Final Report

Research Title: The Development of Concrete with High Resistance to Shrinkage Crack and Evaluation of Restraint Degree (Contract No: RSA5680018)

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Project Duration: 3 Years

Research supported by Thailand Research Fund (TRF) and King Mongkut's University of Technology Thonburi (KMUTT)

Abstract

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The aim of this study is to enhance a technology to prevent shrinkage cracking in concrete structures. The content of this work is divided into two parts. The first part is the development of concrete with high cracking resistance. Concrete improved with various technological concepts are produced and experimentally investigated. It was found that many of the techniques are promising for the prevention of shrinkage crack. Use of bottom ash as an internal curing agent is one of effective method to reduce autogenous shrinkage of concrete. However, in such case, the increase of drying shrinkage may be experience. Use of superabsorbent polymer (SAP) is another way to reduce shrinkage of concrete. Properly-selected type of SAP can absorb water and release water to compensate with moisture reduction in cement paste. Expansive additive can be used to generate suitable amount of expansion during the early age of concrete. The required amount of expansive additive can be conveniently estimated from the relationship between the restrained expansion and the dosage of expansive additive. The combination of expansive additive with Thai fly ash creates more expansion of concrete. This indicates a possibility of more cost-efficient mix proportion design for crack prevention. In the second part, field survey and investigation on degree of restraint is performed. From the site survey, it is found that the careless selection of materials and inappropriate design of mix proportion is one of major causes of shrinkage cracks. Most of shrinkage crack can be prevented with the conventional materials. The degree of restraint can be estimated. The value of degree of restraint can be used for the design against shrinkage cracking of concrete structure.

Keywords : Concrete, Shrinkage, Cracks, Internal Curing, Admixture, Restraint

บทคัดย่อ

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ชื่อโครงการ : การพัฒนาคอนกรีตที่มีความต้านทานการแตกร้าวสูงและการประเมินระดับการยึดรั้ง

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ระยะเวลาโครงการ : 3 years

เป้าประสงค์ของการศึกษานี้คือการปรับปรุงเทคโนโลยีในการป้องกันรอยร้าวเนื่องจากการหดตัวในโครงสร้างคอนกรีต เนื้อหาของงานวิจัยนี้แบ่งเป็นสองส่วน ส่วนแรกคือการพัฒนาคอนกรีตที่มีความสามารถต้านทานการแตกร้าวได้ดี คอนกรีตที่ได้รับการพัฒนาจากเทคโนโลยีหลายๆ แบบได้รับการประเมินด้วยการทดสอบในห้องปฏิบัติการ จากการทดสอบพบว่าเทคนิคหลายๆ อย่างเป็นวิธีการที่มีศักยภาพในการป้องกันรอยร้าวจากการหดตัว การผสมเถ้ากันเตาเพื่อเป็นสารบ่มภายในคอนกรีตช่วยให้สามารถลดการหดตัวแบบออโตจีนัสของคอนกรีตได้ดี อย่างไรก็ตาม ในการใช้เทคโนโลยีดังกล่าวนั้นจำเป็นต้องระมัดระวังว่าการหดตัวแบบแห้งอาจจะเพิ่มขึ้นได้ นอกจากนี้ การใช้ Superabsorbent Polymer (SAP) ก็เป็นอีกวิธีหนึ่งที่สามารถช่วยลดการหดตัวของคอนกรีตได้ SAP ที่เหมาะสมจะสามารถดูดซับน้ำและคายออกมาเพื่อรักษาความชื้นในซีเมนต์เพสต์ สารผสมเพิ่มเพื่อก่อการขยายตัวก็ช่วยให้เกิดการขยายตัวที่เหมาะสมในช่วงอายุต้น ซึ่งจะชดเชยกับการหดตัวที่เกิดขึ้นตามมา ทั้งนี้เราสามารถประเมินปริมาณสารก่อการขยายตัวที่จำเป็นในการป้องกันการแตกร้าวได้จากความสัมพันธ์ระหว่างปริมาณสารก่อการขยายตัวและการขยายตัวภายใต้การยึดรั้ง นอกจากนี้ยังพบว่าการใช้สารก่อการขยายตัวร่วมกับเถ้าลอยไทยสามารถทำให้เกิดการขยายตัวได้มากขึ้น ซึ่งเป็นประโยชน์ในการพัฒนาคอนกรีตที่ต้านทานการแตกร้าวได้ดีและราคาไม่แพงจนเกินไป ในการศึกษาส่วนที่สองเป็นการสำรวจโครงสร้างคอนกรีตจริงที่แตกร้าวเนื่องจากการหดตัวและการวิเคราะห์ระดับการยึดรั้ง ซึ่งพบว่าการเลือกวัสดุผสมคอนกรีตที่ไม่ระมัดระวังและการออกแบบที่ไม่เหมาะสมเป็นสาเหตุหลักของการแตกร้าวเนื่องจากการหดตัวในโครงสร้าง การแตกร้าวที่พบในโครงสร้างคอนกรีตโดยส่วนใหญ่นั้นสามารถที่จะป้องกันได้โดยใช้ส่วนผสมของคอนกรีตที่ใช้อยู่โดยทั่วไป นอกจากนี้ในการศึกษานี้ยังการประเมินระดับการยึดรั้งในโครงสร้าง ซึ่งสามารถนำไปใช้ประโยชน์ในการออกแบบโครงสร้างเพื่อป้องกันการแตกร้าวเนื่องจากการหดตัวได้ดียิ่งขึ้น

คำหลัก: คอนกรีต, การหดตัว, รอยร้าว, การบ่มภายใน, สารผสมเพิ่ม, การยึดรั้ง

Chapter 1: Introduction

1.1 Background

Cracking of concrete structures is an extensively-found problem in construction industry. The cracks may be caused by different reasons. Restrained shrinkage is one of the widely found causes of cracking in Thailand. The shrinkage cracks, once take place, not only adversely affect load carrying capacity but also downgrade durability, serviceability, and appearance of the concrete structures.

The cracking due to shrinkage is a complicated problem because the shrinkage is dependent on many factors; for instance, mix proportion, type of raw materials, environment, curing method, geometry of the concrete structures, and restraining condition. Prevention of the shrinkage cracks therefore needs careful operations in all stages of design and construction. The shrinkage crack problem cannot be solved only by minimizing the shrinkage of concrete, but the good geometry, good control of restraint (such as an arrangement of reinforcing bar or the provision of joints), as well as good curing are also necessary.

This study is aimed to develop some technologies that can be employed to solve the shrinkage cracking problems. This study is divided into two main parts. The first part is the development of various types of concrete that has high resistance to shrinkage cracking and can be produced at reasonable cost. The second part is to investigate restraining conditions (in terms of degree of restraint) of different types of structural element especially the thin members restrained by different adjacent structural elements or internal reinforcement.

In order to obtain the concrete with high resistance to shrinkage cracking, not only low shrinkage but also high cracking strain capacity of concrete must be obtained. Selection of raw materials and optimum design of mix proportion will be conducted in order to produce concrete with high resistance to shrinkage crack. The performance of concrete will be evaluated by experimental works as well as few field tests.

Control of restraining condition is another important factor in the control of shrinkage crack. The degree of restraint always influences the risk of cracking as well as characteristics of cracks (such as average crack width and number of cracks). Despite this well-recognized role of restraint, it is still very difficult to estimate the appropriate value of restraint degree of a RC structure. This study is aimed to produce knowledge on the restraint degree of thin RC structures with different boundary conditions and details of reinforcement by employing FEM analysis. Some field tests and real site observation will be also conducted in order to verify the FEM results.

Once this study is completed, the structural engineers as well as concrete practitioners will be provided with the new type of concrete which has high resistance to shrinkage cracking while the information about the degree of restraint will also allow a convenient risk assessment of shrinkage cracking. And, the problem of shrinkage-induced cracking can be efficiently prevented.

1.2 Objectives

- To develop concrete with high resistance to shrinkage cracking by good material selections and optimized mix proportions. The performance of concrete in various aspects is verified by experiments
- To study the degree of restraint of various types of reinforced concrete structures that is subject to shrinkage crack problem by field investigation as well as numerical simulation such as FEA
- To develop a guideline for material selection and a control of restraint in order to prevent shrinkage crack in concrete structure.

Chapter 2: Literature Review

2.1 Shrinkage of Concrete

Shrinkage of concrete can be caused by many reasons and can be classified as

- Plastic shrinkage which is caused by evaporation of water near exposed surface of concrete in plastic state.
- Autogenous shrinkage which is caused by chemical reactions (chemical shrinkage) and self-desiccation of cement paste.
- Drying shrinkage is caused the drying of concrete in hardened state and the macroscopic volume is reduced.

Since the plastic shrinkage problem takes place before the setting of concrete and can be alleviated by good curing and surfacing works. The plastic shrinkage is therefore not included in the scope of this study. The autogenous shrinkage and drying shrinkage, on the other hand, cannot be easily alleviated and are main targets of this study.

The autogenous shrinkage and the drying shrinkage are different. More autogenous shrinkage can be expected in the concrete with lower water-to-cement ratio (w/c) while the drying shrinkage is more severe in the concrete with higher w/c. In a real environment, a concrete structure is subjected to both types of the shrinkage. Their summation is usually called 'total shrinkage'. In the case that w/c is higher than 0.35, the total shrinkage decreases with higher w/c [1]; while, in the case that w/c is less than 0.35, the total shrinkage increases with higher w/c [2]. More influence of drying shrinkage can be expected in the case that a concrete structure is exposed to low relative humidity, higher wind speed, insufficient curing and has more surface area.

Since the shrinkage of concrete is a behavior that is mainly governed by cement paste and the aggregates usually have more volume stability, the concrete with more aggregates content thus has lower shrinkage if concretes with the same type of binders and water content are compared [3]. Due to this reason, it is typical that cement paste shrinks more than mortar and concrete with the same w/c, respectively [4].

The shrinkage of concrete is also dependent on the type of cement. For instance, the cement with higher C_3A produces larger autogenous shrinkage than the cement with higher C_2S. In addition, the use of different cement-replacing materials influences the shrinkage in different ways. The replacement of cement by a suitable amount of limestone powder or fly ash in a concrete mixture reduces the autogenous shrinkage of the concrete [1] whereas the cement-replacement by silica fume generally increases the shrinkage of concrete [5]. It is noted that the effect of cement-replacing materials is also dependent on the curing provided.

The use of superplasticizer (SP) has a tendency to increase autogenous shrinkage especially when the SP is applied to a concrete with low w/c because the SP improves the dispersion of cement particles and accelerates the hydration reactions [4].

2.2 Techniques for Shrinkage Control

Based on the behaviors of concrete described above, the shrinkage of concrete can thus be controlled by a carful mix design and selection of raw materials; for examples,

(1) Use of cement with low C_3A and high C_2S

(2) Increase aggregate content in concrete mix proportion especially coarse aggregate

(3) Use of suitable cement-replacing materials such as fly ash or limestone powder

(4) Control a dosage of superplasticizer in a suitable range

It is noted here that, in the actual production of concrete, the designer of mix proportion has to consider other properties of concrete such as compressive strength, slump, or other durability properties and these requirements may constraint the mix design and frequently the shrinkage cannot be satisfactorily reduced by merely a mix proportioning.

In the case that a substantial reduction of shrinkage or a complete prevention of shrinkage is required, the use of ‘expansive additive (EA)’ and ‘shrinkage reducing admixture’ (SRA) is one of widely-accepted methods [6,7]. It was reported that the use one of these two materials or the use of these two materials together can remarkably reduce a severity of shrinkage cracking problem. However, the application of these materials results in higher cost of concrete.

The concrete which is produced with expansive additive in order to control shrinkage crack is technically called ‘shrinkage compensating concrete’ in which the suitable early-age expansion is generated in order to compensate with a subsequent long-term shrinkage [8,9]. It was reported, in past research works [10,11], that the concrete with both EA and fly ash provide more expansion in comparison to the concrete with only the same amount of EA (without fly ash). It was also reported that the higher environmental temperature increases the expansion of the concrete with expansive additive. Such information indicates more suitability of the EA application in Thailand where the use of fly ash is popular and a common ambient temperature is high.

However, at present, there is no sufficient knowledge on the application of the expansive additive and shrinkage reducing admixture in tropical zone and there is still no guideline for an efficient and suitable use of such materials in tropical countries; especially, in Thailand.

2.3 Analysis of Concrete Cracking due to Restrained Shrinkage

In past research works [12,13], an analytical method for predicting cracking ages was proposed. The prediction method needs the values of a total free shrinkage, a degree of restraint, and a cracking strain capacity of concrete in the calculation. The method can also be applied in the case that the 'shrinkage compensating concrete' is used.

In the prediction of cracking age, the suitable estimation of the degree of restraint and the cracking strain capacity is necessary. In the case of the cracking strain capacity, there has been some research works [14] that reports the cracking strain capacity of different concretes. The effect of water-to-cement ratio, types of binders, cement paste content, aggregate content, size of coarse aggregate and moisture condition of concrete on the cracking strain capacity were also discussed. The available information can be referred in the selection of a suitable value of cracking strain capacity for the analysis.

On the other hand, there has been no guideline or any reliable information for the suitable 'degree of restraint' for the analysis. The selection of the suitable 'degree of restraint' is complicated and many factors such as shape and size of the structural elements, details of reinforcement, and restraint by adjacent structural members, should be considered. Additional studies in this aspect are clearly required.

2.4 Studies on the Role of Restraints on the Cracking of Concrete

Role of restraint in the cracking of concrete has been receiving more and more interests recently. The restraint includes the arrangement patterns of reinforcement (size, number, spacing, and stiffness of reinforcement), the joints, and connections to adjacent members. However, the past studies [15, 16, and 17] were focused in a specific component of restraint. For instance, Al-Saleh [15] studied the influence of reinforcing bars in the restraint of concrete's shrinkage and reported that the restraint is the highest at the position of the rebars and becomes less at a distance from the rebars. The restraint distribution shows a linear tendency with distance from the rebars when the age of samples increases. In the case of Kianoush's study [16], the wall restrained by floor was studied and analyzed by the finite element method (FEM). In his study, the crack width and the required amount of reinforcement for the crack control could be estimated. And, Miltenberger [17] calculated the joint spacing for a slab-on-ground and proposed an installation of additional reinforcement for controlling different types of cracks. Although the above studies can be referred in the future study, the existing knowledge is however not enough for the design of concrete structures with high resistance to shrinkage crack.

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Chapter 3: Methodology

3.1 Overall Research Structure

In order to develop the technologies for the prevention and alleviation of the shrinkage crack problem, this research work therefore consists of two parts (see Figure 3.1)

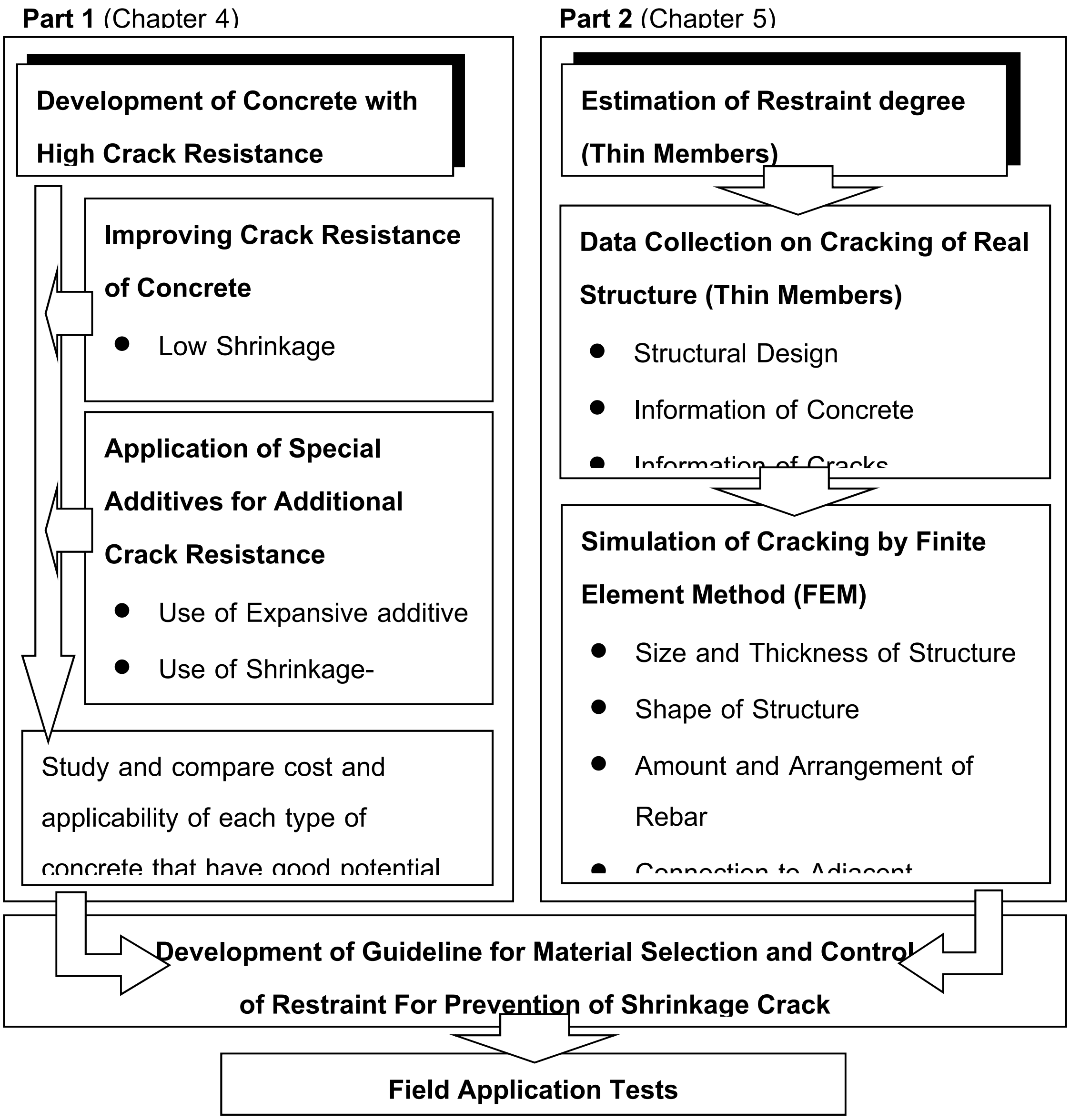


Figure 3.1 Flowchart of this research

Part 1: Development of Concrete with High Crack Resistance

This part emphasizes on the development of concrete with low shrinkage (high volume stability) and high cracking strain capacity by careful selection of materials and mix proportioning. The use of expansive additive and shrinkage reducing admixture will also be studied. The feasibility and suitability of the usage in the real construction will also be evaluated.

In this part, experimental works will be conducted in order to evaluate the crack resistance of concrete with different mix proportions. 'Shrinkage' and 'cracking strain capacity' will be experimentally evaluated and compared. The use of some cement-replacing materials such as fly ash and limestone powder to improve the crack resistance will also be considered and evaluated. The knowledge obtained from the experiments will be very useful for the material selection for a real construction where shrinkage cracks are not acceptable. The use of 'expansive additive' and 'shrinkage reducing admixture' will also be studied. The use of these materials will allow a production of concrete with extraordinary crack resistance. The mechanisms of these materials in the improvement of crack resistance will be investigated in detail. Some microscopic investigation on the microstructure of the concrete may also be performed.

Finally, the concrete mix proportion which shows remarkable potential for a real construction will be selected based on its performance. The cost/benefit analysis of of the proposed concretes as well as an evaluation of their applicability in a real construction will be performed in order to evidently illustrate its merits.

Part 2: Estimation of Restraint degree (for Thin Members)

In this part, the data collection on real structures suffered from shrinkage cracking will be conducted. The survey results will be reproduced by finite element model (FEM) in order to develop a guideline for the estimation of restraint degree. Various parameters such as reinforcement ratio, connectivity to adjacent members will also be included. The case of thin members (floors, roads, and wall) will be emphasized since they suffer more from shrinkage cracking. The guideline will be very useful for improving the crack resistance of concrete structure and selection of appropriate concretes.

Based on the results from Part1 and Part2, the final output will be in a form of guideline for prevention of shrinkage crack by selection of material as well as a control of restraint. The proposed method will finally be applied in the site test.

3.2 Scope of research

- Shrinkage crack problem of thin RC members under the usual environmental conditions of Thailand is considered.
- Raw materials available in Thailand (such as fly ash, limestone powder) are mainly considered. The ‘expansive additive’ and ‘shrinkage reducing admixture’ will also be applied.
- In the FEM simulation, only frequently-applied form (shape & size) of structures will be considered and analyzed.

Chapter 4: Mitigation of Shrinkage

4.1 Background

Shrinkage is one of physical properties of concrete. It is a volume change. Most prominent shrinkage mechanisms that have been proposed are capillary tension and movement of interlayer water. For example, the cause of drying shrinkage is the evaporation of excess water as the concrete dries [1].

The cause of autogenous shrinkage is hydration and physical shrinkage due to self-desiccation [2]. A variation of pore water content caused by drying and wetting cycles seems to be another factor making significant volume changes of concrete

Fly ash is a by-product of the coal-combustion process in thermal power plants. A generating system of fly ash is given in Figure 4.1. The coal burns at a high temperature (>1500 ^{o}C), and the inorganic material melts and agglomerates into spheres. The chemical reaction of coal combustion occurs during heating in the furnace. Upon leaving the combustion zone, the ash particle are quickly cooled and solidified. About 20% of the non-combustible coal falls to the bottom of the furnace where it sinters together to become the bottom ash. Formed from burning coal in the furnace and extracted from flue gas is called fly ash. Fly ash is carried up the chimney stack, along with the combustion gas, is captured by electrostatic precipitator. Fly ash particles are produced from two different sources in Thailand. There are two classes of fly ash: Class F fly ash is produced from anthracite and bituminous coal, having low amount of calcium oxide (CaO), less than 10 % of CaO. The total amount of silica, alumina and ferritic oxide is more than 70%, whereas the amount of SO3 is less than 5%, and class C fly ash is produced from lignite and sub-bituminous coal, having high amount of calcium oxide, more than 10% of CaO. The total amount of silica, alumina and ferritic oxide is than 50%, whereas the amount of SO3 is less than 5%. The specification is ASTM C 618.

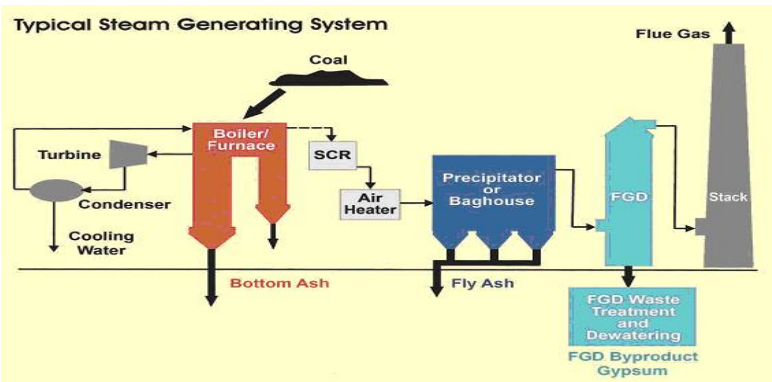


Figure 4.1 Diagram of fly ash generating system [3]

Table 4.1 Type of shrinkage-compensating cements and their constituents

Expansive cement	Principal constituents	Reactive aluminates available for ettringite formation
K	(a) Portland cement (b) Calcium sulfate (c) Portland-like cement containing C_4A_3S	C_4A_3S
M	(a) Portland cement (b) Calcium sulfate (c) Calcium-aluminate cement CA_4 and $C_{12}A_7$	CA_4 and $C_{12}A_7$
S	(a) Portland cement high in C_3A (b) Calcium sulfate	C_3A

Table 4.2 Physical properties and chemical compositions of both “Ettringite series” and ‘Lime series’.

Item		C-S-A type	CaO type
Physical Properties	Specific gravity (%)	3	3.14
	Specific surface area (cm^2/g)	2500	3500
Chemical composition	Ignition loss (%)	0.8	0.4
	SiO_2 (%)	4	9.6
	Al_2O_3 (%)	10	2.5
	Fe_2O_3 (%)	1	1.3
	CaO (%)	51.2	67.3
	MgO (%)	0.6	0.4
	SO_3 (%)	31.9	18
	Total (%)	99.5	99.5

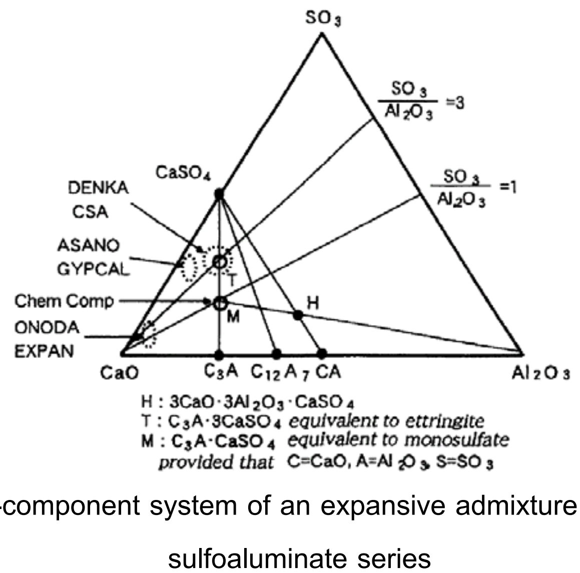


Figure 4.2 Three-component system of an expansive admixture relating to calcium sulfoaluminate series

In US, there are types of expansive cement as follows: K-type: Portland cement mixed with anhydrous Hauyne ($3CaO.3Al_2O3.CaSO_4$), gypsum ($CaSO_4$) and quick lime (CaO).M-type: Portland cement mixed with alumina cement and gypsum (CaSO4) at a reasonable ratio.S-type: normal portland cement mixed with larger amount of tricalcium aluminate(C_3A) and gypsum ($CaSO_4$).

Expansive admixtures include iron powder, alumina powder, magnesia, calcium sulfoaluminate ($CaO\text{-}Al_3O_2\text{-}SO_3$) and calcium oxide (CaO). However, the main groups are the calcium sulfoaluminate series. Definition of expansive additive is that "Additive, when mixed with cement and water, produce ettringite ($3CaO.Al_2O_3CaSO_4.32H_2O$) or calcium hydroxide ($Ca(OH)_2$) by hydration reaction to expand the concrete".

Expansive concrete produced by the expansive cement was popular in USA and Europe. However it was difficult to control the expansion of the expansive concrete because of expansion will generate depending on amount of expansive cement. In Japan, expansive concrete was developed by using expansive additive which is mixed with normal cement concrete, thus, the expansion rate can be easily controlled and managed by adjusting amount of expansive additive.

Bottom ash is the ash form process of electricity of power plant like a fly ash. Bottom ash is generally not considered pozzolan material as with fly ash is because fly ash is much smaller and more reactive than bottom ash so bottom ash a little reactive pozzolan so bottom ash. Lightweight aggregates may actually be minerals, natural rock materials, rock-like products, or by-products of manufacturing processes that are used as bulk fillers in lightweight concrete. It is used in structural building as lightweight concrete to reduce the self-weight of structure. Lightweight aggregate can be used to decorate architectural works such as walls, or suspended ceilings.

Lightweight aggregate could be classified into 4 groups. There are natural lightweight aggregate, manufactured structural lightweight aggregate, by-product lightweight aggregate and manufactured insulating ultra- lightweight aggregate. Firstly, natural lightweight aggregate like pumice or volcanic cinder is produced by mining, crushing, and screening to appropriate size. Secondly, manufactured structural lightweight aggregate like expanded clay, shale or slate which are prepared by crushing, screening, stockpiling, pyro processing them in rotary kilns or on traveling grate sintering machines, cooling. Thirdly, by-product lightweight aggregate like slag, organic cinders or coke breeze is produced by crushing, sizing foamed and granulated and fourthly, manufactured insulating ultra- lightweight aggregate like ground vermiculite, perlite, and diatomite is prepared by pyro-processing. For this research, expanded clay, the manufactured structural lightweight aggregate is used. Lightweight aggregate has low particle density as its cellular structure that emerged by heating the material. Its particle shape and surface texture counts on the source and process of production and directly influence workability, cement content requirements, and water demand in a concrete mixture. The bulk density of lightweight aggregate is low. Lightweight aggregate (LWA) is porous and has the suction ability. There are two types of pores in the LWA: open pore that is interconnected and take part in the permeation and closed pore that is sealed and not interconnected. The simple way to assess the interconnectivity of the pores is by measuring the water absorption property. The lightweight aggregate has high absorption values that may require a modified approach to concrete proportioning [4]. Commonly, the lightweight concrete should be proportioned by the total volume method. After 24 hours immersion of lightweight aggregate, the rate of further absorption will be low [5,6]. Lightweight aggregate would be pre-wetted before mixing to absorb the water which avoids the loss of workability.

Super absorbent polymers (SAP) are compounds that absorb water and swell into many times of their original size and weight. These products are able to absorb up to 200 - 400 times their weight of water through physical absorption by using capillarity, osmosis mechanisms, and hydration of functional groups.

The advantages of SRA are great reduction of cracking caused by drying shrinkage in concrete structures, improvement of crack-reducing performance with an inorganic expansive admixture, and better bonding strength. The mixing method of SRA - used as a part of mixing water in the same way as generally-used air entraining water-reducing admixture. SRA is added to a mixer at batching plant as a part of unit water content and must be mixed uniformly. A quantity of SRA must be necessary deducted from unit water content.

4.2 Materials

The chemical compositions of fly ash are shown in **Error! Reference source not found.**, fly ash were collected from Mae-Moh power plant and BLCP power plant as called as MFA and BFA, respectively. Appearance and SEM image are shown in **Error! Reference source not found.**a and **Error! Reference source not found.**b.

Table 4.3 Chemical compositions of both MFA and BFA

Chemical Composition (%)	**Fly ash- Mae-Moh (MFA)**	**Fly ash- BLCP (BFA)**
Al_2O_3	10.097	13.397
SiO_2	21.193	40.604
P_2O_5	0.316	0.545
K_2O	3.014	1.815
Cao	30.33	26.274
TiO_2	1.055	3.756
V_2O_5	0.07	-
Cr_2O_3	0.028	-
MnO	0.211	0.111
Fe_2O_3	33.353	11.32
NiO	0.019	-
CuO	0.029	0.055
SrO	0.287	0.355

(a) MFA (Mae Moh, Lampang)

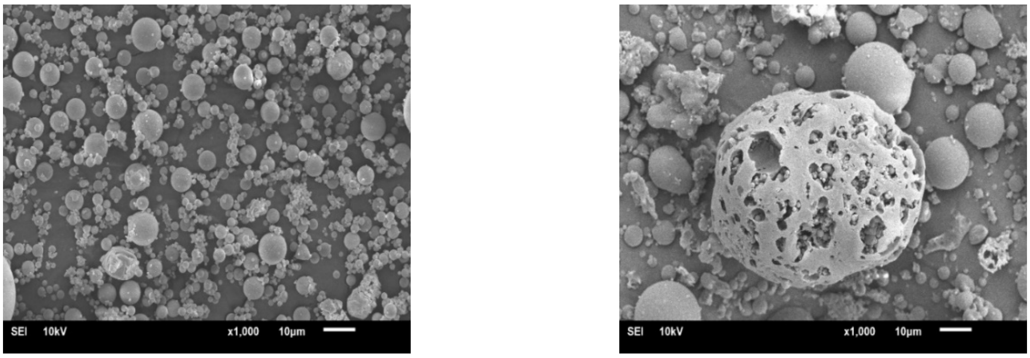
(b) BFA (BLCP, Rayong)

Figure 3 Appearance of fly ash

(a) MBA (Mae Moh, Lampang)

(b) BBA (BLCP, Rayong)

Figure 4.3 Appearance of Bottom ash



(a) MFA (Mae Moh, Lampang) (b) BFA (BLCP, Rayong)

Figure 4 SEM image of fly ash (1000x)

Figure 5 Appearance of lightweight aggregates

Expansive Additives (EA) used in this study is an expansive additive with higher CaO content (CaO type) which induces expansion by the formation of $Ca(OH)_2$ crystal.

Fine aggregate consist of river sand, synthetic lightweight aggregate (LWA) and bottom ash (BA). River sand were used as a control aggregate, its size has smaller than 4.75 mm or pass sieve No 4.

Bottom ash (BA) was used in order to replace river sand for internal curing. BA is the ash form process of electricity of power plant like a fly ash. Bottom ash isn't pozzolan material when compared with fly ash because fly ash is very smaller than bottom ash so bottom ash a little reactive pozzolan so bottom ash isn't suitable to be used instead of cement. Appearance and SEM image were showed in **Error! Reference source not found.**3a and **Error! Reference source not found.**b.

LWA was used in order to replace river sand for internal curing. LWA is a kind of lightweight as expanded clay aggregate that was fired at a temperature higher than 1250 ^{O}C is called “pyroprocessing”. Furthermore, LWA was expanded clay of a single fraction (2.30-4.00 mm) in **Error! Reference source not found.**. The water absorption of synthetic lightweight aggregate by soaking for 72-h was 16.33% and specific gravity in oven dry condition is 1.26 kg/m^3.

Superabsorbent polymer (SAP) are compounds that absorb water and swell into many times their original size and weight. These products are able to absorb up to 200 - 400 times their weight of water through physical absorption by using capillarity, osmosis mechanisms, and hydration of functional groups (see in **Error! Reference source not found.**).

Figure 4.6 Appearance of superabsorbent polymer (SAP)

Shrinkage reducing agent (SRA) can reduce autogenous shrinkage and drying shrinkage. The using of SRA shows that the observed percentage decrease in autogenous shrinkage corresponds to the percentage decrease in the solution surface tension. Moreover, the weight loss of mortar is less than the control mortar in drying condition, because SRA reduces the surface tension of water in capillary pore.

4.3 Test and Determination of Material Properties

The water absorption and bulk specific gravity (SSD) of river sand, synthetic lightweight aggregate (LWA), and boom ash (BA) were tested according to ASTM C128. The absorption, desorption and specific gravity of LWA and BA were tested according to ASTM C1761

The 72-h absorption provides an indication of the water capacity of the lightweight aggregate for internal curing. It is critical to assess how readily this absorbed water is released from the lightweight aggregates to the surrounding cementitious matrix during curing. Desorption test method is used to determine the quantity of absorbed water that is readily released and available to maintain saturation of the capillary pores in the paste. For this standard, an environment of 94 % relative humidity and a temperature of 23.0 $\pm$ 1 ºC is used to assess the desorption capacity.

The method described in this specification is similar in principle to test method C1498 for determining the sorption isotherms of building materials. In this specification, the lightweight aggregate is required to release at least 85 % of its 72-h absorbed water under the stated storage conditions.

Equation 1 can be used to calculate the 72-h absorption which is necessary to determine the mass of lightweight aggregate needed to provide the required quantity of water for internal curing or amount of lightweight aggregate required for internal curing per unit volume of concrete can be estimated using by equation 1

$$A_{72} = \frac{M_{SD} - M_{OD}}{M_{OD}} \times 100\ \% \tag{1}$$

Equation 2 can be used to calculate the specific gravity (OD)

$$G_{OD} = \frac{M_{OD}}{M_{SD} + M_{PW} - M_{PS}} \tag{2}$$

Equation 3 is for calculation of water released at 94 % relative humidity, expressed as a fraction of the oven-dry mass to the nearest 0.01%

$$W_{LWA} = \frac{M_{SD} - M_{94}}{M_{OD}} \tag{3}$$

Equation 4 is the calculation of desorption as a percentage of the 72-h absorption to the nearest 0.1%.

Apparatus

$$D = \left(W_{LWA} / \left(A_{72} / 100\ \%\right)\right) \times 100\ \% \tag{4}$$

The use of lightweight Aggregate (LWA) for Internal Curing of Concrete was tested according to ASTM C 1761. In case of using paper towel method, aggregates have to be soaked for 72 hr. The sample would be wiped at the surface until it is moist (shown in **Error! Reference source not found.** to **Error! Reference source not found.**). The sample is put to the flask 300 g and the water is poured at the volumetric mask. The flask is agitated for 3 minutes and 15 minutes separately to eliminate air bubbles. Then, the sample is placed in the oven for 24 h after weighting the flask which contains both sample and water. The value had to be recorded after the sample is cooled down.

Table 4.4 the absorption and specific gravity of fine aggregate.

Materials	Absorption (%)		specific gravity (kg/m^3)	
	ASTM C 128	ASTM C 1761	ASTM C 128	ASTM C 1761
River sand	0.26	-	2.64	-
LWA	12.75	16.33	1.13	1.26

BA I	12.45	19.19	2.12	1.76
BA II	1.15	13.32	1.84	1.51

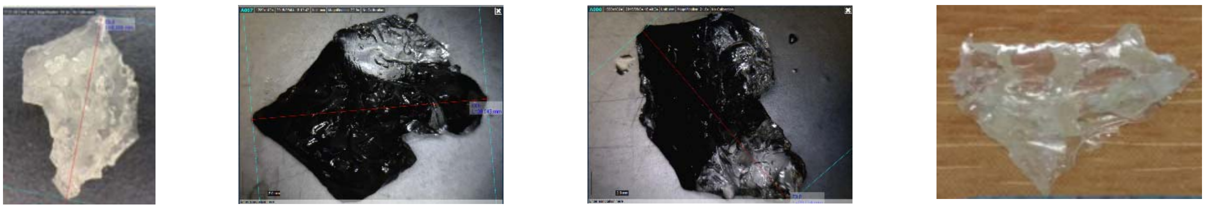

Figure 4.7 Bottom ash (BA) after soaking 72 hrs.


Figure 4.8 Wipe the aggregate by paper towel.


Figure 4.9 Wipe the aggregate until the surface- dry.

The absorption characteristics of SAP was tested after being placed in the drying oven at 105 $^{\circ}$C. There are 4 steps: firstly, the size of SAP was measured before SAP is soaked in **Error! Reference source not found.**a. Secondly, SAP was soaked for 30 min (see in **Error! Reference source not found.**b) and 1 h (see in **Error! Reference source not found.**c). Thirdly, the SAP placed in the drying oven at 105 $^{\circ}$C for 24 h and finally, its size was measured as shown in **Error! Reference source not found.**d.

After SAP was placed in the drying oven at 105 $^{\circ}$C, it was found that the absorption capacity of SAP was not decreased with time. Moreover, the SAP can desorb water by 100% of absorbed water.


a. before being soaked (6.35 mm)

b. after being soaked for 30 min (20.04 mm)

c. after being soaked for 1 h (20.09 mm)

d. after being ovened for 24 h (8.00mm)

Figure 4.10 Appearance of Superabsorbent polymer (SAP) in different steps

Figure 4.11 Hallow cylinder pipe

Figure 4.12 Vibration machine

In this experiment, the effect of porous aggregate as bottom ash (BA) on the water retainability is also investigated [7]. This method was determined the water retainability of porous fine aggregate for design and quality control of fresh concrete"). It is utilized in the replacement of fine aggregate. But one of the main problems for the use of porous aggregate is the moisture property. So the equipment that can help to find the water retainability is showed below: hollow cylinder pipe (shown in **Error! Reference source not found.**) and vibration machine (Shown in **Error! Reference source not found.**)

Error! Reference source not found. to **Error! Reference source not found.** show that the process of determining the water retainability test: water retainability is defined as the water required to completely fill in pores and dependently restricted on surface of aggregate particles under gravitational condition. Moisture property of concrete consists of water absorption and water adsorption and affects the free water amount in fresh concrete. For concrete incorporating porous aggregate, it is reasonable to apply water retainability instead of water absorption to design mixture proportion and to control fresh concrete properties, especially workability. The water retainability of aggregate is specified by the largest mass ratio of water to dry aggregate that creates the zero difference of moisture content between the top and bottom portions (Δm = 0). A relationship between the moisture difference of the tested aggregate and the corresponding trial moisture is plotted, and the water retainability is the moisture value at the interception between the curve and the axis of the trial moisture content.

The 3-5 kg of aggregate was kept at saturated surface dry for 24$\pm$4 h. It was separated up to two-third of its volume, compacted again and filled the cylinder container until overflowing and compacted again. The top of cylinder container was sealed by plastic in order to prevent the loss of moisture. The cylinder container was vibrated for 24 minutes and this material was separated into 2 parts; top and bottom of cylinder container, these 2 parts were weight out of 500±10 g and put into the oven until these 2 parts are constant weight for 72±4 h and then use Equation 5 to calculate the moisture content.

$$m_i = \frac{(B-A)100}{A} \qquad (5)$$

The moisture content between 2 parts: moisture at top cylinder (m_t) and moisture at bottom cylinder (m_b) of cylinder container were compared. The Equation 6 shows difference of the moisture content of the tested aggregate between the top and bottom parts. The result is showed in below (**Error! Reference source not found.** and **Error! Reference source not found.**).

$$\Delta_m = m_b - m_t \qquad (6)$$

Figure 4.13 Fine aggregate and water trial 22-35%.

Figure 4.14 Mix the fine aggregate and water together.

Figure 4.15 Keeping the fine aggregate for 24 hr.

Figure 4.16 Fill the fine aggregate in the container to

two-third.

Figure 4.17 Seal the top of the container and vibrate for 24 minutes.

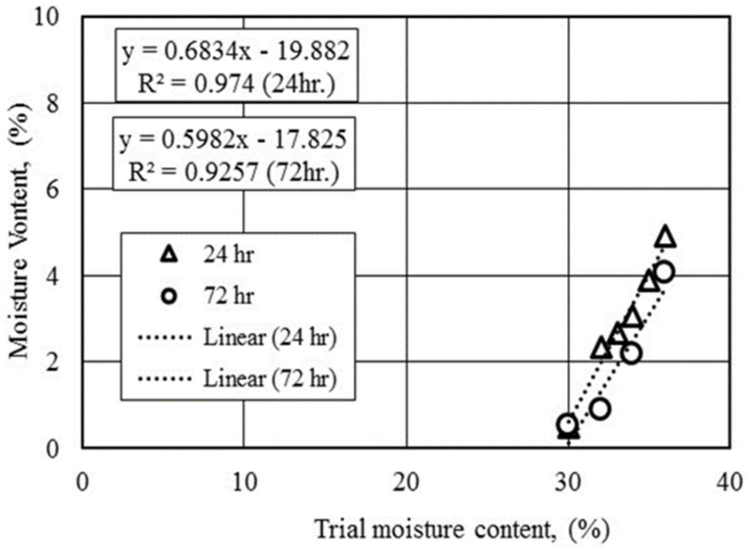


Figure 4.18 Water retainability of bottom ash Mae-Moh (MBA) at 24 and 72 hr.

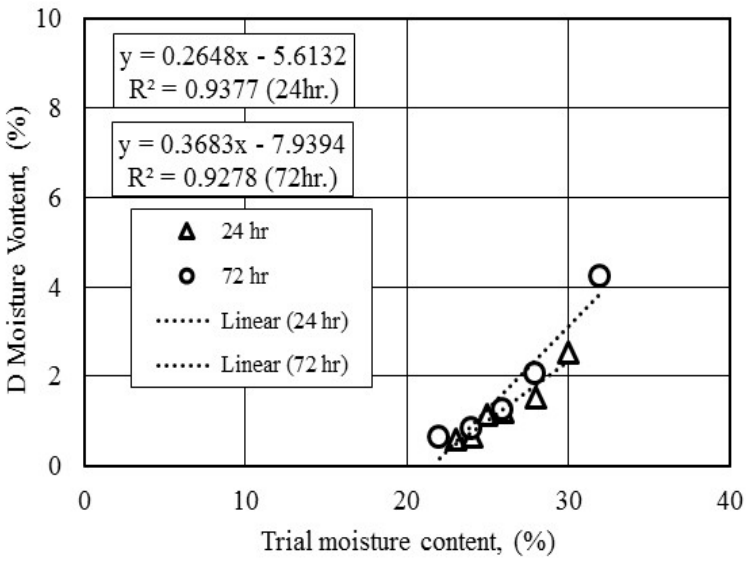


Figure 4.19 Water retainability of bottom ash BLCP (BBA) at 24 and 72 hr.

4.4 Flow and Flow Loss

Flow measurement is one of method to determine consistency of fresh concrete. In research, the flow loss is tested following ASTM C230. Flows of mortar mixtures were determined according to ASTM C1437. Firstly, the mortar mixtures is placed into the flow mold at the center and a layer of mortar mixtures about 25 mm in thickness is placed in the mold and tamp 20 times with the tamper. After filling the mold with mortar and tamp as specified for the first layer, the mortar is cut off to a plane surface. Secondly, the table is wiped from the top and the mold is lifted away from the mortar 1 min after completing the mixing operation. Then, the table was immediately dropped for 25 times in 15 s. Finally, the caliper is used to measure the diameter of the mortar along the four lines scribed on the table top, each diameter is recorded as the number of caliper divisions. For the flow loss, the mortar mixtures were tested every 30 minutes until 2 hours.

The initial flow of mortars with two different condition of bottom ash was tested in oven-dry (OD) and saturated condition (SD). As can be seen from Figure 4.21, the initial flow of control mortar was the lowest and MBA seems to hardly affect the initial flow of mortar with MBA, because round particle shape of MBA can reduce the friction between particle and support flow ability.

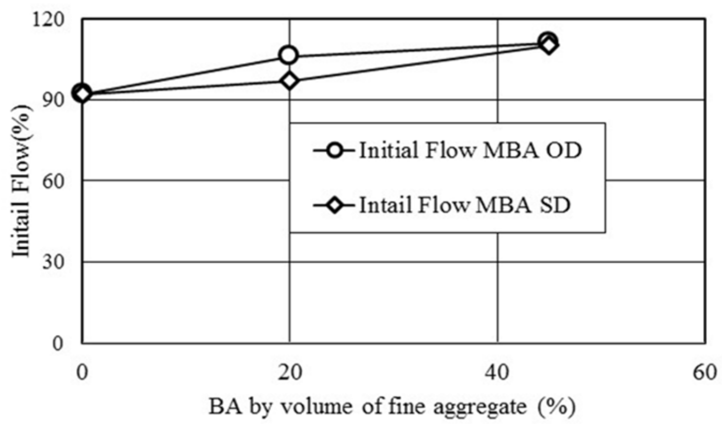


Figure 4.20 Initial flow between MBA in oven-dry and saturated (MBA-OD and MBA-SD) (W/B =0.35)

In case of 20% MBA, the initial flow MBA-OD 20% was higher than MBA-SD 20% because MBA-OD had free water more than MBA-SD. The free water was increased the initial flow of MBA-OD. In contrast, 20% of MBA-SD were already kept the water in side by 72 hours of soak. So, MBA-SD can absorb the water already. Thus, the free water of 20% with MBA-SD was less than MBA-OD. However, in 45% of two sources of MBA-OD and MBA-SD were equal values. So, we could expect in two ways of trends. Firstly, 45% of MBA-OD can absorb the free water and make them decreased with higher than 45%. Secondly, 45% of MBA-SD can desorb the water inside MBA to help the initial flow.

Error! Reference source not found. has two general basic trends between upward and downward in comparison to percentage from 0% to 45%. In 20% and 45% with BBA-SD, the aggregates of BBA-SD contained the water in-side by 72 hours of soak. The mortars with BBA-SD were increased the initial flow, because the quantity of water desorbed from BBA. On the other hand, the initial flows of mortar with 20% and 45% BBA-OD decreased by the smaller average size of BBA particles and higher surface area.

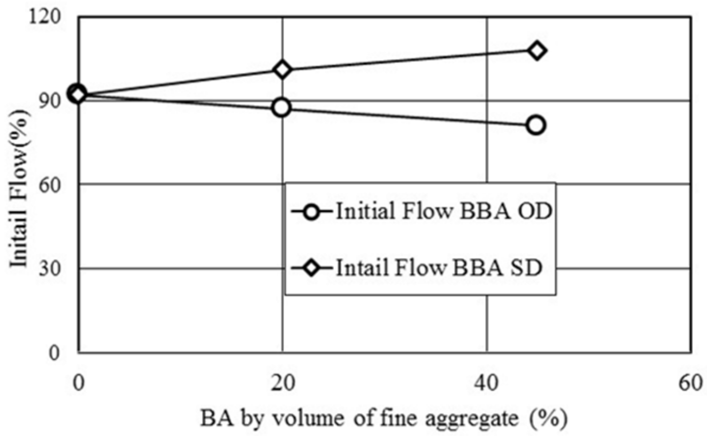


Figure 4.21 Initial flow between BBA-OD and BBA-SD (W/B =0.35)

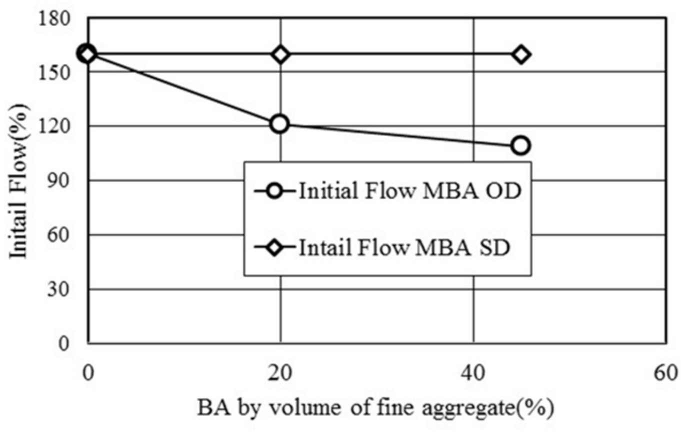


Figure 4.22 Flow loss between MBA-OD and MBA-SD (W/B =0.50)

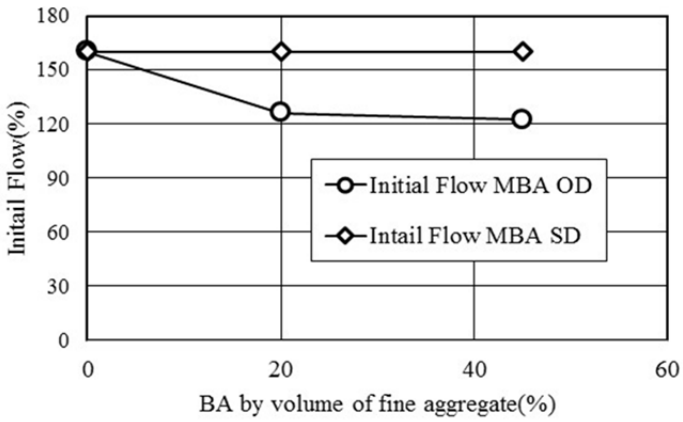


Figure 4.23 Initial flow between BBA-OD and BBA-SD (W/B =0.50)

For w/b of 0.5, as can be seen in **Error! Reference source not found.** and **Error! Reference source not found.**, the initial flow by MBA-OD and BBA-OD were decreased in comparison with control mortar because the particle of BBA-OD was rough and higher porosity than sand. Moreover, the water absorption of BBA-OD was higher than that of river sand.

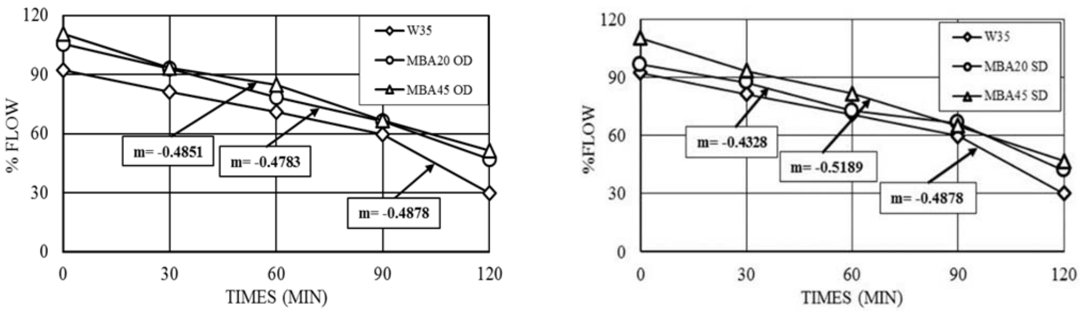


Figure 4.24 Flow loss of MBA20OD, MBA45OD, MBA20SD and MBA45SD (W/B =0.35)

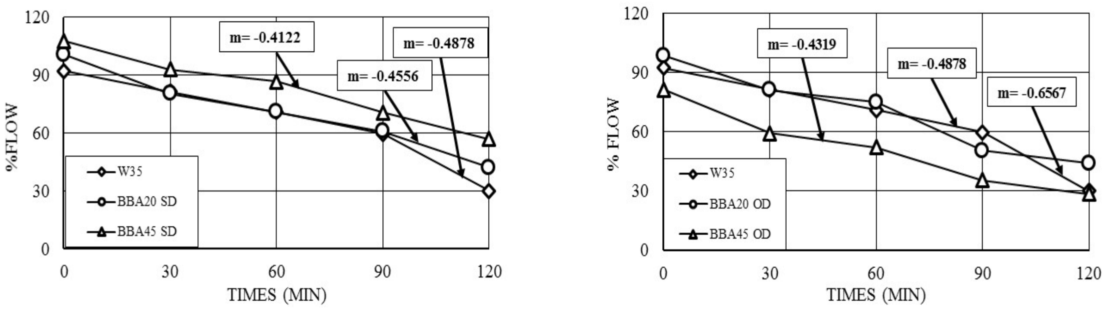


Figure 4.25 Flow loss of BBA20OD, BBA45OD, BBA20SD and BBA45SD (W/B =0.35)

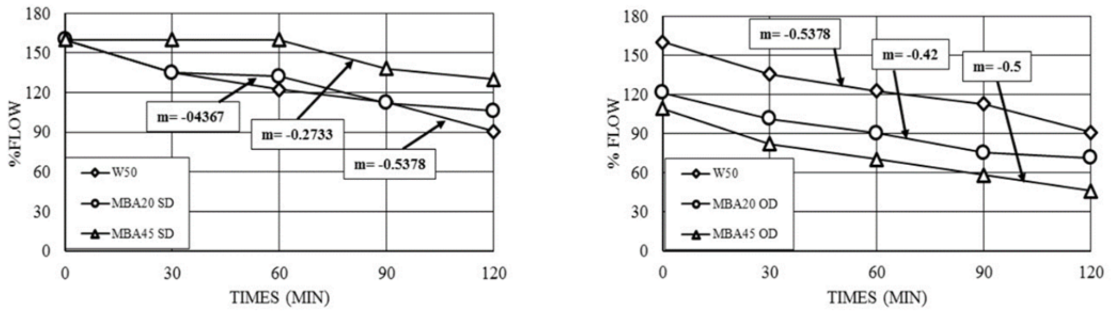


Figure 4.26 Result of the flow loss of MBA20OD, MBA45OD, MBA20SD and MBA45SD by W/B =0.50

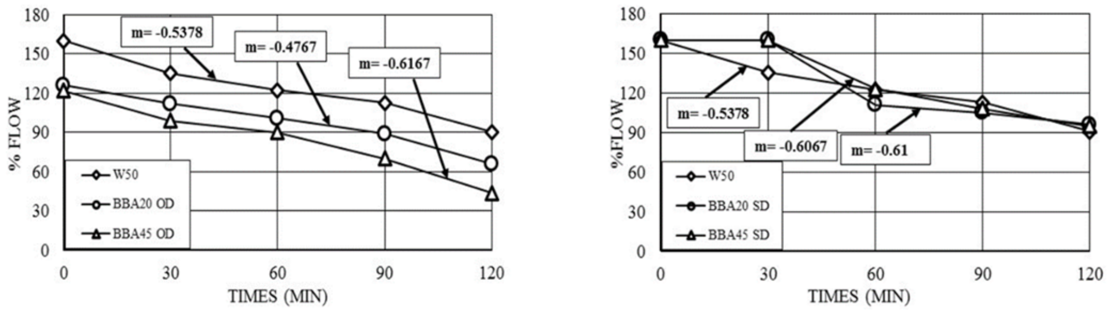


Figure 4.27 Flow loss of BBA20OD, BBA45OD, BBA20SD and BBA45SD (W/B =0.50)

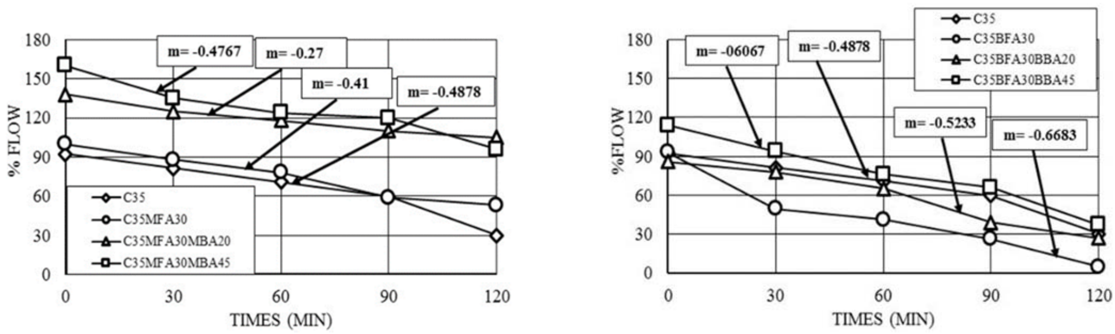


Figure 4.28 Flow loss of MFA30, MFA30MBA20, MFA30MBA45, BFA30, BFA30BBA20, and BFA30BBA45 (W/B =0.35)

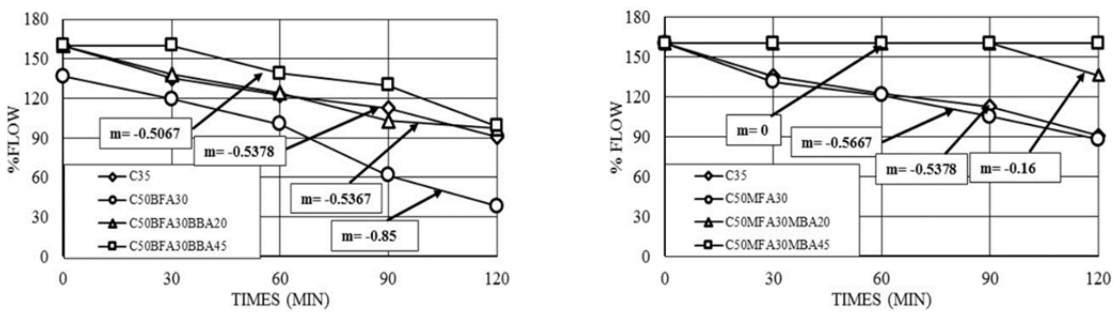


Figure 4.29 Flow loss of MFA30, MFA30MBA20, MFA30MBA45, BFA30, BFA30BBA20, and BFA30BBA45 (W/B =0.50)

Error! Reference source not found. to **Error! Reference source not found.** show that the slope of flow loss decreased with time. The slopes of flow loss were similarly ± 0.05 when BA was added in the mix proportion and compare with control mortar. BA seems to have negligible effect on flow loss.

On the other hand, the slope of flow loss of mortars with the fly ash were increased dramatically caused from the particle of FA sphere shape that helped to decrease the slope of flow loss.

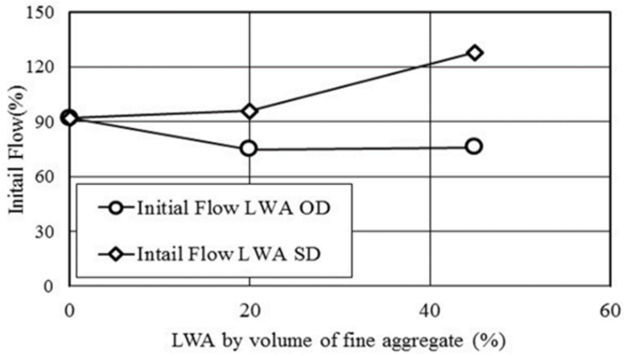


Figure 4.30 Initial flow between LWA-OD and LWA-SD (W/B=0.35)

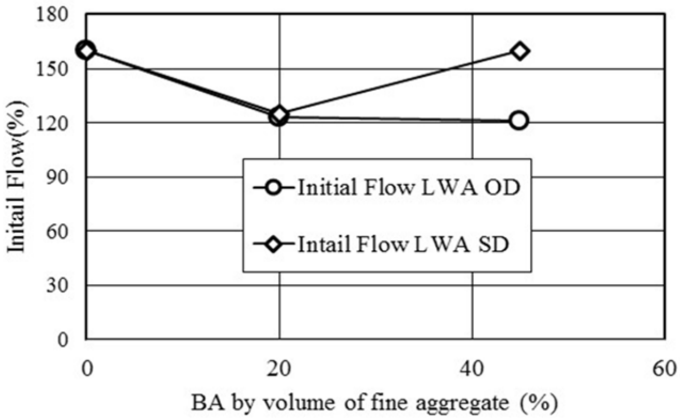


Figure 4.31 Initial flow between LWA-OD and LWA-SD (W/B=0.50)

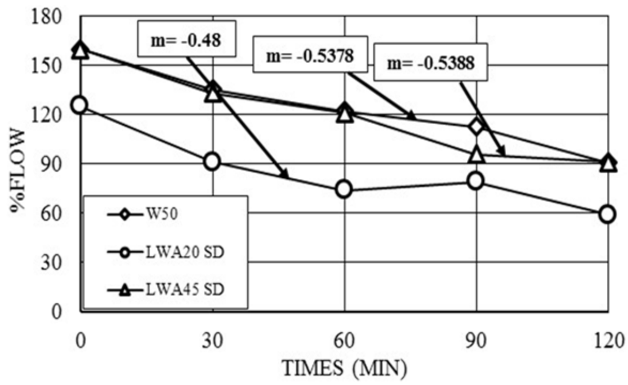


Figure 4.32 Flow loss of LWA20OD, LWA45OD, LWA20SD and LWA45SD W/B=0.35

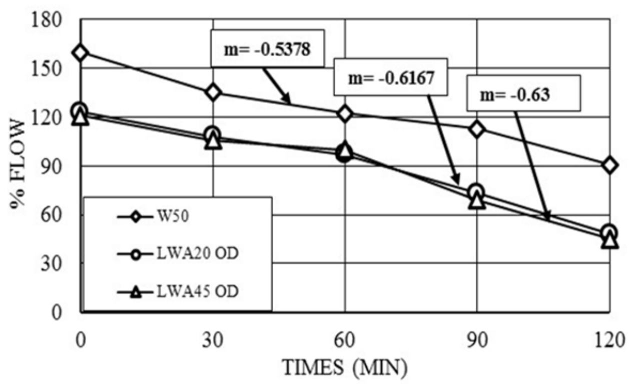


Figure 4.33 Flow loss of LWA20OD, LWA45OD, LWA20SD and LWA45SD W/B=0.50

Error! Reference source not found. and **Error! Reference source not found.** show the initial flow of LWA-OD and LWA-SD. We can see from the LWA-OD, the initial flow was decreased in comparison with control mortar and mortar with LWA-SD because the gradation of LWA. There are more gaps between the particles that make to use more cement paste to fill the gap of particle. So, the initial flow was decreased.

In case of LWA-SD, the water absorption of LWA-SD was higher than that of control mortar and it affected the mortar flow. The viscosity of cement paste with lower W/B was higher than that of cement paste with higher W/B .The W/B of the mortar with LWA-SD was slightly higher than that of the control, because the water absorbed in LWA-SD spilled out during the mixing process. Thus, the viscosity of cement paste with LWASD aggregates was reduced and the initial flow was increased.

Error! Reference source not found. and **Error! Reference source not found.** show that the slope of flow loss between LWA-OD and LWA-SD were similarly in both water binders. The slopes both of them were higher than control mortar because of the gradation, same particle size were more space for cement paste to react with particle. There is more space between the particles that make to use more cement paste to fill the gap of particle. So, the flow loss was occurred more than control mortar.

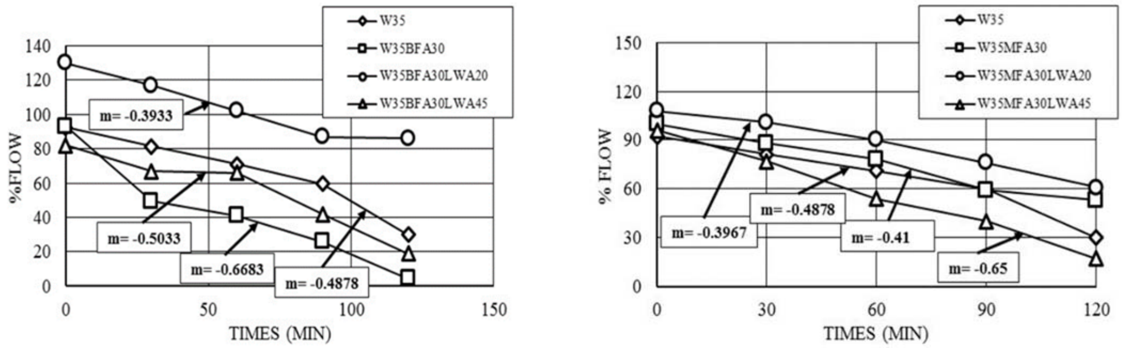


Figure 4.34 Flow loss of MFA30, MFA30LWA20, MFALWA45, BFA30, BFA30LWA20, BFA30LWA45 W/B=0. 35

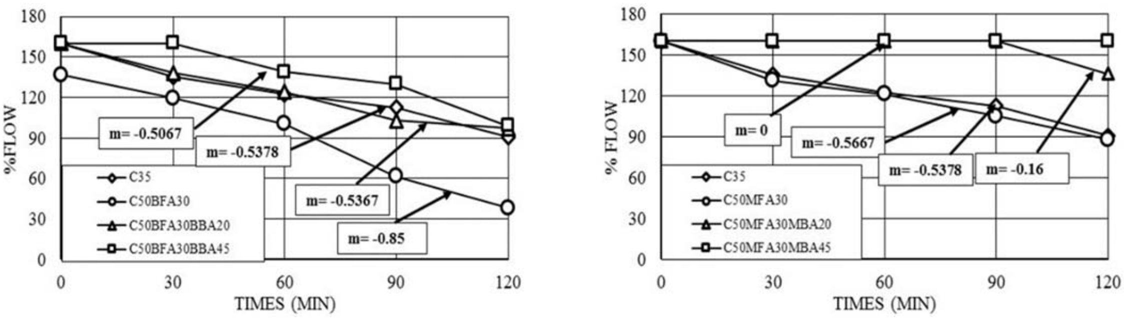


Figure 4.35 Flow loss of MFA30, MFA30LWA20, MFALWA45, BFA30, BFA30LWA20, BFA30LWA45 W/B=0.50

Error! Reference source not found. and **Error! Reference source not found.** show that the flow loss was decreased when fly ash were added replacement cement to the mix proportion because fly ash have finesse lower than cement so when the heat hydration will slowly more than control mortar. Interestingly, some flow ability was higher than 156 that can't keep the value for calculate the flow loss.

The setting times defined next are of no fundamental significance; they merely define two arbitrary points in the general relationship between the time after the addition of water and strength gain. Two setting times are defined: Initial setting time, which indicates that the paste is beginning to stiffen considerably and can no longer be molded and final setting time, which indicates that the cement has hardened to the point at which it can sustain some load.

4.5 Setting Time

Test method for time of setting of mortar according to ASTM C 191. The 1st step; release the rod and allow the needle to settle for 30 s; then take the reading to determine the penetration and record the results of all penetration tests.

The rod The Initial setting time is defined as the time at which the needle penetrates 25 mm. into the specimens and the specimens were measured every 10 minutes until the needle does not sink visibly into the paste. It's called final setting time and then determine the time when a penetration of 25 mm is obtained. This is the initial setting time. The final setting time is when the needle does not sink visibly into the paste. From the testing of setting time of mortar, we found that the characteristics of mortar which containing bottom ash and fly ash were showed in **Error! Reference source not found.** to **Error! Reference source not found.**. The initial and final setting time are showed in the **Error! Reference source not found.**

Table 4.5 Initial and final setting of mortar.

No.	Name of Mix	Initial	Final
1	W35	80	160
2	W35MBA20	79	170
3	W35MBA45	70	145
4	W35BBA20	75	160
5	W35BBA45	70	150
6	W35BFA30	64	120
7	W35MFA30	102	180
8	W35BFA30BBA20	84	180
9	W35BFA30BBA45	95	195
10	W50	107	255
11	W50BBA20	125	210
12	W50BBA45	122	240
13	W50MBA45	79	225
14	W50MBA20	108	225
15	W50FAB30	108	150
16	W50FAM30	139	180
17	W50BFA30BBA20	135	240
18	W50BFA30BBA45	107	225
19	W50MFA30MBA20	143	180
20	W50MFA30MBA45	143	180
21	W35LWA20	65	150
22	W35LWA45	84	225
23	W50LWA20	107	180
24	W50LWA45	83	180
25	W35MFA30LWA20	94	165
26	W35MFA30LWA45	68	150
27	W35BFA30LWA20	88	180
28	W35BFA30LWA45	64	195
29	W50MFA30LWA20	151	255
30	W50MFA30LWA45	148	225
31	W50BFA30LWA20	103	180

32	W50BFA30LWA45	103	195

Error! Reference source not found. show that, the initial setting time on use MBA-OD longer than use saturated because the mix proportion had some water more than MBASD in the system, the water from the calculated of mix design. Likewise, in final of setting time both of 20% and 45%, the result of final of setting time resemble initial of setting time.

As can be seen from the Figure 4.38 that the initial setting time won't affect when BBAOD and BBASD from BLCP were added with 20% but in case 45%, the initial setting when using BBAOD longer than BBASD. Likewise, in final setting time, both of 20% and 45%, the result of final of setting time resemble initial of setting time.

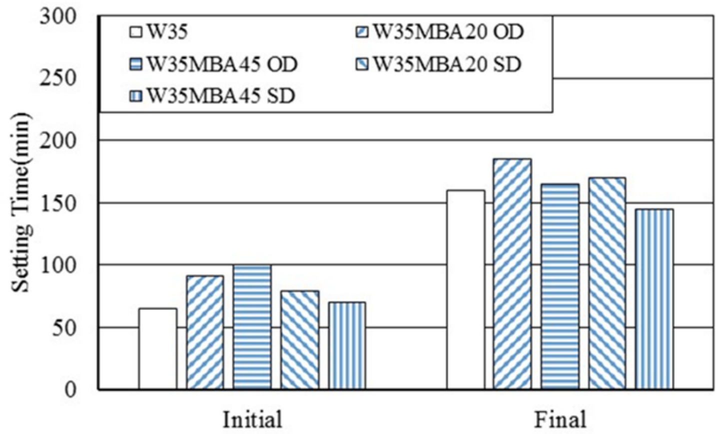


Figure 4.36 Setting time of MBA20OD, MBA45OD, MBA20SD and MBA45SD by W/B =0.35

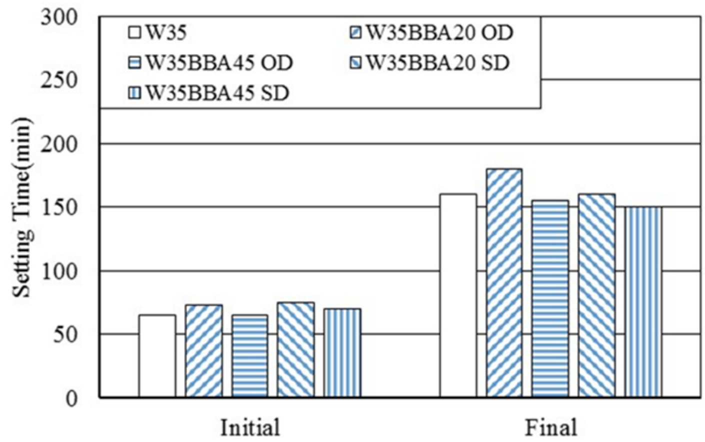


Figure 4.37 Setting time of BBA20OD, BBA45OD, BBA20SD and BBA45SD by W/B =0.35

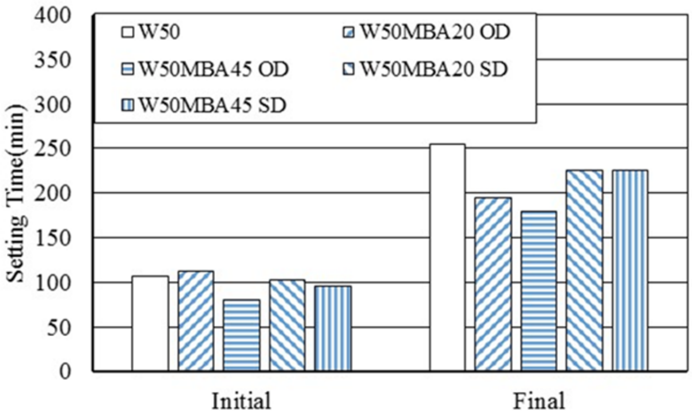


Figure 4.38 Setting time of MBA20OD, MBA45OD, MBA20SD and MBA45SD (W/B = 0.50)

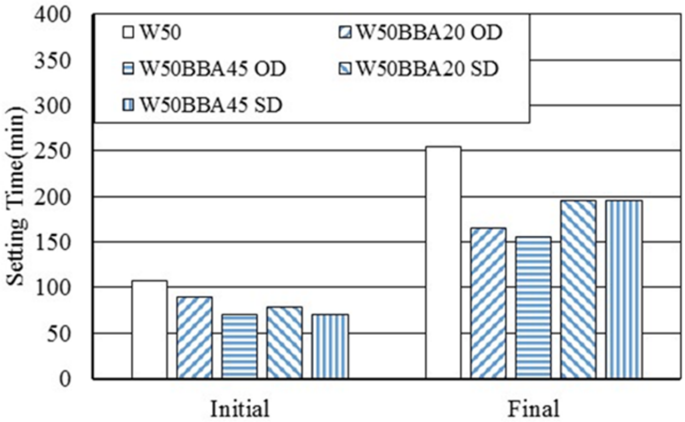


Figure 4.39 Setting time of BBA20OD, BBA45OD, BBA20SD and BBA45SD (W/B =0.50)

From W/B=0.50 in Figure 39 and Figure 40, when we instead MBA and BBA in mix proportion. Both of them won’t effect in initial and final setting time. MBAOD and BBAOD can absorbed the water in mix proportion more than NM aggregates. In case of saturated dry, when compared aggregates between NM aggregates and MBASD, the absorption of BA was higher than NM aggregates. BBA-SD in mix proportion the water in mix proportion was less than the mortar control because they calculated the mix design depend on the absorption of aggregates.

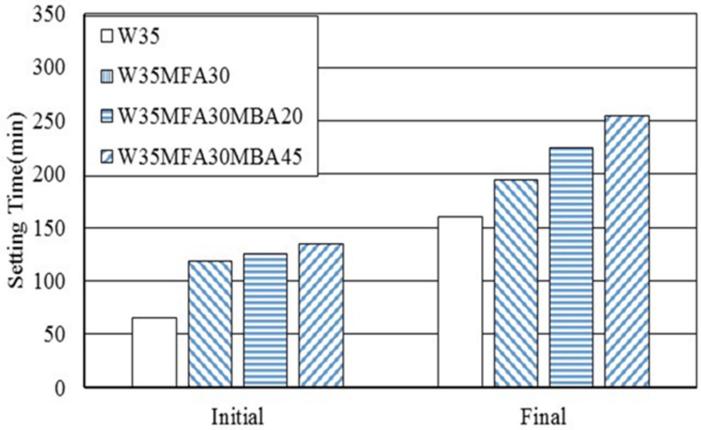


Figure 4.40 Setting time of MFA30, MFA30MBA20 and MFA30MBA45 W/B=0.35

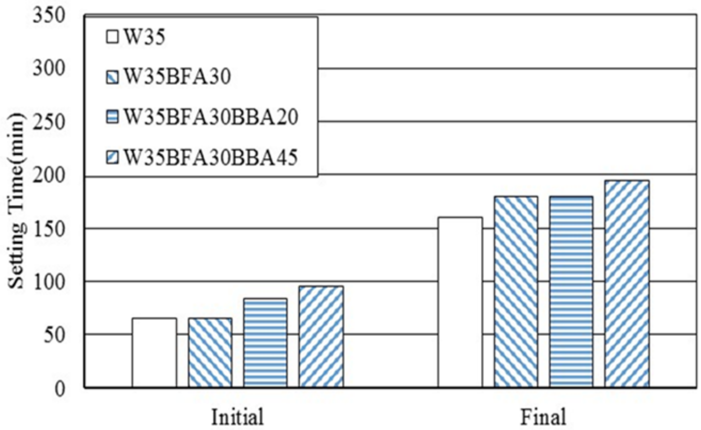


Figure 4.41 Setting time of BFA30, BFA30BBA20 and BFA30BBA45 (W/B=0.35)

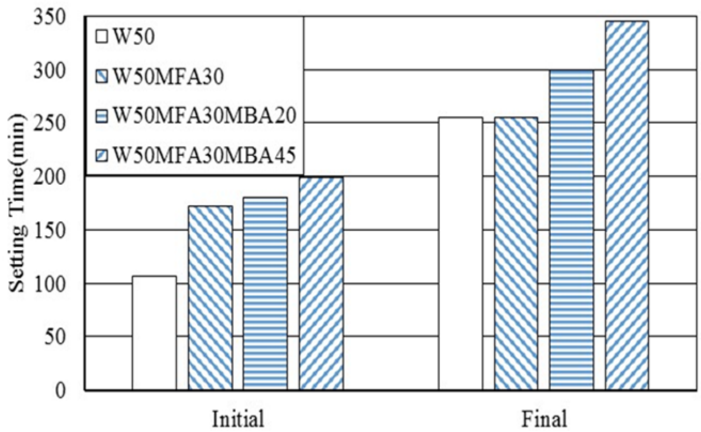


Figure 4.43 Setting time of MFA30, MFA30MBA20 and MFA30MBA45 (W/B=0.50)

Figure 4.41 to Figure 4.44 show that the setting time of initial and final delayed with increase of the replacement level of fly ash and bottom ash in water binder 0.35 and 0.50 when compare with mortar control caused from replacement of fly ash, the bottom ash and the finesse of cement was higher than fly ash. The rate of heat generation from reaction between cement and water will reduce when fly ash was replaced. So, the heat of hydration will decreased when reacts with water.

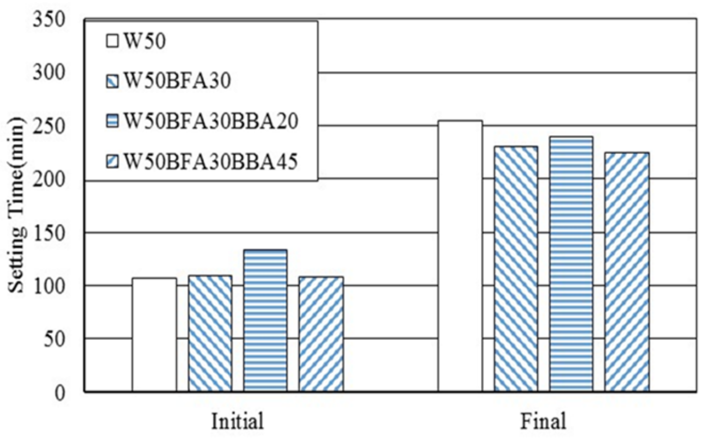


Figure 4.42 Setting time of BFA30, BFA30BBA20 and BFA30BBA45 (W/B=0.50)

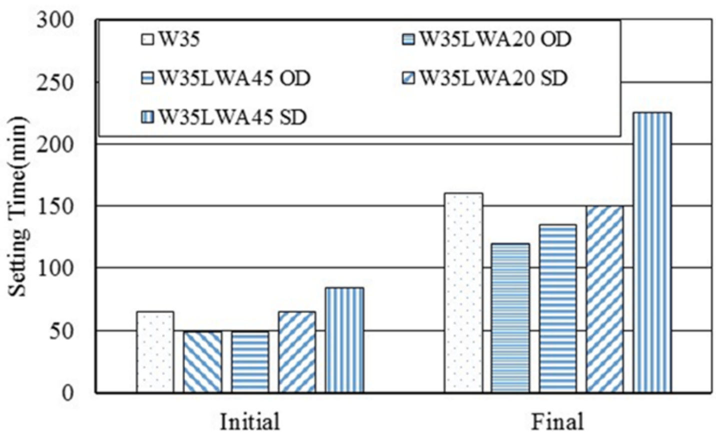


Figure 4.43 Setting time of LWA20OD, LWA45OD, LWA20SD and LWA45SD W/B=0.35

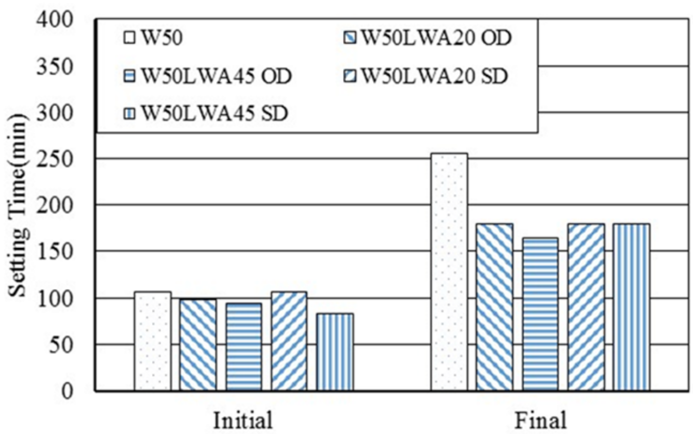


Figure 4.44 Setting time of LWA20OD, LWA45OD, LWA20SD and LWA45SD W/B=0.50

As can be seen from the Figure 4.45 and Figure 4.46, LWAOD does not affect in setting time with W/B=0.35 and 0.50. Saturated LWA (LWA-SD) was increased setting time because the water inside the LWASD spilled out from the aggregates that make water in cement paste was higher. Likewise, the particle of LWA was a material that absorbs water well with an increase in mortar. In case of LWASD the water in mix proportion was appropriated more than W/B=0.35.

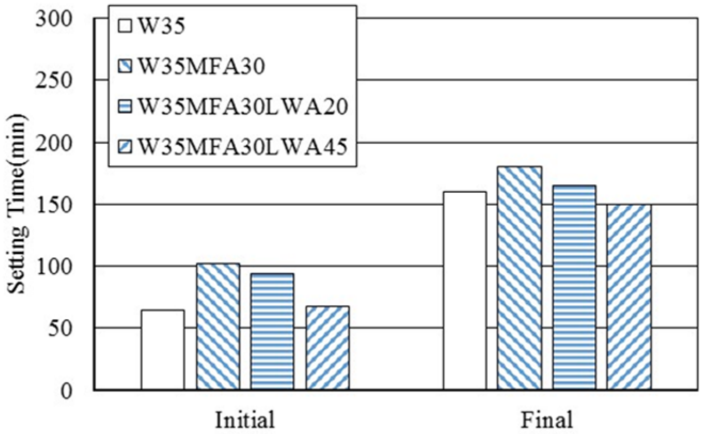


Figure 4.45 Setting time of MFA30, MFA30LWA20 and MFA30LWA45 W/B=0.35

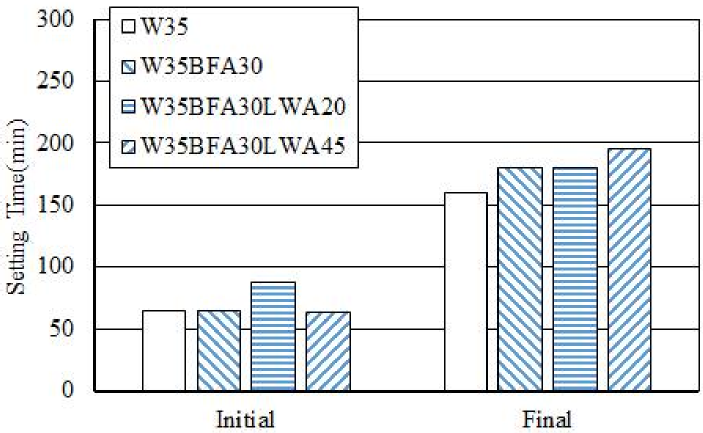


Figure 4.46 Setting time of BFA30, BFA30LWA20 and BFA30LWA45 W/B=0.35

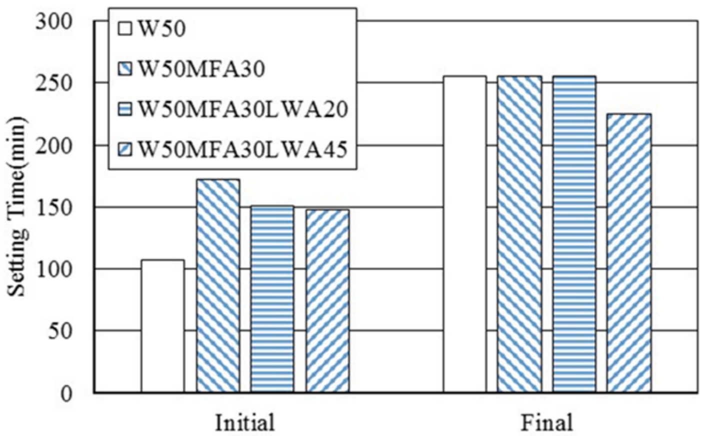


Figure 4.47 Setting time of MFA30, MFA30LWA20 and MFA30LWA45 W/B=0.50

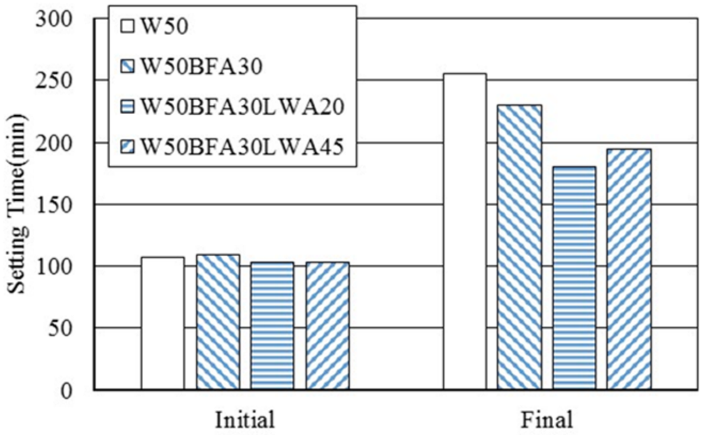


Figure 4.48 Setting time of BFA30, BFA30LWA20 and BFA30LWA45 W/B=0.50

Figure 4.47 to Figure 4.50 show that the setting time of initial and final delayed with increase of the replacement level of fly ash in water binder 0.35 when compare with mortar control caused from replacement of fly ash and the finesse of cement was higher than fly ash. The react of heat hydration between cement and water will reduce when fly ash was replaced. So, the heat of hydration will decreased when reacts with water.

4.6 Compressive Strength

The specimens of size 50 x 50 x 50 mm were used for compressive strength .According to ASTM C109, mortars would be taken out from the mold after 24 hours and curing the specimens sealed with wrap and kept in the container .Then compressive strength were tested at the age of 3, 7, 28 days.

Figure 4.51 shows the different of compressive strength for the period 3, 7 and 28 days. In particular, it compares the W35, W35L45, W50 and W50L45 mix mortar. As can be seen from the bar of W35 has a highest compressive strength of all ages at 3, 7 and 28 days which are 514, 593 and 647 ksc respectively and W35L45 grows the second largest sample whilst W35BFA30L45 grows the lest.

Figure 4.52 shows the different of compressive for the period 3, 7 and 28 days. In particular it compares the W50, W50FBA30, W50L45 and W50FBA30L45 mix mortar. As can be seen from the bar of W35 has a highest compressive strength of all ages at 3, 7 and 28 days which are 385, 404 and 449 ksc respectively and W50L45 grows the second largest sample whilst W50BFA30L45 grows the lest.

Although the water content of the lightweight aggregate had a significant influence on the autogenous shrinkage, it had practically no effect on strength (shown in Figure 4.51 and Figure 4.52.)

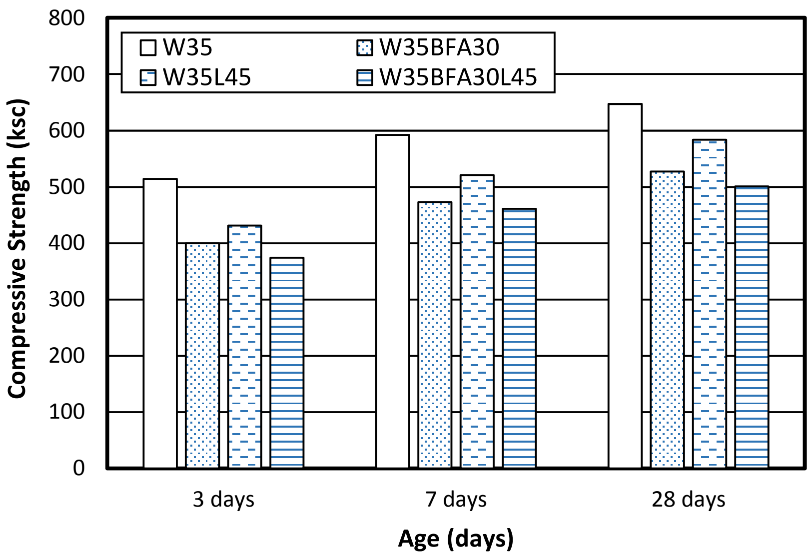


Figure 4.49 Compressive strength of mortar with w/b = 0.35

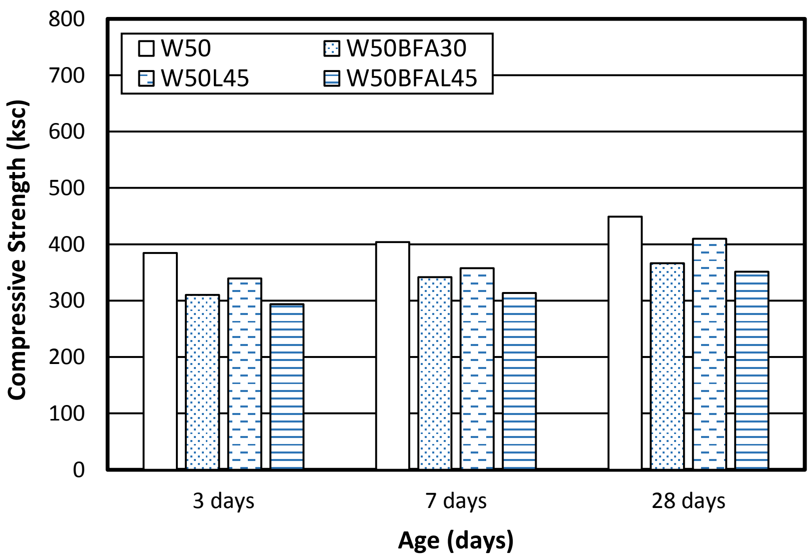
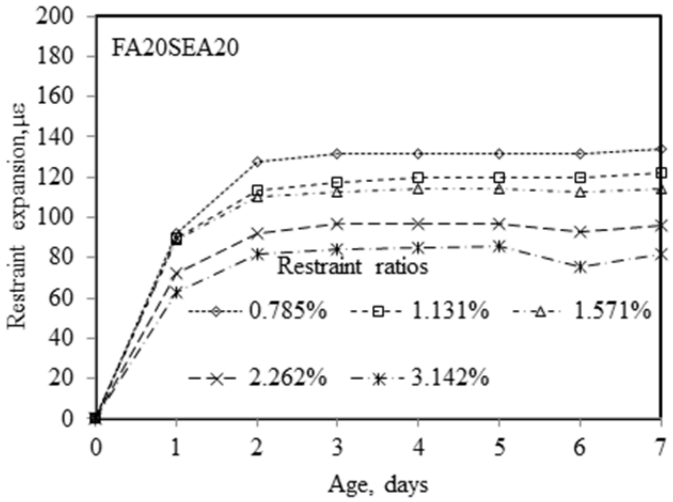


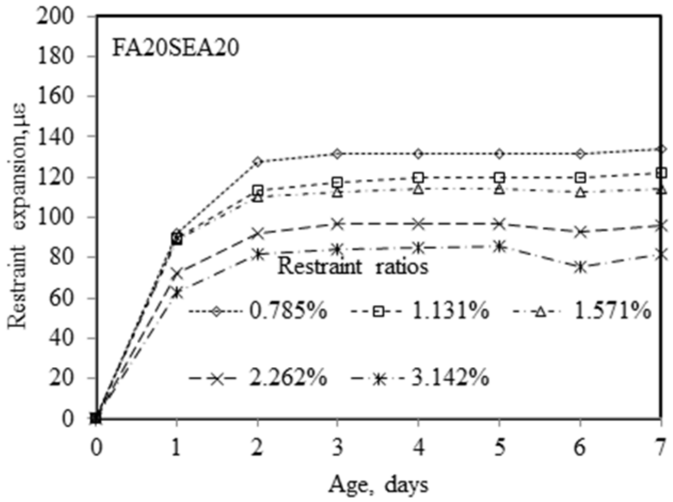
Figure 4.50 Compressive strength of mortar with w/b =0.5

4.7 Restrained Expansion of Expansive Concrete

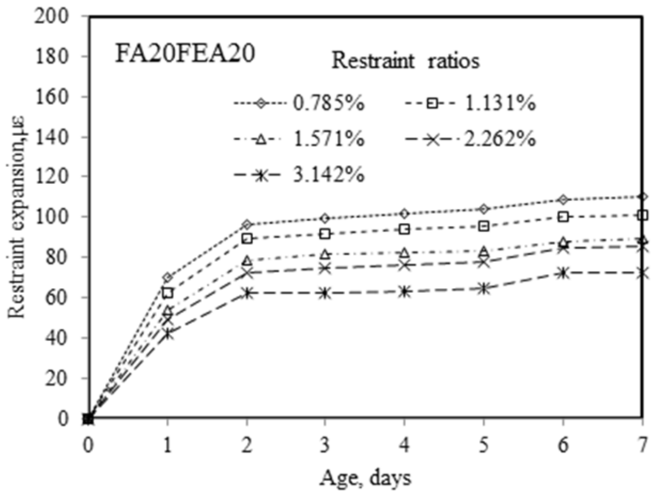
Error! Reference source not found. shows examples of development of restrained expansion. The expansion was rapidly induced during the first three days and became stable (almost constant) subsequently. These results suggested that, for expansive concrete, sufficient curing must be provided at least 2 days to allow the complete reaction of the expansive additives. It should be noted that longer curing (for instance, 7-day curing) can help reducing subsequent shrinkage and thus risk of shrinkage crack.



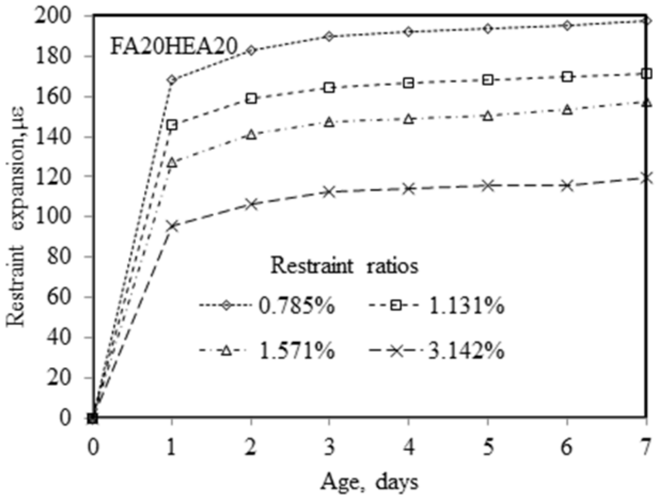
(a) Restrained expansion for FA20SEA20 under different restraint ratios



(b) Restrained expansion for FA20SEA20 under different restraint ratios



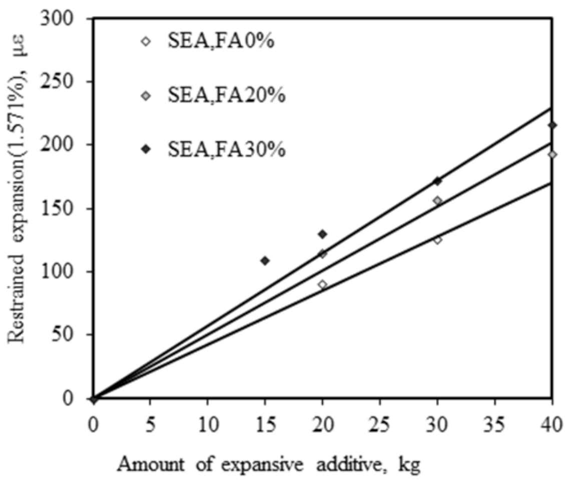
(c) Restrained expansion for FA20FEA20 under different restraint ratios



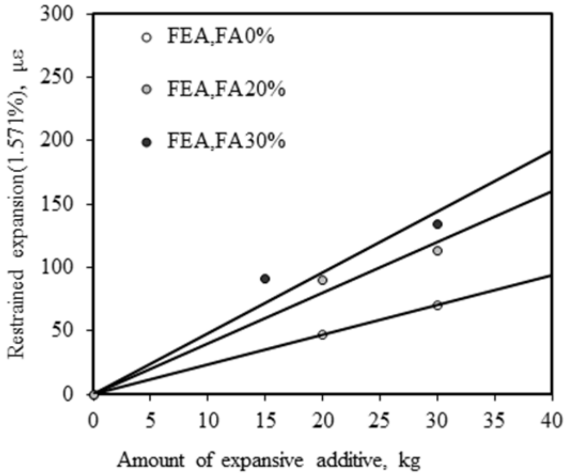
(d) Restrained expansion for FA20HEA20 under different restraint ratios

Figure 4.51 Example of development of restrained expansion

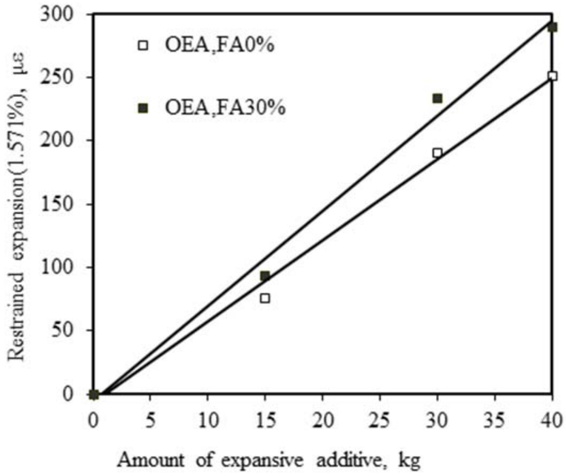
Error! Reference source not found. summarizes the restrained expansions, at the age of 7 days, of expansive concretes with different amounts of fly ash and expansive additives under moist curing condition. **Error! Reference source not found.** (a-d) show the restrained expansion of specimens with SEA, FEA, OEA, and HEA, respectively. Comparing these graphs, it is obvious that the use of different EAs gives different degrees of expansion. Among expansive concretes without fly ash, the one with HEA shows the highest expansion at the same dosage of EA and followed by the ones with OEA, SEA, and FEA, respectively. Tendency is also the same for the case of expansive concrete with fly ash. The difference in the expansion obtained from the use of different expansive additives may result from the difference in chemical composition, phases of chemical components, and fineness of each expansive additive. This point will be further investigated in the future.



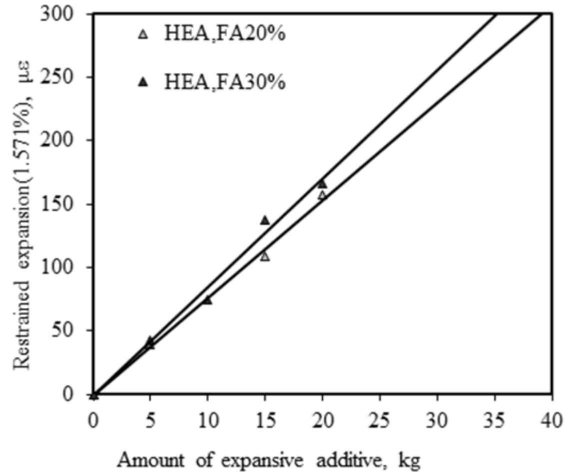
(a) Specimen with SEA



(b) Specimen with FEA



(c) Specimen with OEA



(d) specimen with HEA

Figure 4.52 Restrained expansion of specimen with 1.571% restraint

In **Error! Reference source not found.**, the result also indicate that, in all cases (both expansive concrete with and without fly ash, the restrained expansion (restraining ratio = 1.571%) have somewhat linear relationship with the dosage of expansive additive (in kg/m^3). This tendency was found to be also true in the case of other restraining ratios (0.785%, 1.131%, and 2.262%) This information is useful for the estimation of a required dosage of expansive additive that produces a specific value of expansion in a given restrained condition.

For all types of expansive additive used, the presence of fly ash increases the restrained expansion at 7 days. At the same dosage of expansive additive, concrete samples with 30% fly ash give clearly higher expansion than the concrete with 20% fly ash replacement and also the concrete without fly ash, respectively. This tendency shows that fly ash has good compatibility with all types of expansive additives. This phenomenon is still not fully explainable at present. However, it is believed that the higher expansion may result from at least three following reasons. The first reason is the higher Alumina content (Al_2O_3) in fly ash which may allow more formation of ettringite ($(CaO)_6(Al_2O_3)(SO_3)\bullet 32H_2O$) and the second reason is the lower early age stiffness of cement-fly ash paste system which allows more crystal growths of expanding products. The third is possibly the delay of reaction of expansive additive by fly ash.

In practice, the selection of suitable expansive additive and the estimation of required dosage are key procedures in the application of expansive concrete. It is very often that the performance of each expansive additive must be compared, not only for engineering judgment but also for cost comparison. The use of expansive additive in fly ash concrete may also sometimes be questioned. To response to these challenges, the new term ‘expansion efficiency’ is introduced to quantitatively illustrate the efficiency of each expansive paste system. In this study, the ‘expansion efficiency’ is defined as the expansion created by a unit amount of expansive additive in a given condition. The expansion efficiencies can therefore be calculated as slopes of graphs in **Error! Reference source not found.**

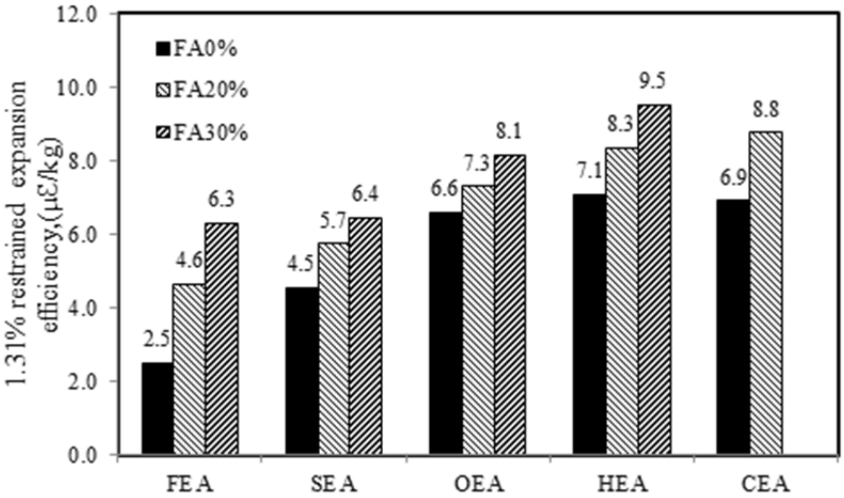


Figure 4.53 Expansion efficiency of each expansive additive (Restraint ratio =1.31%, w/b=0.5)

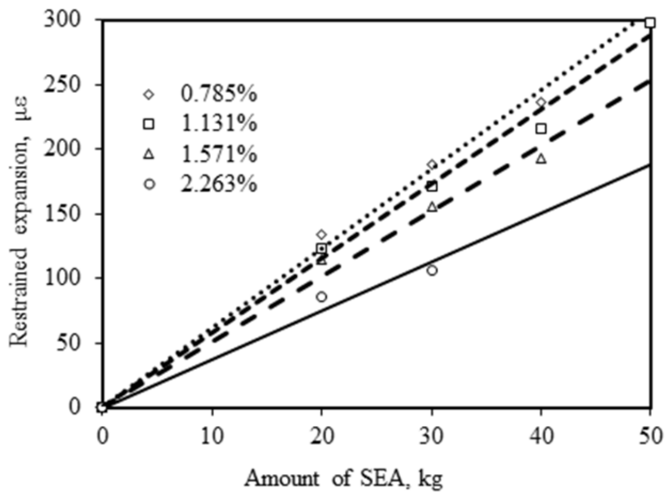


Figure 4.54 Restrained expansion of SEA with 20% FA under different restraint ratios

Error! Reference source not found. compares the expansion efficiencies of the expansive additives tested in this study, which allows a simple comparison of different types of expanding paste system. It can be simply concluded that, at w/b ratio of 0.5, using fly ash at 30% of total binder increases the expansion efficiency of the concrete by at least 15% (when compared to concrete without fly ash). It is noted however that the expansion efficiency was derived from the test results of concrete with w/b ratio of 0.5 and restraint ratio of 1.571%. The expansion efficiency may be substantially changed if the w/b ratio or restraint ratio is changed. **Error! Reference source not found.** shows the relationship between restrained expansion and amount of SEA in the case of concrete with 20% fly ash replacement under different restraint ratios. It is clear that the relationship between dosage of expansive additive and the restrained expansion seems to be linear for the range of restraint ratio from 0.785% to 2.263%. The slope of the graph (and thus expansion efficiency) reduces when the restraint ratio increases.

Figure 4.56 shows an example of a reduction of expansion efficiency (fly ash concrete with 20% fly ash replacement and SEA) due to the increase of restraint ratio. The reduction of restrained expansion due to the increase of restraint seems to be non-linear.

Figure 4.57 (a-d) shows the effect of restraint ratio on restrained expansion at 7 days of all expansive concretes tested in this study. Similar trend could be obtained for all cases. From the results in **Error! Reference source not found.** (a-d), the relative expansion was calculated as a normalized expansion to the expansion at the restraint ratio of 1.131%.

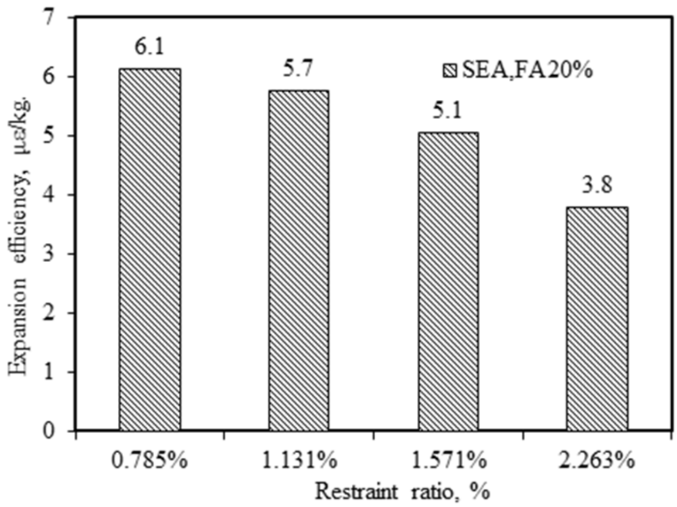
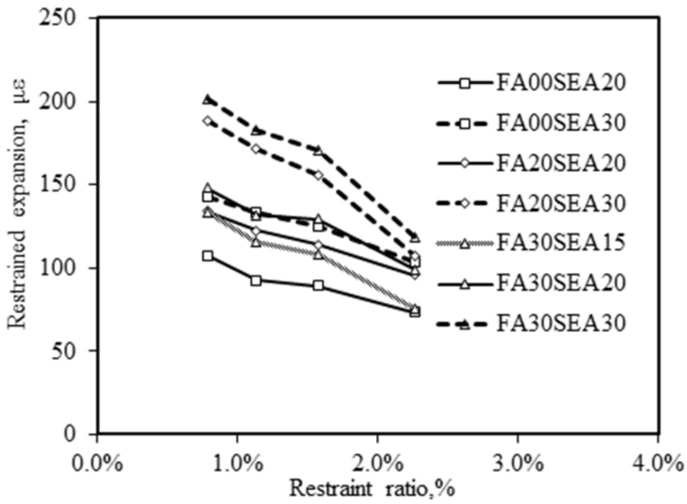
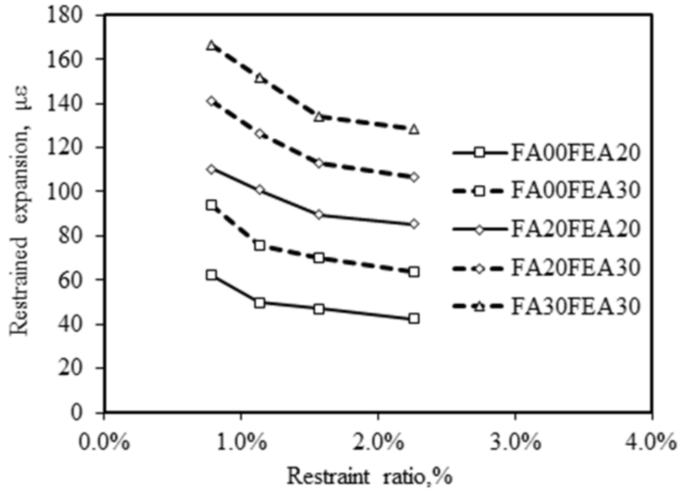


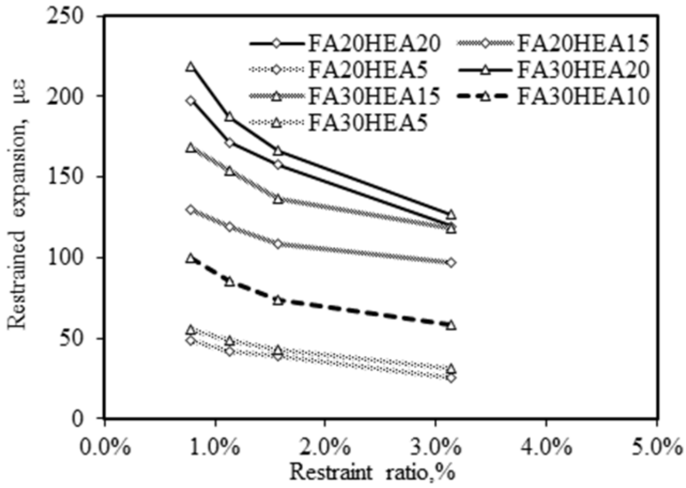
Figure 4.55 Expansion efficiency of SEA with FA20% on different restraint ratios



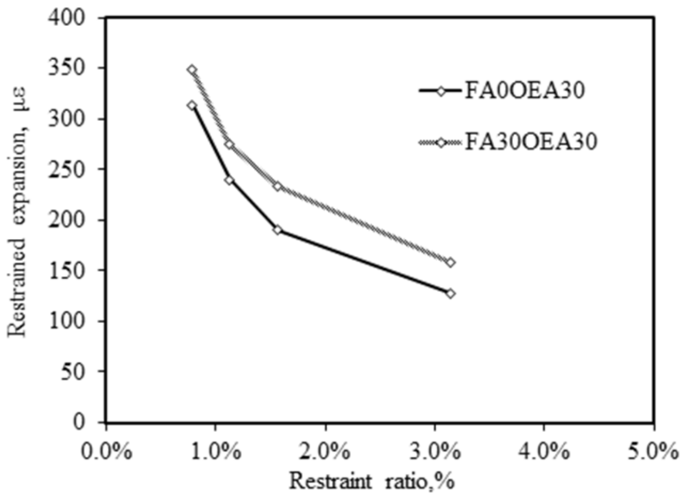
(a) Specimens with SEA



(b) Specimens with FEA

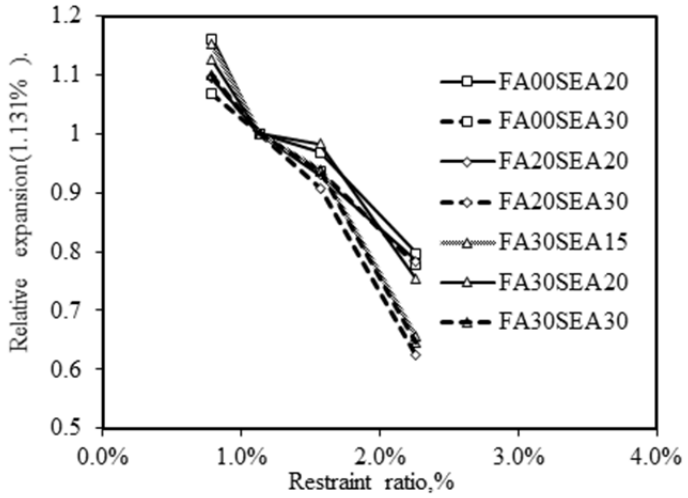


(c) Specimens with HEA

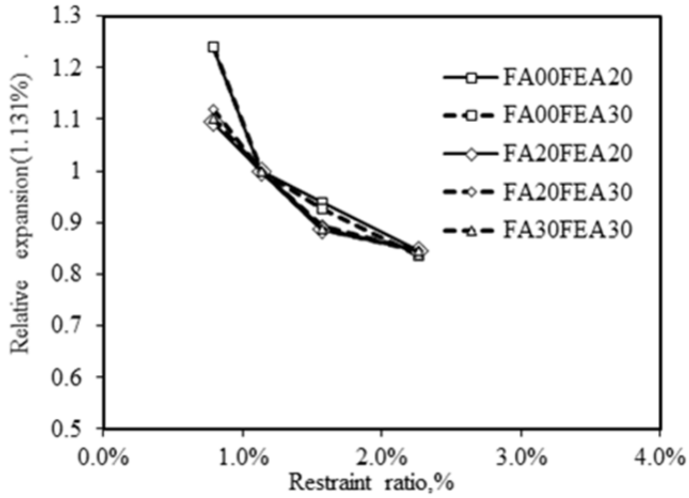


(d) Specimens with OEA

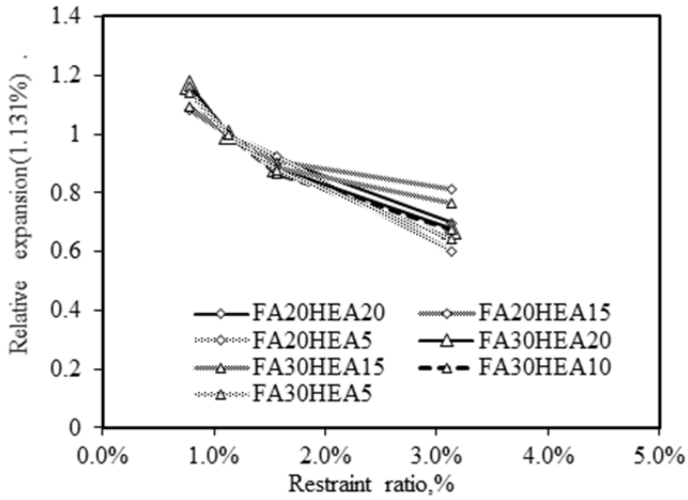
Figure 4.56 Effect of restraint ratio on the restrained expansion at 7days



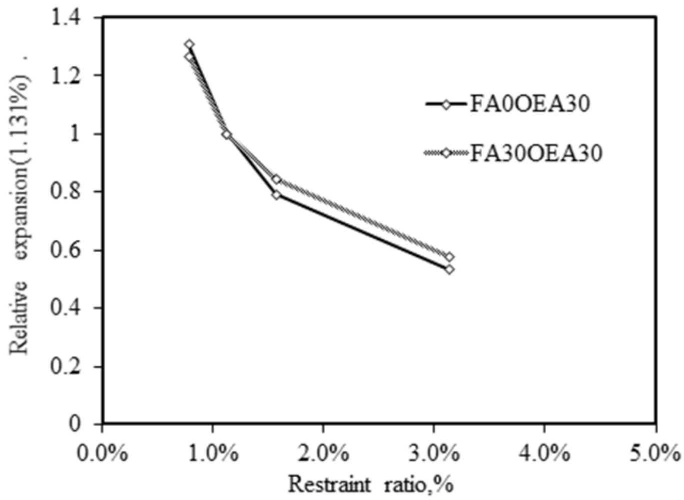
(a) Specimens with SEA



(b) Specimens with FEA



(c) Specimens with HEA



(d) Specimens with OEA

Figure 4.57 Relative expansion

By adding bottom ash into the normal expansive concrete to induce internal curing, it was found that a replacement of 10% in fine aggregate could increase 7-day free expansion by 14% (figure 59) when compared to the expansive concrete without BA.

This effect is also observed in fly ash expansive concrete. The results in **Error! Reference source not found.** show that the 7-day expansion increases by 24% in fly ash expansive concrete with BA10% when compared to the fly ash expansive concrete without BA.

For the effect of fly ash on 7-day free expansion of mixtures with no BA, **Error! Reference source not found.** shows that the replacement of 20% fly ash in total binder (FA20EA20) increases the expansion by approximately15% when compared to expansive concrete with no fly ash (FA00EA20).

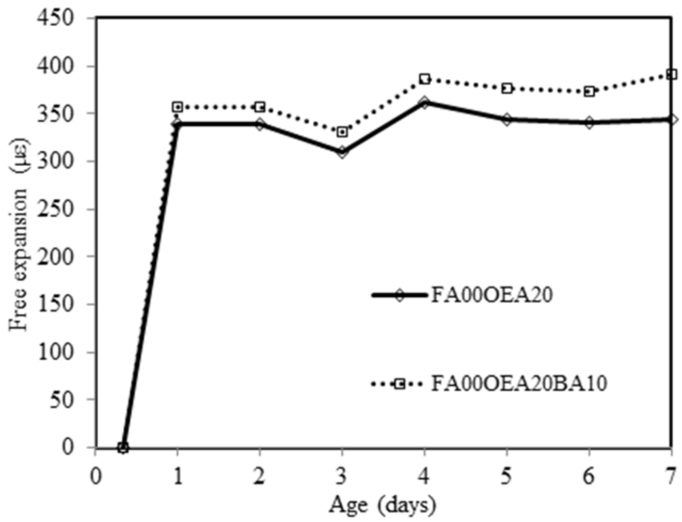


Figure 4.58 Effect of BA on free expansion of normal Expansive concrete

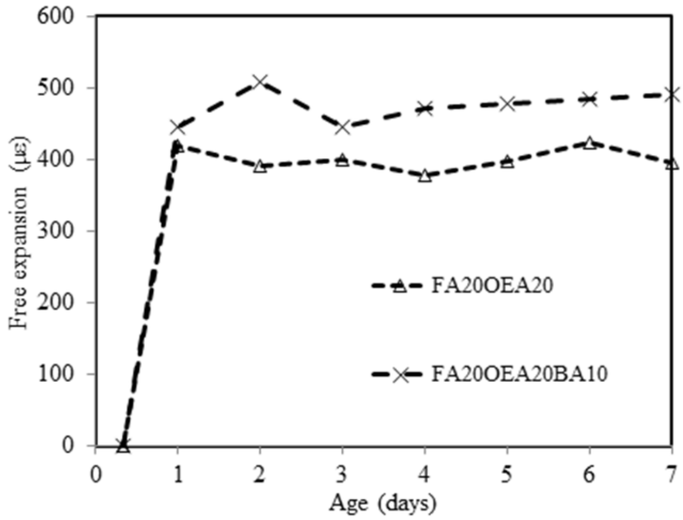


Figure 4.59 Effect of BA on free expansion of Fly Ash Expansive concrete

However, for mixtures with BA an increase of 25% expansion over the mixture without fly ash was obtained when using fly ash with BA (Comparing FA20EA20BA10 with FA00EA20BA10). Moreover, the results of mixtures with 10%BA (FA00EA20BA10) and the mixture with 20%FA (FA20EA20), show that the 7-day free expansion can increase about 14% and 15%, respectively when compared to the control mixture with no FA and no BA (FA00EA20).

The synergy between BA and FA significantly improves expansion of the mixture with both BA and FA when compared to the effect of only fly ash or only bottom ash. If can be seen that the tested 7-day expansion of expansive concrete with both BA and FA increases up to 43% when compared to normal expansive concrete with no BA and no FA (**Error! Reference source not found.**).

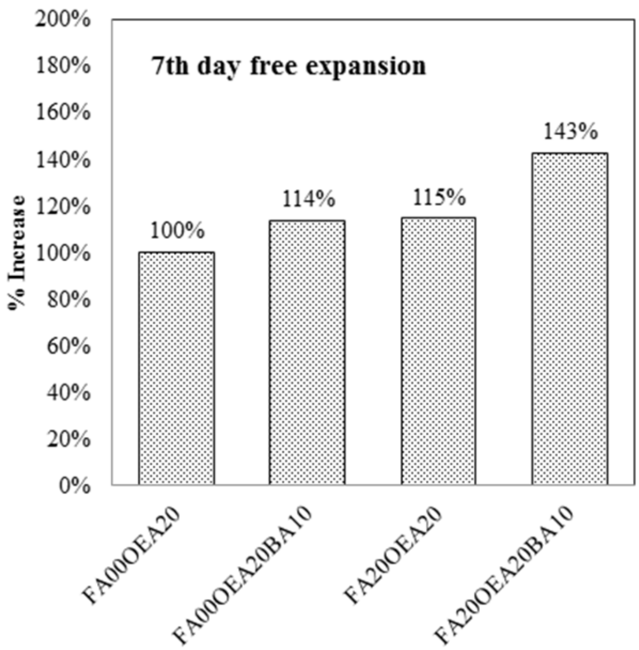


Figure 4.60 Summary results of 7-day free expansion of expansive concrete with FA and BA

Internal curing by using bottom ash at 10% replacement in fine aggregate can increase restrained expansion on the 7th day by 9% when compared to normal expansive concrete with no BA (Figure 4.62). It can also be found that the expansion increased by about 20% for fly ash expansive concrete with BA.

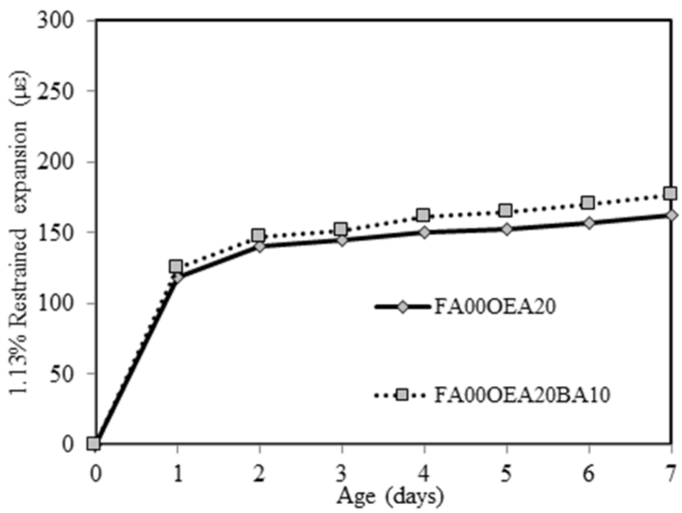


Figure 4.61 Effect of BA on 7-day restrained expansion of normal expansive concrete

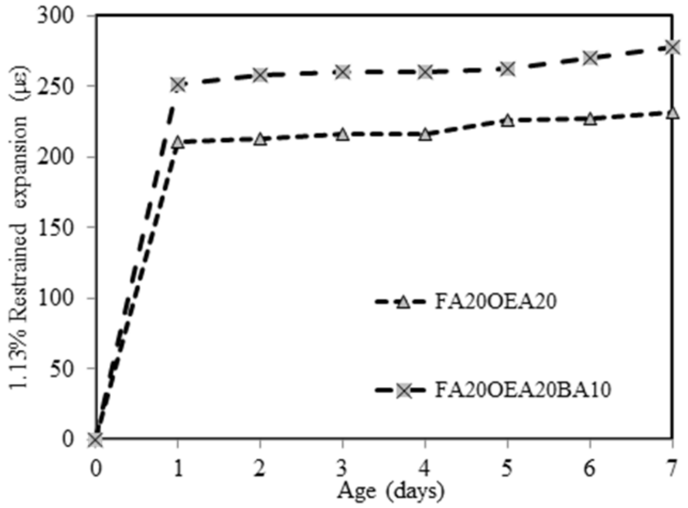


Figure 4.62 Effect of BA on 7-day restrained expansion of normal expansive concrete

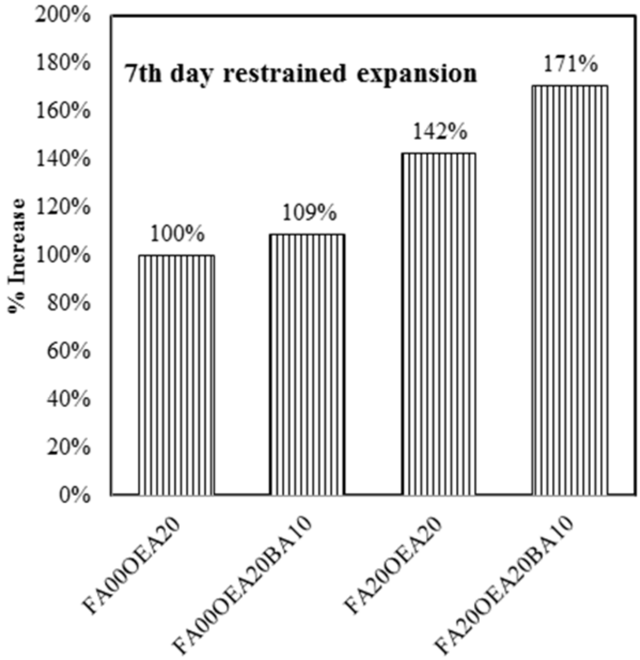


Figure 4.63 Summary results of 7-day restrained expansion of expansive concrete with FA and BA

Figure 4.63 shows that using both FA and BA increases 7-day restrained expansion in expansive concrete, similar to the results in free expansion condition. The application of both FA and BA gain more efficiency by increasing the restrained expansion to 71% when compared to the reference mixture with no BA and FA. The reason that bottom ash can increase expansion of the expansive concrete is probably because bottom ash, by internal curing effect, provides water for the reaction of the expansive agent at the inner part of the concrete where curing water is not possible to be supplied from the normal curing from outside.

It is noted that the effectiveness of shrinkage compensation also depends on the amount of shrinkage of the mixture. If expansion of the mixture is improved, but shrinkage of the mixture is on the other hand much higher, the shrinkage compensation may not be so effective. It is therefore necessary to evaluate the shrinkage behavior of the expansive concrete for a more relevant evaluation. As bottom ash is porous with high water retainability, it is expected to reduce autogenous shrinkage but may increase drying shrinkage of the concrete. The results in **Error! Reference source not found.** show the subsequent shrinkage after curing (after 7-day expansion). The shrinkage up to the age of 49 days of expansive concrete with internal curing is similar to that of the normal expansive concrete. However, at long term, the shrinkage of expansive concrete with internal curing by BA becomes slightly larger than that of the normal expansive concrete. This may be because, when the bottom ash which has high porosity is added into the concrete, the microstructure is not as dense as that of the concrete with natural fine aggregate. The moisture can thus evaporate and migrate out of the sample more easily. This is more obvious at long-term.

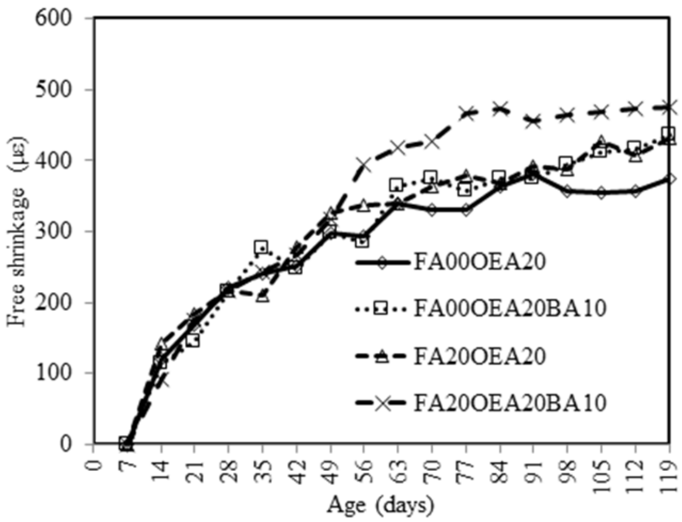


Figure 4.64 Total shrinkage of expansive concrete with BA

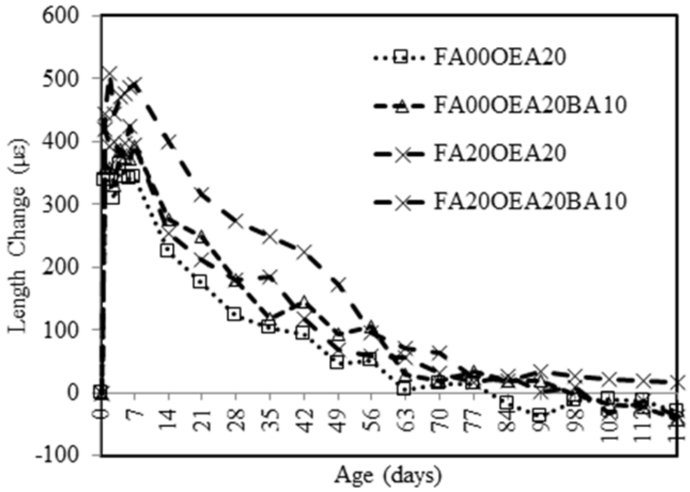


Figure 4.65 Total length change of expansive concrete with BA

Error! Reference source not found. and **Error! Reference source not found.** show that total length change at long term of expansive concrete with bottom ash is approximately same as that of the expansive concrete without bottom ash.

Higher expansion of expansive concrete with fly ash was probably due to the delaying effect of fly ash on the reaction of expansive additive, higher alumina content of the fly ash and lower stiffness of pastes during expansion period, inducing more opportunity to expand. More expansion of expansive concrete with bottom ash was also found due to mitigating autogenous shrinkage and supply of water for reaction of the expansive additive by internal curing. Future investigations are required for confirmation of these mechanisms.

The experimental findings indicate that internal curing can potentially be applied to increase efficiency of expansive concrete however the balance between the enhanced expansion and the subsequent drying shrinkage must be carefully considered. It should be noted that since there is still no study done on the effect of bottom ash on expansion and shrinkage behavior of expansive concrete, for real application in the future, it is necessary to investigate the mechanisms on how fly ash affects the expansion of expansive concrete as well as the quantitative evaluation of expansion of internally cured expansive concrete under different degrees of restraint for possible design in the future.

Chapter 5: Degree of Restraint

5.1 Investigation on Cracks in Real Reinforced Concrete Structure

5.1.1 Shear wall

The data of the cracks which occur in shear wall were measured the crack width by digital microscope. For the cracks, its direction and the time of occurrence within 3 days after casting from thermal contraction crack as well as the autogenous shrinkage crack. The more it has got high strength, the more risky to be impaired in high restraint.

The location of crack mapping of Basement 2 and Basement 3, the minimum and maximum of number of cracks at North Tower, respectively that Basement 2 has got 3 cracks and Basement 3 has got 14 cracks (see in Table 5.1)

The basic information of shear wall is 12 meters long with the end column for wall length, 300 to 400 millimeters for the wall thickness and 3500 millimeters for the wall height.

In concrete, 28 days strength is 450 kg/cm^2 and slump has 17.5 ± 2.5 centimeters

For reinforcement, 300 millimeters for the thick wall of DB12@300m, 400 millimeters for the thick wall of DB16@300m and 40 millimeters of the cover clearance.

In the field investigation, we would identify cracks including repaired cracked, record its location as well as measure the widths of cracks (see in Figure 5.1 and Figure 5.2)

Table 5.1 Type of concrete and reinforcement at North Tower

Floor	Concrete	Reinforcement	Number of Cracks	Max. Crack Width (mm)
B1	C	DB16 @ 150 mm	4 Cracks	0.164
B2	B	DB16 @ 150 mm	3 Cracks	0.243
B3	A	DB16 @ 300 mm	14 Cracks	0.306
B4	A	DB16 @ 300 mm	12 Cracks	0.188

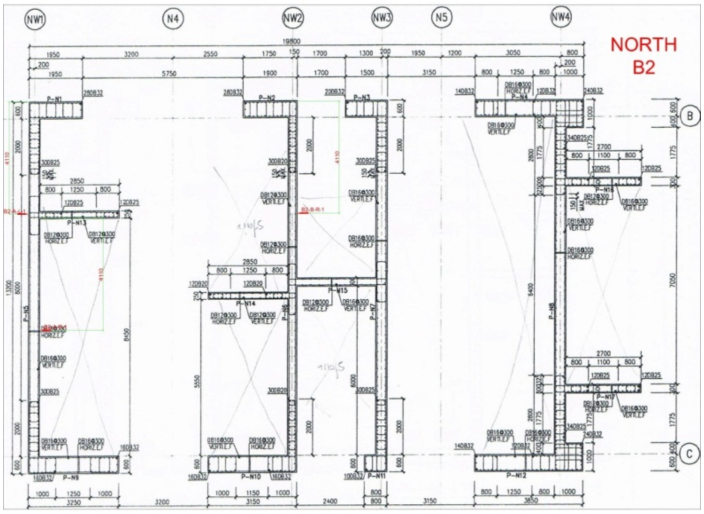


Figure 5.1 Crack Map of North Tower (B2 Floor)

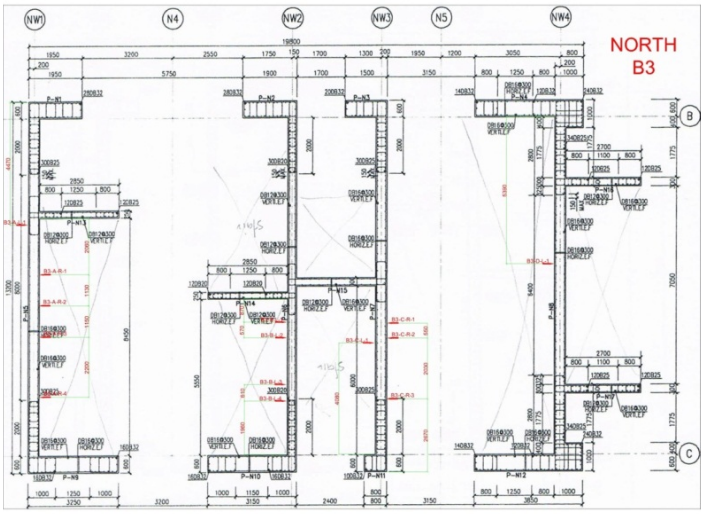


Figure 5.2 Crack Map of North Tower (B3 Floor)

Shear wall is designed to resist the gravity force likes vertical load as well as the lateral force likes wind load and earthquake. For the vertical, the cracking will reduce the rigidity of shear wall but it would not affect to vertical load carrying capacity seeing that it would work on the lateral instead at side sway. Then, for the lateral at the resistance, the force would be distributed to the boundary zone around the edges of the wall and at the effect of lateral; each crack would interact that the shear wall would have to resist more load and moment at the middle part and the sway will also increase as well.

5.1.2 Industrial Floor

For the general causes, there are excessive or unexcessive loading that crack takes the place without any load applied, differential settlement that the direction of cracks are vertical which is not relevant to the settlement crack and durability problems which the most likely causes of deterioration.

However, the number of cracks is able to reduce by increase aggregate content, replace cement with fly ash, and slightly increase water content.

For the prevention, in modification, there is limitation of specification that they focus on strength as well as slump and require prescribed strength earlier than real need and in the reparation, the integrity of overall shear must be restored.

Table 5.2 Mix proportion of concrete

Mix	Cement	Fly Ash	Water	Sand	Gravel	w/b	%FA
A	450	50	160	730	1070	0.32	10%
B	420	80	165	680	1095	0.33	16%
C	376	95	165	785	1005	0.35	20%

Table 5.2 is the mix proportion of concrete that is used in each floors, it is clearly that both Concrete B and C which have the greater w/b and %FA than A are used in basement 1 and 2 have small number of cracking in basement 3 and 4.

In the 2nd year, this document collects the data of the cracks which occur in industrial floor are measured the crack width by crack-mapping, crack-width, and microscope. The longest crack about 11 m and the width crack as to 0.1 to 1.0 mm.

Classifying the crack, there are lengthwise crack that both vertical and horizon which is similar to , +, plus symbol (see in Figure 5.3), Mostly found the cracks around Pile Cap position as well as diagonal crack which crack from the angle pile, stretch along around 45 degrees

Repaired crack which in the area that has already been repaired and found more cracks again, pass the joint which continuous type of crack pass the joint (see in Figure 5.4) and steel row which found the crack parallel with each other vertically as a rule following the steel way.

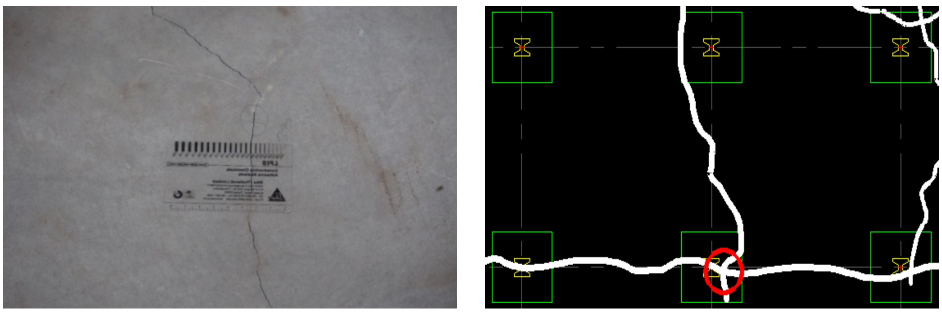


Figure 5.3 lengthwise crack

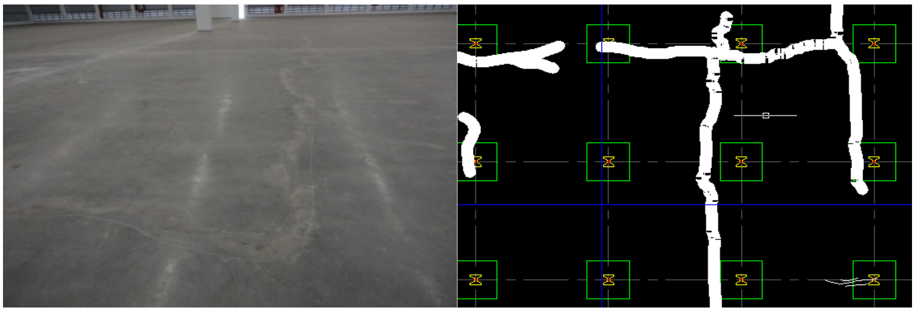


Figure 5.4 Crack that pass the joint

From the angle which is at the coring of concrete around the intersection cracking that exact to the Pile Cap position (see in **Error! Reference source not found.**) and random crack that is able to be found the striped crack randomly disperse the entire factory floor.

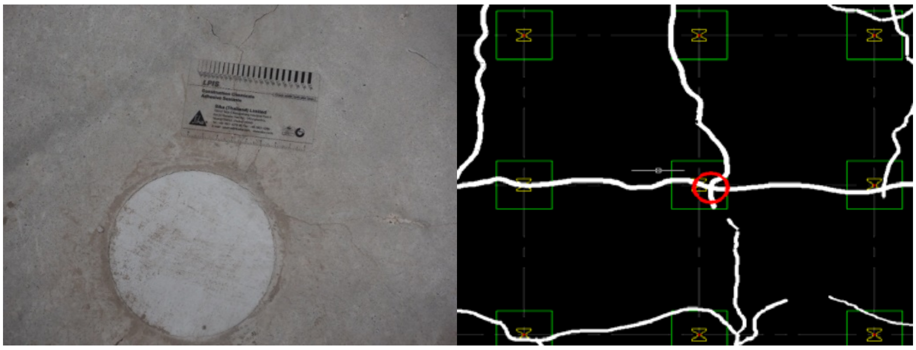


Figure 5.5 Crack from the angle

5.2 Analysis for Degree of Restraint

5.2.1 Degree of Restraint (DOR)

Degree of restraint of concrete is an important parameter which must be accurately assessed for the design for crack prevention. Currently there is no standard guideline for the selection of degree of restraint for each type of structural components probably because the degree of restraint cannot be directly evaluated.

In this section, the numerical technique called “Finite Element Method” is applied in order to evaluate the degree of restraint of each type of structural element. The value obtained from this analysis may be used as a parameter for design against shrinkage crack.

In this study, the “Degree of Restraint (DOR)” is defined as the ratio of the restrained tensile strain to the free shrinkage of the material. Theoretically, the value of DOR must be positive number in a range between 0 and 1. The interpretation of DOR value is described in Table 5.3

Table 5.3 Degree of Restraint (DOR)

DOR	ความหมาย	ผลของการหดตัว
0	No Restraint	No development of tensile strain
0 < x < 1	Partial Restraint	Shrinkage partially develop into tensile strain
1	Full Restraint)	All shrinkage turns into tensile strain

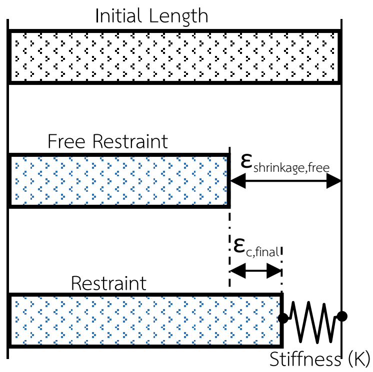


Figure 5.6 Strain due to restraint

The “Degree of Restraint” can be classified into the “internal restraint” and “external restraint”.

The internal restraint usually occurs when there is unbalanced shrinkage/expansion of concrete or the different volume change between concrete and the reinforcing steel bar whereas the external restraint is the restraint induced by adjacent structural members. Examples of structures which is subjected to the external restraint is the concrete wall sit on large concrete footing or concrete wall which links columns or the concrete floor which is supported by large beams, etc.

In the case of concrete wall sit on footing, the effect from external restraint is usually very large. This is because usually wall is constructed quite after the completion of footing construction. In such manner, the shrinkage of wall is usually large and restrained by the footing.

The value of external restraint depends on the restraining condition of each structure as shown in Figure 5.6. It can be seen that the degree of restraint depends on the stiffness of the adjacent structure as well as the length of the main structure.

The external restraint can be calculated by the following equations

$$DOR(\text{External Restraint}) = \frac{KL}{E_c A_c + KL} \tag{1}$$

$$\varepsilon_{c,final} = \left(\frac{KL}{E_c A_c + KL}\right)\varepsilon_{shrinkage,free} \tag{2}$$

Where, K is stiffness of the restraining adjacent structure

L is the initial length of the structure under consideration

E_c is the modulus of elasticity of concrete

A_c is the cross-sectional area of concrete

$\varepsilon_{shrinkage,free}$ is free shrinkage strain

$\varepsilon_{c,final}$ is restrained strain

Equation (1) and Equation (2) is however not realistic because the real concrete wall structure will be restrained at the bottom surface. The restraint degree in each position in the wall is therefore different. Therefore, the equations are not appropriate for the calculation for degree of restraint of the wall constructed on the concrete footing. In this study, the finite element method (FEM) is used to evaluate the degree of restraint instead.

5.2.2 Simulation Model of Concrete Wall and Concrete Footing

In this analysis, the concrete wall sitting on concrete footing is modeled as shown in Figure 5.7. Dimensions of the wall and footing are varied in the analysis. T_w is wall thickness, H_w is wall height, H_f is the thickness of footing, W_f is width of footing and L is the length of both wall and footing. The values of these paramebers are shown in Table 5.4

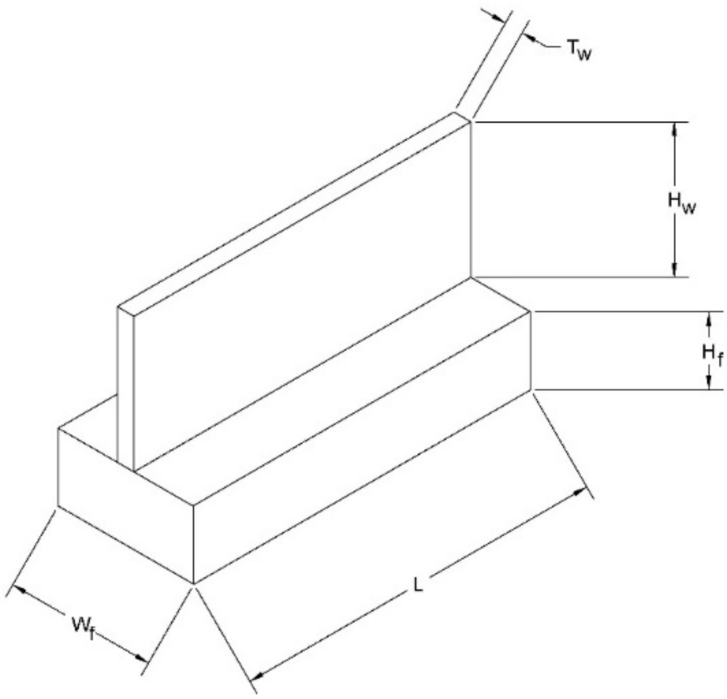


Figure 5.6 Simulation Model for Concrete Wall and Concrete Footing

Table 5.3 Dimension and size of wall & footing model

Dimension	Size (m)
T_w	0.1, 0.2, 0.25, 0.3, 0.4 and 0.5
H_w	0.5, 1, 1.5, 2, 2.5, 3 and 3.5
H_f	0.5, 1, 1.5, 2, 2.5, 3, 3.5 and 4
W_f	1
L	1, 2, 3, 4, 5 and 10

Table 5.4 Material Properties of Concrete

Properties	Wall	Footing
Coefficient Thermal Expansion	$1\text{x}10^{-5}/1\ ^{\circ}\text{C}$	0
Young's Modulus of Elasticity	$3\text{x}10^{10}$ Pa	$3\text{x}10^{10}$ Pa
Poisson's Ratio	0.18	0.18

The boundary conditions of the model are also varied. There are two cases of boundary conditions being assumed. In the first case, the bottom surface of concrete footing is assumed to be perfectly fixed with the underneath surface. While in the other case, it is assumed that bottom surface of concrete footing can slide on the underneath surface without friction. These two cases of analysis will be compared. The interface between concrete wall and concrete footing is assumed to be perfectly bonded.

The material properties is assumed as indicated in Table 5.4

The size of element (mesh element size) is set as 100 mm and the element type is 3D solid element as demonstrated in Figure 5.7.

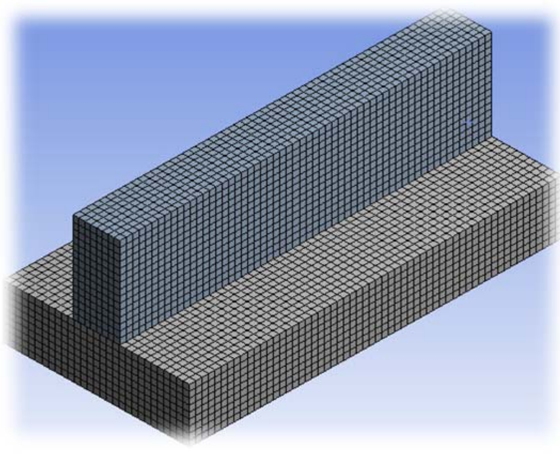


Figure 5.7 3D solid elements

5.2.3 Determination of Degree of Restraint in Concrete Wall by Finite Element Method (FEM)

The analysis of degree of restraint in the concrete wall structure is performed by assuming volume change is caused by thermal change. The analysis is linear as it is valid for the small scale of stress involved in the general cracking problem.

Value of shrinkage is assumed as contraction induced by temperature change. The value of free shrinkage ($\varepsilon_{shrinkage,free}$) is selected as -200 $\mu\varepsilon$. The corresponding change of tempertature is thus -20 $^{\circ}$C. Initial temperature of all element is set at 22 $^{\circ}$C and the final temperature is set at 2 $^{\circ}$C in order to achieve the prescribed contraction.

Degree of restraint at each position is, as shown in Equation 3, evaluated a a ratio between the restrained strain ($\varepsilon_{c,final}$) obtained from the analysis to the free shrinkage strain ($\varepsilon_{shrinkage,free}$) which is set at -200 $\mu\varepsilon$ in this case

$$DOR = \frac{\varepsilon_{c,\,final}}{\varepsilon_{shrinkage,\,free}} \quad (3)$$

5.2.4 Distribution of Stress, Strain, and Degree of Restraint (DOR)

Figure 5.8 demonstrates value of DOR at each height (0 – 3.5 m) in the wall at the center of length (L/2) and at quarter of the length (L/4). This is result from “fixed support” case. The result shows that the degree of restraint (DOR) is higher at bottom and the value is less at higher position. The maximum DOR value of 8.03 occurs at the bottom of wall at mid-length. The minimum value of DOR occurs at the topmost of wall and the value is -0.146. The negative value indicates compressive strain).

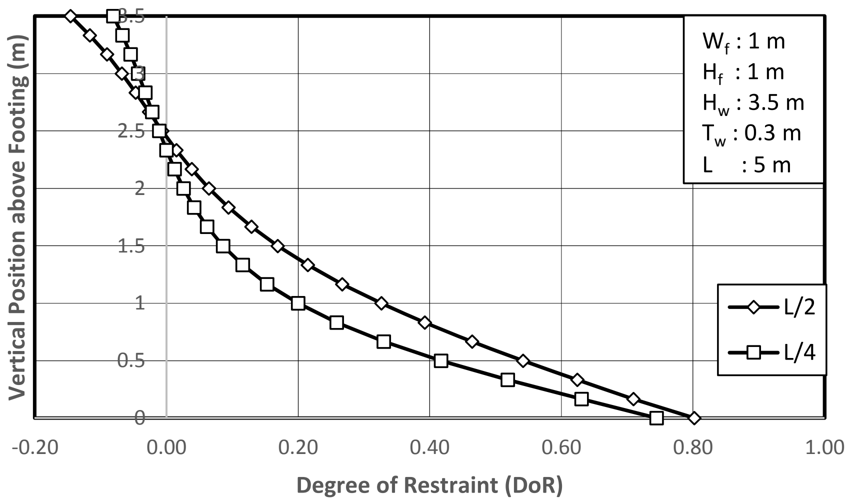
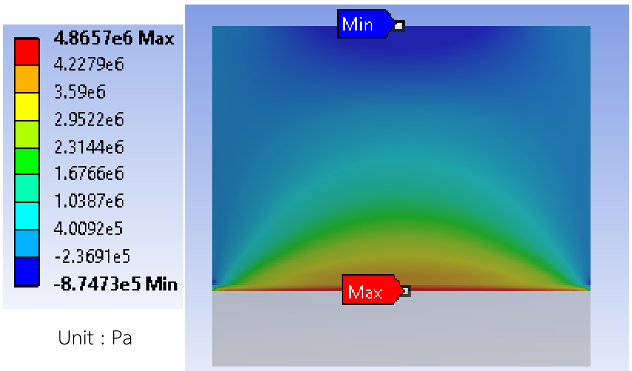
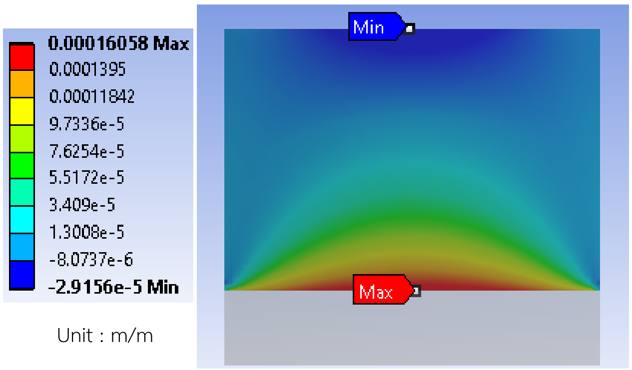


Figure 5.8 Relationship between DOR and height of wall

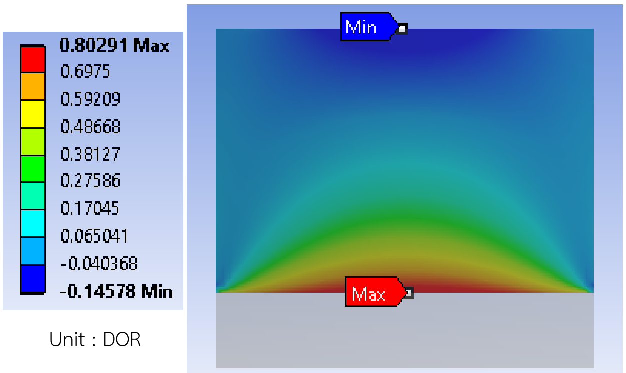
(L=5 m, H_w=3.5 m, T_w=0.3 m, H_f=1 m และ W_f=1 m)



(a) Stress distribution



(b) Strain distribution



(c) Distribution of degree of restraint (DOR)

Figure 5.9 Stress distribution in wall

(L=5 m, H_w=3.5 m, T_w=0.3 m, H_f=1 m และ W_f=1 m)

Figure 5.9 shows the results of stress distribution, strain distribution, and degree of restraint. The position of the maximum value is at the center of bottom face of the wall. Because the wall is symmetrical the stress/strain distribution is symmetric. At the same time, the minimum value occurs at the center of top face of the wall.

It can be seen, from the characteristics of the stress distribution, that the DOR value of any case will be maximum at the middle of the bottom surface. The restrained strain is highest at this point and if cracking takes place, it will always appear here. And, in order to compare the degree of restraint of structure with different geometries. The degree of restraint at the center of bottom surface of wall is mainly considered.

Figure 5.10 shows the relationship between DOR and the thickness of footing in the case that the base footing is on roller support. From the results, it can be seen that when the thickness of footing is relatively small, the increase of the footing will increase the DOR because the stiffness of footing increases. However, when the thickness of footing becomes reaches a certain level, the strain at bottom of footing turns into tensile strain because of the flexural effect. The same effect creates more compressive strain at the top of footing and, as a result, the restraining effect against the shrinkage of wall decrease. This also results in lower degree of restraint. It can be also observed that the DOR value approaches a constant value if the thickness of the footing continues to increase.

It can be also noticed that the degree of restraint (DOR) is more sensitive to this flexural effect when the footing is thinner. In the case where the thickness of footing is 0.5 m, the degree of restraint when the length is 10 m is less than the value when the length is 1 m.

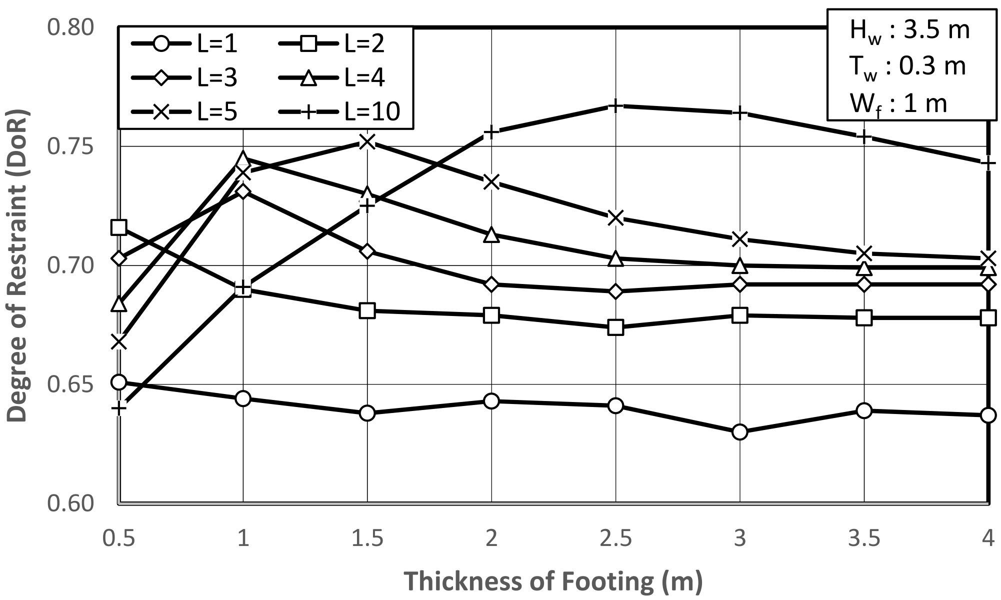


Figure 5.10 Relationship between DOR and thickness of footing

(H_w=3.5 m, T_w=0.3 m and W_f=1 m)

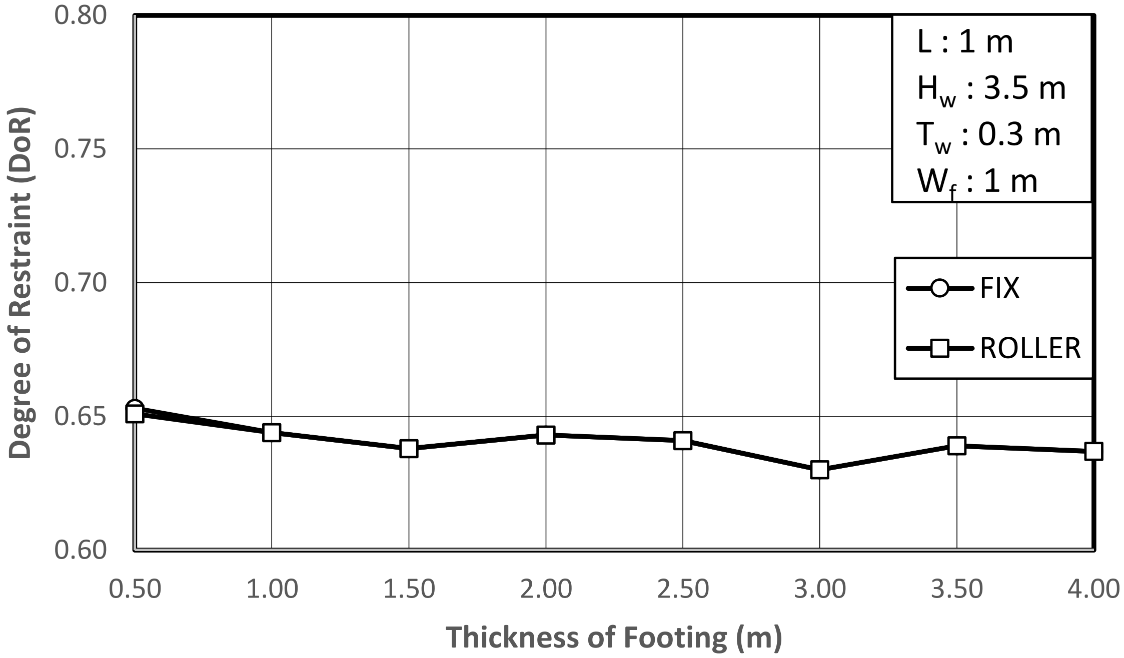


Figure 5.11 Comparison between the cases of fixed base and roller base.

(L=1 m, H_w=3.5 m, T_w=0.3 m and W_f=1 m)

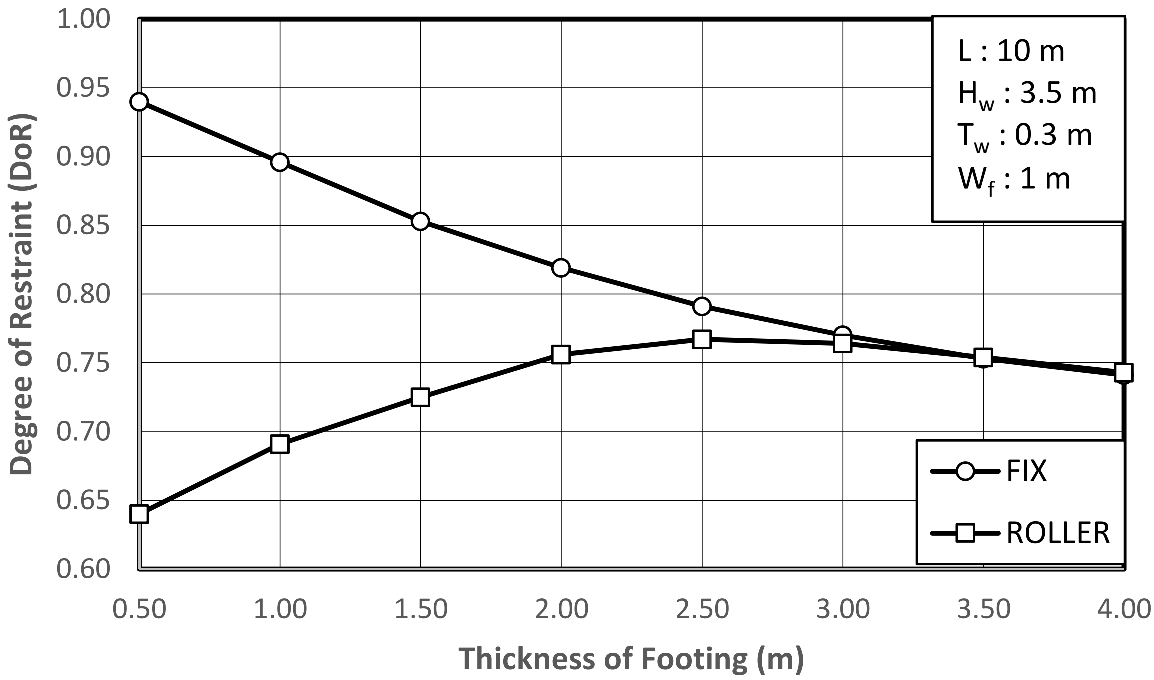


Figure 5.12 Comparison between the cases of fixed base and roller base.

(L=10 m, H_w=3.5 m, T_w=0.3 m andะ W_f=1 m)

Figure 5.11 shows the relationship between DOR and thickness of footing as well as a comparison between the cases where the bottom face of footing is modeled as fixed surface or as roller surface for the case where the length is 1 m and the thickness and height of wall are 0.3 and 3.5 m, respectively. The result indicates that, in this case, the DOR is very same for all values of footing thickness. This is probably because very short footing has high flexural rigidity and therefore there is almost no difference between two cases of the supports.

Figure 5.12 shows the relationship between DOR and thickness of footing for the case where the length of footing, the thickness and the height of wall is 10 m, 0.3 m, and 3.5 m, respectively. The results show very clear difference between the case that the bottom surface of footing is on fixed support or on roller support, especially when the thickness of the footing is small (not more than 2.5 m) because the influence of the boundary condition can influence deformation of footing significantly when the footing is quite thin. When the thickness of footing becomes thicker (at least 3.5 m), the difference between two cases of different support disappears.

In the case of roller-support condition, when the footing become thicker and has higher flexural stiffness, the degree of restraint (DOR) increases. In the case of fixed boundary condition, the degree of restraint (DOR) decreases. In general, the DOR is higher when the fixed boundary condition is applied. Therefore, the condition with fixed boundary condition will be used for further investigation.

5.2.5 Factors influencing Degree of Restraint

In this section, the analysis is performed for the case that the boundary condition at the bottom surface of the footing is fixed. The parameters are varied in order to investigate the sensitivity of DOR to each parameter.

From Figure 5.13, it can be observed that, when the length of wall and footing becomes longer, the value of DOR increases. This is in line with the theory shown in Eq. 1. The increase of DOR is in the range of 7.4% - 31.5% in comparison with the smallest value of DOR (L = 1 m). Another factor that may influence the value of DOR is the ratio between the length and thickness of the wall.

From Figure 5.14, it can also be seen that when the wall thickness increases, the value of DOR decreases. This is because, when the thickness of wall increases the difference of flexural stiffness between wall and footing reduces. And, as a result, the restraining effect become less intense. The influence of the wall thickness is in the range of 0.1% - 9.5% in comparison with the highest value of DOR (T_w = 0.1 m).

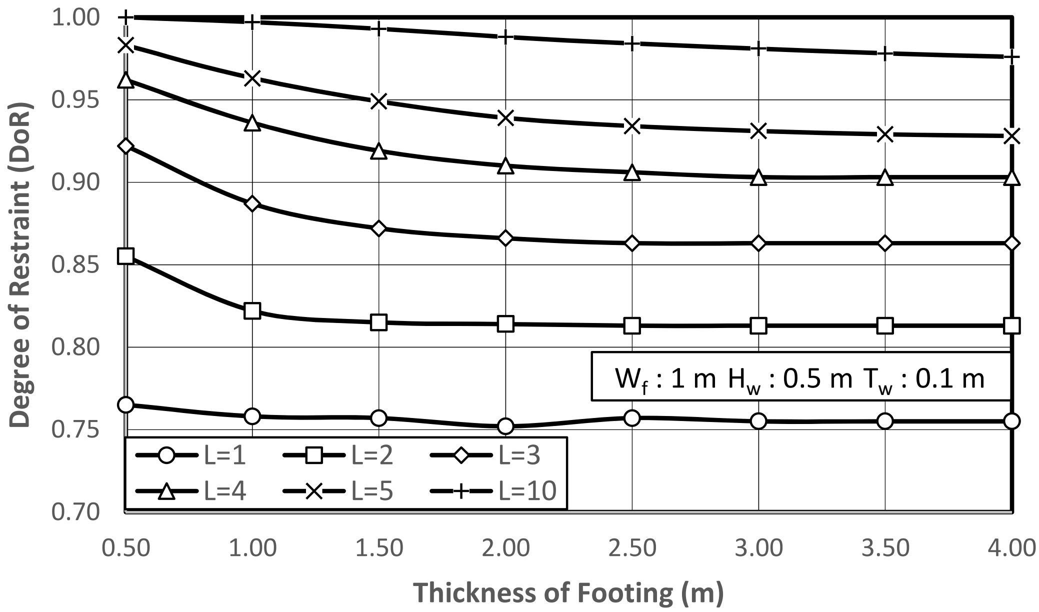


Figure 5.13 Effect of length on DOR

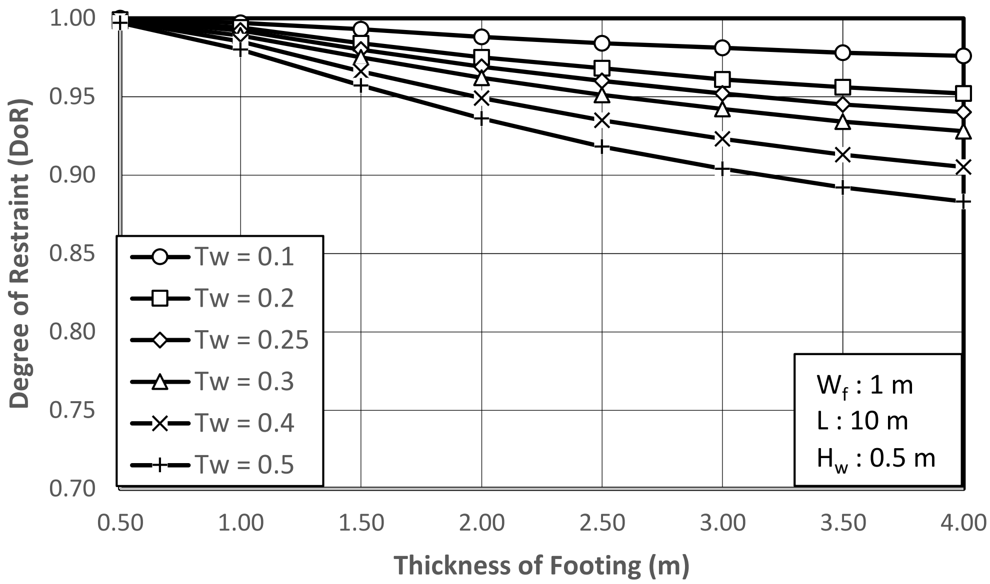


Figure 5.14 Effect of wall thickness on DOR

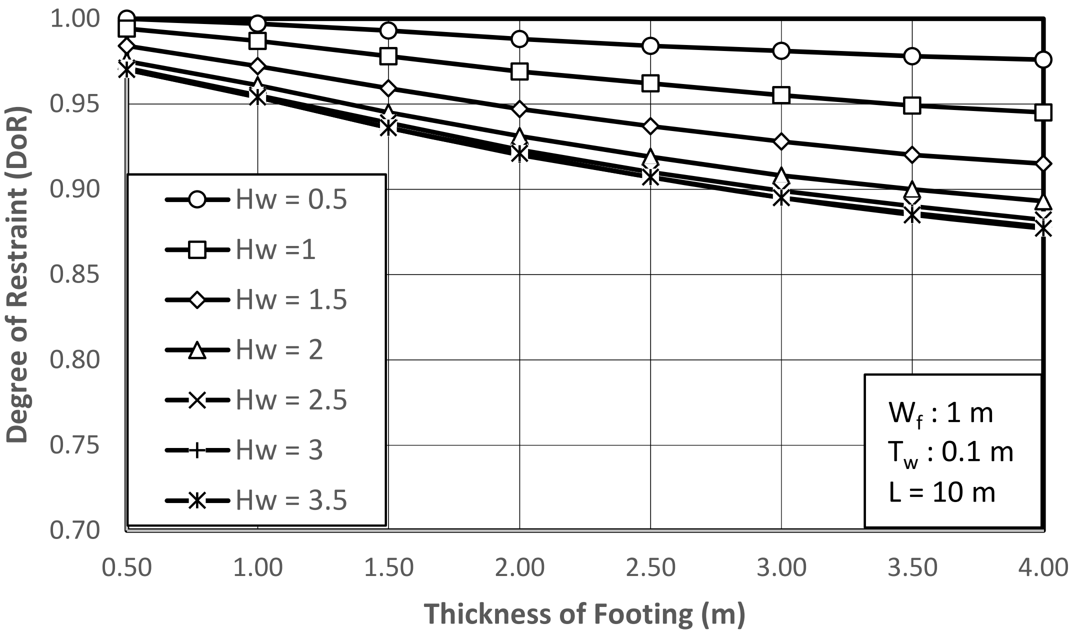


Figure 5.15 Effect of wall height on DOR

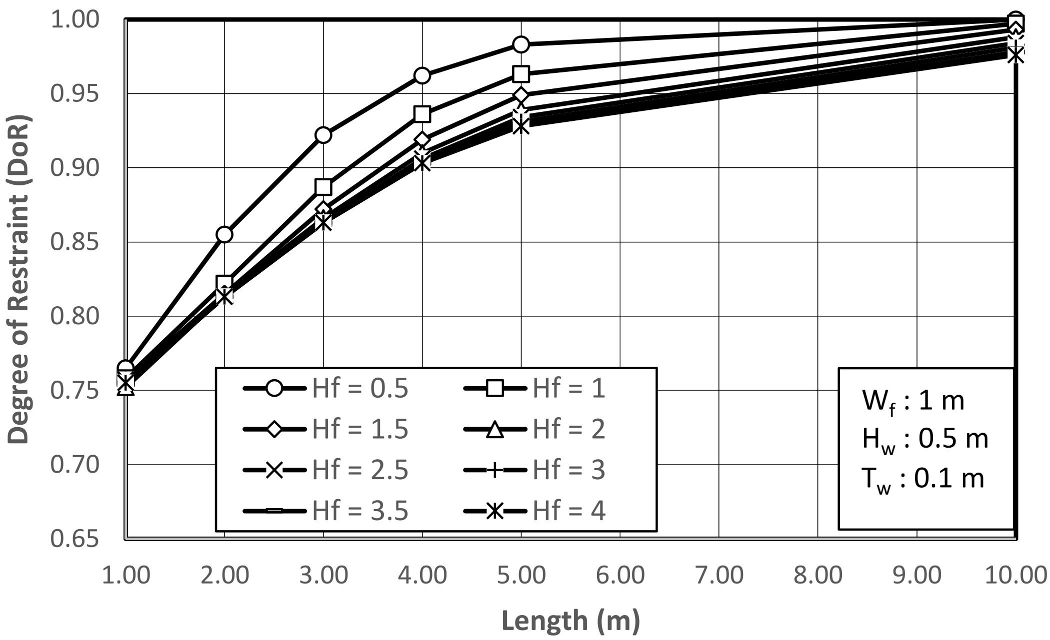


Figure 5.16 Effect of footing thickness on DOR

Figure 5.15 shows that the DOR decreases when the height of wall increases. This is probably because the difference between the stiffness between the wall and that of the footing is smaller. The reduction of DOR due to the change of wall height is in the range of 0.6% - 10.1% when compared with the highest value of DOR (H_w = 0.5 m).

The influence of footing thickness on DOR is shown in Figure 5.16. It can be found that the thicker footing leads to lower value of DOR. This is because, when the thickness of footing increases, the influence of fixed boundary condition at bottom surface of the footing to the wall which locates on top of the footing is weaker. The influence of the thickness of the footing is in the range of 0.3% - 6.4% of the highest value of DOR (H_f = 0.5 m).

According to this parametric study, it can be seen that the factor that mostly influence the change of DOR is the length of the structure. This finding also agrees very well with theoretical formula in Equation 1.

5.3 Analysis for Degree of Restraint

If the restrained tensile strain exceeds the tensile cracking strain capacity of the concrete, the crack will take place. Generally, the cracking strain capacity ($\varepsilon_{cracking}$) is approximately 150 $\mu\varepsilon$. The restrained strain occurs in the real concrete structure under restraint can be calculated by multiplying the degree of restraint (DOR) with the free shrinkage strain of the concrete as shown in Equation 4.

$$\varepsilon_{cracking} < DOR \times \varepsilon_{shrinkage,\, free} \tag{4}$$

The restrained tensile strain calculated from Equation 4 is one of the approaches that can be used to design for prevention against shrinkage cracks. Currently, although there are some prescriptive guidelines in the control of cracking or reducing the degree of restraint, the numeric assessment of degree of restraint and thus the value of restrained strain is still not available. This is because the degree of restraint is dependent on many details of structural design including shape and size of the considered structural element and the adjacent structural elements. The method described in this chapter is one of the approaches that can be used to design for prevention of concrete, especially for wall, in this situation.

Chapter 6: Concluding Remarks

According to the experimental investigation and numerical modeling performed in this study, the following statements can be concluded.

- Use of bottom ash as an internal curing agent is one of effective method to reduce autogenous shrinkage of concrete. However, in such case, the increase of drying shrinkage may be experience. In the case of real structure, the size of structural member is usually large, it can be expected that the adverse effect from the increased drying shrinkage is not critical.
- Use of superabsorbent polymer (SAP) is another way to reduce shrinkage of concrete. Properly-selected type of SAP can absorb water and release water to compensate with moisture reduction in cement paste. This mechanism leads to the reduction of autogenous shrinkage.
- The use of either bottom ash or SAP also helps to increase the flow ability of mortar or concrete. However, too much addition of these materials may leads to drastic loss of compressive strength.
- Expansive additive can be used to generate suitable amount of expansion during the early age of concrete. The required amount of expansive additive can be conveniently estimated from the relationship between the restrained expansion and the dosage of expansive additive.
- The combination of expansive additive with Thai fly ash creates more expansion of concrete. This indicates a possibility of more cost-efficient mix proportion design for crack prevention.
- The use of expansive additive in combination with bottom ash and shrinkage-reducing admixture is also a good alternative to increase the crack resistance of concrete. However, more research on this type of concrete is still required and the cost of the material may be one of the limitations for the real application.
- From the site survey, it is found that the careless selection of materials and inappropriate design of mix proportion is one of major causes of shrinkage cracks. Most of shrinkage crack can be prevented with the conventional materials.
- The degree of restraint can be estimated. The value of degree of restraint can be used for the design against shrinkage cracking of concrete structure.