

**SHEAR STRENGTH OF FIBER REINFORCED CEMENTITIOUS
COMPOSITE DEEP BEAMS**

The seal of Mahidol University is a large, circular emblem in the background. It features a central golden chakra (wheel) with a tiered umbrella (parasol) on top. The chakra is surrounded by a blue ring containing the university's name in Thai script. The entire seal is set against a light blue background.

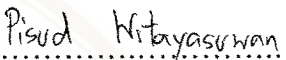
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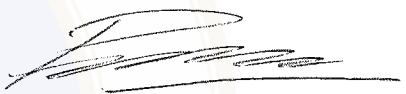
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OF THE REQUIREMENTS FOR
THE DEGREE OF MASTER OF ENGINEERING
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Thesis
entitled
**SHEAR STRENGTH OF FIBER REINFORCED CEMENTITIOUS
COMPOSITE DEEP BEAMS**


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SHEAR STRENGTH OF FIBER REINFORCED CEMENTITIOUS COMPOSITE DEEP BEAMS

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ABSTRACT

This study investigated the application of fiber reinforced cementitious composite (FRCC) in reinforcing bottle-shaped struts in reinforced concrete deep beam members, where the strut-and-tie method is used for the design. Fourteen deep beam specimens were tested to failure. The parameters investigated were strut angles with ties at approximately 28° and 36°, composed of hooked steel fiber with a volume fraction between 0.1 to 2.0%, and of matrix type.

The results showed that the specimen failed by compression strut failure. The use of steel fiber increased the load at the first diagonal crack, reduced the shear damages across the strut, and reduced the strain in the tie. However, the maximum load was not affected by the use of fiber. It is recommended that the use of hooked steel fiber with at least 0.5% volume can satisfy the requirements of minimum transverse reinforcement crossing struts.

**KEY WORDS: BOTTLE-SHAPED STRUT / STRUT-AND-TIE / HOOKED
STEEL FIBER / TRANSVERSE REINFORCEMENT /
REINFORCED CONCRETE**

119 pages

กำลังรับแรงเฉือนของคานาลิกซีเมนต์เสริมเส้นใย

SHEAR STRENGTH OF FIBER REINFORCED CEMENTITIOUS COMPOSITE DEEP BEAMS

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งานวิจัยชิ้นนี้ศึกษาการประยุกต์วัสดุซีเมนต์เสริมเส้นใย เพื่อทดแทนเหล็กเสริมในส่วนรับแรงอัด (Strut) ในคานาลิกคอนกรีตเสริมเหล็ก ที่ออกแบบด้วยวิธี Strut-and-Tie โดยทำการทดสอบตัวอย่างคานาลิก 14 คานาจนวิบัติ มีตัวแปรที่ศึกษาได้แก่ มุมของ Strut กับ Tie ที่ประมาณ 28° และ 36° ปริมาณเส้นใยเหล็กตะขอระหว่าง 0.1 ถึง 2.0 เปอร์เซ็นต์โดยปริมาตร และชนิดของแมทริกซ์

ผลการทดสอบแสดงให้เห็นว่า ตัวอย่างวิบัติโดยแรงอัดใน Struts การใช้เส้นใยช่วยเพิ่มกำลังที่จุดที่เกิดรอยร้าวเพียงครั้งแรก ลดความเสียหายที่ Strut และช่วยลดความเครียดใน Tie การใช้เส้นใยเหล็กตะขอในปริมาณอย่างน้อย 0.5 เปอร์เซ็นต์โดยปริมาตร สามารถทดแทนเหล็กเสริมขั้นต่ำที่กำหนดในมาตรฐานการออกแบบได้

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CHAPTER I

INTRODUCTION

1.1 Problems Statement

Current structural design standards, namely, ACI (ACI Committee 318, 2008) and AASHTO (AASHTO, 2007), have adopted the use of strut-and-tie method (STM) for the strength design of discontinuous regions, such as deep beam, corbels, or post-tension anchorage zones. Traditional designs of these regions rely on empirical methods or designer's experiences. Strut-and-tie model offers a rational approach to the design of these regions by idealizing stress fields as axial members of a truss. In this method, concrete strut carries compressive stress field while tie reinforcement carries tensile stress field. To enhance the efficiency of a bottle-shaped strut, reinforcing steel must be placed around the strut areas to resist transverse tensile stresses. For a complex strut-and-tie model, it may become complicate to place the reinforcing steel across various strut members without intersecting the tie reinforcements.

The application of short discontinuous fibers in concrete can increase post-cracking tensile strength of concrete, improve ductility, and reduce the crack width (ACI Committee 544, 1996). Therefore, it may be possible to use these short fibers to satisfy the transverse reinforcement requirements around the bottle-shaped strut areas. Studies have shown that steel fibers can be used to replace steel transverse reinforcement in deep beams designed using traditional methods (Kwak et al., 2002), (Šalna & Marčiukaitis, 2007). The efficiency of fiber reinforced concrete was found to depend on the fiber types, volume content of the fiber, and various mixture components of concrete. However, very little or no data is available on the effect of steel fibers in concrete strut of deep beams designed using the strut-and-tie method.

1.2 Objectives

The followings are the objectives of this study:

- To study the strength and failure behaviors of steel fiber reinforced concrete deep beams at various fiber content and compare to those of unreinforced concrete deep beams and deep beams containing minimum transverse reinforcement.
- To study the effects of strut angle on the fiber reinforced concrete deep beam.

1.3 Scope of Works

The scope of this research is limited to the behavior of a deep beam failed predominantly by strut failure. The strut angles with tie were limited to approximately 28° and 36°. The strut areas are either reinforced with steel hooked fiber at the volume fractions between 0.1% and 2.0% by volume, with minimum transverse reinforcement ratio according to ACI 318 code, or unreinforced.

1.4 Expected Outcomes

The results obtained from this study shall provide greater understandings of the effects of fiber reinforcement on the behaviors of bottle-shaped struts. These allow designers to appropriately select the amount of fiber content for the design of similar members. In addition, the results serve as a foundation for the applications in other kinds of structure with discontinuous regions.

CHAPTER II

LITERATURE REVIEW

2.1 Reinforced Concrete Deep Beam

ACI Committee 318 (2008) defines deep beams as members with clear span, l_n , equal to or less than four times the overall member depth, d , or regions with a concentrated load acting within twice the member depth from the face of the support. The applied load should act on one face while the member is supported on the opposite face such that compression struts can develop between the loading point and the supports. Figure II.1 shows description of deep and slender beams. Deep beam is commonly found in transfer girders or coupling beams.

The design of deep beam is different from that of a slender beam. In a slender beam where shear span to depth ratio (a/d) exceeds 2, the load is carried by flexure and the entire beam is referred to as B-region, where the B stands for beam theory or Bernoulli. In this case, linear strain distribution is applicable at all sections. However, in a deep beam with a/d less than 2, the load is carried by arch action and the beam is referred to as D-region, where the D stands for discontinuity or disturbed. D-region is assumed to extend about one member depth away from concentrated loads or abrupt changes in the member geometry on either side. In this region, the linear strain distribution assumption is no longer valid. In ACI 318 code(2008), D-region should be designed using a strut-and-tie method.

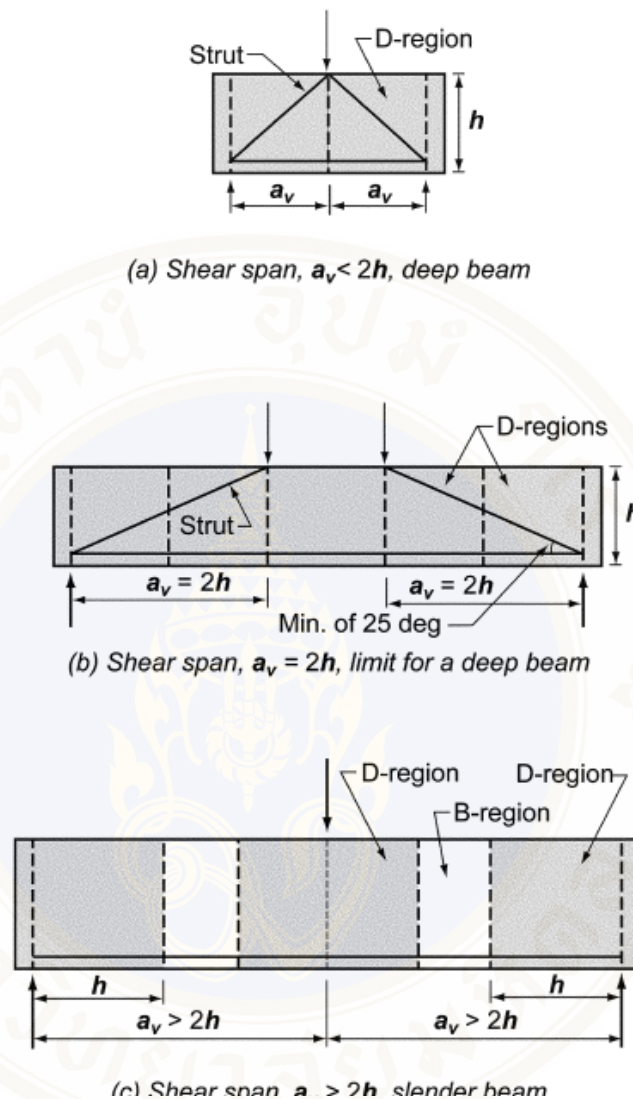


Figure II.1 Descriptions of Deep and Slender Beams (ACI Committee 318, 2008)

2.1.1 Failure Behaviors of Deep Beam

Typically, the principal mode of failure for deep beam is shear with inclined cracks joining the loading point and the support. Shear strength of a deep beam depends on factors such as tensile strength of concrete, shear span to depth ratio, and the amount and distribution of shear reinforcement. Unlike slender beams, the strength of deep beams are not sensitive to the size of the beam (size effect) nor the tensile strain in the longitudinal steel (strain effect) (Collins et al., 2008).

For tensile strength of concrete, the shear strength increases as the tensile strength of concrete increases. Tensile strength of concrete can be evaluated from

either direct tension test, splitting tensile strength, or estimated from the compressive strength of concrete.

For shear span to depth ratio (a/d), the shear strength increases as the shear span to depth ratio decreases.

Shear reinforcement provides a restraint against the growth of inclined crack; thus, increases the efficiency of cracked concrete in resisting shear.

2.1.2 Strut-and-Tie Models

Strut-and-tie is a method that can be used for the design of a deep beam. A strut-and-tie model idealizes the transfer of internal forces between the loading point to the supports as a truss member having compression (strut) and tension elements (tie) carrying only axial loads (Schlaich et al., 1987). The components of a strut-and-tie model are shown in Figure II.2. This method is based on the lower-bound theorem of plasticity in that only the equilibrium of all forces in the strut and tie members must be satisfied through yield strengths of each member while disregarding the kinematic compatibility (Brown et al., 2006). This results in ultimate capacity of a member calculated using the strut-and-tie method be less than or equal to the actual collapse load of the member.

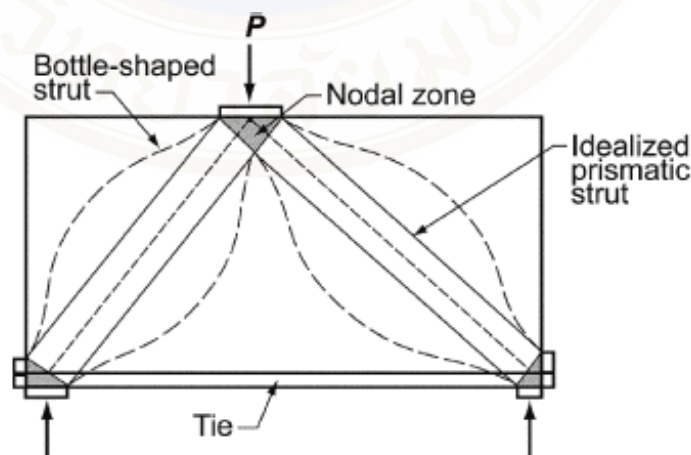


Figure II.2 Components of Strut-and-Tie Model (ACI Committee 318, 2008)

Strut is a compression member in a strut-and-tie model. It is a portion of concrete having compressive stress field, which can be idealized as prismatic,

uniformly tapered compression member, or bottle-shaped. The majority of struts in two dimensional strut-and-tie models are bottle-shaped, where the strut is wider at mid length than at its ends. The lateral spreading of a bottle-shaped strut introduces transverse tensile stress perpendicular to the direction of the strut. If the tensile stress exceeds the tensile strength of concrete, a crack forms in the direction of the strut, as depicted in Figure II.3(a). The transverse tensile stress in a bottle shaped strut can also be modeled as tie elements as shown in Figure II.3(b).

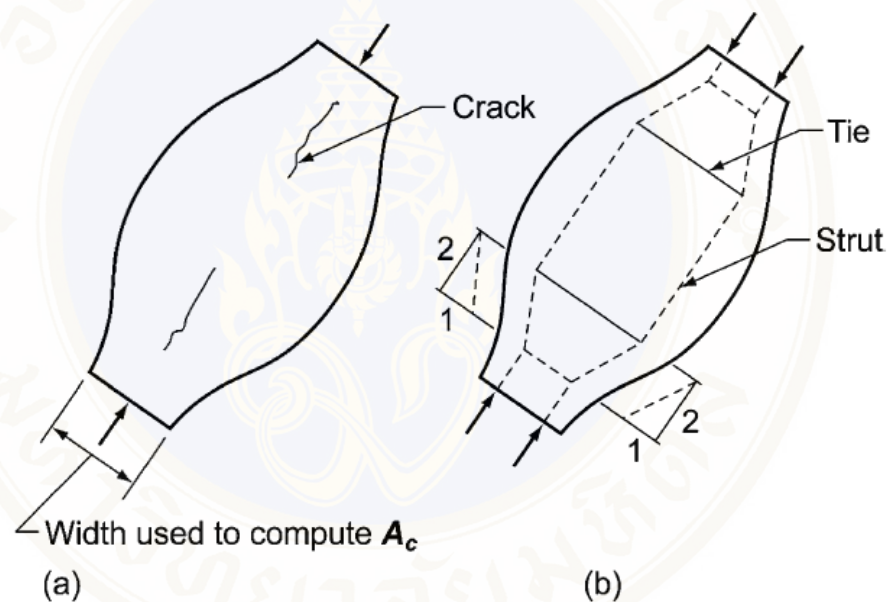


Figure II.3 Bottle-Shaped Strut: (a) Cracking of a Bottle-Shaped Strut; and (b) Strut-and-Tie Model of a Bottle-Shaped Strut (ACI Committee 318, 2008)

Tie is a tension member in a strut-and-tie model. Reinforcing steels shall be placed at the corresponding tie location in a strut-and-tie model.

Nodal zone is a concrete region at the joint in which strut and tie members are connected. There are four types of node depending on the type of forces coming to the node. They are classified as CCC, CCT, CTT and TTT nodes (Figure II.4). A CCC node resists three compressive forces, a CCT node resists two compressive forces and a tensile force, CTT node resists one compressive force and two tensile forces, and TTT node resist three tensile forces.

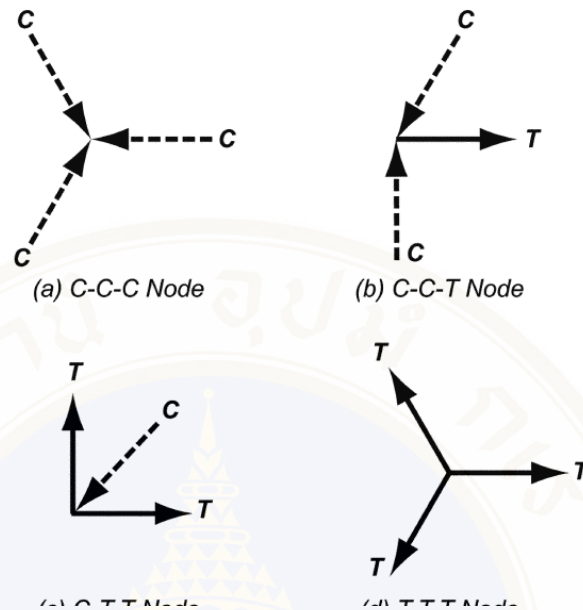


Figure II.4 Classification of Nodes (ACI-318-08)

A member in the strut-and-tie model can fail in one of the four ways:

- The tie could yield in tension.
- Crushing of the strut in compression.
- Bearing failure of the nodal zone.
- Anchorage failure of the tension tie.

2.1.2.1 ACI 318 Provision for Strut-and-Tie Models

In ACI 318 Code (ACI Committee 318, 2008) the internal forces, F_u , due to the factored load acting on a strut, tie, or nodal zones shall be less than the reduced nominal strength of the element as:

$$\phi F_n \geq F_u \quad (\text{II.1})$$

where:

- F_u = the factored force acting in a strut, in a tie, or on one face of a nodal zone
- F_n = the nominal strength of the strut, tie, or nodal zone
- ϕ = a strength reduction factor

It should be noted that the strength reduction factor used in a strut-and-tie model is taken equal to that for shear (0.75) regardless of whether the compression or tension failure is investigated.

ACI Code Section A.3.1 presents the nominal compressive strength of a strut without longitudinal reinforcement as:

$$F_{ns} = f_{ce} A_{cs} \quad (\text{II.2})$$

$$f_{ce} = 0.85 \beta_s f'_c \quad (\text{II.3})$$

where:

F_{ns} = nominal compressive strength of a strut

A_{cs} = area of concrete strut

f_{ce} = effective compressive strength

β_s = strut efficiency factor

f'_c = the concrete compressive strength

The strut efficiency factor depends on the type of the strut and whether the strut has any steel reinforcement crossing it. There are five classes of strut having different values for β_s as shown in Table II.1.

Table II.1 β_s for Strut

Strut Type	β_s
Uniform Width Strut	1.0
Bottle-Shaped strut with Reinforcement	0.75
Bottle-Shaped strut without Reinforcement	0.60
Strut in Tension Members or Tension Flanges	0.40
All Other Cases	0.60

ACI Committee 318 (2008) specifies a minimum area of steel reinforcement to resist transverse tensile stress for bottle-shaped strut with reinforcement, which take into account the angle of the reinforcements and the axis of the strut. When determining the area of steel, both longitudinal and transverse steels shall be included.

$$\sum \frac{A_{si}}{b_s s_i} \sin \alpha_i \geq 0.003 \quad (\text{II.4})$$

where:

A_{si} = the total area of reinforcement in the i^{th} direction as shown in Figure II.5

s_i = spacing of reinforcing bar in the i^{th} direction

b_s = width of strut

α_i = angle between the axis of a strut and the bars in the i^{th} direction

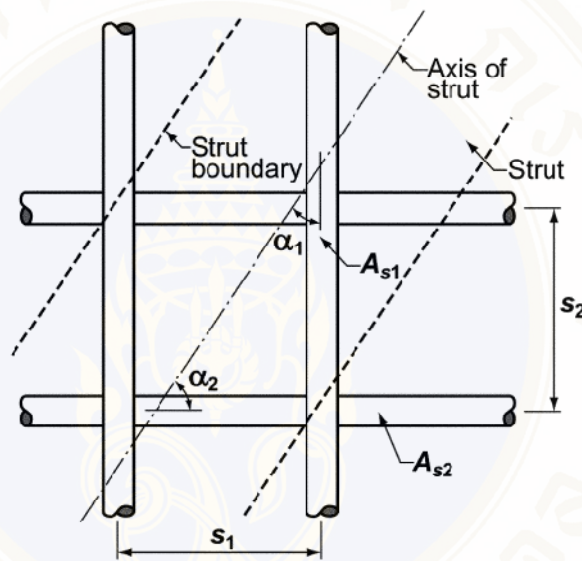


Figure II.5 Reinforcement Crossing a Strut (ACI-318-08)

ACI Code Section A.4.1 specifies the nominal strength of a tie as:

$$F_{nt} = A_{ts} f_y + A_{tp} (f_{se} + \Delta f_p) \quad (\text{II.5})$$

where:

F_{nt} = nominal strength of a tie

A_{ts} = area of nonprestressed reinforcement in a tie

A_{tp} = area of prestressing steel in a tie

f_{se} = effective stress in prestressing steel

Δf_p = increase in stress in prestressing steel due to factored loads

ACI Code Section A.5.1 specifies the nominal compressive strength of a nodal zone as:

$$F_{nn} = f_{ce} A_{nz} \quad (\text{II.6})$$

$$f_{ce} = 0.85 \beta_n f'_c \quad (\text{II.7})$$

where:

F_{nn} = nominal compressive strength of a nodal zone

A_{nz} = area of a face of a nodal zone

β_n = the nodal zone efficiency factor

Nodal zone efficiency factor depends on the type of node. Values of β_n are given in Table II.2.

Table II.2 β_n for Nodal Zones

Nodal Zone Type	β_n
In nodal zones bounded by struts or bearing areas, or both	1.0
In nodal zones anchoring one tie	0.80
In nodal zones anchoring two or more ties	0.60

2.1.2.2 AASHTO Provision for Strut-and-Tie Models

AASHTO LRFD (2007) Section 5.6.3 specifies that the factored resistance, P_r , of struts and ties shall be taken as that of an axially loaded component:

$$P_r = \phi P_n \quad (\text{II.8})$$

where:

P_n = nominal resistance of strut or tie (N)

ϕ = resistance factor

It should be noted that the resistance factor used in a strut-and-tie model is taken equal to that for shear, which equals to 0.70.

The nominal resistance of an unreinforced compressive strut and limiting compressive stress shall be taken as:

$$P_n = f_{cu} A_{cs} \quad (\text{II.9})$$

$$f_{cu} = \frac{f'_c}{0.8 + 170 \varepsilon_l} \leq 0.85 f'_c \quad (\text{II.10})$$

$$\varepsilon_l = \varepsilon_s + (\varepsilon_s + 0.002) \cot^2 \alpha_s \quad (\text{II.11})$$

where:

- f_{cu} = limiting compressive stress (MPa)
 A_{cs} = effective cross-sectional area of strut (mm^2)
 α_s = the smallest angle between the compressive strut and the adjoining tension tie
 ε_s = the tensile strain in the concrete in the direction of the tension tie (mm/mm)
 f'_c = specified concrete compressive strength (MPa)

The efficiency of strut is a function of the tensile strain in the steel tie and the angle between the strut and tie. The strut efficiency factor was based on the Modified Compression Field Theory (MCFT). The higher the tensile strain, the lower the strut efficiency. The strut efficiency is limited to 0.85 and approaches zero as the strut aligns in the same direction as the tie, as shown in Figure II.6. The decrease in strut efficiency with the strut angle is consistent with the findings of Matamoros and Wong (2003).

Based on the collected and/or test data Tuchscherer et al. (2011a) and Brown et al. (2006) reported that the AASHTO's provisions for strut efficiency led to less conservative results when compared with ACI 318's provisions. The AASHTO LRFD provisions also had higher scatter of the data than the ACI's method.

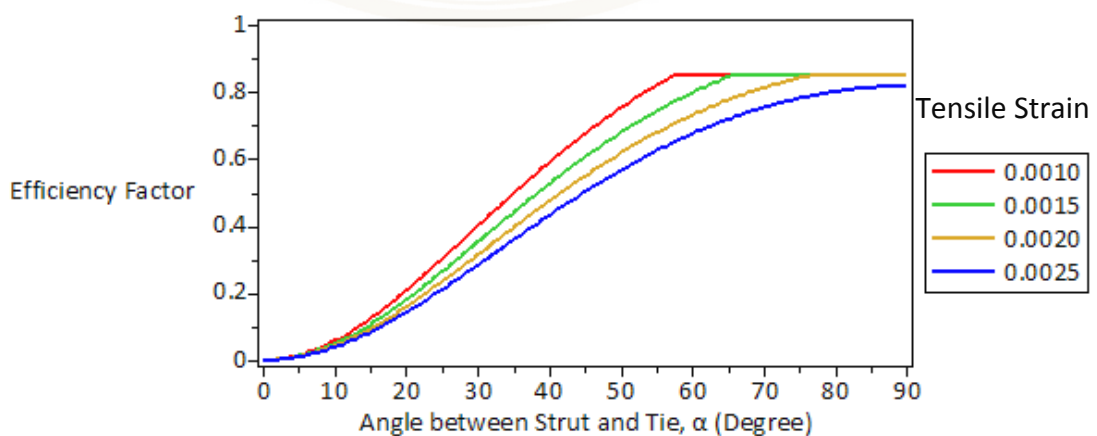


Figure II.6. Strut Efficiency in AASHTO LRFD

AASHTO LRFD 5.6.3.6 specifies minimum crack control reinforcement crossing all areas in the form of grid reinforcements having spacing less than 300 mm and having reinforcement ratio of at least 0.003 in each direction (i.e. the minimum total reinforcement ratio of 0.0042).

AASHTO LRFD section 5.13.2.3, detailing requirements for deep beam. The factored tensile resistance, N_R in N, of transverse pair of reinforcing bars shall satisfy:

$$N_R = \phi f_y A_s \geq 0.83 b_v s \quad (\text{II.12})$$

where:

- A_s = area of steel in distance spacing of reinforcement (mm^2)
- b_v = beam width (mm)
- s = spacing of transverse reinforcement (mm)
- f_y = yield strength of transverse reinforcement (MPa)

2.1.3 Other Prediction Methods for Shear Strength of Deep Beam

Choi et al. (2007a) developed a model to predict shear strength for both slender beams and deep beams with and without shear reinforcement. The shear strength of a beam can be attributed to the failure of the concrete compression zone. The shear strength of beam can be determined as follows:

$$V_n = V_c + V_s \quad (\text{II.13})$$

$$V_c = \lambda_s \sqrt{f_t [f_t + \bar{\sigma}_{ct}]} b \cdot [c(\alpha_{x1} \varepsilon_0) - C_c] + \sqrt{f'_c [f'_c + \bar{\sigma}_{cc}]} b C_c \quad (\text{II.14})$$

$$V_s = \rho_v f_{vy} b (d - 2C_c) \quad (\text{II.15})$$

where:

- V_n = shear strength of beam
- V_c = shear contribution of concrete
- V_s = shear contribution of transverse reinforcement
- λ_s = size effect factor
- f_t = $0.292 \sqrt{f'_c}$ (MPa), which equals to tensile strength of concrete

$\bar{\sigma}_{cc}, \bar{\sigma}_{ct}$	=	average compressive normal stresses developing in failure surface of compression crushing and inclined tensile cracking
$c(\alpha_{x1}\epsilon_0)$	=	depth of compression zone at critical section
C_c	=	depth of failure surface of compression crushing
ρ_v	=	ratio of transverse web reinforcement
f_{vy}	=	yield strength of transverse web reinforcement
b	=	width of beam
d	=	effective depth of beam

Zararis (2003) developed a model to predict the shear strength of a deep beam. The shear strength depends on the strength of concrete, longitudinal steel ratios, shear reinforcement ratios, and shear span to depth ratios. For deep beam, the following equation was proposed:

$$V_u = \frac{bd}{a/d} \left[\frac{c_s}{d} \left(1 - 0.5 \frac{c_s}{d} \right) f'_c + 0.5 \rho_v f_{yv} \left(1 - \frac{c_s}{d} \right)^2 \left(\frac{a}{d} \right)^2 \right] \quad (\text{II.16})$$

where:

V_u	=	ultimate shear force
a	=	shear span
c_s	=	depth of compression zone above critical diagonal crack

Senturk and Higgins(2010) proposed a modification to Zararis's model to account for the possibility that the strength of a deep beam is limited by the strength of the longitudinal steel, instead of the concrete compression zone. In addition, the modification includes the horizontal shear force contribution of the vertical web reinforcement. The process involves checking whether the longitudinal steel reinforcement is sufficient to be in equilibrium with the compression force.

$$T_{\max} = A_s f_y \geq C = 0.85 f'_c b c_s \quad (\text{II.17})$$

If it is satisfied, the strength of the beam is governed by concrete strength and its strength is given by Zararis's equations. Otherwise, a horizontal shear force contribution of the vertical web reinforcement must be added to the longitudinal steel

reinforcement force and check against the compression force from concrete in the compression zone.

$$T_{\max} + 0.4V_s \tan \theta = A_s f_y \geq C = 0.85 f'_c b c_s \quad (\text{II.18})$$

where:

θ = Angle between the critical diagonal crack and the vertical direction

If it is satisfied, the shear strength of concrete is calculated as:

$$V_u = \frac{T_{\max} (1 - 0.5c_s / d) + 0.5V_s (a / d) + 0.5V_{sd, \text{limited}}}{(a/d)} \quad (\text{II.19})$$

where:

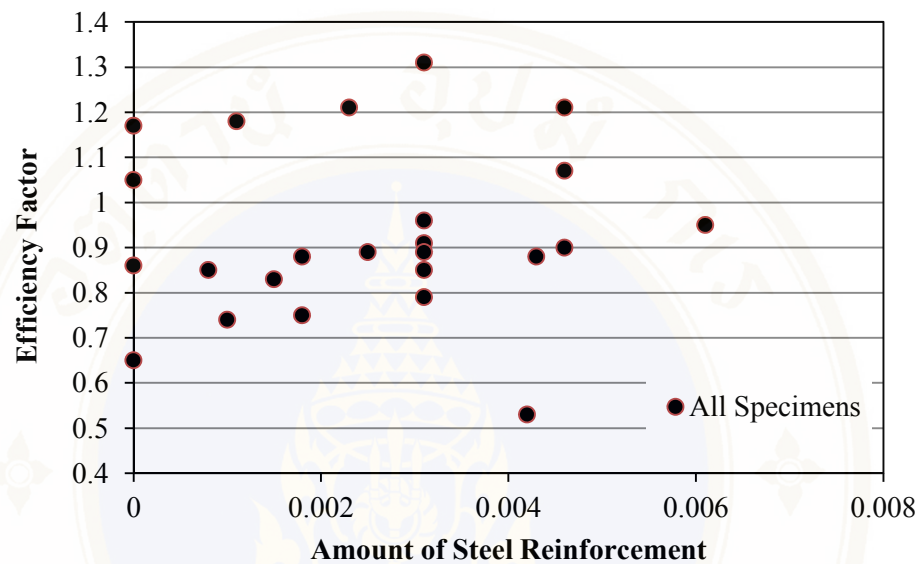
$$V_{s, \text{limited}} = C - T_{\max}$$

If not, the strength of the deep beam is limited by the strength of reinforcing steel.

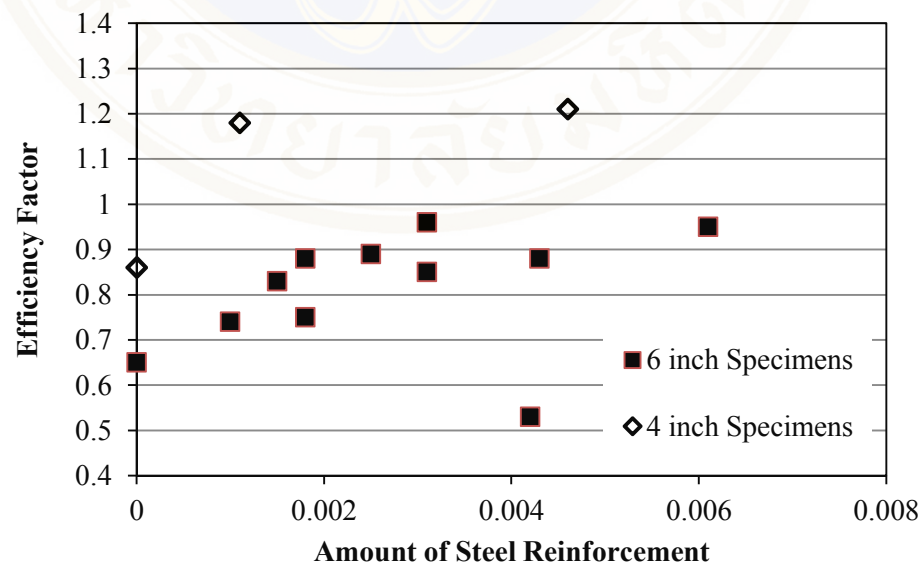
2.1.4 Effects of Transverse Reinforcement on Shear Behavior of Reinforced Concrete Deep Beam

Brown et al. (2006) studied behaviors of single strut in reinforced concrete panels having various reinforcing layout and reinforcement ratios (expressed as $\sum (A_{si} / b_s s_i) \sin \alpha_i$) and reported that the reinforcement ratio does not have significant effects on the strength of a single strut. The primary factor affecting the strut efficiency appeared to be the thickness of the member – the thicker the member, the lower the strut efficiency. A scatter plot of their data is shown in Figure II.7a. However, if only the data from specimens that had unconventional reinforcement layout was removed, leaving with only specimens with more or less distributed reinforcement (either one direction or grid pattern), and separate the specimens with different thicknesses, Figure II.7b is obtained. From the Figure II.7b, it can be observed that the strut efficiency increases slightly as the reinforcement ratio increases, especially between zero and about 0.003. After that, the strut efficiency became constant. This is consistent with Brown and Bayrak (2006), which reported that there is a minimum amount of transverse reinforcement required to maintain the equilibrium of force crossing a strut. Beyond this amount of transverse reinforcement,

an increase in transverse reinforcement ratio did not lead to an increase in the strength of a strut, but may reduce the crack width for serviceability concerns.



(a) All Specimens



(b) Specimens Containing only Distributed Reinforcement

Figure II.7 Strut Efficiency vs. Amount of Transverse Reinforcement

The effect of transverse reinforcement to the strength of deep beam according to Choi, et al. (2007a) is shown in Figure II.8 for the beam with shear span to depth ratio (a/d) of 1.23 and concrete compressive strength of 30 MPa. In this model, the shear strength of a beam is given by the combination of the shear contributions from concrete (V_c) and steel transverse reinforcement (V_s). Although the shear contribution of concrete depends on the amount of transverse reinforcement, very little change of V_c was observed as the transverse reinforcement increases. On the other hand, a significant increase of the overall shear strength was observed as the transverse reinforcement amount increases, which can be attributed to the V_s term.

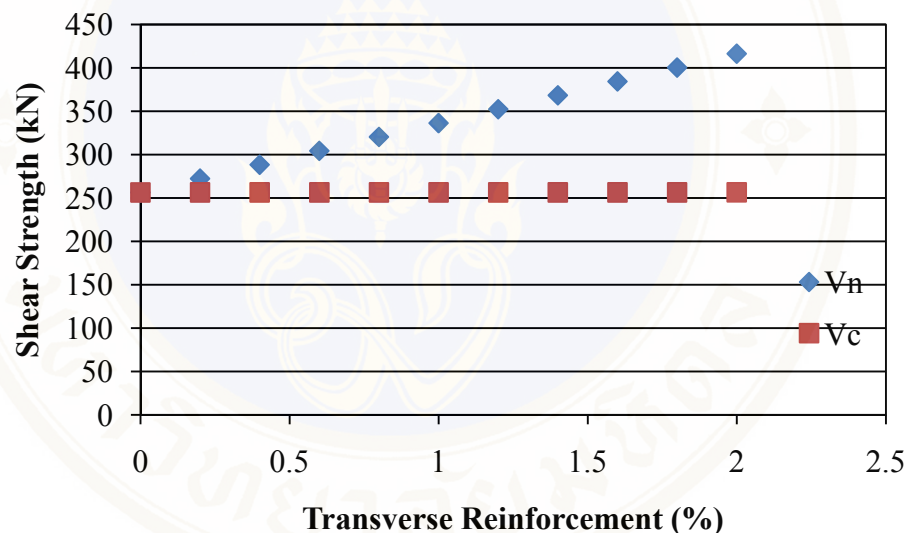


Figure II.8 Shear Strength vs. Amount of Transverse Reinforcement from Choi, et al. (2007a) Model

For the Zararis (2003) model, the shear strength of the beam comprises two terms. For the purpose of this investigation, the first term is called V_c , which is the contribution of concrete strength. The effect of transverse reinforcement to the shear strength of a deep beam having shear span to depth ratio (a/d) of 1.23 with concrete compressive strength of 30 MPa is shown in Figure II.9. It was observed that the shear strength of beam increased with an increase in transverse reinforcement. However,

increasing transverse reinforcement was found to decrease the shear strength of concrete.

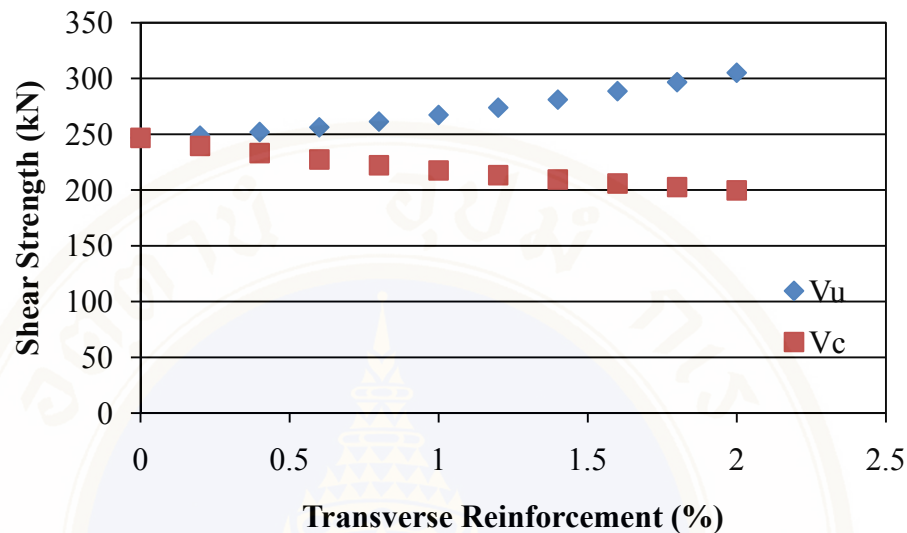


Figure II.9 Shear Strength vs. Amount of Transverse Reinforcement from Zararis (2003) Model

Tuchscherer et al. (2011b) studied the influence of the distribution of steel stirrups across the web of deep beams. The specimens having shear span to depth ratio a/d between 1.84 and 1.85 and vertical web reinforcement ratio between 0.2% and 0.3%. It was observed that an increase of vertical web reinforcement ratio from 0.2 to 0.3% did not significantly improve the shear capacity as shown in Figure II.10. However, for similar transverse reinforcement ratios, providing four stirrups distributed across the web led to about 15% higher strength and smaller crack width than when only two stirrups were provided near the side faces.

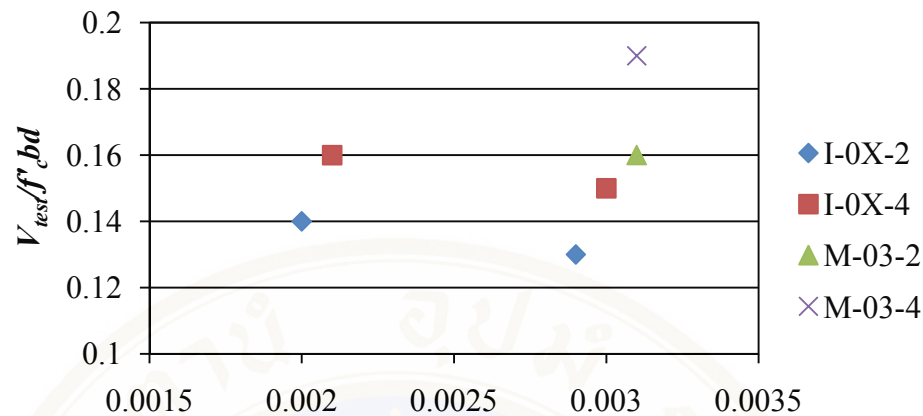


Figure II.10 $V_{test}/f'_c b d$ vs. Transverse Reinforcement Ratio (Tuchscherer, et al., 2011b)

2.2 Fiber Reinforced Concrete

2.2.1 Introduction

Concrete is strong in compression and weak in tension. When subjected to tensile stresses, unreinforced concrete will crack and fail abruptly. Therefore, concrete is normally reinforced with steel reinforcing bars to resist the tension force after cracking. Alternatively discontinuous fibers may be added into concrete matrix to improve concrete tensile strength after cracking, ductility, toughness, impact resistance, and fatigue resistance (ACI Committee 544, 1996).

Typically, fibers are made up of steel, glass, synthetic polymer, or natural fiber materials. The role of fibers in increasing concrete residual tensile strength and a strain capacity is shown schematically in Figure II.11. For plain concrete, when the tensile strength is reached, the concrete specimen cracks and fails abruptly. On the other hand, for fiber reinforced concrete, the fibers can bridge across the tensile crack resulting in some residual strength after the first cracking. The ability of fiber to carry tension increases as fiber content increases. At some point, it may be possible to increase the tensile strength beyond the strength at the first cracking. This is called strain-hardening and deflection-hardening for behaviors observed in direct tensile and bending tests, respectively. Typically, this behavior is accompanied by a significant increase in strain capacity and substantially larger energy absorption capacity. In order

to obtain strain-hardening or deflection hardening behaviors, appropriate fibers must be used at sufficiently large amount in a compatible matrix.

In tension, the ability of fibers to enhance concrete post-cracking behavior primarily depends on the fiber tensile strength, fiber stiffness, and bond behavior with surrounding concrete matrix (ACI Committee 544, 1996). Thus, the behavior of fiber reinforced concrete is highly dependent on the fibers and should be evaluated experimentally from direct tension or flexural strength tests.

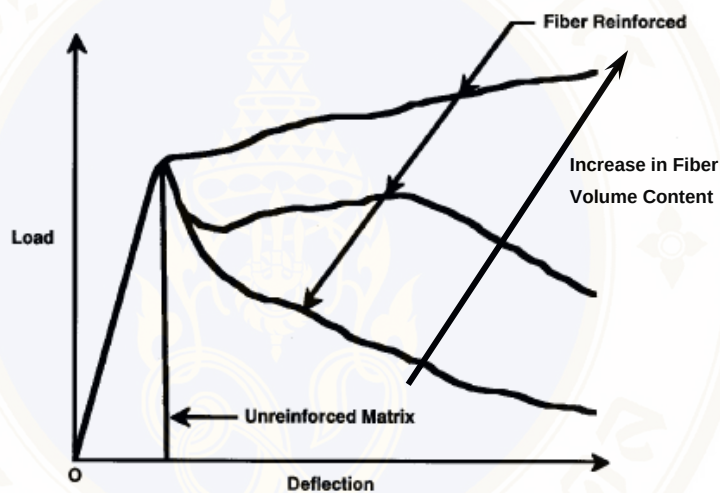


Figure II.11 Range of Load versus Deflection Curves for Unreinforced Matrix and Fiber Reinforced Concrete (ACI Committee 544, 1996).

2.2.2 Application as Shear Reinforcement in Slender Beams

Due to the ability of fibers to carry tensile stress, several studies have been conducted on the use of fibers as shear reinforcement in slender beams, with or without steel stirrups, for either strain-softening or strain-hardening materials, and for both monotonic and reversed cyclic loadings (Li et al., (1992); Adebar et al., (1997); Furlan Jr. and de Hanai, (1997); Campione and Mindess, (1999), Fischer and Li, (2002); Parra-Montesinos and Chompreda, (2007)). The general conclusion is that the fibers can be used as a replacement for transverse reinforcement in slender beams. The load carrying capacity of the beam increases as fiber content increases, until the flexural failure controls the strength of the member.

2.2.3 Application as Shear Reinforcement in Deep Beams

Limited data is available for the use of fibers as reinforcement in deep beams.

Kwak et al. (2002) investigated the shear strength of FRC beam with shear span to depth ratio between 2 and 4. In these specimens, steel fibers were used at volume fractions of 0.5 and 0.75%. For the deep beam with shear span depth ratio of 2, the beam without steel fiber abruptly failed by shear failure before flexural capacity was reached as shown in Figure II.12. For the beam with steel fiber, the failure mode changed to a combination of shear and flexure. The maximum load and deflection capacity increased as the fiber volume fraction increased.

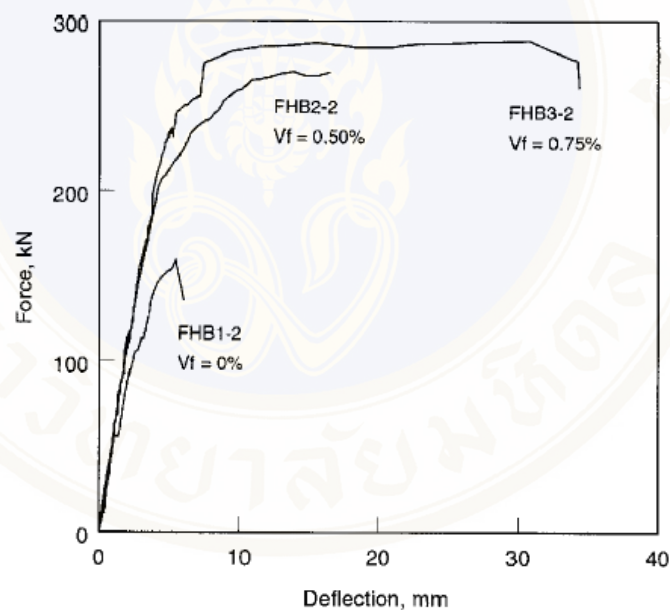


Figure II.12 Force-Deflection for Beams with $(a/d) = 2$ (Kwak, et al., 2002)

Šalna and Marčiukaitis (2007) studied the influence of the amount of steel fiber and shear span to depth ratio on the strength of fiber reinforced concrete beam. Beams specimens having shear span to depth ratio (a/d) between 1 and 2 with steel fiber volume content between 1-2% by volume were tested and compared to the plain concrete specimen. It was found that the greatest increase in load carrying capacity

was observed at shear span to depth ratio of 1.0, when the steel fiber is added. For shear span to depth ratio between 1.5 and 2, the increase of load capacity is less significant as shown in Figure II.13. It was also found that an increase in steel fiber content between 1.0-2.0% led to only slight increases in load carrying capacity.

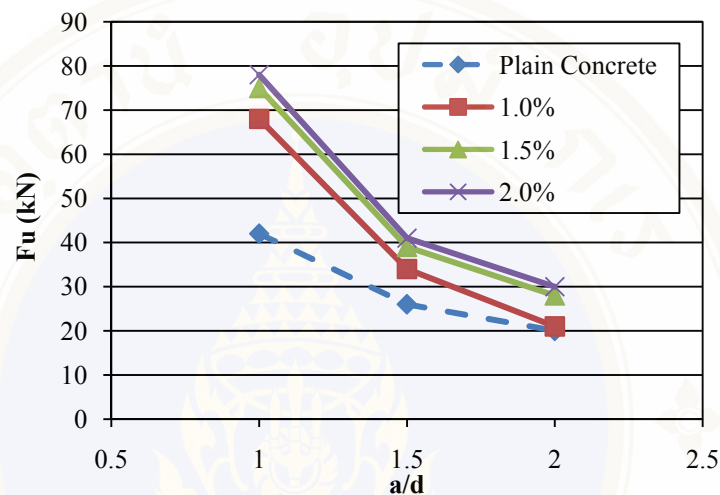


Figure II.13 Influence of Fiber Volume V_f and Shear Span Ratio a/d on Load Capacity. (Šalna and Marčiukaitis, 2007)

Shah and Mishra (2004) investigated the effect of steel fiber in concrete on crack and deformation characteristics of reinforced concrete deep beams. The specimens contained steel fiber in volume fraction of 1%, with and without transverse reinforcement, and clear span to overall depth ratio of 1.5, 2.0, 3.0 and 4.0. It was found that the inclusion of steel fibers in concrete deep beam resulted in reduced crack widths and reduced deflection. It was observed that the deflection of FRC beams were reduced up to about 70% compared to the plain concrete beams, as shown in Figure II.14. In addition, an inclusion of steel fibers in concrete deep beams resulted in reduced crack widths at all stages of loading.

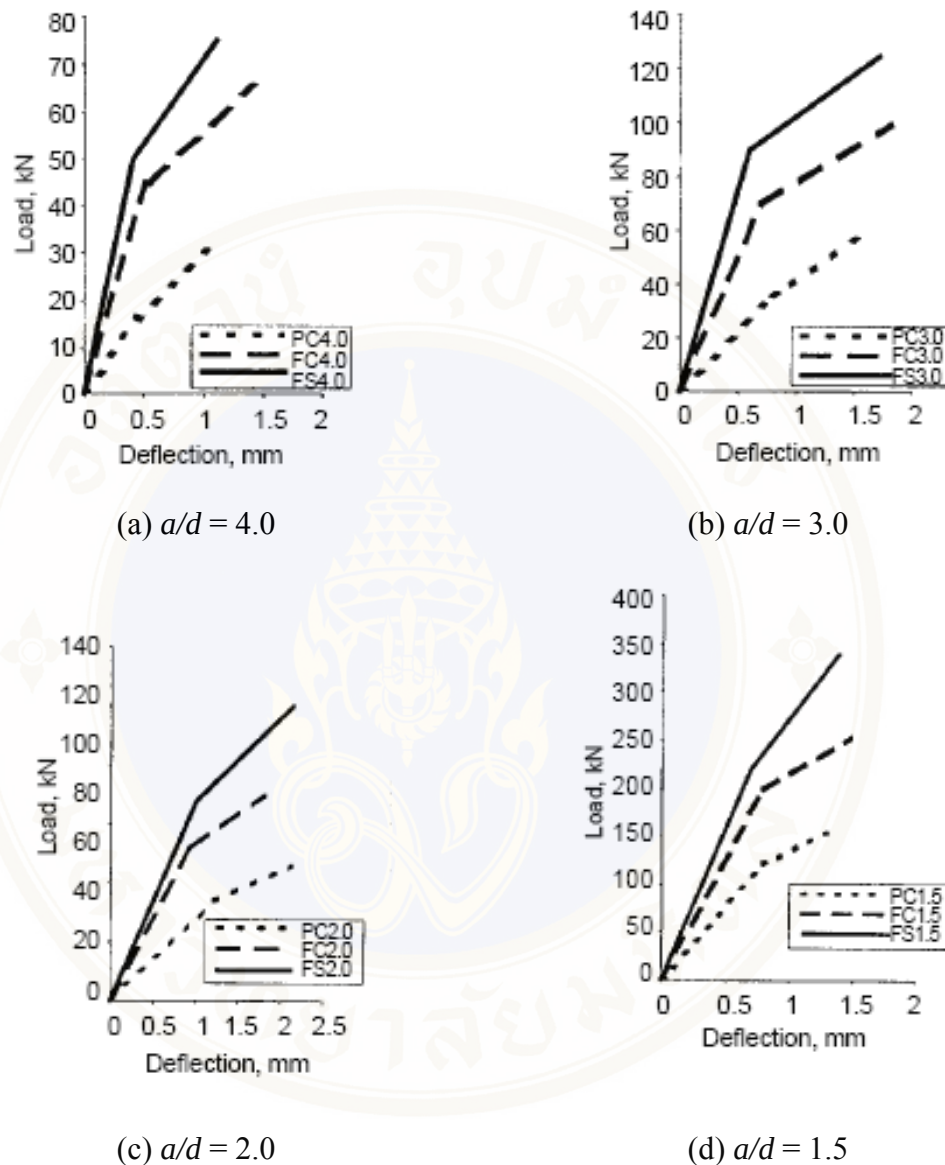


Figure II.14 Comparison of Load Deflection Curves of Deep Beams with Varying Clear Span to Depth Ratios (Shah and Mishra, 2004)

Madan et al. (2007) studied the behaviors of FRC deep beams with shear span to effective depth ratio of 0.75, 1.0 and 1.25. Steel fibers were added to concrete matrix at the volume fraction of 1.0 and 1.25%. The FRC specimens were compared with RC specimens with web reinforcement at the ratio of 0.25 and 0.5%. It was found that the first diagonal cracking load and the maximum load increases when the fiber

content increases. However, the increase in the maximum load appears to be very minimal (less than 5%). In addition, it was found that FRC specimens had slightly higher maximum loads than those of the RC specimens with transverse reinforcement. The trends appeared to be consistent in all shear span to effective depth ratios. For the same type of specimen, the shear strength increased when the shear span to effective depth ratios decreased.



CHAPTER III

EXPERIMENTAL PROGRAM

3.1 Parameters Studied

The parameters studied were the angle of strut with tie and the type of matrix. Deep beam specimens were used with two different span lengths, 600 and 800 mm, corresponding to the strut angle of approximately 28° and 36° respectively. Five types of fiber reinforced cementitious composites (FRCCs) were studied consisting of four concrete matrices having fiber volume fraction of 0.1, 0.5, 1.0 and 2.0%, and one mortar matrix FRCC having fiber volume fraction of 2.0%. Two control specimens, with plain concrete matrix, were made for each strut angle. One contained the minimum transverse reinforcement and the other had no transverse reinforcement. The experimental program is shown in Figure 3.1.

The notation for the deep beam specimens contained series of letters and number. The first letter, B, indicates that this is for the deep beam specimen. Next, series of letters and numbers indicates the matrix type, where:

RC = Concrete matrix with minimum transverse reinforcement

PC = Plain concrete matrix

F*.* = Concrete matrix containing steel fibers at *.* volume fraction

FM*.* = Mortar matrix containing steel fibers at *.* volume fraction

The last set of numbers indicates the span length (center to center of the support) in mm. For example, BF0.5-600 is a 600 mm span deep beam specimen with concrete matrix containing steel fibers at 0.5% by volume.

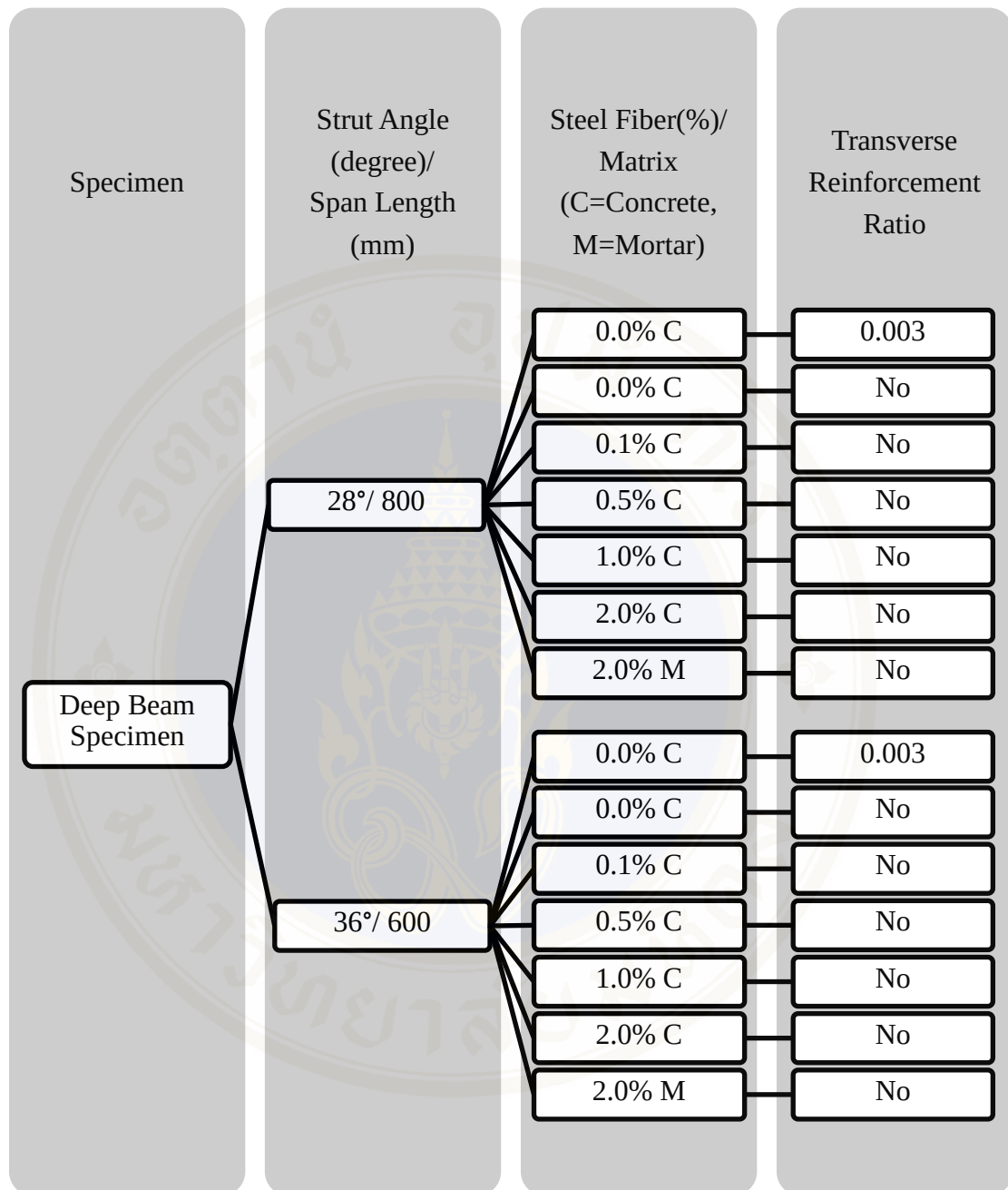


Figure 3.1 Experimental Program

3.2 Material

This section describes the properties of materials used in this study.

3.2.1 Fiber

The fibers used in this study were ZP308 steel hooked fibers manufactured by Bekaert company, as shown in Figure 3.2. This fiber had a circular cross-section with its both ends hooked. Each fiber had a length of 30 mm and a diameter of 0.75 mm with an aspect ratio of 40. The tensile strength of the wire was 1100 MPa.



Figure 3.2 Steel Hooked Fibers

3.2.2 Material and Mixture Proportioning

For all concrete and mortar mixtures, Portland cement Type I conforming to ASTM C150 (2007) was used. The coarse aggregate was crushed limestone with the maximum size of 9.5 mm. The fine aggregate was river sand with the maximum size of 0.8 mm. All the mixtures had a design compressive strength of 30 MPa. Superplasticizer was used to increase the workability of concrete matrix. Table 3.1 summarizes the mixture proportions.

Table 3.1 Mixture Proportion

Mixture Type	Cement (kg/m ³)	Water (kg/m ³)	Coarse Aggregate (kg/m ³)	Fine Aggregate (kg/m ³)	w/c	Superplasticizer (%by weight)
Concrete	342	205	897	870	0.6	1-2
Mortar	592	296	-	1300	0.5	-

3.2.3 Steel Reinforcement

For all deep beam specimens, 20 mm diameter deformed bars were used as primary tie reinforcements. It conformed to TIS. 24-2548 (Thai Industrial Standards Institute, 2005) grade SD40. 6 mm diameter round bars were used as transverse reinforcements. It conformed to TIS. 20-2543 (Thai Industrial Standards Institute, 2000) grade SR24. Their properties are summarized in Table 3.2.

Table 3.2 Properties of Reinforcing Steel

Reinforcing Steel	Grade	Yield Strength, F_y	Ultimate Strength, F_u
		(MPa)	(MPa)
RB6	SR24	368	464
DB20	SD40	515	657

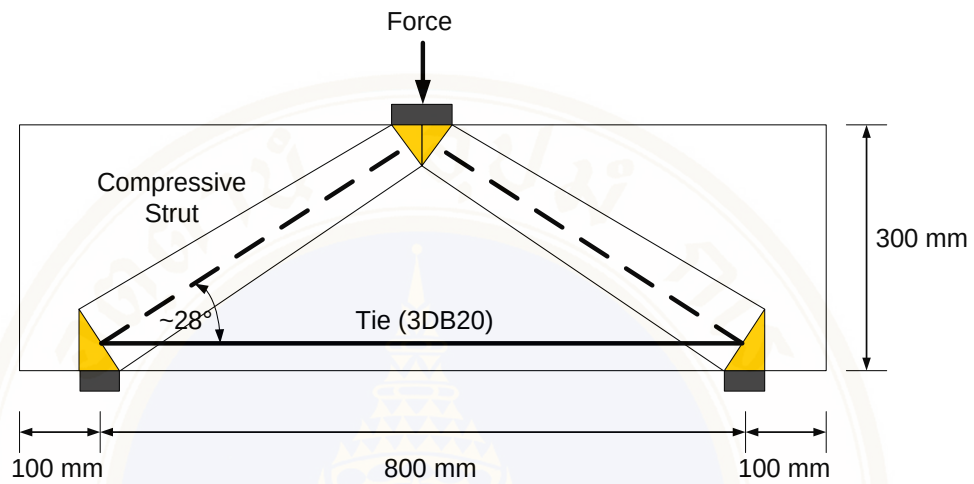
3.3 Deep Beam Specimen

3.3.1 Details

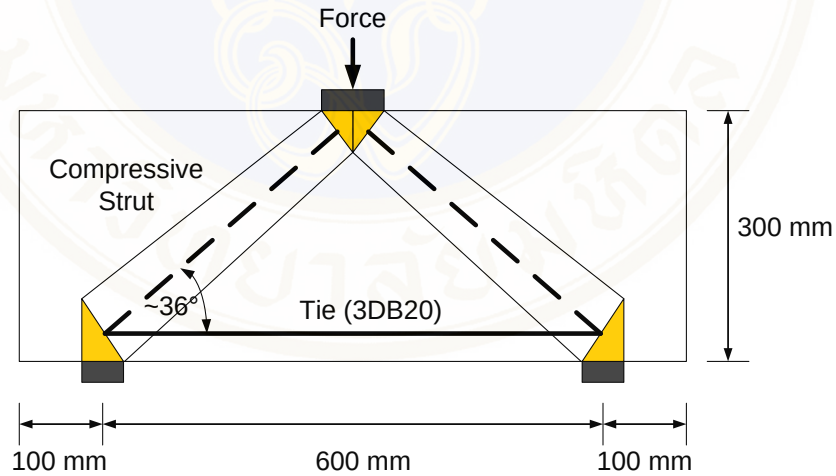
Deep beam specimens were classified into two groups based on their span lengths. In the first group, the deep beams specimens were 150×300×1000 mm with the span length of 800 mm, corresponding to the strut angle of 28° (Figure 3.3a). In the second group, the specimens were 150×300×800 mm with the span length of 600 mm, corresponding to the strut angle of 36° (Figure 3.3b). The effective depth were all 260 mm. Three 20 mm deform bars were used as longitudinal reinforcement for all of the beams. The differences among the specimens were transverse reinforcement ratio and the volume content of steel fibers in the matrix. The properties of the specimens are provided in Table 3.3.

Deep beam specimens were designed to have bottle-shaped strut on both sides with a tie at the bottom of the beam (Figure 3.3). Brown et. al. (2006) reported that for beam with shear span to depth ratio less than two, as in the case for the deep beam specimens here, a single panel truss mechanism dominates the behavior. This is especially true when the beam have low transverse reinforcement ratio. The load was applied at the top through a bearing plate at the middle while the specimen was supported on the bottom with two bearing plates. The tie and bearing plates were

designed such that the failure would occur by strut failure. This was achieved by providing sufficiently large tie area and large bearing areas, as shown in Figure 3.4.



(a) Span Length of 800 mm

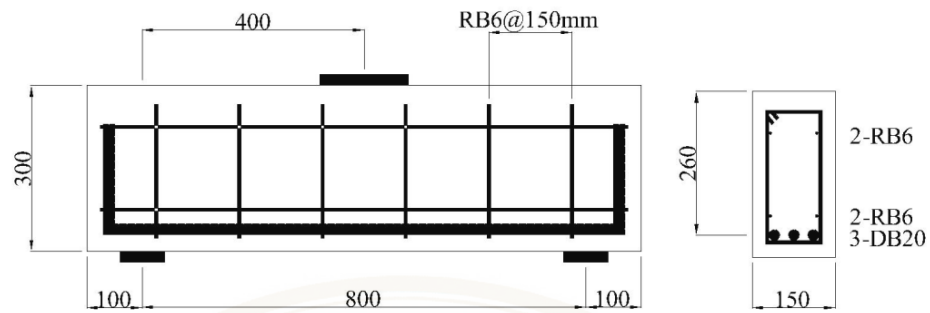


(b) Span Length of 600 mm

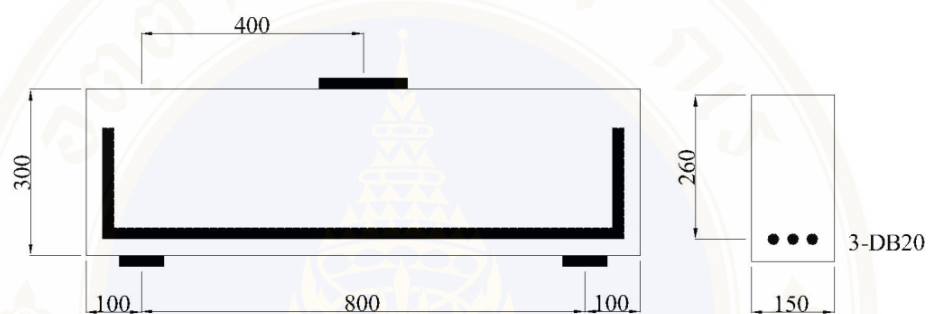
Figure 3.3 Strut-and-Tie Model of Deep Beam Specimens

Table 3.3 Properties of Specimens

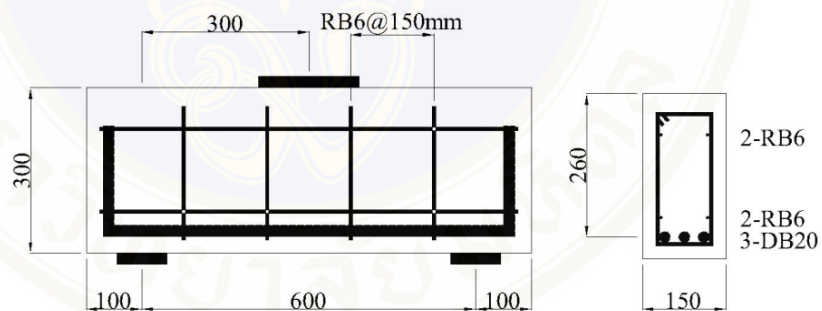
Group	Specimen	Width (mm)	Effective Depth (mm)	Span Length (mm)	Shear Span (mm)	Shear Span to Depth Ratio	Fiber Volume Fraction (%)	Transverse Reinforcement Ratio
A	BRC-800	150	260	800	400	1.54	0.0	0.003
	BPC-800	150	260	800	400	1.54	0.0	-
	BF0.1-800	150	260	800	400	1.54	0.1	-
	BF0.5-800	150	260	800	400	1.54	0.5	-
	BF1.0-800	150	260	800	400	1.54	1.0	-
	BF2.0-800	150	260	800	400	1.54	2.0	-
	BFM2.0-800	150	260	800	400	1.54	2.0	-
B	BRC-600	150	260	600	300	1.15	0.0	0.003
	BPC-600	150	260	600	300	1.15	0.0	-
	BF0.1-600	150	260	600	300	1.15	0.1	-
	BF0.5-600	150	260	600	300	1.15	0.5	-
	BF1.0-600	150	260	600	300	1.15	1.0	-
	BF2.0-600	150	260	600	300	1.15	2.0	-
	BFM2.0-600	150	260	600	300	1.15	2.0	-



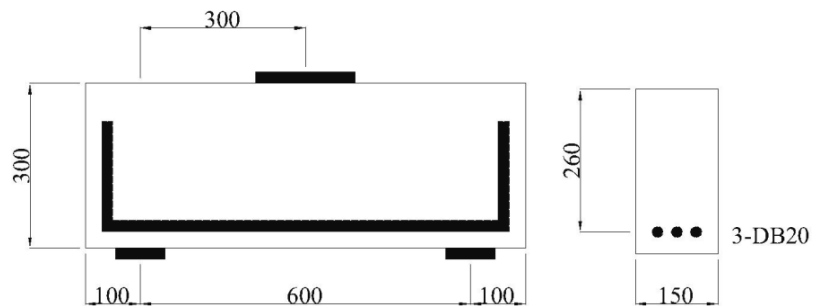
(a) Deep Beam Specimen with Transverse Reinforcement for Group A.



(b) Deep Beam Specimen without Transverse Reinforcement for Group A.



(c) Deep Beam Specimen with Transverse Reinforcement for Group B.



(d) Deep Beam Specimen without Transverse Reinforcement for Group B.

Figure 3.4 Reinforcement Details

3.3.2 Making and Curing

Before casting of concrete, a strain gauge was attached to one of the longitudinal reinforcements at the midspan location. The deep beam specimens were cast in steel molds (Figure 3.5). Three cylinders, three dog-bone shaped and beam specimens were also cast for each mixture for the determination of compressive strength, tensile strength, and flexural strength, respectively.

After casting, the specimens were left under a plastic sheet in the laboratory for one day before being demolded. Then they were moved into a moist curing room, having relative humidity greater than 95%, for 21 days before being tested.

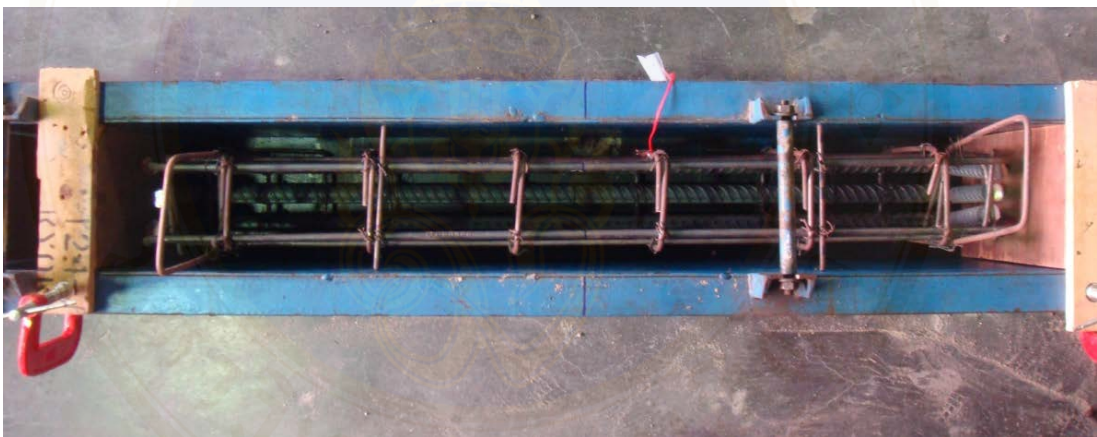


Figure 3.5 Deep Beam Specimen Ready for Casting

3.3.3 Instrumentations

The instrumentations consisted of a strain gauge to monitor reinforcing steel strain and diagonally placed Linearly Variable Displacement Transducers (LVDTs) to measure shear strains across the strut regions. An additional LVDT was installed at the midspan to measure the deflection of the deep beam. Figure 3.6 shows the instrumentations layout. The strain gauge and LVDTs were connected to a data acquisition system for continuous recording of data throughout the duration of the test. A LabVIEW software was written specially for this test.

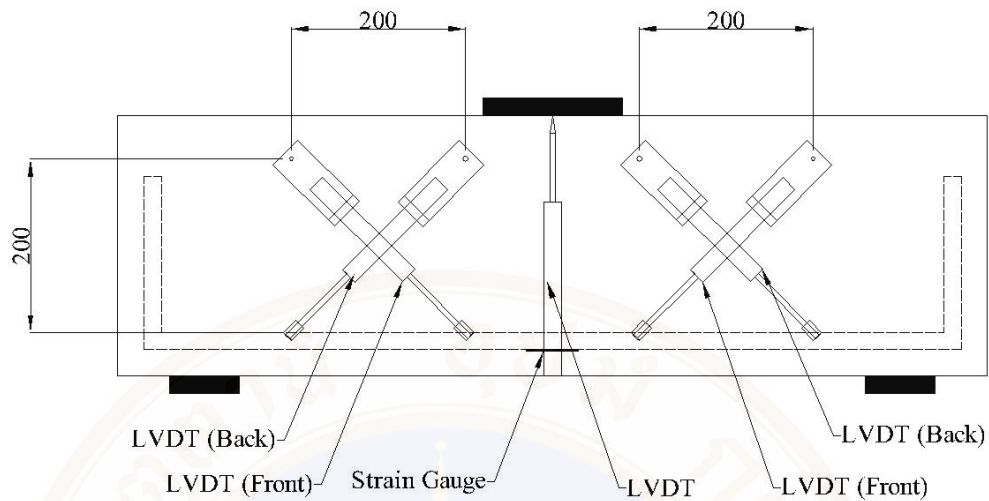


Figure 3.6 Instrumentation Layout

The strain gauge for measuring the strain was of type FLA-5-11, manufactured by Tokyo Sokki Kenhyujo Co., Ltd. The gauge length was 5 mm, with a gauge factor of 2.11 and a resistance of 120 Ω .

The installation of the strain gauge involved grinding the surface of the reinforcement until smooth. The strain gauge was glued to the reinforcement by cyanoacrylate adhesive. It was protected from water by butane tapes and aluminum foil tapes. Figure 3.7 shows the strain gauge installed on a steel reinforcement.



Figure 3.7 Strain Gauge

The shear strain in the strut zone was determined from the data obtained through LVDT placed diagonally over the strut region as shown in Figure 3.8. Two LVDTs were used on each side, one in the back and the other in the front. The average shear strain (γ) was calculated from Equation 3.1.

$$\gamma = \frac{(d'_1 - d_1)d_1 - (d'_2 - d_2)d_2}{2HL} \quad (3.1)$$

where the variables in Equation 3.1 and dimensions are defined in Figure 3.8.

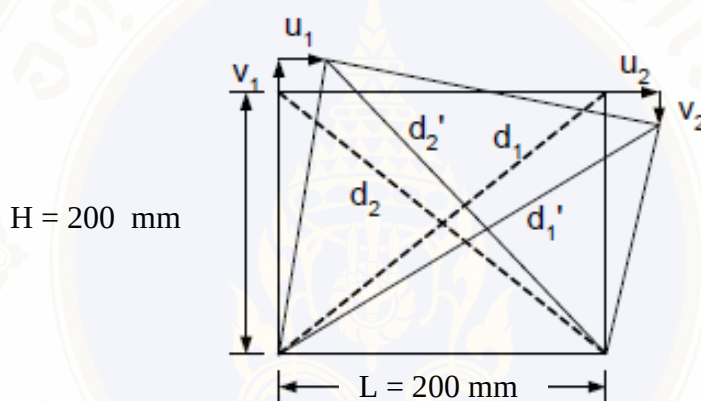


Figure 3.8 Calculation of Shear Strain (adapted from Sittipunt and Wood, (1995))

3.3.4 Testing Procedures

The deep beam specimens were tested at the age of 21 days. Shortly before testing, bearing plates were installed at the support through thread rods embedded in concrete with epoxy adhesive. The bearing plates were 80×150×25 mm for group A (800 mm span length) and 90×150×25 mm for group B (600 mm span length). After installing the bearing plates, the deep beam specimen was placed on the roller support of the testing machine. Then another bearing plate was placed at the top. The bearing plates were 160×150×25 mm for group A and 180×150×25 mm for group B.

Next, LVDTs were attached to both sides across the strut zone and the midspan of the beam through thread rods embedded in concrete with epoxy adhesive. The LVDTs and strain gauges were then connected to the data acquisition system. Figure 3.9 shows the experimental setup.

The load was applied at the midspan by an Instron universal testing machine. The loading rate used was a constant displacement of 1 mm/min. During the testing, the crack patterns of the beam were traced.

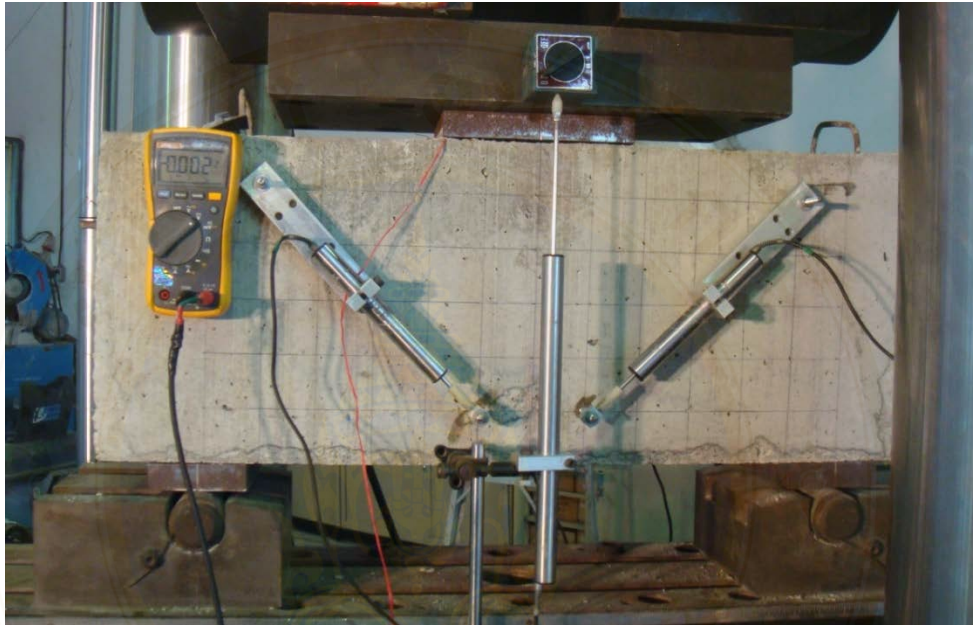


Figure 3.9 Experimental Setup

3.3.5 Material Tests

Compressive strength, direct tensile strength, and flexural strength of the concrete and FRCC mixtures were determined for each mixture. The compressive strength test followed the ASTM C39 (1999) standards. The specimens used were standard Ø100×200 mm cylinder. The ends of the cylinders were capped with neoprene pads with steel retaining rings to ensure uniform load distribution. An ELE 3600 kN capacity compression testing machine was used. The test setup is shown in Figure 3.10.

The compressive strength was computed as:

$$f'_c = \frac{F}{A} \quad (3.2)$$

where:

F = maximum compressive load
 A = cross-sectional area (7854 mm²)



Figure 3.10 The Compressive Strength Test Setup

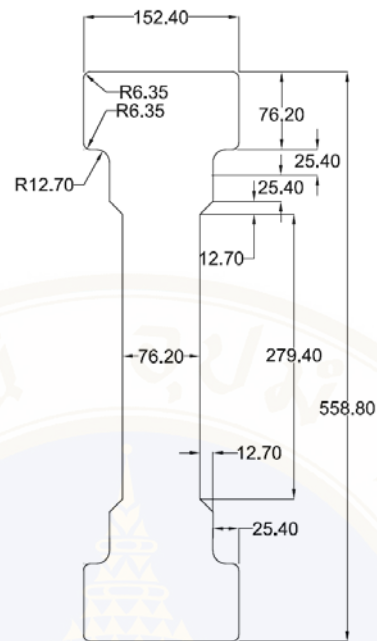
For the direct tensile test, dog-bone shaped specimens having 75×75mm cross-section at the middle portion were used. The dimensions of the specimen are shown in Figure 3.11. The test was performed using an Instron universal testing machine (Figure 3.12). The tensile strains were measured using two LVDTs attached to both sides of each specimen. The loading rate used was a constant displacement of 1 mm/min until the maximum tensile strength is reached. Then, it was increased to 2 mm/min until failure occurred.

The tensile strength (f_t) was calculated as:

$$f_t = \frac{F_t}{A} \quad (3.3)$$

where:

F_t = maximum tensile load
 A = cross-sectional area (5669 mm²)



All dimensions are in mm

Figure 3.11 Dog-Bone Shaped Specimen



Figure 3.12 Tensile Strength Test Setup

For the flexural strength test, 100×100×350 mm beam specimens were used. The test followed ASTM C1609 (2006) standards. The test was performed using an Instron universal testing machine (Figure 3.13). The test rate was 0.1 mm/min until the first crack appeared, then the test rate was increased to 0.4 mm/min until the deflection reached 0.5 mm, then it was further increased to 1 mm/min until the deflection of approximately 5 mm was reached.

The flexural strength (f_r) was calculated as:

$$f_r = \frac{F_b L}{bd^2} \quad (3.4)$$

where:

- F_b = maximum load
- L = span length (300 mm)
- b = width of the section (100 mm)
- d = depth of the section (100 mm)



Figure 3.13 Flexural Strength Test Setup

In addition, the tensile strength of reinforcing bar was performed to determine the yield strength and the ultimate strength of the bar. The strain was measured using an Instron extensometer with 5 mm gauge length. The tensile test followed ASTM A370 (2003) standard. Figure 3.14 shows a typical setup of the tensile test.



Figure 3.14 Reinforcing Steel Direct Tensile Test Setup

CHAPTER IV

RESULTS AND DISCUSSION

4.1 Material Properties

The section presents the material properties of all matrices used. The specimens were cast one mixture a day consecutively. This ensures similar materials and curing condition for all the mixtures.

4.1.1 Compressive Strength

Figure 4.1 shows the average compressive strengths for all the FRCC matrices obtained by testing of three cylindrical specimens. Table 4.1 provides the compressive strength value for each specimen. It can be observed from Figure 4.1 that the compressive strength decreases slightly as fiber content increases. This is confirmed by a linear regression analysis at a significance level (α) of 0.05 (see Appendix B.1). The average compressive strength of FRCC containing 2% of fiber was 12.7% lower than that of the plain concrete matrix. This was probably caused by poorer workability of matrices containing fibers, which led to higher porosity and lower compressive strengths. The overall average compressive strength of concrete matrices was 35.82 MPa. The average compressive strength of the mortar matrices was 42.21 MPa.

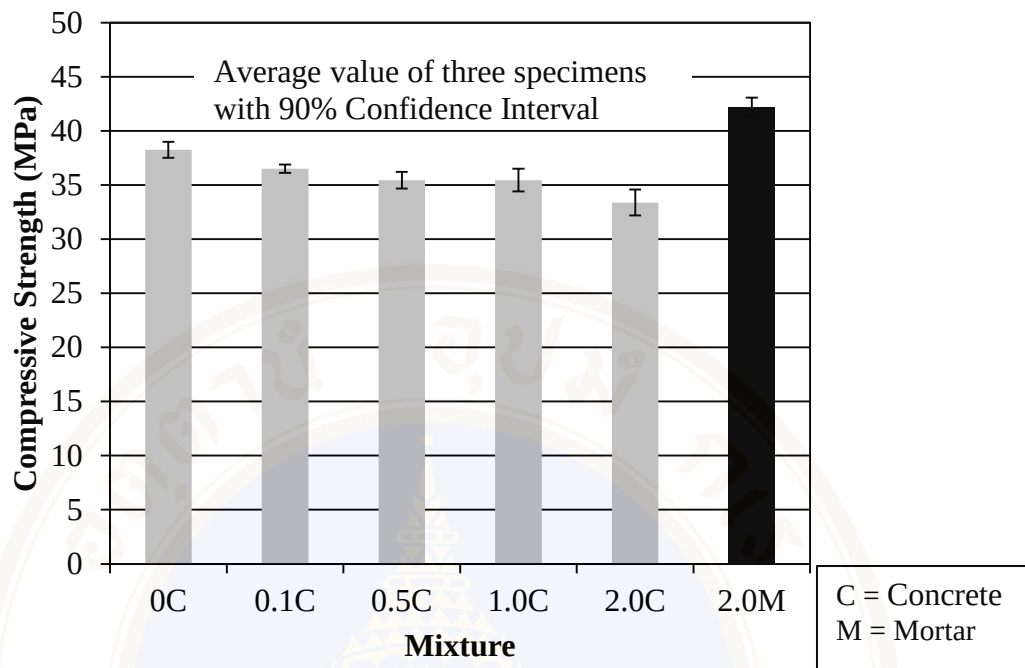


Figure 4.1 Average Compressive Strength

Table 4.1 Compressive Strength Results

Mixture Designation	Compressive Strength (MPa)				CV (%)
	Specimen 1	Specimen 2	Specimen 3	Average	
0C	39.15	37.97	37.68	38.27	2.02
0.1C	35.95	36.05	37.57	36.52	2.49
0.5C	34.34	35.19	36.82	35.45	3.56
1.0C	36.60	34.40	35.41	35.47	3.10
2.0C	32.92	33.66	33.60	33.40	1.22
2.0M	43.12	41.97	41.54	42.21	1.93

4.1.2 Direct Tensile Strength

Figure 4.2 shows representative tensile stress vs. strain plots from the direct tensile tests. It can be observed that plain concrete (no fiber) specimens and FRCC specimens with 0.1% fiber completely lost their tensile capacity once the tensile stress reached the first cracking strength. For specimens having steel fiber contents between 0.5 and 2.0% by volume, there were some residual tensile strengths

after the first cracking. A strain-hardening behavior was observed in specimens having steel fiber contents of 2.0% for both concrete and mortar matrices while FRCC with steel fibers contents of 1.0% or less exhibited strain-softening behaviors. Comparisons of the failure patterns for some of the direct tensile specimens are provided in Figure 4.3.

A plot between average direct tensile strength and the fiber content is shown in Figure 4.4. It can be seen that the direct tensile strength increases with an increase in fiber content for the concrete matrices even in strain-softening mixtures. A regression analysis shows that this trend is significant at 0.05 level (Appendix B.2). For mortar matrix, the data point appeared to have a different trend compared with those of concrete matrices. The direct tensile test results are summarized in Table 4.2.

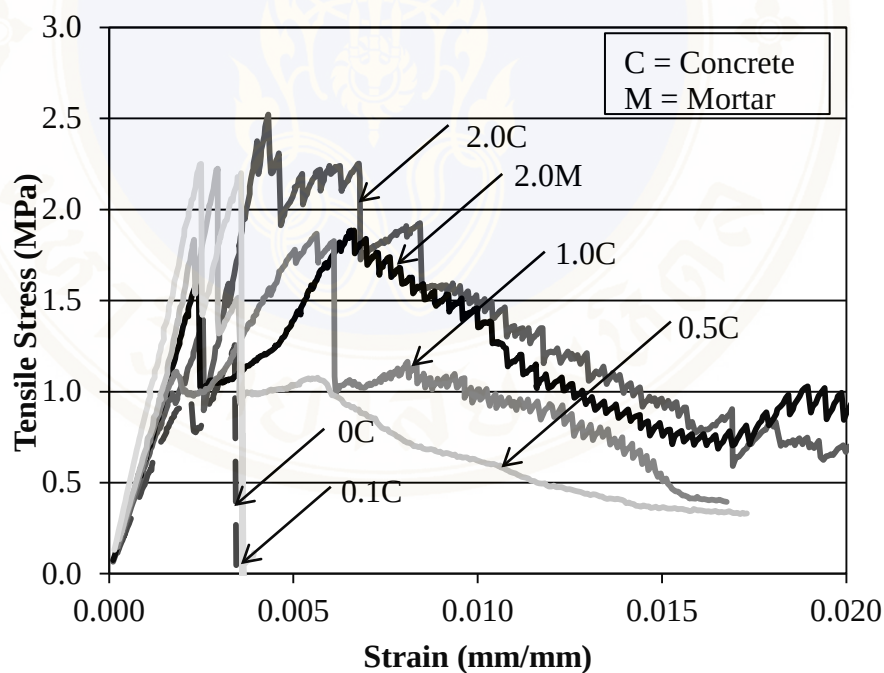


Figure 4.2 Tensile Stress vs. Strain

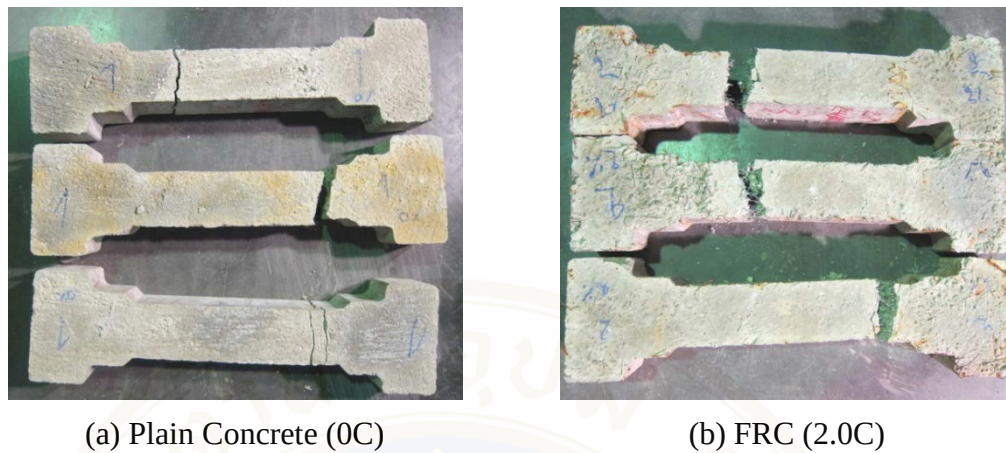


Figure 4.3 Failure Modes of Direct Tensile Strength Specimens

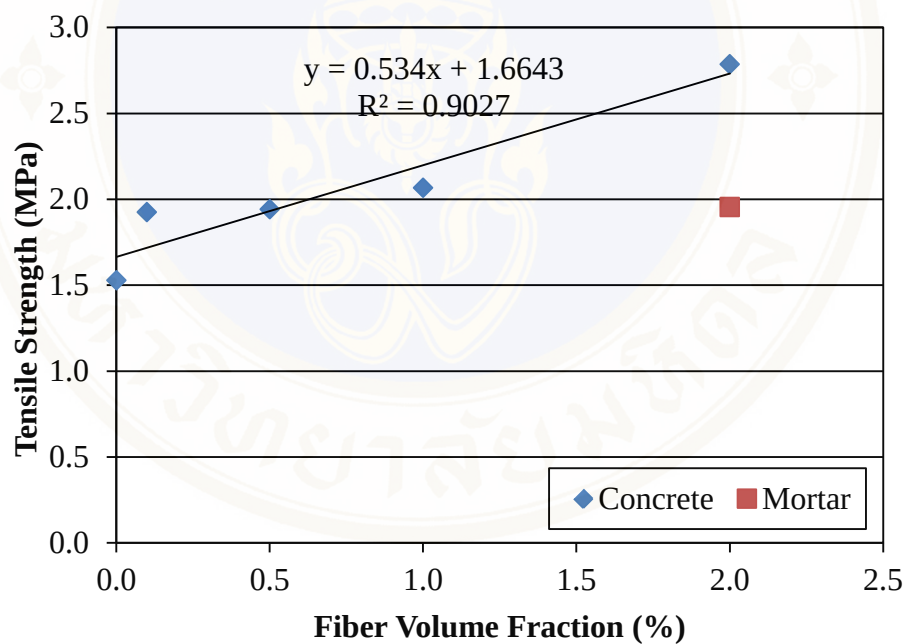
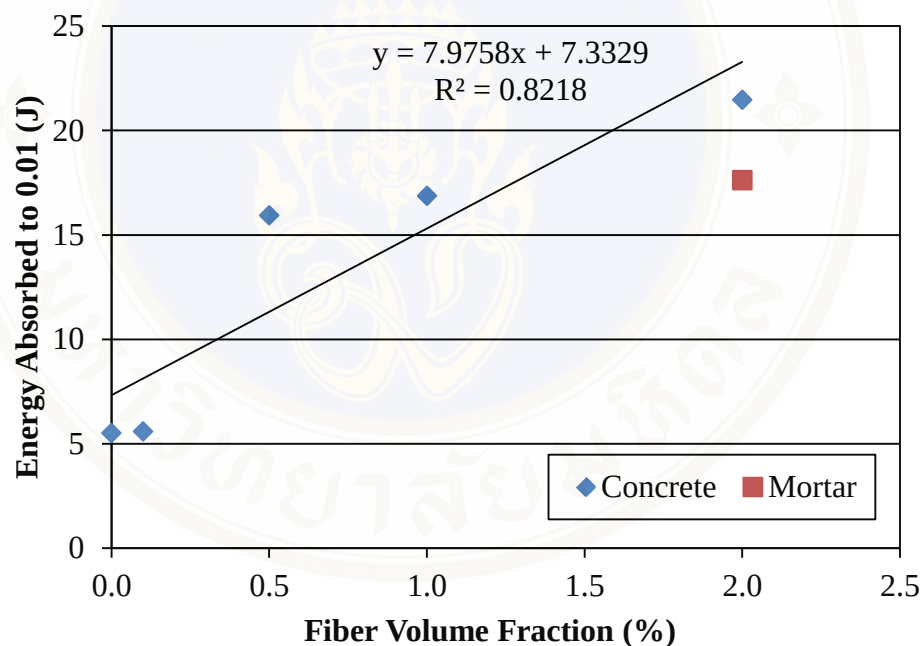


Figure 4.4 Tensile Strength vs. Fiber Volume Fraction

Figure 4.5 shows the relationship between the average energy absorption capacity and fiber content. The energy absorption capacity was defined in this test as the area under the tensile stress-strain curve, to the strain of 0.01mm/mm. It can be observed that the energy absorbed to 0.01 strain increases as the fiber content increases.

Table 4.2 Direct Tensile Test Results

Mixture Designation	Tensile Strength (MPa)					Energy Absorbed to 0.01 Strain	
	Specimens			Avg.	CV (%)	Strain	
	1	2	3			Average (J)	CV (%)
0C	1.70	1.63	1.25	1.53	15.60	5.51	34.07
0.1C	-	1.60	2.25	1.92	23.98	5.58	41.61
0.5C	1.66	2.23	-	1.94	20.68	15.93	13.20
1.0C	1.87	2.26	-	2.07	13.53	16.88	5.38
2.0C	3.08	2.75	2.52	2.79	10.15	21.48	9.33
2.0M	2.27	1.71	1.88	1.96	14.85	17.62	8.90

**Figure 4.5 Energy Absorption Capacity to 0.01 Strain vs. Fiber Volume Fraction**

4.1.3 Flexural Strength

Figure 4.6 shows representative flexural stress-deflection relationships for the mixtures in this study. It can be observed that plain concrete specimens and FRCC specimens with 0.1% fiber content lost their flexural strength immediately after the first cracking. For specimens having steel fiber content between 0.5 and 2.0%, there was some flexural capacity remaining after the first cracking. The greater the fiber

content, the higher the residual flexural strength. Deflection-hardening behaviors were observed in specimens containing 2% steel fibers, for both concrete and mortar matrices. On the other hand, deflection-softening behaviors were observed for FRCCs with steel fibers contents of 1.0% or less. The flexural strength test results are summarized in Table 4.3.

The failure of the all the specimens is from a single crack in the middle third of the span as show in Figure 4.7.

Figure 4.8 shows the plot between average flexural strength and the fiber content. A linearly increasing trend between the flexural strength and the fiber content can be observed, even in deflection-softening mixtures. A regression analysis shows that this linear trend is significant at 0.05 level (Appendix B.3).

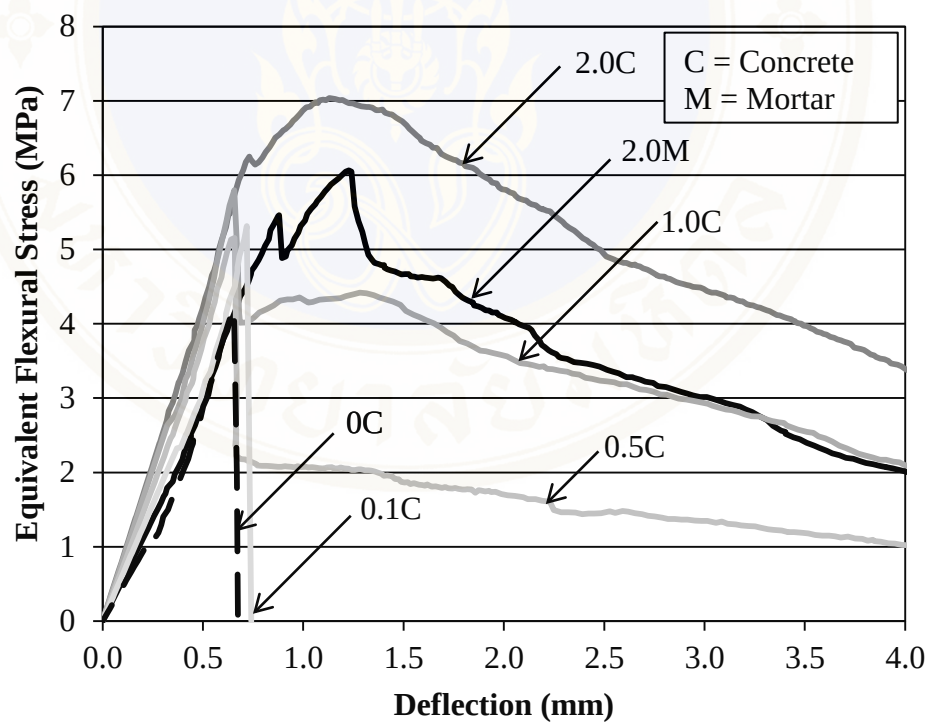


Figure 4.6 Flexural Stress vs. Deflection

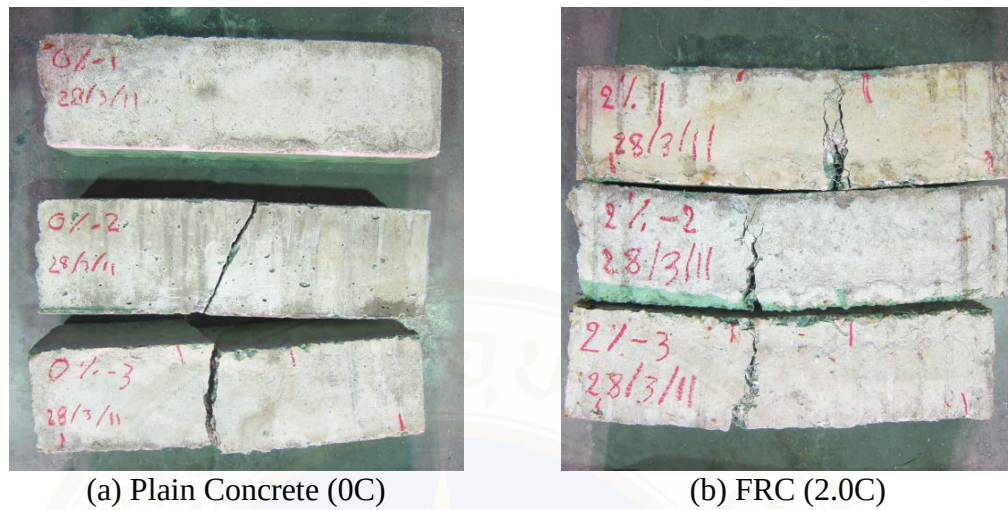


Figure 4.7 Failure Modes of Flexural Strength Specimens

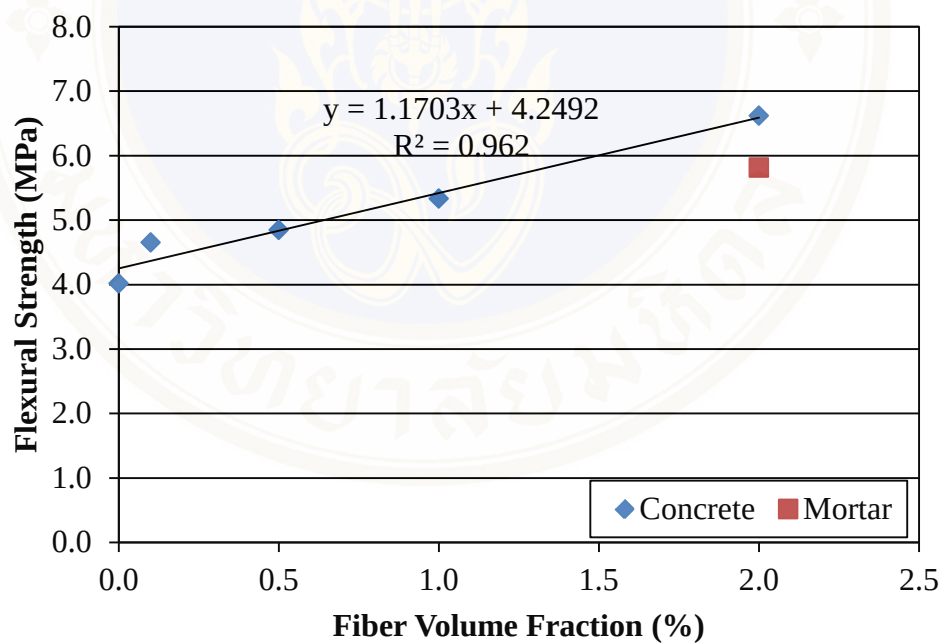


Figure 4.8 Flexural Strength vs. Fiber Volume Fraction

Figure 4.9 shows the relationships between the fiber content and average energy absorption to the deflection of L/300 and L/150, respectively. It can be observed that the energy absorption capacity increases linearly as the fiber content increases.

Table 4.3 Flexural Strength Test Results

Mixture Designation	Flexural Strength (MPa)					Energy Absorbed (J)			
	Specimens			Avg.	CV (%)	L/300		L/150	
	1	2	3			Avg.	CV (%)	Avg.	CV (%)
0C	3.65	4.19	4.20	4.01	7.79	4.71	25.31	4.71	25.31
0.1C	5.32	4.48	4.16	4.65	12.88	5.03	9.47	5.03	9.47
0.5C	5.16	4.77	4.61	4.85	5.74	7.40	5.18	13.18	14.03
1.0C	5.40	4.80	5.80	5.33	9.45	9.74	7.27	23.70	3.86
2.0C	7.12	5.69	7.04	6.62	12.11	13.41	15.05	33.15	13.55
2.0M	-	5.57	6.06	5.81	6.06	10.15	7.87	26.42	2.61

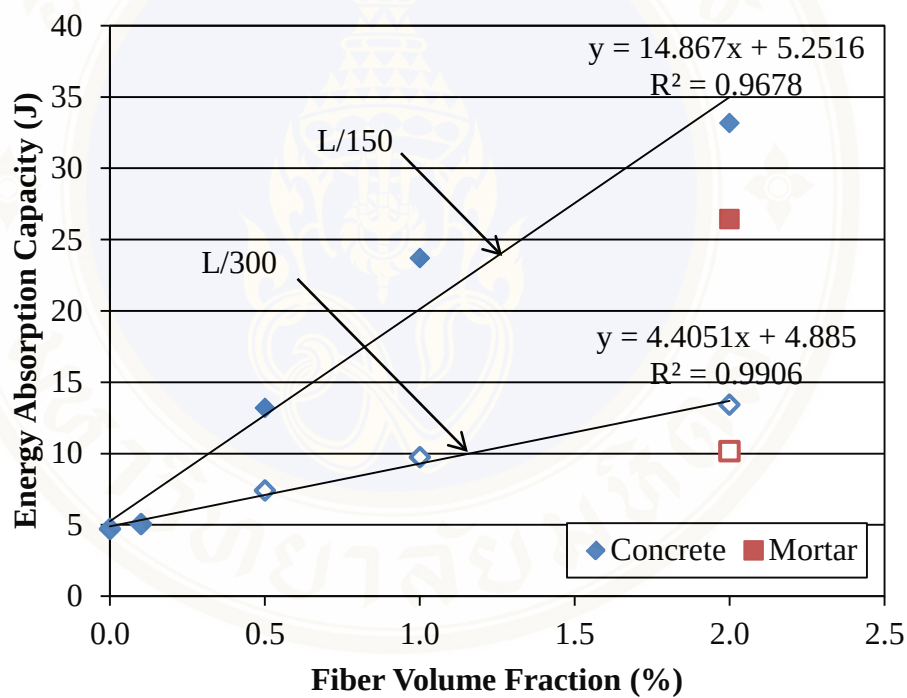
**Figure 4.9 Energy Absorption Capacity vs. Fiber Volume Fraction**

Figure 4.10 shows the plot of average tensile strength vs. average flexural strength. It can be observed that the tensile strength correlates well with the flexural strength.

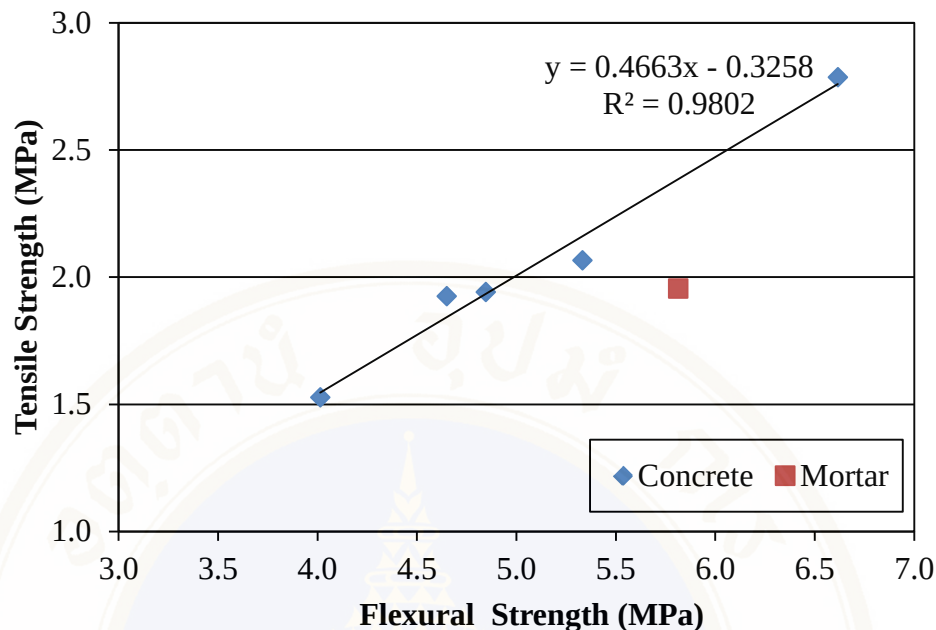


Figure 4.10 Tensile Strength vs. Flexural Strength

4.2 Deep Beam Specimens

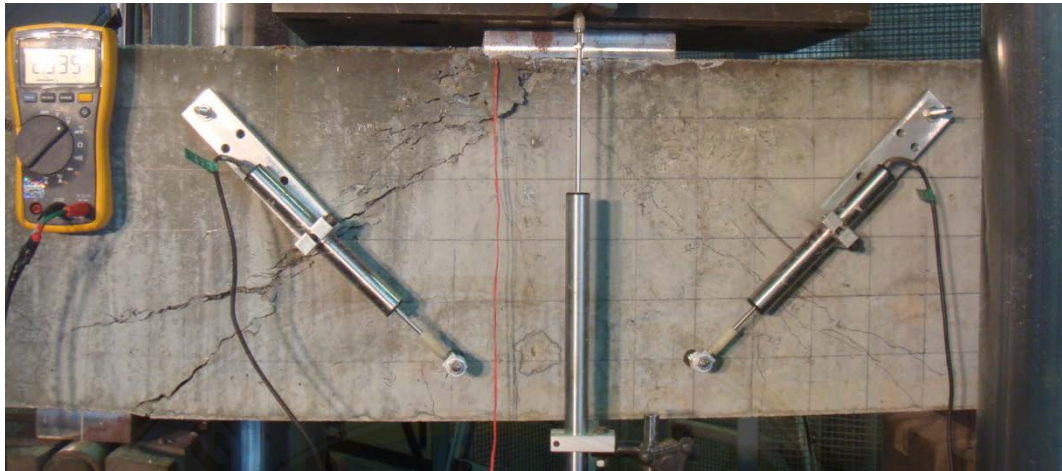
This section presents the test results and the analyses of the data for the deep beam specimens.

4.2.1 Overall Behavior

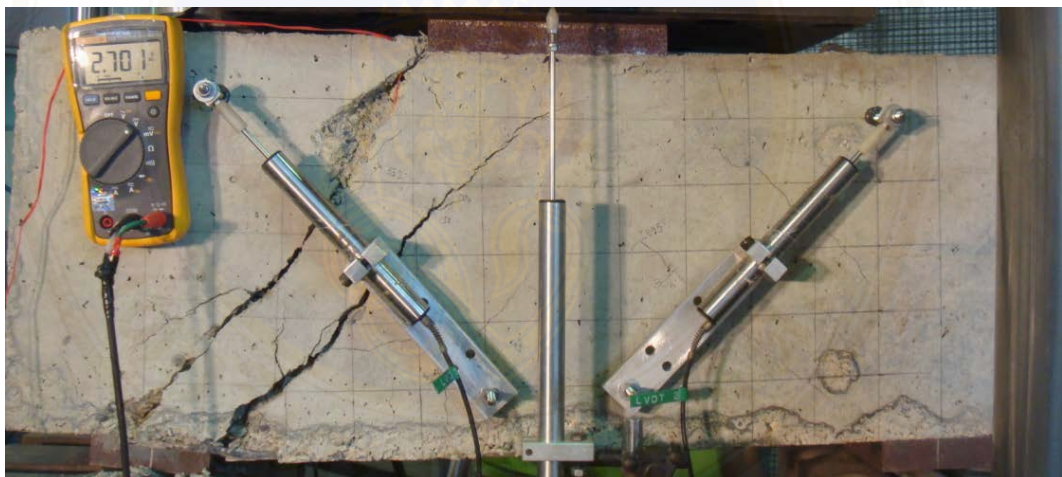
When a deep beam specimen was loaded using a displacement controlled mode, diagonal cracks initially formed on both left and right sides of the specimens. However, only one of the diagonal cracks increased in width as the imposed displacement increased, while the other remained small in width. The critical diagonal cracks occurred randomly on either side, depending on which side had a lower strength. In addition, the critical diagonal crack was not necessarily the first diagonal crack formed.

At some point, the maximum load-carrying capacity was reached. The critical diagonal crack continued to increase in width and led to a decrease in the load-carrying capacity. Typical cracking patterns of deep beam specimens at failure are shown in Figure 4.11.

Test results of the deep beam specimens are summarized in Table 4.4 and Table 4.5.



(a) 800 mm Span Specimen



(b) 600 mm Span Specimen

Figure 4.11 Typical Failure Modes

Table 4.4 Summary of Test Results for 800 mm Specimens

Specimen	BPC-800	BRC-800	BF0.1-800	BF0.5-800	BF1.0-800	BF2.0-800	BFM2.0-800
Maximum Load (kN)	488.76	308.38	356.86	451.86	355.10	384.18	295.90
First Diagonal Cracking (kN)	150.00	150.00	120.00	175.00	140.00	210.00	180.00
Maximum Shear Stress (f'_c , MPa)	0.34	0.23	0.33	0.30	0.28	0.00	0.19
Maximum Shear Stress ($\times \text{sqrt}(f'_c)$, MPa)	2.06	1.36	1.73	1.87	1.59	1.54	1.20
Deflection at Peak (mm)	4.53	3.23	4.68	3.67	5.35	3.50	2.81
Shear Strain at Peak (mm/mm)*	0.006	0.003	0.004	0.004	0.002	0.002	0.002
Failure Side	Right	Left	Left	Left	Left	Left	Left
Strut Angle (degree)*	40	45	40	40	45	40	42
Max Strut Stress (ratio of f'_c)	1.36	0.86	1.33	1.22	1.02	0.98	0.74
Deflection at Failure (mm)	4.59	5.79	5.61	4.35	7.83	4.56	6.79
Deflection at Failure (% peak)	101%	179%	120%	119%	146%	130%	241%
Shear Strain at Failure (mm)	0.007	0.008	0.010	0.008	0.005	0.005	0.023
Shear Strain at Failure (% peak)	109%	253%	229%	215%	253%	231%	10210%
Compressive Strength (f'_c , MPa)	37.17	33.97%	27.91	38.36	32.92	40.71	34.86
Subgroup for Strength	H	N	N	H	N	H	H

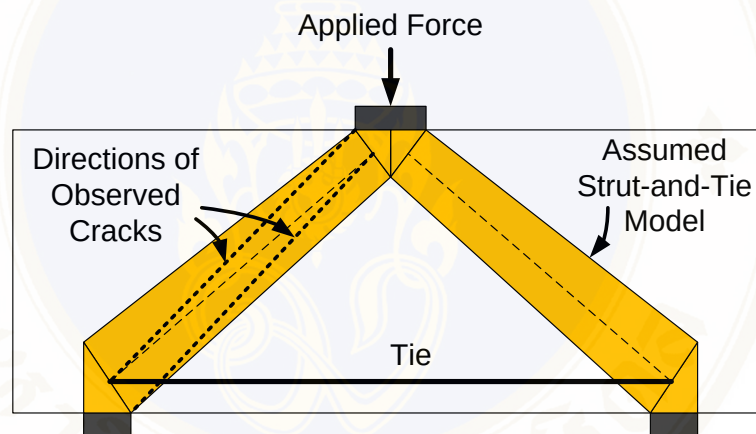
* Measured on the failure side

Table 4.5 Summary of Test Results for 600 mm Specimens

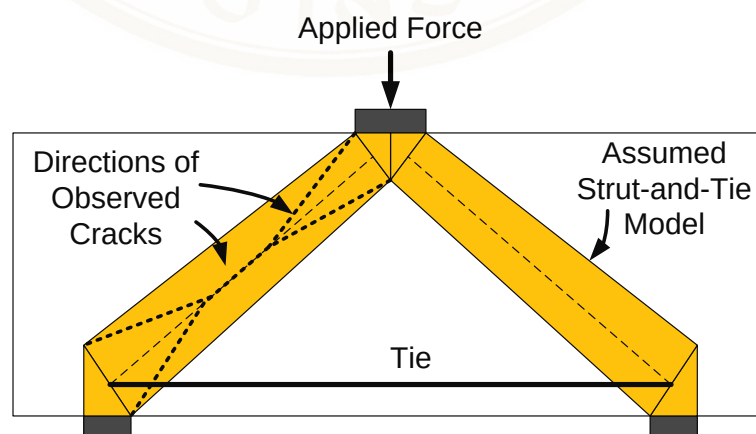
Specimen	BPC-600	BRC-600	BF0.1-600	BF0.5-600	BF1.0-600	BF2.0-600	BFM2.0-600
Maximum Load (kN)	332.76	459.73	562.65	458.83	492.50	496.77	617.50
First Diagonal Cracking (kN)	160.00	140.00	150.00	200.00	190.00	200.00	200.00
Maximum Shear Stress (f'_c , MPa)	0.42	0.54	0.38	0.55	0.61	0.50	0.40
Maximum Shear Stress ($\times \text{sqrt}(f'_c)$, MPa)	1.89	2.52	2.33	2.54	2.76	2.48	2.51
Deflection at Peak (mm)	1.74	3.43	2.76	2.89	3.12	3.74	4.30
Shear Strain at Peak (mm/mm)*	0.002	0.005	0.004	0.002	0.003	0.007	0.003
Failure Side	Right	Left	Left	Right	Right	Left	Left
Strut Angle (degree)*	55	50	55	55	50	57	60
Max Strut Stress (ratio of f'_c)	1.33	1.83	1.20	1.74	2.05	1.49	1.20
Deflection at Failure (mm)	1.83	5.34	3.69	5.54	4.26	9.92	7.67
Deflection at Failure (% peak)	105%	156%	134%	191%	137%	265%	178%
Shear Strain at Failure (mm)	0.002	0.012	0.007	0.014	0.009	0.040	0.023
Shear Strain at Failure (% peak)	117%	251%	176%	819%	296%	598%	673%
Compressive Strength (f'_c , MPa)	20.36	21.90	38.23	21.47	20.87	26.45	39.78
Subgroup for Strength	L	L	H	L	L	L	H

* Measured on the failure side

The diagonal crack was formed by transverse tensile stress across the compressive strut extending from one of the supports to the loading point. However, the cracks locations appeared to deviate from the assumed strut-and-tie model. For specimens with fiber contents of 1.0% by volume or less, it was observed that the diagonal crack either extended from the edge of the top bearing plate to the centerline of the tie, or from the edge of the support bearing plate to about the middle of the assumed nodal size of the top node (determined based on the strength of the concrete and the applied force at the top node), as shown in Figure 4.12a. In some specimens, both cracks can be seen. But in most specimens, only one of the two was found.



(a) Specimens with 1.0% Fiber Content or Lower

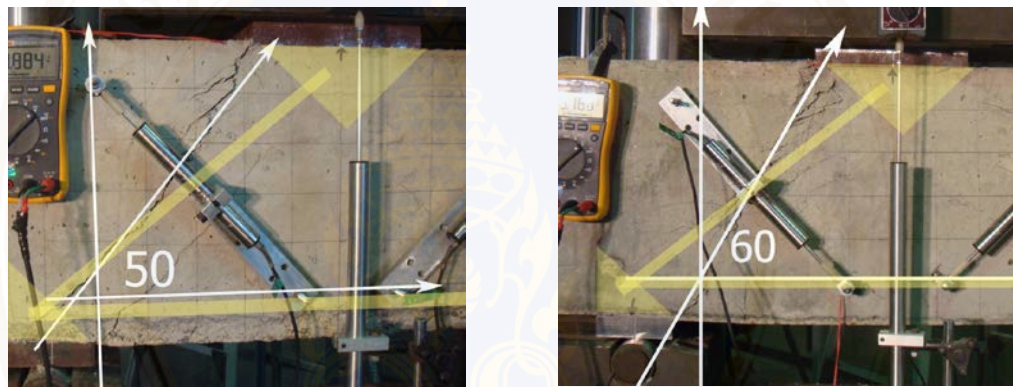


(b) Specimens with 2.0% Fiber Content

Figure 4.12 Locations of Diagonal Cracks Relative to Assumed Model

On the other hand, for specimens with fiber content of 2.0% (either mortar or concrete matrix), a different crack pattern was observed. Y-shaped crack patterns were formed next to the top and bottom nodes (Figure 4.12b). Typical examples of these two types of crack pattern are shown in Figure 4.13.

In any cases, the diagonal cracks had steeper angles than those assumed during the design. The actual crack angles were measured after the test. Figure 4.14 shows the plot between crack angle and the volume content of steel fibers. Overall, the crack angle was about the same for each span length.



(a) Specimens with 1.0% Fiber Content or Lower (BRC-60) (b) Specimens with 2.0% Fiber Content (BFM2.0-60)

Figure 4.13 Crack Patterns Relative to Assumed Model

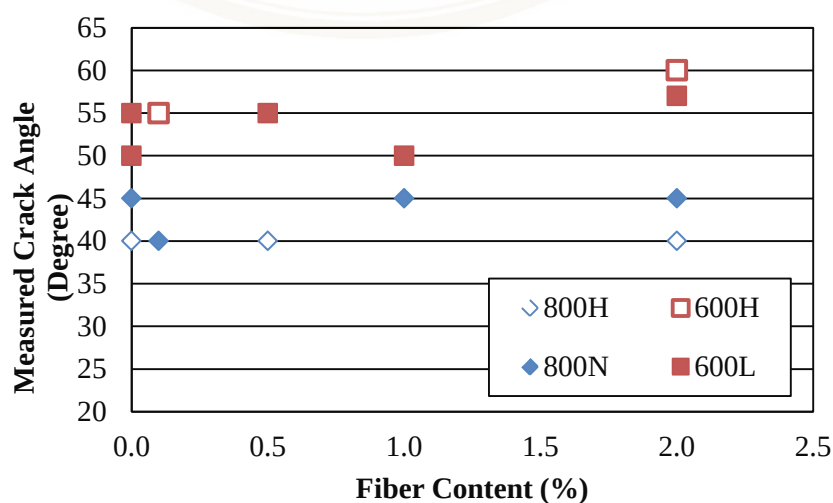


Figure 4.14 Angle of Strut vs. Fiber Content

A parametric study was conducted to investigate whether the steeper crack angle can be caused by the fact that the actual nodal efficiency was different from those assumed in the design (during the design, it was assumed that the strut efficiency was equal to 1.0 while ACI 318's nodal efficiency was used for all nodes), resulting in shallower depth of the top nodes (Figure 4.15). In addition, the effective area resisting the bearing force at the supports may be less than the full length of the bearing plate (Figure 4.15). The combinations of the two may result in strut angles higher than those assumed and closer to the measured angles.

Changes in the nodal dimensions leads to the changes in the strut force and the ratio between the strut force and the strength of the strut at each end. This ratio is called strut efficiency. The strut efficiency at each end varies with the height of the upper node and length of the lower node, shown as surface plot in Figure 1.16. At failure of the member, the internal stresses in the beam should be aligned such that the strut efficiency is constant throughout the member, which is the intersection between the two surfaces. Further, if we assumed that the failure occurs by strut failure (as in the case for this test) and at the strut efficiency of 1.0, the height of the upper node and the length of the lower node can be solved, and the strut angle corresponding to this condition can be calculated.

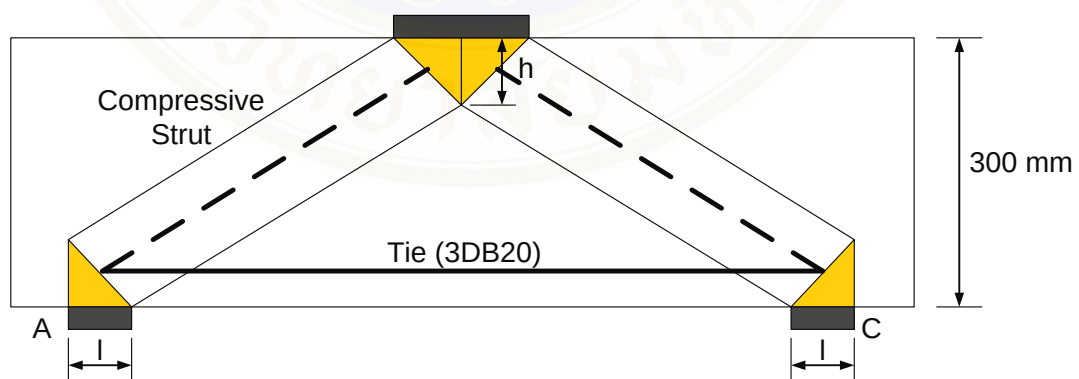
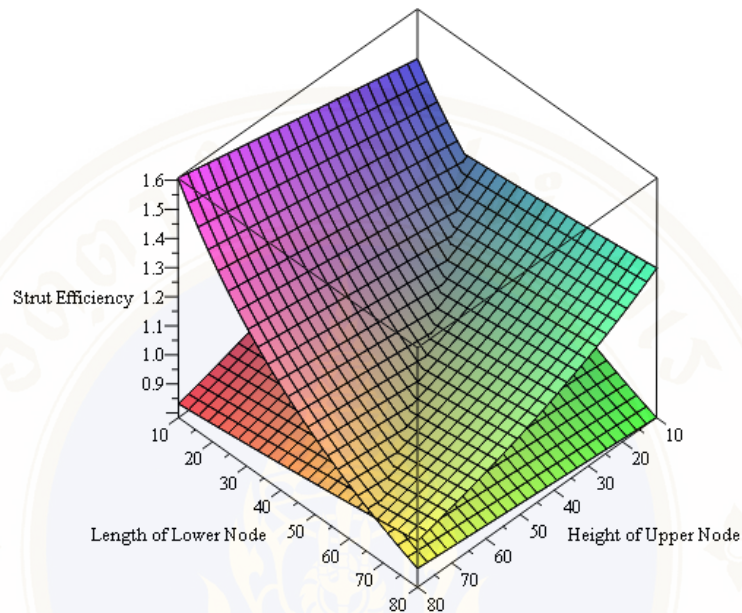


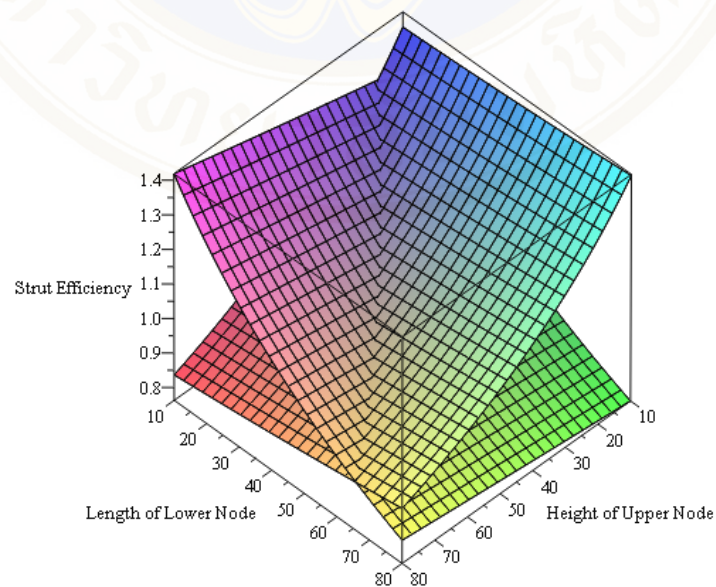
Figure 4.15 Strut and Node Dimensions

At the strut efficiency equal to 1, it can be observed that, depending on the assumptions used for the strut and node efficiencies, the angle between strut and tie for the specimens with 800 mm and 600 mm span lengths were between 27.9°-32.2° and

36.3°-41.1°, respectively. In any cases, these angles were not substantially more than the assumed angle and are much lower than the measured crack angles (Figure 4.14).



(a) 800 mm Span



(b) 600 mm Span

Figure 4.16 Strut Efficiency vs. Node Dimensions

Table 4.6 Strut Angle for Various Assumptions of Strut and Node Efficiencies

Case	Strut and Node Efficiency			800 mm Span Specimens			600 mm Span Specimens		
	β_s	$\beta_{n,upper}$	$\beta_{n,lower}$	h_{upper} (mm)	l_{lower} (mm)	α (deg)	h_{upper} (mm)	l_{lower} (mm)	α (deg)
1 (Design)	1.0	0.85	0.80	80	80	28.8	80	90	36.3
2	1.0	1.0	1.0	48.2	29.5	32.2	42.2	36.8	41.1
3	1.0	1.0	0.9	51.6	51.6	31.3	45.4	53.8	40.1
4	1.0	0.90	0.85	69.7	73.7	29.6	64.0	69.2	38.2
5	1.0	0.85	0.80	83.9	102.2	27.9	77.5	88.2	36.5

It should be noted that no significant sign of bearing or anchorage failure was observed from the test. Some vertical cracks near the midspan of the beam and splitting cracks along the reinforcements were observed in some specimens but they remained hairline-sized throughout the test.

It was observed that the specimens without fibers or transverse reinforcement failed almost immediately after the formation of the diagonal crack. For the other specimens, they were able to support additional loads beyond the point of first diagonal cracking and deformed beyond the point of peak load.

Figure 4.17 and Figure 4.18 show the comparison of damages on the critical strut at the deflection of approximately $L/120$ for all the FRCC matrices. For specimens without transverse reinforcements, it can be observed that the higher the fiber content, the smaller the maximum crack width. In specimens with relatively small amount of fibers, namely 0.1 and 0.5%, only one diagonal crack was observed. For specimens having fiber volume content greater than 1.0%, several parallel diagonal cracks can be seen. This effectively resulted in a reduction of the crack width of the main diagonal cracks. Compared to specimens with transverse reinforcement, FRCC specimens with 0.5% or more fiber contents exhibited comparable damage.

The specimens in this project were made and tested during a period of over a year. It was found that variations of the cement quality and curing conditions greatly affected the quality of concrete obtained, as can be seen by the variations in the compressive strength for the concrete used for the deep beam specimens. Because the behaviors of deep beam may be affected by the concrete strength (as summarized in Table 4.4 and Table 4.5), the specimens were analyzed in subgroups with similar

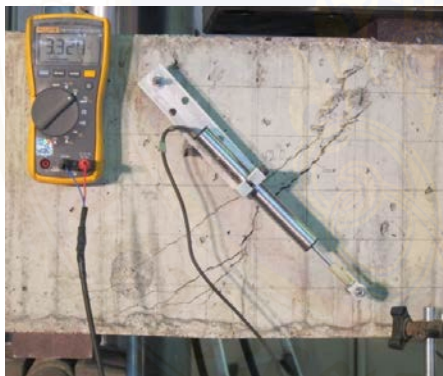
compressive strength to avoid misleading results. There are two subgroups in each of the span length group. The subgroups are denoted in Table 4.4 and Table 4.5 as H, N, and L for high, normal, and low compressive strengths, respectively.



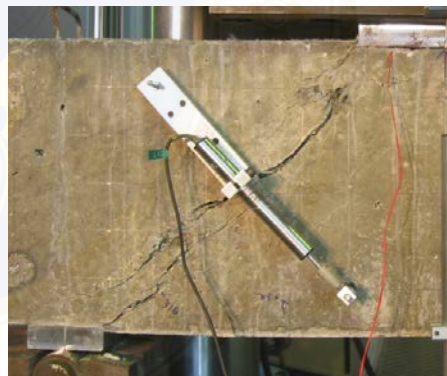
(a) BRC-800



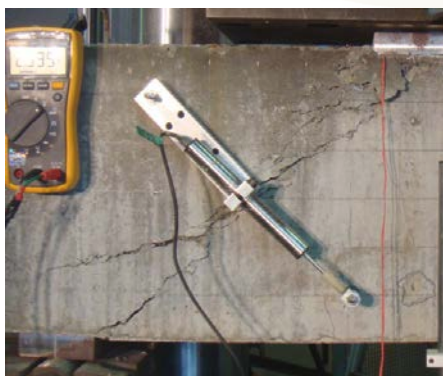
(b) BF0.1-800



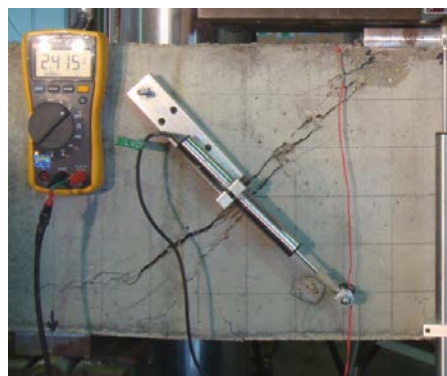
(c) BF0.5-800



(d) BF1.0-800



(e) BF2.0-800



(f) BFM2.0-800

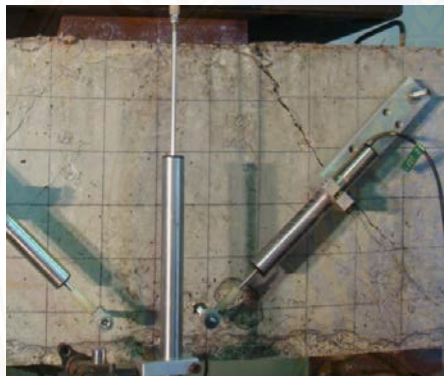
Figure 4.17 Damages at the Deflection of Approximately $L/120$ for 800 mm Span Specimens



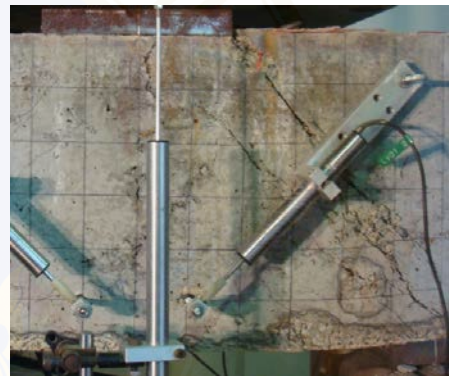
(a) BRC-600



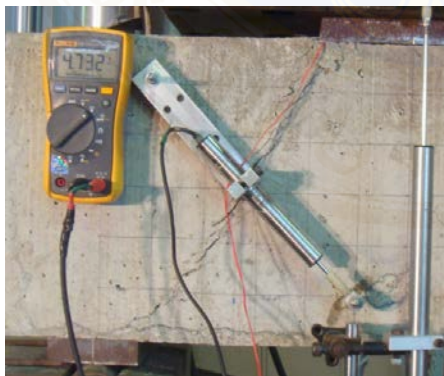
(b) BF0.1-600



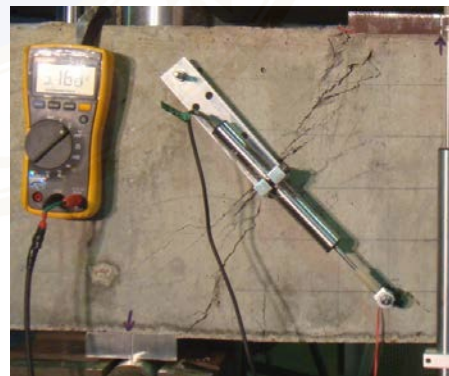
(c) BF0.5-600



(d) BF1.0-600



(e) BF2.0-600



(f) BFM2.0-600

Figure 4.18 Damages at the Deflection of Approximately $L/120$ for 600 mm Span Specimens

4.2.2 Load vs. Deflection

Figure 4.19 and Figure 4.20 show load-deflection relationships from the tests. In general, the load-deflection followed a linear trend until almost the point of the maximum load. A slight change in the load-deflection slope can be observed at the load of between 140-210 kN for 800 mm span specimens, and between 140-230 kN for 600 mm span specimens. These were identified as the points where diagonal cracks occurred. After the maximum load, most specimens can retain some of the capacity without failure.

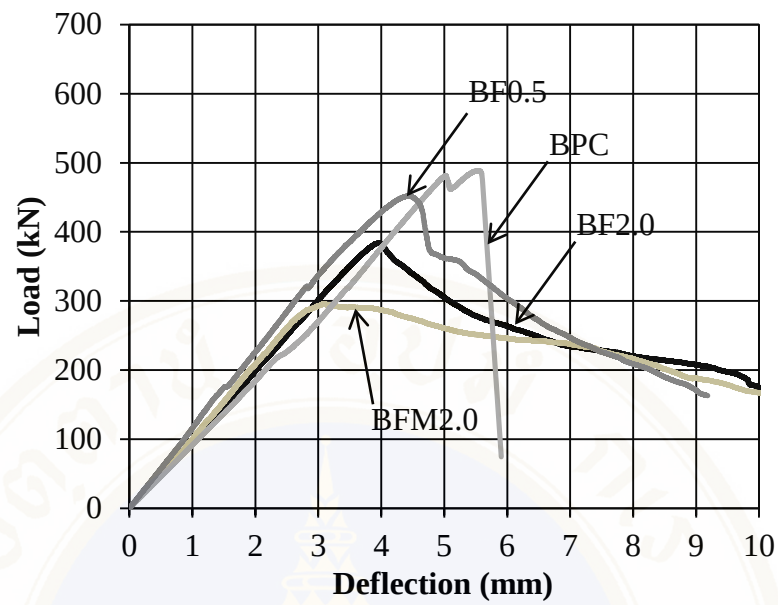
The load at the first cracking and the maximum load are plotted with the volume fraction of fiber in Figure 4.21 and Figure 4.22, respectively. For reinforced concrete, Brown et al. (2006) found that the load at first diagonal cracking was unaffected by the transverse steel reinforcement ratio. However, Figure 4.21 shows that the load at the first diagonal crack tended to increase slightly as the fiber content increases, especially at fiber content between 0-0.5%. This can be explained by an increase in tensile strength with the fiber content (Figure 4.4).

On the other hand, contradicting trends were observed for the maximum load (Figure 4.22) between the specimens with 800 mm and 600 mm span lengths. For specimens with 600 mm span length, the maximum load increases slightly as the fiber content increases. However, for specimens with 800 mm span length, the maximum load is not noticeably affected by the fiber content, or even decreases as the fiber content increases.

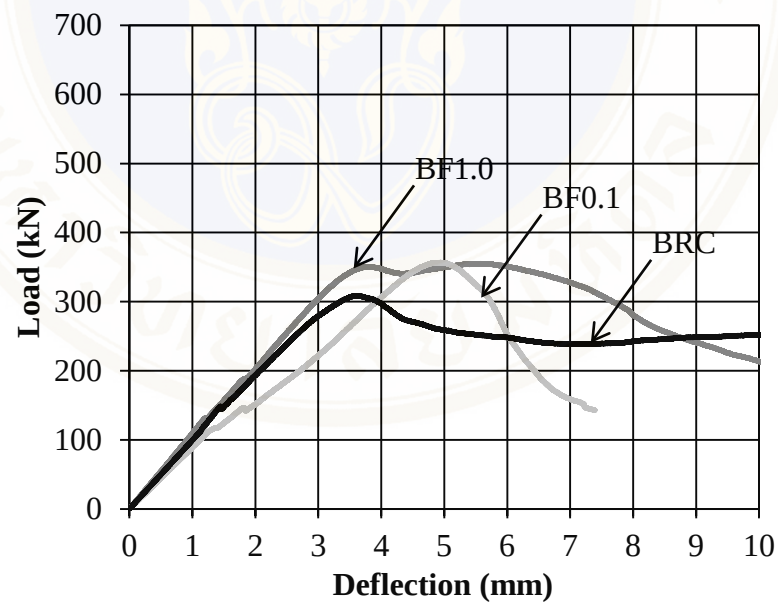
A similar conclusion can be drawn by observing the load-deflection curves (Figure 4.19 and Figure 4.20). In specimen with 600 mm span, load-deflection behavior became more ductile as fiber content increases. However, this effect was not as noticable in 800 mm span specimens. This indicates that the angle of strut has an effect on the efficiency of fiber and the strength of the deep beam member.

Figure 4.23 shows the plot between deflection at failure and the fiber volume content. It was observed that the deflection at failure tended to increase as fiber content increases.

In general, the use of steel fiber at a volume content of 0.5% provides comparable, if not better, behavior to the RC specimens containing minimum transverse reinforcements.

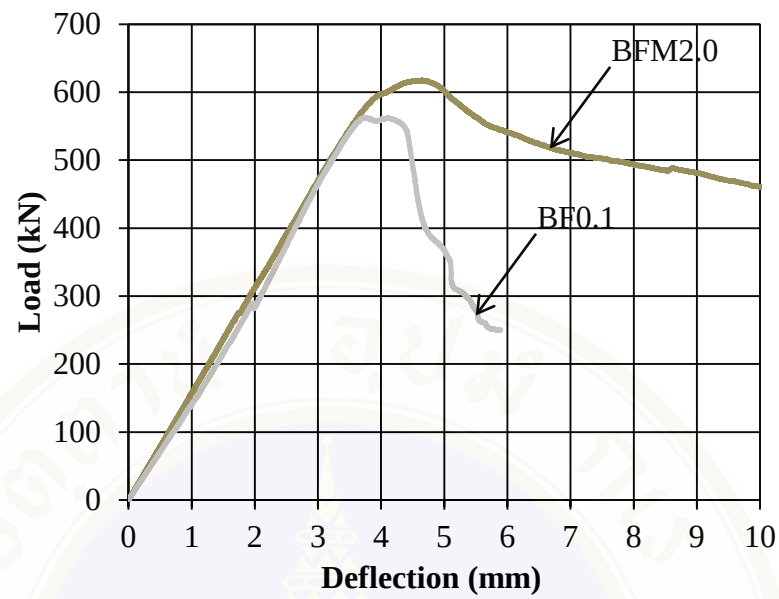


(a) Subgroup H

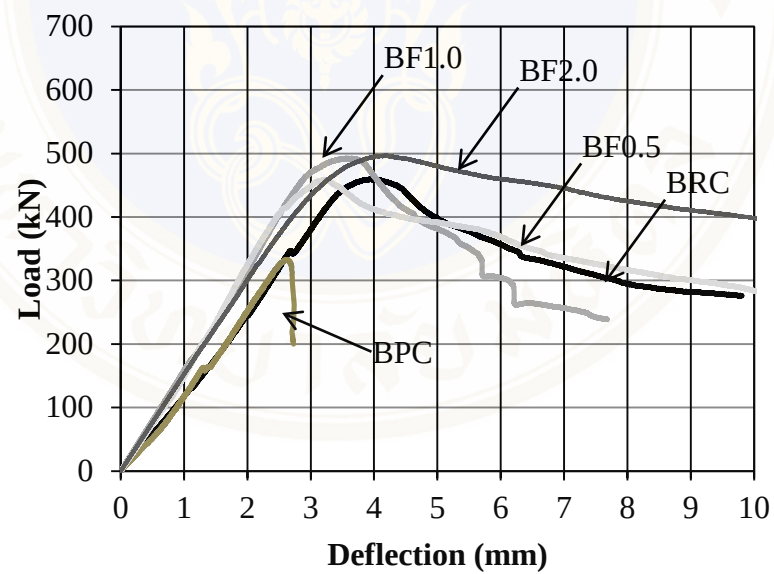


(b) Subgroup N

Figure 4.19 Load vs. Deflection for 800mm Span Specimen



(a) Subgroup H



(b) Subgroup L

Figure 4.20 Load vs. Deflection for 600mm Span Specimen

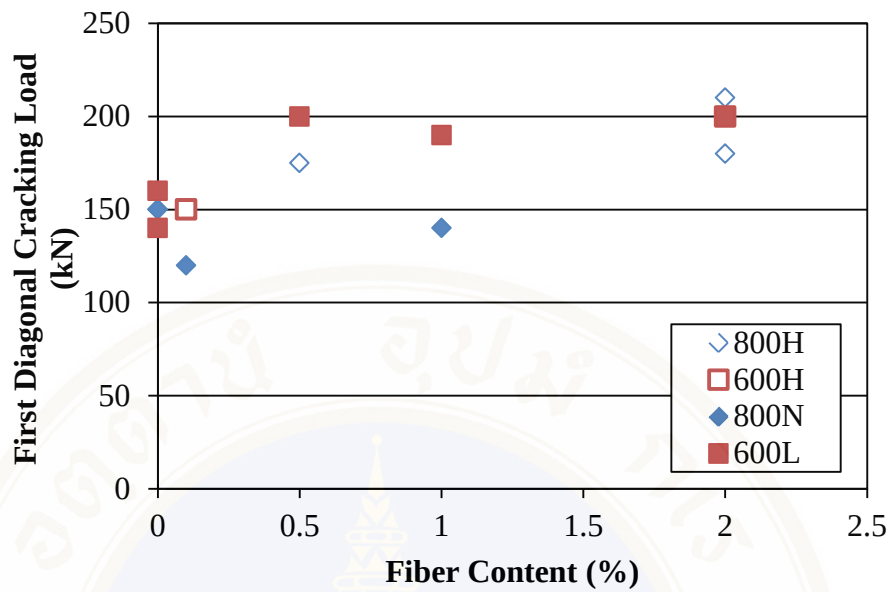


Figure 4.21 First Diagonal Cracking Load vs. Fiber Content

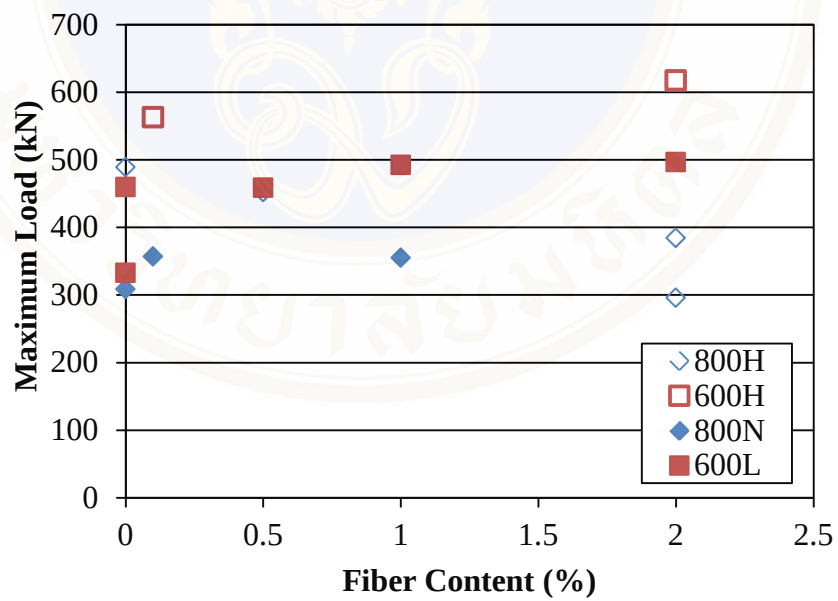


Figure 4.22 Maximum Load vs. Fiber Content

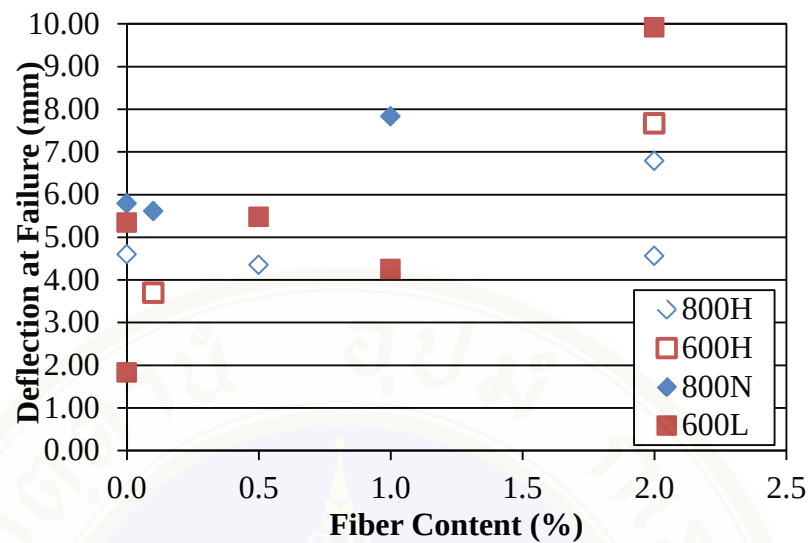


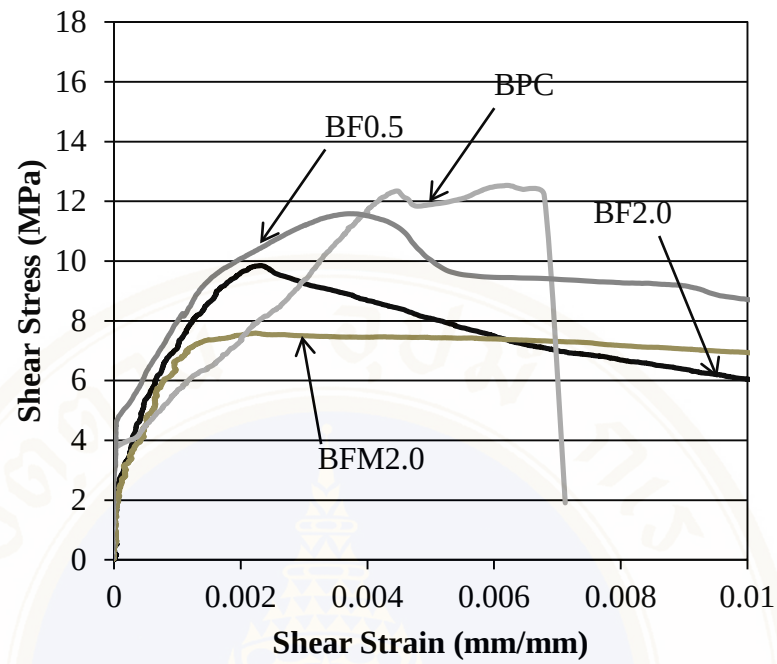
Figure 4.23 Deflection at Failure vs. Fiber Content

4.2.3 Shear Stress vs. Shear Strain

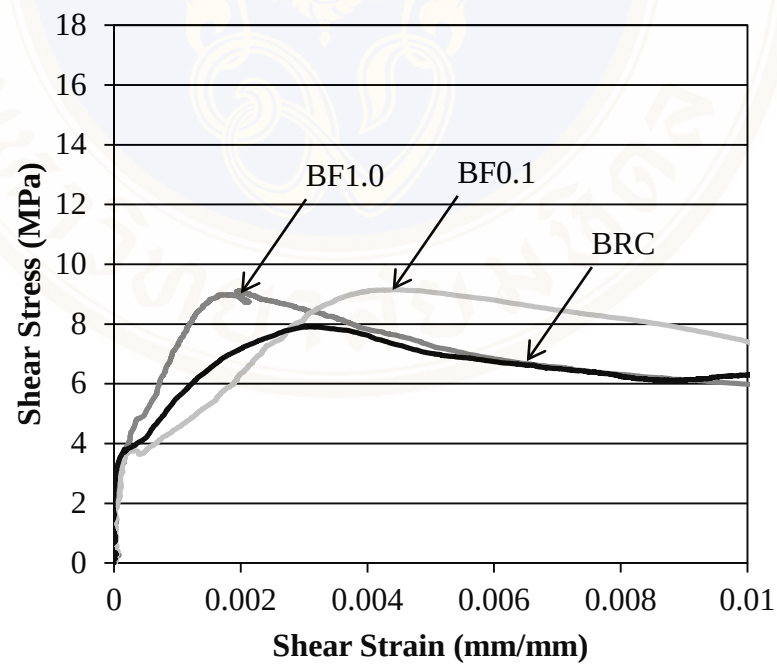
The shear stress is defined as a ratio of the applied load and the product of beam width and effective depth. The shear strain of the beam is calculated based on the measurements of diagonal LVDTs using the formula in section 3.3.3.

Figure 4.24 and Figure 4.25 show the plot of shear stress vs. shear strain on the failure side. In general, the shear strain was almost negligible at shear stress, below 2-4 MPa. After that, diagonal crack occurred and led to a sudden increase in the shear strain.

In most cases, the peak shear stress is reached when the shear strain was about 0.002-0.004. Figure 4.26 provides comparison of the diagonal crack at various levels of shear strain.

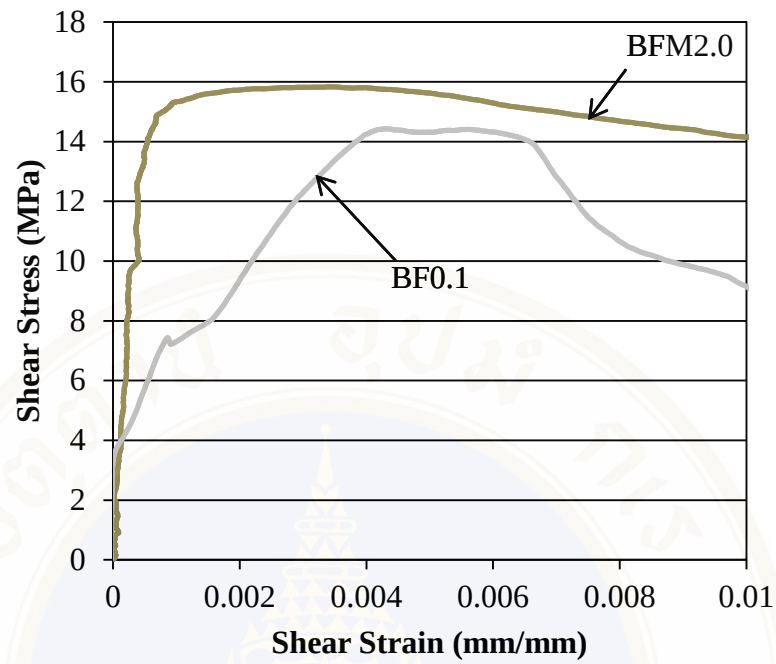


(a) Subgroup H

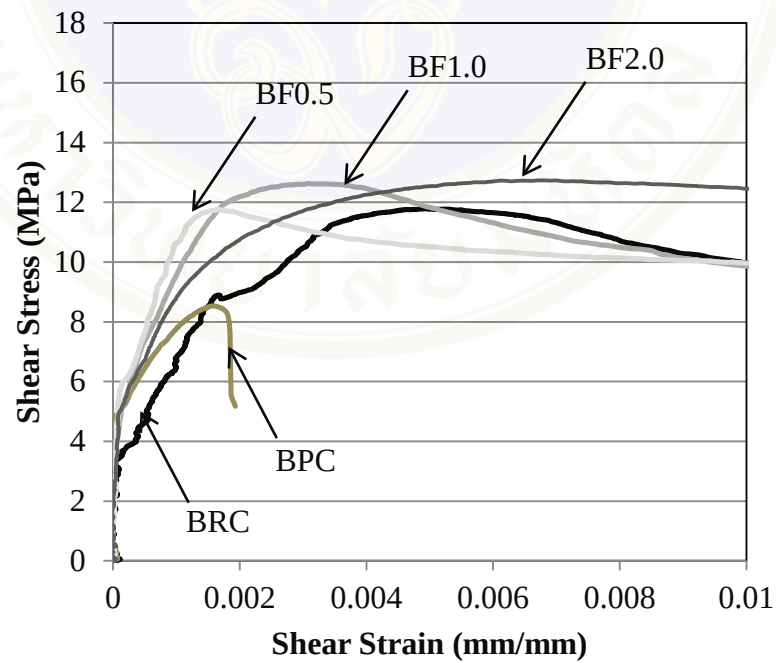


(b) Subgroup N

Figure 4.24 Shear Stress vs. Shear Strain for 800mm Span Specimens

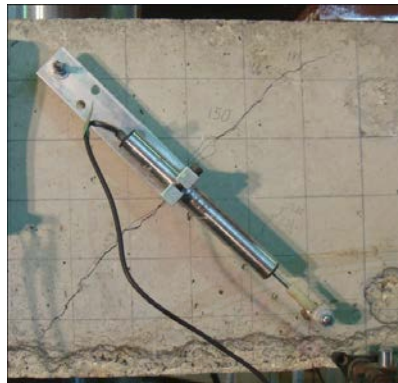


(a) Subgroup H



(b) Subgroup L

Figure 4.25 Shear Stress vs. Shear Strain for 600mm Span Specimens



(a) Shear Strain = 0.002



(b) Shear Strain = 0.004



(c) Shear Strain = 0.006



(d) Shear Strain = 0.008

Figure 4.26 Diagonal Crack Damages at Various Levels of Shear Strain

For the shear stresses between the first occurrence of diagonal crack and the peak shear strength, the plain concrete specimens without transverse reinforcement and FRCC specimens with 0.1% fibers had noticeably larger shear strain than the FRCC specimens with 0.5% fiber or more. This indicates the ability of fibers to reduce the shear damages and confirms the findings in Figure 4.17 and Figure 4.18. Nonetheless, for fibers volume fraction between 0.5-2.0%, no substantial reduction of the shear strain was observed with an increase in fiber content. For specimens with transverse reinforcements (BRC), it was observed that, for a given shear stress before the peak, the shear strains were larger than those of the FRCC specimens with 0.5% fiber content or more. Therefore, the transverse reinforcements were not as effective as fibers in reducing the shear damages.

After the peak shear stress, the shear stress vs. shear strain curves for plain concrete specimens dropped suddenly, which indicates a brittle shear failure. However, the curve for FRCC specimens with 0.1% fiber dropped at a slower rate. For other FRCC specimens and RC specimens, more gradual decreases in shear stress were observed.

Figure 4.27 show the plot between the volume fraction of fiber and shear strain at failure as a ratio of the shear strain at the peak load. It can be observed that the shear strain at failure tends to increase as the fiber content increases, especially for the 600 mm span specimens. Note that an unusually high value of shear strain at failure was observed for BF0.5-600. This was because the failure occurred almost identically on both sides, leading to much more gradual decay in shear stress – shear strain relationship after peak than other specimens.

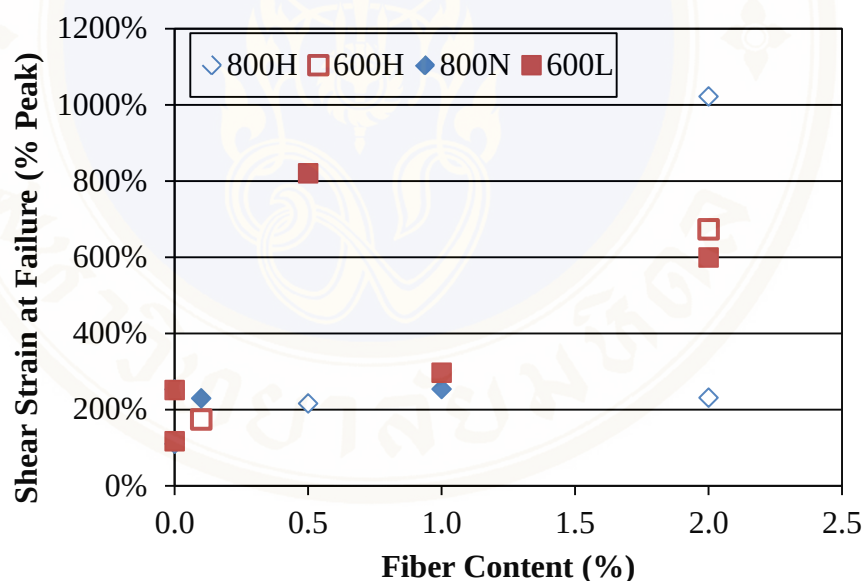
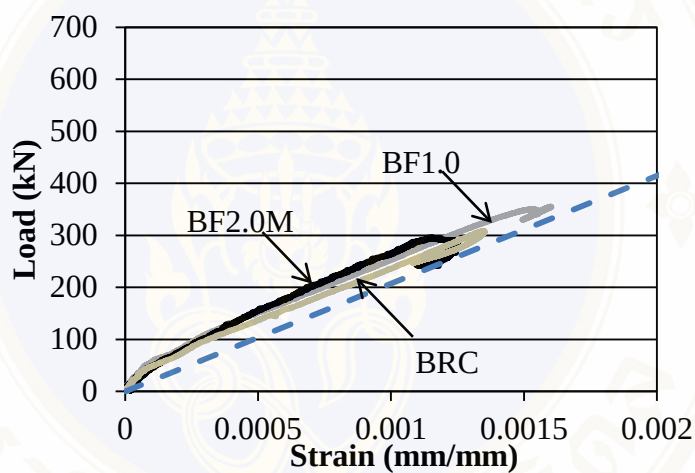


Figure 4.27 Shear Strain at Failure vs. Fiber Content

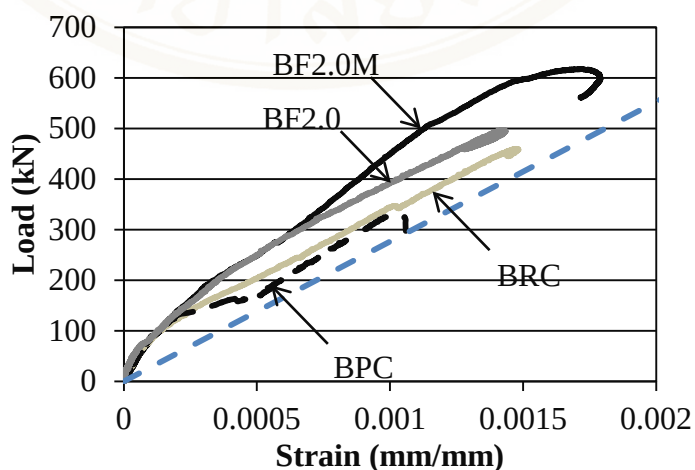
4.2.4 Tie Strain

Figure 4.28 shows the relationship between load and strain in the steel tie reinforcement measured from a strain gauge at the midspan location. It can be observed that no yielding of reinforcement had occurred, indicating that the failures of the beams were primarily caused by strut failures, as it was intended during the design stage. Dashed lines in the Figure 4.28 indicated the predicted steel strain based on the

tie force at a given load level. It can be seen that the measured strains in the tie were less than the predicted strain because the predicted strain did not account for the initial loading when concrete was uncracked. In addition, it was noticed that the strain in the steel tie reinforcement decreased slightly as the fiber content increased for the same load level. This can be considered as an additional benefit of using FRCC in a deep beam. There are two possibilities for this occurrence. The first possibility is that the angle between strut and tie was increased as the fiber content increases, due to reduced area of the top node. The second is that some of the tension force may be resisted by the FRCC matrix.



(a) 800 mm Span



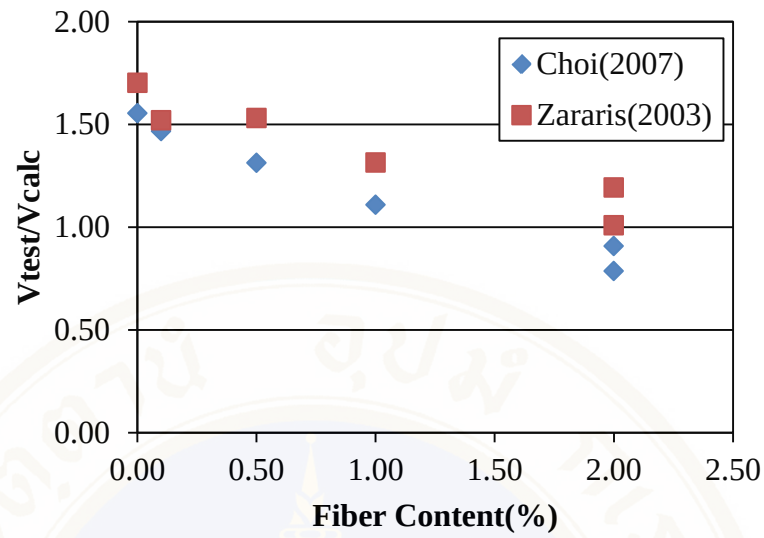
(b) 600 mm Span

Figure 4.28 Load vs. Tie Strain

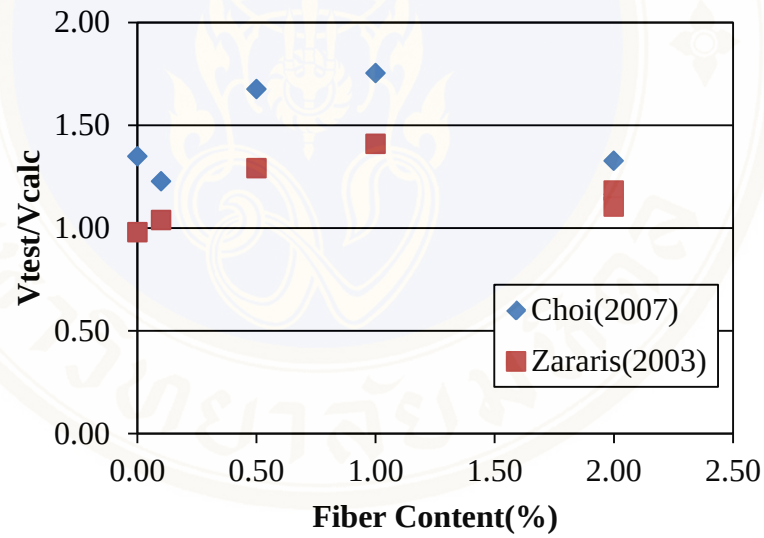
The angle of strut vs. fiber content was shown in Figure 4.14, in which large scatter can be observed due to the fact that the cracks were not perfectly straight. Nonetheless, there was no clear trend for the increase of crack angle with the fiber content. Thus, it may be possible that the matrix can contribute some tensile resistances in the tie. It was observed that there were some tensile cracks perpendicular to the tie axis for all specimens. The FRCC, with its ability to bridge the crack, allow some tensile stress transfer between cracked sections in the vicinity of the tie; thus, relieving some tensile force from the tie.

4.2.5 Compare the Results to the Other Methods

Figure 4.29 shows the plot between the volume fraction of fiber and the shear strength predicted by the other method is compared with the results of test. It can be observed that similar trend between Choi (2007b) method and Zararis (2003) method. In specimen with 800 mm span, the shear strength predicted by the other method is compared with the results of test decreases as fiber content increases but different trend for 600 mm span specimen having steel fiber contents between 0.1 and 1.0% by volume.



(a) 800 mm span



(b) 600 mm span

Figure 4.29 V_{test}/V_{calc} vs. Fiber Content

CHAPTER V

CONCLUSIONS

5.1 Summary

This study investigated the use of fiber reinforced cementitious composite (FRCC) in reinforcing the bottle-shaped strut in reinforced concrete deep beam member, where strut-and-tie method is required for the design. The parameters studied were strut angle of approximately 28° and 36° , hooked steel fiber volume fraction between 0.1 and 2.0%, and the type of matrix. Fourteen deep beam specimens were tested monotonically to failure. Compressive strength, direct tensile strength, and flexural strength of FRCC mixtures were tested for each mixture.

5.2 Conclusions

The following conclusions can be drawn from this study:

5.2.1 Material Test

- The compressive strength was not significantly affected by the use of hooked steel fiber in the fiber volume fraction between 0.1 and 2.0%.
- Direct tensile strength and flexural strength increased as the fiber content increases between 0.1 and 2.0% by volume.
- Strain-hardening and deflection-hardening were observed in FRCC with fiber volume fraction of 2.0% for both concrete and mortar matrices.
- The energy absorptions under direct tensile and flexure tests increased linearly as the fiber volume fraction increases between 0.1 and 2.0%.

5.2.2 Deep Beam Test

- RC specimen without transverse reinforcement and FRCC specimen having the volume fraction of 0.1% failed immediately by strut failure after reaching the maximum load.
- FRCC specimens having hooked steel fiber contents between 0.5 and 2.0% without transverse reinforcement exhibited load-deflection behaviors comparable or better than RC member with transverse reinforcement.
- The maximum load of the deep beam specimen was not significantly affected by the use of hooked steel fiber in the fiber volume fraction between 0.5 and 2.0%.
- The actual angles of diagonal crack were steeper than the strut angles assumed in strut-and-tie models.
- Two parallel diagonal cracks observed in RC specimen were reduced to only one diagonal crack in FRCC specimens.
- The fiber content had relatively small effect on the load deflection behaviors for specimens with strut angle of about 42° (800-mm specimens).
- Load deflection curve exhibited more ductile failure as the fiber content increases between 0.1-2.0% by volume for specimens with strut angle of about 55° (600-mm specimens).
- A drop in load-carrying capacity occurred when the shear strain across strut was approximately 0.002-0.004.
- The load at first diagonal cracking increased as the fiber content increased.
- Lower shear stress-shear strain stiffnesses before the peak were observed for RC specimens with and without transverse reinforcement and FRCC specimen having the volume fraction of 0.1%, compared with RC specimens with transverse reinforcement or FRCC specimens with fiber volume fraction of 0.5% or more.

- Shear-related damages decreased as the steel fiber contents increased between 0.5 and 2.0% by volume. The use of transverse reinforcement did not reduce the shear damage.
- The use of FRCC decreased the tensile force in tie element by allowing tension transfer across cracked concrete surrounding the tie.

5.3 Design Recommendations

Based on the test data, the use of steel fiber at the fiber content between 0.5 and 2.0% provide a significant increase in load at the first diagonal crack and reduce the shear strain. It is recommended that hooked steel fiber be used at a volume content of at least 0.5% to satisfy the requirement of minimum transverse reinforcement crossing struts.

5.4 Recommendations for Future Research

The results in this study have shown that a comparable behavior to a properly-reinforced member can be obtained through the use of steel fiber at 0.5% volume fraction. Additional study should be conducted for the steel volume fraction between 0.1-0.5% to see whether the fiber amount can be further reduced. In addition, the strength of concrete mixture was found to be one of the important factor affecting the post-peak behavior of the deep beam. The effect of the concrete strength should be further investigated. Moreover, studies should be conducted on other types of fibers (steel at other aspect ratios or shapes, polymeric fibers, etc.).

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APPENDIX A

DESIGN OF DEEP BEAM SPECIMEN USING STRUT-AND-TIE METHOD

METHOD

A.1 800 mm Span Specimen

The strut-and-tie model for this specimen is shown in Figure A.1.

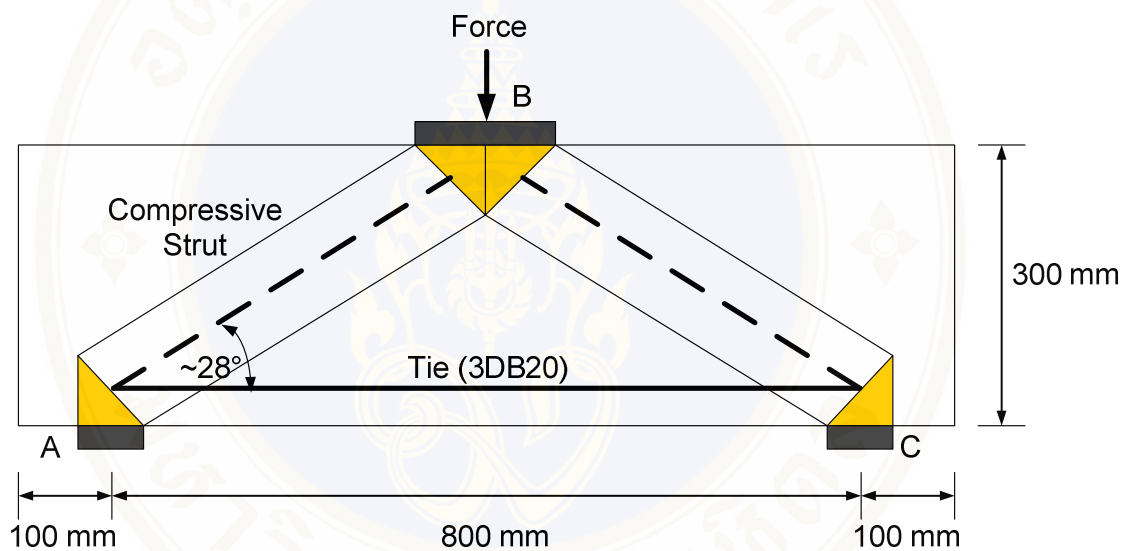


Figure A.1 Strut-and-Tie Model of 800 mm Span Specimen

Assume the height and width of nodes as shown in Figure A.2.

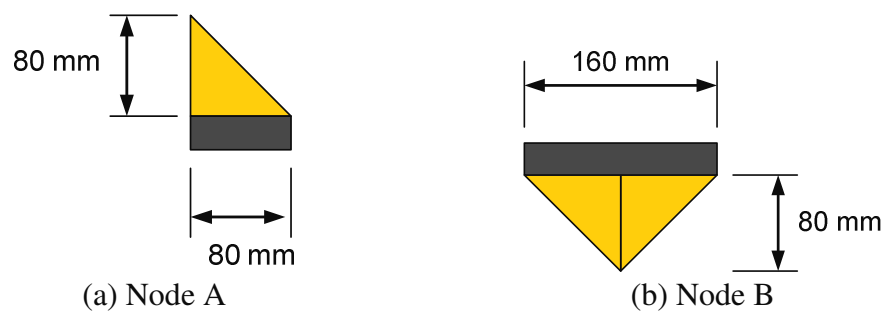


Figure A.2 Assumed Nodal Size of 800 mm Span Specimen

Assume the following section and material properties

$$\begin{aligned} b &= 150 \text{ mm} \\ f_y &= 520 \text{ MPa (obtained from material tests)} \\ f'_c &= 30 \text{ MPa} \end{aligned}$$

Calculate the strength of strut AB at A by assuming the strut efficiency factor (β_s) of 1.0 and assume that the bearing plate is 80 mm in width.

$$\begin{aligned} \beta_s &= 1.0 \\ d_s &= h \cos \alpha + l \sin \alpha \\ &= 80 \cos(28.81) + 80 \sin(28.81) \\ &= 108.65 \text{ mm} \\ f_{ce} &= 0.85 \beta_s f'_c \\ &= 0.85 \times 1.0 \times 30 \\ &= 25.50 \text{ MPa} \\ f_{ce} d_s b &= 25.50 \times 108.65 \times 150 \\ &= 415589 \text{ N} \end{aligned}$$

At the strut failure, the strut-and-tie model can be shown in Figure A.3.

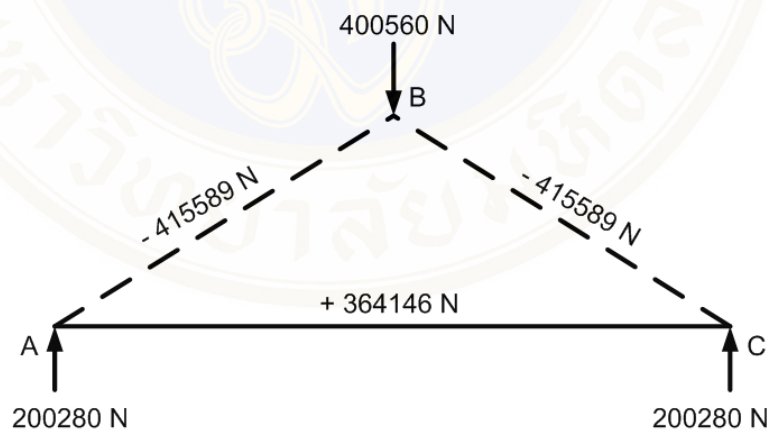


Figure A.3 Force in Strut-and-Tie Model at Strut Failure of 800 mm Span Specimen

Provide reinforcement to resist the tie force without yielding. Try using 3DB20,

$$\begin{aligned} T &= A_{st} f_y \\ &= ((3 \times \pi \times 20^2) / 4) \times 520 \\ &= 490088 \text{ N} \end{aligned}$$

$$T > F_{AC} = 364146 \text{ N} \quad \text{OK.}$$

Check the bearing at support to ensure no nodal failure at the strut failure.

Node A and node C are CCT nodes.

$$\begin{aligned} \beta_n &= 0.8 \\ f_{ce} &= 0.85 \beta_n f'_c \\ &= 0.85 \times 0.8 \times 30 \\ &= 20.40 \text{ MPa} \\ f_{ce} lb &= 20.40 \times 80 \times 150 \\ &= 244800 \text{ N} \end{aligned}$$

At strut failure

$$\begin{aligned} R_s &= 200280 \text{ N} \\ f_{ce} lb &> R_s \quad \text{OK} \end{aligned}$$

For specimen with transverse reinforcement, the reinforcement must satisfy the minimum reinforcement crossing strut requirements.

Using RB6

$$\begin{aligned} A_{si} &= (\pi \times 6^2) / 4 \\ &= 28.27 \text{ mm}^2 \\ 0.003 &\leq \sum \frac{A_{si}}{b_s s_i} \sin \alpha_i \\ 0.003 &\leq \frac{(2 \times 28.27 \sin(28.81)) / (150 \times s) + (2 \times 28.27 \sin(61.19)) / (150 \times s)}{s} \\ s &\leq 170.67 \text{ mm} \end{aligned}$$

Use RB6 @ 150 mm as transverse reinforcement in the horizontal and vertical directions.

This corresponds to the true transverse reinforcement ratio of:

$$\begin{aligned} \sum \frac{A_{si}}{b_s s_i} \sin \alpha_i &= \frac{(2 \times 28.27 \sin(28.81)) / (150 \times 150) + (2 \times 28.27 \sin(61.19)) / (150 \times 150)}{150} \\ &= 0.0034 \end{aligned}$$

A.2 600 mm Span Specimen

The strut-and-tie model for this specimen is shown in Figure A.4.

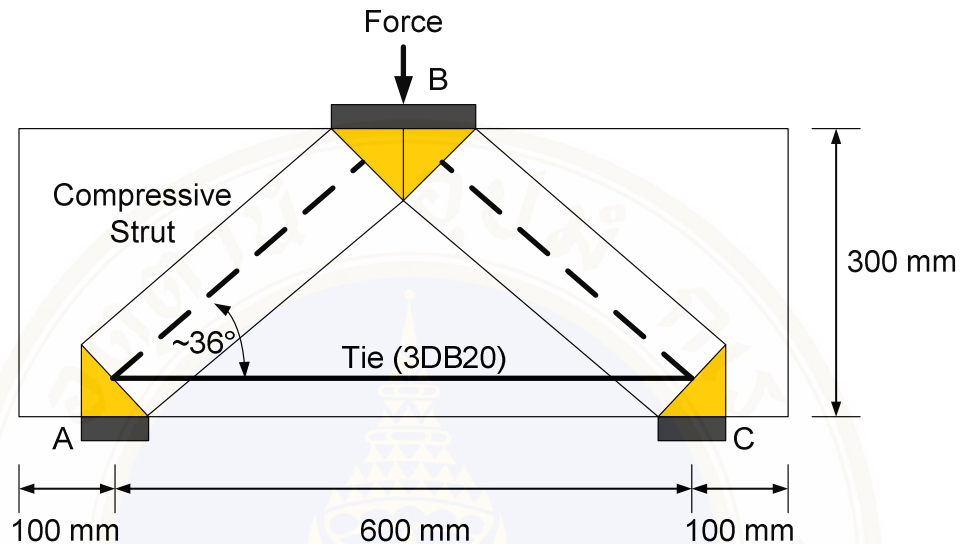


Figure A.4 Strut-and-Tie Model of 600 mm Span Specimen

Assume the height and width of nodes as shown in Figure A.5.

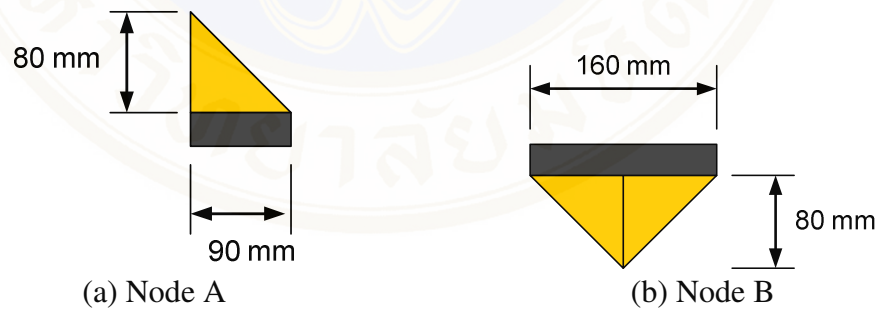


Figure A.5 Assumed Nodal Size of 800 mm Span Specimen

Assume the following section and material properties

b	=	150	mm
f_y	=	520	MPa (obtained from material tests)
f'_c	=	30	MPa

Calculate the strength of strut AB at A by assuming the strut efficiency factor (β_s) of 1.0 and assume that the bearing plate is 80 mm in width

$$\begin{aligned}
 \beta_s &= 1.0 \\
 d_s &= h \cos \alpha + l \sin \alpha \\
 &= 80 \cos(36.25) + 80 \sin(36.25) \\
 &= 117.74 \text{ mm} \\
 f_{ce} &= 0.85 \beta_s f'_c \\
 &= 0.85 \times 1.0 \times 30 \\
 &= 25.50 \text{ MPa} \\
 f_{ce} d_s b &= 25.50 \times 117.74 \times 150 \\
 &= 450337 \text{ N}
 \end{aligned}$$

At the strut failure, the strut-and-tie model can be shown in Figure A.6.

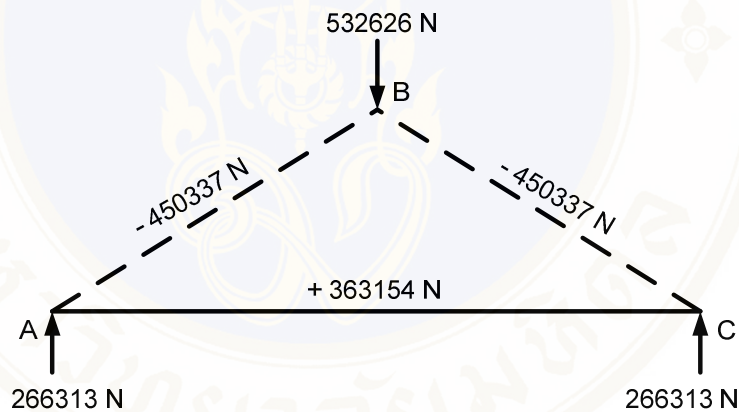


Figure A.6 Force in Strut-and-Tie Model at Strut Failure of 600 mm Span Specimen

Provide reinforcement to resist the tie force without yielding. Try using 3DB20,

$$\begin{aligned}
 T &= A_s f_y \\
 &= ((3 \times \pi \times 20^2) / 4) \times 520 \\
 &= 490088 \text{ N} \\
 T &> F_{AC} = 363154 \text{ N} \quad \text{OK.}
 \end{aligned}$$

Check the bearing at support to ensure no nodal failure at the strut failure. Node A and node C are CCT nodes.

$$\beta_n = 0.8$$

$$\begin{aligned}
 f_{ce} &= 0.85\beta_n f'_c \\
 &= 0.85 \times 0.8 \times 30 \\
 &= 20.40 \quad \text{MPa} \\
 f_{ce} lb &= 20.40 \times 90 \times 150 \\
 &= 275400 \quad \text{N}
 \end{aligned}$$

At strut failure

$$\begin{aligned}
 R_s &= 266313 \quad \text{N} \\
 f_{ce} lb &> R_s \quad \text{OK}
 \end{aligned}$$

For specimen with transverse reinforcement, the reinforcement must satisfy the minimum reinforcement crossing strut requirements.

Using RB6

$$\begin{aligned}
 A_{si} &= (\pi \times 6^2) / 4 \\
 &= 28.27 \quad \text{mm}^2 \\
 0.003 &\leq \sum \frac{A_{si}}{b_s s_i} \sin \alpha_i \\
 0.003 &\leq \frac{(2 \times 28.27 \sin(36.25)) / (150 \times s) + (2 \times 28.27 \sin(53.75)) / (150 \times s)}{s} \\
 &\leq 176 \quad \text{mm}
 \end{aligned}$$

Use RB6 @ 150 mm as transverse reinforcement in the horizontal and vertical directions.

This corresponds to the true transverse reinforcement ratio of:

$$\begin{aligned}
 \sum \frac{A_{si}}{b_s s_i} \sin \alpha_i &= \frac{(2 \times 28.27 \sin(36.25)) / (150 \times 150) + (2 \times 28.27 \sin(53.75)) / (150 \times 150)}{150} \\
 &= 0.0035
 \end{aligned}$$

APPENDIX B

REGRESSION ANALYSIS OF MATERIAL PROPERTIES

B.1 Linear Regression Analysis for Compressive Strength

Linear regression analysis results of the compressive strength are summarized in Table B.1. It was found that the compressive strength (f'_c) for the mixture used in this study relates to the steel hooked fiber fraction (v_f) as $f'_c = 37.25 - 1.99 v_f$. Figure B.1 shows the regression line with the data points.

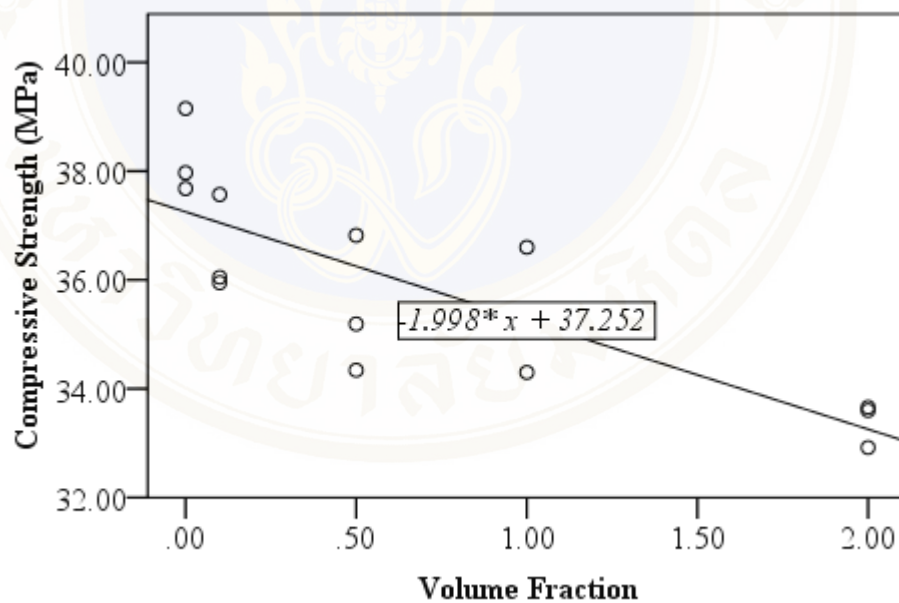


Figure B.1 Compressive Strength vs. Volume Fraction

Table B.1 Linear Regression Analysis Results for Compressive Strength

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	0.825 _a	0.680	0.655	1.07561

a. Predictors: (Constant), Volume Fraction

ANOVA_b

Model	Sum of Squares	df	Mean Square	F	Sig.
Regression	31.941	1	31.941	27.608	0.000 _a
Residual	15.040	13	1.157		
Total	46.981	14			

a. Predictors: (Constant), Volume Fraction

b. Dependent Variable: Compressive Strength

Coefficients_a

Model	Unstandardized Coefficients		Standardized Coefficients		
	B	Std. Error	Beta	t	Sig.
(Constant)	37.252	0.390		95.531	0.000
Volume Fraction	-1.998	0.380	-0.825	-5.254	0.000

a. Dependent Variable: Compressive Strength

B.2 Linear Regression Analysis for Direct Tensile Strength

Linear regression analysis results of the direct tensile strength are summarized in Table B.2. It was found that the direct tensile strength (f_t) for the mixture used in this study relates to the steel hooked fiber fraction (v_f) as $f_t = 1.64 + 0.55 v_f$. Figure B.2 shows the regression line with the data points.

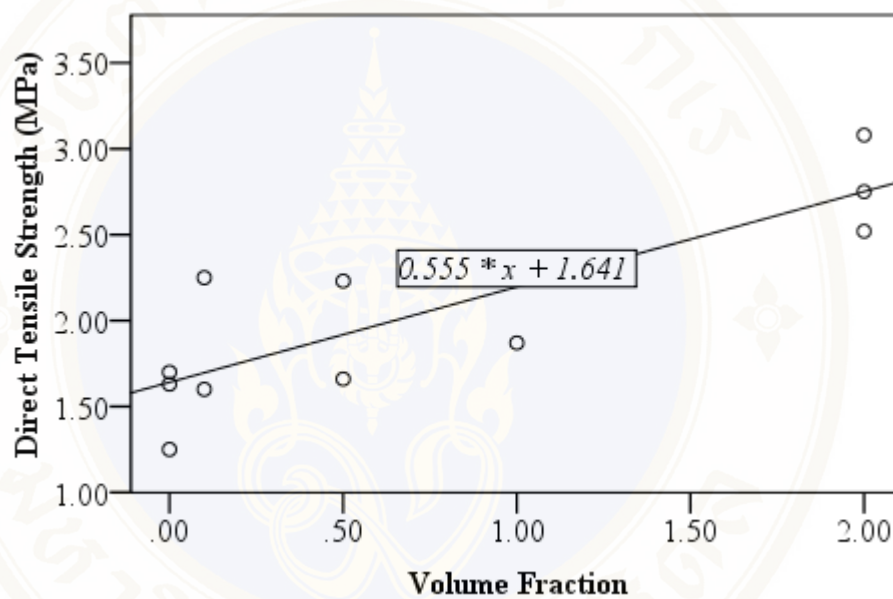


Figure B.2 Direct Tensile Strength vs. Volume Fraction

Table B.2 Linear Regression Analysis Results for Direct Tensile Strength

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	0.827 _a	0.717	0.689	0.30133

a. Predictors: (Constant), Volume Fraction

ANOVA_b

Model	Sum of Squares	df	Mean Square	FF	Sig.
Regression	2.303	1	2.303	25.363	0.001 _a
Residual	0.908	10	0.091		
Total	3.211	11			

a. Predictors: (Constant), Volume Fraction

b. Dependent Variable: Direct Tensile Strength

Coefficients_a

Model	Unstandardized Coefficients		Standardized Coefficients		
	B	Std. Error	Beta	t	Sig.
(Constant)	1.641	0.121		13.527	0.000
Volume Fraction	0.555	0.110	0.847	5.036	0.001

a. Dependent Variable: Direct Tensile Strength

B.3 Linear Regression Analysis for Flexural Strength

Linear regression analysis results of the flexural strength are summarized in Table B.3. It was found that the flexural strength (f_r) for the mixture used in this study relates to the steel hooked fiber fraction (v_f) as $f_r = 4.40 + 1.11 v_f$. Figure B.3 shows the regression line with the data points.

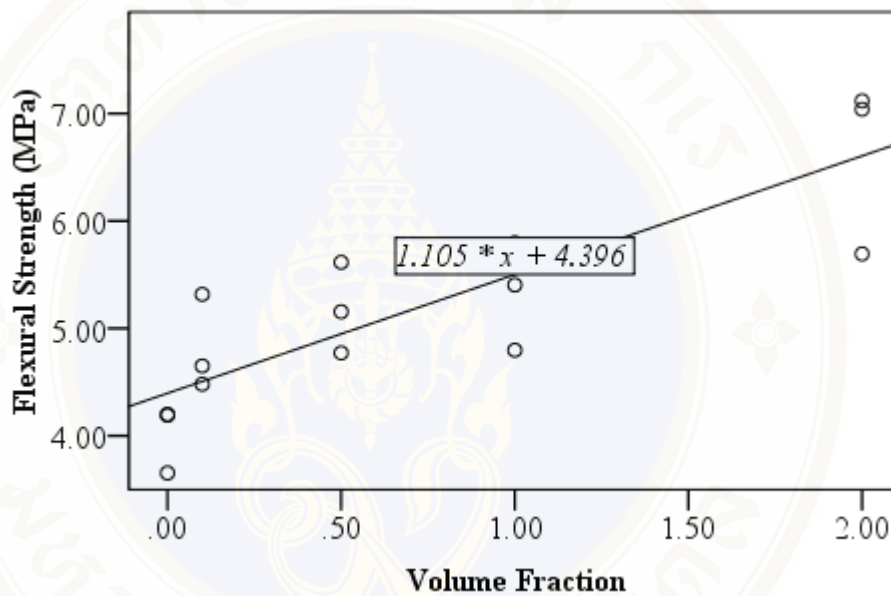


Figure B.3 Flexural Strength vs. Volume Fraction

Table B.3 Linear Regression Analysis Results for Flexural Strength

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	0.851 _a	0.725	0.704	0.53382

a. Predictors: (Constant), Volume Fraction

ANOVA_b

Model	Sum of Squares	df	Mean Square	FF	Sig.
Regression	9.766	1	9.766	34.269	0.000 _a
Residual	3.705	13	0.285		
Total	13.470	14			

a. Predictors: (Constant), Volume Fraction

b. Dependent Variable: Flexural Strength

Coefficients_a

Model	Unstandardized Coefficients		Standardized Coefficients		
	B	Std. Error	Beta	t	Sig.
(Constant)	4.396	0.194		22.717	0.000
Volume Fraction	1.105	0.189	0.851	5.854	0.000

a. Dependent Variable: Flexural Strength

B.4 Tensile Stress vs. Strain from the Direct Tensile Strength

Figure B.4 to Figure B.9 show tensile stress vs. strain plots from the direct tensile tests for each mixture.

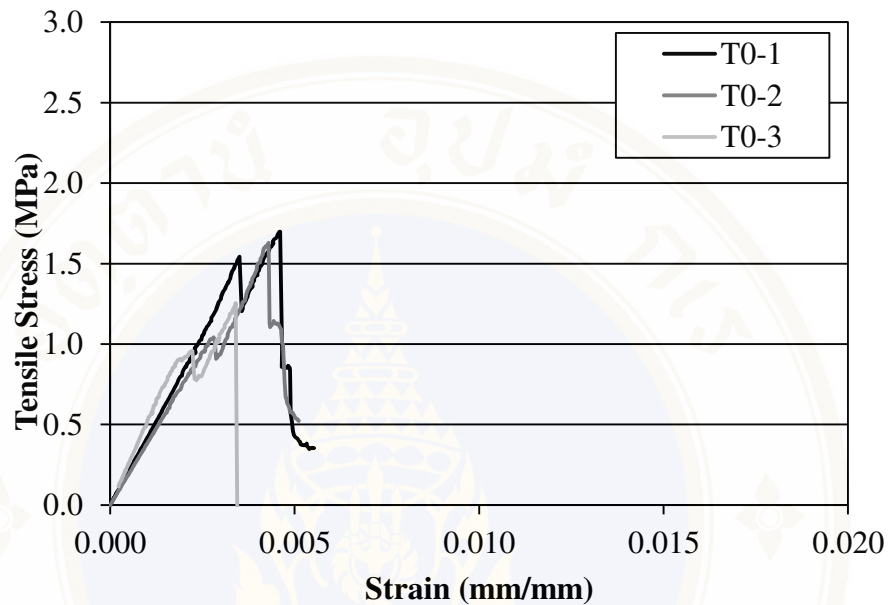


Figure B.4 Tensile Stress vs. Strain for Plain Concrete (0C) Specimen

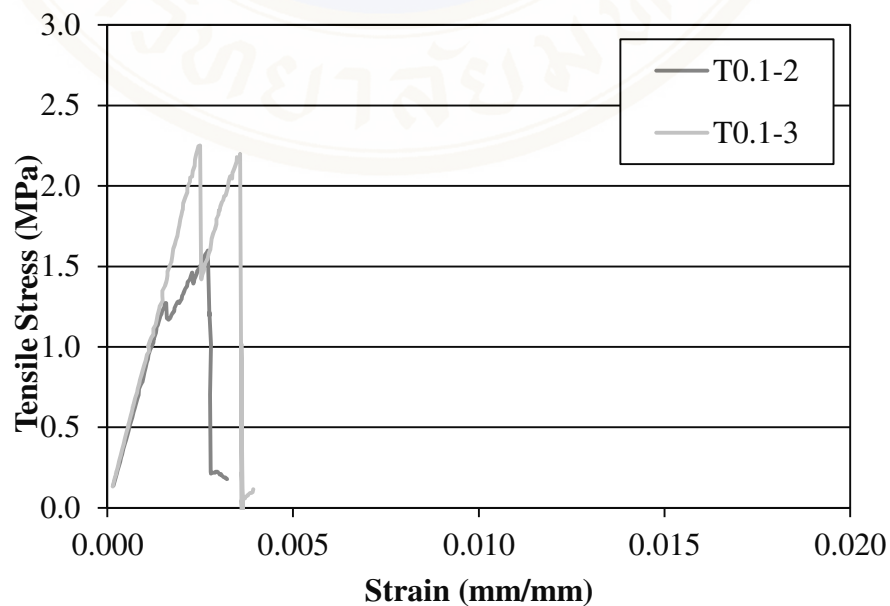


Figure B.5 Tensile Stress vs. Strain for Specimens with 0.1% Fiber Content (0.1C) Specimen

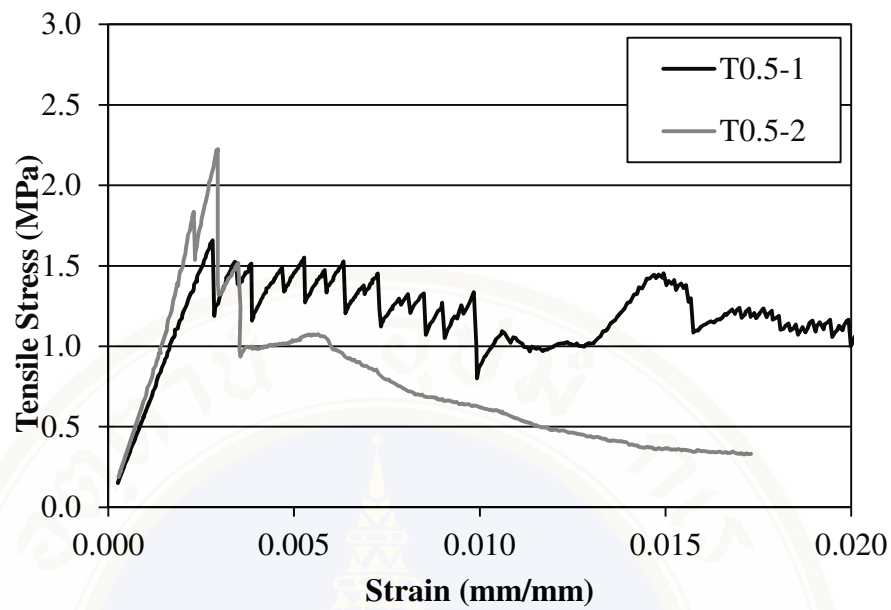


**Figure B.6 Tensile Stress vs. Strain for Specimens with 0.5% Fiber Content
(0.5C) Specimen**

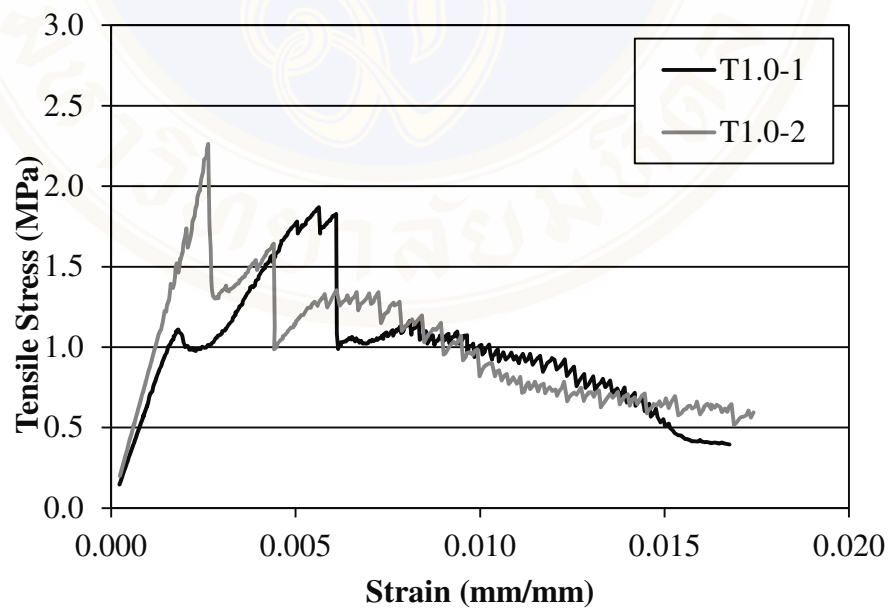


**Figure B.7 Tensile Stress vs. Strain for Specimens with 1.0% Fiber Content
(1.0C) Specimen**

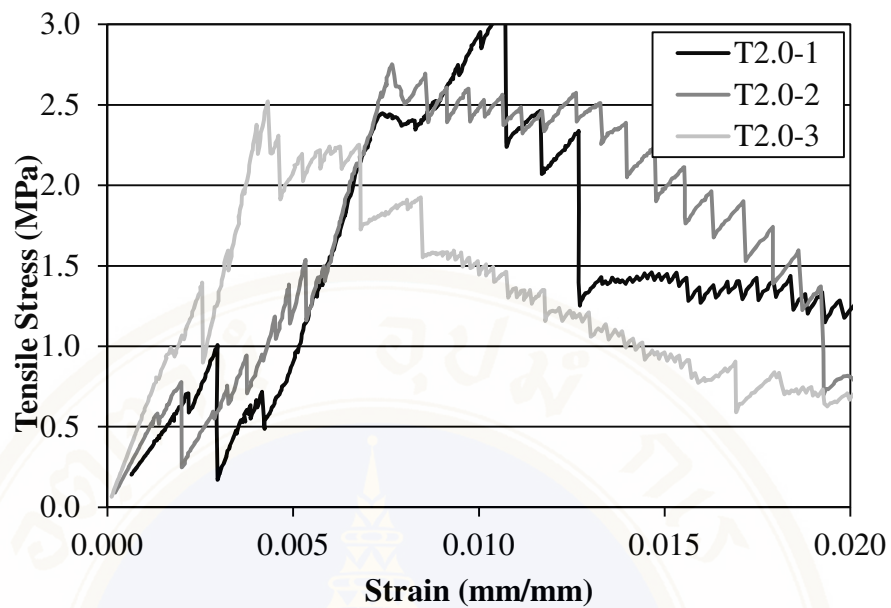


**Figure B.8 Tensile Stress vs. Strain for Specimens with 2.0% Fiber Content
(2.0C) Specimen**

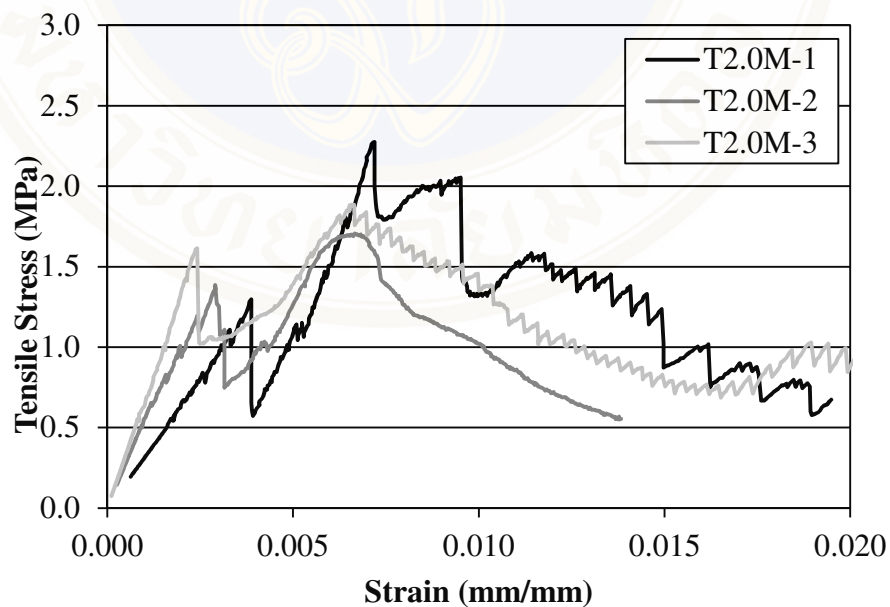


**Figure B.9 Tensile Stress vs. Strain for Specimens with 2.0% Fiber Content
(2.0M) Specimen**

B.5 Flexural Stress vs. Deflection from the Flexural Strength

Figure B.10 to Figure B.15 show flexural stress-deflection relationships from the flexural strength test for each mixture.

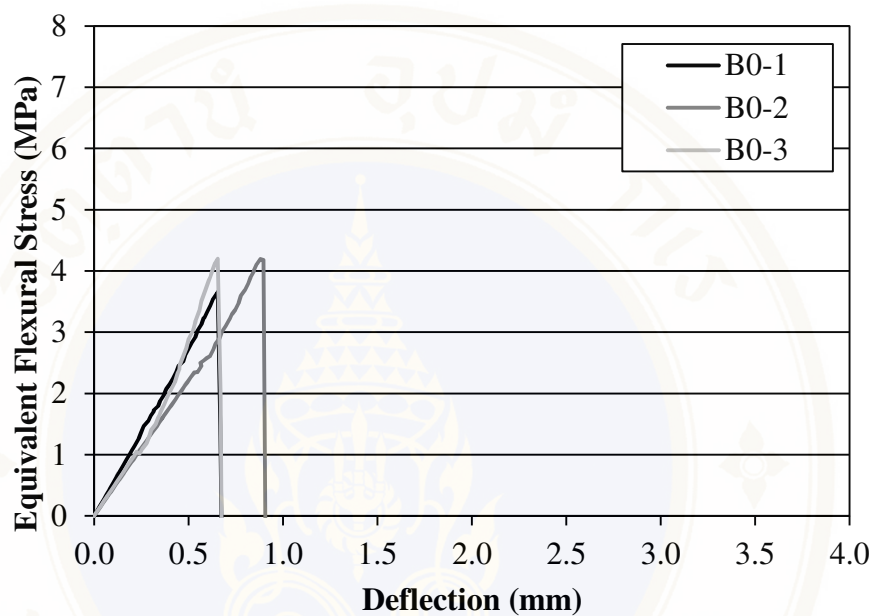


Figure B.10 Flexural Stress vs. Deflection for Plain Concrete (0C) Specimen

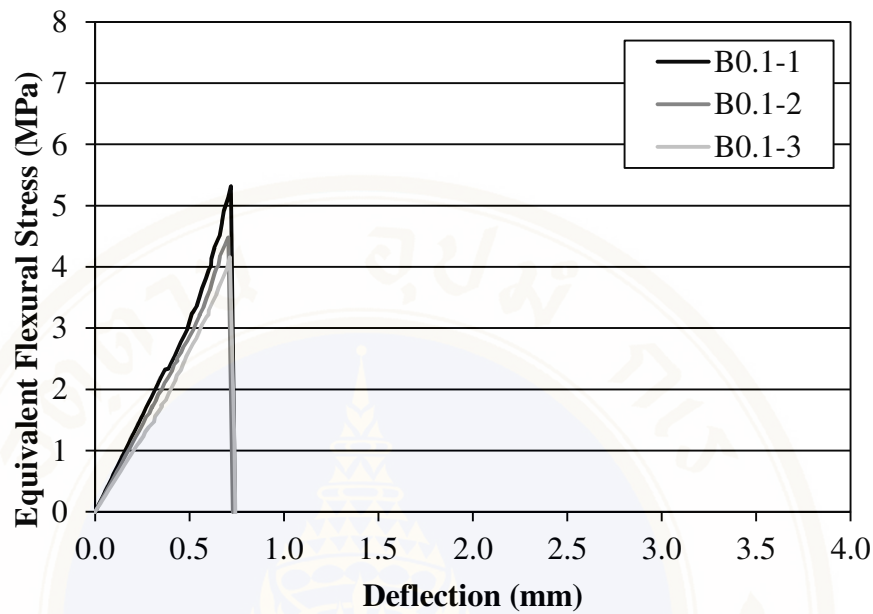


**Figure B.11 Flexural Stress vs. Deflection for Specimens with 0.1% Fiber Content
(0.1C) Specimen**

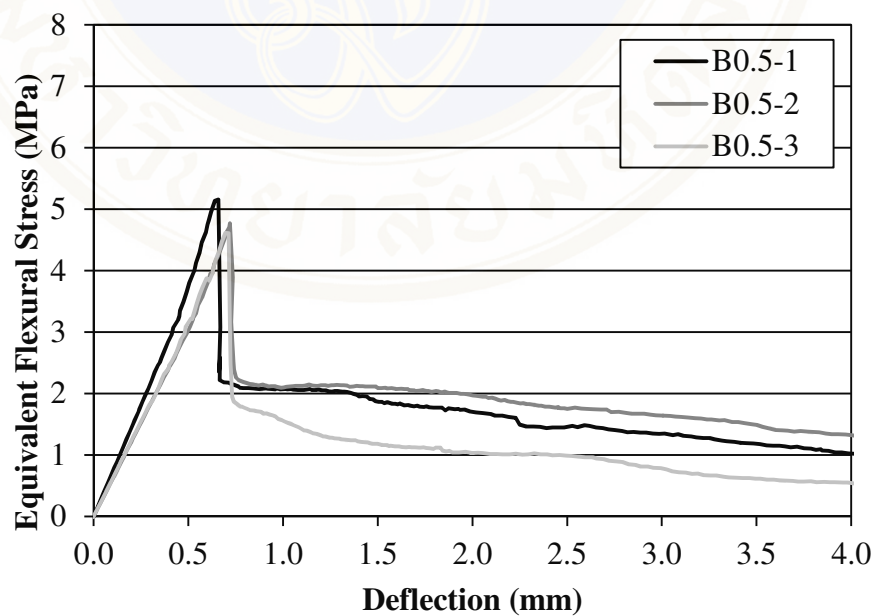


**Figure B.12 Flexural Stress vs. Deflection for Specimens with 0.5% Fiber Content
(0.5C) Specimen**

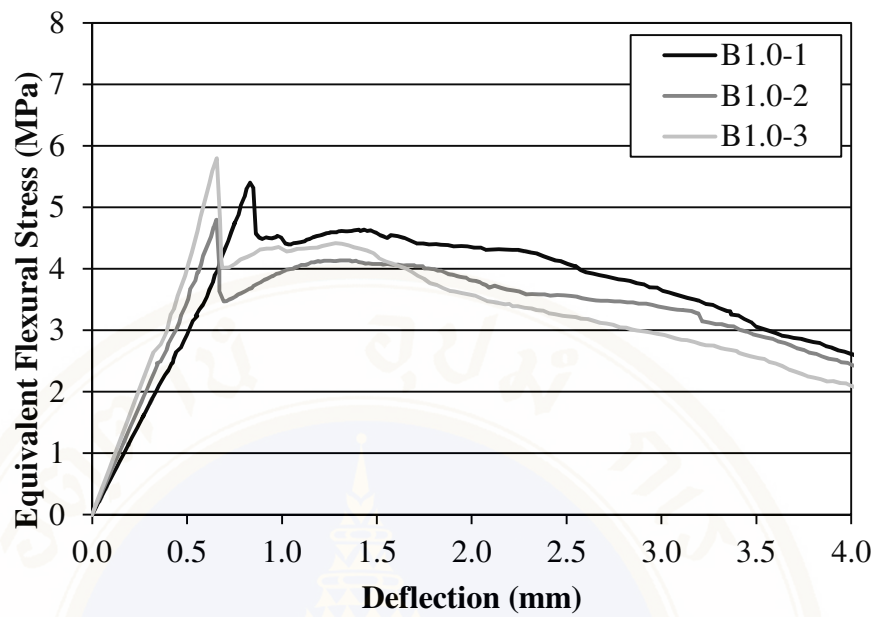


**Figure B.13 Flexural Stress vs. Deflection for Specimens with 1.0% Fiber Content
(1.0C) Specimen**

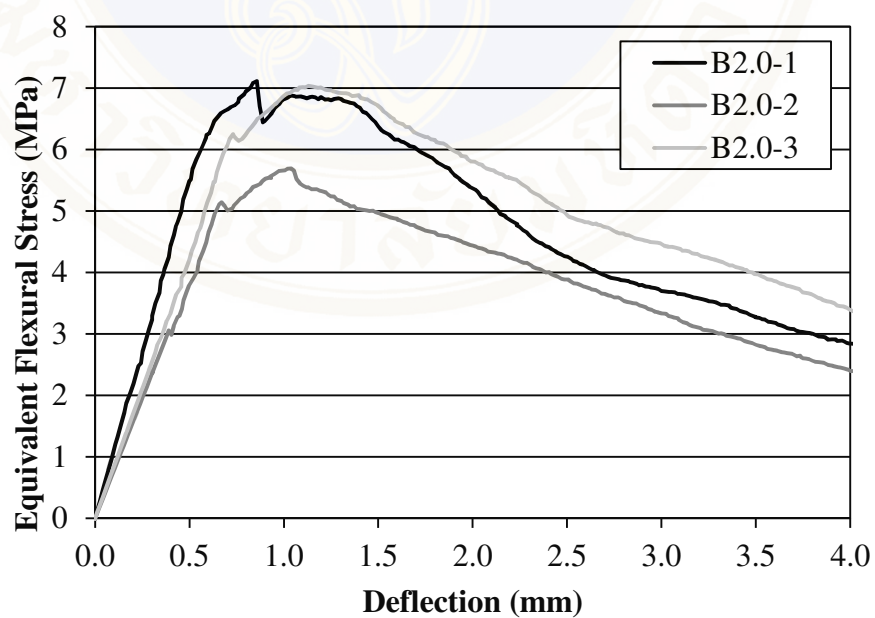


**Figure B.14 Flexural Stress vs. Deflection for Specimens with 2.0% Fiber Content
(2.0C) Specimen**

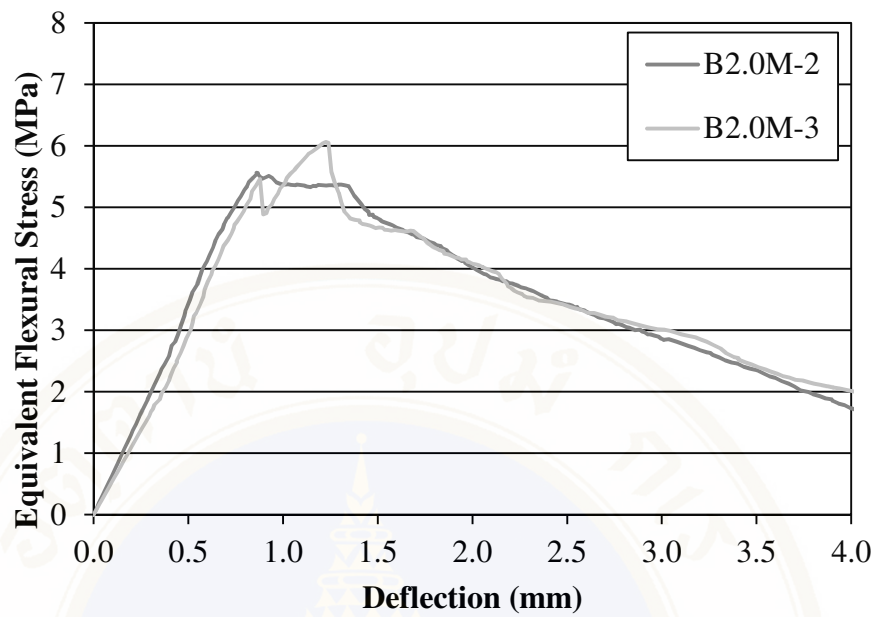


Figure B.15 Flexural Stress vs. Deflection for Specimens with 2.0% Fiber Content (2.0M) Specimen

APPENDIX C

DEEP BEAM SPECIMENS

C.1 Load vs. Deflection

Figure C.1 to Figure C.7 show load-deflection relationships from the tests for 800 mm span specimens.

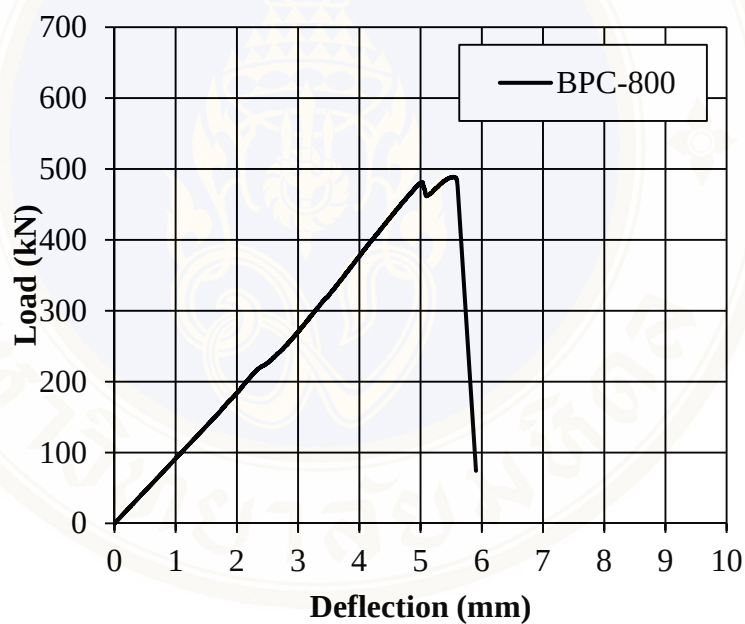


Figure C.1 Load vs. Deflection for BPC-800

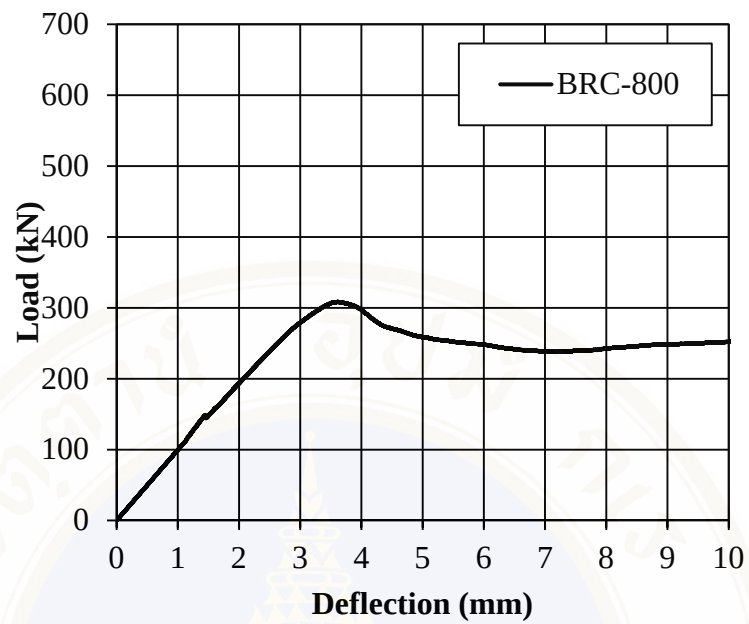


Figure C.2 Load vs. Deflection for BRC-800

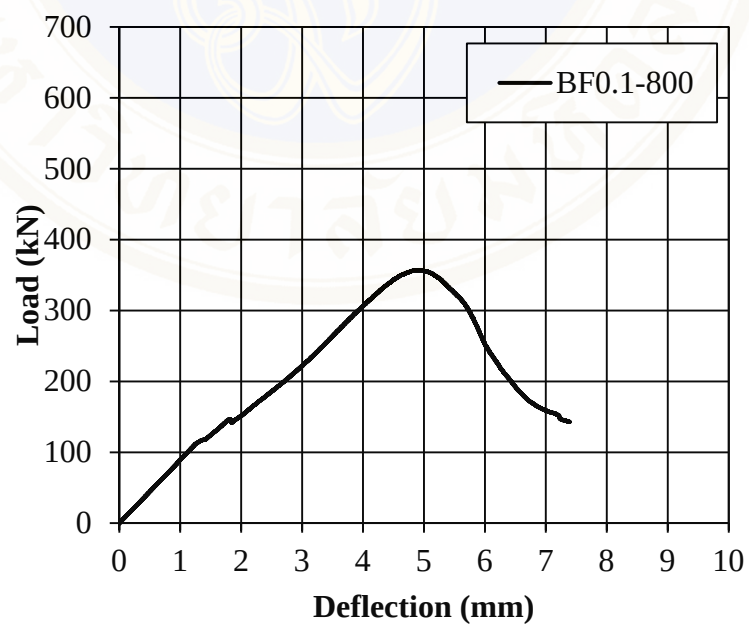


Figure C.3 Load vs. Deflection for BF0.1-800

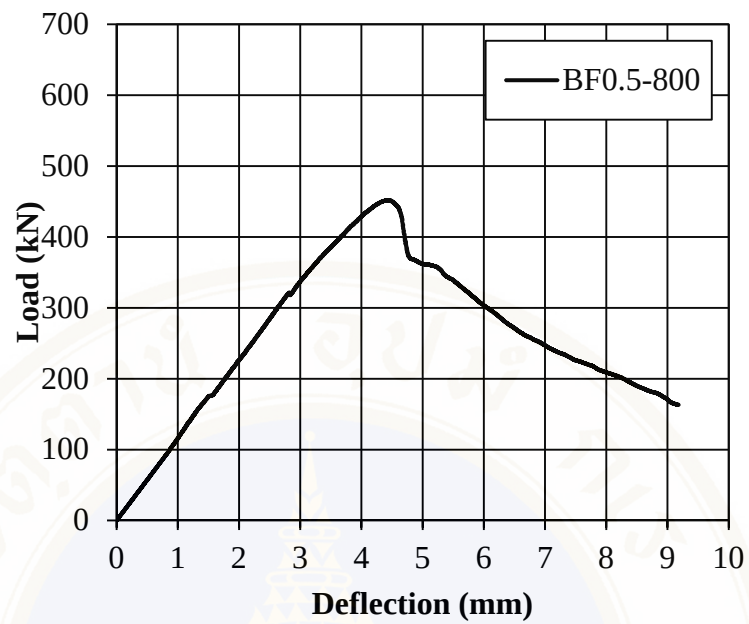


Figure C.4 Load vs. Deflection for BF0.5-800

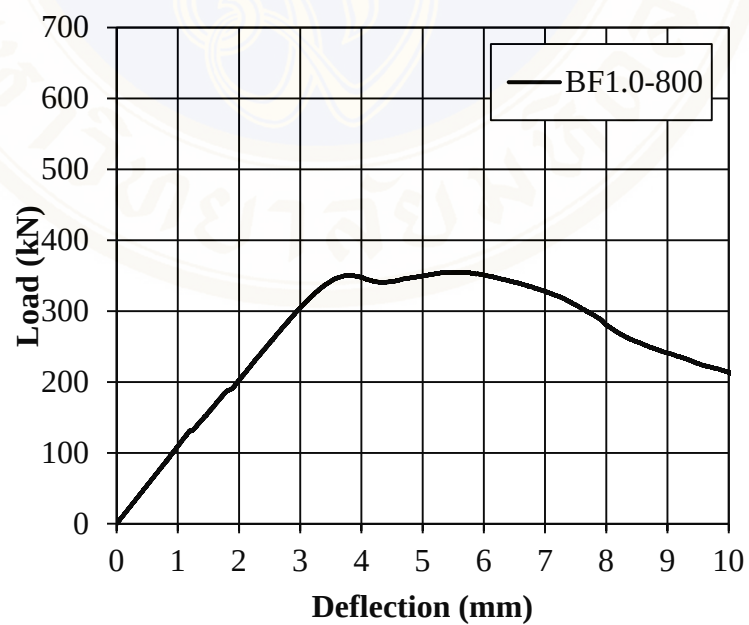


Figure C.5 Load vs. Deflection for BF1.0-800

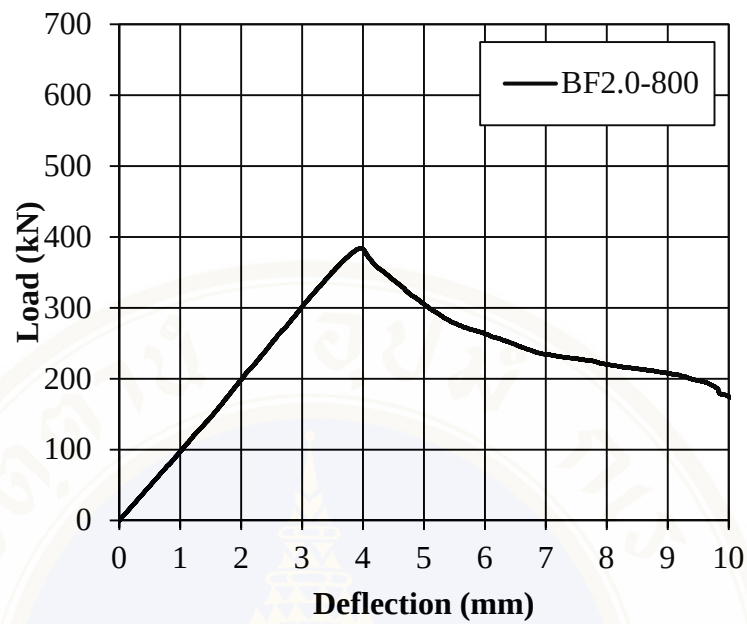


Figure C.6 Load vs. Deflection for BF2.0-800

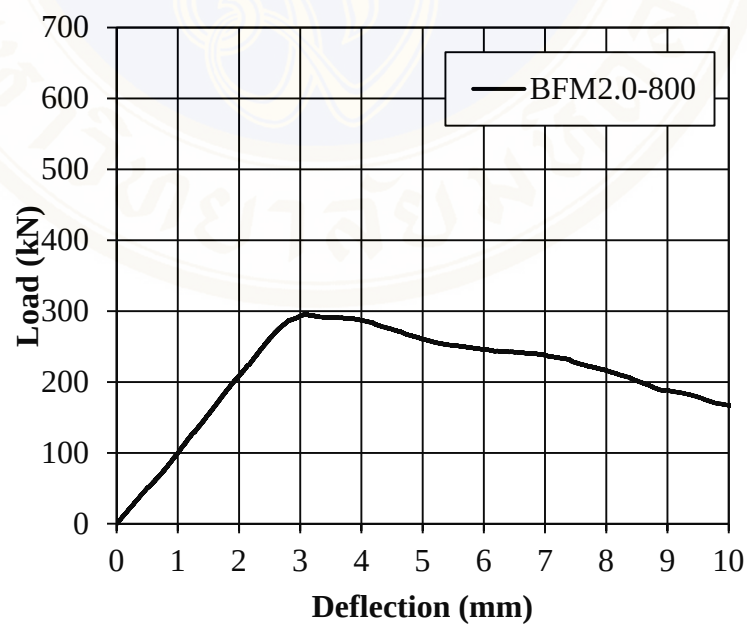


Figure C.7 Load vs. Deflection for BFM2.0-800

Figure C.8 to Figure C.14 show load-deflection relationships from the tests for 600 mm span specimens.

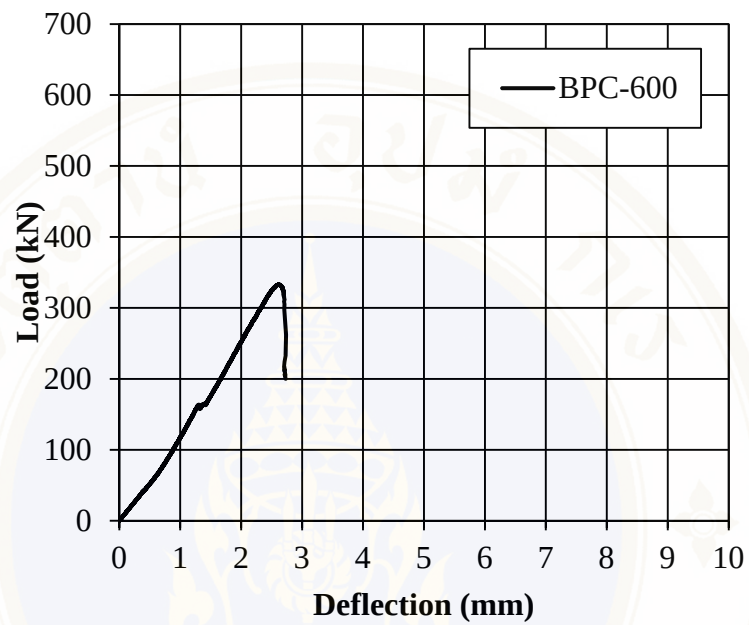


Figure C.8 Load vs. Deflection for BPC-600

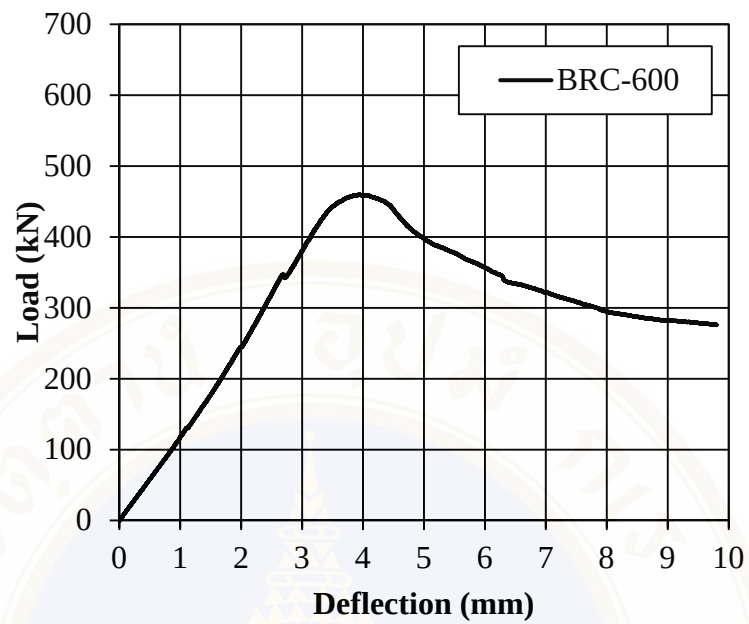


Figure C.9 Load vs. Deflection for BRC-600

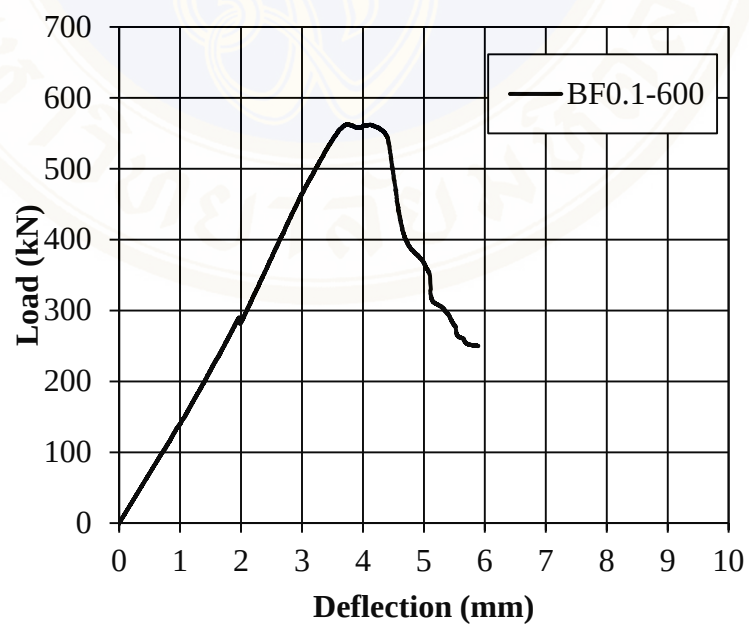


Figure C.10 Load vs. Deflection for BF0.1-600

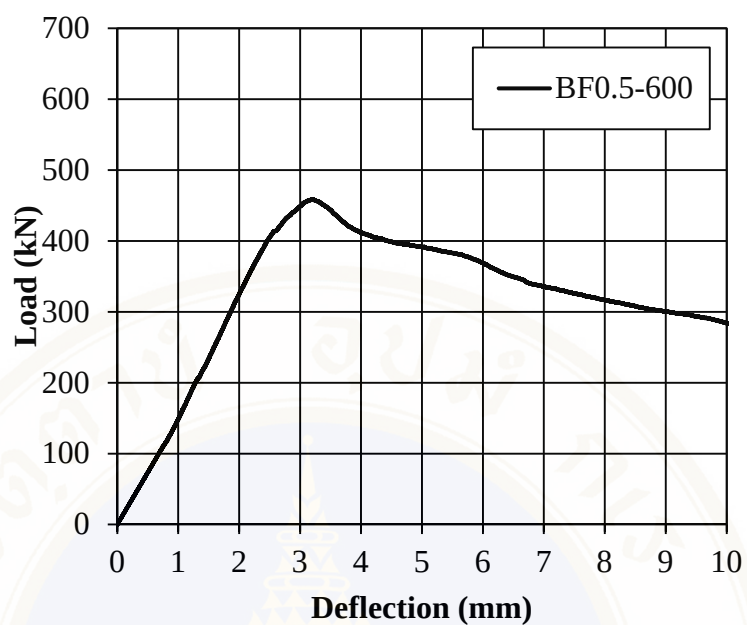


Figure C.11 Load vs. Deflection for BF0.5-600

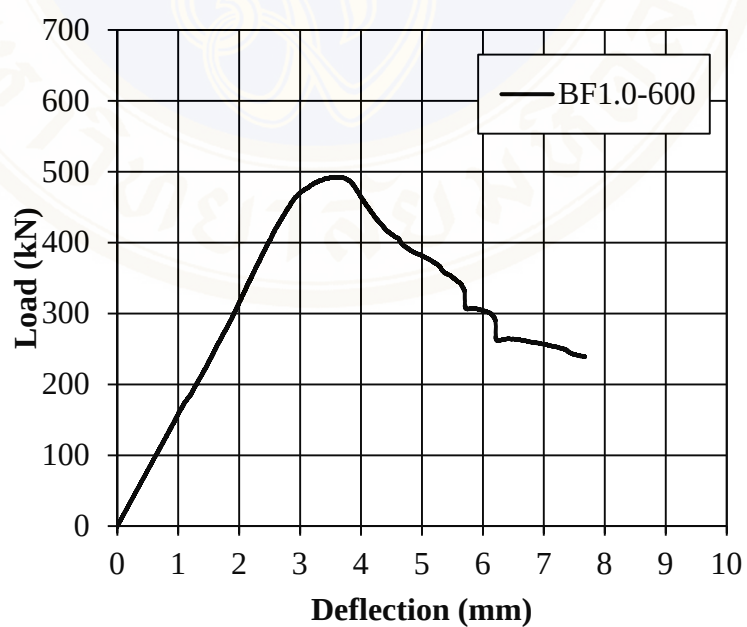


Figure C.12 Load vs. Deflection for BF1.0-600

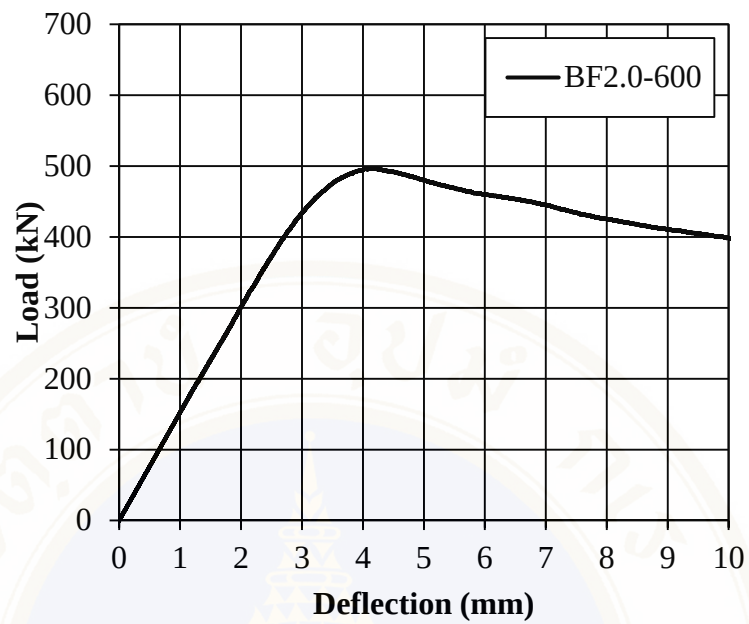


Figure C.13 Load vs. Deflection for BF2.0-600

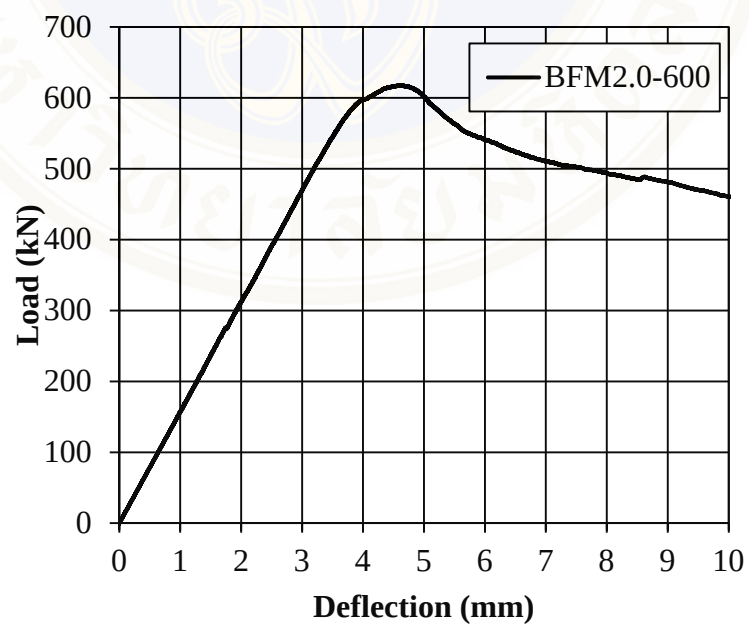


Figure C.14 Load vs. Deflection for BFM2.0-600

C.2 Shear Stress vs. Shear Strain

Figure C.15 to Figure C.21 show the plot of shear stress vs. shear strain on the failure side for 800mm span specimens.

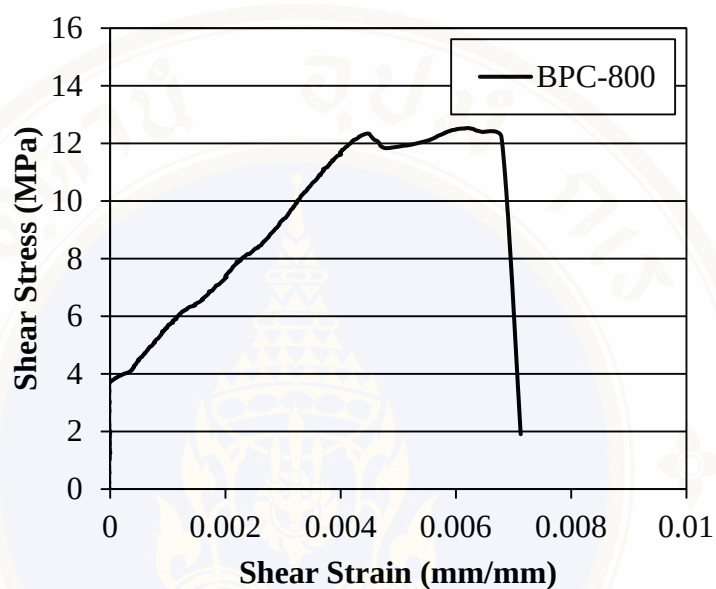


Figure C.15 Shear Stress vs. Shear Strain for BPC-800

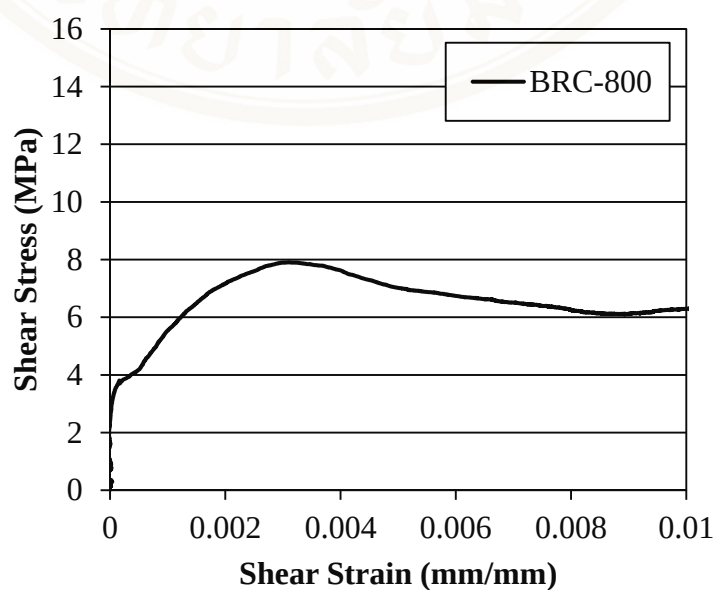


Figure C.16 Shear Stress vs. Shear Strain for BRC-800

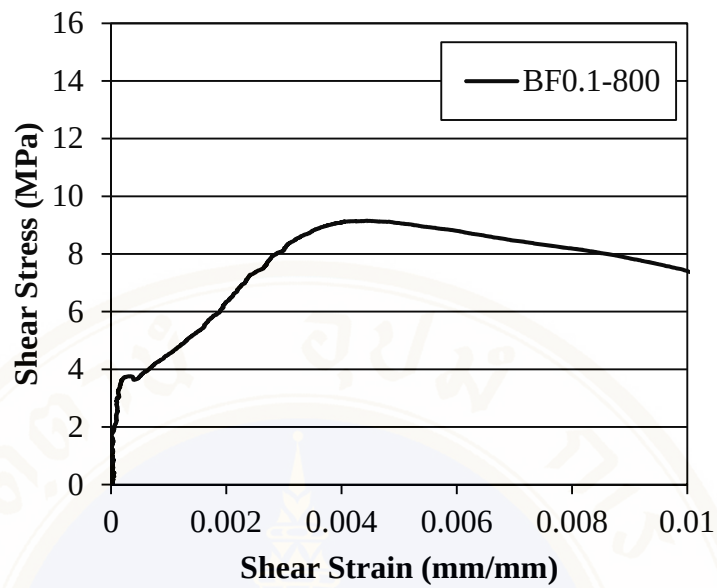


Figure C.17 Shear Stress vs. Shear Strain for BF0.1-800

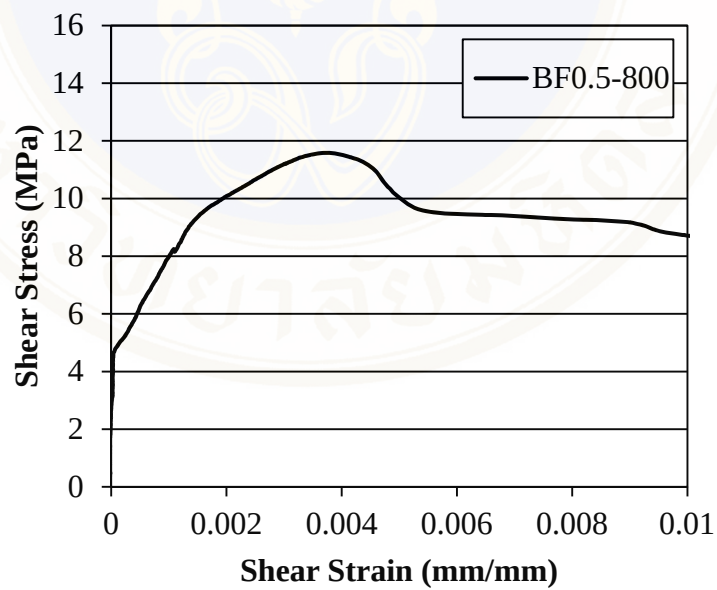


Figure C.18 Shear Stress vs. Shear Strain for BF0.5-800

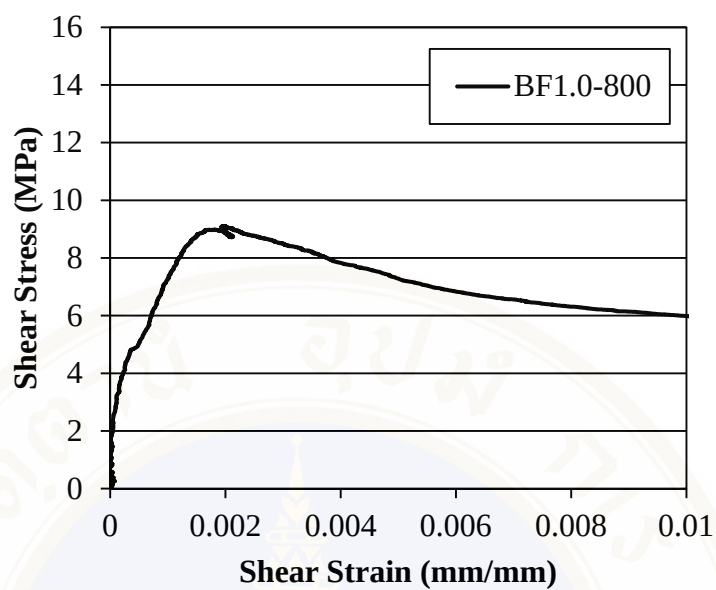


Figure C.19 Shear Stress vs. Shear Strain for BF1.0-800

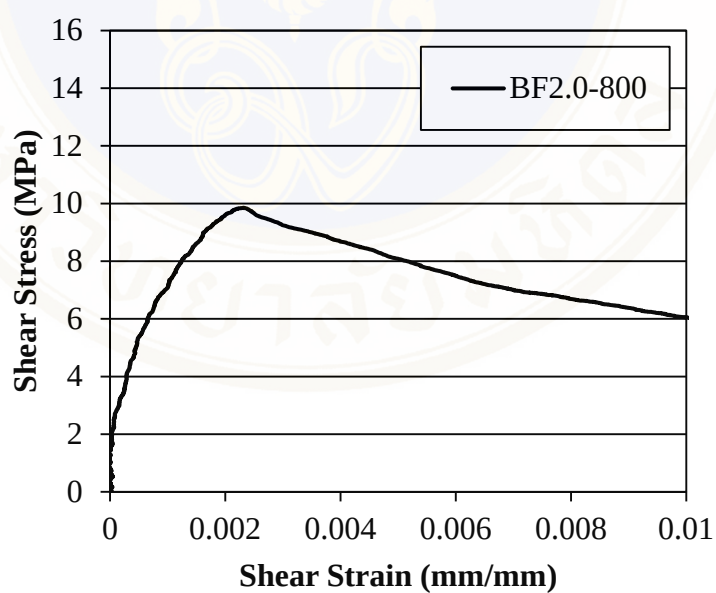


Figure C.20 Shear Stress vs. Shear Strain for BF2.0-800

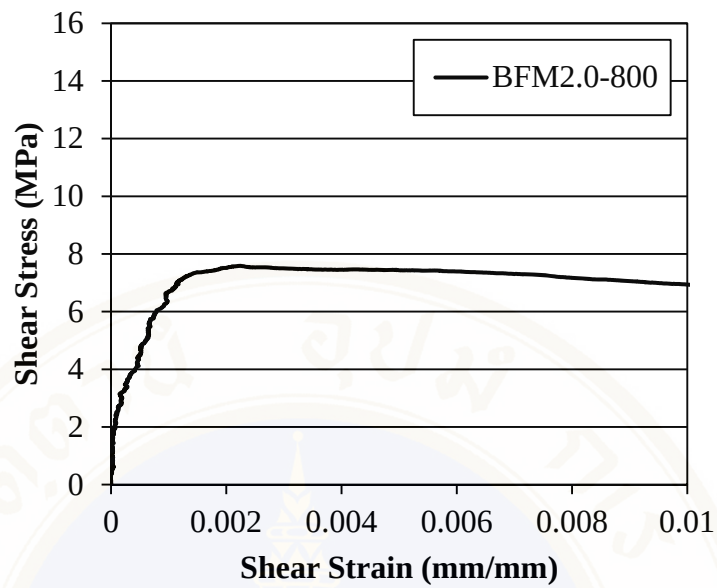


Figure C.21 Shear Stress vs. Shear Strain for BFM2.0-800

Figure C.22 to Figure C.28 show the plot of shear stress vs. shear strain on the failure side for 600mm span specimens.

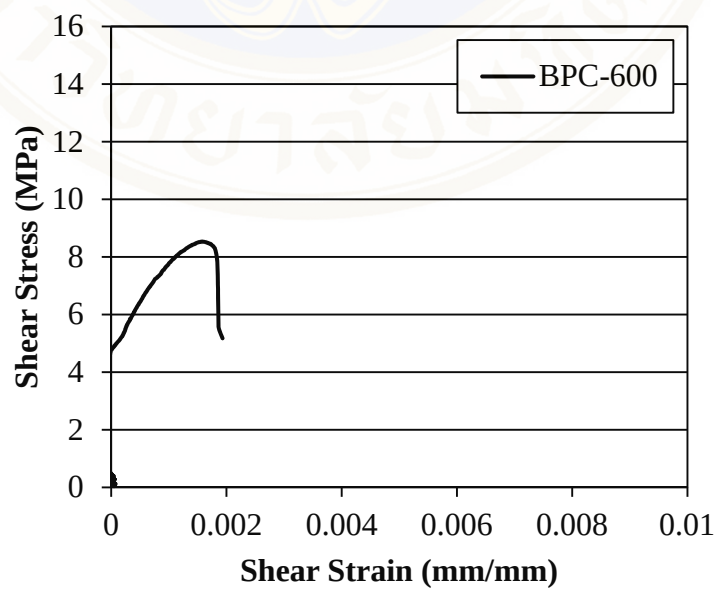


Figure C.22 Shear Stress vs. Shear Strain for BPC-600

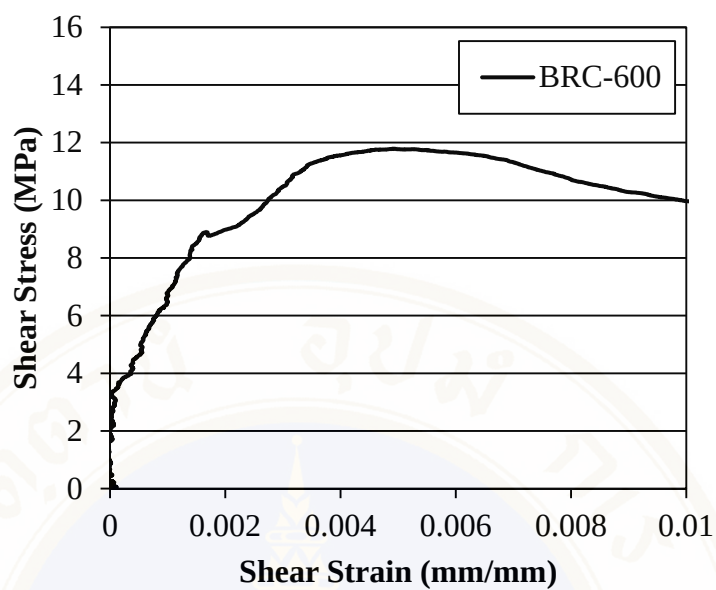


Figure C.23 Shear Stress vs. Shear Strain for BRC-600

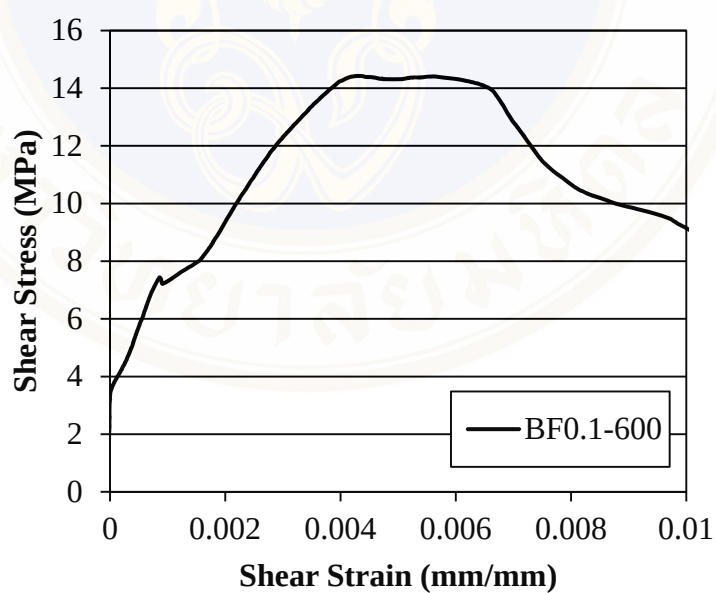


Figure C.24 Shear Stress vs. Shear Strain for BF0.1-600

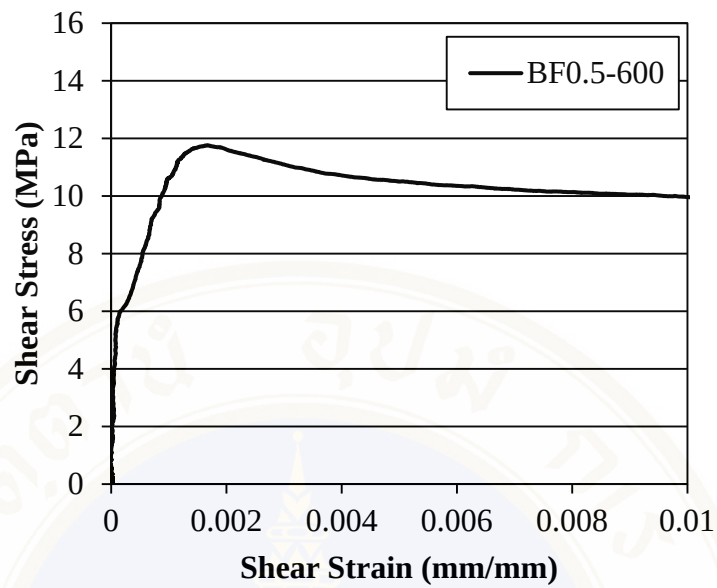


Figure C.25 Shear Stress vs. Shear Strain for BF0.5-600

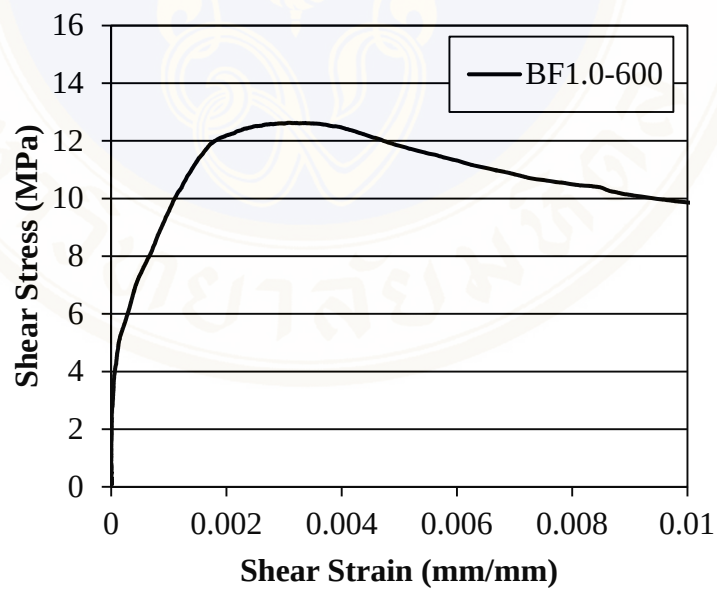


Figure C.26 Shear Stress vs. Shear Strain for BF1.0-600

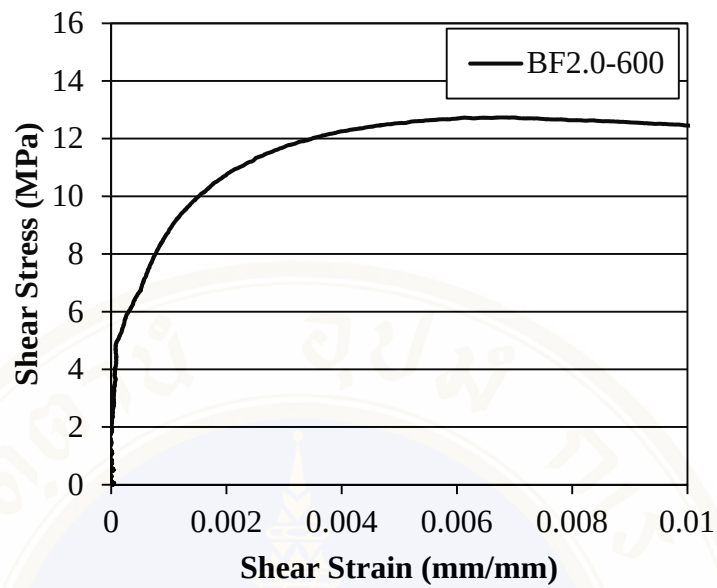


Figure C.27 Shear Stress vs. Shear Strain for BF2.0-600

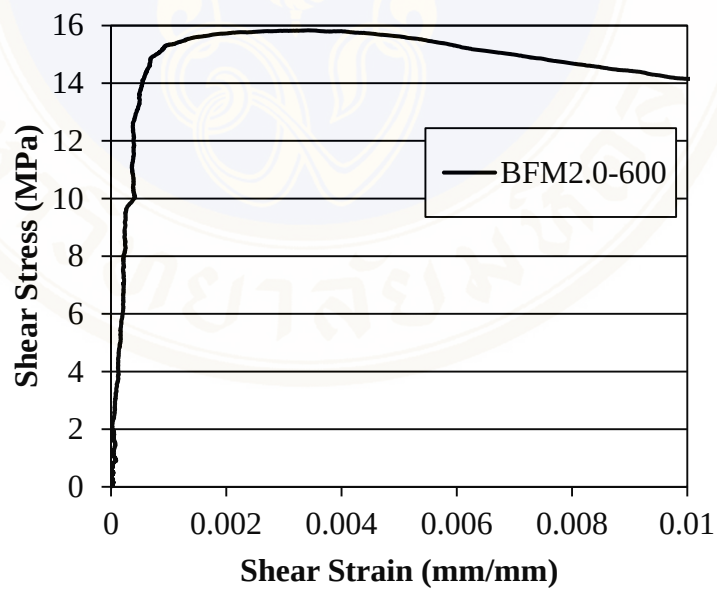


Figure C.28 Shear Stress vs. Shear Strain for BFM2.0-600

C.3 Nodal Zone and Strut Angle

Figure C.29 and Figure C.30 shows the pictures specimens superimposed with the assumed nodal zone, assumed strut direction, and measured strut angle for 800 mm and 600 mm span specimens, respectively.



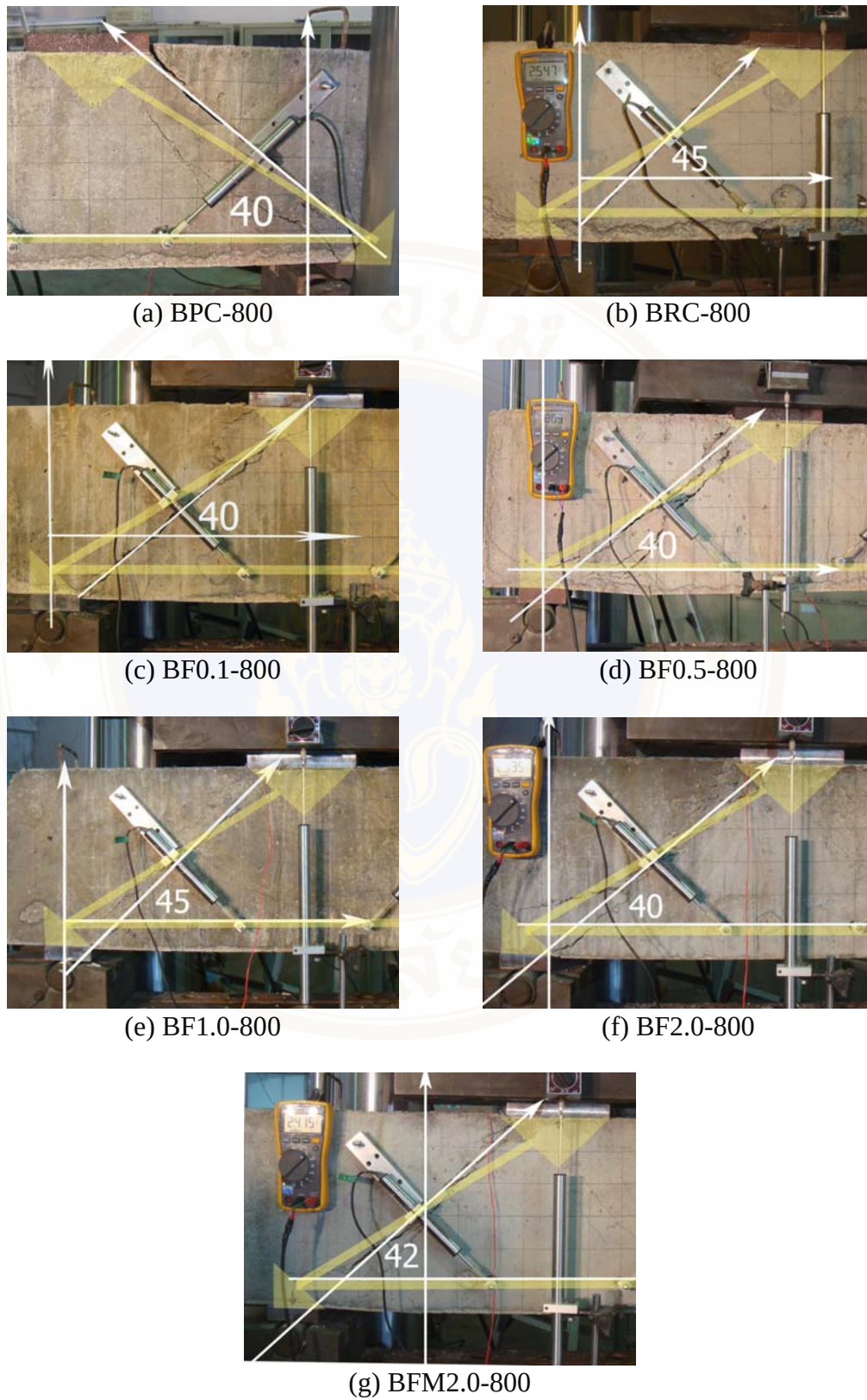


Figure C.29 Nodal Zone and Strut Angle for 800 mm Span Specimens

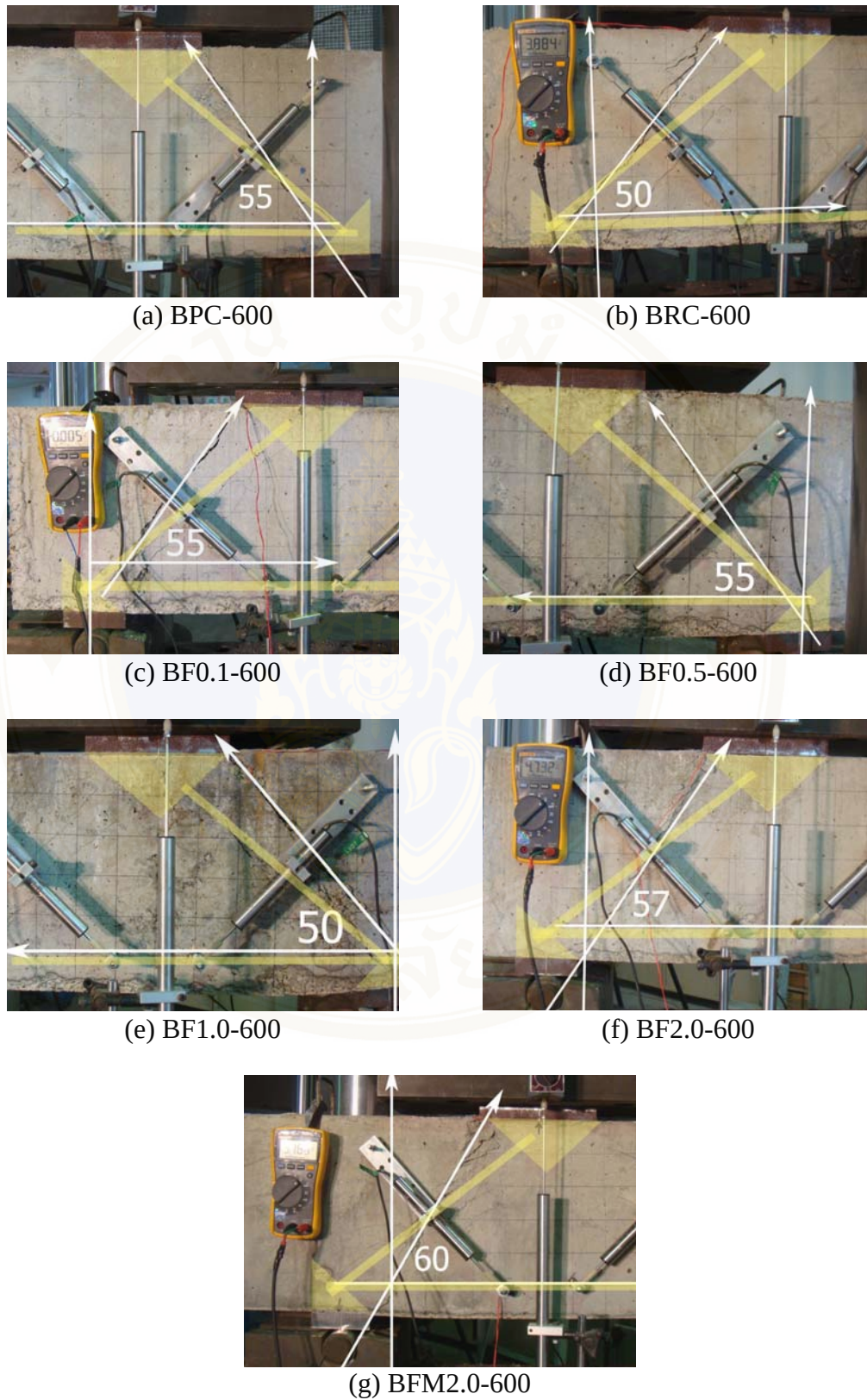


Figure C.30 Nodal Zone and Strut Angle for 600 mm Span Specimens

C.4 Failure Modes

Figure C.31 to Figure C.37 show the failure modes for 800 mm span specimens. Figure C.38 to Figure C.44 show the failure modes for 600 mm span specimens.

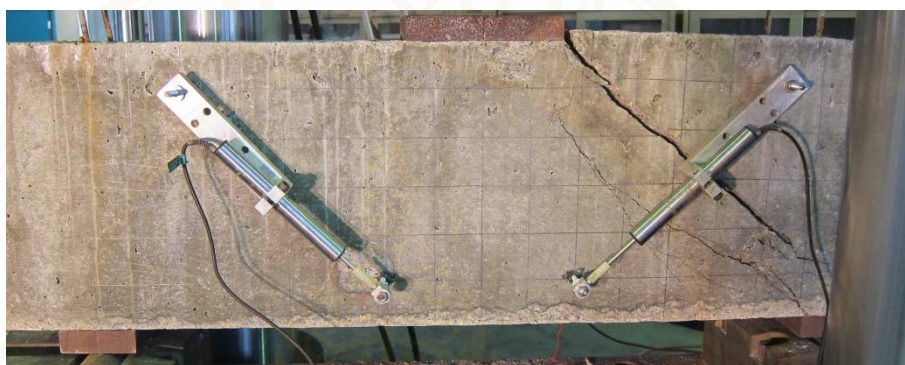


Figure C.31 Failure Mode for BPC-800

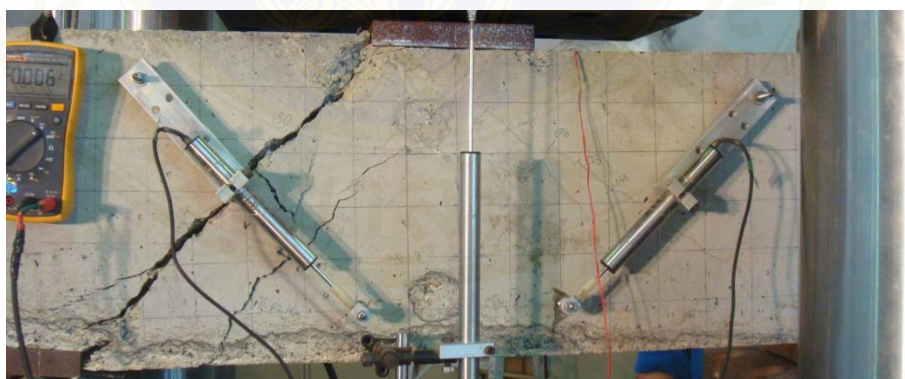


Figure C.32 Failure Mode for BRC-800



Figure C.33 Failure Mode for BF0.1-800



Figure C.34 Failure Mode for BF0.5-800



Figure C.35 Failure Mode for BF1.0-800

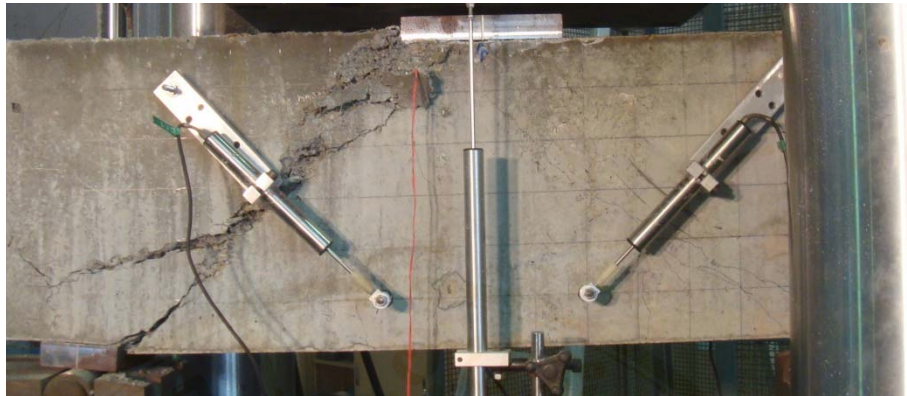


Figure C.36 Failure Mode for BF2.0-800

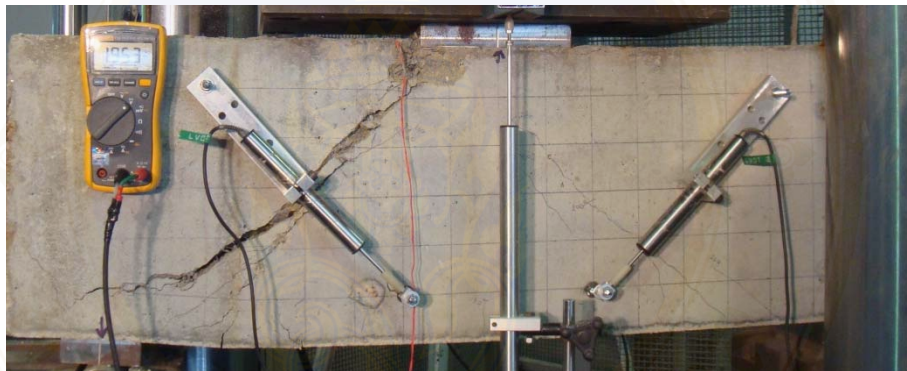


Figure C.37 Failure Mode for BFM2.0-800



Figure C.38 Failure Mode for BPC-600

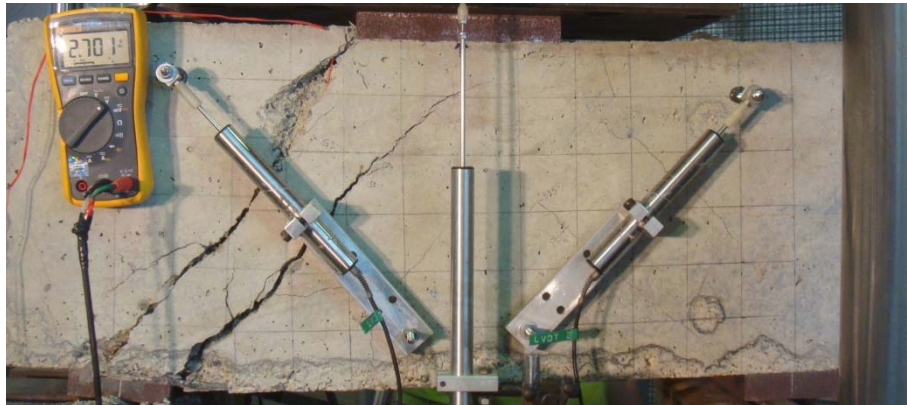


Figure C.39 Failure Mode for BRC-600

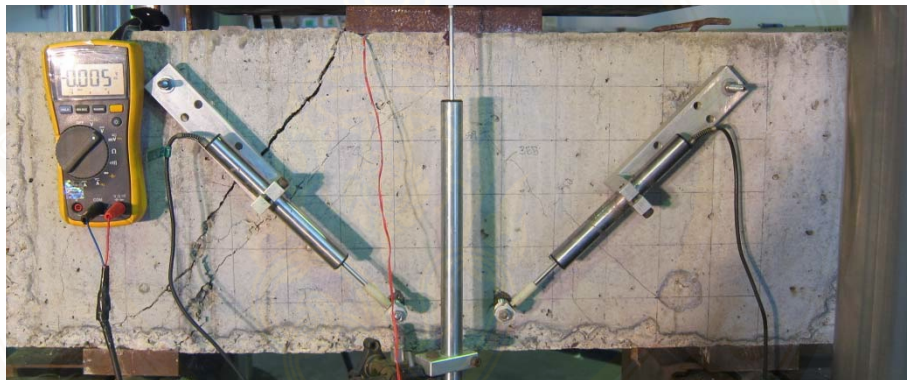


Figure C.40 Failure Mode for BF0.1-600

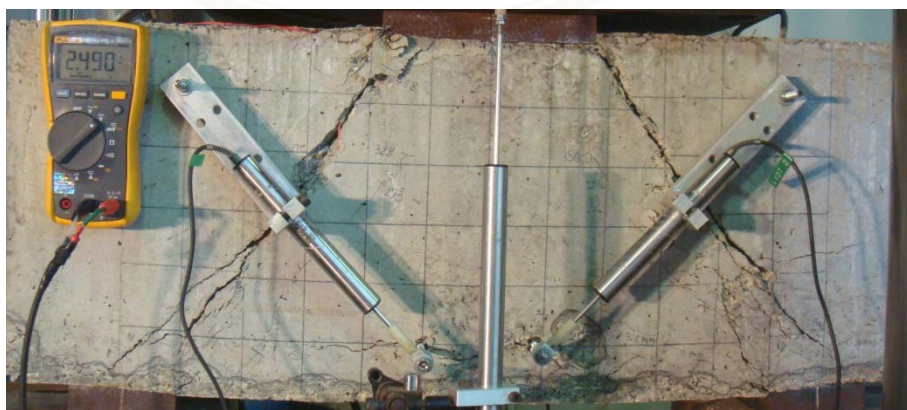


Figure C.41 Failure Mode for BF0.5-600

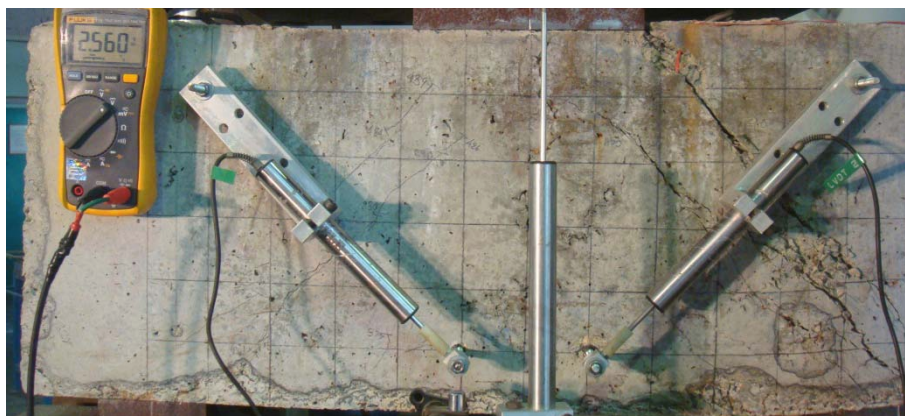


Figure C.42 Failure Mode for BF1.0-600

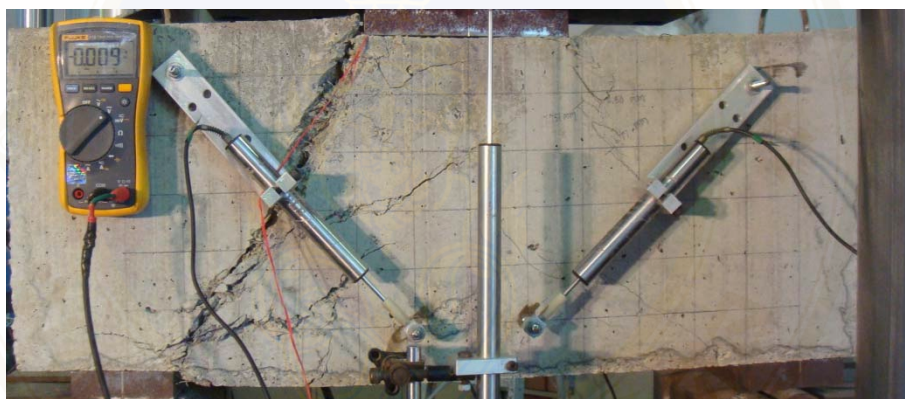


Figure C.43 Failure Mode for BF2.0-600

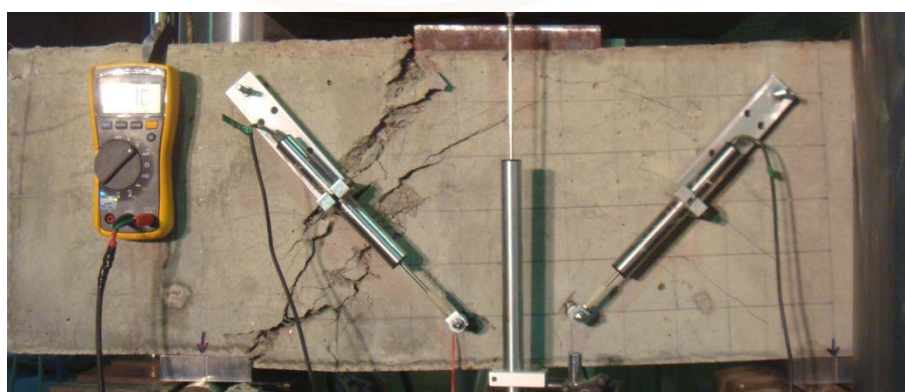


Figure C.44 Failure Mode for BFM2.0-600

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