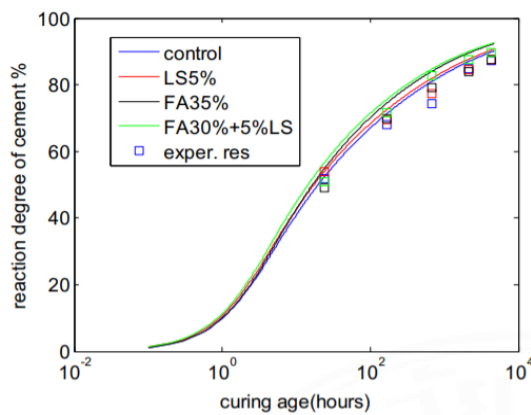


Figure 2.11a Reaction degree of cement of control specimen, LS5, FA35, and FA30 with LS5

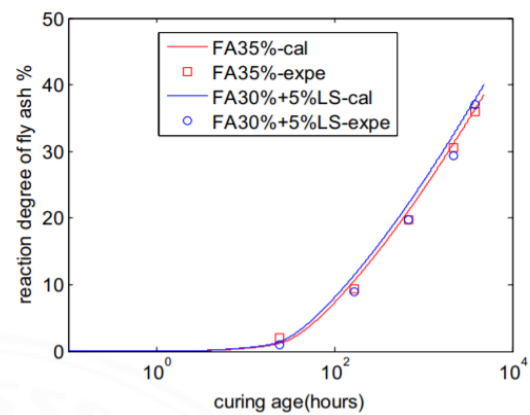


Figure 2.11b Reaction degree of fly ash of FA35 and FA30 with LS5

Figure 2.11 Comparison of experimental result and calculation model result reaction degree of cement and reaction degree of fly ash (Wang, 2018)

Slump and slump loss are some of the important parameters for fresh concrete to indicate workability and working time. There are many reports about the benefits of fly ash to improve slump and slump loss that can provide higher flowability and delay setting time. However, limestone powder can cause a lower slump and accelerate the setting time in concrete due to the irregular shape and filler effect on cement hydration, respectively.

According to Kaewmanee and Tangtermsirikul (2014) studied about ternary biner of fly ash with limestone powder and conducted initial slump and slump loss evaluations. Fly ash concrete gives the highest initial slump. However, cement incorporating with limestone powder concrete improved the initial slump compared to that of the control mixture. Moreover, a ternary binder that contains limestone powder shows a slightly faster slump loss than no limestone powder containing mixture and the rate of slump loss tends to be faster when higher limestone powder was replaced as shown in Figure 2.12

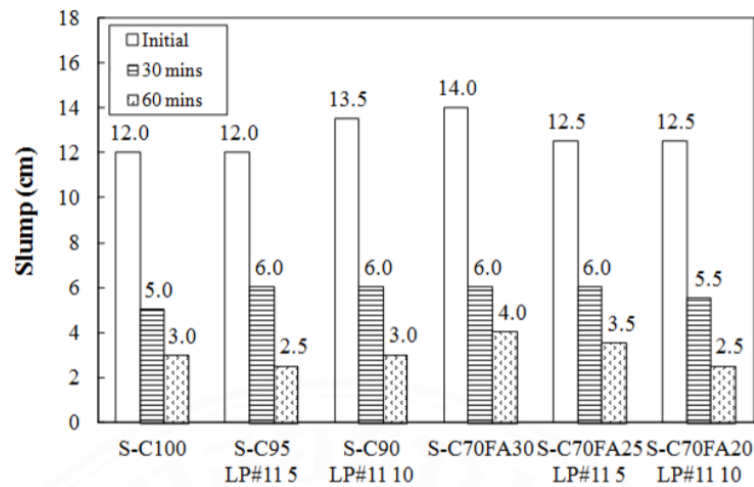


Figure 2.12 Comparison of slump and slump loss of normal concrete, LP5, LP10, FA30, FA25LP5, and FA20LP10 (Kaewmanee et al., 2014)

Normally, there are many studies and reports about the positive inclusion of fly ash in a mixture that can produce higher late compressive strength than OPC only mixture and the higher early compressive strength can be found when using limestone powder in the mix design. So the introduction of fly ash with limestone powder as cement replacing materials can be used to compensate and improve the compressive strength and also have an advantage on mass concrete construction because of lower cement content that can generate heat due to the cement hydration process.

De Weerd et al. (2010) found that limestone powder concrete gave higher compressive strength than other mixtures and the higher limestone powder replacing fly ash would give higher compressive strength as shown in Figures 2.13 and 2.14.

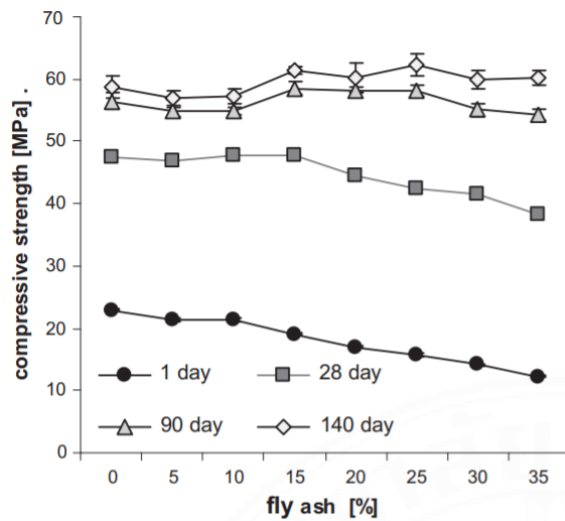


Figure 2.13a Fly ash replacing levels 0, 5, 10, 15, 20, 25, 30, and 35

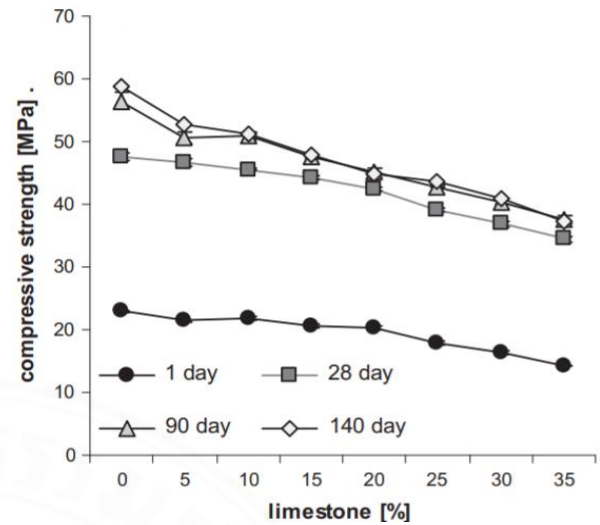


Figure 2.13b Limestone powder replacing levels 0, 5, 10, 15, 20, 25, 30, and 35

Figure 2.13 Compressive strength of concrete at the testing age of 1, 28, 90, 140 days (De Weerd et al., 2010)

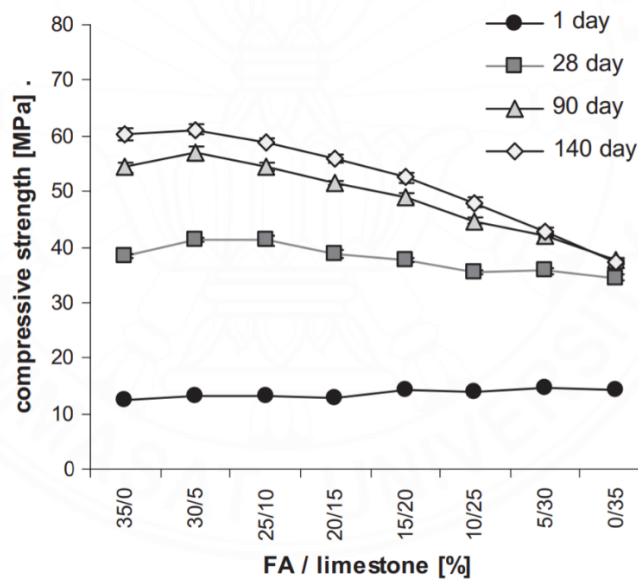


Figure 2.14 Compressive strength of concrete at various fly ash and limestone powder replacing levels at the testing age of 1, 28, 90, 140 days (De Weerd et al., 2010)

Mounanga et. al (2010) also studied the multi-partial replacement of fly ash and/or limestone powder replacing cement to formulate binary and ternary binder systems and conducted the compressive strength on mortars. Fly ash has shown no benefit on for early-age strength development with compressive strength decreasing with increasing fly ash replacement level, Limestone powder mortars show lower

compressive strength than no limestone powder mixture with replacement levels of 25% and 50%. as shown in Figure 2.15.

Mortars	Compressive strength R_c (MPa)		
	2 days	7 days	28 days
100% C	22.7	31.1	44.0
75% C + 25% BFS	19.0	28.4	44.4
50% C + 50% BFS	11.4	22.0	39.6
75% C + 25% FA	18.7	27.0	33.2
50% C + 50% FA	9.5	15.6	21.9
75% C + 25% LF	19.7	34.7	37.3
50% C + 50% LF	11.3	15.8	17.0
75% C + 17% BFS + 8% LF	17.9	29.2	44.0
75% C + 8% BFS + 17% LF	19.5	31.2	39.3
50% C + 33% BFS + 17% LF	14.0	26.8	39.1
50% C + 17% BFS + 33% LF	11.0	20.6	32.9
75% C + 17% FA + 8% LF	17.1	31.6	35.0
75% C + 8% FA + 17% LF	19.2	31.8	36.6
50% C + 33% FA + 17% LF	9.9	17.6	21.1
50% C + 17% FA + 33% LF	9.5	18.9	22.1

Figure 2.15 Compressive strength of mortars at various fly ash and limestone powder replacement levels (Mounanga et al., 2010)

2.6 Properties of the maximum size of aggregate (G_{max}) equal to 1-inch concrete

Nowadays, G_{max} widely use in Thailand construction is 3/4 inch and G_{max} is one of the parameters that affect binder content which is used to design mix proportion. Less water and cement are used for large size aggregate mixtures than for small size aggregate mixtures, due to decreasing in void ratio and total aggregates surface area at the same total aggregate mass so by shifting G_{max} to be larger, 1 inch, binder content can be reduced. For mass concrete, lower binder content will have an advantage in term of temperature rise.

Lagundžija and Thiam (2017) studied and determined the methods to reduce maximum temperature rise in concrete and the larger aggregate size is also one of the conducted methods. The assumption is when larger aggregate is used, the less cement is required so the heat generation due to hydration reaction will be lower. The temperature rise history was recorded and show the lower temperature rise by using

larger aggregates than the reference mix proportion, cement only with the normal size aggregate, the maximum temperature rise and temperature gradient was reduced as shown in Figure 2.16 and 2.17. However, there are some limitations to use large aggregate because of onsite concrete casting, i.e. pumpability and reinforcement spacing which can lead to inhomogeneous concrete structure.

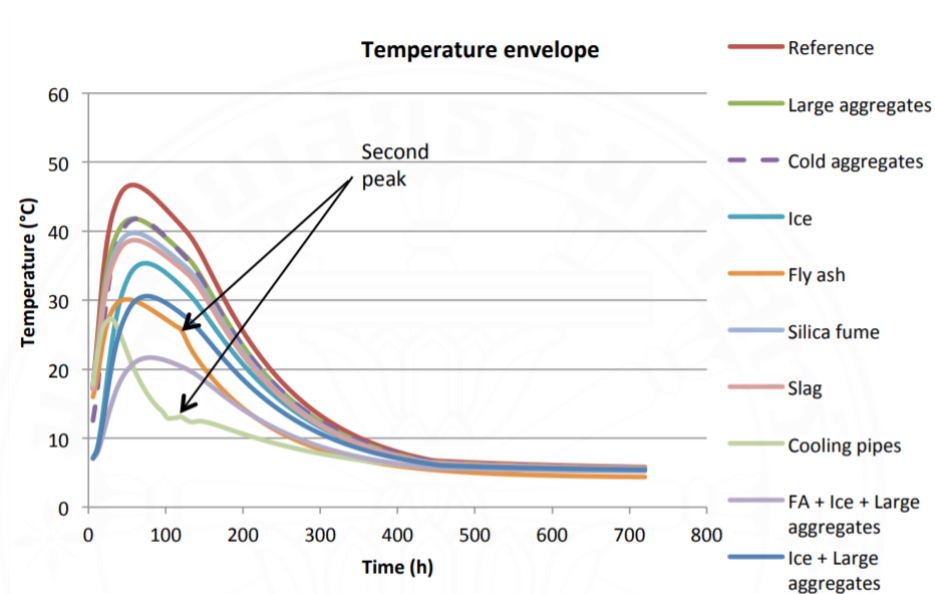


Figure 2.16 Temperature development of concrete (Lagundžija et al., 2017)

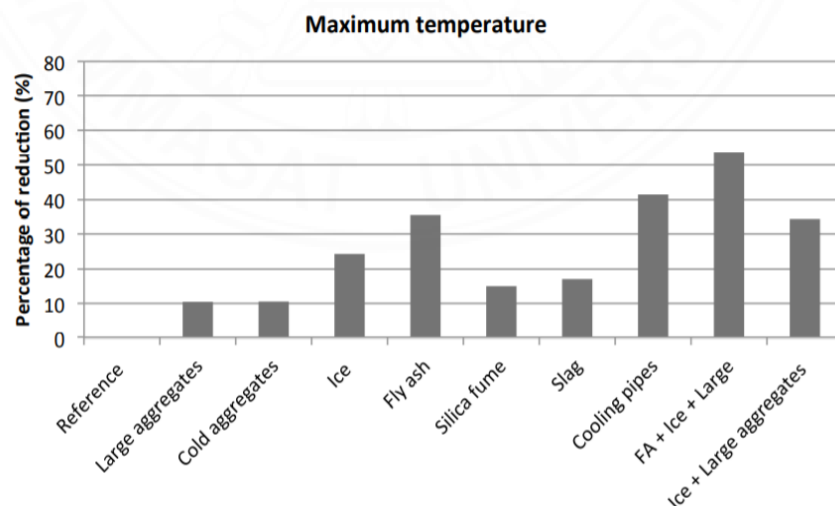


Figure 2.17 Maximum temperature development of concrete (Lagundžija et al., 2017)

As earlier mentioned, the increase of maximum coarse aggregate size requires less water and cement when compared to the smaller size. As a result, the increase in G_{max} will affect the slump of concrete due to the changed binder content.

According to Vilane and Sabelo (2016), the slump was measured and it was found that increasing G_{max} with all other factors remaining constant produced higher workability (slump) due to the lower surface area to be wetted.

Meddah, Zitouni, and Belâabes (2009) found that the improvement of compressive strength of concrete was also affected by G_{max} and believed to be after 7 days as shown in Figure 2.18. This is because of the higher compactness of the granular skeleton that led to low void content in concrete and results in higher compressive strength. With the lower void content due to the higher compactness of the concrete skeleton, high concrete compressive strength is expected.

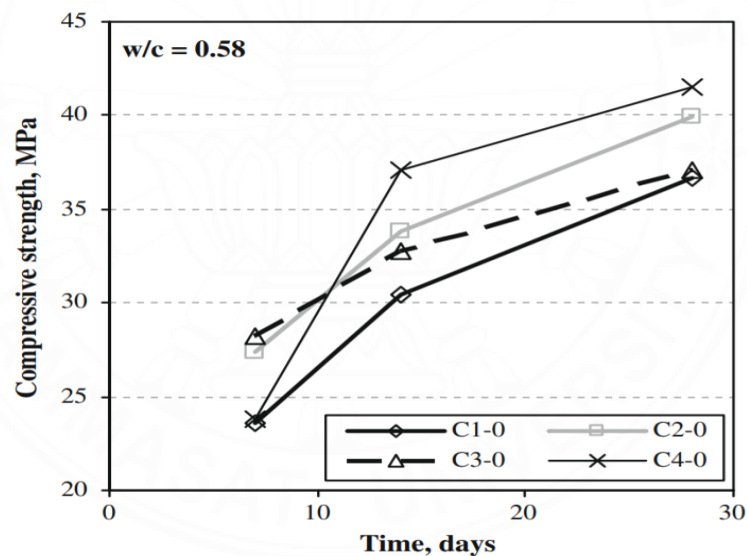


Figure 2.18 Compressive strength of concrete at 7, 14, and 28 days (Meddah et al., 2009)

According to the study of Vilane and Sabelo (2016), three different sizes of coarse aggregate were used and investigated on compressive strength. The smaller maximum size of coarse aggregate contains a larger wetted area. This area can leave the pores when the wetted area is dried out from the curing process and the micro-cracks can be started.

Ekwulo and Eme (2017) found that the larger coarse aggregate used in mix design exhibited the higher compressive as shown in Figure 2.19. However, they also stated that to use non-uniform gradation of coarse aggregate in mix proportion gave the lowest compressive strength compared to other specimens, denoted as B.

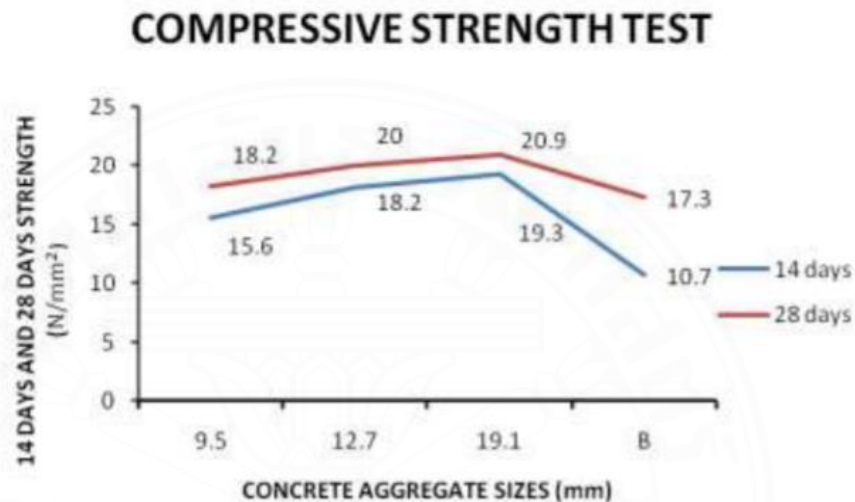


Figure 2.19 Compressive strength of concrete for the coarse aggregate maximum size of 9.5, 12.7, and 19.1 and non-uniform gradation. (Ekwulo et al., 2017)

2.7 Parameters affecting heat generation, and thermal properties of concrete

The development of the prediction model to control thermal cracking is also one way to minimize the problem. Heat generation and thermal properties of concrete are two of the many important parameters that needed to be considered to improve the accuracy of the model and make the model reliable. There are many reports which studied the parameters affecting heat generation and thermal properties of concrete such as the effects of binder, the maximum size of coarse aggregate, water to binder ratio, mix proportion, and properties of concrete ingredients.

Kishi and Dorjparamyn (1999) described the effects of limestone powder introduced into the mixtures as a cement replacing material. They reported that cement hydration is accelerated by limestone powder existence based on Blaine's fineness and amount of limestone powder. The acceleration can be observed as shown in Figure 2.20 which can conclude that a higher amount and Blaine's fineness of limestone powder can lead to the higher hydration heat rate.

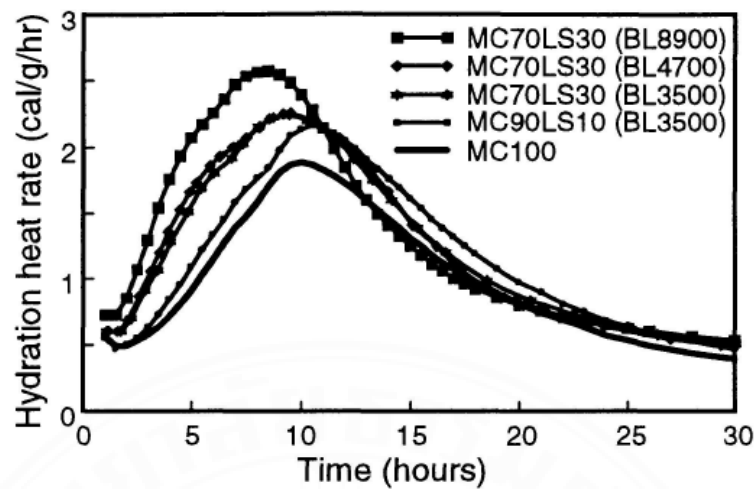


Figure 2.20 Hydration heat rate of cement pastes containing limestone powder at 0, 10, and 30 with Blaine's fineness 3500, 4700, and 8900. (Kishi et al., 1999)

De Weerd (2007) reported the calorimetry result of the control specimen, the result of the heat development of OPC containing 50% of fly ash shows considerable delaying compared to the cement only specimen. Heat development rate of binary binder, 50% fly ash replacement, is almost similar to those with the ternary binder, both 10% and 20% of micro-sized limestone powder. However, it clearly shows that the addition of nano-sized limestone powder instead of micro-sized accelerates and compensates for the retarded early hydration of binary binder, cement with fly ash only, as shown in Figure 2.21.

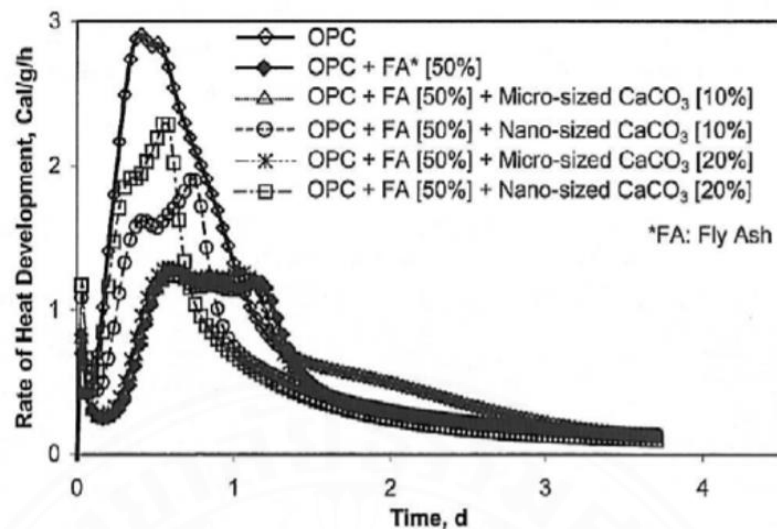


Figure 2.21 Rate of heat development for cement pastes containing fly ash and limestone powder. (De Weerd 2007)

De Weerd et. al (2010) investigated the isothermal calorimetry and compare the result of the rate of heat hydration for systems with cement only. Cement with fly ash system delays the maximum rate of hydration heat. By replacing limestone powder in both cement only and cement with fly ash mixtures, the hydration heat production is almost the same, with the slightly higher acceleration, during the first 24 hours when compared to mixture containing no limestone powder in the mixture as shown in Figure 2.22.

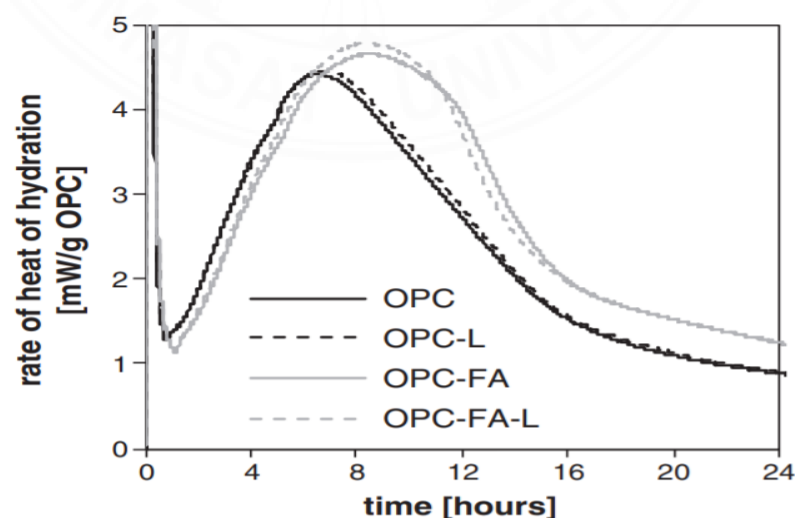


Figure 2.22 Rate of the heat of hydration for cement pastes with fly ash and limestone powder. (De Weerd 2010)

The hydration heat flow curves show that to replace cement with fly ash results in lower maximum hydration heat flow due to the lower heat release from the pozzolanic reaction and cement hydration as shown in Figure 2.23a. The same phenomena can be found for binary binder. The use of cement together with the limestone powder causes a reduction of maximum hydration heat flow as shown in Figure 2.23b. The combination of fly ash limestone powder as a cement replacing material has been formulated and studied. The acceleration of maximum heat flow peak can be found except the mixture that cement has been replaced by 50% by 33% fly ash and 17% limestone powder as shown in Figure 2.24.

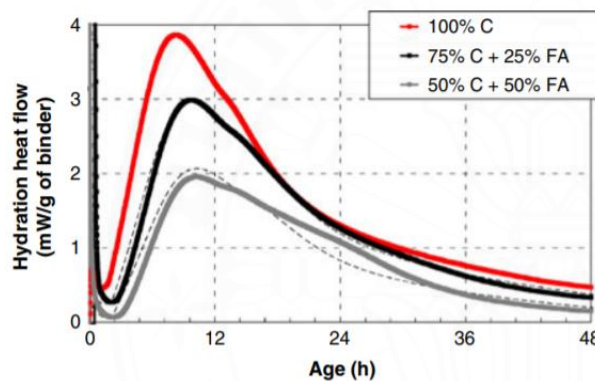


Figure 2.23a Fly ash replacing levels at 25% and 50%

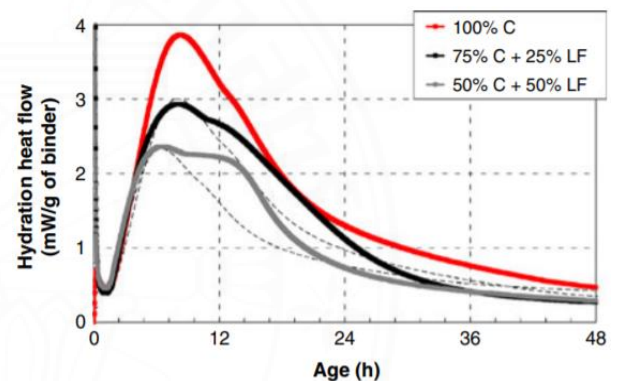


Figure 2.23b Limestone filler at replacing levels 25 % and 50%

Figure 2.23 Hydration heat flow of cement pastes containing fly ash or limestone filler (Mounanga et al., 2010)

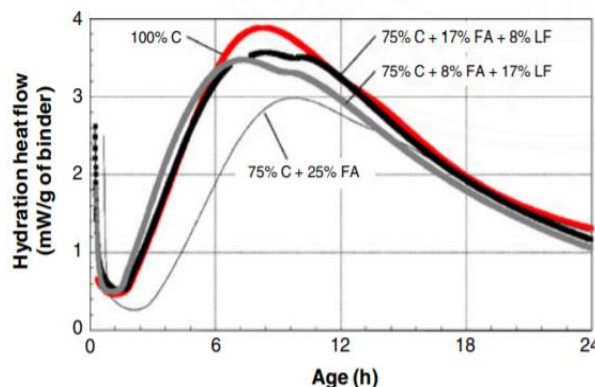


Figure 2.24a Fly ash with limestone filler replacing levels at 25% with 0%, 8% with 17%, and 17% with 8%

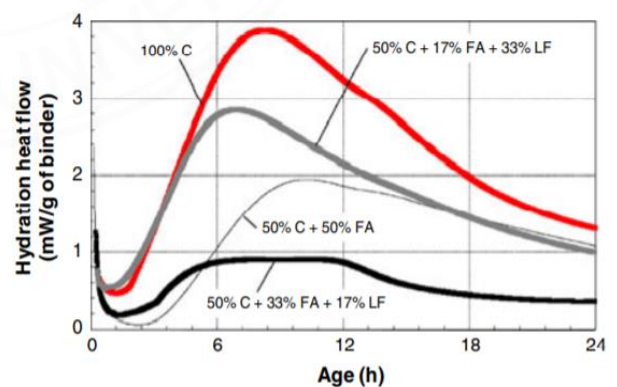


Figure 2.24b Fly ash with limestone filler replacing levels at 50% with 0%, 17% with 33%, and 33% with 17%

Figure 2.24 Hydration heat flow of cement pastes containing fly ash and limestone filler (Mounanga et al., 2010)

Pailyn, Watcharapong, and Arnon (2014) stated that the heat release of cement with 30% replacement of fly ash is retarded. In the case of ternary binder paste sample, it can accelerate heat evolution rate compared to that of binary binder system, cement-fly ash system, and tends to accelerate more with the higher limestone powder replacement level in the mixture as shown in Figure 2.25.

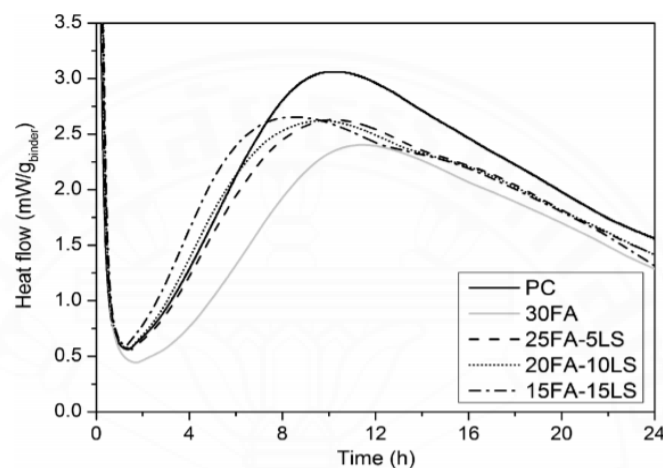


Figure 2.25 Heat flow of cement pastes containing fly ash and limestone powder (Pailyn et al., 2014)

Thermal properties are necessary factors related to the performance of concrete behavior evaluation. The specific heat and thermal conductivity will be focused on in this literature review. The addition of limestone powder or fly ash in a binary binder system to investigate these two thermal properties has been mentioned in many studies, especially for fly ash but many researches recently conclude that to evaluate these two thermal properties, it is required to formulate the time-dependent prediction model to improve the accuracy and be more realistic on behavior of real mass concrete structures.

Saengsoy (2002) and Choktaweekarn (2008) studied and developed the time-dependent specific heat prediction model and stated that moisture content plays more important role than any other ingredients in concrete. Based on their studies, the specific heat was calculated considering the ingredients of concrete with respects to time.

The water to binder ratio, fly ash replacement in a binary binder, and moisture content in concrete was studied to investigate their effects on thermal conductivity

according to Sun et. al (2016). The study shows that these three parameters strongly affected the thermal conductivity. The prepared concrete specimen which is stated as a porous material obtained the best performance and the heat conductivity coefficient of fly ash concrete was reported to be $0.112 \text{ W/(m}\cdot\text{k)}$ at 28 curing days.

Asadi et. al (2018) studied and reviewed the measurement methods and parameters affecting the thermal conductivity of concrete. There are two methods commonly determined for thermal conductivity measurement. The first one is the steady-state method, the material will be tested based on the constant heat transfer which is not time-dependent and suitable for the homogenous materials. Conversely, the transient method which is usually applied for the heterogeneous material with moisture content is the time-dependent technique and relies on temperature change. The various factors mentioned in this study are humidity, temperature, characteristics of aggregate, moisture content, and properties of binders that can affected the thermal conductivity. The thermal conductivity will increase normally with the increase of moisture content and moistened or saturated specimens give higher thermal conductivity value than those with dry condition. The declining trend of thermal conductivity can be achieved with the increasing temperature but the fluctuation of value is also reported at different temperature increments. The type and amount of aggregate are also important parameters to consider. The study revealed that increasing the volume of a coarse aggregate fraction without change for a sand ratio can increase the thermal conductivity of concrete.

However, there are some studies stated that the availability of fly ash in mixture results in higher thermal conductivity than no-fly ash due to the capability of water absorption which can carry moisture content in specimens according to Sun et. al (2016) and Shin and Kodide (2012).

The investigation on limestone powder as a cement replacing material about specific heat and thermal conductivity is still limited. Choktaweekarn (2010) studied the use of limestone powder to simulate specific heat and thermal conductivity by computerized prediction models and these thermal prosperities of limestone powder are reported to be higher than cement which is agreed with Wei et. al (2017). They simulated and analyze thermal properties of limestone power concrete by thermal

analysis and finite element method. They found that the availability of limestone powder generated higher specific heat and thermal conductivity of concrete modestly.



CHAPTER 3

THEORETICAL BACKGROUND

3.1 Adiabatic temperature rise prediction model introduction

The adiabatic temperature rise prediction model is proposed by Saengsoy (2002). The study proposed a computerized program to determine the temperature rise generated in mass concrete. The model was found to be satisfactory by conducting the temperature rise test and to confirm the accuracy of the model with the test results. The comparison was also made with the others' results. The adiabatic temperature rise model consists of four parameters which are also the model to be the factor to simulate the adiabatic temperature rise. The four parameters are the degree of hydration of cement, and degree of pozzolanic reaction of fly ash, total heat generation considering exothermic reactions of binders, free water model which is related with the consumed water by reactions and gel water, and specific heat model which is the function of summation of specific heat of all compositions of concrete with known the specific heat of each ingredient as shown in Figure 3.1.

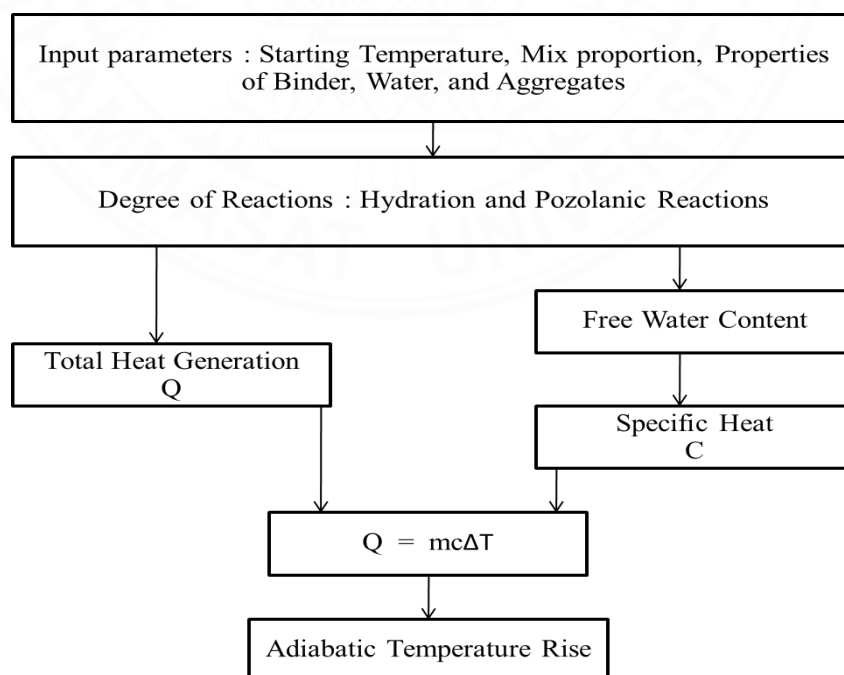


Figure 3.1 Adiabatic temperature rise chart based on Saengsoy (2002)

3.2 Semi-adiabatic temperature rise prediction model introduction

According to the proposed adiabatic temperature rise model by Saengsoy (2002), it can predict the adiabatic temperature rise accurately. However, in the real condition for mass concrete, the heat loss to the environment will exist. Therefore, the semi adiabatic temperature rise model is proposed by Choktaweekarn (2008). The proposed model aims not only for temperature rise prediction in mass concrete considering the effect of heat loss but also to evaluate cracking in mass concrete. The input parameters will include the same parameters from the adiabatic temperature rise model by Saengsoy (2002). However, the additional parameters are included as shown in Figure 3.2. According to this, the additional thermal properties which are heat conductivity and thermal expansion coefficient are considered to analyze thermal cracking in mass concrete.

The flow chart illustrates the adiabatic temperature rise prediction model steps with the verification by semi-adiabatic temperature rise test results. Firstly, the input parameters to calculate the adiabatic temperature rise are the initial temperature of concrete, mix proportion, and the properties of the materials. The degree of hydration of cement and degree of pozzolanic reaction of fly ash are calculated, then the total heat generation, free water content, and thermal properties can be provided respectively. Then, the adiabatic temperature rise of concrete can be computed and the commercial FEM program is used with total heat generation and thermal conductivity as the factors to generate the semi-adiabatic temperature rise profile of concrete and the verification will be done by the comparison between the computerized semi-adiabatic temperature rise results and the experimental results.

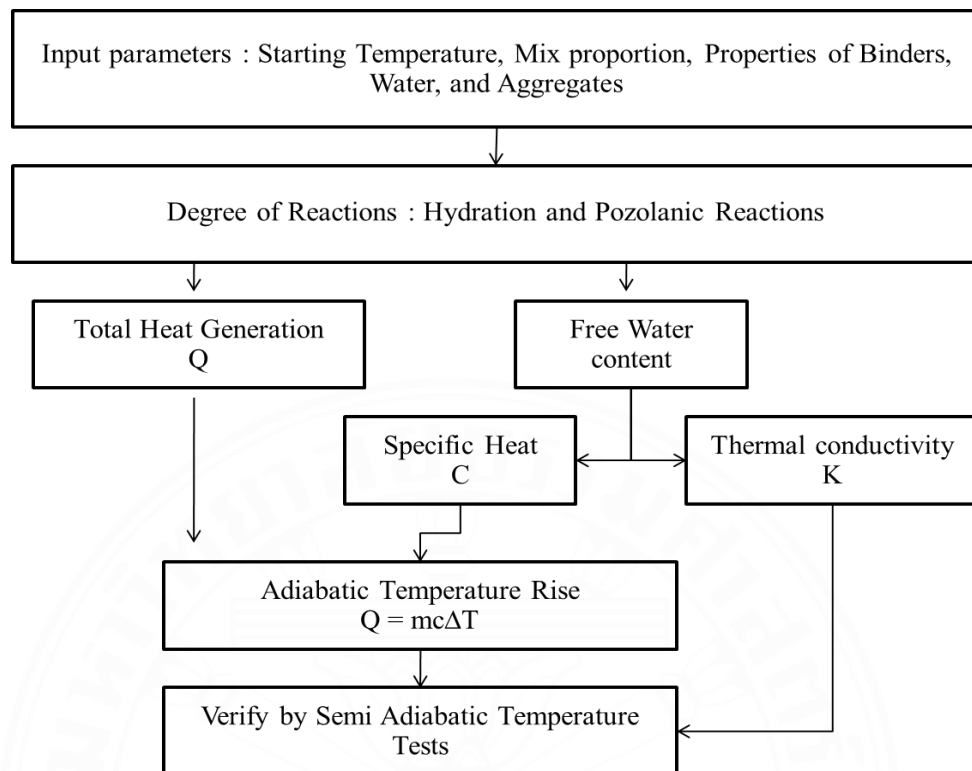


Figure 3.2 Semi-adiabatic temperature rise chart based on Choktaweekarn (2008)

CHAPTER 4

FREE WATER CONTENT

General

Water is one of the most important concrete compositions to provide the availability for reactions to take place. The main important role of water in the concrete is the hydrate activator for the cement and other binders which require water to initiate their reactions. It enables the reactions of cement and supplementary cementitious materials, water has been classified into 3 types according to the hydration reaction which is free water, chemically bound, and gel water. Firstly, evaporable water or free water is the unreacted water and free to access for the reaction. Then, as the reaction proceeds, free water reacts chemically or consumed by the cementitious materials and becomes non-evaporable or chemically bound water. Entrapped water in the reacted product is generated and called gel water. Moreover, free water content has a great effect on specific heat of the concrete

The prediction model and experimental results are still not available for the ternary binder system. As a result, the investigation on experiment and the development of prediction model of free water content for the ternary binder system are required.

4.1 Free water content prediction model

Free water content refers to the water that has not been consumed by the reactions, i.e., cement reaction and fly ash reaction. The water that has already reacted with binders is considered as water consumed by the reaction. The water that is entrapped in the product formed by binder reactions (hydration reaction and the pozzolanic reaction) is considered as gel water.

As a consequence, the free water content calculation model for ternary binder concrete can be done by the subtraction of the initial water content in the mixtures with gel water and water reacted in the reactions of binders as shown in Equation (4.1). As the reaction continues, the free water content is reduced with time. The addition of limestone powder as a cement replacing material for ternary binder system

causes gel water and water consumed by reaction to be lower because of the lower taking place of hydration from the lower cement content in the mixtures.

$$W_{fw}(t) = W_{fw0}(t) - W_{whp}(t) - W_{wgel}(t) \quad (4.1)$$

where $W_{fw}(t)$ is free water content weight at the age t in the concrete mixture (kg/m^3). W_{fw0} is the unit free water content in the mixture (kg/m^3). $W_{whp}(t)$ is the weight of water consumed by reaction at the age t (kg/m^3). $W_{wgel}(t)$ is the weight of gel water in the mixture at the age t (kg/m^3).

As the hydration reaction and the pozzolanic reaction proceed, the water will be consumed as the reaction activator for cement compounds and fly ash which are the hydration reaction and the pozzolanic reaction, respectively. The chemical reactions of cement and fly ash properties are considered to formulate the chemically bound water content or processed water. Equation (4.2) expresses how chemically bound water is calculated.

$$W_{whp}(t) = \theta_{hpc} \cdot W_{uc0} \cdot \frac{\alpha_{hy}(t)}{100} + \theta_{hpf} \cdot W_{ufa0} \cdot \frac{\alpha_{poz}(t)}{100} \quad (4.2)$$

where θ_{hpc} is the minimum ratio of water to binder to complete hydration reaction. θ_{hpf} is minimum ratio of water to complete pozzolanic reaction to obtain the maximum pozzolanic reaction. W_{uc0} and W_{ufa0} are unit weights of cement and fly ash (kg/m^3). $\alpha_{hy}(t)$ is the average degree of hydration of cement (%), $\alpha_{poz}(t)$ is the degree of pozzolanic reaction of fly ash in (%), and t is the age (days).

Gel water is considered as entrapped water from the product of hydration reaction and pozzolanic reaction. Gel water content will increase with the increase of age. Equation (4.3) expresses how to calculate gel water content.

$$W_{\text{wgel}}(t) = \left(0.0126 + \frac{0.0026}{-1.009 + \exp(0.1414 \cdot w/b)} \right) \cdot W_{\text{uc0}} \cdot \frac{\alpha_{\text{hy}}(t)}{100} + \frac{(6.177 \cdot r^2 + 9.681 \cdot r) \cdot (8.87 \cdot w/b^{8.44} + 0.1156) \cdot W_{\text{ufa0}} \cdot \frac{\alpha_{\text{pozz}}(t)}{100}}{rfa \leq 0} \quad (4.3)$$

where $W_{\text{wgel}}(t)$ is gel water weight in the mixture at the considered time t (kg/m^3 of concrete). W_{uc0} and W_{ufa0} are unit weights of cement and fly ash in the mixture (kg/m^3). rfa is fly ash replacement in total binder, w/b is the water to binder, $\alpha_{\text{hy}}(t)$ is the average degree of hydration of cement (%), $\alpha_{\text{pozz}}(t)$ is the degree of pozzolanic reaction of fly ash (%), and t is the age (days).

4.2 Materials

Ordinary Portland cement, Mae-Moh fly ash, and limestone powder were used as cement and SCMs. The cement complied with ASTM-C150 and TIS-15 and fly ash was categorized as class C and class 2b according to ASTM C618 and TIS 2135, respectively. Limestone powder which mainly contains CaCO_3 was used to replace cement. Chemical compositions and physical properties of the binders are shown in Table 4.1.

Table 4.1 Chemical compositions and physical properties of binders for free water content tests

Properties	Cement	Fly ash	Limestone powder
Chemical composition (%)			
SiO ₂	18.98	36.18	0.79
Al ₂ O ₃	5.33	20.21	0.42
Fe ₂ O ₃	3.16	13.89	0.23
CaO	65.34	18.74	54.66
MgO	1.15	2.69	0.59
SO ₃	2.65	3.74	N/A
LOI	2.54	0.25	43.21
Na ₂ O	0.16	1.14	0.02
K ₂ O	0.21	2.29	0.01
TiO ₂	0.29	0.4	0.01
P ₂ O ₅	0.06	0.21	0.03
MnO	0.05	0.12	0.01
SrO	0.03	0.09	N/A
Cr ₂ O ₃	0.01	N/A	N/A
ZnO	0.02	N/A	N/A
Propoerties			
Specific gravity (g/cm ³)	3.05	2.57	2.67
Blaine fineness (cm ² /g)	3,141	2,254	5,226

4.3 Specimen preparation

Ternary binder pastes were cast based on the cement, fly ash, and limestone powder. The water to binder ratios of the mixtures were varied at 0.25 and 0.4. In the case of binary binder, limestone powder to binder ratio was 0.1, and fly ash to binder ratios were 0.3 and 0.5. In the case of ternary binder, limestone powder was used to replace cement at a limestone powder to total binder ratio of 0.1 and the replacement levels of fly ash were kept constant at 0.3 and 0.5. All specimens were cast in the completely sealed containers to prevent water to come in and go out from the specimen and kept in the control temperature and humidity chamber until the testing age. Figures 4.1 and 4.2 show the apparatus for preparing specimens. Each paste was cast and tested for the free water content at the ages of 1, 3, 7, and 28 days. Table 4.2 illustrates the mix proportions used in this experiment. In the mix designation, “w”, “opc”, “fa”, and “lp” refer to water to binder ratio, cement content only, fly ash replacement ratio, and limestone powder replacement ratio, respectively.



Figure 4.1 Plastic molds completely sealed for the free water content specimen



Figure 4.2 Container for keeping the free water content testing molds in the control room

Table 4.2 Mix proportions for free water content tests

Mixtures	w/b	Proportions of binders		
		opc	fa	lp
w25opc	0.25	1.0	0	0
w25fa30	0.25	0.7	0.3	0
w25fa50	0.25	0.5	0.5	0
w25lp10	0.25	0.9	0	0.1
w25fa30lp10	0.25	0.6	0.3	0.1
w25fa50lp10	0.25	0.4	0.5	0.1
w40opc	0.4	1.0	0	0
w40fa30	0.4	0.7	0.3	0
w40fa50	0.4	0.5	0.5	0
w40lp10	0.4	0.9	0	0.1
w40fa30lp10	0.4	0.6	0.3	0.1
w40fa50lp10	0.4	0.4	0.5	0.1

4.4 Experimental procedures

The same technique as that applied by Choktaweeakarn (2008) was used in this study. Based on the definition, free water content of paste is the water that is evaporable when it is subjected to drying at 105 °C. At the test age, the specimens were removed from the sealed container and quickly weighed. The volume determination of specimens was conducted by specimen weight in water deducted with the specimen weight in the air, referred to the ASTM C127. Then the specimens were brought to drying oven at 105 °C for 24 hours to dry the water out from the specimens. Finally, the specimens were weighed and calculated the difference of weight of specimens between before drying and after drying. For each tested mixture at each test age, 3 specimens were tested to obtain average of free water content.

4.5 Experimental results

The tested specimens show all the same tendency that free water content reduces with the increase of time, indicating that water was consumed by binder reactions.

4.5.1 Effects of water to binder ratio

It was found from the experiment that the free water content of pastes decreases with time. Pastes with the lower water to binder ratio (0.25) have lower free water content than that with the higher water to binder ratio (0.4) according to the lower water content in the mixture and the lower initial water of the specimen. in Figure 4.3.

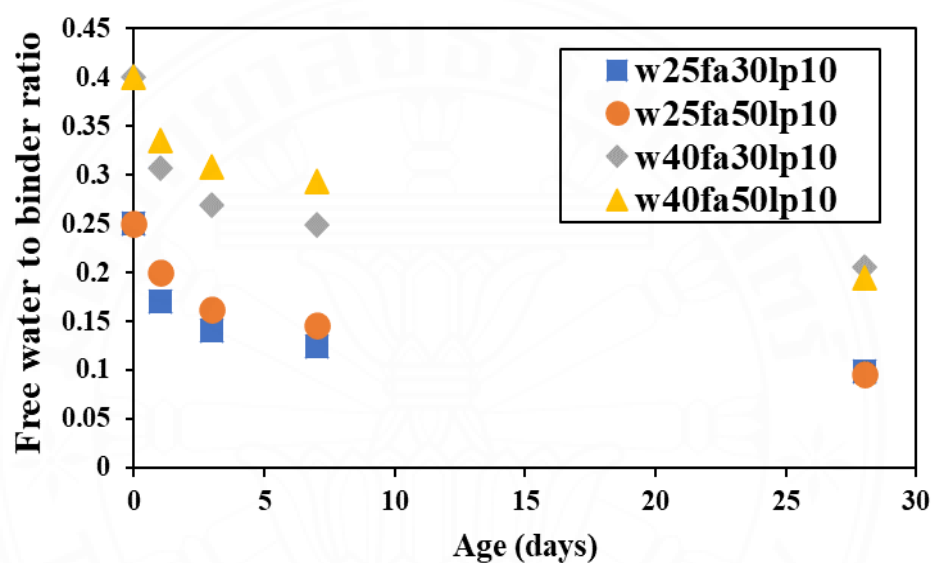


Figure 4.3 Free water content results of cement pastes with w/b ratios of 0.25 and 0.4

4.5.2 Effects of binders

The effects of limestone powder or fly ash as the cement replacing material are shown in Figures 4.4 and 4.5. The values of free water content of pastes with lower cement content are higher than those with higher cement content, especially at young age. The replacement of limestone powder or fly ash is the reason causing higher free water content in the mixture at the specimen age up to 7 days due to the non-reactive substance of limestone powder and the slow pozzolanic reaction of fly ash.

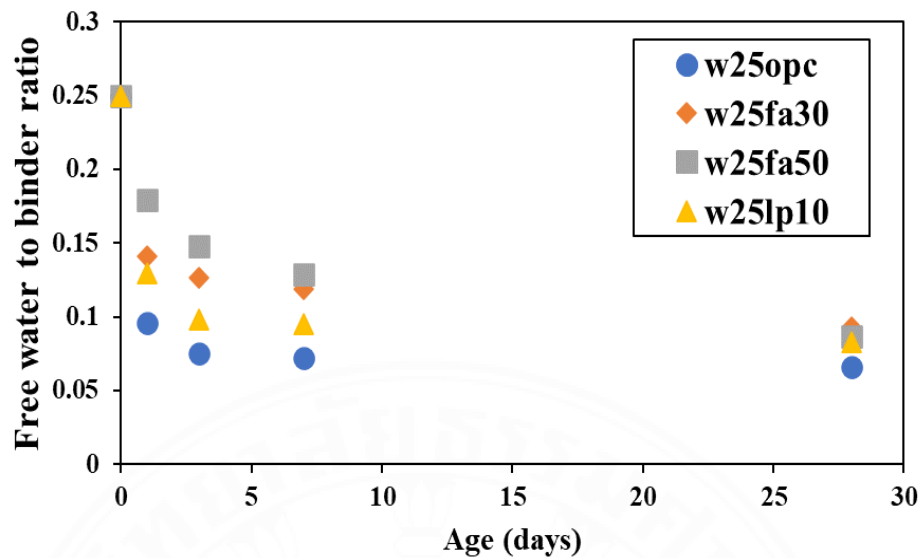


Figure 4.4 Free water content results of binary binder pastes with OPC, FA30, FA50, and LP10 at w/b ratio of 0.25

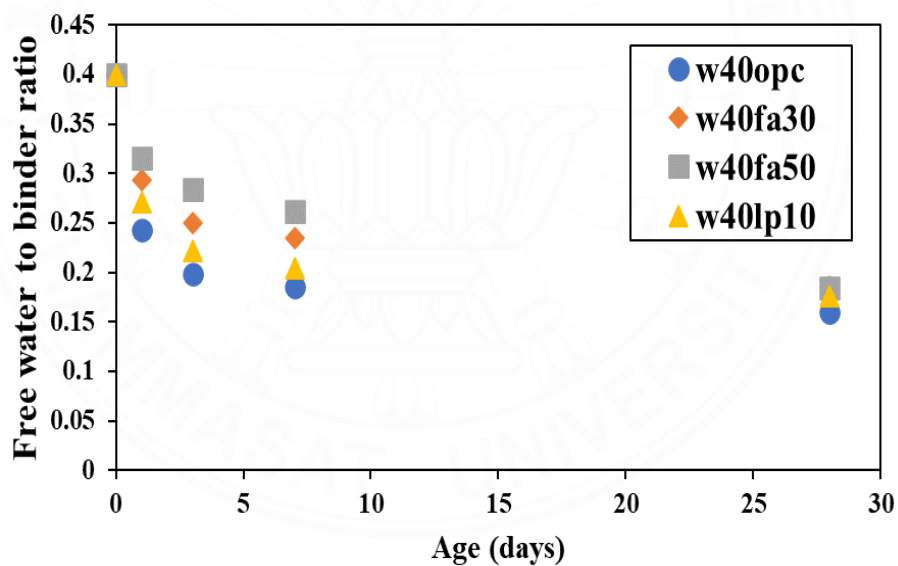


Figure 4.5 Free water content results of binary binder pastes with OPC, FA30, FA50, and LP10 at w/b ratio of 0.4

Figures 4.6 to 4.9 show the effects of limestone powder used for ternary binder as cement replacing material added into the binary binder mixture of fly ash-cement pastes. The results show that the ternary binder mixture has higher free water content than the binary binder mixture due to the lower cement content. The free

water content of fly ash pastes continues to decrease in long term when compared to no-fly ash pastes because of the continuing pozzolanic reaction.

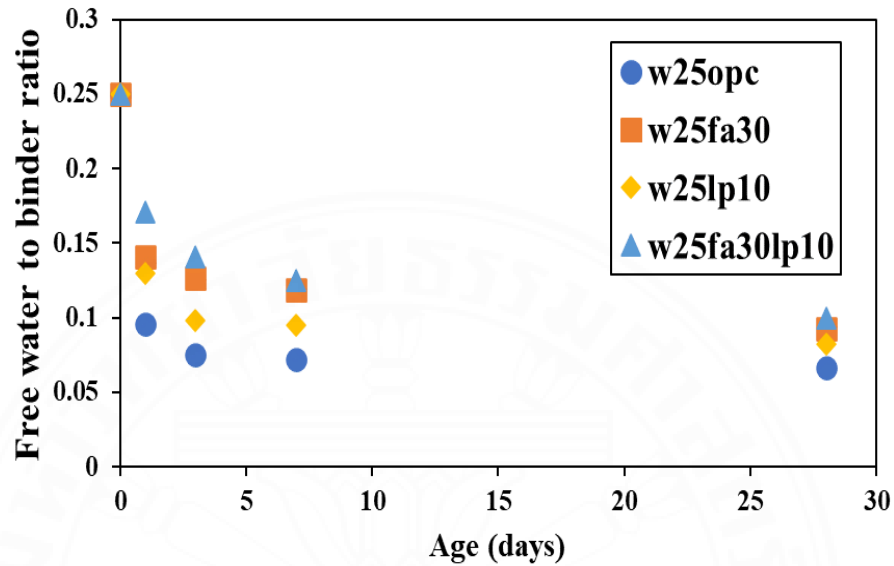


Figure 4.6 Free water content results of cement pastes with OPC, FA30, LP10, and FA30LP10 at w/b ratio of 0.25

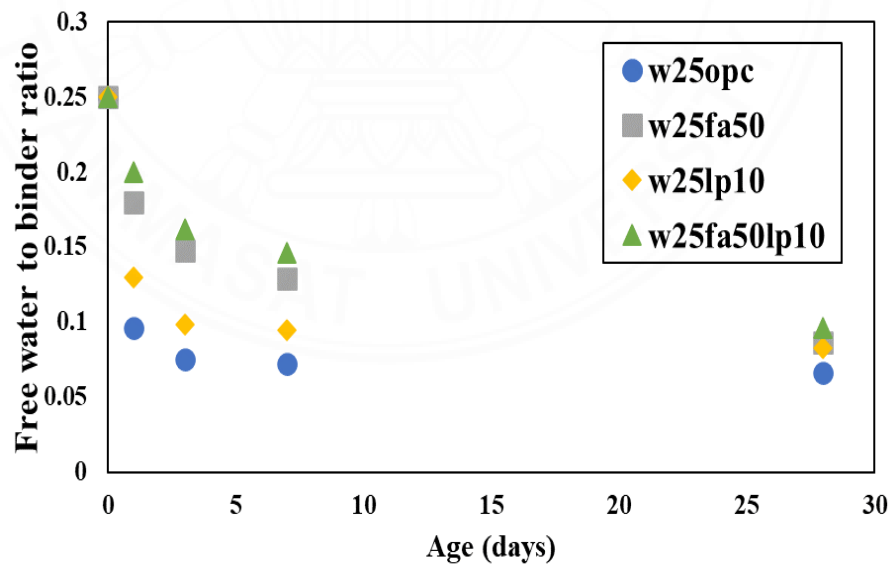


Figure 4.7 Free water content results of cement pastes with OPC, FA50, LP10, and FA50LP10 at w/b ratio of 0.25

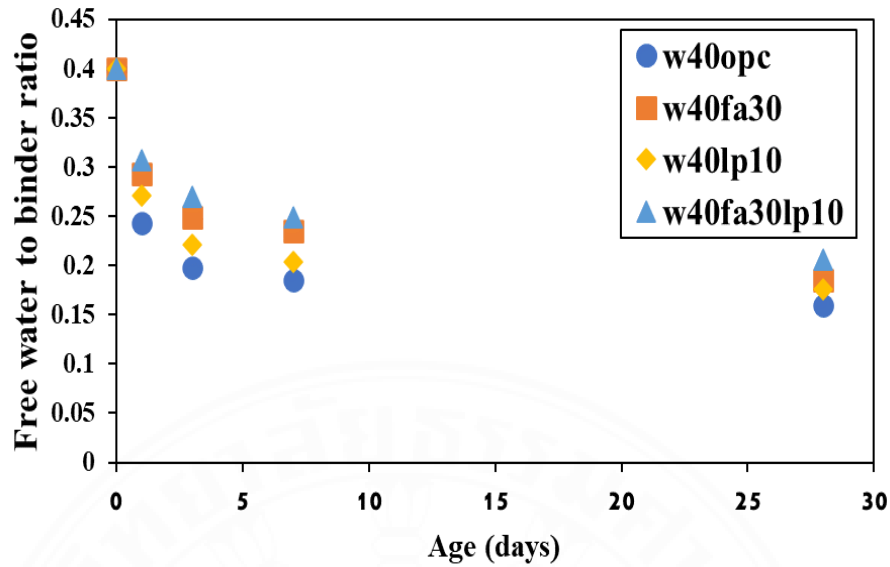


Figure 4.8 Free water content results of cement pastes with OPC, FA30, LP10, and FA30LP10 at w/b ratio of 0.4

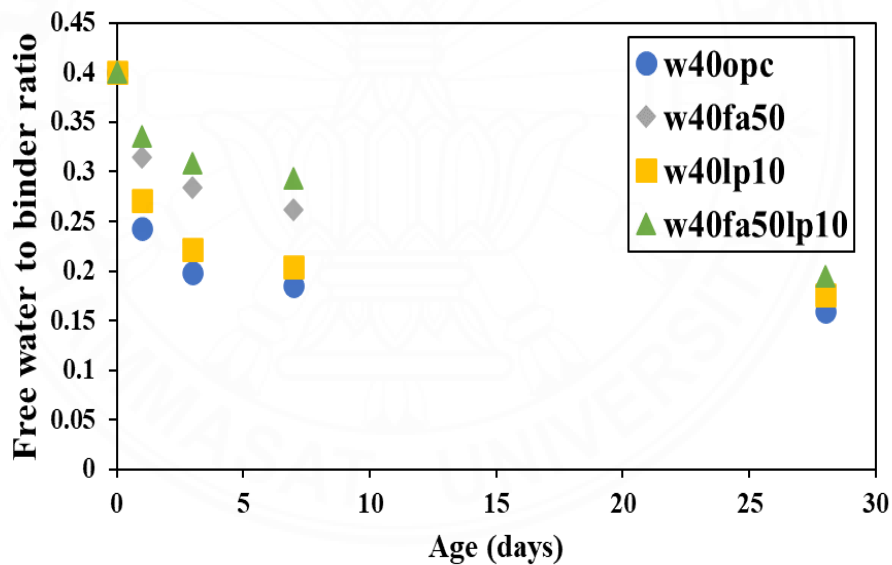


Figure 4.9 Free water content results of cement pastes with OPC, FA50, LP10, and FA50LP10 at w/b ratio of 0.4

4.6 Verification of free water content prediction model

The proposed free water content prediction model was used to generate free water content profile at various ages and for various mix proportions. Then, the verification is done. The computerized model shows the higher free water content for the lower cement content especially at young age and decrease in long term.

Moreover, the satisfactory accuracy of the prediction model with experimental results can be achieved and clearly seen from Figures 4.10 and 4.11.

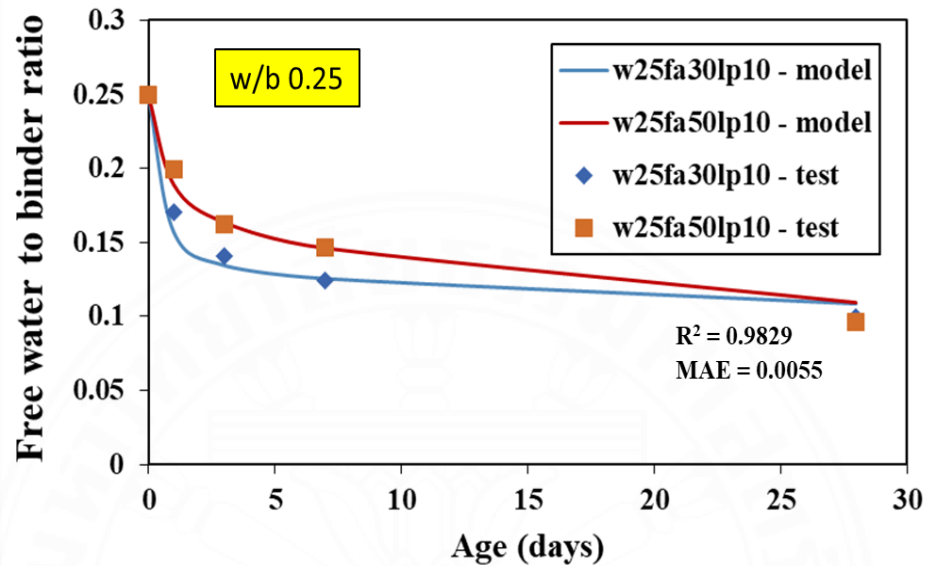


Figure 4.10 Comparison between free water content experimental result and model of pastes with FA30LP10 and FA50LP10 at w/b ratio of 0.25

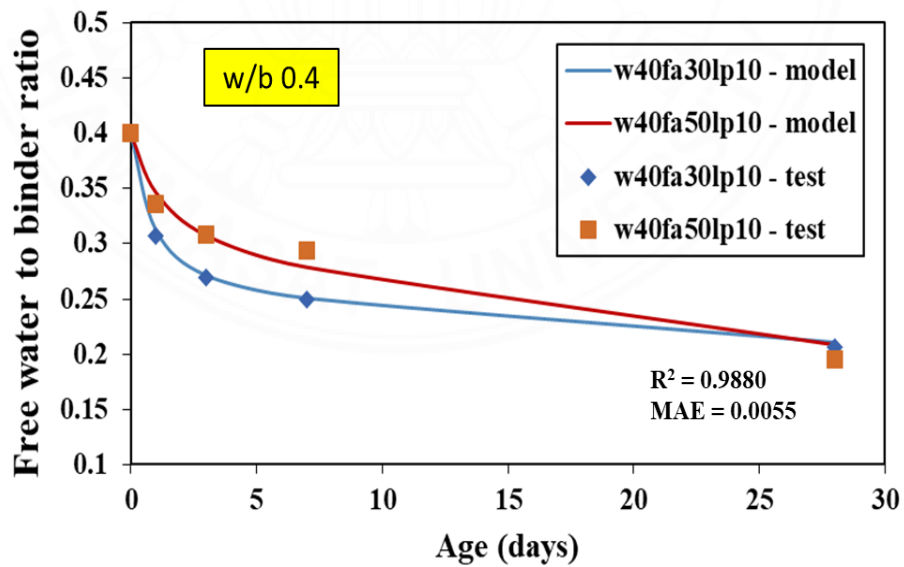


Figure 4.11 Comparison between free water content experimental result and model of pastes with FA30LP10 and FA50LP10 at w/b ratio of 0.4

CHAPTER 5

SPECIFIC HEAT

General

Improvement of temperature rise prediction model can be done by incorporating many consideration factors including thermal properties. Specific heat is one of the necessary thermal properties to improve the accuracy of the temperature rise prediction model. The definition of specific heat is heat supply needed to change the temperature of 1 unit mass of material by 1°C.

Specific heat of concrete normally has been influenced by free water which has the highest individual-specific heat value when compared to the other ingredients. Specific heat at an early age decreases rapidly due to the rapid decrease in the amount of free water content in the concrete. The previous studies for specific heat calculation models still have not covered for the ternary binder system which is the system containing cement, fly ash, and limestone powder. Also, experimental data for the ternary binder system is still limited. As a result, the investigation on experiment and prediction model development of specific heat for ternary binder system is required.

5.1 Specific heat prediction model

For the ternary binder concrete, the specific heat prediction model is still not available. As a result, this study aims to propose the specific heat prediction model for ternary binder with the verification with experimental results. The specific heat of ternary binder concrete is calculated based on weight fraction (w_i) of ingredients and their individual specific heat values (c_i) as shown in Equations 5.1 and 5.2. As the reactions proceed, free water content, weight of cement and fly ash decrease while the amount of hydrated product increases. It is obvious that the hydrated products are from cement and fly ash. Like aggregates, the weight of limestone powder is constant since it is an inert material and does not react to form any products. Specific heat is reduced with time due mainly to the decreasing of free water content. Table 5.1 provides the specific heat of each ingredient in concrete.

$$c(t) = w_g c_g + w_s c_s + w_{air} c_{air} + w_{fw}(t) c_{fw} + w_{uc}(t) c_c + w_{ufa}(t) c_{fa} + w_{hp}(t) c_{hp} + w_{lp} c_{lp} \quad (5.1)$$

$$w_{hp}(t) = 1 - w_g - w_s - w_{fw}(t) - w_{uc}(t) - w_{ufa}(t) - w_{ulp} \quad (5.2)$$

where $c(t)$ is the specific heat of concrete at the considered age t (kcal/kg °C). w_g , w_s , w_{air} , and w_{lp} are the weight ratios of coarse aggregate, fine aggregate, air, and limestone powder per unit weight of concrete, respectively. $w_{fw}(t)$, $w_{uc}(t)$, $w_{ufa}(t)$, and $w_{hp}(t)$ are the weight ratios of free water, unhydrated cement, unreacted fly ash, and hydrated products, respectively, at the considered age t . c_g , c_s , c_{air} , c_{fw} , c_c , c_{fa} , c_{hp} , and c_{lp} are the values of specific heat of coarse aggregate, fine aggregate, air, free water, cement, fly ash, hydrated product, and limestone powder, respectively (kcal/kg °C). t is the considered age (day).

Table 5.1 Specific heat values of concrete ingredients by Choktaweekarn (2008 and 2010)

Ingredients	Specific heat (kcal/kg °C)
Water	1.00
Limestone powder	0.23
Coarse aggregate	0.20
Fine aggregate	0.19
Air	0.24
Cement	0.18
Fly ash	0.17
Hydrated product	0.10 (cement with fly ash)
Hydrated product	0.15 (cement with limestone powder)

5.2 Materials

Ordinary Portland cement, Mae-Moh fly ash, and limestone powder were used as cementitious materials for the tested specimens. The cement complying with ASTM-C150 and TIS-15 was used. Fly ash which was categorized as class C and class 2b according to ASTM C618 and TIS 2135, respectively, and mainly containing high CaO content. Limestone powder was also used to replace cement for the tested specimens. Table 5.2 provides the chemical compositions and physical properties of the cementitious materials.

Table 5.2 Chemical compositions and physical properties of binders for specific heat tests

Properties	Cement	Fly ash	Limestone powder
Chemical composition (%)			
SiO ₂	18.98	36.18	0.79
Al ₂ O ₃	5.33	20.21	0.42
Fe ₂ O ₃	3.16	13.89	0.23
CaO	65.34	18.74	54.66
MgO	1.15	2.69	0.59
SO ₃	2.65	3.74	N/A
LOI	2.54	0.25	43.21
Na ₂ O	0.16	1.14	0.02
K ₂ O	0.21	2.29	0.01
TiO ₂	0.29	0.4	0.01
P ₂ O ₅	0.06	0.21	0.03
MnO	0.05	0.12	0.01
SrO	0.03	0.09	N/A
Cr ₂ O ₃	0.01	N/A	N/A
ZnO	0.02	N/A	N/A
Propoerties			
Specific gravity (g/cm ³)	3.05	2.57	2.7
Blaine fineness (cm ² /g)	3,141	2,254	5,270

Remarks: N/A is referred to not available

5.3 Specimen preparation

Twelve mix proportions were designed and tested by varying w/b ratios at 0.25 and 0.4. Limestone powder to binder ratio was used at 0.1 and fly ash to binder ratios were 0.3 and 0.5 for binary binder. For ternary binder, limestone powder was used to replace cement at a limestone powder to total binder ratio of 0.1 and the replacement levels of fly ash were kept constant at 0.3 and 0.5. The mix proportions of the tested mixtures are shown in Table 5.3. The specimens were tested for the specific heat at the ages of 1, 3, 7, and 28 days. Specimens were cast in PVC cylinder molds with a radius of 1.25 cm and a height of 5.0 cm. with the inserted thermocouple at the center and penetrated up to the mid-depth level. The container of specimen was covered by aluminum foil to prevent loss of water after the finish of casting. The molds were removed 24 hours after casting, and the specimens were firmly enclosed

with aluminum foil as shown in Figure 5.1. All specimens were kept in a control temperature and relative until reaching the test ages.



Figure 5.1a Prepared cylinder molds



Figure 5.1b Aluminum foil covered after finished casting specimen



Figure 5.1c Firmly wrapped specimen with aluminum foil



Figure 5.1d Zip-lock plastic used for covering specimens until the testing age

Figure 5.1 The specimen preparation for specific heat testing

The test was conducted on specimens without aluminum foil removal since the aluminum foil has very high heat conductivity and low specific heat and its thickness is very thin so the aluminum foil wrapping effect on the specific heat of the tested specimens can be neglected. In the mix designation, “w”, “opc”, “fa”, and “lp” refer to water to binder ratio, cement only, fly ash replacement ratio, and limestone powder replacement ratio.

Table 5.3 Mix proportions for specific heat tests

Mixtures	w/b	Proportions of binders		
		opc	fa	lp
w25opc	0.25	1.0	0	0
w25fa30	0.25	0.7	0.3	0
w25fa50	0.25	0.5	0.5	0
w25lp10	0.25	0.9	0	0.1
w25fa30lp10	0.25	0.6	0.3	0.1
w25fa50lp10	0.25	0.4	0.5	0.1
w40opc	0.4	1.0	0	0
w40fa30	0.4	0.7	0.3	0
w40fa50	0.4	0.5	0.5	0
w40lp10	0.4	0.9	0	0.1
w40fa30lp10	0.4	0.6	0.3	0.1
w40fa50lp10	0.4	0.4	0.5	0.1

5.4 Experimental procedures

The test method of specific heat of hardened specimens was based on Choktaweeakarn (2008). The heat supplying medium is hot water with the insulated container cover which is provided for submerging the specimens. The temperature increase of the specimens and the decrease of temperature of hot water were recorded by a data logger at every 5 seconds of interval, simultaneously. The apparatus for testing and the test setup are shown in Figures 5.2 to 5.4.

**Figure 5.2** The insulated container for specific heat testing



Figure 5.3 The apparatus for specific heat testing

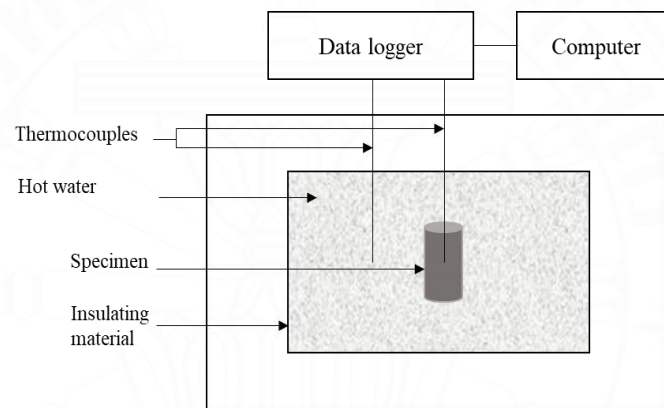


Figure 5.4 Experimental setup for specific heat testing

The specific heat of specimen can be calculated based on Equation (5.3).

$$c_{sp} = \frac{m_w c_w (\Delta T_w - \Delta T_{loss})}{m_{sp} \Delta T_{sp}} \quad (5.3)$$

where m_w and m_{sp} are the water and specimen mass, respectively (kg.) c_w and c_{sp} are the water and specimen specific heat, respectively (kcal/kg/°C). ΔT_w is the temperature decrease of the water (°C), ΔT_{sp} is the temperature increase of the specimen (°C), and ΔT_{loss} is the decrease of water temperature due to heat loss to the environment.

ΔT_{loss} can be determined by the water only system measurement. The heat loss to environment causing water temperature change and was recorded by a data

logger at an interval of 5 seconds. The average value determination was done by repeating this calibration for 3 cycles.

5.5 Experimental results

It can be clearly seen that the tested specimens show all the same tendency that specific heat decreases with time which is the same trends of the free water content.

5.5.1 Effects of water to binder ratio

According to the experiment, pastes with lower water to binder ratio (0.25) have lower specific heat than those with the higher water to binder ratio (0.4) due to lower water contents in the mixtures. Figures 5.5 exhibits the effects of water to binder ratio.

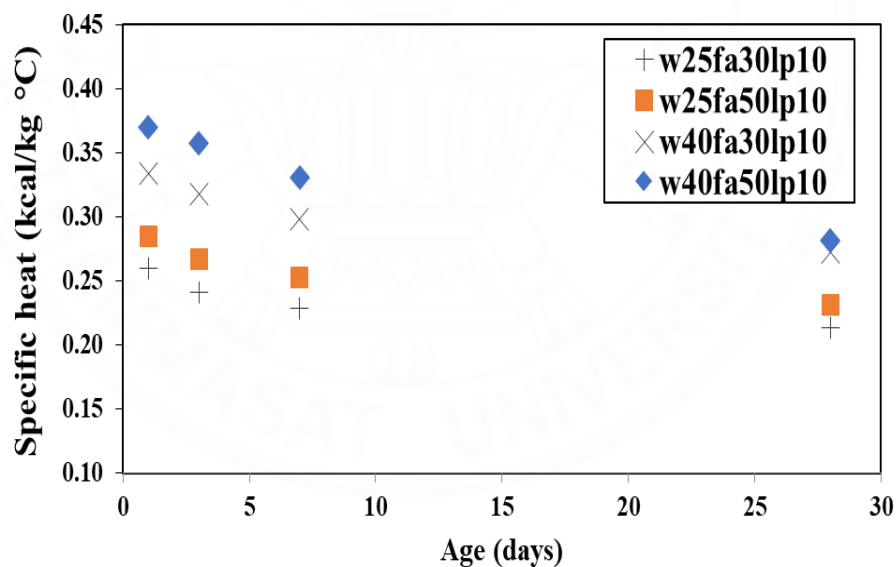


Figure 5.5 Specific heat results of cement pastes with FA30LP10 and FA50LP10 at w/b ratios of 0.25 and 0.4

5.5.2 Effects of binders

Figures 5.6 and 5.7 show the effects of limestone powder or fly ash as the cement replacing material. The values of specific heat of pastes with higher supplementary cementitious materials (SCMs) content are higher than those with lower SCMs, especially at young age by the reason of the inactive property of

limestone powder and the slow pozzolanic reaction of fly ash. As a result, the incorporation of these two SCMs causes higher free water content in mixtures.

The effects of limestone powder used for ternary binder as cement replacing material added into the binary binder mixture are shown in the Figures 5.8 to 5.11. The results show that the ternary binder mixture has higher specific heat than the binary binder mixture due to the lower cement content. Specific heat of pastes with fly ash continues to decrease in later age correlated to pastes with no-fly ash because of continuing pozzolanic reaction. The higher specific heat of ternary binder system pastes than those with binary binder and control specimen at young age reveals that using fly ash incorporating with limestone powder as cement replacing materials is useful for temperature reduction of mass concrete.

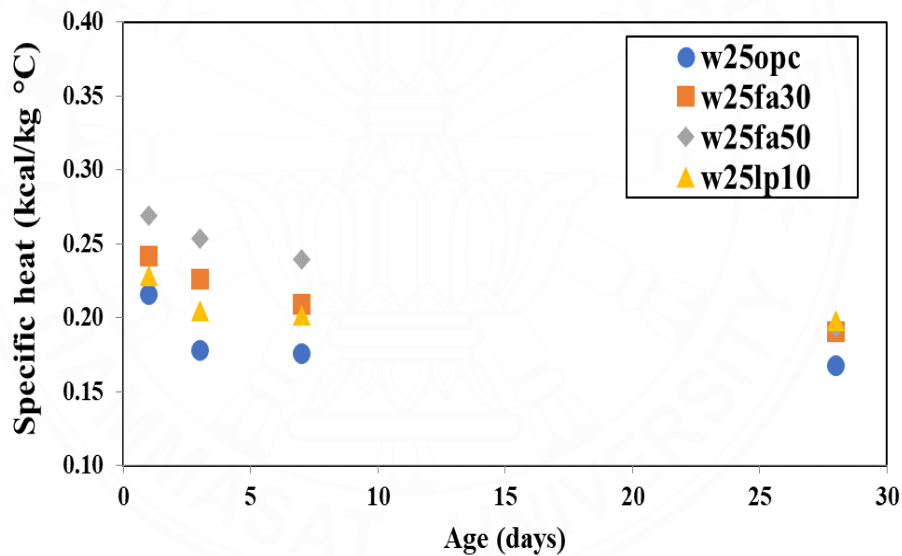


Figure 5.6 Specific heat results of cement pastes with OPC, FA30, FA50, and LP10 at w/b ratio of 0.25

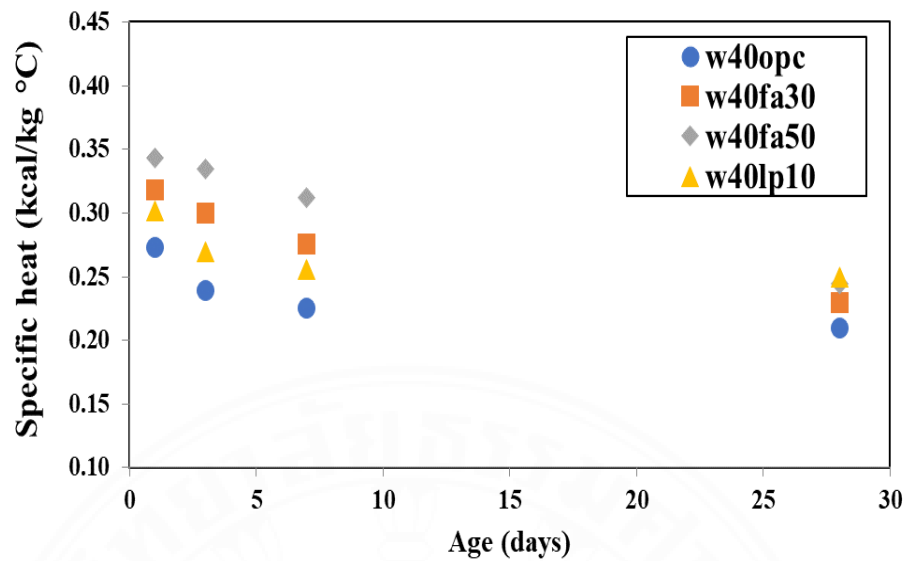


Figure 5.7 Specific heat results of cement pastes with OPC, FA30, FA50, and LP10 at w/b ratio of 0.4

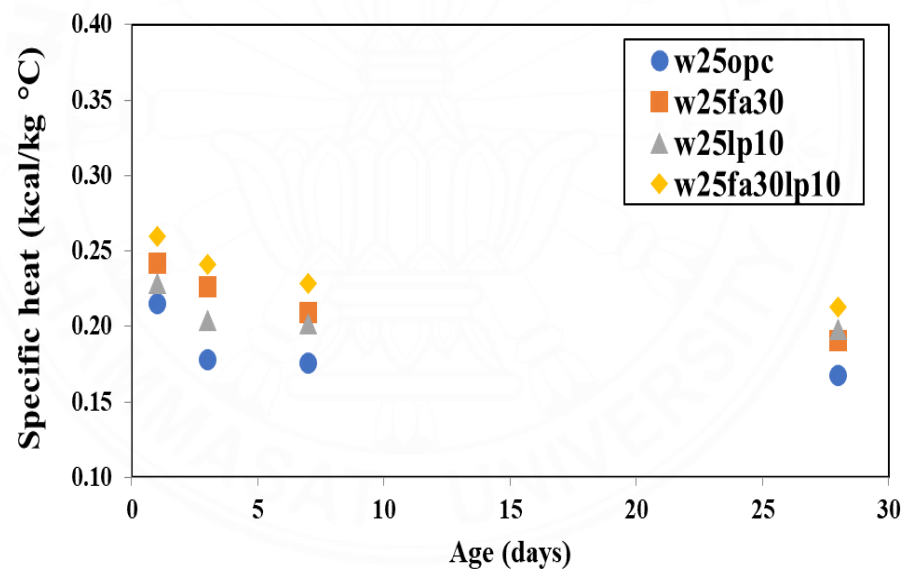


Figure 5.8 Specific heat results of cement pastes with OPC, FA30, LP10, and FA30LP10 at w/b ratio of 0.25

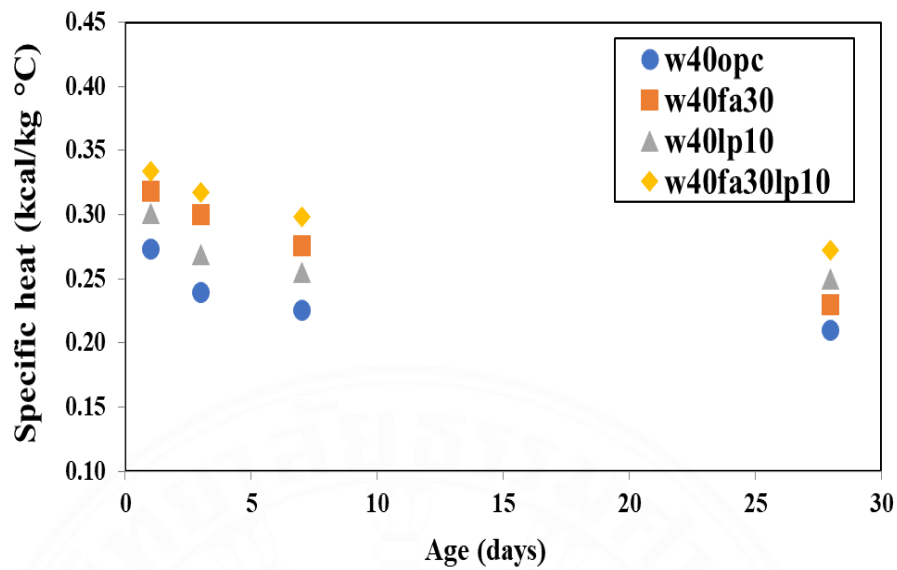


Figure 5.9 Specific heat results of cement pastes with OPC, FA30, LP10, and FA30LP10 at w/b ratio of 0.4

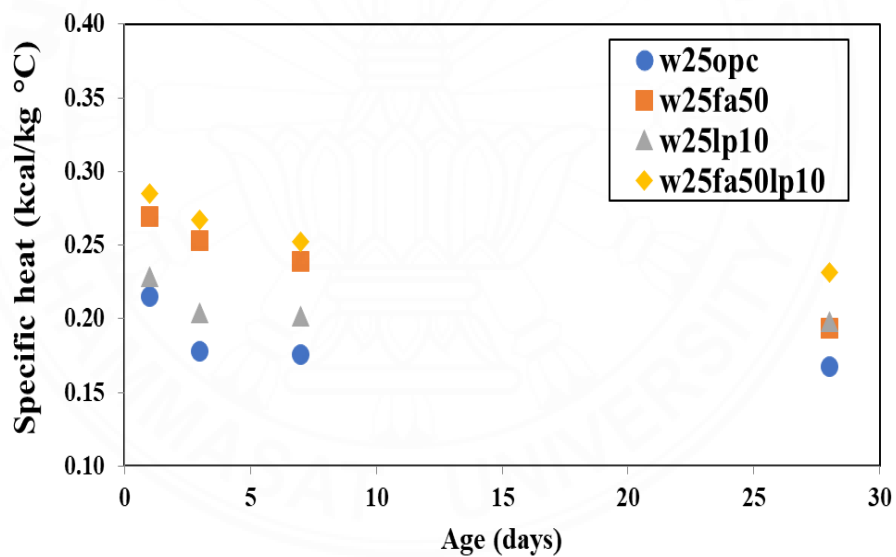


Figure 5.10 Specific heat results of cement pastes with OPC, FA50, LP10, and FA50LP10 at w/b ratio of 0.25

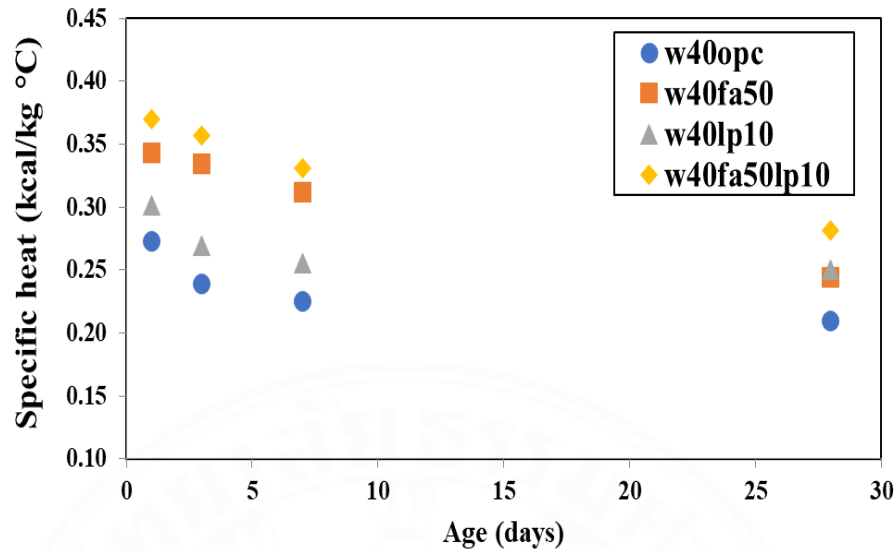


Figure 5.11 Specific heat results of cement pastes with OPC, FA50, LP10, and FA50LP10 at w/b ratio of 0.4

5.6 Verification of specific heat prediction model

As mentioned above, the proposed equations for specific heat prediction were used to calculate the specific heat at various ages and of various mix proportions. Then, the computed results and the experimental results were compared and shown in Figures 5.12 and 5.13. The figures show that the proposed model can cover the behaviors of the experimental results. The figures show higher specific heat for mixtures with lower cement content, especially at young age. The prediction model also displays that the specific heat of the mixtures with fly ash tends to decrease in later age. The specific heat prediction model was found quantitatively satisfactory with the experimental results and can be used to preliminarily predict the specific heat of the tested specimens with satisfactory accuracies.

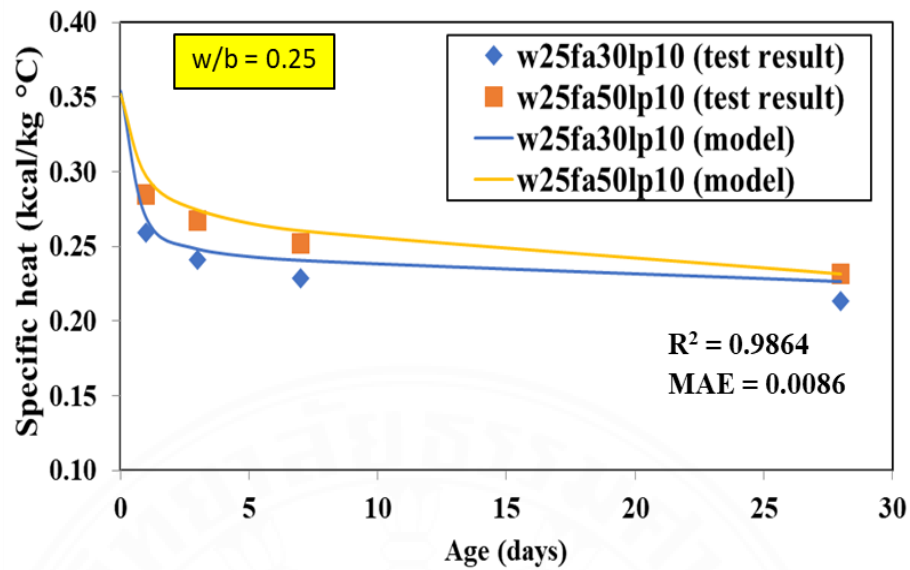


Figure 5.12 Comparison between specific heat experimental result and model of pastes with FA30LP10 and FA50LP10 at w/b ratio of 0.25

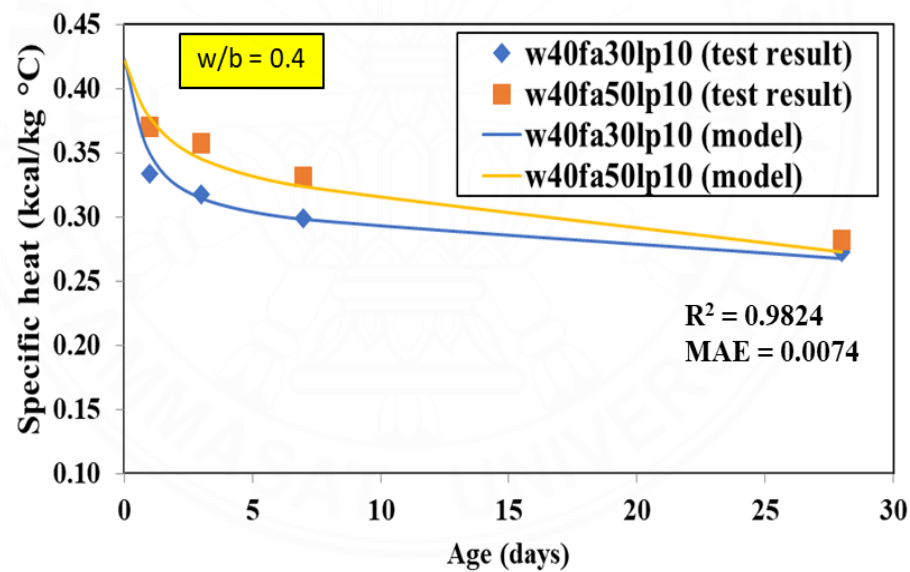


Figure 5.13 Comparison between specific heat experimental result and model of pastes with FA30LP10 and FA50LP10 at w/b ratio of 0.4

CHAPTER 6

DEGREE OF HYDRATION REACTION

General

The process of hydration reaction in ordinary Portland cement will take place immediately when cement is mixed with water. Hydration reaction of is the process of cement reacting with water, giving the products that make the mixture strong and turning from fluid state into hardened state based on Bullard et. al (2011). Neville (2011) explained the hydration process as the reactions taking place in a water-reacted cement paste of ordinary Portland cement after turning into a bonding agent. The degree of reaction is the term to express how much the cement compounds have already reacted with water or processed. Equations 6.1 to 6.4 were used to briefly illustrate the hydration reactions of each main cement compounds which are C_3S , C_2S , C_3A , C_4AF .

C_3S or tricalcium silicate is a cement component from the silicate compound group. The compressive strength development is related to this component due to the hydration product, C-S-H. Based on Neville (2011), physical properties of this component lead to the impact on setting and hardening process of cement. Equation (6.1) presents the hydration reaction of the C_3S .

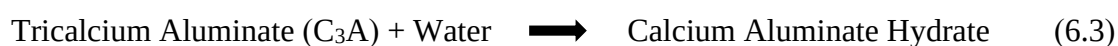


C_2S or dicalcium silicate is another cement component from the silicate compound group. One of the hydrated products is C-S-H leading to strength development. The physical properties of this component lead to the impact on setting and hardening process of cement according to Neville (2011). Equation (6.2) presents the hydration reaction of the C_2S .

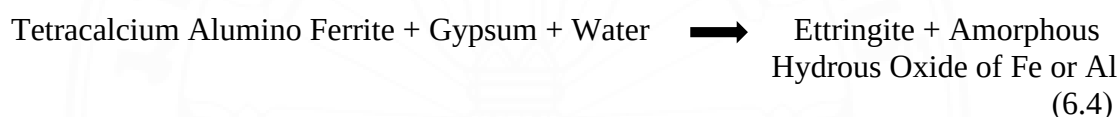


C_3A or tricalcium aluminate is a cement component from the aluminate compound group. The negative effects of water reacted cement paste especially for

mass concrete can be found from the hydration reaction of tricalcium aluminate. This component contributes very little to the strength of concrete and is considered to be undesirable. The reaction between water and this component only generates heat and stiffen immediately, known as the false setting. Gypsum is introduced to prevent this phenomenon, Neville (2011).



C₄AF or tetracalcium aluminato ferrite is another cement component from the aluminate compound group. There is not a much important role in strength development due to this component but it is essential for heat generation. This component can react with water and gypsum and form ettringite as shown below.



6.1 Degree of hydration reaction prediction model

The cement hydration is the reaction of major cement components reacting chemically with water and turns into hydrated products. The degree of hydration reaction prediction model for ternary binder was set to be time-dependently with the function of water to binder, concrete temperature at the considering time, and age. This study adopted the already proposed degree of hydration reaction prediction model from the previous studies by Saengsoy (2002) and Choktaweekarn (2008) to formulate degree of hydration reaction model for ternary binder system. From the previous models, the time-dependent degrees of hydration reaction for cement compounds which are C₃S, C₃A, C₂S, and C₄AF have been proposed and will be used in this study.

As a consequence, the degree of hydration reaction calculation models from Choktaweekarn (2008) were adopted in this study. The equations for degree of hydration at any concrete temperature are formed as shown in Equations (6.5) to (6.8). Any constant values to be used in the equations are shown in the previous studies.

$$\alpha_{C_2S}(t) = \frac{1 - \exp[(A/(B + \exp(C \cdot w/b))) \cdot \tan^{-1}(D \cdot (w/b)^E \cdot t)]}{1 + \exp[F \cdot t^G]} \times 100 \quad (6.5)$$

$$\alpha(t)_{C_3A} = \eta_{C_3A} \cdot \frac{1 - \exp[A \cdot \tan^{-1}(B \cdot (w/b)^C \cdot t)]}{1 + \exp[(D \cdot (w/b)^3 + E \cdot (w/b)^2 + F \cdot w/b + G) \cdot \tan^{-1}(H \cdot t)]} \times 100 \quad (6.6)$$

$$\alpha(t)_{C_3S} = \eta_{C_3S} \cdot \frac{1 - \exp[(A/(B + \exp(C \cdot w/b))) \cdot \tan^{-1}(D \cdot t)]}{1 + \exp[(E \cdot (w/b)^3 + F \cdot (w/b)^2 + G \cdot w/b + H) \cdot \tan^{1/(I + \exp(K \cdot w/b))}]} \times 100 \quad (6.7)$$

$$\alpha(t)_{C_4AF} = \frac{1 - \exp[A \cdot \tan^{-1}(B \cdot (w/b)^C \cdot t)]}{1 + \exp[(D \cdot (w/b)^3 + E \cdot (w/b)^2 + F \cdot w/b + G) \cdot \tan^{-1}(H \cdot t)]} \times 100 \quad (6.8)$$

where $\alpha_i(t)$ is the degree of hydration at any consideration time t of cement compound i (%). i is C_3A , C_3S , C_2S , or C_4AF . w/b is the water to binder ratio at time t which is the age of concrete (days).

6.1.1 Effects of fly ash and limestone powder on degree of hydration reaction

The roles of fly ash and limestone powder in the degree of hydration reaction are not just only to lower cement content in the mixture but also the physical phenomena that enhances the degree of hydration reaction. Many previous studies of fly ash concrete and limestone powder concrete have also stated and proposed the impacts of fly ash or limestone powder on the degree of hydration reaction, especially on two cement compounds which are C_3S and C_3A such as the studies of Saengsoy (2002) and Choktaweekarn (2008). The physical acceleration factor from fly ash and limestone powder for C_3S and C_3A hydrations are introduced and calculated from the degree of hydration, age of concrete, Blaine's fineness of replacing materials, weight of cement, and replacement ratio of replacing material, and unit weight of concrete. In this study, fly ash with limestone powder are also assumed to enhance the hydration reaction simulation models and the physical acceleration factors of C_3S and C_3A hydrations for ternary binder concrete are proposed. The weight ratio concept is applied for physical acceleration factors of C_3S and C_3A . The model parameters are Blaine's fineness of replacing materials, weight of cement and replacing materials, and replacement ratio of replacing material. Then the physical acceleration factor of C_3S and C_3A calculation models are proposed as in Equations (6.9) to (6.10).

$$\phi_{C_3S} = -1.6612 \left(\frac{\alpha_{C_3S}(t-1)}{100} \right)^4 + 3.8991 \left(\frac{\alpha_{C_3S}(t-1)}{100} \right)^3 - 2.4864 \left(\frac{\alpha_{C_3S}(t-1)}{100} \right)^2 - 0.2045 \left(\frac{\alpha_{C_3S}(t-1)}{100} \right) + 0.942 \tan^{-1} \left(\left(\frac{(F_f \times rfa) + (F_{lp} \times rlp)}{rfa + rlp} \right) - \left(\frac{W_c + W_f + W_{lp}}{UW} \right) \right)^{rfa + rlp} \quad (6.9)$$

$$\phi_{C_3A} = 1.0281 \left(\frac{\alpha_{C_3A}(t-1)}{100} \right)^4 - 1.5095 \left(\frac{\alpha_{C_3A}(t-1)}{100} \right)^3 + 0.4181 \left(\frac{\alpha_{C_3A}(t-1)}{100} \right)^2 - 0.1354 \left(\frac{\alpha_{C_3A}(t-1)}{100} \right) + 0.78 \tan^{-1} \left(\left(\frac{(F_f \times rfa) + (F_{lp} \times rlp)}{rfa + rlp} \right) - \left(\frac{W_c + W_f + W_{lp}}{UW} \right) \right)^{rfa + rlp} \quad (6.10)$$

where ϕ_{C_3S} and ϕ_{C_3A} are the physical acceleration factors for C_3S and C_3A hydrations. F_f and F_{lp} are Blaine's fineness of fly ash and limestone powder, respectively (cm^2/g). rfa and rlp are the replacement level of fly ash and limestone powder, respectively. W_c , W_f , and W_{lp} are the weight of cement, fly ash, and limestone powder, respectively (kg). UW is the unit weight of concrete (kg/m^3).

The studied mix proportions used to demonstrate the proposed physical acceleration factor of C_3S and. Fly ash or limestone powder is used to replace cement for binary binder as the studied mix proportions. Fly ash and limestone powder replacement ratios are 0.3 and 0.1, respectively. Then limestone powder replacement ratio of 0.1 is used to replace cement by keeping the replacement ratio of fly ash constant for the ternary binder. The proposed physical acceleration factors of C_3S and C_3A hydrations for ternary binder are illustrated as in the Figures 6.1 to 6.4.

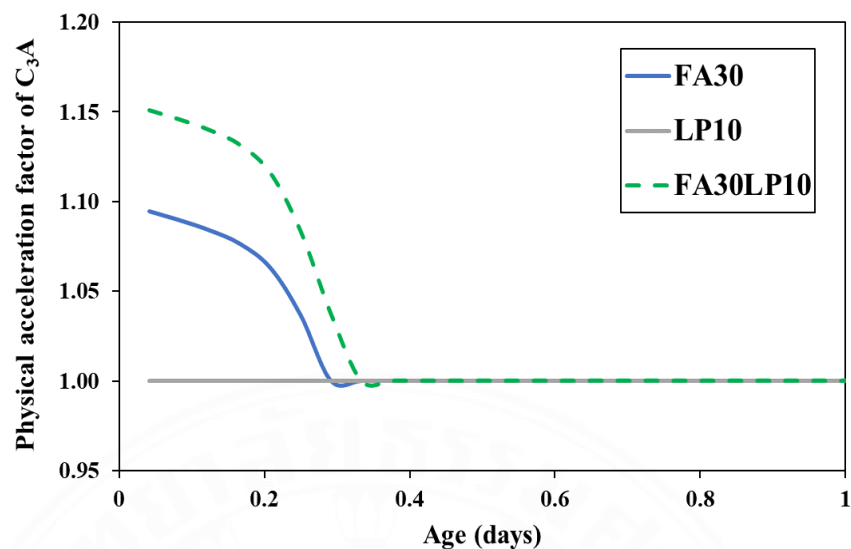


Figure 6.1 Physical acceleration factor of C_3A with FA30, LP10, and FA30LP10

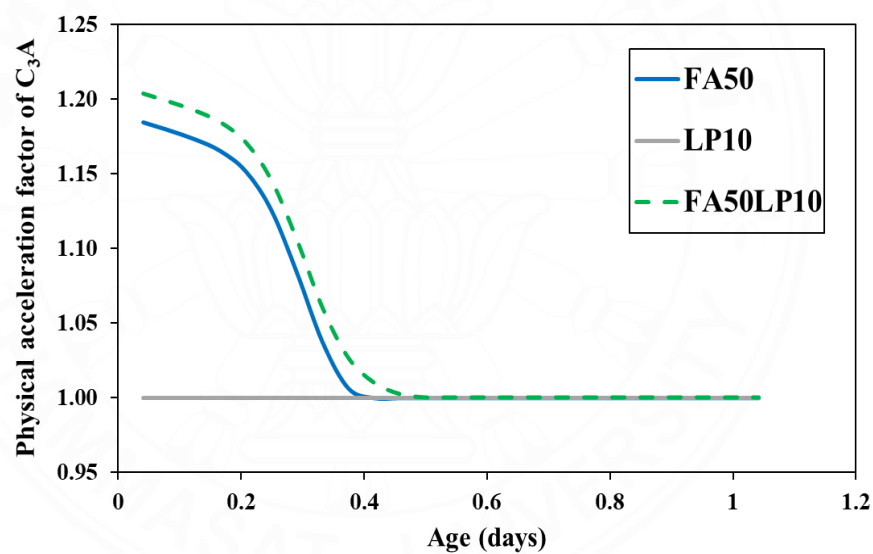


Figure 6.2 Physical acceleration factor of C_3A with FA50, LP10, and FA50LP10

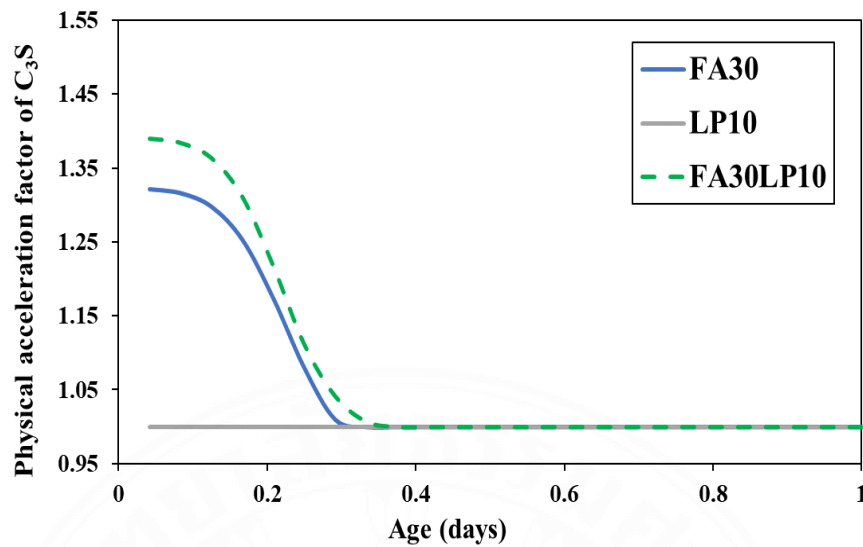


Figure 6.3 Physical acceleration factor of C_3S with FA30, LP10, and FA30LP10

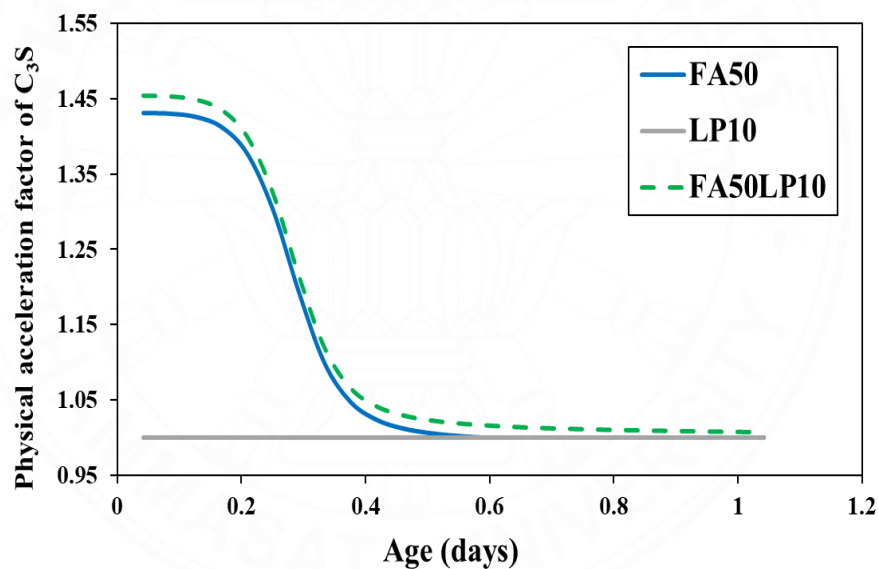


Figure 6.4 Physical acceleration factor of C_3S with FA50, LP10, and FA50LP10

6.1.2 Effects of fly ash on retardation of C_3S reaction

Tatong (2001) stated that fly ash chemically retards the hydration reaction of cement and proposed a retardation factor (ψ). Choketaweekarn (2008) found that the retardation effect on C_3S reaction of fly ash varies with temperature. In this study, it is assumed that fly ash in ternary binder concrete retards the C_3S reaction in the same behavior as in the previous studies.

The retardation factor, proposed by Choketaweekarn (2008), is calculated as in Equation 6.11.

$$\psi = \kappa \left[\{0.0866 \cdot (w/b)^{-2.443} - 1.032 \cdot (w/b)\} \cdot rfa^{2.151} + 1 \right] \cdot \left(\frac{\alpha_{C3S}(t)}{100} \right)^{3.409 \cdot rfa^{1.952}} \quad (6.11)$$

where ψ is the retardation factor of C_3S hydration by fly ash, $\alpha_{C3S}(t)$ is the degree of hydration of C_3S for the mix without fly ash (%), rfa is replacement ratio of fly ash in total binder, w/b is water to binder ratio and κ is the temperature factor.

$$10^\circ\text{C} < T < 20^\circ\text{C}, \quad \kappa = 0.2$$

$$20^\circ\text{C} < T < 30^\circ\text{C}, \quad \kappa = 0.08 \cdot T - 1.4$$

$$T > 30^\circ\text{C}, \quad \kappa = 1.0$$

where T is the temperature ($^\circ\text{C}$).

6.2 Average degree of hydration reaction

Based on the basic assumption that hydration reaction of each cement compound is independent, the average degree of hydration reaction simulation for ternary binder system was formulated as a function of average weight fraction of the hydration degree with all cement compounds as shown in Equation 6.12.

$$\alpha(t)_{hy} = \frac{\sum_{i=1}^4 m_i \alpha_i(t)}{\sum_{i=1}^4 m_i} \quad (6.12)$$

where $\alpha(t)_{hy}$ is the cement average degree of hydration (%), i is the cement compound i which is C_3S , C_3A , C_2S , or C_4AF . m_i is unit mass of each compound (kg/m^3). $\alpha(t)_i$ is the individual hydration degree of each cement compound (%).

Figure 6.5 exhibits an example of simulation of average degree of hydration reaction for ternary binder and shows that the mixtures containing limestone powder produce slightly higher average degree of hydration reaction than the mixtures with no limestone powder at the very early age of concrete caused by the physical acceleration effect from limestone powder.

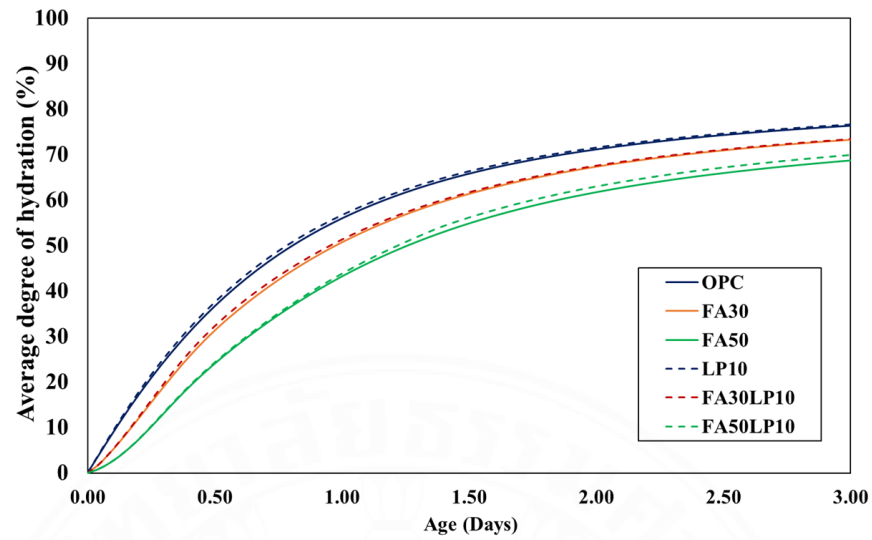


Figure 6.5 Average degree of hydration of cement paste with FA30, FA50, LP10, FA30LP10 and FA50LP10

CHAPTER 7

TOTAL HEAT GENERATION

General

By the assumption that all of the cementitious reactions in concrete are considered as the exothermic reactions, which means the reactions generate heat when the compounds reacted with water. Based on Neville (2011), the hydration reaction of cement is also common with many chemical reactions which produce heat when the certain substances react with water and the heat evolution of cement depends on both properties of cement. The total heat generation in concrete is the sums of the cumulative hydration heat of each cement compound including monosulphate and ettringite formations and the pozzolanic reaction of fly ash if any. There are some previous studies providing total heat generation model for fly ash concrete and limestone powder concrete but the total heat generation model for ternary binder concrete is still not available and will be proposed in this study.

7.1 Total heat prediction model

The total heat generation for ternary binder concrete is calculated by the addition of the heat generation due to the reaction of each cement compound, ettringite and monosulphate formations, and pozzolanic reaction of fly ash. Limestone powder, possessing the inactive property, is considered not to react with water so the hydration reaction and the heat generation due to limestone powder is not provided. The total heat generation model prediction for fly ash and limestone powder concrete is proposed as in Equation (7.1).

$$Q(t) = Q_{C3S}(t) + Q_{C2S}(t) + Q_{C3A}(t) + Q_{C4AF}(t) + Q_{C3AET}(t) + Q_{C4AFET}(t) + Q_{FA}(t) \quad (7.1)$$

where $Q(t)$ is the cumulative heat of concrete at any consideration time t (kcal/kg of concrete). $Q_{C3S}(t)$, $Q_{C2S}(t)$, $Q_{C3A}(t)$, $Q_{C4AF}(t)$, $Q_{FA}(t)$, $Q_{C3AET}(t)$, and $Q_{C4AFET}(t)$ are the cumulative hydration heat generation of C_3S , C_2S , C_3A , C_4AF , ettringite

formation by C_3A , ettringite formation by C_4AF , respectively, at the considered time t (kcal/kg of concrete).

Six mix proportions were designed to study the total heat generation model for ternary binder concrete. They are single binder which is cement only represented as OPC, binary binder, and ternary binder. Fly ash, represented as FA, was used to replace cement at replacement ratios of 0.3 and 0.5 for binary binder. For ternary binder, the replacement level of fly ash was kept constant and limestone powder was used as cement replacing materials at replacement ratio of 0.1 and represented as LP. The proposed total heat generation simulation model shows that LP10 shows the highest amount of total heat generation due to the dispersion effect that accelerates hydration reaction. Moreover, the total heat generation is higher in the mixture with higher cement contents as shown in Figure 7.1.

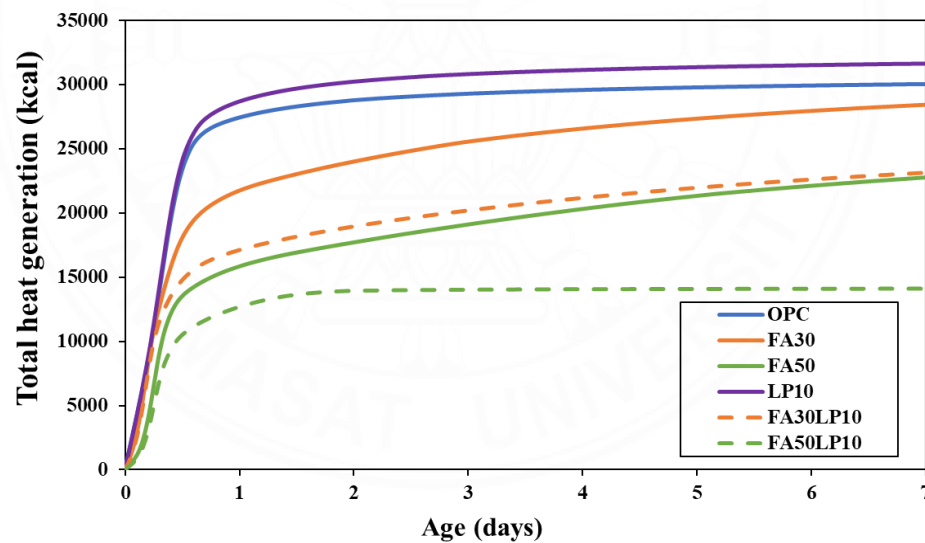


Figure 7.1 Total heat generation simulation results for OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10

7.2 Cumulative heat generation

7.2.1 Cumulative heat generation of each cement compound

The exothermic reaction which is the heat releasing process can be observed in the cement hydration reaction. As the reaction proceeds, each cement compound which is C_3S , C_3A , C_2S , or C_4AF generates heat. The cumulative heat generation

model for ternary binder concrete was proposed and it is calculated individually for each cement compound as shown in Equation (7.2).

$$Q_i(t) = \frac{\alpha_i(t)}{100} \cdot Q_{i, \max} \cdot w_i \cdot \Phi_i \quad (7.2)$$

where $Q_i(t)$ is the cumulative heat generation of cement compound i at the consideration age (kcal/kg). $\alpha_i(t)$ is degree of hydration of cement compound i (%). $Q_{i, \max}$ is the cumulative heat generation at complete hydration of cement compound i (kcal/kg). w_i is the effective weight ratio of cement compound i . Φ_i is the dispersion factor of each cement compound. i is the cement compound C_3S , C_3A , C_2S , or C_4AF .

The dispersion factor was introduced to take into account for the better reaction of some part of un-hydrated cement substances in the hydration process. The presence of fly ash with limestone powder can enhance the hydration reaction of cement of each cement compound based on Thongsanitgarn (2014). However, the calculation model of fly ash together with limestone powder in the ternary binder system has still not developed. As a result, the dispersion faction calculation model for the ternary binder is proposed in this study. It is assumed that the presence of both limestone powder and fly ash in ternary binder concrete resulted in better dispersion of cement particles. The dispersion factor of ternary binder concrete is computed from the multiplication of dispersion factor of limestone powder and fly ash as in Equation 7.3.

$$\phi_{i,lp+fa} = \phi_{i,lp} \cdot \phi_{i,fa} \quad (7.3)$$

where $\phi_{i,lp+fa}$ is the dispersion factor of cement compound i of ternary binder concrete. $\phi_{i,lp}$ and $\phi_{i,fa}$ are the dispersion factors of each cement compound i due to limestone powder and fly ash, respectively, and can be calculated from Equations 7.4 and 7.5.

$$\phi_{i,lp} = -19.74rlpc^6 + 66.83rlpc^5 - 88.55rlpc^4 + 57.89rlpc^3 - 19.24rlpc^2 + 2.939rlpc + 0.8 \quad (7.4)$$

$$\phi_{i,fa} = -6.19rfa^6 + 14.3rfa^5 - 9.16rfa^4 - 0.2rfa^3 + 1.06rfa^2 + 0.38rfa + 0.8 \quad (7.5)$$

where rlpc is the weight ratio of limestone powder to cement with limestone powder and rfa is the weight ratio of fly ash to total binder, respectively.

The mix proportions used to demonstrate are cement only, fly ash replacing cement at replacement ratios of 0.3 and 0.5 for binary binder, and limestone powder as cement replacing materials at replacement ratios of 0.1 and 0.2 for the ternary binder. The proposed dispersion factor for ternary binder shows that at the same replacement level of fly ash, the replacement of limestone powder causes the dispersion factor to be lower and the higher limestone powder replacement, the lower dispersion factor as shown in Figures 7.2 and 7.3 for the mixtures with 30% fly ash and 50% fly ash, respectively.

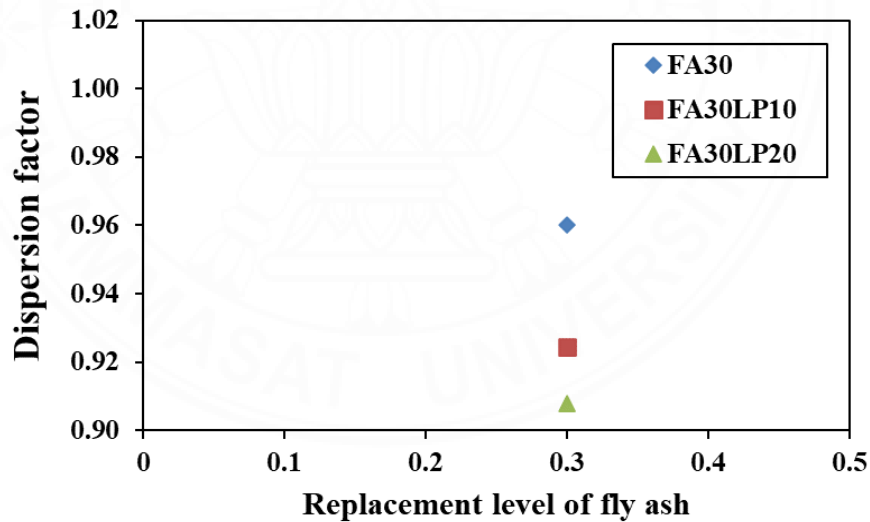


Figure 7.2 Dispersion factor for concrete with FA30, FA30LP10, and FA30LP20

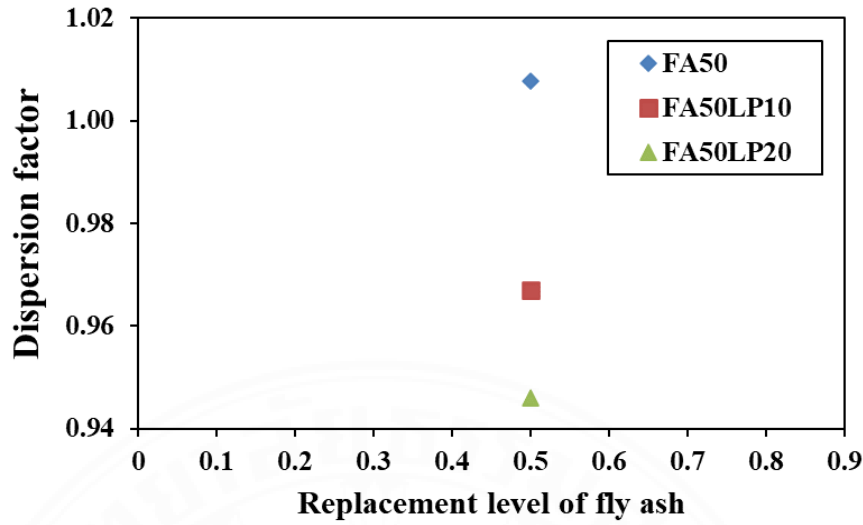


Figure 7.3 Dispersion factor for concrete with FA50, FA50LP10, and FA50LP20

7.2.2 Cumulative heat generation of fly ash

Choktaweekarn (2008) proposed the cumulative heat generation from fly ash which can be computed as shown in Equation 7.6.

$$Q_{FA}(t) = \frac{\alpha_{poz}(t)}{100} \cdot Q_{FA,max} \cdot w_{fa} \quad (7.6)$$

where $Q_{FA}(t)$ is the cumulative heat generation of fly ash at age t (kcal/kg of concrete), $\alpha_{poz}(t)$ is the degree of pozzolanic reaction at age t (%), $Q_{FA,max}$ is the maximum cumulative heat generation of fly ash (kcal/kg of fly ash), and w_{fa} is the weight ratio of fly ash in 1 m³ of concrete to the unit weight of concrete (no unit).

The maximum cumulative heat generation of fly ash proposed by Choketaweekarn (2008) is calculated according to the function of calcium oxide content in fly ash as shown in Equation 7.7.

$$Q_{FA,max} = 100 + 1.75 \cdot \%CaO_{fa} \quad (7.7)$$

where $\%CaO_{fa}$ is calcium oxide content in fly ash (% by weight of fly ash)

The degree of pozzolanic reaction proposed by Saengsoy (2002) was adopted in this study.

7.2.3 Cumulative heat generation of ettringite and monosulphate

The cumulative heat from the ettringite and monosulphate production at a constant concrete temperature from the reaction of C_3A , and C_4AF with gypsum as proposed by Saengsoy (2002) can be computed from Equations 7.8 and 7.9.

$$Q_{C_3AET}(t) = \frac{(0.4973 \cdot T^{1.1571} + 58.1539) \cdot t}{-0.9147 + \exp[(0.0089 \cdot T^{0.9615} + 0.4303) \cdot t]} \cdot w_{Gyp} \quad (7.8)$$

$$Q_{C_4AFET}(t) = \frac{(0.1186 \cdot T^{1.1571} + 13.8753) \cdot t}{-0.9147 + \exp[(0.0089 \cdot T^{0.9615} + 0.4303) \cdot t]} \cdot w_{Gyp} \quad (7.9)$$

where $Q_{C_3AET}(t)$ and $Q_{C_4AFET}(t)$ are cumulative heat generation at age t by the ettringite and monosulphate formation from the reaction of gypsum with C_3A , and C_4AF , respectively (kcal/kg of concrete). w_{Gyp} is the weight ratio of gypsum to the unit weight of concrete. T is concrete temperature ($^{\circ}C$) and t is the age of concrete (days).

7.3 Calculation of adiabatic temperature rise

Based on the heat generation equation, the adiabatic temperature for ternary binder concrete can be calculated based on Equation 7.10.

$$Q(t) = m \cdot c(t) \cdot \Delta T(t) \quad (7.10)$$

where $Q(t)$ is the total heat generation at the considered age (kcal/kg). m is the concrete mass (kg/m^3). $c(t)$ is the concrete specific heat at the considered age (kcal/kg $^{\circ}C$). $\Delta T(t)$ is the temperature change in concrete at the considered age ($^{\circ}C$). t is the considered age of concrete (days).

According to the time-dependent total heat generation simulation model for ternary binder concrete which has been already mentioned in this chapter and specific heat prediction model for ternary binder which is also time-dependent and was

already proposed in Chapter 5, the simulation of adiabatic temperature rise profile of ternary binder concrete can be executed.



CHAPTER 8

THERMAL CONDUCTIVITY

General

The improvement of accuracy for temperature rise analysis of concrete can be done by considering real condition of structure with the existing of heat loss to environment. Thermal conductivity is introduced and used as one of the inputs in the commercial FEM software to formulate the semi-adiabatic temperature rise profile of concrete. The definition of thermal conductivity is the transfer of heat via a certain sectional area of considered mass for a unit temperature gradient. As previously mentioned in Chapter 2, higher accuracy for thermal conductivity analysis in concrete structure can be achieved with time dependent simulation model instead of time independent parameter analysis.

For fly ash together with limestone powder concrete, the thermal conductivity prediction model has still not been proposed. As a result, the thermal conductivity prediction model for the ternary binder is developed in this study.

8.1 Thermal conductivity prediction model

Thermal conductivity increases with time due to the time-dependent increase of hydrated products. The thermal conductivity of ternary binder concrete is calculated based on volumetric ratio (n_i) of concrete ingredients and their individual thermal conductivity values (k_i) as shown in Equations 8.1 and 8.2. It is assumed that the hydrated products are from cement and fly ash while limestone powder is an inert material and does not react to form any products. Therefore, the volume of limestone powder is constant. Table 8.1 provides thermal conductivity of each ingredient in concrete.

$$k(t) = n_g k_g + n_s k_s + n_{ra} k_{ra} + n_{fw}(t) k_{fw} + n_{uc}(t) k_c + n_{ufa}(t) k_{fa} + n_{hp}(t) k_{hp} + n_{lp} k_{lp} \quad (8.1)$$

$$n_{hp}(t) = 1 - n_g - n_s - n_{fw}(t) - n_{uc}(t) - n_{ufa}(t) - n_{lp} \quad (8.2)$$

where $k(t)$ is the thermal conductivity of concrete at the considered age (kcal/m.hr °C). n_g , n_s , n_{ra} , and n_{ulp} are volumetric ratios of coarse aggregate, fine aggregate,

air, and limestone powder, respectively. $n_{fw}(t)$, $n_{uc}(t)$, $n_{ufa}(t)$, and $n_{hp}(t)$ are the volumetric ratios of free water, unhydrated cement, unreacted fly ash, and hydrated products, respectively, at considered age. k_g , k_s , k_{ra} , k_{fw} , k_c , k_{fa} , k_{hp} , and k_{lp} are thermal conductivity of coarse aggregate, fine aggregate, air, free water, cement, fly ash, hydrated products, and limestone powder, respectively (kcal/m.hr °C)

Table 8.1 The thermal conductivity values of concrete ingredients by Choktaweekarn (2008 and 2010)

Ingredients	Thermal conductivity
	(kcal/m hr °C)
Water	0.51
Limestone powder	2.29
Coarse aggregate	2.20
Fine aggregate	3.00
Air	0.026
Cement	1.33
Fly ash	0.65
Hydrated product	1 (cement with fly ash)
Hydrated product	1.35 (cement with limestone powder)

Six mix proportions were used to verify the proposed models. The mix proportions were designed with water to binder of 0.4. Fly ash replacement ratios were 0.3 and 0.5 and limestone powder replacement ratio was 0.1 for binary binder. In the case of ternary binder, limestone powder was used to replace total binder at 0.1 and keep the level of fly ash replacement ratio constant. The studied mix proportion are shown in Table 8.2. For the mix designation, “OPC”, “FA”, and “LP” are referred to cement only, fly ash replacement ratio, and limestone powder replacement ratio, respectively.

Table 8.2 Mix proportions for studying the proposed thermal conductivity prediction model for ternary binder concrete

Mix Designation	Cement (kg./m ³)	Fly ash (kg./m ³)	Limestone powder (kg./m ³)	Water (kg./m ³)	River sand (kg./m ³)	Crushed limestone (kg./m ³)	Admixture (kg./m ³)
OPC	406	0	0	163	791	1069	3.45
FA30	277	119	0	159	791	1069	2.58
FA50	195	195	0	156	791	1069	1.75
LP10	364	0	40	162	791	1069	3.43
FA30LP10	236	118	39	158	791	1069	2.36
FA50LP10	155	194	39	155	791	1069	1.36

The computed thermal conductivity results for the ternary binder concrete are shown in the Figures 8.1 and 8.2. Thermal conductivity increases with time due to the higher density as stated by Choktaweekarn (2008). The model results show that the thermal conductivity increases with time and the higher replacement ratio of fly ash results in lower thermal conductivity due to the lower value of thermal conductivity of fly ash compared to the other ingredients of concrete. Ternary binder mixtures show the higher thermal conductivity than binary binder because limestone powder has higher thermal conductivity value than other binders and hydrated products.

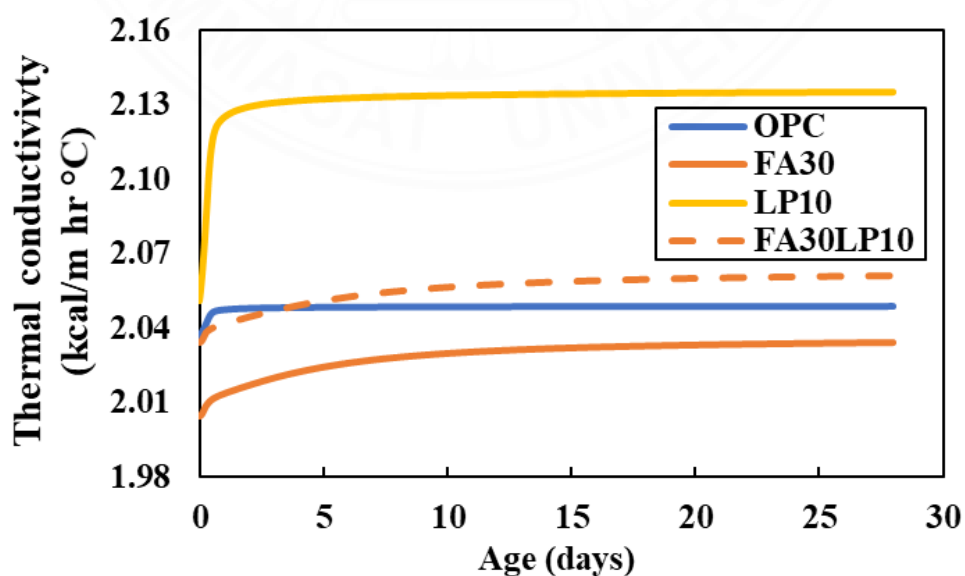


Figure 8.1 Thermal conductivity prediction model results for concrete with OPC, FA30, LP10, and FA30LP10

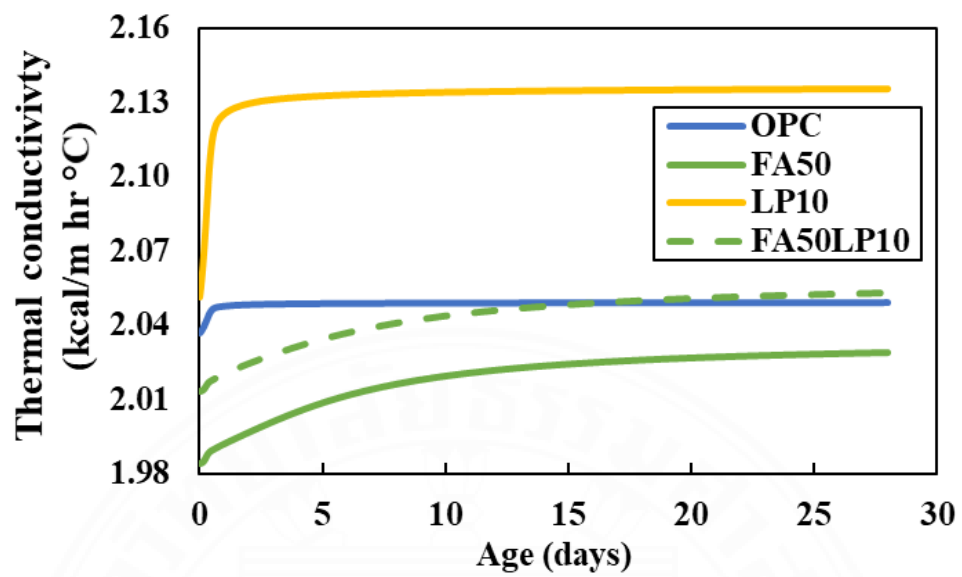


Figure 8.2 Thermal conductivity prediction model results for concrete with OPC, FA50, LP10, and FA50LP10

CHAPTER 9

EFFECTS OF FLY ASH AND LIMESTONE POWDER ON CONCRETE PROPERTIES OF TERNARY BINDER CONCRETE AND VERIFICATION WITH THE PROPOSED MODELS

General

The adiabatic temperature rise model for fly ash including limestone powder as cement replacing materials for ternary binder concrete is still not available and the experimental data on temperature rise are still limited. As a consequence, the adiabatic temperature rise prediction model for ternary binder is proposed and the experiments on temperature rise are conducted to verify the proposed model. Observation on effects of fly ash with limestone powder concrete on slump and slump loss, compressive strength, and tensile strain capacity are also carried out.

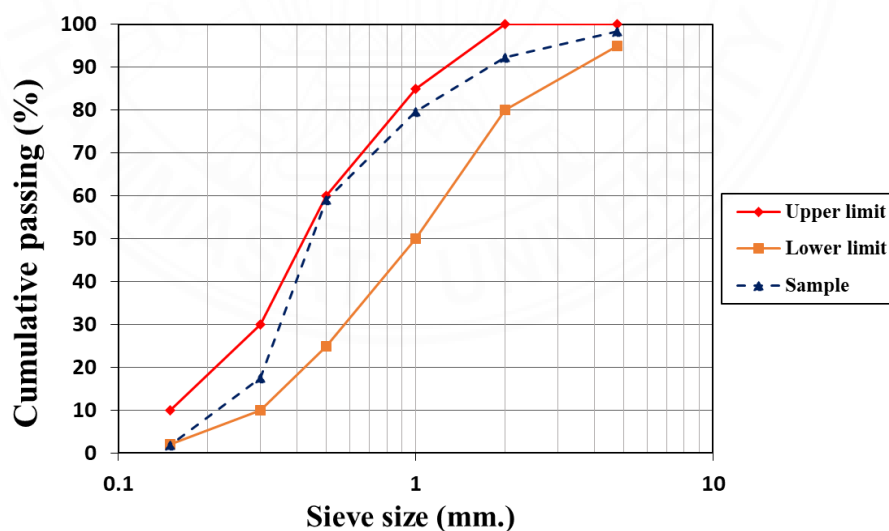
9.1 Materials and mix proportions

Ordinary Portland cement, Mae-Moh fly ash, and limestone powder were used for preparing the tested specimens. Ordinary Portland cement type I (OPC) according to ASTM-C150 and TIS-15 was used. The fly ash used in the tests and was categorized as class C and class 2b according to ASTM C618 and TIS 2135, respectively. Limestone powder which mainly contains CaCO_3 was also used to replace cement for mixture design. Chemical compositions and physical properties of the studied binders are shown in Table 9.1.

Fine aggregate is river sand with gradation based on ASTM C33 as shown in Figure 9.1, having specific gravity and absorption equal to 2.59 and 1.12, respectively. Crushed limestone coarse aggregate with a gradation according to ASTM C33 as shown in Figure 9.2 was used in the experiments. Its specific gravity and absorption are 2.75 and 0.55, respectively.

Table 9.1 Chemical compositions and physical properties of tested materials

Properties	Cement	Fly ash	Limestone powder
Chemical composition (%)			
SiO ₂	18.98	36.18	0.79
Al ₂ O ₃	5.33	20.21	0.42
Fe ₂ O ₃	3.16	13.89	0.23
CaO	65.34	18.74	54.66
MgO	1.15	2.69	0.59
SO ₃	2.65	3.74	N/A
LOI	2.54	0.25	43.21
Na ₂ O	0.16	1.14	0.02
K ₂ O	0.21	2.29	0.01
TiO ₂	0.29	0.4	0.01
P ₂ O ₅	0.06	0.21	0.03
MnO	0.05	0.12	0.01
SrO	0.03	0.09	N/A
Cr ₂ O ₃	0.01	N/A	N/A
ZnO	0.02	N/A	N/A
Propoerties			
Specific gravity (g/cm ³)	3.05	2.57	2.70
Blaine fineness (cm ² /g)	3,141	2,254	5,270

**Figure 9.1** Particle size distribution of fine aggregate based on ASTM C33

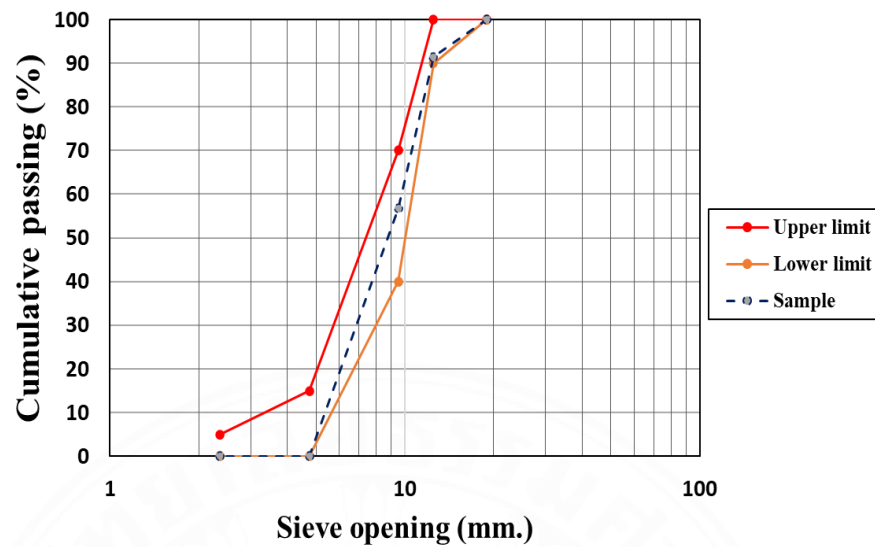


Figure 9.2 Particle size distribution of coarse aggregate based on ASTM C33

Mix proportion was designed based on the paste to volume of void of aggregate phase, water to binder ratio, replacement levels of fly ash and limestone powder. The water to binder was controlled 0.4. Fly ash was used to replace cement at fly ash to binder ratios of 0.3 and 0.5 and limestone powder was used to replace cement at limestone powder to binder ratio of 0.1 for the binary binder. Then, the replacement level of fly ash will be kept constant at 0.3 and 0.5 and limestone powder was used to replace cement at 0.1 of total binder for ternary binder. Six mixtures were designed and used thorough the experimental program as shown in Table 9.2. As the mix designation, “OPC”, “FA”, and “LP” refer to cement content only, fly ash replacement ratio, and limestone powder replacement ratio. The Kao-Mighty MX-T naphthalene-based water-reducing admixture was used to control the initial slump of the tested concrete to be within 10 ± 2.5 cm. It can be observed that the lower amount of admixture was used for the mixtures with the lower cement contents.

Table 9.2 Mix proportions of the tested concrete

Mix Designation	Cement (kg./m ³)	Fly ash (kg./m ³)	Limestone powder (kg./m ³)	Water (kg./m ³)	River sand (kg./m ³)	Crushed limestone (kg./m ³)	Admixture (kg./m ³)
OPC	406	0	0	163	791	1069	3.45
FA30	277	119	0	159	791	1069	2.58
FA50	195	195	0	156	791	1069	1.75
LP10	364	0	40	162	791	1069	3.43
FA30LP10	236	118	39	158	791	1069	2.36
FA50LP10	155	194	39	155	791	1069	1.36

9.2 Experimental programs

9.2.1 Slump and slump loss

As mentioned in Chapter 2, slump and slump loss tests are conducted to determine the performance of the workability of fresh concrete and working time for casting concrete. Fly ash normally possess the enhancing ability of the workability of concrete and delays the setting times which leads to lower rate of slump loss compared to no-fly ash mixture. The opposite phenomena can be found when introducing limestone powder as cement replacing material. Limestone powder concrete leads to lower initial slump and shorten working time with higher slump loss. In this study, ternary binder, cement with fly ash and limestone powder, is investigated to study the effects of limestone powder replacing cement with the constant ratio of fly ash replacement on the initial slump and slump loss.

After the mixing process was just finished, initial slump, based on ASTM C143, was measured on the concrete sample immediately. Then, the slump was observed every 30 minutes until 2 hours after mixing or until there is no measurable slump.

9.2.2 Compressive strength

Fly ash concrete has shown great improvement of later age compressive strength instead of early-age performance. On the contrary, limestone powder improves the early age compressive strength of concrete but worse performance can be observed for later ages. Therefore, the all-age enhancement of compressive

strength can be expected by the combination of fly ash and limestone powder addition into mix proportion. The ternary binder system is used to investigate compressive strength of concrete.

The specimens were cast in 10x10x10 cm plastic cube and wrapped with plastic to keep moisture. After 24 hours, the forms were removed and the specimen were cured in the water tank until the test ages at 3, 7, 28, and 91 days. Figures 9.3 and 9.4 show the preparation and test setup for the compressive strength test. The average value of compressive strength of each mixture can be achieved from at least 3 specimens.



Figure 9.3 The specimen's preparation in the plastic cube covered with the plastic wrap



Figure 9.4 The experimental setup for the compressive strength testing

9.2.3 Tensile strain capacity

Tensile strain capacity is described as the maximum tension capacity of the concrete before the tensile failure occurs. Tensile strain capacity is used to evaluate instead of tensile strength due to the accuracy and real characteristic of the structure. Direct tension test and flexural test have been discussed and attempted to evaluate tensile strain capacity. However, in term of convenience and real stress distribution across the member of structure, the flexural test will be used rather than direct tension based on Tongaroonsri and Tangtermsirikul (2008). Tensile strain capacity can be calculated based on Nguyen et. al (2010) which stated that flexural tensile strain can be directly converted to direct tensile cracking strains by multiplying with 0.7. The use of both fly ash and limestone powder to replace cement in mixtures are investigated and conducted in this study for tensile strain capacity of concrete.

The specimens were cast in 10x10x35 cm beam shape and covered with plastic wrap to prevent moisture changing. After 24 hours, the specimens were removed and cured in a water tank until the test ages of 1, 3, 7, and 28 days. Figures 9.5 to 9.7 show the preparation and test setup for the tensile strain capacity.

The method of flexural test based on ASTM C78 was used. A data logger was prepared for data recording until the specimen cracked. Two specimens were used to obtain the average for tensile strain capacity for each concrete mixture.



Figure 9.5 Preparation for concrete beam casting in the steel molds



Figure 9.6 Apparatus for tensile strain capacity testing



Figure 9.7 Experimental setup for tensile strain capacity testing

9.2.4 Semi-adiabatic temperature rise

Six boxes plywood formworks with the dimension of 50 cm x 50 cm x 50 cm were prepared for specimen casting. For each box, four styrofoam panels with a thickness of 5 cm. were attached at four sides of the plywood formwork. Silicone was used to seal all joints of styrofoam for leakage prevention. Thermocouples type K were placed at the center and 5 cm. away from the styrofoam side surface at 20 cm. above the bottom surface of styrofoam to record the temperature rise profile. The specimens were cast with the dimensions of 40 x 40 x 40 cm. Figures 9.8 and 9.9 show the test setup for the semi-adiabatic temperature rise and Figure 9.10 displays the temperature rise history recording system.

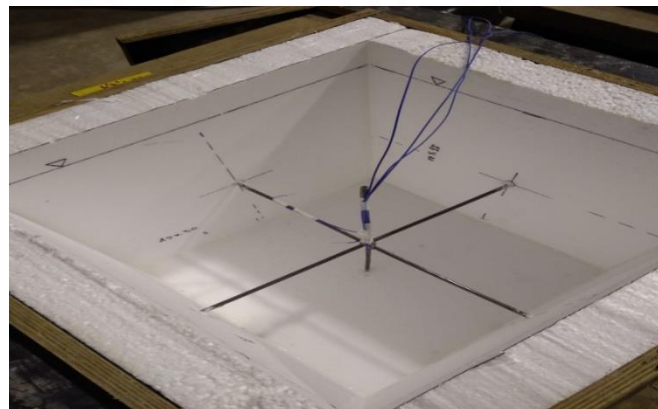


Figure 9.8 The inside of the prepared boxes for the semi-adiabatic temperature rise testing



Figure 9.9 Experimental setup for the semi-adiabatic temperature rise testing



Figure 9.10 Specimens with the temperature rise history recording system

9.2.5 Verification of the proposed model by semi-adiabatic temperature rise

Based on Equation (7.1), the total heat generation in ternary binder concrete with fly ash and limestone powder was calculated from the summation of cumulative heat generation which is time-dependent from the major reactions in concrete such as exothermic reactions of binders. Together with the specific heat prediction model for ternary binder concrete that was already proposed in the earlier section, the adiabatic temperature rise model of mass concrete containing fly ash and limestone powder can be obtained.

The proposed adiabatic temperature prediction model was used to generate the temperature rise results and the total heat generation and thermal properties based on

ternary binder concrete were used as the parameters for FEM to formulate the semi-adiabatic temperature rise profiles and then the verification with the laboratory test was performed.

9.3 Experimental results

9.3.1 Slump and slump loss

The experimental results of slump and slump loss for the tested mix proportions are shown in Figure 9.11. Initial slump was controlled at 10 ± 2.5 cm for all mixed concrete specimens. Firstly, the slump of concrete decreased with the time increase for all six mixtures.

For binary binder, the presence of fly ash in concrete caused slower slump loss than the cement only concrete. Higher replacement of fly ash is observed to decrease the rate of slump loss. This is because fly ash can retard hydration reaction. Faster slump loss is reported in the limestone powder concrete. Fly ash-limestone powder concrete lost their slump faster than fly ash concrete when compared at the same replacement level of fly ash in binary binder because of the enhancement the hydration reaction from the existence of limestone powder cause concrete to harden faster.

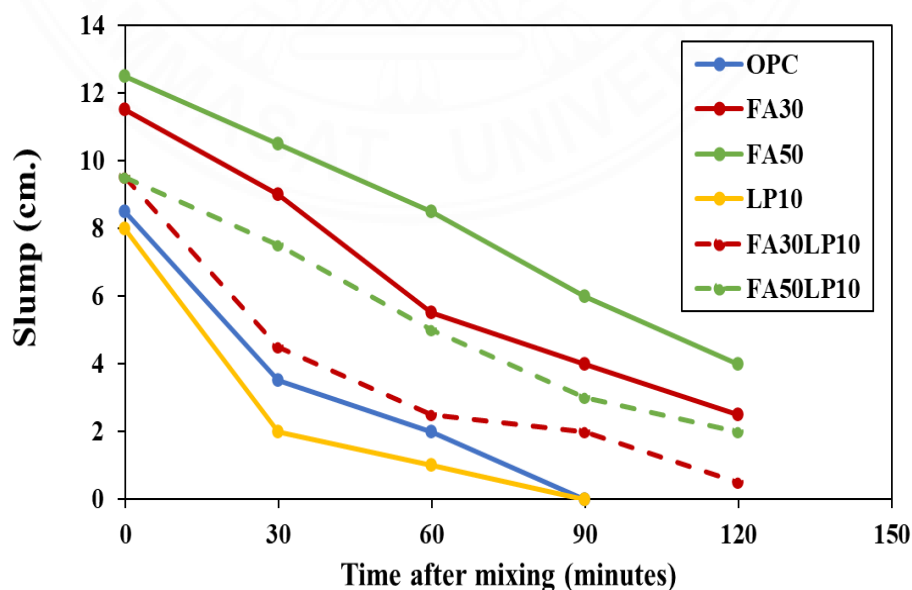


Figure 9.11 Initial slump and slump loss for concrete with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4

9.3.2 Compressive strength

Figure 9.12 exhibits the experimental results for compressive strength in the tested specimen. The experimental results clearly exhibit that compressive strength increases with time. Up to 28 days, the compressive strength is higher with the higher cement content except for LP10 which shows almost equal compressive strength with cement only at the age of 3 days and FA30 that yields the highest compressive strength compared to the other mixtures at 28 and 91 days because of the improvement of early compressive strength of limestone powder and the pozzolanic reaction proceeding at the later age, respectively. At 91 days, the compressive strength trending is FA30, OPC, FA30LP10, LP10, FA50, and FA50LP10 (in order from higher strength).

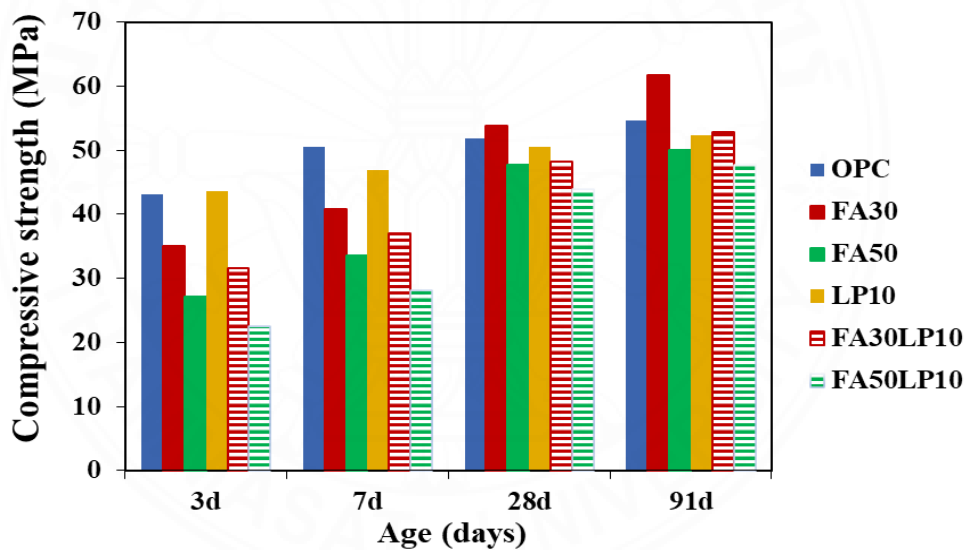


Figure 9.12 Compressive strength results for concrete with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4

9.3.3 Tensile strain capacity

Tensile strain capacity results of concrete up to 28 days of age are shown in Figure 9.13. The figure shows that the tensile strain capacity increases with the increase of time. The higher cement content in the mixtures gives the higher tensile strain capacity for all testing ages.

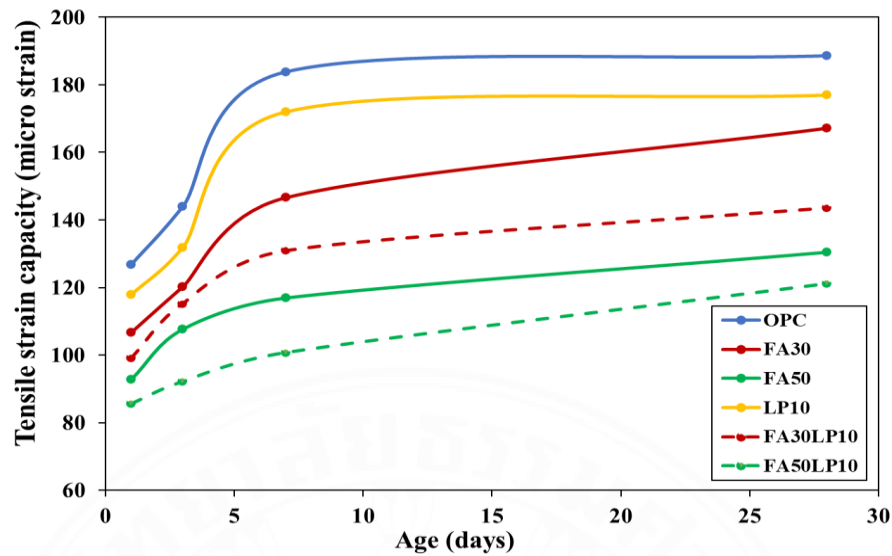


Figure 9.13 Tensile strain capacity results for concrete with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4

9.3.4 Relationship between Compressive strength and Tensile strain capacity

Tests on both compressive strength and tensile strain capacity of ternary binder concrete were conducted and investigated to observe the relationship as shown in Figure 9.14. Tensile strain capacity has a tendency to be proportional to compressive strength. The tensile strain capacity is higher with the higher compressive strength and can be observed with all six mix proportions.

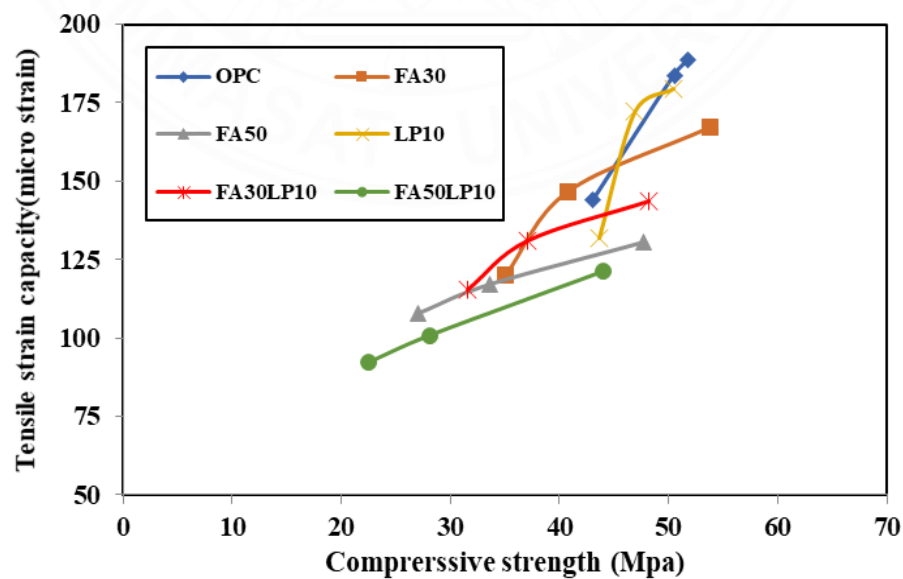


Figure 9.14 Relationship between compressive strength and tensile strain capacity with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4

9.3.5 Semi-adiabatic temperature rise

The experiment results of semi-adiabatic temperature rise of different tested mixtures are shown in Figure 9.15. The results show that the higher maximum temperature rise can be observed in the mixtures with higher cement content than those with the lower cement content because of the higher heat production from the exothermic hydration reaction of cement components resulted from the higher cement content.

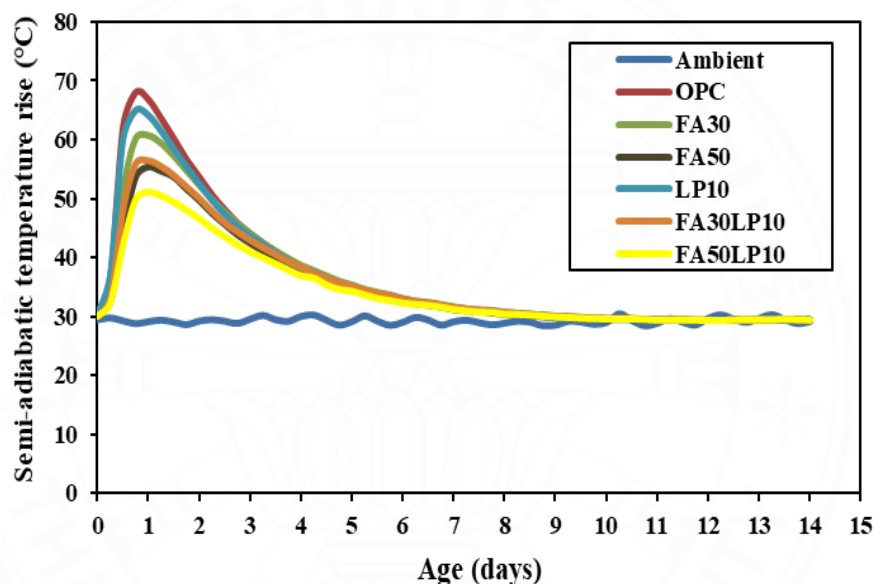


Figure 9.15 Semi-adiabatic temperature rise profile for concrete with OPC, FA30, FA50, LP10, FA30LP10 and FA50LP10 at w/b ratio of 0.4

In case of binary binder, fly ash and limestone powder as cement replacing materials, give the lower maximum temperature rise than the control cement-only specimen. Moreover, the higher fly ash replacement level shows the lower maximum temperature rise for the tested concrete.

Limestone powder was used to replace cement at replacement level of 0.1 of total binder and kept fly ash replacement level constant for ternary binder. The results show that the ternary binder mixtures produce lower maximum temperature rise than the binary binder mixtures at the same replacement levels of fly ash. It indicates that a ternary binder is useful and contains an advantage to improve thermal resisting mass concrete due to the lower maximum temperature rise production.

9.3.6 Verification of the proposed model by semi-adiabatic temperature rise

Comparison between temperature profiles which were generated from the adiabatic temperature rise simulation model with commercial FEM software and experimental data of ternary binder concrete was done as shown in Figures 9.16 to 9.21. The accuracy of the proposed adiabatic temperature rise model prediction for ternary binder concrete was found to be satisfactory with the laboratory tested specimens.

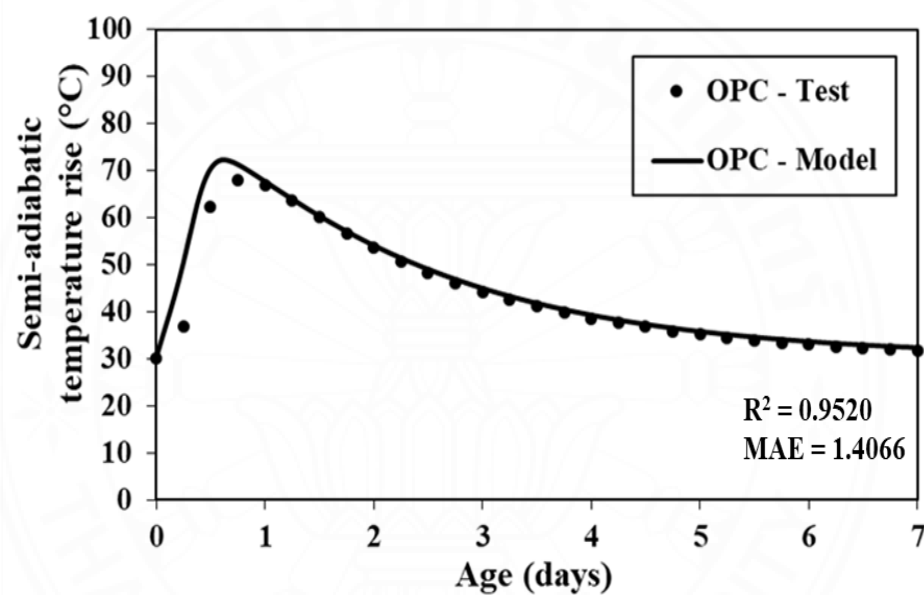


Figure 9.16 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with OPC

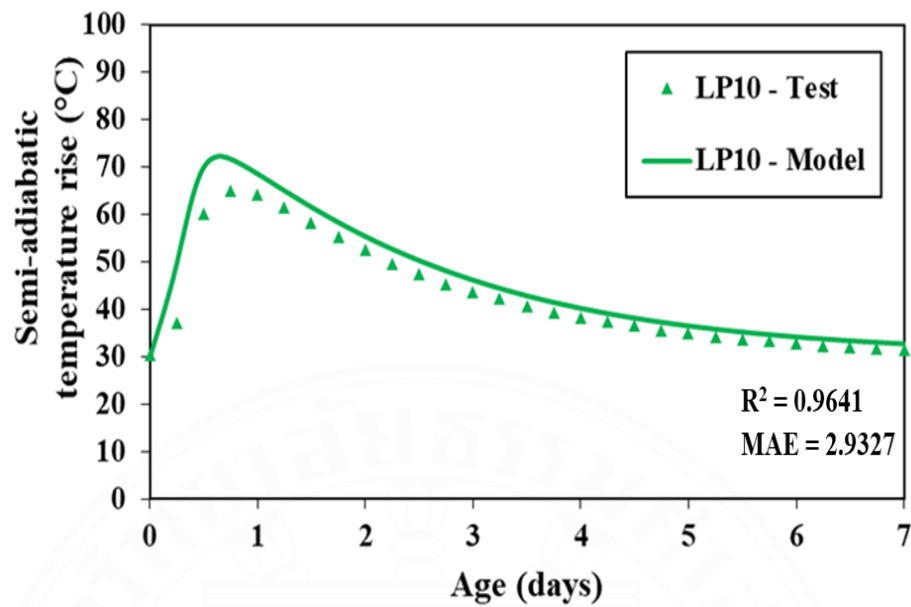


Figure 9.17 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with LP10

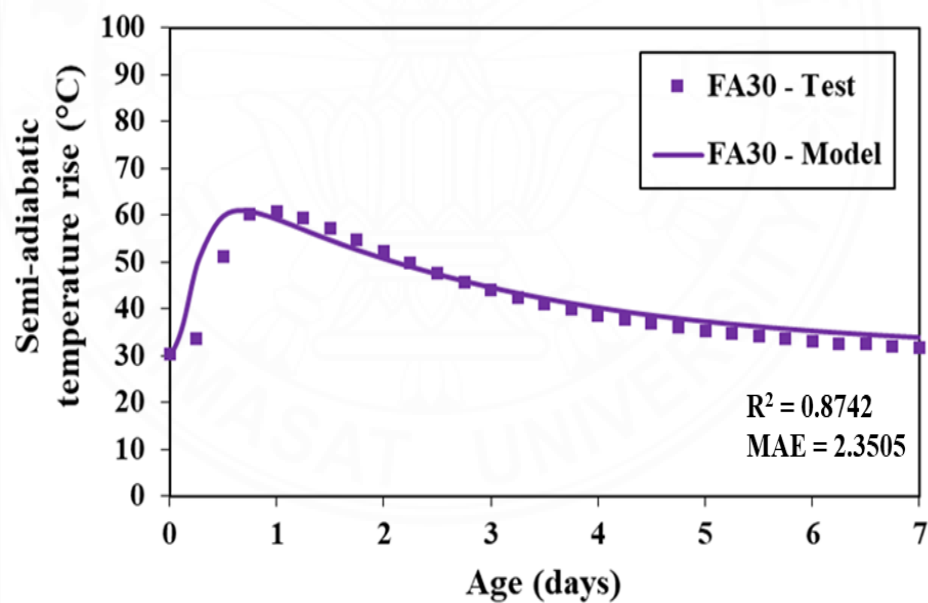


Figure 9.18 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA30

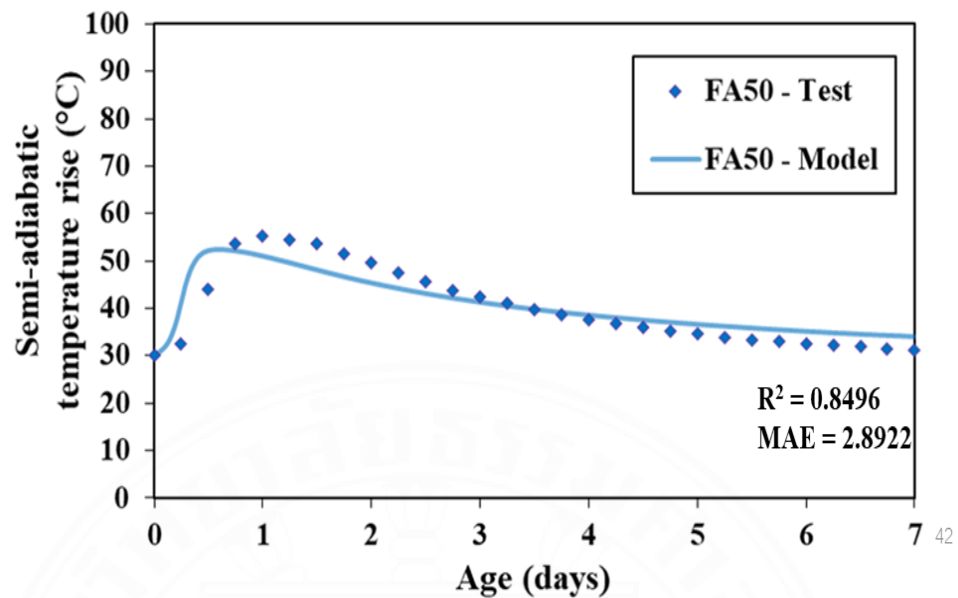


Figure 9.19 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50

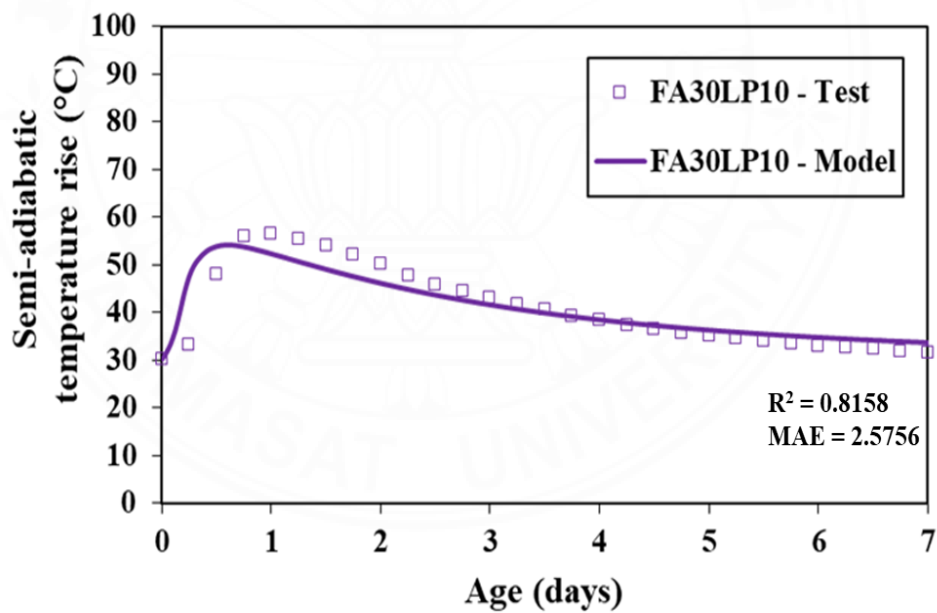


Figure 9.20 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA30LP10

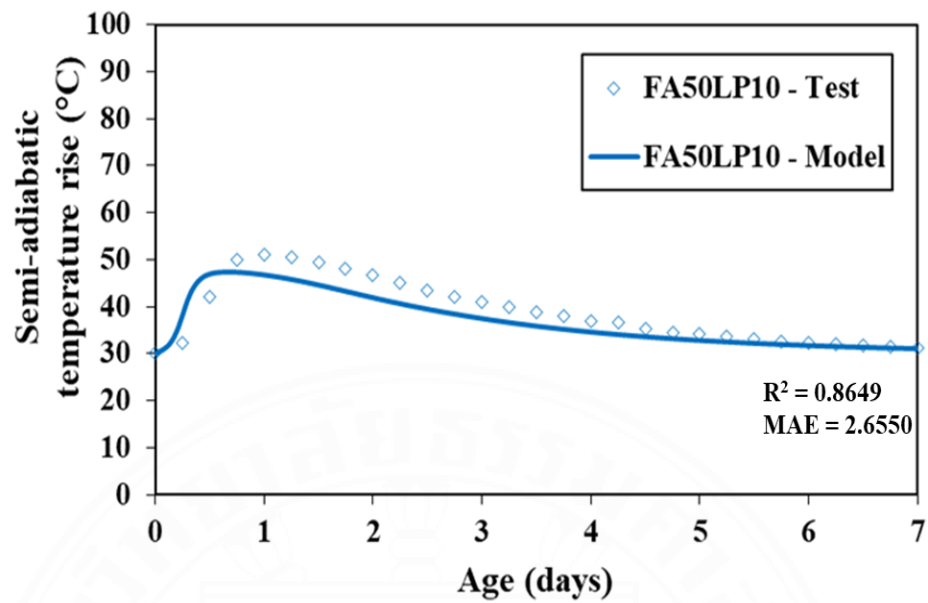


Figure 9.21 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50LP10

CHAPTER 10

EFFECTS OF THE MAXIMUM SIZE OF COARSE AGGREGATE ON TERNARY BINDER CONCRETE

General

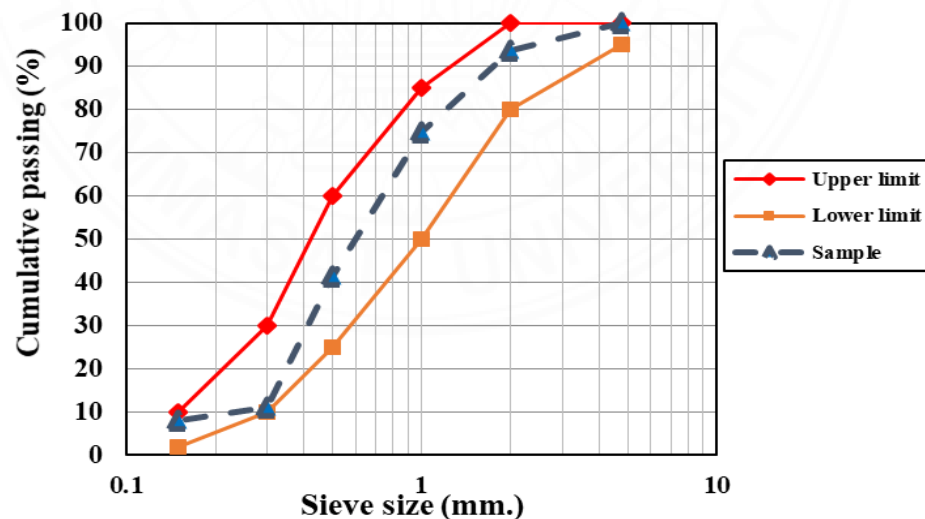
Coarse aggregate with two different maximum sizes (G_{max}) were used for mix proportion design to investigate the behaviors of concrete with the varying G_{max} . Cement content in the mixtures can be reduced according to the lower void content of the G_{max} 1 inch than that of the G_{max} $\frac{3}{4}$ inch. The latter is the normal G_{max} used in concrete in Thailand. The mix proportions according to the study of binder system in the previous chapter are selected based on the maximum temperature rise reduction efficiency to design mix proportions for the study of the effects of G_{max} in this chapter. The investigation of G_{max} 1 inch concrete and G_{max} $\frac{3}{4}$ inch concrete on slump and slump loss, compressive strength, tensile strain capacity, and temperature rise are performed with the analysis of experimental data.

10.1 Materials and mix proportions

The specifications of Ordinary Portland cement, Class C fly ash, limestone powder, and admixture type F follow the same standard as section 9.1 and Table 10.1 provides the chemical and physical properties of the materials. River sand with the gradation according to ASTM C33 as shown in Figure 10.1 is used with the specific gravity and absorption equal to 2.59 and 1.03, respectively.

Table 10.1 Chemical compositions and physical properties of tested materials

Chemical composition (%)	Cement	Fly ash	Limestone powder
SiO ₂	18.98	32.94	0.79
Al ₂ O ₃	5.33	16.68	0.42
Fe ₂ O ₃	3.16	14.04	0.23
CaO	65.34	26.06	54.66
MgO	1.15	3.37	0.59
SO ₃	2.65	2.67	N/A
LOI	2.54	0.7	43.21
Na ₂ O	0.16	1.78	0.02
K ₂ O	0.21	2.05	0.01
TiO ₂	0.29	0.35	0.01
P ₂ O ₅	0.06	0.26	0.03
MnO	0.05	0.14	0.01
SrO	0.03	0.15	N/A
Cr ₂ O ₃	0.01	N/A	N/A
ZnO	0.02	N/A	N/A
Physical Properties			
Specific gravity (g/cm ³)	3.05	2.56	2.77
Blaine fineness (cm ² /g)	3,141	2,500	5,220

**Figure 10.1** Particle size distribution of fine aggregate based on ASTM C33

Two gradations of crushed limestone coarse aggregate according to ASTM C33 are used in this study. The first gradation is ASTM C33 grade 7 as commonly used for concrete which contains G_{max} equals to $\frac{3}{4}$ inch as shown in Figure 10.2. The

second gradation is ASTM C33 grade 7 and grade 67 which contains G_{max} equals to 1 inch as shown in Figure 10.3. Specific gravity of the coarse aggregate is 2.76 and absorption is 0.58.

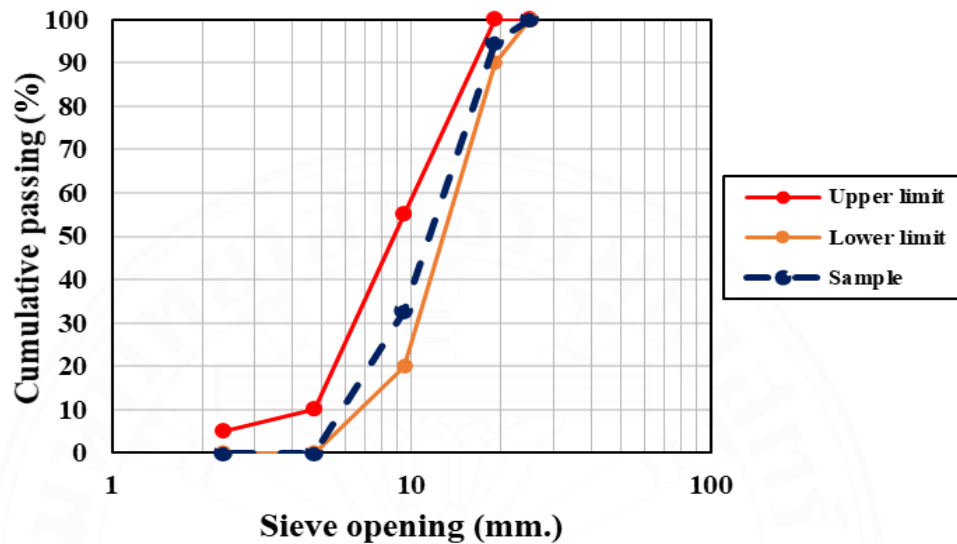


Figure 10.2 The particle size distribution of coarse aggregate with G_{max} equals to 1 inch based on ASTM C33 grade 67

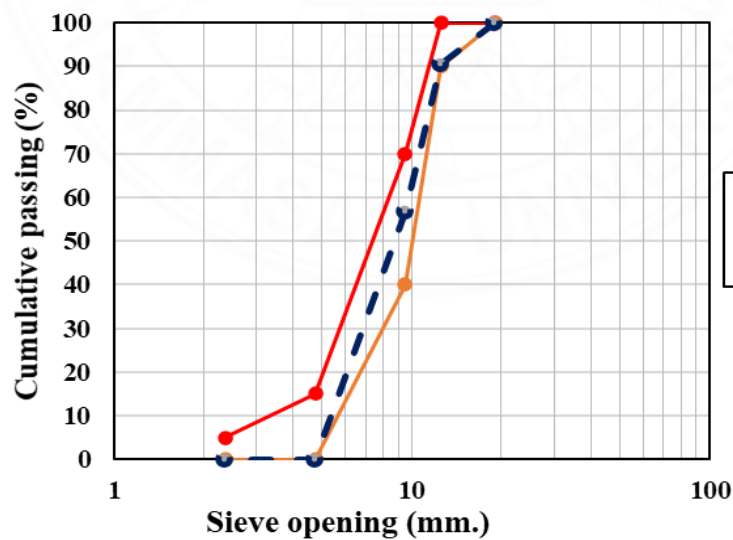


Figure 10.3 The particle size distribution of coarse aggregate with G_{max} equals to $\frac{3}{4}$ inch based on ASTM C33 grade 7

The mix proportions were selected by the consideration on the capability of the maximum temperature rise in concrete. As a consequence, two mixtures were selected. Fly ash replacing cement at replacement ratio of 0.5 of total binder which is binary binder and fly ash replacing cement at replacement ratio of 0.5 of total binder with limestone powder replacing cement at 0.1 of total binder for ternary binder are selected as the studied mix proportions to investigate the effects of G_{max} in concrete. Figure 10.4 displays the comparison of the upper and lower limits of the ASTM C33 grade 67 and ASTM C33 grade 7. It shows that ASTM C33 grade 67, G_{max} 1-inch, contains the higher maximum size of coarse aggregate and rougher gradation distribution than those with ASTM C33 grade 7, G_{max} $\frac{3}{4}$ -inch.

Based on the experiment on void content of aggregate referred to ASTM C29, coarse aggregate with G_{max} 1 inch and coarse aggregate with G_{max} $\frac{3}{4}$ inch shows the same sand to total aggregate ratio that gives the lowest void content. Coarse aggregate with G_{max} 1 inch gives the lower void content for all sand to total aggregate ratios compared to coarse aggregate with G_{max} $\frac{3}{4}$ inch as shown in Figure 10.5. To calculate mix proportion, void content is also one of the parameters affecting the amount of binder in the mixture. The studied mix proportions were designed based on the same paste content and the same ratio of sand to total aggregate with the different void content ratios. The lower void content from coarse aggregate with G_{max} 1 inch results in lower binder content consequently. Then, mix proportions were designed at the water to binder ratio of 0.4 together with the initial slump was controlled at 10 ± 2.5 cm. by the use of Kao-Mighty MX-T naphthalene-based water-reducing type F admixture according to ASTM C494. It can be seen that the lower amount of admixture was used with the lower cement content mixtures at the same G_{max} concrete and the concrete containing G_{max} 1 inch requires higher amount of admixture to control the same initial slump than the concrete containing G_{max} $\frac{3}{4}$ inch at the same replacement levels of fly ash and limestone powder.

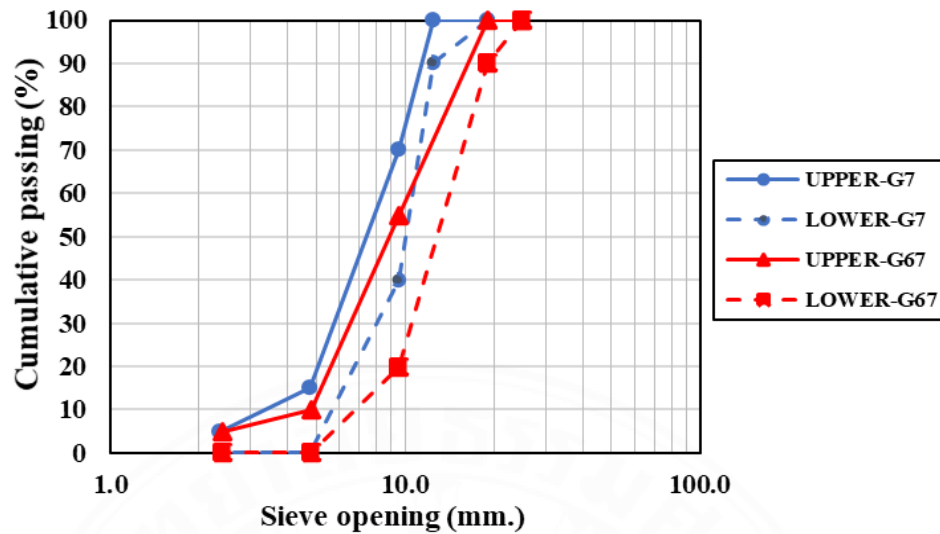


Figure 10.4 The comparison of upper and lower limit of coarse aggregate grade 67 and 7 based on ASTM C33

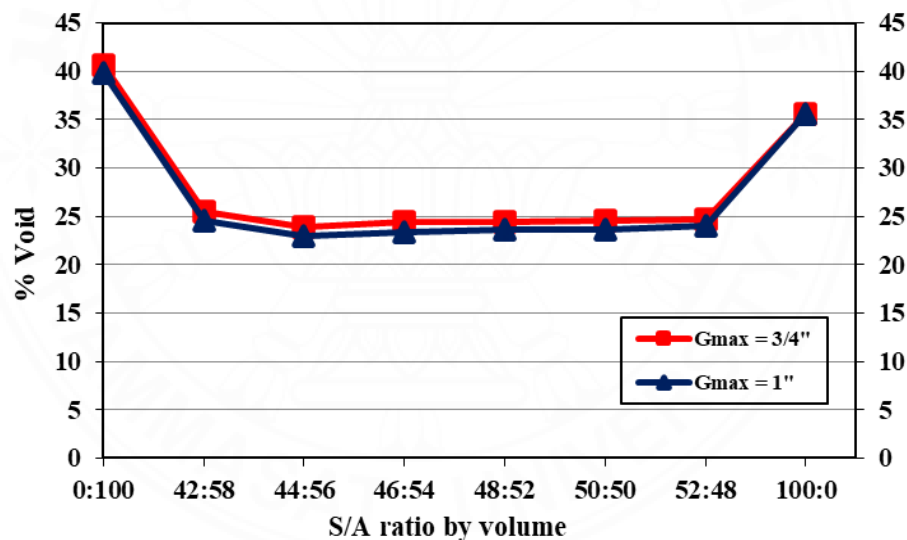


Figure 10.5 The comparison of void content between coarse aggregate with Gmax equal to $\frac{3}{4}$ inch and 1 inch

Finally, four mixtures are used as the studied mix proportions which are binary binder and ternary binder, as stated early, with two varied maximum sizes of coarse aggregate which are ASTM C33 grade 7 and grade 67 as shown in the Table 10.2. In the mix designation, “FA”, “LP”, “G1”, and “G3/4” refer to fly ash replacement ratio, limestone powder replacement ratio, Gmax equal to 1 inch, and Gmax equal to $\frac{3}{4}$ inch, respectively.

Table 10.2 Mix proportions of the tested concrete

Mix Designation	Cement (kg./m ³)	Fly ash (kg./m ³)	Limestone powder (kg./m ³)	Water (kg./m ³)	River sand (kg./m ³)	Crushed limestone (kg./m ³)	Admixture (kg./m ³)
G1 - FA50	192	192	0	154	799	1081	1.92
G $\frac{3}{4}$ - FA50	199	199	0	159	787	1064	0.80
G1 - FA50LP10	153	191	38	153	799	1081	1.34
G $\frac{3}{4}$ - FA50LP10	158	198	40	158	787	1064	0.40

10. 2 Experimental programs

The observation on the experiments which are slump and slump loss, compressive strength, tensile strain capacity, and semi-adiabatic temperature rise of Gmax 1 inch concrete was done and compared with the results of the Gmax $\frac{3}{4}$ inch concrete. The general information of each test has been stated in the earlier section.

10. 3 Experimental results

10.3.1 Slump and slump loss

Slump and slump loss of different tested mixtures are displayed in Figure 10.6 and shows that slump decreases with the increase of time. The results show that the slump loss of mixtures with limestone powder is faster than mixtures with no limestone powder for the same Gmax. The slump loss of mixtures with Gmax 1 inch is faster than those with Gmax $\frac{3}{4}$ inch when compared at the same fly ash or fly ash with limestone powder replacement ratio because of the lower binder content in the mixtures with higher Gmax.

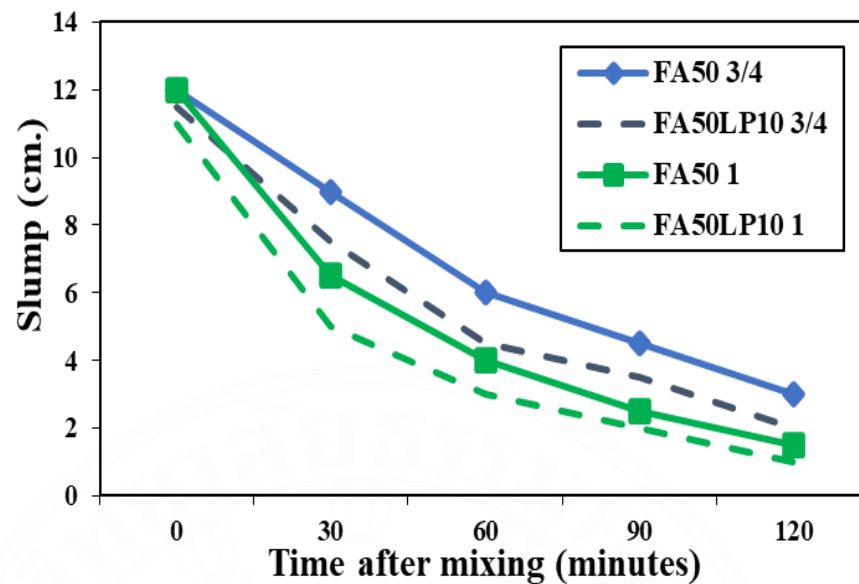


Figure 10.6 Initial slump and slump loss of concrete with FA50 and FA50LP10 with G_{max} $\frac{3}{4}$ inch and 1 inch at w/b ratio of 0.4

10.3.2 Compressive strength

Compressive strength of concrete with various mix proportions up to 91 days are shown in Figure 10.7. The figure shows that with the increase of time, the compressive strength of concrete increases. In the higher cement content mixtures, the compressive strength is higher than those with lower cement content when compared at the same G_{max} . At the same replacement levels of SCMs, the compressive strength of mixtures with G_{max} 1 inch are higher than those with G_{max} $\frac{3}{4}$ inch because of the lower paste content of G_{max} 1-inch concrete.

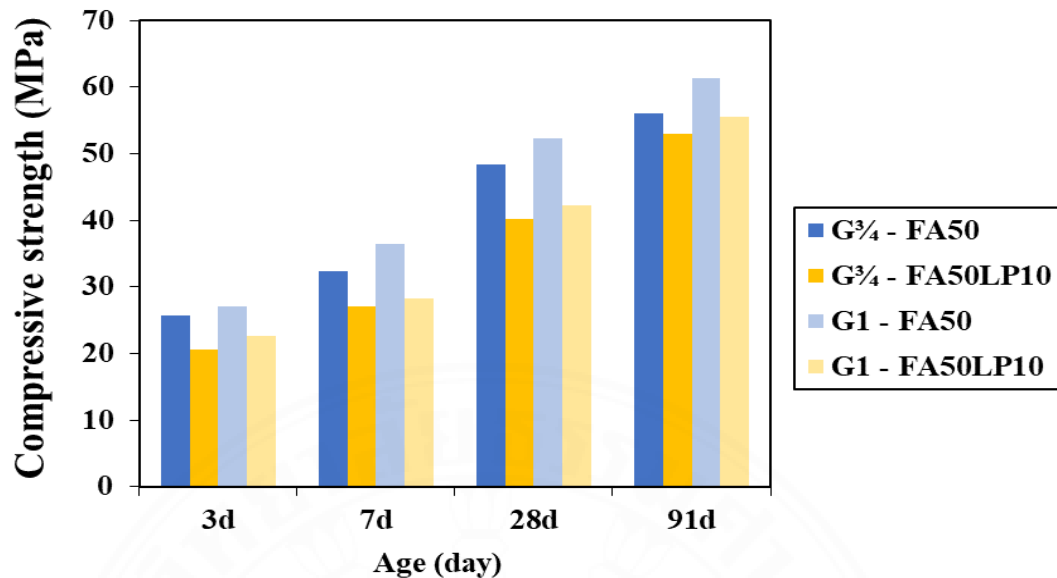


Figure 10.7 Compressive strength of concrete with FA50 and FA50LP10 with $G_{\max}$ $\frac{3}{4}$ inch and 1 inch at w/b ratio of 0.4

10.3.3 Tensile strain capacity

Experimental results of tensile strain capacity are shown in Figure 10.8 for the various mix proportions up to 28 days. The tensile strain capacity clearly increases with time for all tested specimens. The tensile strain capacity in higher cement content mixtures is higher than those with lower cement content for the comparison at the same maximum size of coarse aggregate. Moreover, $G_{\max}$ 1 inch concrete shows higher tensile strain capacity than $G_{\max}$ $\frac{3}{4}$ inch concrete when compared at the same fly ash or fly ash with limestone powder replacement level.

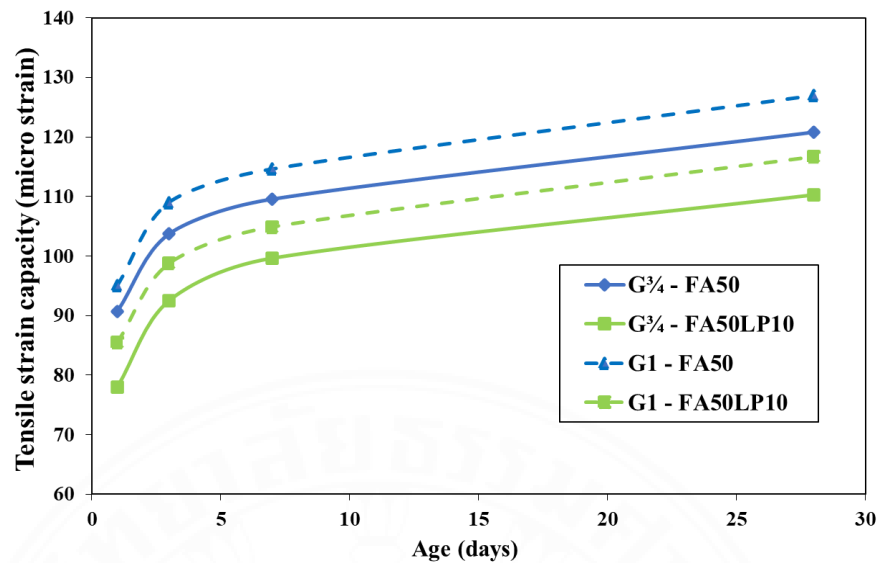


Figure 10.8 Tensile strain capacity of concrete with FA50 and FA50LP10 with G_{max} $\frac{3}{4}$ inch and 1 inch at w/b ratio of 0.4

10.3.4 Relationship between Compressive strength and Tensile strain capacity

Relationship between compressive strength and tensile strain capacity was also observed for the study of effects of G_{max} on ternary binder concrete. The relationship was illustrated as shown in Figure 10.9. The proportional relationship of compressive strength and tensile strain capacity is seen for both G_{max} 1 inch concrete and G_{max} $\frac{3}{4}$ inch concrete.

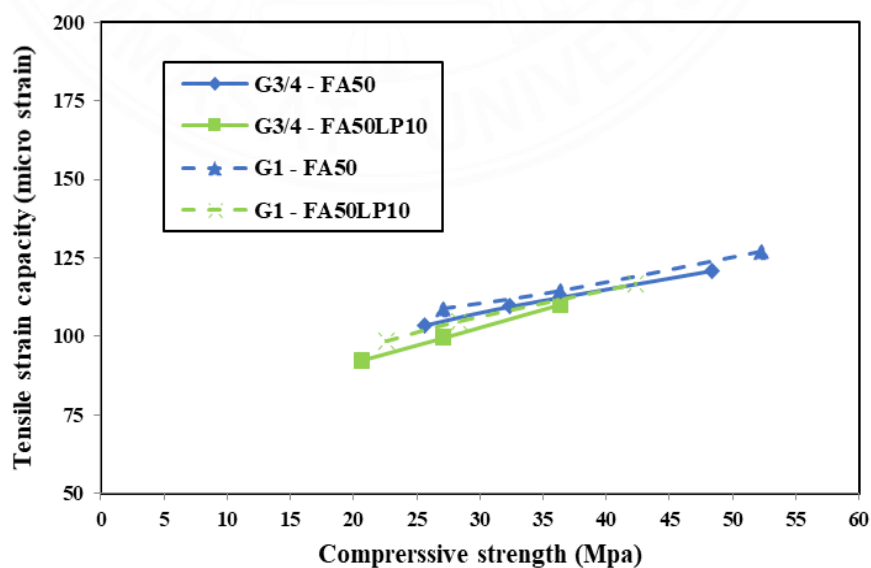


Figure 10.9 Relationship between compressive strength and tensile strain capacity with FA50 and FA50LP10 with G_{max} $\frac{3}{4}$ inch and 1 inch at w/b 0.4

10.3.5 Semi-Adiabatic temperature rise

Temperature rise profiles are exhibited in Figure 10.10. The figure shows that the maximum temperature rise in lower cement content mixtures is lower than those with the higher cement content when compared at the same G_{max} . The maximum temperature rise of mixtures with G_{max} 1 inch is lower than those with G_{max} $\frac{3}{4}$ inch because of the lower binder content in the mixtures.

As a result, the combination of ternary binder which is replacing cement with fly ash and limestone powder together with the use of G_{max} equal to 1 inch can be found effectively to reduce the maximum temperature rise in concrete.

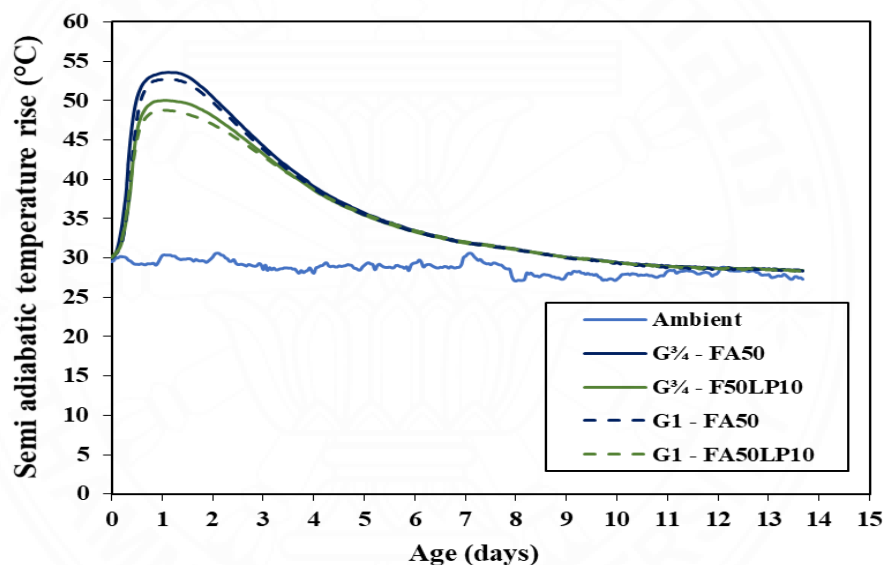


Figure 10.10 Semi-adiabatic temperature rise profiles of concrete with FA50 and FA50LP10 with G_{max} $\frac{3}{4}$ inch and 1 inch at w/b 0.4

10.3.6 Verification of the proposed model by semi-adiabatic temperature rise with FA50 and FA50LP10 with G_{max} equals to $\frac{3}{4}$ and 1 inch concrete

Comparison between temperature profiles which were generated from the proposed adiabatic temperature rise simulation model with commercial FEM and laboratory results of FA50 and FA50LP10 with G_{max} equal to $\frac{3}{4}$ and 1 inch concrete was done as shown in Figures 10.11 to 10.14. The accuracy of the proposed adiabatic temperature rise model prediction for ternary binder concrete was found to be satisfactory with the laboratory tested specimens.

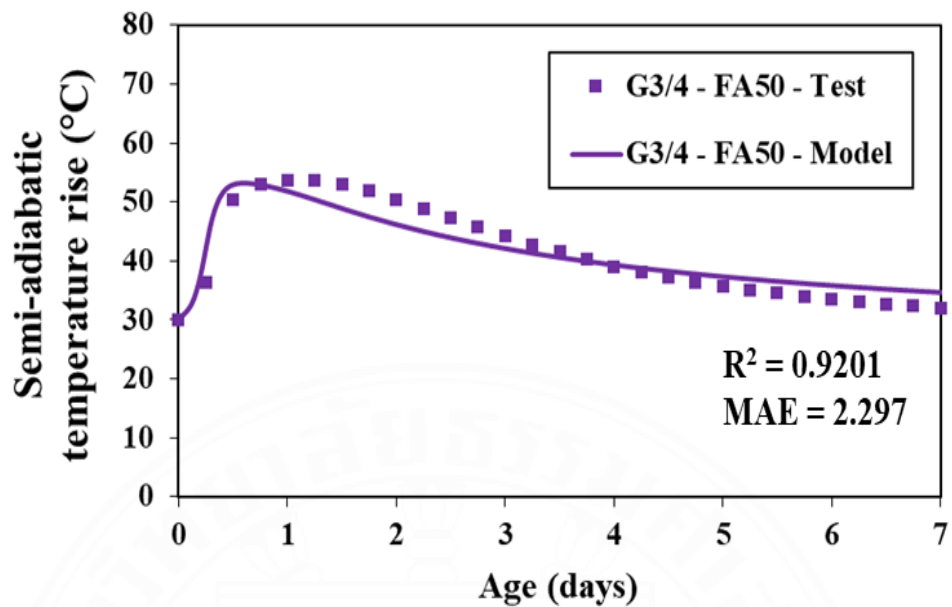


Figure 10.11 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50 with Gmax 3/4 inch at w/b 0.4

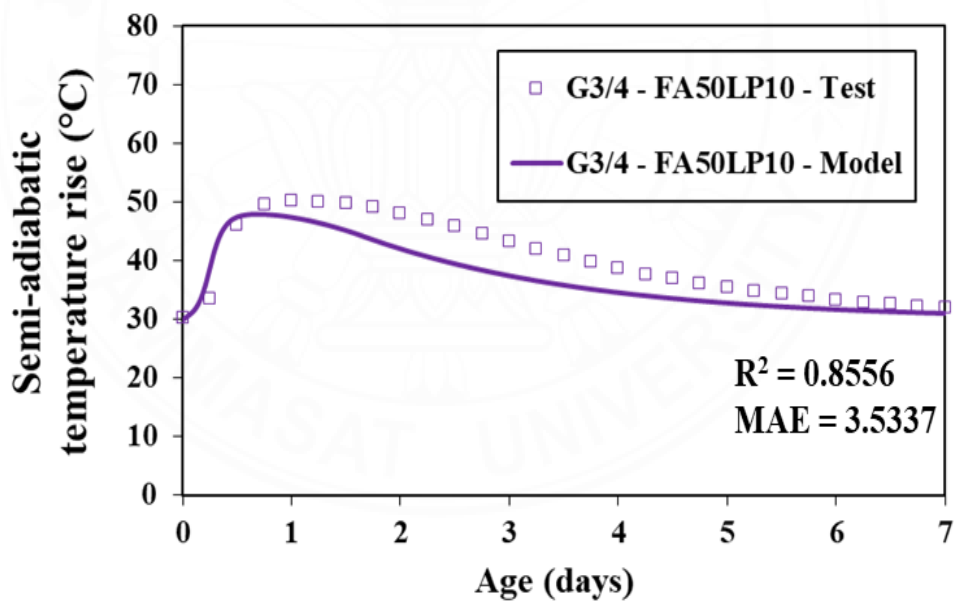


Figure 10.12 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50LP10 with Gmax 3/4 inch at w/b 0.4

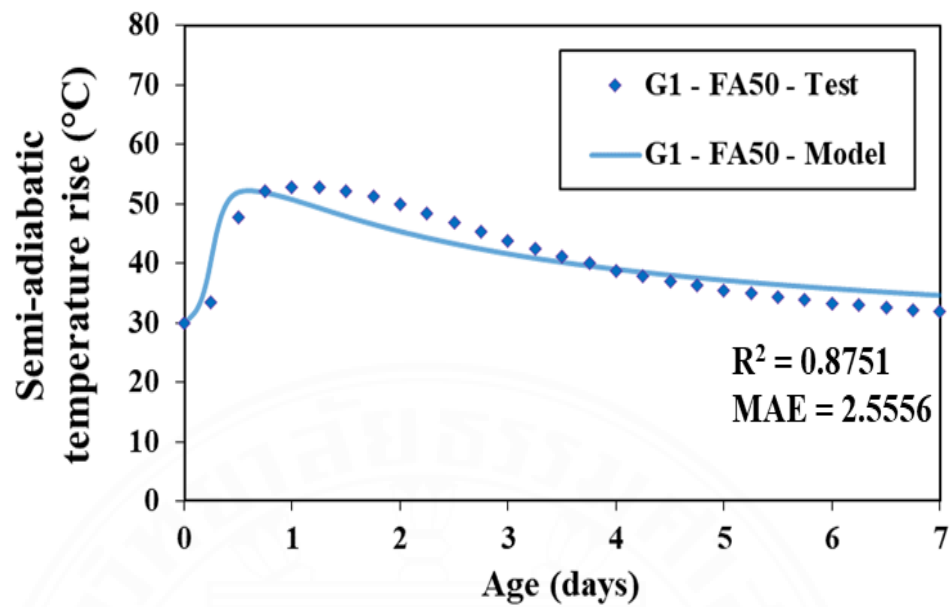


Figure 10.13 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50 with Gmax 1 inch at w/b 0.4

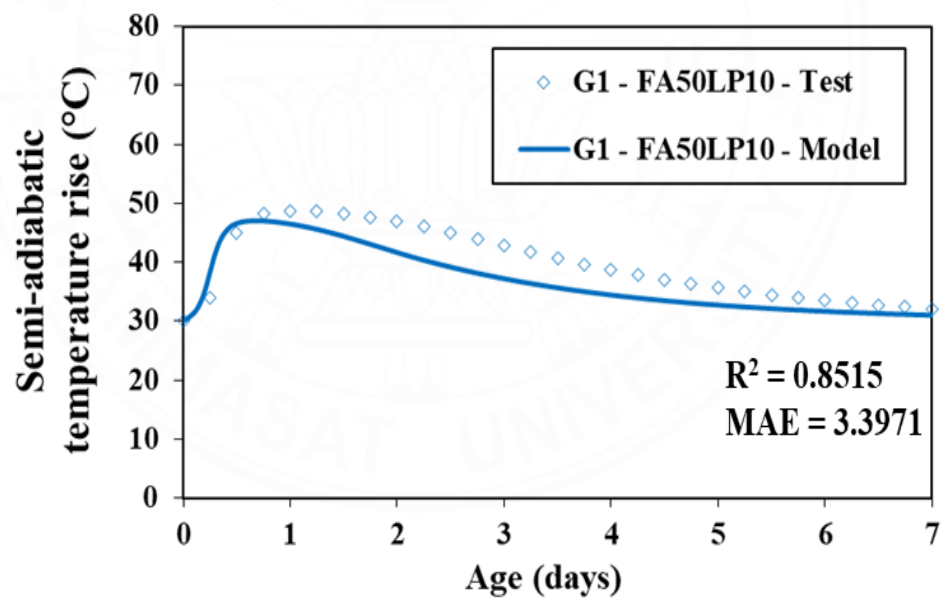


Figure 10.14 Comparison of semi-adiabatic temperature rise between model result and experimental result for concrete with FA50LP10 with Gmax 1 inch at w/b 0.4

CHAPTER 11

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

11.1 Conclusions

11.1.1 The study of binder system

1. Free water content prediction model and specific heat prediction model for ternary binder system were developed. Verification was done by the comparison between computerized results and experimental results. The proposed prediction models were found quantitatively satisfactory and able to capture the behavior of the laboratory results.

2. The prediction models of degree of hydration with the physical acceleration factor and total heat generation with the cumulative heat containing dispersion factor were considered with ternary binder assumption. The proposed models can be used to simulate average degree of hydration reaction and total heat generation for ternary binder system.

3. Thermal conductivity prediction model was proposed for ternary binder system and was proved to be able to simulate thermal conductivity for ternary binder concrete.

4. According to the study with binder system for the ternary concrete, the adiabatic temperature rise prediction model for ternary binder concrete was developed. The proposed prediction model can generate the adiabatic temperature rise profiles and were compared with the laboratory results for verification. It was found that the proposed model for ternary binder could be used to preliminarily predict the temperature rise of the tested specimens with satisfactory accuracies.

5. Amount of cement in the mixtures greatly affected the studied mechanical properties of concrete and temperature rise. Higher cement content mixtures give higher compressive strength, tensile strain capacity, and temperature rise.

6. Amount of admixture used to control the initial slump of fresh concrete was lower for the lower cement content mixtures. The faster rate of slump loss was

observed in the higher cement content mixtures and concrete containing limestone powder.

7. For compressive strength, the higher cement content mixtures show higher compressive strength at the test age up to 28 days except for the binary binder concrete with limestone powder which shows almost equal compressive strength with the cement-only mixture at the test age of 3 days. The concrete with 30% fly ash replacement shows the most improvement of compressive strength at 28 days.

8. Tensile strain capacity results show that the higher cement content mixtures yield the higher tensile strain capacity for tested concrete at the test ages of 1, 3, 7, and 28 days.

9. Both fly ash and limestone powder used as the cement replacing materials for binary binder system and ternary binder system can minimize the maximum temperature rise of concrete compared to the cement-only mixture. For the binary binder, higher fly ash replacing level can cause maximum temperature rise of concrete to be lower. The ternary binder concrete is reported to improve performance of concrete to reduce the maximum temperature rise, especially for concrete with 50% fly ash replacement and 10% limestone powder replacement which gave the lowest maximum temperature rise in concrete compared to the other tested mixtures.

11.1.2 The study of aggregate system

1. According to the experimental results, the lower binder content due to the use of larger G_{max} in concrete equal to 1 inch instead of $\frac{3}{4}$ inch gives positive effect on the compressive strength, tensile strain capacity, and temperature rise.

2. When compared at the same fly ash or fly ash with limestone powder replacement ratio, the G_{max} 1 inch concrete lost their slump faster than those with G_{max} $\frac{3}{4}$ inch concrete.

3. Compressive strength of concrete was improved when G_{max} 1-inch was used in concrete instead of G_{max} $\frac{3}{4}$ inch at the same fly ash or fly ash with limestone powder replacement ratio for all test ages.

4. G_{max} 1 inch concrete shows higher tensile strain capacity than G_{max} $\frac{3}{4}$ inch concrete for the test ages of 3, 7, and 28 days when compared at the same fly ash or fly ash with limestone powder replacement level.

5. Tensile strain capacity is proportional to compressive strength for all four mix proportions with concrete containing both G_{max} equals to 1 inch and $\frac{3}{4}$ inch.

6. G_{max} 1 inch concrete causes the lower maximum temperature rise when compared to G_{max} $\frac{3}{4}$ inch concrete at the same fly ash or fly ash with limestone powder replacement level. The study can be inferred that the use of ternary binder system concrete with G_{max} equal to 1 inch improves the performance in term of maximum temperature rise reduction in mass concrete.

11.2 Recommendations for future study

11.2.1 The study of binder system

1. According to the verification of free water content prediction model with experimental data, the overestimation of simulation results has been existed and need more investigation on both simulation model part and experimental results to clarify and increase the accuracy of the model prediction.

2. The fly ash-limestone powder interaction has been reported by the thermogravimetry technique. Further examination is needed for the improvement of total heat prediction model.

3. Some studies also report the effect of fineness of cementitious materials on heat development in the process of reaction. As a result, fineness of binders should be another parameter considered for dispersion factor simulation model.

4. As stated earlier, the interaction between fly ash and limestone powder existed so the effects of limestone powder on pozzolanic reaction is also required for further investigation and considered to be one of the parameters for pozzolanic reaction prediction model for ternary binder concrete.

5. According to the limitation of tests, the test results of thermal conductivity of ternary binder mixtures are still not provided. Experiments on thermal conductivity to investigate the effects of fly ash with limestone powder as cement replacing materials are necessary to verify the proposed thermal conductivity prediction model for ternary binder concrete.

6. Adiabatic temperature rise prediction model requires further modifications by considering more ranges of parameters.

7. The coefficient of thermal conductivity or CTE of ternary binder concrete should be investigated together with the restrained strain analyzing in mass concrete to evaluate the risk of cracking with tensile strain capacity.

11.2.2 The study of aggregate system

1. When using G_{max} 1 inch instead of G_{max} $\frac{3}{4}$ inch for ternary binder concrete, void content and total binder content is reduced. The negative properties of concrete may occur as shown in the experimental result on slump loss. Therefore, it is required to investigate for other concrete properties to achieve the maximized benefits of concrete containing G_{max} 1 inch.

2. According to the amount of admixture used to control the initial slump, it was reported that the G_{max} 1-inch concrete required higher amount of admixture than those with G_{max} $\frac{3}{4}$ -inch concrete and this may cause concrete containing G_{max} 1 inch to be more expensive than those with the normal concrete containing G_{max} $\frac{3}{4}$ inch. As a result, the cost analysis should be done with the real condition for the benefit optimization.

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The seal of Thammasat University is a large, faint, circular watermark in the background. It features a central emblem with a crown on top, a lotus flower in the middle, and a base with Thai script. The words "THAMMASAT UNIVERSITY" are written in English around the bottom half of the circle, and Thai script is written around the top half.

APPENDICES

APPENDIX A

FREE WATER CONTENT RESULTS

Table A.1 Free water content of cement pastes (w/b = 0.25) at 0, 1, 3, 7, and 28 days

Age (days)	w25opc	w25fa30	w25fa50	w25lp10	w25fa30lp10	w25fa50lp10
0	0.2500	0.2500	0.2500	0.2500	0.2500	0.2500
1	0.0961	0.1410	0.1795	0.1295	0.1706	0.1996
3	0.0751	0.1266	0.1477	0.0983	0.1410	0.1619
7	0.0719	0.1187	0.1288	0.0948	0.1247	0.1463
28	0.0664	0.0929	0.0864	0.0826	0.0995	0.0960

Table A.2 Free water content of cement pastes (w/b = 0.40) at 0, 1, 3, 7, and 28 days

Age (days)	w40opc	w40fa30	w40fa50	w40lp10	w40fa30lp10	w40fa50lp10
0	0.4000	0.4000	0.4000	0.4000	0.4000	0.4000
1	0.2432	0.2936	0.3151	0.2714	0.3071	0.3355
3	0.1984	0.2494	0.2844	0.2215	0.2694	0.3084
7	0.1853	0.2349	0.2614	0.2036	0.2490	0.2940
28	0.1593	0.1850	0.1841	0.1759	0.2061	0.1947

APPENDIX B

VERIFICATION BETWEEN MODEL RESULTS AND EXPERIMENTAL RESULTS OF FREE WATER CONTENT

Table B.1 Experimental and model results of free water content of cement pastes
(w/b = 0.25 and 0.40) at 0, 1, 3, 7, and 28 days

w/b	Age (days)	Experimental result		Model result	
		FA30LP10	FA50LP10	FA30LP10	FA50LP10
0.25	0	0.2500	0.2500	0.2500	0.2500
	1	0.1706	0.1996	0.1575	0.1893
	3	0.1410	0.1619	0.1347	0.1636
	7	0.1247	0.1463	0.1258	0.1462
	28	0.0995	0.0960	0.1087	0.1093
0.40	0	0.4000	0.4000	0.4000	0.4000
	1	0.3071	0.3355	0.3127	0.3464
	3	0.2694	0.3084	0.2714	0.3073
	7	0.2490	0.2940	0.2508	0.2786
	28	0.2061	0.1947	0.2108	0.2082

APPENDIX C

SPECIFIC HEAT RESULTS

Table C.1 Specific heat of cement pastes (w/b = 0.25) at 1, 3, 7, and 28 days

Age (days)	w25opc	w25fa30	w25fa50	w25lp10	w25fa30lp10	w25fa50lp10
1	0.215348	0.228187	0.241881	0.269107	0.259455	0.284964
3	0.178085	0.204206	0.22598	0.253279	0.241131	0.267104
7	0.17563	0.201711	0.209	0.239205	0.228721	0.252293
28	0.167609	0.198000	0.19032	0.193441	0.213217	0.231499

Table C.2 Specific heat of cement pastes (w/b = 0.40) at 1, 3, 7, and 28 days

Age (days)	w40opc	w40fa30	w40fa50	w40lp10	w40fa30lp10	w40fa50lp10
1	0.272985	0.301338	0.318028	0.343409	0.333492	0.370222
3	0.23923	0.269179	0.300108	0.334212	0.317526	0.357273
7	0.225212	0.255285	0.275537	0.311758	0.298492	0.331009
28	0.209551	0.249999	0.229915	0.244472	0.272355	0.281608

APPENDIX D

VERIFICATION BETWEEN MODEL RESULTS AND EXPERIMENTAL RESULTS OF SPECIFIC HEAT

Table D.1 Experimental and model results for specific heat of cement pastes
(w/b = 0.25 and 0.40) at 1, 3, 7, and 28 days

Water to binder	Age (days)	Experimental result		Model result	
		FA30LP10	FA50LP10	FA30LP10	FA50LP10
0.25	1	0.2595	0.2850	0.2688	0.2967
	3	0.2411	0.2671	0.2482	0.2743
	7	0.2287	0.2523	0.2407	0.2604
	28	0.2132	0.2315	0.2266	0.2313
0.40	1	0.3335	0.3702	0.3489	0.3767
	3	0.3175	0.3573	0.3144	0.3450
	7	0.2985	0.3310	0.2980	0.3235
	28	0.2724	0.2816	0.2673	0.2726

APPENDIX E

THE RESULTS OF THE PROPOSED THERMAL CONDUCTIVITY PREDICTION MODEL

Table E.1 Model results for thermal conductivity of concrete (w/b = 0.40)

Age (days)	Thermal conductivity (kcal/m hr °C)					
	OPC	FA30	FA50	LP10	FA30LP10	FA50LP10
0	2.0370	2.0047	1.9840	2.0513	2.0342	2.0134
1	2.0476	2.0137	1.9920	2.1253	2.0417	2.0204
2	2.0481	2.0171	1.9967	2.1296	2.0445	2.0246
3	2.0483	2.0200	2.0011	2.1312	2.0469	2.0283
4	2.0485	2.0224	2.0051	2.1321	2.0491	2.0316
5	2.0486	2.0244	2.0086	2.1327	2.0509	2.0345
6	2.0486	2.0259	2.0116	2.1331	2.0525	2.0369
7	2.0487	2.0272	2.0141	2.1334	2.0537	2.0391
8	2.0487	2.0283	2.0162	2.1337	2.0548	2.0409
9	2.0487	2.0291	2.0180	2.1339	2.0557	2.0424
10	2.0488	2.0298	2.0195	2.1341	2.0564	2.0438
11	2.0488	2.0304	2.0208	2.1343	2.0570	2.0449
12	2.0488	2.0309	2.0219	2.1344	2.0575	2.0459
13	2.0488	2.0313	2.0228	2.1345	2.0580	2.0468
14	2.0488	2.0317	2.0236	2.1347	2.0584	2.0476
15	2.0488	2.0320	2.0243	2.1348	2.0587	2.0483
16	2.0489	2.0323	2.0250	2.1348	2.0590	2.0489
17	2.0489	2.0326	2.0255	2.1349	2.0593	2.0495
18	2.0489	2.0328	2.0260	2.1350	2.0595	2.0499
19	2.0489	2.0330	2.0265	2.1351	2.0598	2.0504
20	2.0489	2.0332	2.0269	2.1351	2.0600	2.0508
21	2.0489	2.0333	2.0272	2.1352	2.0601	2.0512
22	2.0489	2.0335	2.0275	2.1352	2.0603	2.0515
23	2.0489	2.0336	2.0278	2.1353	2.0604	2.0518
24	2.0489	2.0338	2.0281	2.1353	2.0606	2.0521
25	2.0489	2.0339	2.0284	2.1354	2.0607	2.0524
26	2.0489	2.0340	2.0286	2.1354	2.0608	2.0526
27	2.0489	2.0341	2.0288	2.1355	2.0609	2.0528
28	2.0490	2.0342	2.0290	2.1355	2.0610	2.0530

APPENDIX F

RESULTS OF SLUMP AND SLUMP LOSS IN SECTION 9.3.1

Table F.1 Slump and slump loss results of concrete with OPC, FA30, FA50, LP10, FA30LP10, and FA50LP10 at water to binder 0.4

Mixture	Time after mixing (minutes)				
	0	30	60	90	120
OPC (cm.)	8.5	3.5	2.0	0	0
FA30 (cm.)	11.5	9.0	5.5	4.0	2.5
FA50 (cm.)	12.5	10.5	8.5	6.0	4.0
LP10 (cm.)	8.0	2.0	1.0	0	0
FA30LP10 (cm.)	9.5	4.5	2.5	2.0	0.5
FA50LP10 (cm.)	9.5	7.5	5.0	3.0	2.0



APPENDIX G

RESULTS OF COMPRESSIVE STRENGTH IN SECTION 9.3.2

Table G.1 Compressive strength of concrete (w/b = 0.40) at 3, 7, 28, and 91 days

Compressive strength (MPa)				
Mixture	3 days	7 days	28 days	91 days
OPC	43.11	50.54	51.83	54.67
FA30	35.08	40.76	53.77	61.75
FA50	27.06	33.56	47.71	49.98
LP10	43.63	46.92	50.52	52.37
FA30LP10	31.59	37.09	48.25	52.83
FA50LP10	22.54	28.11	43.97	47.75

APPENDIX H

RESULTS OF TENSILE STRAIN CAPACITY IN SECTION 9.3.3

Table H.1 Tensile strain capacity of concrete ($w/b = 0.40$) at 1, 3, 7, and 28 days

Tensile strain capacity (micro strain)				
Mixture	1 day	3 days	7 days	28 days
OPC	126.78	143.96	183.79	188.54
FA30	106.65	120.19	146.59	167.15
FA50	92.78	107.64	116.89	130.42
LP10	117.88	131.86	172.03	176.98
FA30LP10	99.06	115.24	130.94	143.63
FA50LP10	85.52	92.12	100.71	121.18

APPENDIX I

RESULTS OF SEMI-ADIABATIC TEMPERATURE RISE IN SECTION 9.3.5

Table I.1 Test results of semi-adiabatic temperature of cement-only concrete and fly ash concrete (w/b = 0.40)

Age (days)	Ambient (°C)	OPC (°C)	FA30 (°C)	FA50 (°C)
0	29.55	30.10	30.45	30.20
1	29.22	66.88	60.66	55.29
2	29.32	53.74	52.23	49.62
3	29.65	44.19	44.01	42.36
4	30.12	38.58	38.71	37.59
5	29.23	35.15	35.40	34.52
6	29.12	32.93	33.12	32.53
7	29.21	31.58	31.71	31.13
8	28.95	30.68	30.82	30.34
9	28.61	30.08	30.13	29.76
10	29.08	29.72	29.67	29.47
11	28.95	29.58	29.51	29.36
12	29.82	29.42	29.44	29.26
13	29.79	29.50	29.48	29.35
14	29.24	29.50	29.47	29.39

Table I.2 Test results of semi-adiabatic temperature of binary concrete with limestone powder and ternary concrete (w/b = 0.40)

Age (days)	LP10 (°C)	FA30LP10 (°C)	FA50LP10 (°C)
0	30.40	30.10	30.00
1	64.04	56.49	51.12
2	52.46	50.09	46.59
3	43.59	42.99	40.96
4	38.19	38.39	36.88
5	34.96	35.16	34.25
6	32.67	33.00	32.24
7	31.39	31.57	31.15
8	30.52	30.69	30.36
9	29.94	30.08	29.86
10	29.59	29.70	29.55
11	29.44	29.39	29.36
12	29.32	29.40	29.29
13	29.40	29.38	29.39
14	29.37	29.31	29.37

APPENDIX J

VERIFICATION BETWEEN MODEL RESULTS AND EXPERIMENTAL RESULTS OF SEMI-ADIABATIC TEMPERATURE RISE IN SECTION 9.3.6

Table J.1 Experiment results and model results of semi-adiabatic temperature of cement-only concrete and fly ash concrete (w/b = 0.40)

Age(days)	OPC (°C)		FA30 (°C)		FA50 (°C)	
	Model	Experiment	Model	Experiment	Model	Experiment
0	30.00	30.10	30.00	30.45	30.00	30.20
1	67.66	66.88	59.24	60.66	51.13	55.29
2	54.05	53.74	50.78	52.23	45.45	49.62
3	44.92	44.19	44.61	44.01	41.38	42.36
4	39.22	38.58	40.26	38.71	38.65	37.59
5	35.73	35.15	37.31	35.40	36.71	34.52
6	33.59	32.93	35.28	33.12	35.23	32.53
7	32.28	31.58	33.87	31.71	34.11	31.13

Table J.2 Experiment results and model results of semi-adiabatic temperature rise of binary concrete with limestone powder and ternary concrete (w/b = 0.40)

Age(days)	LP10 (°C)		FA30LP10 (°C)		FA50LP10 (°C)	
	Model	Experiment	Model	Experiment	Model	Experiment
0	30.00	30.40	30.00	30.10	30.00	30.00
1	68.42	64.04	52.28	56.49	46.80	51.12
2	55.24	52.46	46.10	50.09	41.94	46.59
3	46.04	43.59	41.57	42.99	37.51	40.96
4	40.11	38.19	38.42	38.39	34.64	36.88
5	36.40	34.96	36.26	35.16	32.87	34.25
6	34.09	32.67	34.73	33.00	31.77	32.24
7	32.64	31.39	33.63	31.57	31.09	31.15

APPENDIX K

RESULTS OF SLUMP AND SLUMP LOSS IN SECTION 10.3.1

Table K.1 Slump and slump loss of concrete with water to binder of 0.40 (Effect of G_{max} ; $\frac{3}{4}$ inch and 1 inch)

Mixture	Time after mixing (minutes)				
	0	30	60	90	120
G $\frac{3}{4}$ - FA50 (cm.)	12	9	6	4.5	3
G1 - FA50 (cm.)	12	6.5	4	2.5	1.5
G $\frac{3}{4}$ - FA50LP10 (cm.)	11.5	7.5	4.5	3.5	2
G1 - FA50LP10 (cm.)	11	5	3	2	1



APPENDIX L

RESULTS OF COMPRESSIVE STRENGTH IN SECTION 10.3.2

Table L.1 Compressive strength of concrete with water to binder of 0.40 at 3, 7, 28, and 91 days (Effect of Gmax; $\frac{3}{4}$ inch and 1 inch)

Compressive strength (MPa)				
Mixture	3 days	7 days	28 days	91 days
G $\frac{3}{4}$ - FA50	25.65	32.40	48.40	56.06
G $\frac{3}{4}$ - FA50LP10	20.63	27.07	40.23	52.89
G1 - FA50	27.11	36.39	52.29	61.29
G1 - FA50LP10	22.64	28.33	42.29	55.57

APPENDIX M

RESULTS OF TENSILE STRAIN CAPACITY IN

SECTION 10.3.3

Table M.1 Tensile strain capacity of concrete with water to binder of 0.40 at 1, 3, 7, and 28 days (Effect of Gmax; $\frac{3}{4}$ inch and 1 inch)

Tensile strain capacity (micro strain)				
Mixture	1 day	3 days	7 days	28 days
G $\frac{3}{4}$ - FA50	91	104	110	121
G $\frac{3}{4}$ - FA50LP10	78	92	100	110
G1 - FA50	95	109	115	127
G1 - FA50LP10	86	99	105	117

APPENDIX N

RESULTS OF SEMI-ADIABATIC TEMPERATURE RISE IN

SECTION 10.3.5

Table N.1 Results of semi-adiabatic temperature of concrete with water to binder of 0.40 (Effect of Gmax; $\frac{3}{4}$ inch and 1 inch)

Age (days)	Ambient (°C)	G $\frac{3}{4}$ - FA50 (°C)	G $\frac{3}{4}$ - F50LP10 (°C)	G1 – FA50 (°C)	G1 – FA50LP10 (°C)
0	29.50	30.00	30.17	29.93	30.06
1	30.26	53.56	50.08	52.74	48.78
2	30.08	50.48	48.10	49.88	46.96
3	28.52	44.26	43.27	43.82	42.87
4	28.07	39.08	38.65	38.81	38.66
5	29.06	35.72	35.43	35.49	35.62
6	29.35	33.47	33.31	33.32	33.51
7	30.18	32.02	31.89	31.86	32.08
8	27.10	31.13	31.05	31.04	31.14
9	27.89	30.09	30.04	29.99	30.12
10	27.35	29.43	29.40	29.40	29.49
11	28.33	29.01	28.87	28.78	29.11
12	28.22	28.67	28.65	28.61	28.64
13	27.88	28.62	28.57	28.54	28.52

APPENDIX O

VERIFICATION BETWEEN MODEL RESULTS AND EXPERIMENTAL RESULTS OF SEMI-ADIABATIC TEMPERATURE RISE IN SECTION 10.3.6

Table O.1 Experiment results and model results of semi-adiabatic temperature of FA50 and FA50LP10 with Gmax $\frac{3}{4}$ inch concrete (w/b = 0.40)

Age(days)	G $\frac{3}{4}$ - FA50 (°C)		G $\frac{3}{4}$ - F50LP10 (°C)	
	Model	Experiment	Model	Experiment
0	30.00	30.00	30.00	30.17
1	51.87	53.56	47.31	50.08
2	46.22	50.48	42.00	48.10
3	42.14	44.26	37.45	43.27
4	39.36	39.08	34.61	38.65
5	37.36	35.72	32.85	35.43
6	35.86	33.47	31.76	33.31
7	34.68	32.02	31.09	31.89

Table O.2 Experiment results and model results of semi-adiabatic temperature rise of FA50 and FA50LP10 with Gmax 1 inch concrete (w/b = 0.40)

Age(days)	G1 – FA50 (°C)		G1 – FA50LP10 (°C)	
	Model	Experiment	Model	Experiment
0	30.00	29.93	30.00	30.06
1	50.62	52.74	46.58	48.78
2	45.28	49.88	41.73	46.96
3	41.52	43.82	37.28	42.87
4	38.96	38.81	34.50	38.66
5	37.12	35.49	32.78	35.62
6	35.72	33.32	31.72	33.51
7	34.58	31.86	31.06	32.08

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