Size Effect on Shear Strength of Reinforced Concrete Beams without Stirrups

by

Muhammad Hafez Bin Ismail

Dissertation submitted in partial fulfilment of

the requirements for the

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CERTIFICATION OF APPROVAL

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Approved by,

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(Dr. Teo Wee)

UNIVERSITI TEKNOLOGI PETRONAS
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September 2012

CERTIFICATION OF ORIGINALITY

This is to certify I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

MUHAMMAD HAFEZ BIN ISMAIL

ABSTRACT

Given the great discord concerning the mechanisms that govern shear failure, the shear behaviour of concrete beam elements with no transverse reinforcement is investigated.

The variables introduced in the experimental program are member depths and amount of longitudinal steel reinforcement. The effects of these variables on the shear stress at failure of the concrete are investigated. Two geometrically similar series of beams with different steel reinforcements are prepared. The dimensions of beams are 200 mm (w) x 400 mm (d) x 2000 mm (l) and all the beams have a constant a/d ratio of 2.0.

Apart from that, the beams casted will be in two different kinds of states, one is under-reinforced beam and the other beam is over-reinforced beam. Plus, the beams will be analyzed the changes in the shear strength with smaller dimension beams with similar, distinct and constant longitudinal steel percentages (1.3% for under-reinforced and 2.34% for over-reinforced).

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CHAPTER 1

INTRODUCTION

1.1 PREAMBLE

Decades ago, the concrete construction industries have faced with a very significant challenge in terms of the deterioration of the infrastructures, and there is currently a crucial need for the rehabilitation and repair of bridges, buildings, and highways in a large number of cases. The effects of environment such as harsh climate, de-icing salts, seismic activity and also the design of older structures, which may seem adequate when comparing it with the contemporary codes but it now against the current codes already. These factors are basically some of the driven factors that contribute to the infrastructures to become either structurally deficient or functionally obsolete.

Shear failure of reinforced concrete, more properly called diagonal tension failure, is very difficult to predict accurately. In spite of many decades of experimental researches and the use of highly sophisticated analytical tools, it is not yet fully understood. Furthermore, if a beam without properly designed shear reinforcement is overloaded to failure, shear collapse is likely to occur suddenly, with no advance warning of distress. This is in strong contrast with the nature of flexural failure. For typically under reinforced beams, flexural failure is initiated by gradual yielding of the tension steel, accompanied by obvious cracking of the concrete and large deflections, giving ample warning and providing the opportunity to take corrective measures.

As of now, there are so many techniques available for extending the useful life of those mentioned structures above. For instance, replacing non-structural toppings with structural toppings or adding extra reinforcement like in form of either externally bonded steel plates or fiber reinforced polymer (FRP) plates. Despite of all the methods suggested as aforementioned, it is better to have a detail and better understanding on the analysis of shear strength of RC beams, which do not have web reinforcement as if it will convey useful insights for the explanation of failure mechanism in beams in general.

1.2 MOTIVATION

It is known that in spite of numerous extensive studies over the last 50 years, the problem of how shear failures occur in RC beams remains largely, unresolved. A strong evidence of this is the fact that the international codes, such as American Concrete Institute (ACI) code (ACI 1999) or the Euro code2 [European Committee for Standardization (CEN) 1992], are based on rather (semi-) empirical considerations. Plus, there is a great discrepancy between design codes of different countries. Many of these codes do not even account for some basic and proven factors affecting the shear capacity of concrete members. Of these factors, much confusion is expressed with regards to the effect of absolute member size on the shear capacity of beam elements. On this subject, there is a lack of consensus in the approach to the problem due to the limited amount of experiments dedicated to this effect.

1.3 OBJECTIVE

The focus of this research is to evaluate the ‘size effect’ in normal concrete beams without web reinforcement in order to better understand the mechanisms involved. An experimental program is planned to investigate the following as well:

1.3.1 The reduction in shear stress at failure as the size of beams increases.

1.3.2 The effect of amount of longitudinal steel reinforcement on the shear stress at failure.

1.3.3 To investigate the modes of failure and shear behaviour of RC beam without transverse reinforcement in it.

1.3.4 To get the strain diagram inside the shear span of the beam.

1.4 SCOPE OF STUDY

This project will be solely focusing on the aspects related to the analysis of size effect to shear strength of RC beams without stirrups. This is happened due to time limitation that has been allocated for final year project. The study will be focusing on analyzing the failure of beam whereby an experimental research that involves laboratory work testing will be conducted. 2 beams will be casted and tested and with the steel reinforcement ratio; (ρ) has been set constant they will be compared with the other beams with different sizes. There are two beams dimensioning 200mm (w) x 400 mm (d) x 2000 mm (l) with different tensile reinforcement bars, which are using 10T-16 and 10T-12 to ensure one beam is under reinforced state and the other beam is over reinforced.

1.5 PROJECT FEASIBILITY

For the first phase of the Final Year Project (FYP 1), it will involve of doing literature review upon the analysis of shear strength of RC beams without using any stirrups. During the second phase of Final Year Project (FYP 2), only then, it will involve the casting of beams followed by an experimental testing on the beams. After that, an analytical study and interpretation of the results from the experiment will be done. With the aid of resources that are available in laboratory for tools and advice from supervisor it is feasible that for the project to be carried out and within the period of approximately seven (7) months, this is definitely can be done.

CHAPTER 2

LITERATURE REVIEW

2.1 MECHANISM OF SHEAR FAILURE

Shear failure occurs when the shear capacity of reinforced concrete beam section is exceeded and a sliding failure develops on the beam induced by potential shear crack, which is perpendicular to the direction of tensile force. A few researches namely Zhang and Zhu[1], Sharif et al[2], and Altin et al[3] mentioned that shear failure of concrete structures is catastrophic and brittle in nature. It happens without advance warning prior to failure, thus making it harmful. Hence, it can be concluded that RC beams must be designed to develop their full flexural capacity and assure a ductile flexural failure mode under extreme loading, as it is desirable for beams to fail under flexural rather than in shear.

Another researcher, Kani[4] mentioned that the concerned shear failure is diagonal failure in reinforced concrete beams. In his research, a series of test had been done, whereby when loaded; failure was induced by cracks outside the central section of the beam. Hence, it can be concluded that the failure is caused by constant shear force at the ends of the beams where cracks appeared. Therefore, it was decided that shear stress is responsible for such failure.

2.2 SHEAR SPAN TO DEPTH RATIO, ($^{a}/_{d}$)

Kim and Park (1996), they have mentioned that according to the shear span-to-depth ratio ($^{a}/_{d}$), shear failure of a reinforced concrete beam without web reinforcement is divided into two modes, as shown in Figure 2-1. For ($^{a}/_{d}$) less than 2.0~3.0, the inclined cracking loads exceeds the shear compression failure load. Within the formation of the inclined crack, a beam without web reinforcement becomes unstable and fails. This type of failure is usually called "diagonal tension failure". For $\left({}^{a}/_{d}\right)$ less than 2.0~3.0, however, the failure load exceeds the inclined cracking load. If sufficient anchorage length is provided, after the inclined crack develops, failure may occur by concrete crushing in the upper end, and this type of failure is called "shear compression failure".

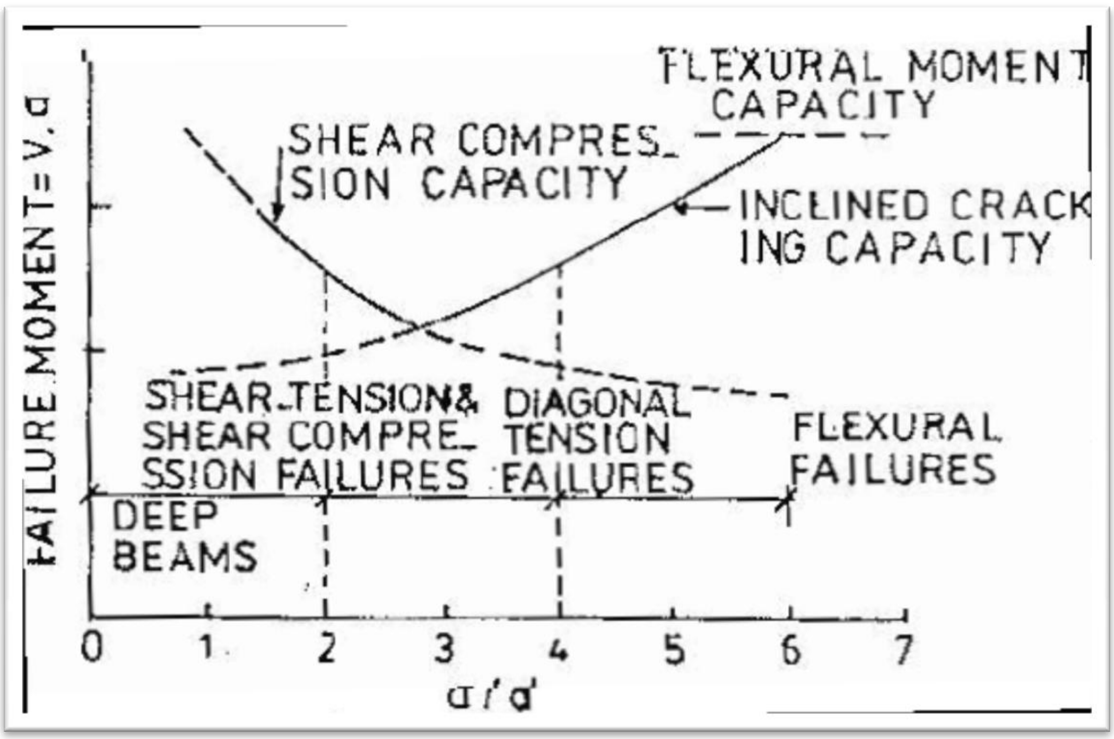

Figure 2-1: Effect of a/d on shear failure mode

Shear force in reinforced concrete member is transferred in various ways. For slender beams where $\left({}^{a}/_{d}\right)$ is greater than 2.0~3.0, shear force is carried by the shear resistance of uncracked concrete in the compression zone, the interlocking action of aggregates along the rough concrete surfaces on each side of the crack, and the dowel action of the longitudinal reinforcement. For relatively short beams, however, after the breakdown of beam action, shear force is resisted mainly by arch action.

Test results have shown that the shear strength of reinforced concrete beams without web reinforcement depends mainly on concrete strength, longitudinal steel ratio, shear span-to-depth ratio, and effective depth. Factors such as maximum aggregate size, diameter of the bars, and spacing of the flexural cracks show some minor contribution. A part of all these primary factors are included in the existing shear strength prediction models, but the effects of these factors are estimated differently according to the models.

Recently, high-strength concrete has been increasingly used in practice. With the development of concrete technology and the introduction of super plasticizers and silica fume, the compressive strength of concrete in the field of ready-mixed concrete reached 100 MPa (14,300 psi) and higher. Since the mechanical properties of concrete are changed in high-strength concrete, a reevaluation of the prediction model is necessary to reliably estimate the shear strength of beams made with high-strength concrete; more accurate predictions of shear strength of reinforced concrete members are required.

Apart from that, the shear failure of reinforced concrete beams without web reinforcement has been known to be a typical case of brittle failure and indicates significant size effect. In 1981, Reinhardt[1] introduced fracture mechanics in the prediction of shear strength. He analyzed limited test data for shear failure based on linear elastic fracture mechanics. Subsequently, it was established that the size effect implied by linear elastic fracture mechanics is too strong in the case of concrete, and that brittle failures of concrete structures are better described by nonlinear fracture mechanics. Meanwhile, simple and approximate size effect laws on the basis ofnonlinear mechanics was proposed by Bazant[2]. Several studies[3-6] have shown that Bazant's size effect law is in good agreement with test results. However, there is some discrepancy between the prediction by Bazant's size effect law and the test data, particularly for large-sized specimens. Recently, Kim and Eo[7] proposed a modified Bazant's size effect law to reduce the discrepancy.

In the present study, a simple and accurate equation predicting the shear strength of reinforced concrete beams without web reinforcement is proposed based on basis mechanisms of shear transfer and a modified Bazant's size effect law deduced by Kim and Eo, and it was verified by the published test data. In addition, a simplified equation is also proposed for a practical design purposes. The equations that include the effects of all the factors previously mentioned are supported by test results and are compared with other prediction equations for the shear strength of beams without web reinforcement.

2.3 BASIC SHEAR TRANSFER MECHANISM

The factors assumed to be carrying shear force in cracked concrete to the supports when no shear reinforcement is provided for the member, are illustrated in the following free body diagram (Figure 2-2)

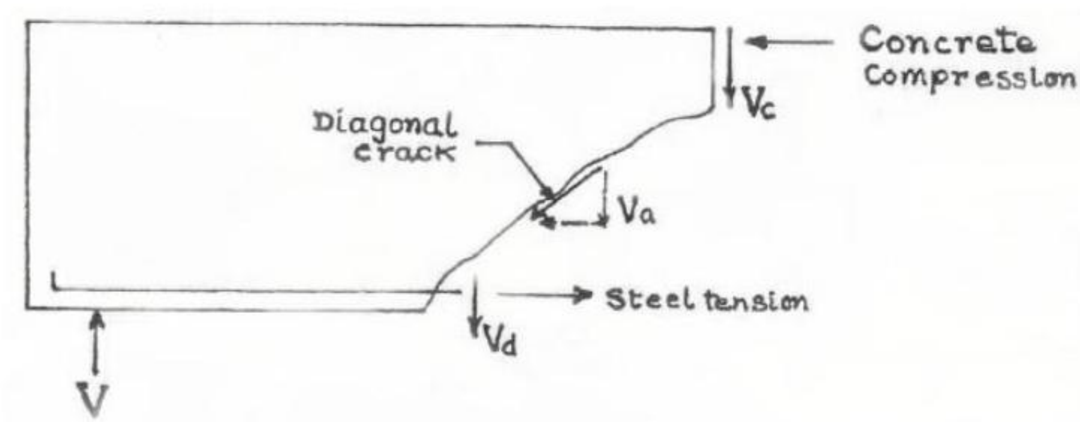

Figure 2-2: Shear transfer mechanism

These three factors are the sum of beam action. In addition to beam action, arch action also contributes to the shear resistance.

Many investigators have tried to determine the contribution from each of the elements of beam action to shear resistance. It was concluded by some that after inclined cracks developed in the concrete, the contribution from each of the following V_d, V_a and V_c altered between 15-25%, 33-50% and 20-40% (Ziara, 1993) and (Kim & Park, 1996)

2.3.1 Concrete Compression Zone (V_c)

Gradually inclined cracks widen in the concrete, the shear resistance from V_a decereases while V_c and V_d increase. Finally when the aggregate interlock reaches failure, large shear force transfers rapidly to the compression zone causing sudden and often explosive failure to the beam when arch action contribution is low.

2.3.2 Aggregate Interlock (V_a)

It is generally believed that aggregate interlock transfers a large part of the total shear force to the supports. Width of the cracks, aggregate size and concrete strength are the most important variables. When the longitudinal reinforcement ratio is increased with added bars to the beam, the width of the flexural cracks get smaller due to increased shear resistance and consequently the contribution of V_d decreases.

According to previous investigations (Sherwood, Bentz, & Collins, 2007), the shear resistance of the normal weight large concrete beams (d=1400m), increased by 24% when varying the maximum aggregate size between 9.5 mm and 51 mm. The increase reduced in the smaller beams (d=280 mm) to 6 %. Figure 2-3 illustrates their test results.

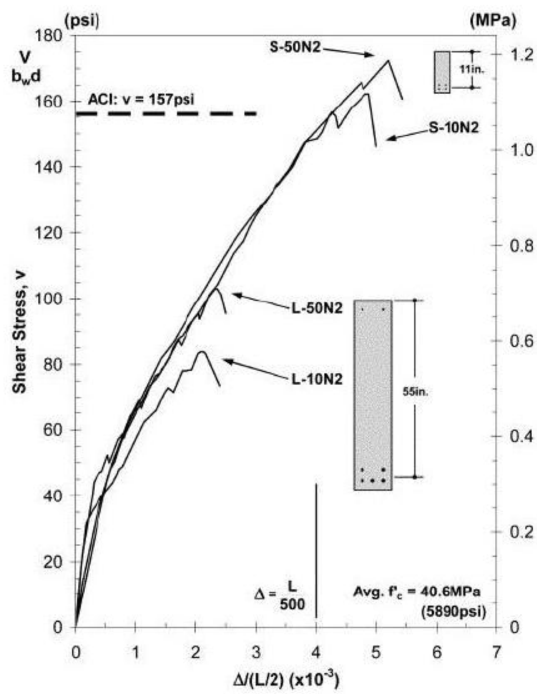

Figure 2-3: Change in aggregate size in normal weight concrete beams (Sherwood, Bentz, & Collins, 2007)

This proved their suspicion that large coarse aggregates can increase shear resistance because the surface of the crack is rougher (Figure 2-4).

On the other hand when the concrete strength was increased in one of the larger specimen > 70 MPa the load at failure scored beneath the lower strength concrete. The reason is that the surface of the crack was much smoother because all of the aggregates had fractured.

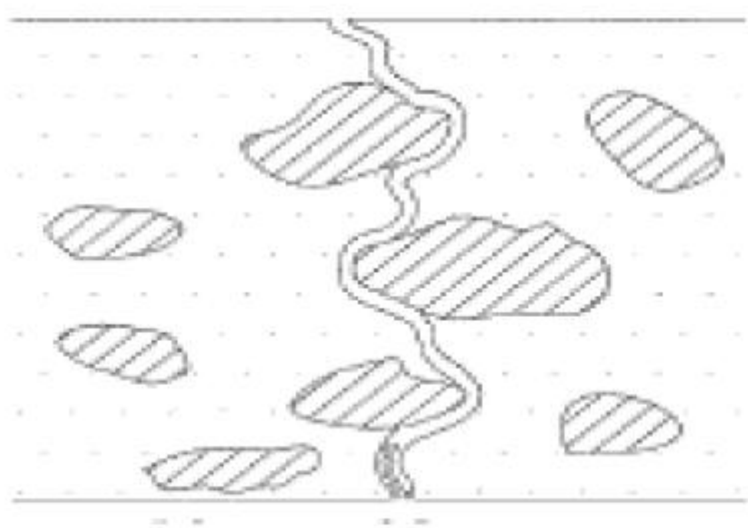

Figure 2-4: Crack passing through the concrete, around the aggregates

2.3.3 Dowel Action (V_d)

Shear resistance caused by dowel action increases as the shear reinforcement decreases. Consequently it has a significant effect in members where no shear reinforcement is provided. When inclined crack cross the longitudinal reinforcing bar, forces act on the dowel due to e.g. deflection of the bar at the face of the crack (Figure 2-5). Aggregates around the bar try to resist the deflection by interlocking with each other and those entire forces sum up as the total shear resistant of dowel action (El-Ariss, 2006) & (Dileep Kumar, 2008)

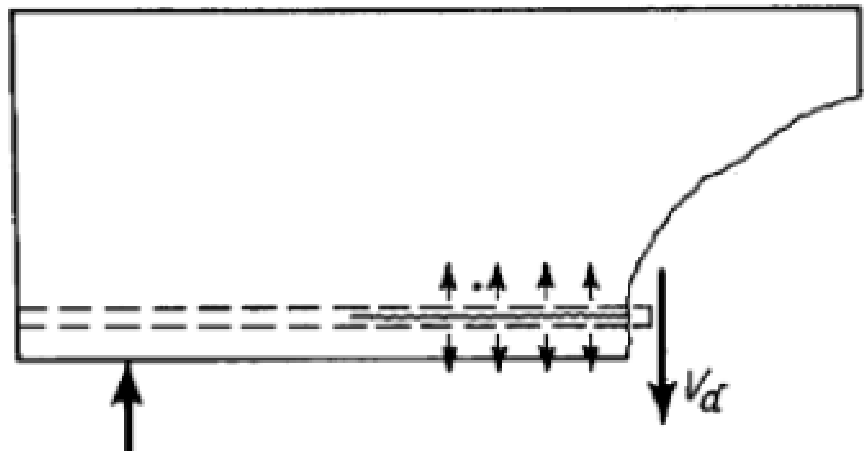

Figure 2-5: Dowel action

2.3.4 Arch Action

When beams develop a flexure-shear interaction, the shear resistance consists of two different mechanisms, beam and arch mechanisms. The former governs when the a/d ratio is above the critical (transition) point and the latter when it is below (Figure 2-6). When the arch action begins to contribute more than beam action, the member can achieve considerably more load than at diagonal cracking.

To predict failure mode of the member, Russo et al. (Russo, Zingone, & Puleri, 1991) concluded that when arch action governs, shear-compression (SC) failure should be expected and diagonal-tension (DT) should be expected if beam action governs.

When talking about flexure-shear, it’s when bending moment and shear force act together in a cross section a/d=M/ (V.d).

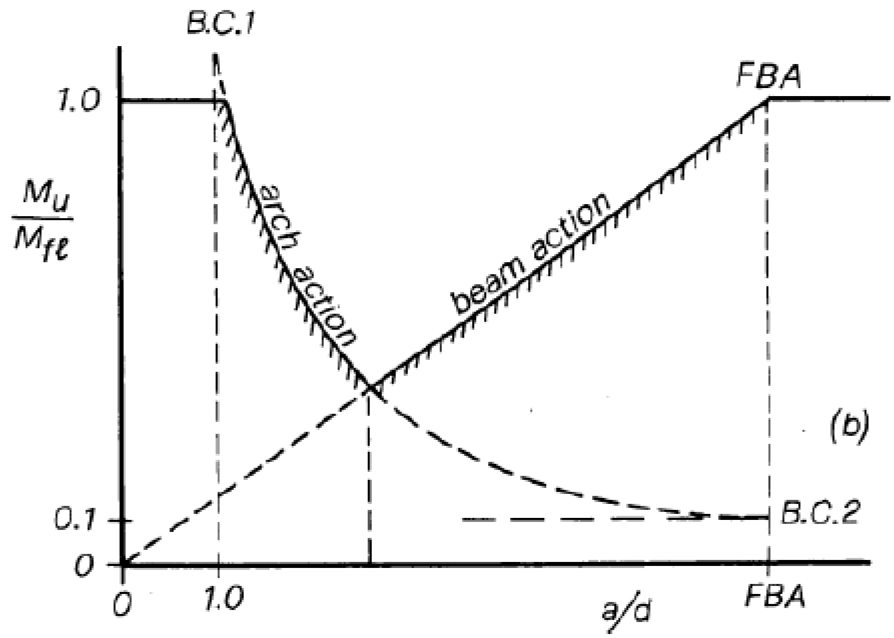

Figure 2-6: Model for flexure-shear interaction (Russo, Zingone, & Puleri, 1991)

(Kani, 1964) has described the arch action as followed:

"Under increasing load reinforced concrete beam transforms into a comb-like structure. In the tensile zone the flexural cracks create more or less vertical concrete teeth, while the compression zone represents the backbone of the concrete comb. The analysis of this structural system has revealed that two rather different mechanisms are possible: as long as the capacity of the concrete teeth is not exceeded the beam like behaviour governs; after the resistance of the concrete tooth has been destroyed a tied arch, having quite different properties, remains."

Figure 2-7 illustrates Kani's concrete tooth and backbone of the concrete comb.

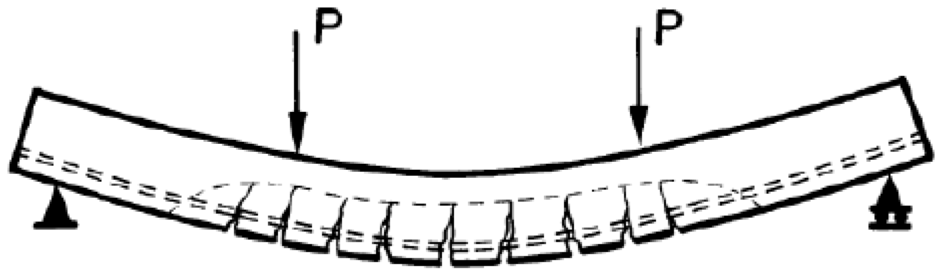

Figure 2-7: Flexural failure and concrete teeth (Kani, 1964)

2.4 PREVIOUS SIZE EFFECT INVESTIGATIONS.

In 1955, the Wilkins Air Force Depot Warehouse in Shelby, Ohio, collapsed due to the shear failure of 36 in. (914 mm) deep beams, which did not contain any stirrups at the location of failure (Collins and Kuchma, 1997 and Collins and Mitchell, 1997). These beams had a longitudinal steel ration of only 0.45%. They failed at a shear stress of only about 0.5 MPa whereas the ACI Building Code of the time (ACI Committee 318, 1951) permitted an allowable working stress of 0.62 MPa for the 20 MPa concrete used in the beams. Experiments conducted at the Portland Cement Association (Elstner and Hognestad, 1957) on 12 in. (305 mm) deep model beams indicated that the beams could resist about 1.0 MPa. However, the application of an axial tension stress of about 1.4 MPa reduced the shear capacity by about 50%. It was thus concluded that tensile stresses caused by thermal and shrinkage movements were the reason for the beam failures.

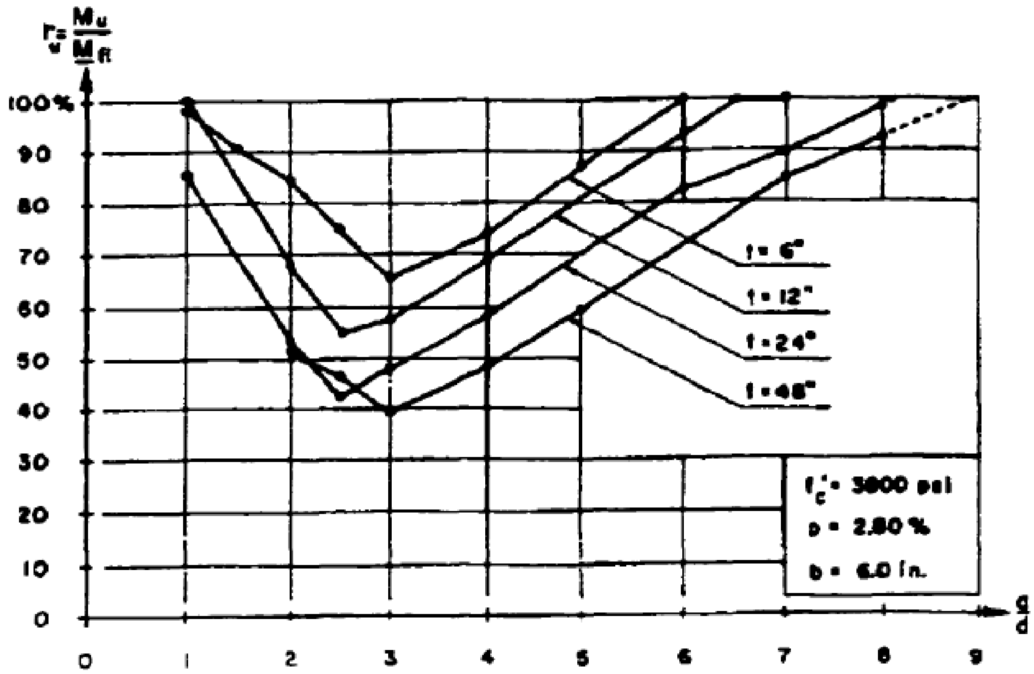

Figure 2-8: Relative strength (Ultimate moment/flexural moment) vs. a/d ratio (Kani 1967)

Kani (1966 and 1967) was amongst the first to investigate the effect of the absolute member size on concrete shear strength after the dramatic warehouse shear failures of 1955 (Collins and Kuchma, 1997 and Collins and Mitchell, 1997). His work consisted of beams without web reinforcement with varying member depths, d, longitudinal steel percentages, ρ, and shear span-to-depth ratios, a/d. He determined that member depth and steel percentage had a great effect on shear strength and that there is a transition point at a/d≈ 2.5 at which beams are shear critical (i.e. the value of the bending moment at failure was minimum) (see Fig. 2-8)

Kani found this value of a/d to be the transition point between failure modes and is the same for different member sizes and steel ratio. Below an a/d ratio value of about 2.5 the test beams developed arch action and had a considerable reserve of strength beyond the first cracking point. For a/d values greater than 2.5 failures was sudden, brittle and in diagonal tension soon after the first diagonal cracks appeared. This transition point is more emphasized in test beams containing higher reinforcement ratios and almost disappears in specimens with lower reinforcement ratios. In addition, Kani found a clearly defined envelope bounded by limiting values of ρ and a/d. Inside this envelope diagonal shear failures are predicted to occur and outside of this envelope flexural failures are predicted to occur. These conclusions regarding the influence of both ρ and a/d were similar for all beam depths tested. Kani also looked at the effect of beam width and found no significant effect on shear strength.

More recently, Bazant and Kim (1984) derived a shear strength equation based on the theory of fracture mechanics. This equation accounts for the size effect phenomenon as well as the longitudinal steel ratio and incorporates the effect of aggregate size. This equation was calibrated using 296 previous tests obtained from the literature and was compared with the ACI Code equations. It was noted after the comparison that the practice used in the ACI Code of designing for diagonal shear crack initiation rather than ultimate strength does not yield a uniform safety margin when different beam sizes are considered. It was also found, according to the new equation, that for very large specimen depths the factor of safety in the ACI Code almost disappears. However, no experimental evidence was available ye to confirm that fact as all the tests performed up to that time were on relatively small specimens. This equation was improved by Bazant and Sun (1987) to account for the maximum aggregate size distinctly from the size effect phenomenon and was extended to cover the influence of stirrups. This formula was calibrated using a larger set of test data consisting of 461 test results compiled from the literature.

Later on, Bazant and Kazemi (1991) performed tests on geometrically similar beam with a size range of 1:16 and having a constant a/d ratio of 3.0 and a constant longitudinal steel ratio, ρ. Beams tested varied in depth from 1 inch (25 mm) to 16 inches (406 mm). The main failure mode of the specimens tested was diagonal shear but the smallest specimen failed in flexure. This study confirmed the size effect phenomenon and helped corroborate the previously published formula. However, the deepest beam tested was relatively small and the authors concluded that for beams larger than 16 inches (406 mm) additional reductions in shear strength due to size effect were likely.

Kim and Park (1994) performed tests on beams with a higher than a normal concrete strength (53.7 MPa). Test variables were longitudinal steel ratio, ρ, shear span-to-depth ratio, a/d, and effective depth, d. Beams heights varied from 170mm to 1000mm while the longitudinal steel ratio varied from 0.01 to 0.049 and a/d varied from 1.5 to 6.0. Their findings were similar to Kani's from which it was concluded that the behavior of the higher strength concrete is similar to that of normal-strength concrete. However, since only one concrete strength was investigated no general conclusions could be made with respect to concrete strength and shear capacity.

Shioya (1989) conducted a number of tests on large-scale beams in which the influence of member depth and aggregate size on shear strength was investigated. In this study, lightly reinforced concrete beams containing no transverse reinforcement were tested under a uniformly distributed load. The beam depths in this experimental program ranged from 100mm to 3000 mm. Shioya found that the shear stress at failure decreased as the member size increased and as the aggregate size decreased. It is interesting to note that the beams tested by Shioya contained about the same amount of longitudinal reinforcement as the roof beams of the Air Force warehouse which collapsed in 1955 (Collins and Kuchma, 1997 and Collins and Mitchell, 1997). The warehouse beams had an effective depth of 850 mm and failed at a shear stress of about $0.10\sqrt{f'_c}$MPa. This shear stress observed in beams having a depth of 1000 mm in the Shioya tests. It is important to mention that there was a tendency for reduced shear stress at failure even with tests including 3000 mm deep beams. Figure 2-9 illustrates the results obtained by Shioya.

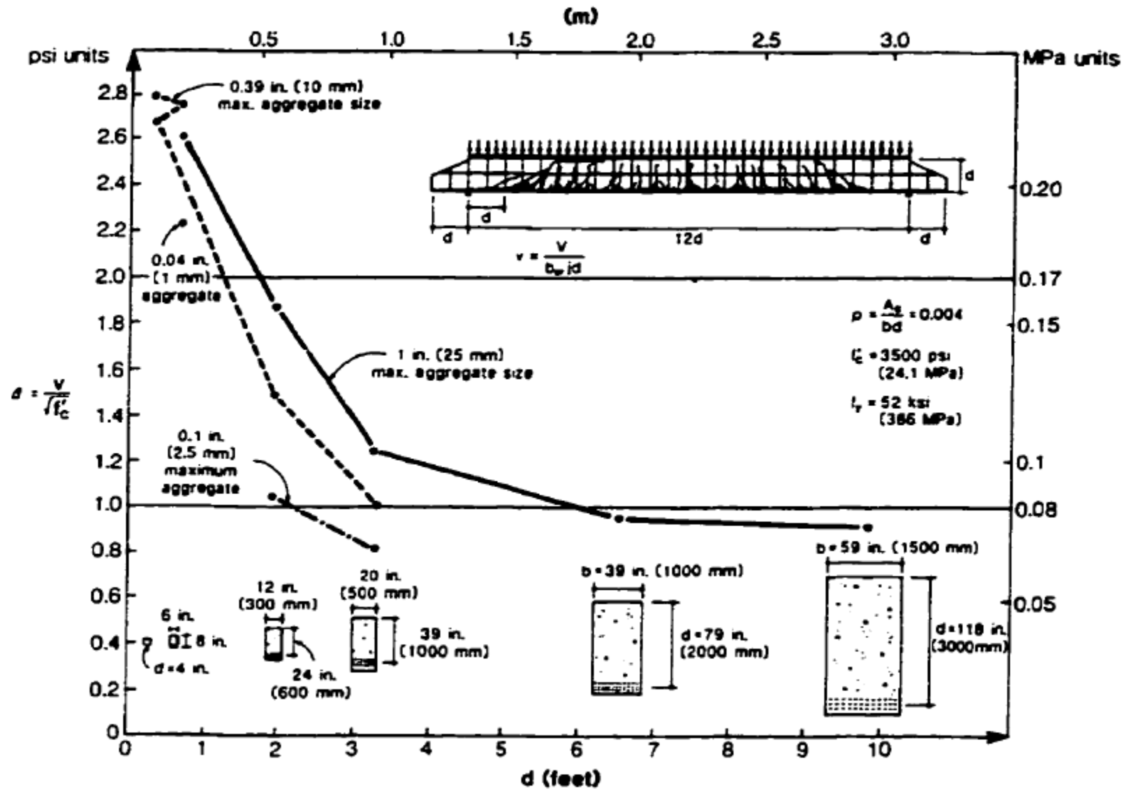

Figure 2-9: Influence of member depth and aggregate size on shear stress at failure for tests carried out by Shioya 1989, taken from Collins and Mitchell, 1997.

Stanik (1998) performed tests on a wide range of beam specimens at the University of Toronto. The specimens tested had a varying depths, d, ranging from 125 mm to 1000 mm, varying amounts of longitudinal steel (0.76% to 1.3%) as well as varying concrete strength, $f'_{c,}$ ranging from 37 MPa to 99 MPa. The longitudinal reinforcement was distributed in some specimens along the sides and some specimens contained the minimum amount of transverse reinforcement recommended by the CSA Standard (CSA 1994). In the series with longitudinal bars along the sides, a set of wider beams was also tested. The purpose was to evaluate the influence of the amount, as well as the distribution of the longitudinal steel on the shear strength. Stanik found that the size effect is very pronounced in lightly reinforced deep members. Members containing the minimum amount of transverse reinforcement or side distributed steel performed better than their counterparts with only bottom longitudinal reinforcing bars. Deep members with side distributed reinforcement performed nearly as well as the shallow members containing only bottom longitudinal reinforcement. As well, the wider members containing side distributed steel were weaker than the narrower ones with similar side distributed steel. Stanik concluded that the size effect is more related to measures controlling crack widths and crack spacing rather than the absolute depth of the member.

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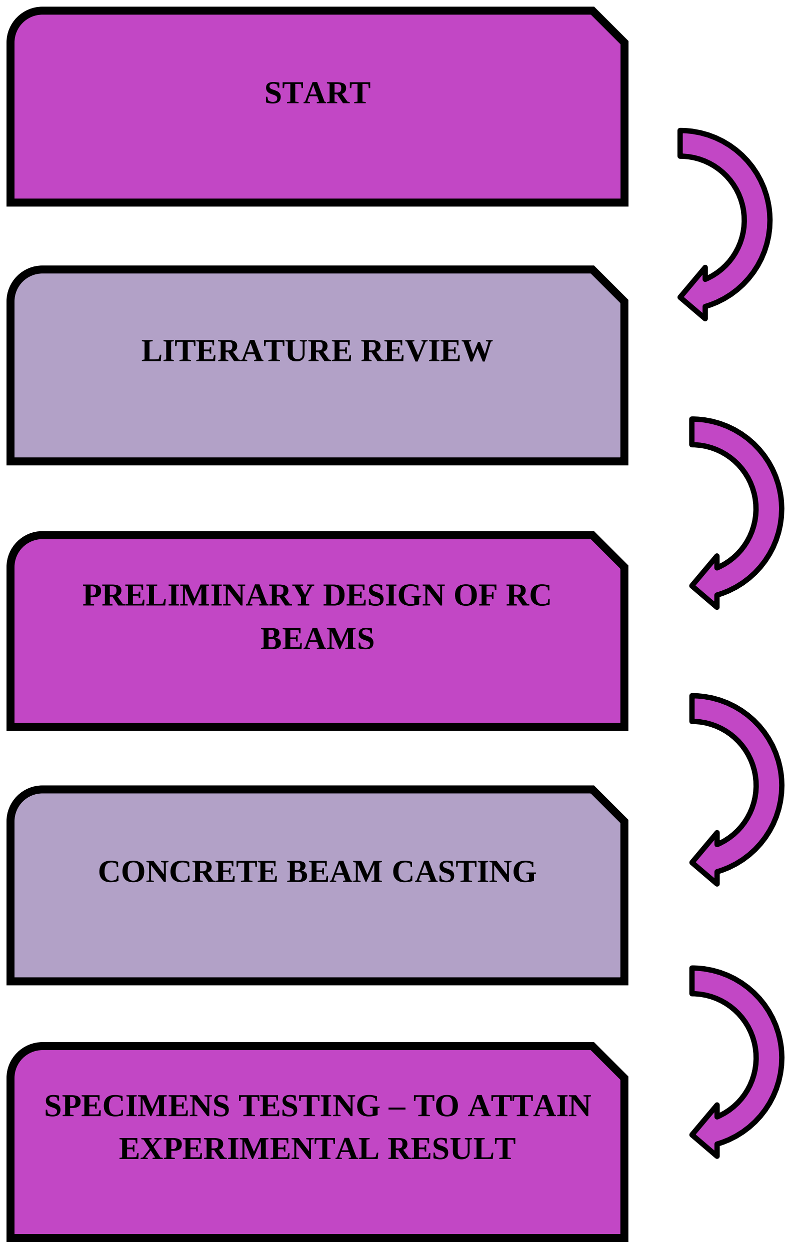
CHAPTER 3

METHODOLOGY

3.1 PROJECT PROCESS FLOW



3.2 BEAM DETAILS

An experiment will be conducted to know the size effect of the beams to the shear strength of it and a total of two beams with geometrically the same in shape and dimension but differs in which one of them will be under reinforced concrete beam and the other one will be over reinforced concrete beam.

The design calculations are done based on BS Code 8110 to prove the suitability of beam section used.

- Beam size : 200 mm (w) x 400 mm (d) x 2000 mm (l)
- Reinforcement beam 1 : 10T-12
- Reinforcement beam 2 : 10T-16
- Cover : 20 mm
- Effective depth of beam 1 : 400-20-12-10 = 358 mm
- Effective depth of beam 2 : 400-20-16-10 = 354 mm
- f_y : 460 MPa
- f_{cu} : 30 MPa

3.3 PROJECT ACTIVITY PHASES

The execution of each activities in this project have been phased out accordingly in order to ensure that for every activities, they will be conducted as per planned and follow the timeline that has been set up earlier. Apart from that, as this project has been categorized into several phases, it is hoped that the process of implementation for every phase will be much easier and can be completed on time. The phases of activity that will be conducted are as followed:

1. Preliminary design of RC beams
2. Concrete Casting
3. Testing of RC beams.

As of current situation in FYP 1, the preliminary design of RC beams will take place and as for the rest will be done in FYP 2

Preliminary design of RC beams

Beam dimension is: 200 mm (w) x 400 mm (d) x 2000 mm (l) with a cover of 20 mm. The proposed idea is that to come out with two different states of the beams in which one will be in under reinforced state and the other one will be in over reinforced state.

Under-reinforced beam

The beam in which the tension capacity of the tensile reinforcement is **smaller** than the combined compression capacity of the concrete and the compression steel (under-reinforced at tensile face).When the reinforced concrete element is subject to increasing bending moment, the tension steel yields while the concrete does not reach its ultimate failure condition. As the tension steel yields and stretches, an “under-reinforced” concrete also yields in a ductile manner, exhibiting a large deformation and warning before its ultimate failure. In this case, the yield stress of the steel governs the design.

Over-reinforced beam

The beam in which the tension capacity of the tension steel is **greater** than the combined compression capacity of the concrete and the compression steel (over-reinforced at tensile face). So the “over-reinforced concrete” beam fails by crushing of the compressive –zone concrete and before tension zone steel yields, which does not provide any warning before failure as the failure is very instantaneous.

1) With the proposed reinforcement of 10T-12 reinforcement bars in order to have an **under-reinforced beam**, it will contribute an area of 1130.97 mm^2 with an effective depth of 358 mm.

Compressive force, $C = 0.67fcu(0.9x)(b)$ **Assume f_{cu}= 30 N/mm^2**

$= 0.603f_{cu}bx$

= 0.603(30) (200) x

= 3618x

Tensional force, T $= A_sf_y$ **Assume f_y = 460 N/mm^2**

= 1130.97(460)/1000

= 520.246 kN

Compressive force = Tensional force

3618x = 520.46 kN

$$x = \frac{520.46}{3618}$$

According to BS8110 – For under-reinforced beam $x \leq 0.5d$

X = 144 mm < 0.5d = 0.5(358) = 179 mm

Reinforced Concrete Design is OK!

2) For **over-reinforced beam**, it is proposed to have 10T-16 reinforcement bars with an area of 2010.62 mm^2 with an effective depth of 354 mm.

Compressive force, $C = 0.67fcu(0.9x)(b)$ **Assume f_{cu}= 30 N/mm^2**

$= 0.603 f_{cu} bx$

$= 0.603(30)(200)x$

$= 3618x$

Tensional force, T $= A_s f_y$ **Assume f_y = 460 N/mm^2**

$= 2010.62(460)/1000$

$= 924.89$ kN

Compressive force = Tensional force

3618x = 924.89 kN

$$x = \frac{924.89}{3618}$$

According to BS8110 – For under-reinforced beam x ≥ 0.5d

X = 256 mm > 0.5d = 0.5(354) = 177 mm

Reinforced Concrete Design is OK!

3) Prediction of the shear capacity of the beam

3.1) According to BS 8110 – 1:1997 Structural use of concrete, part 1 code of practice for design and construction

For T-12

$$V_c = \frac{0.79}{\gamma_m}\left(\frac{fcu}{25}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4}\left(100\frac{As}{bd}\right)^{1/3}$$

Where

γ_m = 1.25 (safety factor)

Fcu = Concrete Strength

d = Effective Depth

b = Width

As = Area of Reinforcement

$$V_c = \frac{0.79}{\gamma_m}\left(\frac{30}{25}\right)^{1/3}\left(\frac{400}{358}\right)^{1/4}\left(100\frac{1130.97}{200x358}\right)^{1/3}$$

$V_c =$ (0.632)(1.0627)(1.0281)(1.1646)

$V_c =$ 0.804 N/mm^2

For T-16

$$V_c = \frac{0.79}{\gamma_m}\left(\frac{fcu}{25}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4}\left(100\frac{As}{bd}\right)^{1/3}$$

Where

γ_m = 1.25 (safety factor)

Fcu = Concrete Strength

d = Effective Depth

b = Width

As = Area of Reinforcement

$$V_c = \frac{0.79}{\gamma_m}\left(\frac{30}{25}\right)^{1/3}\left(\frac{400}{354}\right)^{1/4}\left(100\frac{2010.64}{200x354}\right)^{1/3}$$

$V_c =$ (0.632)(1.0627)(1.0310)(1.784)

$V_c =$ 1.236 N/mm^2

3.2) EN 1992-1-1, 2004 (EUROCODE)

10T-12

$$V_{Rk,c} = \left[C_{Rk,c}.K.((100.\rho l.fck)^{\frac{1}{3}})\right].bw.d$$

Where $C_{Rk,c} = 0.18$

K = 1 + $\sqrt{\frac{200}{d}} \leq 2.0$

Fck = concrete strength

ρl = $\frac{A_{sl}}{b_w d}$

$b_w = width$

d = effective depth

K = 1 + $\sqrt{\frac{200}{358}} \leq 2.0$

K = 1.74

ρl = $\frac{1130.97}{200 \ x \ 358} \leq 0.02$

ρl = 0.016

$$V_{Rk,c} = \left[C_{Rk,c}.K.((100.\rho l.fck)^{\frac{1}{3}})\right] .bw.d$$

$V_{Rk,c}$ = [(0.18) (1.74) ((100 x 0.023 x 30)*(1/3))]*200*358

$V_{Rk,c}$ = 91978.11 N

10T-16

$$V_{Rk,c} = \left[C_{Rk,c}.K.((100.\rho l.fck)^{\frac{1}{3}})\right].bw.d$$

Where $C_{Rk,c} = 0.18$

K = 1 + $\sqrt{\frac{200}{d}} \leq 2.0$

Fck = concrete strength

ρl = $\frac{A_{sl}}{b_w d}$

$b_w = width$

d = effective depth

K = 1 + $\sqrt{\frac{200}{354}} \leq 2.0$

K = 1.75

ρl = $\frac{2010.62}{200\ x\ 354} \leq 0.02$

ρl = 0.028

$V_{Rk,c} = \left[C_{Rk,c}.K.((100.\rho l.fck)^{\frac{1}{3}})\right] .bw.d$

$V_{Rk,c}$ = [(0.18) (1.74) ((100 x 0.028 x 30)*(1/3))]*200*354

$V_{Rk,c}$ = 97113.9 N

3.3) ACI 318-08, 2007 (American Concrete Institution)

10T-12

$$V_c = \left(\left(\lambda\sqrt{f'c}\right)/6\right)bw.d$$

Where

λ = 1.0

Fc = Concrete Strength

bw = Width

d = Effective Depth

$V_c = \left(\left(1.0\sqrt{30}\right)/6\right)200\ x\ 358$

V_c = 65361.55 N

10T-16

$$V_c = \left(\left(\lambda\sqrt{f'c}\right)/6\right)bw.d$$

Where

λ = 1.0

Fc = Concrete Strength

bw = Width

d = Effective Depth

$V_c = \left(\left(1.0\sqrt{30}\right)/6\right)200\ x\ 354$

V_c = 64631.26 N

4) With the proposal of choosing 10T-12 and 10T-16 reinforcement bars, the longitudinal steel ratio need to be maintained in accordance to the existing steel ratio for smaller beams so that it will be geometrically the same in terms of longitudinal steel ratio during the analysis of shear strength soon. For the under-reinforced is 1.3% and for the over-reinforced is 2.34%.

Thus, with the proposal of having 10T-12 and 10T-16, there is necessary to check the longitudinal steel ratio both of them concerning the existing steel ratio.

Therefore, OK! **Therefore, OK!**

10T – 12	**10T – 16**
As = $(10 \times \pi \times 12^2) / 4 = 1130.97$ As/bd = 0.013 0.013 = (1130.97) / (200 x 358) $0.013 \approx 0.016$ **Therefore, OK!**	As = $(10 \times \pi \times 16^2) / 4 = 2010.62$ As/bd = 0.0234 0.0234 = (2010.62) / (200 x 354) $0.0234 \approx 0.028$ **Therefore, OK!**

3.4 BEAM LAYOUT PREPARATION

For the first few weeks of the FYP II, only a minor job that has been done while waiting for the suitable contractors to take up the job of making formwork and prepare the concrete mixture. Once the contractor has been identified, a proper layout of beam has been prepared by using software, AutoCAD. In the drawing, all the detailing and dimensioning have been mentioned, thus assisting the contractors in preparing the formwork within time.

Figure 3-1 illustrates the sample of beam layout:

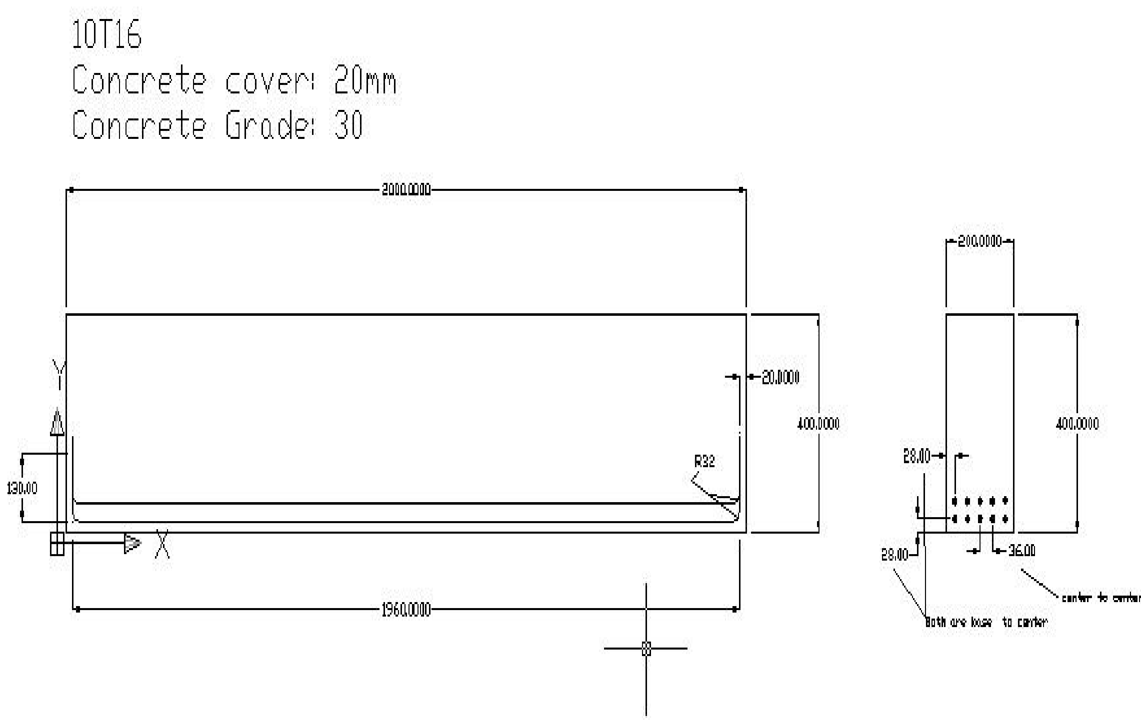


Figure 3-1: Sample of beam layout

3.5 FORMWORK PREPARATION

A way of communication between a client and contractor is via drawing. Thus, prior the work starts, the detailed layout of beams have been submitted and within few days the formworks are done. The formwork is very much significance in doing the concreting works as if it will be used as a temporary structure to support the fresh (i.e., uncured) concrete until it is already strong enough to support itself and applied loads on it. Below is the ongoing process of making the formworks:

Figure 3-2: Contractors are doing formwork

3.6 REINFORCEMENT-BAR BENDING

In the beam layout, it has been calculated and specified on how long the span of the reinforcement bar is and the spaces in between of those rebar (minimum 20 mm) as soon when the concrete mix is poured, it has to ensure that the aggregates must be able to pass through the gaps to fill the bottom part of the formwork.

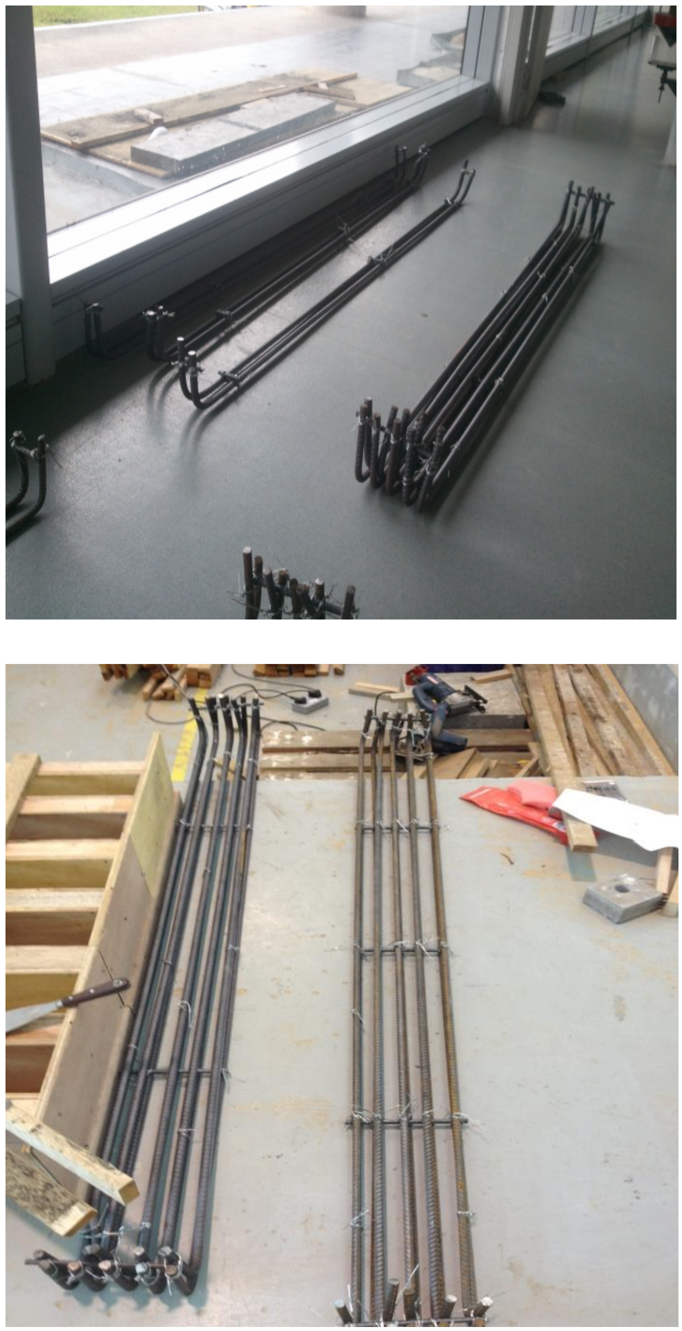


Figure 3-3: The reinforcement bars that have been bent

3.7 HOOK PREPARATION

The use of hooks is basically to assist the process of transferring the beam into the lab for testing soon as the beams are casted outside the laboratory. For each of the hooks, the length specified is about 75mm each and will cut down using cutter machine. The safety is really most emphasized in handling the machine. Once cut into pieces, the rebar are bent into U-shape with little extension at their ends by using a G-Clamp.

It will be placed on top of the beams after the concrete has been poured and it also marks a significance sign of the top and bottom of a beam.

Figure 3-4: The bar is cut into pieces with the same length

3.8 CASTING OF THE BEAMS

There are some minor works that need to be done prior to start casting the beams. Cubes of 20mm thickness need to be prepared and these will act as the supporting medium to the rebar as they will be put on side to side of the rebar and also below the rebar. So that the rebar is in suspension thus giving allowances for the concrete mix to get through it and form concrete cover for the beam. Other than that, the cube will ensure that the rebar will be in stationary and not affected by the concrete mix when they will poured into the formwork.

Figure 3-5: Cube is marked to the desired dimension

Figure 3-6: Cubes that have been marked

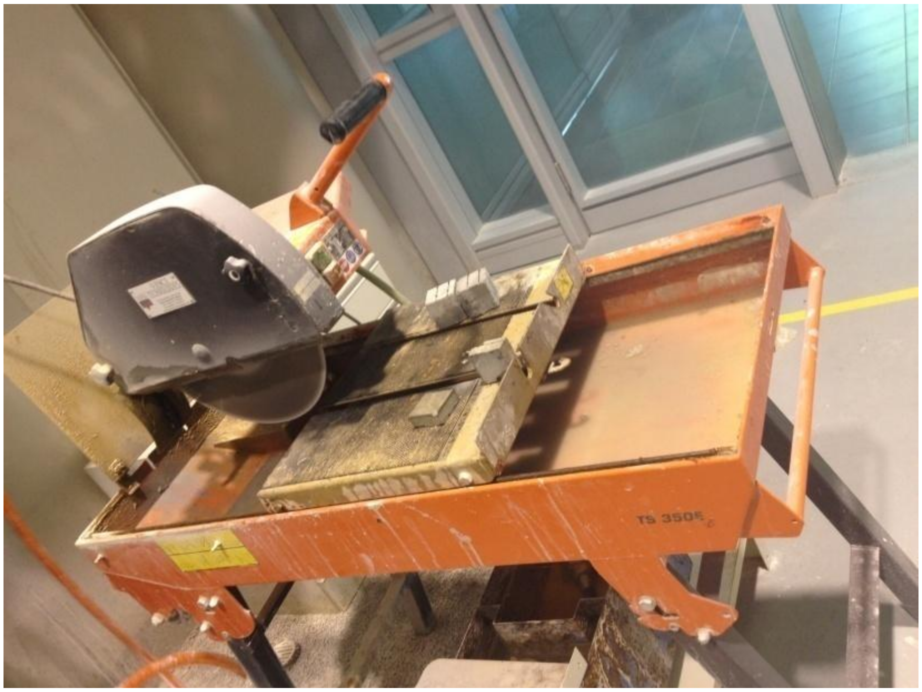


Figure 3-7: The cutter machine

It is a necessary for the formworks and the moulds to be applied with grease on their interior surfaces that will make contact with the concrete mix. This is done as to ensure those surfaces are not become adhesive to concrete soon.

Figure 3-8: Grease applied to the cylinder mould


Figure 3-9: Grease applied to the cube mould


Figure 3-10: Grease applied to the formwork

After all the interior surfaces of the beams and moulds have been thinly coated with the grease, the next step is to place the reinforcement bars accordingly into the respective formworks. The placement should be in correct way so that when the concrete mix is poured into the formwork, the rebar are in stationary position and do not move close enough to the side of formwork which eventually will cause the blockage of concrete mix to pass through it to the bottom. To have a safety measure about that, that is the function of the cube that have been cut earlier. They will be placed to some spots along the rebar and at underside as well just to give enough allowances to the concreting process later on.

Figure 3-11: The position of reinforcement bars inside of the formwork

The contractor that supply and provide the concrete mix services, ORKA comes to the site and pours the concrete into the formworks gradually and it is noted that the concrete mix needs to be compacted as sometimes it looks they have filled up the formworks somehow rather it is only water and there are still a lot of air spaces inside the formwork. Therefore, a vibrator or poker is needed to assist the compaction process. The vibrator is dip and submerged into the formwork and move alongside so that the concrete mix will settle down and get compacted. After it is confirmed that the concrete mix is fully compacted into the formwork, the surface finishing is done and then the hooks that have been prepared earlier will be placed on top of the finished surface. The formworks then are left hardened and on the next day the curing process will be done immediately.

Figure 3-12: The concrete mix is ready to be poured into the formworks

Figure 3-13: Vibrator or Poker

The looks at the surface of formwork and cube concrete after being compacted and finished.

Figure 3-14: The cubes filled with the concrete

Figure 3-15: The looks of the cylinder mould just before the finishing is done

3.9 CURING THE BEAMS AND CUBES

On the very next day, after the beams and concrete cubes and cylinders have been casted, they are ready to be cured. The need of curing is basically to prevent hydration of the concrete. There are two methods of curing that have been done, according to the availability of materials and their physical characteristics.

For the beams, as they are very heavy after being filled with the concrete mixture and with the availability of gunny sack, thus the practice is that, the beams are covered with the gunny sack and they will be sprayed with water to dampen the gunny sack and after they are sure being wet, the beams will be covered with plastic cover again. This curing process will be done on daily basis in order to ensure that the beams are always in wet condition.

Figure 3-16: Gunny sacks are sprayed with water to make sure they becomes damp

Figure 3-17: Beams have been covered with plastic cover

For the concrete inside of moulds, firstly they are demoulded and then they are marked with pens and immediately after that, they will be put inside of water tanks that purposely made for curing process in the laboratory. The mark will help the process of recognition soon as the water becomes cloudy and there are many other cubes that are cured at the same time in the tanks, so the marks will avoid the confusion later on.

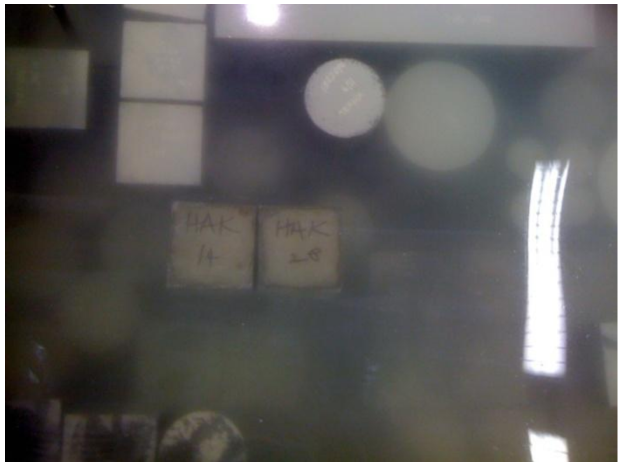


Figure 3-18: Concrete cubes inside water tanks for curing process

3.10 COMPRESSION TEST

Upon the concrete cubes attain their seven days curing; a compression test has been conducted to know their strengths. Three cubes have been taken out from the water tank and brought into the compression machine room. The results obtained will be discussed in the next chapter.

Figure 3-19: seven days curing cubes

Figure 3-20: Compression Test Machine

3.11 TENSILE STRENGTH TEST

The test is conducted to check on the tensile strength of the reinforcement bar. The steel reinforcing bar is used in concrete construction to provide tensile strength, complementing concrete's excellent compressive properties. Rebar also helps maintain structural integrity as concrete cracks from expansion and contraction cycles. Tensile testing of rebar is relatively straightforward. One pair of T10 and T16 rebar has been prepared for this and the results obtained will be further elaborated in the next chapter.

Figure 3-21: The rod before and after the testing

3.12 EXPERIMENTAL SETUP

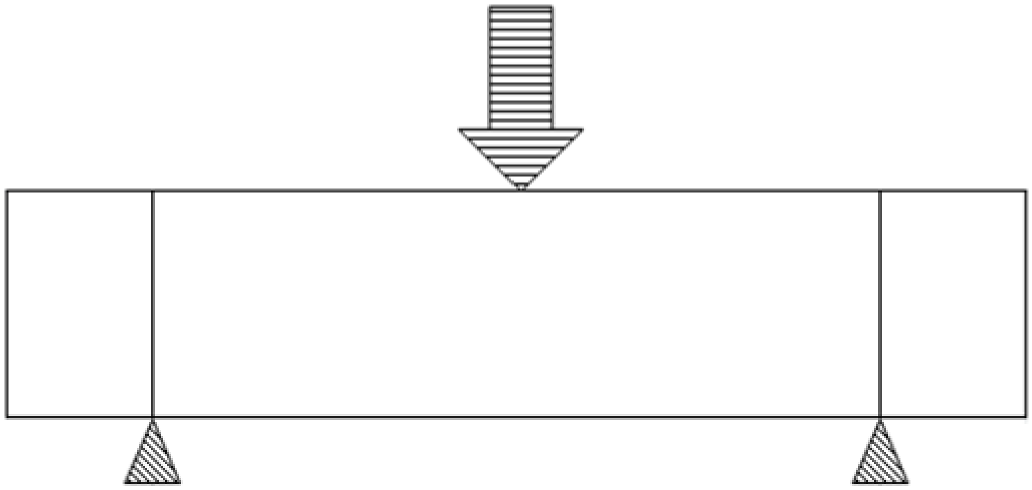


Figure 3-22: One-point loading experimental setup

The beam will be loaded with one-point loading until maximum deformation occurs. The point will be maintained at constant position for all beams to ensure a constant shear span ratio. The strain gauges will be attached at outside and inner side of the beam and diagonal deformation and cracking will be monitored visually.

3.13 GANTT CHART AND KEY MILESTONE

Final Year Project 1																	
	Action Items	1	2	3	4	5	6	7		8	9	10	11	12	13	14
1	Selection of Project Title															
2	Preliminary Research Work															
3	Submission of Extended Proposal								Mid Semester Break							
4	Proposal Defense															
5	Project Work															
6	Submission of Interim Draft															
7	Submission of Interim Report															

Final Year Project 2																
	Action Items	1	2	3	4	5	6	7		8	9	10	11	12	13	14
1	Project Work Continues															
2	Submission of Progress Report															
3	Project Work Continues															
4	Pre-EDX															
5	Submission of Draft Report								Mid Semester Break							
6	Submission of Dissertation															
7	Submission of Technical Paper															
8	Oral Presentation															
9	Submission of Hard Bound Dissertation															

	Process
	Suggested Milestone

3.14 TOOLS AND EQUIPMENT

To conduct the project, the tools and equipment that are available in laboratory will be utilized and used for work progress. Below are the list of tools and equipment that are necessary and needed for the project:

- ✓ Concrete mix of Grade 30
- ✓ Reinforcement Bars (10T-12 & 10T-16)
- ✓ Laboratory tools and equipment

CHAPTER 4

RESULTS AND DISCUSSIONS

4.0 FORMULA USED FOR TENSILE STRENGTH TEST

4.1 Shear Stress Formula

$$\boxed{\text{Stress, } \tau = \frac{F}{A}}$$

Where:

τ = the shear stress;

F = the force applied;

A = the cross-sectional area of material with area parallel to the applied force vector.

4.2 Strain Formula

$$\boxed{\text{Strain, } \varepsilon = \frac{Lchange}{L}}$$

4.3 Tabulated Data for Rebar T-16 Tensile Strength Tests.

AVAILABLE DATA	
Area, mm^2	201.06
Original Length, mm	670
Maximum Load, kN	141.392
Maximum Stress, kN/mm^2	0.703233

Table 4-1

AVAILABLE DATA	
Area, mm^2	201.06
Original Length, mm	670
Maximum Load, kN	137.625
Maximum Stress, kN/mm^2	0.68449717

Table 4-2

4.4 Graphs obtained from the tabulated data

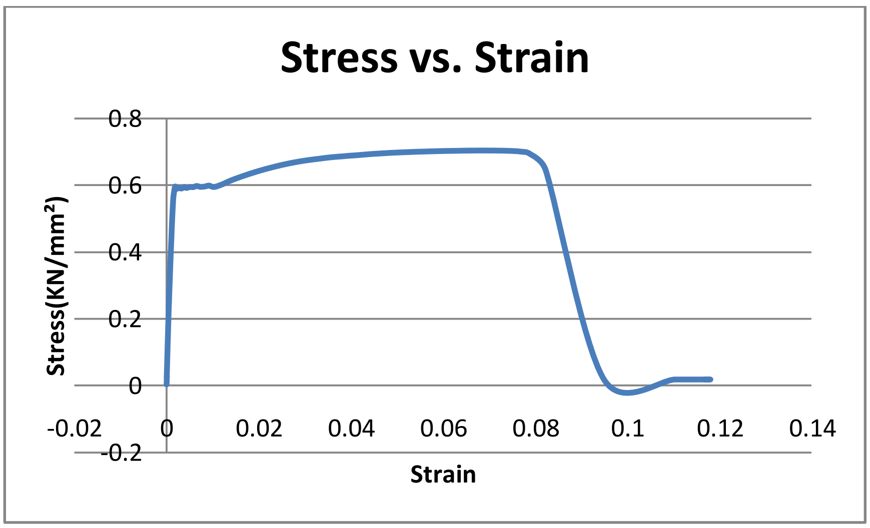


Figure 4-1: Graphs from Table 4-1

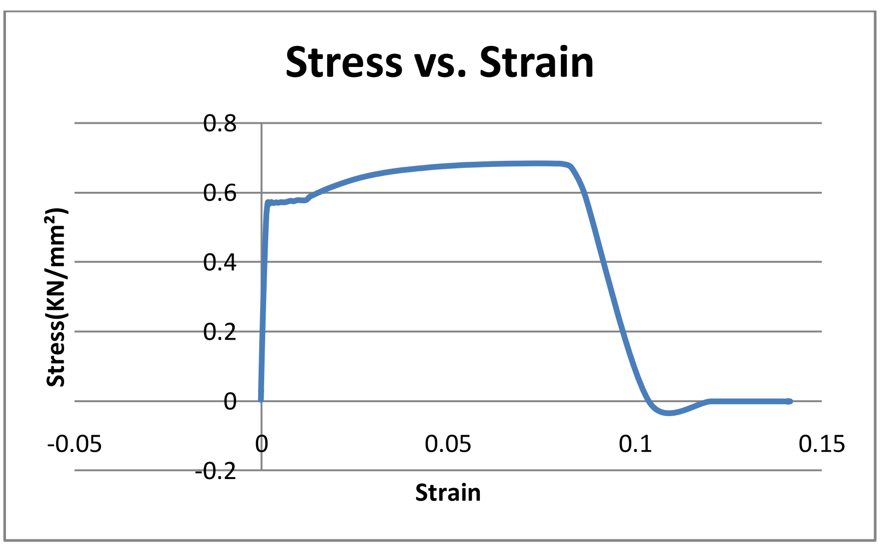


Figure 4-2: Graphs from Table 4-

4.5 Tabulated Data for Rebar T-12 Tensile Strength Tests.

AVAILABLE DATA	
Area, mm^2	113.1
Original Length, mm	670
Maximum Load, kN	73.358
Maximum Stress, kN/mm^2	0.648612

Table 4-3

AVAILABLE DATA	
Area, mm^2	113.1
Original Length, mm	670
Maximum Load, kN	74.474
Maximum Stress, kN/mm^2	0.658479

Table 4-4

4.6 Graphs obtained from the tabulated data

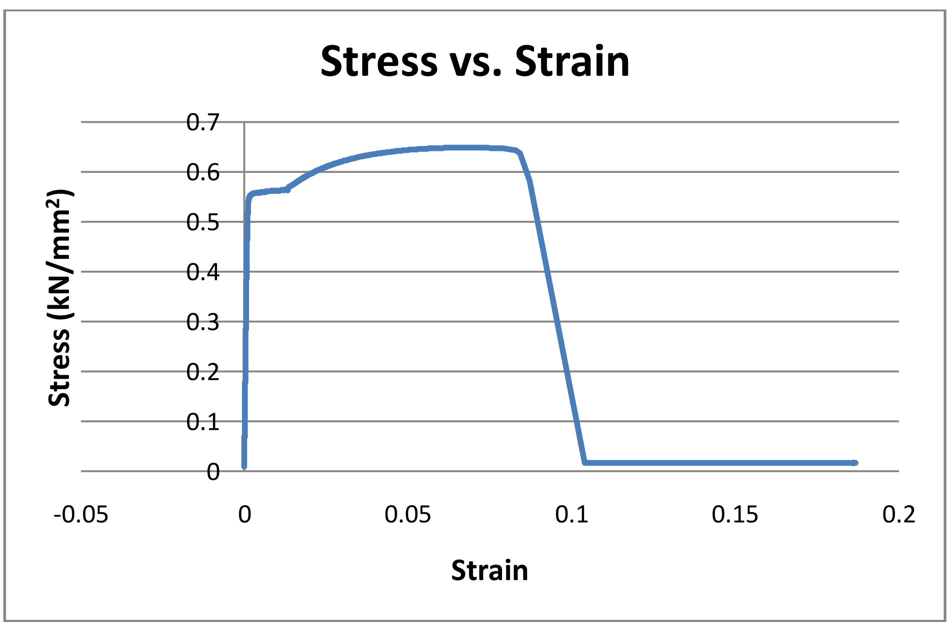


Figure 4-3: Graphs from Table 4-3

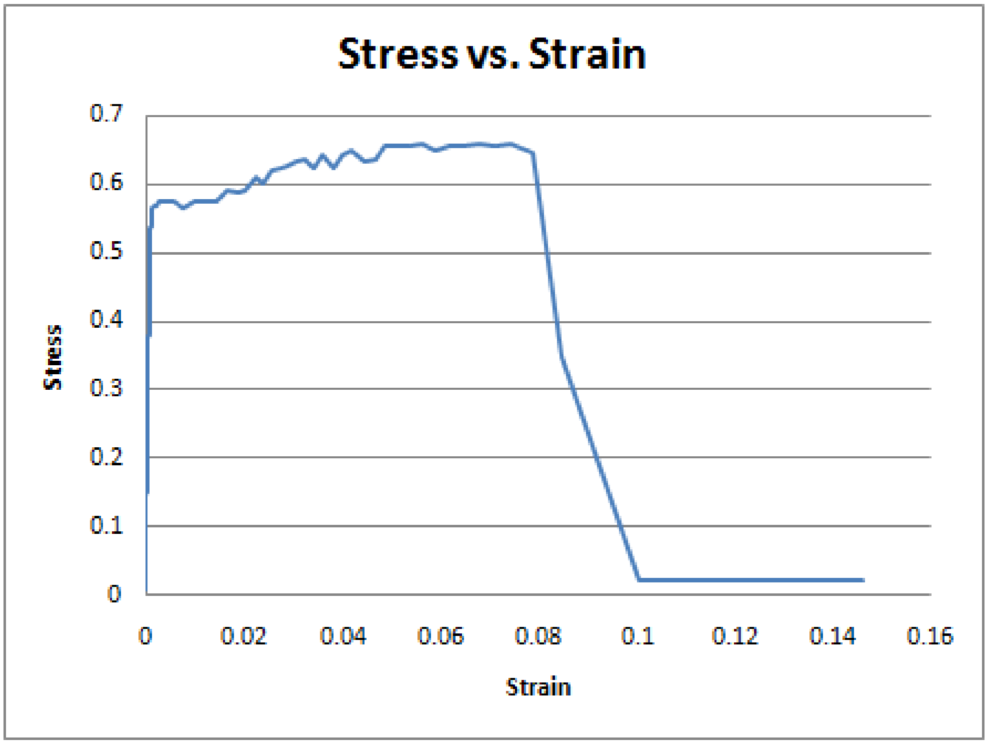


Figure 4-4: Graphs from Table 4-4

4.7 Compressive Strength Test

4.7.1 Seven-Days Cured Cubes Compressive Strength Test

As the cube concretes that has been cured reach it seven-days curing, thus can be used as an early indication of the official 28-day strength. Normally, for typical Portland cement concrete, the 7-day strength is about two-thirds or three-fourths of the 28-days strength.

Cube Dimension: 100 x 100 x 100			
7-Days Curing Cubes			
Units	**Max Load (kN)**	**Max. Stress (MPa)**	**Average (MPa)**
1	155.7	15.57	
2	176.0	17.60	
3	155.8	15.58	16.25
28-Days Curing Cubes			
1	323.2	32.32	
2	344.3	34.43	
3	361.6	36.16	
4	352.7	35.27	
5	328.2	32.82	
6	352.3	35.23	34.37

Table 5-1: Result for Cubes Compression Test

To conduct experiment, the specimen must be properly centered in the testing machine in order to avoid the asymmetric failure modes.

Based on the compressive strength test that has been conducted, it shows a positive result and gives and insightful view that the beams are already fit for testing.

4.8 Test Set-Up

Figure 6-1 shows a photograph of the test set-up in the Concrete Technology Laboratory at the Universiti Teknologi PETRONAS. The load-controlled actuator is used to apply the downward concentrated point load to the specimen with the rate of 0.1 kN/s. Both ends of the beam specimen rest upon a support assembly which made up of steel. The support distance from the end of the beam is 250 mm and the point load is located at mid-span of the beam. There are three LVDTs mounted under the test specimen to enable the determination of the beam's displacement. In order to monitor the strain in the longitudinal reinforcement, the demec digital mechanical strain gauge is used. Before carrying out the test, contact chips are glued to the concrete surface with adhesive.

Figure 6-1: Experimental Test Set-Up

Beam Specimen with 10T-12

AVAILABLE DATA			
Shear force at failure (kN)	-93.59		
Breaking Time (min)	42 mins 14 secs		
Total Deflection at failure (mm)	Left	Middle	Right
	2.35	6.036	4.865

Beam Specimen with 10T-16

AVAILABLE DATA			
Shear force at failure (kN)	-92.21		
Breaking Time (min)	40 mins 40 secs		
Total Deflection at failure (mm)	Left	Middle	Right
	6.69	3.675	3.645

4.9 Failure Mode

Both of these beams exhibit similar effect when the first shear crack occurs, it does not lead to total failure of the beams yet. The shear crack begins to appear on the side of the beam and only after that, they will fail and unlike for the smaller size beams where the shear failure is very sudden.

From the experiment conducted as well, the failure of the beams is observed to be as diagonal tension failure whereby the incline cracking occurs. The diagonal shear failure starts with the development of a few fine vertical cracks followed by the destruction of the bond between the reinforcing steel and surrounding concrete at the support.

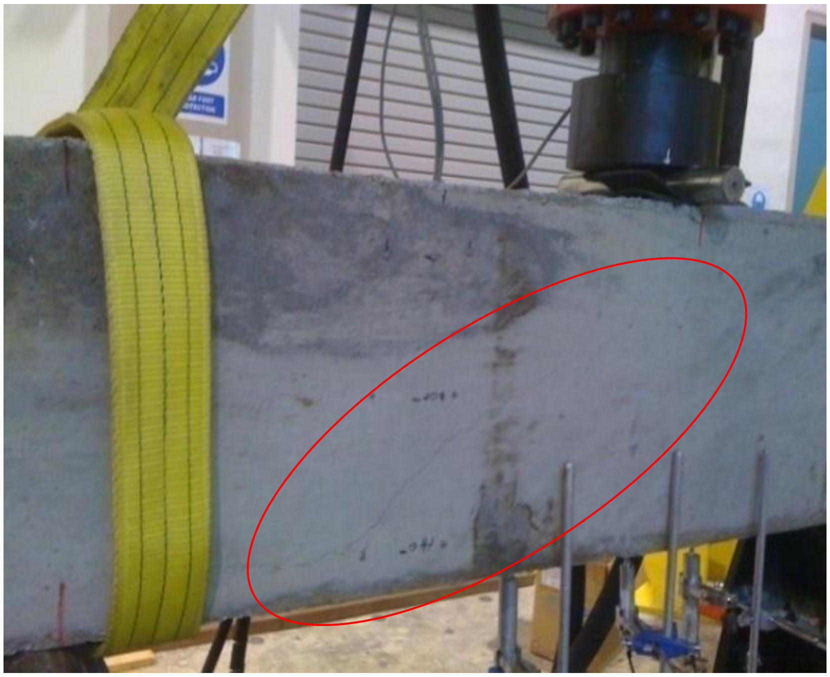


Figure 6-2: Shear cracking starts to emerge prior to total failure.

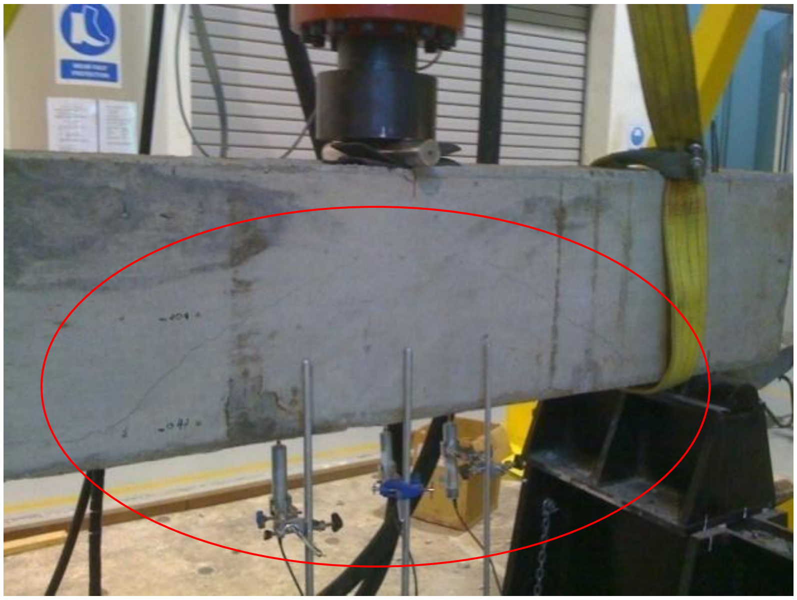


Figure 6-3: Shear cracking become more visible and start inclining toward point load.

Figure 6-4: Diagonal Shear Failure is observed.

Apart from that, from this experiment, the strain diagram cannot be figured out as if to measure the strain manually is basically physically impossible. The measurement to check on the strain has been done for every 5 kN of the load imposed by the actuator towards the beam, somehow the readings remain to be the same throughout the experiment and it is physically dangerous as well as some of the beams (e.g: smaller size beams) fail without prior notice and very sudden.

4.10 Results Comparison

1. For 10T-12

BS 110-1:1997		
Theoretical	Experimental	% Error
61.128 kN	93.59 kN	34.59
EN 1992-1-1, 2004 (EUROCODE)		
Theoretical	Experimental	% Error
81.846 kN	93.59 kN	12.54
ACI 318-08, 2007 (American Concrete Institution)		
Theoretical	Experimental	% Error
65.361 kN	93.59 kN	30.16

2. For 10T-16

BS 110-1:1997		
Theoretical	Experimental	% Error
87.51 kN	92.21 kN	5.1
EN 1992-1-1, 2004 (EUROCODE)		
Theoretical	Experimental	% Error
97.11 kN	92.21 kN	5.3
ACI 318-08, 2007 (American Concrete Institution)		
Theoretical	Experimental	% Error
64.631 kN	92.21 kN	29.9

3. Shear Force Comparison

Shear Force Comparison			
T-12		T-16	
A	B	A	B
93.59 kN	23.44 kN	92.21 kN	25.93 kN
% Difference = 74.95		% of Difference = 71.88	

Whereby

A denotes as bigger beam size = 400 x 200 x 2000 (in mm)

B denotes as smaller beam size = 200 x 100 x 2000 (in mm)

4.Shear Stress Comparison

By using formula of Shear Stress

V = v/bd

Whereby:

V = Shear Stress (kN/mm^2)

v = Shear Force (kN)

b = width of the beam (mm)

d = Effective Depth of the beam (mm)

Shear Force Comparison			
T-12		T-16	
A	B	A	B
93.59 kN	23.44 kN	92.21 kN	25.93 kN
1.31 kN/mm^2	1.35 kN/mm^2	1.30 kN/mm^2	1.51 kN/mm^2

Whereby

A denotes as bigger beam size = 400 x 200 x 2000 (in mm)

B denotes as smaller beam size = 200 x 100 x 2000 (in mm)

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION

As a conclusion, based on the experiment that has been conducted, some of the objectives are met and there is one objective which is not achieved which is to get the strain diagram in the shear span of the beam.

The size effect is clearly demonstrated where the shear stress decreases with the increase in depth. (See Appendix)

From the data obtained, it shows that the beam specimens with the 10T-12 and 10T-16 have almost the same shear force at failure, thus showing that the size of the reinforcement bar does not give a significant effect towards the shear failure of the beam.

From this experiment also, it is proved that the mode of failure of the beam is in diagonal failure. The slight difference between the smaller size beams (200 mm x 100 mm x 2000 mm) and deeper beams (400 mm x 200 mm x 2000 mm) is that the failure of the small beams is very sudden without having any notice prior to that meanwhile for the deeper beams, the incline cracking start to emerge starting from the supports towards the point load and then fail diagonally.

5.2 RECOMMENDATIONS

a. In order to get the better result for strain, a new way of measuring strain may be introduced such as by using a more efficient device so that it will not depends on manual measurement as of now which is dangerous and not practical.

b. For us to conduct the experiment, safety and awareness must be prioritized as some of the beams just have a sudden failure and some of the debris may scatter to nearby area. All the tools and apparatus must be handled with extra careful.

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APPENDICES

APPENDIX 1: *MATERIALS PROPERTIES*

APPENDIX 1: MATERIAL PROPERTIES

Ready Mix Concrete Grade 30 Aggregate Grading

OKA	QUALITY CONTROL MECHANICAL ANALYSIS	QAM/QC/008 dd : 30/09/05 rev : 1

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Type of Material : Single-sized 20 mm crushed granite

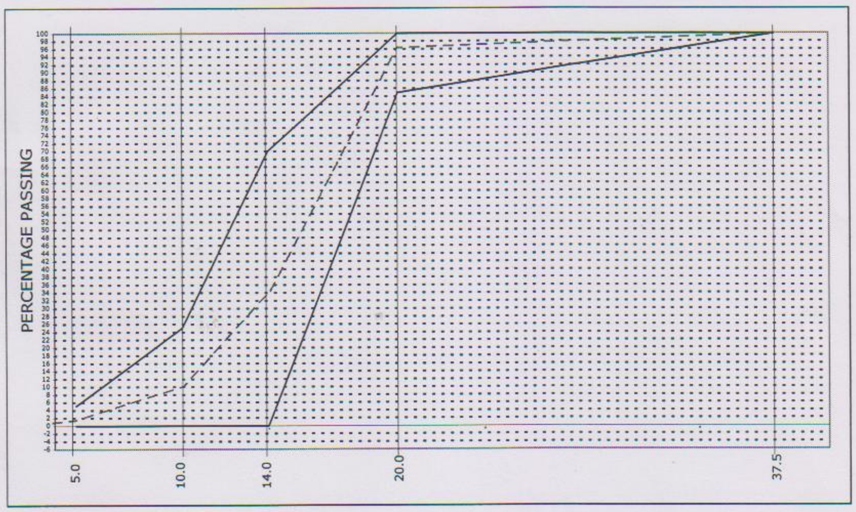
Location : Batu Gajah

Sample No : 13406 (AGF 688) Simfoni Saujana

Sampling Date : 24/10/2012

Testing Date : 24/10/2012

SIEVE SIZE	WT. RETAINED (gm)	% RETAINED	% PASSING	SPEC. LIMIT MS 29	SPEC. LIMIT Others	SPEC. LIMIT Remarks
37.5 mm	0	0.00	100	100		
20.0 mm	53	3.69	96.31	85 - 100		
14.0 mm	900	62.59	33.73	0 - 70		
10.0 mm	339	23.57	10.15	0 - 25		
5.0 mm	125	8.69	1.46	0 - 5		
Pan	21	1.5	0.00			
	1438	100				
Silt Content						
Visual Inspection	Pass					



Remarks :

Tick where appropriate ACCEPT REJECT

QC Supervisor

QM / FM

DIUJI DAN DISAHKAN
OKA CONCRETE INDUSTRIES