THE SHEAR ASSESSMENT OF CONCRETE BEAMS WITH A HONEYCOMBED ZONE PRESENT IN THE HIGH SHEAR REGION

by

WAHID OMAR

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ABSTRACT

A honeycombed zone in a concrete structure is among the forms of defects that can be found in concrete structures. In this study, the effects on the shear capacity of a concrete beam are investigated when a honeycombed zone is present in the high shear region. The honeycombed concrete was simulated using an uncompacted no-fines concrete mix. Tests were carried out on rectangular beams with a honeycombed zone introduced at various locations within the high shear region of the beam. The honeycombed zone was square with the size of about a third of the effective depth of the beam. Two strengths of normal concrete were investigated, with average strengths of about 50 N/mm^2 and 35 N/mm^2. The average strengths of the honeycombed concrete were about 23 N/mm^2 and 12 N/mm^2. A total of 56 beam specimens with a shear span ratio of 2.0 were tested. Six beam specimens with a shear span ratio of 3.5 were also examined.

The study found that, in a beam with a honeycombed zone, a diagonal crack could form early and the shear capacity of a beam could be reduced. The severity of these effects varies with the location of the honeycombed zone. Comparisons with BS 8110 and BD 44/95 show that, depending on the locations and also on how the strength of the honeycombed concrete is treated, assessment using both methods can be either unsafe or very conservative. Recommendations are made for a consistently conservative assessment.

Modifications were proposed to an existing plastic analysis method to take into account the strength of the honeycombed concrete. Condition factors derived from the test data were also proposed to be applied to the plastic analysis in assessing beams with a honeycombed zone.

A honeycombed zone simulating a construction joint in a beam was also investigated analytically and compared with the data from 4 tests.

This work is specially dedicated to Emak and Abah, Laili......and to all my brothers and sisters..

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NOTATION

A_s : the area of the longitudinal reinforcement

A_{sv} : the area of shear reinforcement

a : shear span

b : width of beam

d : effective depth

d_a : maximum size of aggregate

h : height of beam

E_s : Elastic modulus of longitudinal reinforcement

E_{sv} : Elastic modulus of shear reinforcement

f_c : compressive cylinder strength of normal concrete

f_{cav} : 'weighted average' strength of concrete

f_{ch} : compressive cylinder strength of honeycombed concrete

f_{cu} : compressive cube strength of normal concrete

f_{cuh} : compressive cube strength of honeycombed concrete

f_y : the yield strength of bottom longitudinal reinforcement

f_t : tensile strength of concrete

f_{yv} : the yield strength of shear reinforcement

l_c : length of plastic mechanism in normal concrete

l_h : length of plastic mechanism in honeycombed concrete

l_j : length of plastic mechanism in joint

l_m : length of plastic mechanism

M : bending moment

s_v : spacing of shear reinforcement

V : shear force

V_c : diagonal cracking shear force

V_u : ultimate shear force

W_I : internal work done by the plastic yield line

u : the displacement of the yield line

α : the angle of the yield line displacement to the failure mechanism

β : angle of inclination of the failure mechanism to the horizontal axis

γ : angle of plastic mechanism to the vertical axis

θ : angle of displacement to plastic mechanism in joint

φ : angle of internal friction

ν : the effectiveness factor

ν_c : the effectiveness factor for normal concrete

ν_h : the effectiveness factor for honeycombed concrete

ρ : percentage of longitudinal reinforcement

τ : average shear stress

Φ : degree of longitudinal reinforcement

ψ : degree of shear reinforcement

TABLE OF CONTENTS

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

Assessment and monitoring of existing structures have become major tasks for structural engineers. Today, structural engineers probably spend more time carrying out assessment works and monitoring existing structures than being involved in designing and developing new structures. It should be emphasised that repairs and rehabilitation of existing structures are usually very costly. A realistic but at the same time a safe assessment is therefore very important. One way to achieve this is to provide comprehensive guidance to the assessing engineers. This requires more research work to be carried out.

There has been a lot of work to produce guidance for assessment work. The guidance for structural inspection and condition survey has been systematically formulated and well established among assessing engineers. In the UK, the department of transport produced a series of assessment documents including the latest edition of BD 44/95(1) for concrete bridges. In order to prepare a comprehensive assessment document, a lot of research has been carried out to study various forms of concrete deterioration and their effects on structural capacity. A number of research works have also been carried out to study the effect of defects in structures such as inadequate reinforcement anchorage.

Concrete structures are assessed for various reasons. In the early years the needs for the assessment arose mainly because of the deterioration of concrete due to aging and environmental effects, the need to extend the service life of structures, to upgrade structures for an increase in loading and a change of use. Those are traditionally among the main causes which require structures to be assessed. As the structural assessment develops and expands, it has later been found that often structures with defects in concrete due to design and/or construction faults also exist.

Assessing engineers must be prepared to face various forms of concrete defects in existing structures. They may encounter structures with non-uniform concrete qualities, low quality concrete, and structures containing a honeycombed or a weak concrete zone. In a survey carried out in the UK on 200 concrete bridges(2), 1% of the structures surveyed contained honeycombed concrete. It has been reported(3) that a concrete strength of as low as 5 N/mm^2 was found in an existing concrete structure.

Reports of research on the effects of a honeycombed zone on structural capacity is, however, almost non-existent. When there is any, it is always in the form of adhoc field investigations and the work is not well reported. Probably many thought that the honeycombed concrete can always be easily detected at the construction stage and, the associated problems can be rectified immediately. In today's world, where engineers are involved in a massive upgrading and rehabilitation program of existing structures, the possibilities that the assessment engineer may have to assess the structural capacity of honeycombed structures cannot simply be ruled out. Therefore it is important that research work be carried out to study this problem.

In this study, experimental and analytical investigations were carried out to study the effects of a honeycombed zone present in the high shear region of a concrete beam on shear behaviour and shear capacity of the beam. Currently neither experimental data nor a rational analytical method are readily available to be used in assessing a beam with a honeycombed zone.

1.2 HONEYCOMBED ZONE IN CONCRETE STRUCTURES

A honeycombed zone in a concrete beam is not an uncommon occurrence. Inadequate compaction, low workability concrete, congested reinforcement, leakage of the formwork, unsystematic sequence of pouring due to any reason are among the factors that can cause the formation of a honeycombed zone. Those factors are associated with poor concreting practice.

A zone of honeycombed concrete may form anywhere in a concrete beam, with higher probabilities in certain places like at construction joints and at the area of highly congested reinforcement. Honeycombed concrete is associated with voids. Kaplan(4) showed that with only 5% voids, concrete may lose 30% of its compressive strength. Thus, the presence of a honeycombed zone in a concrete beam can significantly affect the strength of the beam.

A honeycombed zone present in the high shear region of a concrete beam may have an adverse effect on shear capacity of the beam. This is especially so in a beam with a low shear span ratio. This can be understood by examining the shear behaviour in a concrete beam.

1.3 SHEAR IN CONCRETE BEAM

The transfer of shear force in a concrete beam involves complex mechanisms. Shear can be transmitted from one plane to another in various ways. A diagonal crack forms when shear stress interacts with tensile and compressive stresses to produce principal tensile stresses in excess of the tensile strength of the concrete. Once an inclined crack is formed, shear can be transferred through aggregate-interlock action, concrete compressive struts and also through dowel action. Under certain conditions, a beam with inclined cracks can act as a tied-arch structure and shear is resisted by the anchorage of the reinforcement and concrete at the crown of the arch. For a short shear span beam, shear force can be predominantly transferred to the support by a concrete compressive strut.

From the above description, it is highly possible that if a honeycombed zone is present in the shear zone, it may affect the shear transfer mechanism. It may alter the distribution of stresses within the shear zone. It may accelerate the formation of the diagonal cracking and result in early failure of the beam. Its presence within the potential compressive strut or within the area of concrete arch may reduce the ultimate shear capacity of a concrete beam.

With regard to the structural assessment often it has been found that inadequate shear strength caused a structure to fail the assessment(5). This indicates that shear is the most critical strength parameter for a concrete beam. The brittle nature of shear failure implies that failure can often occur without warning. These points emphasise the need for a study to be done on this problem.

1.4 SHEAR ANALYSIS AND ASSESSMENT

For decades the analysis of shear in concrete beams relied on empirical solutions. This happened because the mechanisms of shear transfer are complex and various parameters contribute to the mechanisms and consequently to the shear strength of a beam. In the last 30 years efforts have been made to approach the shear problem rationally. Plastic theory is one of the approaches that seems to provide a rational solution and gives a good prediction of shear behaviour.

Currently there is no general analytical solution which is readily available and which can be applicable to all ranges of concrete beams under all conditions. For example, engineers having to assess the shear capacity of honeycombed beams will find no analytical solution readily available to be used. The current practice of assessment of shear in many circumstances also relies on procedures proposed for design rather than assessment.

It is unclear how to apply the design rules which are largely empirical to the assessment of structures with details and/or material properties which vary from those assumed in the derivation of the empirical formulae. For example if a zone of honeycombed concrete is found in a beam, the question is how to apply the empirical solution to the problem?

The easy solution, as often adopted, is to use the lowest strength of concrete obtained from the site investigation and apply it into the recommended shear expression. Ideally the distribution of concrete strength would be obtained from cores taken from a number of locations. However, in general, sufficient cores are not available or cannot be taken for practical reasons. Hence, it is necessary to augment the available core data with non-destructive techniques such as a rebound hammer and ultrasonic pulse velocity measurement. However, such techniques should only be used quantitatively when their results have been site calibrated against core data.

However, using such a strength in the shear expression can result in a risky unsafe assessment or it can also result in a very conservative assessment. To date there are no test data against which a measure of the safety of such approaches can be made. The location of the honeycombed zone may be in such a position that it has no effect at all on the shear capacity of the beam. In that case the assessment will be conservative. It may also happen that the honeycombed zone is located at a position at which it can cause a significant reduction in the shear capacity of the beam. In this situation, the procedure adopted may be insufficient to give a safe assessment.

It is thus difficult and at times can be very risky because so far no work to quantify the shear capacity of such beams has been reported. It is unknown to what extent the size of the honeycombed zone can affect the shear capacity of a beam. It is also unknown where are the locations at which a honeycombed zone can pose a critical adverse effect on shear capacity of a concrete beam. Another unknown parameter is at which level of strength the honeycombed concrete can be a problem in reducing the shear capacity of a concrete beam.

Probably the assessment codes may have allowed for various defects which are normally encountered in a concrete beam. However it is unknown whether such allowance can be extended to a beam with a honeycombed zone without actually comparing it with test data.

After many years of research there are various analytical solutions available for shear. However since shear is a complex phenomenon, and it can be influenced by several parameters, no single solution can be claimed as truly rational and can be used in a rational manner to treat shear in a concrete beam with a honeycombed zone. Probably some of the analytical methods can be extended and be applicable to predict shear capacity of a honeycombed beam. A plastic theory for predicting shear, for example, may enable defects and deterioration along with other structural information to be taken into account in a logical manner. Tests are required for verification.

Hence, there is a need to extend the current rational shear strength prediction methods to cover defects and deterioration in concrete and to provide test data against which to check the predictions.

1.5 RESEARCH SIGNIFICANCE

In particular for a honeycombed beam there is a need for:

1. Providing experimental data of the effect of a honeycombed zone on the shear capacity of a reinforced concrete beam. The test data will be of great benefit especially for guidance in structural assessment. Currently there has been no report of such work in which the method of assessment as mentioned above is quantified against test data.

2. Development of an analytical method in order to assess the shear capacity of beams with a honeycombed zone or non-uniform concrete in the high shear region. In this study the existing plasticity analysis for predicting shear capacity of a concrete beam is utilised and, with the proposed modification based on test results, it can be a rational assessment tool to evaluate shear in a honeycombed beam.

1.6 SCOPE OF RESEARCH

In general the current investigations comprise experimental and analytical works. The scope of the experimental work of this research are as follow:

1. Simulation of honeycombed concrete: An uncompacted no-fines mix was used to simulate honeycombed concrete. The method for the inclusion of a honeycombed zone into a beam was also examined. A cast insitu honeycombed zone using a specially designed mould proved to be more suitable compared to a precast honeycombed zone. This is due to the fact that a cast insitu method creates a chemical bond between the honeycombed and the normal concrete.

2. Experimental investigations on the effect of a honeycombed zone on shear behaviour and shear capacity of a concrete beam were carried out. All beams were rectangular and designed to fail in shear. A honeycombed zone was deliberately introduced at a number of locations within the high shear region of a beam. The major variable was

the location of the honeycombed zone. Two levels of strength of normal concrete and honeycombed concrete respectively were investigated. The effect of the size of a honeycombed zone was also included in the tests. The major portion of the work was devoted to study the effect on a beam with a short shear span, with a shear span ratio of 2.0, and without shear reinforcement. Comparisons were made with beams with a shear span ratio of 3.5. Beams were subjected to a point load.

3. Honeycombed beams with shear reinforcement were also examined. Four controls and four honeycombed specimens were tested.

4. A comparison was also made between honeycombed beams and beams with a void instead of a honeycombed zone. This was examined because it was initially thought that the mode of shear behaviour in a beam with a void could be compatible to a beam with a honeycombed zone, and if this were so, probably the analytical method developed for the former could be extended and applied to the latter.

5. Beams with a construction joint were also tested. Honeycombed concrete was placed in a beam at an angle, simulating a construction joint. Four such specimens were examined.

6. Control specimens of normal and honeycombed concrete were also prepared and comprised cubes, cylinders and prisms. Since the plastic analysis was adopted as the analytical tool which was to be extended, relationships between cube and cylinder compressive strength had to be determined from the test of the control specimens.

7. The use of ultrasonic pulse velocity measurement in detecting the zone of honeycombed concrete was also carried out.

The analytical work includes:

1. The extension of the plastic analysis to evaluate shear capacities of beams with a construction joint. Analytical investigations were carried out and the summary of the modes of possible failure are presented. The variables were the strength of the honeycombed concrete, the angle of inclination of the joint, the amount of longitudinal reinforcement and the shear span ratio.

2. An elastic finite element analysis was used to check the stress distribution within the shear region due to the presence of a honeycombed zone.

3. A plastic analysis was chosen as an analytical tool for shear in a honeycombed concrete beam. Comparisons were made with the test data and modifications required to the existing theory were worked out.

4. Test results were compared with BS 8110 and BD 44/95 methods of prediction. The predictions were made using the strength of the normal concrete and also the strength of the honeycombed concrete. The diagonal cracking shear and the ultimate loads were compared.

1.7 SCOPE OF THESIS

A literature review giving the background of shear work in relation to concrete beams is presented in **Chapter 2**. **Chapter 3** describes the experimental work. Experimental results and the analysis of the results are presented in **Chapter 4**. **Chapter 5** presents the analytical work which includes the extension of the plastic analysis for application in a honeycombed beam and analytical study of beams with a construction joint. The finite element analysis and comparisons between the plastic analysis and test results are reported in **Chapter 6**. The comparisons between the test results and the predictions of BS 8110 and BD 44/95 are also presented in **Chapter 6**. The conclusions and the recommendations for future work are presented in **Chapter 7**.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The complex nature of shear in concrete members makes it an interesting area of research. In structural engineering, the subject has been one of the most widely researched for about a century. The extensive research in the past has probably covered in great detail if not all but almost every single aspect of shear behaviour and strength in concrete beams. The influence and the role of each single parameter and the relationships between the parameters that are involved in the mechanisms of shear transfer have been thoroughly studied through numerous experimental works. A considerable number of shear analytical models have been studied and, consequently, various analytical methods such as a highly empirical approach, fracture mechanics, finite element and plasticity theory approach have been proposed. In spite of this there is still no consensus regarding an analytical model for the shear resistance of concrete beams.

With regard to the present study on honeycombed concrete beams, no report of work of a similar nature has so far been found in the literature. It is strongly believed that this is the first attempt in which the effect of the presence of a honeycombed zone in the high shear region of a concrete beam has been studied. In order to carry out the present study, a general background of research work on shear in concrete beams needed to be reviewed. Some relevant information, such as the typical shear behaviour and the mode of failure, the mechanism of shear transfer and parameters affecting the shear strength of concrete beams that have been gathered from previous research are summarised in this review. A review on the previous studies of analytical solutions is also presented.

The review is divided into three main sections. First, a brief general review of shear in concrete beams is presented in **Section 2.2**. The discussion in this section includes the description of the typical shear behaviour of concrete beams, the mechanisms of shear transfer and the factors affecting the shear strength of concrete beams. The development of the analytical methods is briefly reviewed and discussed in **Section 2.3**. **Section 2.4** discusses some aspects of the assessment of existing structures. Some detailed aspects of the analytical methods that are important to the current study are also discussed accordingly. The summary of the literature review is presented in **Section 2.5**.

2.2 SHEAR IN CONCRETE BEAMS

2.2.1 General

As a result of the extensive studies carried out in the past, researchers have in principle been able to identify a general pattern of shear behaviour in various types of concrete beams. Researchers have also recognised the mechanisms of shear transfer and the various components involved in the transfer of shear stress. Various factors affecting the shear behaviour and strength have also been recognised. In this review these aspects are discussed in general as a summary of information from the previous research.

2.2.2 The Typical Behaviour of Shear in Concrete Beams

Researchers have long been agreed(6),(7),(8),(9) that the fundamental character of shear behaviour in a concrete beam is the formation of a 'diagonal tension crack' in the shear zone. The crack which is also called an 'inclined crack' is formed when a shear stress interacts with the tensile and compressive stresses to produce principal compressive and tensile stresses. The latter cause a diagonal or an inclined crack to be formed since concrete is very much weaker in tension than in compression. This crack may be developed independently or as an extension of the previously developed flexural cracks. The independently formed diagonal tension crack is always referred to as 'web-shear crack' and the second type is often identified as 'flexural-shear crack'. The first type of shear crack can generally be observed in prestressed concrete members or in ordinary reinforced concrete members with a short shear span. The second type is typical for reinforced concrete members having a normal range of shear span of the order of two to four times the effective depth.

The formation of the diagonal tension crack produces a complex redistribution of the internal stresses, and beams, depending on their properties, the nature of loading, shear span, and many other parameters, may behave in various ways leading to the failure of the member. There are cases in which diagonal cracking is immediately followed by a member failure and in other cases, the cracks stabilise and substantially more shear force can be applied before the beam ultimately fails. In some instances, secondary cracks often result from splitting forces developed by the deformed bars when slip between concrete and steel reinforcement occurs, or from dowel action forces in the longitudinal bars transferring shear across the crack.

Although shear behaviour can be influenced by various parameters, researchers(6),(7),(8) found that the manner in which inclined cracks develop and grow and the mode of failure that subsequently develops is strongly affected by the relative magnitudes of the shearing stress, v and longitudinal flexural stress, f_x. It has been shown(8) that, if all other variables are kept constant, both parameters can be represented in terms of the ratio of shear span(a) to the effective depth(d) of the member, a/d. This is generally applied to rectangular beams loaded with concentrated loads. However, the amount of the shear reinforcement present in the member is also an important parameter and can influence the mode of the behaviour(6).

Researchers have observed that the shear behaviour of beams prior to the formation of diagonal cracking, and the subsequent redistribution of internal stresses and the mode of behaviour up to the failure can generally be classified according to *a/d* values. It must be remembered however, that it is often found that shear behaviour can vary even for beams with nominally identical properties. This has been shown in many tests. Krefeld(10) for example, observed this phenomena.

This phenomenon occurs due to the fact that shear behaviour and strength are influenced by many other factors. Bresler(8) once stated that to deal with all the factors would be a 'monumental task' and listed some of them. They include the proportions and shape of the beam, the structural restraints and the interaction of the beam with other components in the system, the amount and arrangement of tensile, compressive and transverse reinforcement, the load distribution and loading history, the properties of the concrete and steel, the placement of concrete and curing, and the environmental history.

R.Taylor(11) classified diagonal tension failures into two modes: confined-shear and shear-compression, refer to **Figure 2.1**. A confined-shear failure mode can occur in beams with a shear span ratio of less than 2.0. Inclined cracks occur independently, initiated by the principal tensions across the line joining the load and the reaction, in a manner similar to an indirect tension test. The effect of flexure is negligible.

Beams with shear reinforcement may fail by a crushing failure at the reaction and/or at the arch and accompanied by crushing along the inclined crack. The failure load is generally greater or much greater than the diagonal cracking load, because propagation of the crack is restrained or confined by the stresses adjacent to the load and reaction, forcing the failure crack to propagate at a steeper angle than 45^{o}. Beams without shear reinforcement and shear span ratios close to, or greater than, 1.0 can fail by diagonal tension with the lower face of the diagonal crack separating away from the upper face, finally causing the compression zone at the 'arch' (i.e. adjacent to the point load) to become so shallow that it crushes.

The shear-compression failure usually occurs in beams with a shear span ratio more than 2.0. In this range of shear span ratio an inclined crack is generally the result of a flexural crack which extends vertically from the tension surface of the beam to just above the reinforcement and then becomes inclined and curved and propagates towards the compression face. With further increase in load the diagonal tension crack will continue to propagate at the upper end, first becoming shallower and then eventually becoming steeper again to force the compression zone to become so shallow that it crushes. For beams with a longer shear span, with a shear span ratio greater than about 2.5, the failure may occur almost immediately after the formation of the diagonal crack. Shear-compression failures are a result of combined flexure and shear.

An alternative failure mode can occur if the crack propagates backwards along the longitudinal reinforcement and cause a loss of bond. At this stage the beam acts as a tie and arch, and further increase of load results in the anchorage failure of the longitudinal reinforcement. This type of failure where crushing of the concrete at the top of the arch is merely a secondary failure mode is called a shear-tension failure and should always be avoided in practice with nominal links and adequate end anchorage.

It has been observed that(8), at a shear span ratio of about 6.0, the effect of shear in a beam will be negligible.

2.2.3 The Mechanisms of Shear Transfer

Observations made from numerous experimental works in the past have shown that shear is transmitted from one plane to another in various ways in reinforced concrete beams and the behaviour, including the failure modes, depends on the method of shear transmission. It is a highly inconsistent phenomenon and many researchers such as Kani(12) noted that the shear stress at failure is far from being constant even in the case of the same concrete, cross section and reinforcement. In 1973, the Joint ASCE-ACI Task Committee 426 on Shear and Diagonal Tension(6), despite reviewing numerous works on shear, only presented a 'tentative' evaluation of the contribution of each shear carrying components.

Five main types of shear transfer in concrete beams that have been identified are(6): (i) shear stress in the uncracked concrete; (ii) interface shear transfer; (iii) dowel action; (iv) arch action; and (v) shear reinforcement. These mechanisms occur to widely varying extents in various types of structural elements and depend on various parameters. Some of them are already mentioned in **Section 2.2.2**. H.Taylor(13) quantified that for a typical reinforced concrete beam a shear force is carried in the following approximate proportions: compression zone; 20-40%, dowel action, 15-25%; and aggregate interlock, 35-50%.

In uncracked members and in the uncracked regions of cracked members, shear stresses interact with tensile and compressive stresses producing principal stresses which cause the inclined diagonal cracking or a crushing failure. Once the inclined cracking is formed, shear can be transferred through the action of the aggregate interlock, concrete compressive zone and through the dowel action of the longitudinal reinforcement. In the situation where the beam acts as a tied-arch structure, load is carried by a mechanism which is dependent on the anchorage of the reinforcement and the concrete resistance at the crown of the arch.

The shear reinforcement carries part of the applied shear, but it is only significantly functional once inclined cracks have formed(6),(7). Its presence restricts the growth of a diagonal crack and reduces the penetration of the diagonal crack into the compression zone, thus contributing to the capacity of the member by increasing or maintaining the shear transferred by interface shear transfer, dowel action and arch action, in addition to resisting some of the shear force directly

2.2.4 Factors Affecting Shear Behaviour and Strength of Concrete Beams

Since the early years, researchers had realised that there are various factors that can influence the shear in concrete beams. Talbot as early as 1909(7) found that the amount of longitudinal reinforcement and the span to depth ratio played important roles in the shear strength of concrete beams without shear reinforcement. Moretto(14) and Clark(15) in their studies on shear in the 40's and 50's also recognised the effects of the two parameters apart from concrete strength. Researchers in the following years have conducted various experimental works and studied in detail the contribution of each factor. The joint ASCE-ACI Task Committee 426(6) summarises most of the work and discusses the effects of each factor in detail. Only a brief review is presented here.

Studies have shown that shear behaviour and strength can be affected by the cross-section of the beam. Beams with different size and shape may exhibit different strength in shear. Tests by Taylor(16), however, have shown much less size effect if the size of the coarse aggregate is changed in the same proportion as the beam size. Statistical analysis of test results(17) shows that the beam size has no significant effect on the ultimate strength of beams with shear reinforcement. For rectangular beams, statistical studies of a large amount of data(18) showed no significant variation of strength if the breadth(b) to depth(d) ratio, b/d is in the range of 0.25 to 1.0.

The longitudinal reinforcement details also have significant effects on shear. The term representing the amount of longitudinal reinforcement appears in all expressions of shear strength reviewed, including expressions given in BS 8110(19), BS 5400(20) and ACI codes(6) as well as in the plasticity theory. Tests by Kani(21) and Rajagopalan and Ferguson(22) indicated that for a low percentage of longitudinal reinforcement; in the range of below 1.2%, it may reduce the shear strength. The explanation for this is that the smaller percentage of longitudinal reinforcement reduces the dowel action and also the flexural cracks extend higher into the beam and are wider, reducing both shear compression zone and the interface shear transfer.

Regarding the effects of the yield strength of longitudinal reinforcement, tests(6) indicated that for a beam with *a/d* ratios from 1.5 to 3.8, the shear strength is independent of the yield strength. However, it was found that if the increase in yield strength is offset by a reduction in the amount of longitudinal reinforcement to give a constant moment capacity, shear strength would be reduced. The above reasons can also be applied to explain this situation.

Previous tests(6),(7) generally indicated that the primary role of shear reinforcement is to accommodate the redistribution of internal forces when diagonal cracking occurs. This is accomplished in two ways. First, the shear reinforcement will accept a portion of the redistributed internal forces through a sudden increase in tensile strain and, hence stress on formation of the diagonal crack. Secondly, the shear reinforcement restrains the diagonal crack development, thus preventing deep penetration of the diagonal crack into the compression zone. Shear reinforcement near the bottom of diagonal cracks was found to be effective in preventing dowel splitting cracks and in increasing the bond strength by providing confinement. Placas and Regan(23) found that vertical and inclined stirrups had equal efficiency.

As mentioned in **Section 2.2.2** the ratio of the shear span to the effective depth, *a/d,* has been found as an important variable in the shear strength of a concrete beam. For general use, including uniformly loaded beams, the ratio is always expressed as *M/Vd*, where *M* and *V* are the bending moment and shear force at the section considered respectively. This term is part of the ACI's diagonal crack expression (refer to **Section 2.4.2.1**). However, such a transformation raises the question as to whether the variable is really a geometrical effect (*a/d*) or a loading effect (*M/Vd*). It would seem more logical for it to be a geometrical effect.

There are many other factors that can affect the shear strength and behaviour of a concrete beam. The shear behaviour and strength of a member may change due to the way the load is applied to it(6). Axially applied loading in compression can increase the shear strength of a concrete beam(6). However, with regard to the current work those are not within the scope of study.

Probably the work of Clark and Thorogood(65) on the behaviour of transverse strips of a circular voided bridge deck in shear can be related to the current study. The study examined the effect of voids on shear behaviour. It was however particularly directed towards the need to reinforce the section in order to control the diagonal crack.

2.3 ANALYTICAL METHODS OF SHEAR IN CONCRETE BEAMS

2.3.1 General

A large number of publications discussing the development of the analytical solutions for shear in concrete are available. References (6), (7), (8), (9) and (24) review and discuss most of the work that has been done very comprehensively. In this thesis, only some of them that are relevant to the present work are discussed.

The following discussion is divided into two main sections. **Section 2.3.2** discusses the historical background of the development of analytical methods. In this section the early development in the formulation of analytical solutions is briefly reviewed. **Section 2.3.3** concentrates the discussion on a more rational approach of analytical solutions, which comprises the plasticity theory and modified compression field theory.

2.3.2 Historical Background

Researchers have been working on the analytical solution for shear in concrete beams for about a century. Among the pioneers was Morsch(25) who, after suggesting diagonal tension as the cause of inclined cracking, established the classical expression below,

$$v = \frac{V}{bjd} \tag{2.1}$$

v	=	shear stress
V	=	vertical shear at the section
b	=	width of the section
jd	=	internal moment arm

The above expression has been widely used as a measure of the diagonal tension stress(7). CP114 used the expression to obtain the shear cracking load by replacing v with the empirically determined tensile strength of concrete(24). Today the same equation is modified by replacing jd with d, the effective depth of beam, and design codes such as BS 8110(19) and BS 5400(20) use it to obtain the nominal shear stress in a concrete beam, although Evans et al(26) noted that the distribution of the shear stress across a flexurally cracked beam was not understood and an accurate determination of the magnitude of v is impossible.

The earlier solutions like the one proposed by Morsch were simple, but later was found as over-simplified and based on very limited information. In the early design, the expression gave the measure of diagonal tension and related it to the cylinder strength of concrete, f_c by restricting the stress, v to a safe limit, which was a certain fraction of f_c. This implied that shear strength was only related to concrete strength. The ACI-ASCE Committee 326(7) examined this procedure against tests data and found that only a small portion of the variation in cracking shear stress was due to concrete strength variation.

Throughout the period of between 1940's to 1970's, as research on shear progressed and more information on shear emerged through numerous experimental works, researchers studied varieties of models of shear behaviour and proposed a substantial number of analytical solutions. More parameters were taken into account, but in general almost all of them were empirically formulated, complex and applicable to only a limited range of beams. In general, separate analytical formulations were given for the shear cracking load and the collapse load.

The analytical solutions will be discussed in the following section according to the classification of the methods.

2.3.2.1 Empirical Solutions

Among the early researchers to adopt empirical solutions were Moretto(14) and Clark(15). Moretto suggested an empirical equation which took into account the percentage of the longitudinal reinforcement. Clark presented an empirical expression which includes four major variables, the ratio of the tensile reinforcement, the concrete strength, the ratio of the depth to the shear span, and the ratio of the shear reinforcement. The most popular and widely used empirical solution has been proposed by ACI-ASCE Committee 326(7). The solution was intended for the design of concrete beams in shear. The basis of the solution was to relate the diagonal cracking load to the maximum principal stress in an element subjected to shear and flexural tension. The computation of this solution was extremely complex and considerable simplifications were required. After considerable simplifications, an expression has been proposed for predicting a diagonal cracking shear load. The expression was formulated empirically, based on a considerable number of test data and incorporated three major variables: the ratio of shear span to the effective depth of the beam, the amount of longitudinal reinforcement and concrete tensile strength. As it was developed specifically for design, the solution satisfies the requirement that it should be simple to facilitate the everyday design work.

However being an empirical expression, it does not actually represent the physical behaviour of shear in beams. Inadequacies of the analytical solution such as those given by the ACI-ASCE approach are clearly indicated through comparison with test data(6),(7),(24). **Figure 2.2** taken from reference (27) demonstrates this fact. Zsutty(27), through his statistical study, highlighted the imperfections of the 1962 ACI-ASCE empirical approach. He noted that the proposed expressions did not take into account the two separate types of beam behaviour, arch action and beam action, and also the theoretical principal stress formulation of shear cracking behaviour did not properly represent or weight the beam properties such as the concrete strength and the percentage of longitudinal reinforcement that govern shear strength.

2.3.2.2 Truss Analogy

The truss model introduced at the beginning of this century has been the most popular analytical solution for shear strength of concrete beams with shear reinforcement for decades(28). The original truss analogy postulates that a diagonally cracked reinforced concrete beam acts as a truss with parallel longitudinal chords, and with a web composed of diagonal concrete compression struts inclined at 45° and shear reinforcement acting as tension ties. It predicts that the failure of beams is caused by the yielding of the shear reinforcement.

It has later been shown that this popular 45° truss model was extremely over-simplified and ignored the concrete contribution to the shear strength. It also completely ignored the favourable interaction between shear reinforcement and the aggregate-interlock capacity and the dowel force capacity. The Shear Study Group(24) noted that some of the main theoretical objections to the approach are, (i) it ignores the ability of the concrete compressive zone to support shear; (ii) it appears to predict that failure is caused by the shear reinforcement reaching its yield stress, while in fact shear failure of beams with shear reinforcement is generally due to the compressive failure of the concrete above a shear crack, and (iii) the assumption that all web compressive forces, or in effect all shear cracks are at 45° to the main steel is an over simplification. There is also no need to assume that the truss is based on 45° model. In fact the lower-bound plastic method (see **Section 2.3.3.1**) generalises the truss model.

2.3.2.3 Other Methods

Many other researchers worked on various shear models other than those classified above. For example, Kani(12) attempted what seems to be a more realistic approach by addressing the problem of the bending of the teeth of concrete between flexural cracks. The work of Kani, was later improved by Lorentsen(29) and also Fenwick et al(30), by including arch action, aggregate interlock and dowel actions in their investigation.

Regan(9) discusses the work of some of the researchers working on shear models based on equilibrium analysis. These models were applicable for beams observed to fail in shear-compression. The earlier version of this category of models was proposed by Laupa et al(31) and was highly empirical. Walther(32) presented a complicated solution and assumed unrealistically that the angle of crack was at 45°. Krefeld et al(33) idealised a tied-arch action of a beam in shear and assumed the inclined portion of a diagonal crack as a smooth profile. Krefeld then formulated a shear failure equation assuming that the dowel action cracking as the usable maximum load. The equation needed an empirical correction factor. Regan(34) and his work with Placas(35) contributed towards a more practical solution.

With numerous tests data available, Zsutty(27) attempted to approach the problem of an analytical solution of shear by carrying out a statistical analysis and proposed equations comprised of concrete strength, longitudinal steel ratio and shear span ratio. A fracture mechanics approach has been proposed by Bazant(36). The approach by Bazant is one of the solutions which have a rational basis. Bazant's method will be further discussed in **Section 2.4.2.3.**

There are a lot more studies on analytical solutions for both diagonal cracking load and collapse load. Some of the work studies in detail the role and the contribution of specific components of the mechanisms of shear transfer. Walraven(37), for example carried out an extensive study of the force transfer across a crack and found that the crack width, the aggregate size and the concrete strength were the important parameters. Walraven's work has been incorporated in the modified compression field theory which will be discussed in **Section 2.3.3.2.** Detail of such discussion can be found in References (6),(7),(8),(9),(24).

The brief review above indicates that the highly inconsistent behaviour of shear has resulted in a variety of shear models being introduced. The complexities of shear also lead, at some stages, researchers to make incorrect assumptions such as in the truss analogy with a 45^{o} diagonal crack angle. Researchers have proposed various models and studied various factors. However, from the above review it can be seen that no solution can give a close shear prediction without empirical factors included. Although researchers could not generally agree on a single solution, the numerous research studies nevertheless have been able to provide vital information regarding shear behaviour and the parameters involved.

With regard to the applicability of the above reviewed methods of solution to the problems studied, only ACI-ASCE, BS 8110 and Bazant methods will be further discussed in **Section 2.4**. They were chosen because, for ACI's method as well as the BS 8110 method, they have some rationality and widely accepted for design. In the case of Bazant's it represents one of the solutions that has some basis of rationality.

2.3.3 Rational Methods

Although some of the shear models discussed previously such as the work of Kani may be regarded as a step towards a rational solution for shear, a significant development towards a rational solution for shear in concrete beams was probably started when researchers realised that the angle of the diagonal cracking was not necessarily 45^{o} at failure, but could be derived from compatibility and equilibrium conditions. This improved truss model has been developed significantly in Europe(9),(38). Kupfer for example derived web inclination from compatibility conditions by minimising the strain energy of the web with respect to the compression angle. The solutions based on this approach provides a more general basis for explaining the behaviour of reinforced concrete beams in shear.

2.3.3.1 Plastic Analysis

Another stage of influential development of truss models has taken place in Copenhagen and Zurich(9). The model was based not on complete constitutive relationships but rather on plastic theory and extensive studies have been carried out by many researchers such as Nielson and Braestrup (39),(40),(41) and Grob and Thurlimann(42).
In the plastic analysis, concrete is assumed as rigid and perfectly plastic and its tensile strength is neglected. These assumptions mean that any elastic deformations and work-hardening effects are neglected and unlimited ductility is assumed. In reality concrete is a type of material which has a limited ductility. In order to allow for this lack of ductility, the actual compressive strength is reduced by a factor called an 'effectiveness factor'. In plastic analysis this is the only factor that has to be determined experimentally and will be further discussed later in this section.

Nielsen et al(44),(45) have shown that the plastic analysis works in cases where shear failure is governed by web crushing. Nielsen and Braestrup(43) went a step further to apply the theory to beams where there was no shear reinforcement to assist with the necessary redistribution of stress, and found that the results were encouraging. Of course close prediction of the theory is only obtained when the concrete compressive strength is reduced. As usual, in the plasticity approach two solutions are possible: (i) lower-bound solution and (ii) upper-bound solution.

A lower-bound solution requires a statically admissible, safe stress field to be constructed and is well suited for design purposes. The plastic shear theory applies a truss model with variable inclination of the concrete diagonals. It consists of the longitudinal reinforcing bars acting as stringers, shear reinforcement as vertical ties and the concrete diagonals as inclined struts forming a continuous compression field. The examples of the model for beams with and without shear reinforcement constructed by Nielsen(40) are shown in **Figure 2.3(a)** and **(b)**. Equilibrium equations for the forces in the stress field are developed. The solution for the web crushing criterion is then obtained by setting the shear reinforcement stress to the yield stress and the web concrete stress to a crushing limit, which is a reduced strength of the concrete in compression. The lower-bound solution has been incorporated in the European design code(9).

The example of the upper-bound models both for a beam with and without shear reinforcement is shown in **Figure 2.4(a)** as a failure mechanism consisting of a shear deformation at an inclined yield line. An alternative form of displacement field is shown in **Figure 2.4(b)**. Equating the rate of internal work to the rate of the external work, and optimising the angle of the inclination of the yield line, the lowest upper-bound shear failure load can be obtained. It is always found that a failure mechanism extending from the point of loading to the point of support would give the lowest upper bound solution in the absence of shear reinforcement. It is also found that, except for small amounts of longitudinal reinforcement, the shear deformation is vertical.

For beams with a small amount of longitudinal reinforcement, the shear deformation is not necessarily vertical(43). Study by Nielsen and Braestrup(43) showed that longitudinal reinforcement would contribute to the shear capacity, thus the shear deformation would be at an angle to the vertical shear plane if its amount, calculated in term of the ratio of the yield force to the crushing force in concrete is less than half of the effectiveness factor. Otherwise shear capacity is determined by the concrete alone.

Nielsen and other researchers have shown that for beams with and without shear reinforcement, both lower and upper bound approaches produced the same solution, indicating that the solution is exact in terms of plasticity theory. In the upper-bound solution, the yield line inclination is found as twice the angle of the compressive strut inclination. Nielsen noted that this shows that the shear stresses are transferred in the yield lines by the aggregate interlock action.

It has been shown(40) that despite its simplicity, and some unrealistic assumptions, especially with respect to the physical description of the concrete, the plastic solution can be applied to a wide range of shear failures in reinforced and prestressed concrete and it can predict remarkably closely the experimental shear failure loads provided that the concrete compressive strength is reduced. Nielsen noted, for the upper-bound solution, that despite the fact that the assumed failure mechanism is not necessarily the one always observed in reality, this does not affect the validity of the solution, since the failure mechanism of a rigid-plastic body is not uniquely determined. Nielsen(43) also claims that since it is possible to imagine a shear failure mechanism (upper-bound solution) and a stress state (lower-bound), corresponding to the same ultimate load, it is sufficient to ensure the validity of the solution.

The problem with the theory is one parameter called an effectiveness factor that has to be applied to the concrete compressive strength. The need for this empirical factor in the plastic solution (in many cases the concrete strength is reduced as low as 50%), leads to some concern of other researchers to use this method. Bazant and Kim(36), who worked on fracture mechanics expressed concern over the ability of this method to give a correct prediction on actual structures.

The effectiveness factor has to be determined by calibrating the theoretical predictions against experimental test results. This factor is necessary to take into account that concrete is not a perfectly plastic material, and has a limited ductility, and also to absorb all other shortcomings of the theory. **Figure 2.5** shows the typical curve of the actual concrete behaviour in compression and the model of plastic behaviour assumed in this method. The effectiveness factor 'adjusts' the stress-strain curve.

For a beam with shear reinforcement, Nielsen(44) proposed a simple expression for the effectiveness factor. It is only a linear function of the concrete compressive strength. For beams without shear reinforcement, the factor is a more complicated function of a number of parameters and the expression is given by Nielsen et al(45). From the expression given by Nielsen et al (see **Section 2.4.2.3**) it shows that the factor increases with a decrease in concrete strength. This shows that it can generally be accepted that the effectiveness factor does reflect the ductility of concrete since the stronger the concrete the more brittle it is. Nielsen and Braestrup(43) also claim that empirically the dependence of the factor on $\sqrt{f_c}$ in the absence of shear reinforcement is in line with the existing relationships between tensile strength of concrete to its compressive strength, hence confirms that the effectiveness factor is also a measure of concrete in tension.

The work of Vecchio and Collins(46) on strain-softening effects in cracked concrete clearly explains the phenomenon of the requirement of the effectiveness factor in plastic analysis. They tested concrete panels subjected to in-plane loads including shear and found that the compressive strength of cracked concrete reduces as the transverse tensile strain increases. As a result, the compressive strength of concrete found from uncracked cylinders or cubes should be multiplied by a reduction factor when applied to cracked concrete. The reduction factor is a function of the transverse tensile strain. **Figure 2.6** shows the sketch of the typical relationships between concrete strength and the increase in transverse strain as found by Vecchio and Collins.

2.3.3.2 Compression Field Theory

Collins also proposed a method for predicting the shear capacity of a concrete beam using compression field theory(38). The theory was first introduced to treat torsion in concrete members(47). Generally the theory was developed based on the variable angle truss model and it resembles the theory proposed by Wagner in 1929 to study the post-buckling shear resistance of thin webbed metal beams(48). The early versions of the theory ignored the concrete in tension and it was only applicable to sections where the effects of flexural moment were negligible(38). Following the extensive work in Toronto, Vecchio and Collins(46) introduced the modified compression field theory which is applicable to concrete beams. The theory is capable of predicting the complete response of a particular concrete section of a member subjected to shear, moment and axial load. The method has been adopted in the Canadian design codes(49).

The theoretical model treats cracked concrete as a new material with its own stress-strain characteristics. Strain-softening effects in the response of the concrete were taken into account. Equilibrium, compatibility and constitutive relationships are formulated in terms of average stresses and average strains. The variability in the angle of inclination of the struts is taken into account. The theory assumes the principal stress direction coincides with the principal strain direction. A comparison with test data indicated that this was a reasonable assumption(46). Consideration is also given to local stress conditions at crack locations which take into account the ability of aggregate interlock action to transfer shear and Walraven's work(37) on aggregate interlock has been incorporated in the solution(50),(51).

Using the theory, the cross-section of a member can be divided into a series of horizontal concrete layers and longitudinal steel elements and assigned with their own properties. For each layer the biaxial stresses and strains are determined by considering equilibrium, compatibility and stress-strain requirement. The only section compatibility requirement used is that plane sections remain plane, which implies that the profile of longitudinal strains are linear. The solution is obtained by an iterative process and with the support of a computer program the complete shear response of a given cross-section can be determined. The theory showed good correlations with experimental results(52).

From all the analytical methods discussed, probably the modified compression field theory is the most rational and contains no empirical constant. However, with the complex nature of shear, of course, the Collin's theory cannot be a comprehensive analytical model. Regan(9), for example noted that it is rather inaccurate to assume linear profiles of longitudinal strains in the presence of a shear crack. Other weaknesses include, because of its sectional character, the theory may not be capable of predicting the local effects caused by support and loading details as well as the effect of material discontinuity. As a result, it may underestimate the shear capacity of regions where a significant portion of the load is carried by direct strut action(52). Proper detailing of the section considered must be ensured in order for the theory to make a reliable prediction of shear response, which, in the general situation of the assessment of an actual structure, is beyond the control of the assessing engineer.

2.4 THE ASSESSMENT OF SHEAR IN CONCRETE BEAMS

2.4.1 Design and Assessment Codes

At the initial stage of structural assessment practice, structures have often been assessed according to the design requirement. The design codes were normally used and if a particular structure complied with the code then the structure would be classified as safe. It was later realised that such a straightforward procedure could in many circumstances lead to a conservative assessment result, especially with regard to the shear capacity. This happens because the old structures have been designed according to the old design codes whereas assessment is carried out according to the present codes. With regard to shear, the current codes are more onerous than the older in several respects(53).

In dealing with existing structures any decision whether to repair or demolish, following the results of an assessment, may inflict much greater and wider implications than building a new structure. Apart from the cost for repair or demolishing, any disturbance to the usage of the existing structure may incur extra costs. The closure of a bridge for example, will cause delays and disruptions and this will involve extra costs.

As a result of that, in the UK effort has been made to produce a special code for assessment. A series of assessment documents were produced by the Department of Transport and the latest in the series was BD 44/95(1). Clark(54),(55) has discussed the background of the development of these assessment documents. There are essentially modified versions of design codes. For example, the shear assessment expressions in BD 44/95 is the modified version of the BS 5400 shear design equation obtained by adopting a lower-bound rather than a mean best fit to test data. There is also the facility for using a reduced partial safety factor if the worst credible strength of the concrete is determined.

Many researchers such as Clark(55) are of the opinion that with regard to the shear assessment, the first principle analytical procedure and/or test data should be employed rather than using the procedure from the assessment or design codes. Among the reasons for that trend of opinion are; the procedures used in the code are a blend of empiricism and theory with certain imposed limitations. As discussed in **Section 2.3**, the complexities of shear behaviour result in such a design approach. For example, in BS 8110 and BS 5400, the clauses for shear design are based upon a 45° truss model with the addition of a concrete term. The unrealistic assumption of a 45° truss model was already discussed in **Section 2.3.2.2**. The concrete contribution, which depends on the detailing of longitudinal reinforcement is evaluated separately. This approach ignores the advantage of having both shear and longitudinal reinforcement evaluated as acting together(55).

Of course reverting to the basic analytical method will require much extra information. For example, in order to employ the shear expressions available, detailed information on reinforcement and sections are required. For certain analytical methods, for example the modified compression field theory (discussed in **Section 2.3.3.2**), in order for them to produce a correct prediction the detail of the section assessed must comply with the assumptions during the derivation stage. However, in an assessment, engineers can often obtain much information during the assessment, especially with the aid of various non and semi-destructive equipments that are available today. This advantage will facilitate the use of analytical methods in the assessment.

In dealing with a honeycombed or a weak spot of concrete in a beam, in the normal situation engineers will use the lowest strength of concrete obtained from the assessment and evaluate the capacity of the beam using either design, assessment or any analytical methods. It is unknown whether this method is valid without evidence of comparisons with the test data. All the shear studies were previously made on perfect beams. In the design or assessment codes, a factor of safety is applied to the expressions developed from tests. The factor of safety may take into account all the possibilities of experimental error and normal deficiencies such as variation in material properties, but not the presence of honeycombed concrete.

2.4.2 Analytical Approach

In the following section, theoretical evaluations are made with respect to the applicability of some of the analytical methods to the problem under study.

2.4.2.1 ACI Method

The ACI's diagonal cracking expression is given as below(56):

$$V_c = \frac{bd}{7}\left(\sqrt{f_c} + 120\rho\frac{V.d}{M}\right) \tag{2.2}$$

Where,

V_c	=	shear strength at diagonal cracking
b,d	=	width and effective depth of the section respectively
f_c	=	cylinder compressive strength of concrete
ρ	=	the ratio of the longitudinal steel
V	=	shear force at the section considered
M	=	bending moment at the section considered

The expression was developed based on the assumption that the diagonal crack was caused by excessive principal tensile stress occurring in the shear zone(7). Based on the equation for principal stress at a point in a shear zone, a rational relationship for diagonal tension cracking load was developed. There was great difficulty in computing an equation of principal stress in cracked concrete. The magnitude of the stresses was influenced by the presence of cracks. It could not be computed on the assumption of uncracked sections, neither could it be computed directly from the assumption of cracked sections. As a result major simplifications were introduced in order to produce the above expression in its present form, and empirical constants were determined by correlation with a huge number of test data. However, it did have a rational theoretical basis and contained all the major variables that affected the formation of diagonal cracking: the shear span ratio, the ratio of longitudinal reinforcement and the concrete strength.

Although it has a theoretical basis, but due to the degree of simplifications made, and the fact that it has empirical factors in order for it to work, the expression is essentially an empirical solution for shear in a beam with uniform concrete. However, although the empirical factors would have taken into account variations in concrete properties in the beams, as opposed to the control specimens, and minor defects, they would not have taken account of the presence of a honeycombed zone. The state of stress and its distribution within the shear zone could be modified by the honeycombed zone and the assumptions made in deriving the empirical equation may no longer be valid.

It may look simple enough to replace the concrete strength with the properties of the honeycombed concrete, but in the absence of a clear rationality of the expression in representing the shear behaviour, it is unclear how the properties of the honeycombed concrete can be incorporated into it. It should be conservative to adopt the properties of the honeycombed concrete, but the presence of honeycombed concrete could modify the structural response in a currently unpredictable manner.

2.4.2.2 BS 8110 and BD 44/95

The expressions given by BS 8110 and BS 5400 were also empirically derived(24). From BS 8110(19), the expression to predict a diagonal cracking shear, V_c, in a concrete beam is given as,

$$V_c = 0.79.\sqrt[4]{\frac{400}{d}}.\sqrt[3]{\frac{100A_s}{bd}}.bd.\sqrt[3]{\frac{f_{cu}}{25}} \tag{2.3}$$

From BD 44/95(1), the expression to evaluate the diagonal cracking shear in concrete is given as,

$$V_c = 0.24.\sqrt[4]{\frac{500}{d}}.\sqrt[3]{\frac{100A_s}{bd}}.\sqrt[3]{f_{cu}}.bd \tag{2.4}$$

where,

d	=	effective depth of beam
A_s	=	the area of longitudinal steel
f_{cu}	=	the cube strength of concrete

The equations were empirically formulated. As for the ACI method, there is no rational way to incorporate the presence of a honeycombed zone into the expressions. The expression, especially that given by BD 44/95 may be able to give a safe prediction of shear capacity of honeycombed beams, in view of the fact that at the derivation stage all the variation due to structural and material deficiencies were taken into account. However, without comparing it with real test data on honeycombed beams no safe and valid conclusion can be made.

A honeycombed zone can exist in the shear zone in different locations and in different sizes. Substituting the strength of honeycombed concrete into the expressions without considering the difference in the magnitude of the effect that it can cause on the shear capacity of the beam is inappropriate. With the above expressions no rational analytical consideration of the varying effect of the honeycombed zone can be carried out.

Note also that the expressions above give the diagonal cracking shear. It has been shown in many research works that beams, especially those categorised as a short shear span beam, can sustain a substantial amount of loading following the formation of the diagonal cracking before reaching the ultimate failure. In the assessment, this potential benefit should be utilised rather than resort to an uneconomic assessment approach.

2.4.2.3 Bazant's Method

The expression derived by Bazant(36) based on a fracture mechanics approach is given in the following form:

$$V_u = (bd)k_1.(r)^p\left((f_c)^q + k_2\frac{\sqrt{r}}{(a/d)^r}\right)\frac{1}{\sqrt{1+d/25d_a}} \tag{2.5}$$

Where,

V_u	=	ultimate shear strength
d_a	=	maximum aggregate size
k_1, k_2, p, q, r	=	empirical constants

All other symbols are as defined before

The expression gives the ultimate shear strength of a concrete beam. The last term on the right-hand side outside the bracket is the size effect, which is developed based on the fracture mechanics theory. The rest of the expression was derived on the basis that the shear strength of a concrete beam was contributed by a combination of beam and arch actions. The beam action was represented by the concrete tensile strength which was an indirect measure of the bond action of the longitudinal reinforcement. After some simplifications the arch action was finally represented by the geometry of the shear span. It has a rational theoretical basis, but all the empirical parameters in the expression were obtained from test data.

Theoretically it is very doubtful if the method proposed by Bazant can give a safe prediction of shear strength of a honeycombed beam under study. Probably it will give a good prediction of ultimate shear strength if a honeycombed zone is located at the anchorage zone and thus controls the failure of the beam, and the strength of the honeycombed concrete is used in the expression. It is unclear how this theoretical prediction can be used in cases such as a honeycombed zone located at a particular location which leads to an arch action failure, since the strength of concrete does not appear in the arch action component of the expression.

2.4.2.4 Upper Bound Plasticity Theory

According to the upper bound plasticity theory, the following work equation can predict the ultimate shear capacity of a concrete beam without shear reinforcement(45).

$$V_u = \frac{1}{2\sin(\alpha+\beta)} \nu f_c (1-\sin\alpha) \frac{bh}{\sin\beta} - A_s f_y \cos(\alpha+\beta) \qquad (2.6)$$

V_u	=	ultimate shear force
f_c	=	cylinder compressive strength of concrete
b, h	=	width and height of the beam
A_s	=	the area of the longitudinal reinforcement
f_y	=	the yield strength of longitudinal reinforcement
u	=	the displacement of the yield line
ν	=	the effectiveness factor

β = angle of inclination of the failure mechanism relative to the horizontal axis

α = the angle of the yield line displacement to the plastic mechanism

All other symbols are as defined before

The assumed failure mechanism upon which the above equation is based is shown in **Figure 2.4(a)**.

Plasticity theory seems a suitable method of analysis if the work equation above is modified to accommodate the strength of the honeycombed concrete when the assumed failure mechanism passes through the honeycombed zone in the shear zone. It looks logical that, if the failure mechanism passes through both normal and honeycombed zones, the concrete strength to be used is the 'weighted average' of the two concretes, averaged according to the lengths which they contribute to the mechanism.

However this may not be appropriate in all cases because the magnitude of the effect of honeycombed concrete on shear strength may be different at different locations. For example, honeycombed concrete at the bottom section near to the support may produce lower ultimate shear strength than if it is located at the top near to the loading point, although their amount of contribution to the 'average' strength of concrete is the same.

This occurs because the presence of the honeycombed zone near to the support may accelerate the failure through the early formation of a shallow diagonal crack and subsequent rapid transfer of the force to the anchorage. The resistance of the anchorage is also weak because bond that exists between the honeycombed concrete and reinforcement is weak. If the honeycombed zone is located at the top near to the loading point, the diagonal crack will be initiated by a flexural crack, and it will be a steep crack. This will turn the structure into a tied-arch and the behaviour of the honeycombed beam in the compression zone will influence the behaviour of the beam.

The effectiveness factor which is determined from tests and intended to take into account not only the limited ductility of concrete but all types of variation in shear behaviour of beams will not be able theoretically to accommodate the variation in shear behaviour due to the presence of the honeycombed zone. All the tests that have been done have never considered defects such as a honeycombed zone present in the beams. The resolution of these problems would of course involve a substantial collection of test data, especially if the solution is required to cover a wide range of honeycombed problems in concrete beams.

With regard to the effectiveness factor Nielsen et al (45) gives an empirical formula in order to evaluate the effectiveness factor. The expression for a beam without shear reinforcement is complicated and based on beams with normal concrete. It is a function of concrete strength, the depth of the beam, longitudinal reinforcement ratio and the shear span ratio and is given in the following form:

$$\nu = f_1(f_c) f_2(h) f_3(\rho) f_4\left(a/h\right) \tag{2.5}$$

in which,

$$f_1(f_c) = 3.5/\sqrt{f_c}$$

$$f_2(h) = 0.27\left(1 + 1/\sqrt{h}\right)$$

$$f_3(\rho) = 0.15\rho + 0.58$$

$$f_4\left(a/h\right) = 1.0 + 0.17\left(a/h - 2.6\right)^2$$

Where,

h = overall depth of beam section

All other symbols are as defined before.

As already mentioned, since the effectiveness expression given by Nielsen et al is based on perfect beams, it is very unlikely that it can be used for beams with a honeycombed zone. At present however no alternative is available. In order to formulate one, a large number of tests would certainly be required. The possible modification in order to take into account the presence of honeycombed concrete is to use the same 'average' strength of concrete that is computed for the work equation as described above and insert it into the effectiveness expressions. Again, this would need a substantial series of tests for validation.

2.4.3 Assessment Criteria- Diagonal Cracking or Ultimate Failure

The UK and US design codes specify that the diagonal cracking load is the criterion for the design of ultimate shear capacity of concrete beam without shear reinforcement. No doubt this is the safest limit of safety for any type of beam. However, as often observed, for beams with short shear span, the ultimate load is always substantially greater than the diagonal cracking load, the amount of which will depend on the main reinforcement and its anchorage. Bazant(36) argues that designing against the diagonal crack initiation does not assure a uniform safety margin and thus recommends that a concrete beam should be designed against the ultimate failure. For a beam with shear reinforcement, the formation of diagonal cracking is just the beginning of the failure process, where stresses are being transferred to shear reinforcement.

It is thus difficult to give definitive general guidance on which criterion should be used in assessment. A short beam assessed on the basis of its diagonal cracking load may result in a very conservative assessment, while for a medium to slender range of shear spans beams may just produce safe assessment results.

At this stage, it is not a matter of selecting the appropriate criterion for shear loading assessment, but rather it is selecting the most rational and accurate method of shear assessment that matters.

2.5 SUMMARY AND CONCLUSION

The above discussion clearly shows that shear is a complex phenomenon. Researchers for years have been busy with studies to understand the basic nature of shear. Numerous experimental works have been carried out in order to identify the basic factors that influence the shear behaviour and its strength. The scale of the complexities of shear is reflected when researchers take about 60 years after Morsch and Ritter introduced their classical theory, to really embark their research on a rational approach. For decades shear design procedures have been mostly based on unrealistic assumptions and analytical expressions that are empirically developed.

The review of the literature shows that shear behaviour and its mechanism of force transfer rely very significantly on the concrete. The compressive strength of concrete appears in all expressions of shear strength prediction, reflecting concrete as the important parameter in determine the shear strength of a concrete beam. Although studies showed that other parameters such as the amount of the longitudinal and shear reinforcement also play an important role in the shear transfer mechanism and in determining the shear strength, concrete plays the most significant role. In shear it is concrete that transfers and distributes the force to the other components of the beam. Once shear force is being transferred to a concrete beam, it is the properties of the concrete that will determine at which stage the cracks, flexural or diagonal or a combination of both, will develop and a complex redistribution of stresses occur within the concrete and other components of the beam.

The important role of concrete is more crucial for members with a medium to short shear span in which shear can be transferred through concrete compressive struts. In such cases the presence of a weak concrete within the potential path of the struts can certainly affect the shear capacity of the member. Shear transferred through other components of the mechanisms can also be affected by the properties of the concrete. For example, the transfer of force through the action of the aggregate interlock depends on the properties of the concrete. The resistance provided by the anchorage of the longitudinal reinforcement is determined by the bonding between steel reinforcement and concrete, and the magnitude of the bonding is partly dependent on the concrete strength. The presence of a honeycombed zone at the crown of the arch, in the case of beam acting as a tied-arch, can directly affect the shear strength of the member.

Prior to cracking, a honeycombed zone may alter the distribution and redistribution of tensile and compressive stresses within the high shear zone once the beam is subjected to an applied load. This would certainly affect the magnitude and direction of the principal stresses and the subsequent shear behaviour of the beam. Also, the inconsistency and localised nature of shear behaviour in concrete members raise concern over the presence of a honeycombed zone at any spot within the shear zone. The overall shear behaviour can be altered and possibly lead to a premature failure of the member.

All the analytical solutions that have been developed in many years of research are generally meant for design, and as such, imperfect conditions such as the presence of honeycombed zones in the member have never been considered. All the methods studied in the past treated the shear zone as containing a uniform quality of concrete. Until the present work, neither experimental nor analytical work has been carried out to study the effect of honeycombed zones on the shear capacity of concrete beams.

None of the analytical solutions that are reviewed in the previous section can be categorised as fully rational. Some have a rational theoretical basis, but eventually they all need empirical parameters in order to give a close prediction of shear strength for a range of concrete beams. Probably the least empirical is the modified compression field theory. Note however, its unrealistic assumption of linear distribution of the longitudinal strain in cracked concrete. This happens because shear behaviour cannot be generalised under all conditions. It is very sensitive to a change in parameters such as the section of the beam, shear span ratio and steel reinforcement. It can even happen that nominally identical beams behave differently. This occurs due to the fact that shear behaviour and strength are influenced by several parameters and probably there are parameters or the relationships between various parameters yet to be uncovered.

Currently it seems that almost all of the available analytical solutions cannot be rationally applied to the problem of honeycombed concrete beams under study. Even those solutions that seem to be amenable to take into account the presence of honeycombed concrete need substantial experimental verifications and the establishment of empirical parameters looks inevitable. It would not be expected that the work is straight forward.

The ACI-ASCE and British design codes analytical solutions may be an acceptable tool for structural design. However, the application of this method to assessment may not be the best solution even for a perfect member. Apart from being empirical, the solution treats the contribution of concrete and reinforcement to the shear capacity of a beam separately. Clark(54) noted that in the situation where all the information such as longitudinal and shear reinforcement are fully defined, it is worthwhile to consider non-elastic methods where a more accurate structural response can be obtained.

The modified compression field theory seems to be the most general and rational in comparison to all other analytical methods. Its prediction of shear capacity agrees well with the test data. The layer approach of the modified compression field theory can possibly be extended to solve the current problem, in which a honeycombed zone can be located anywhere within the shear zone. Its ability to predict a complete response of a structure is advantageous to the assessment. The sectional character of the method is not a problem when dealing with a honeycombed zone.

The plasticity based variable angle truss model and its associated upper bound collapse mechanism can possibly be modified and apply to the current problem. There are some rationalities in the method in taking into account the presence of a honeycombed zone. The dependence of the shear strength on the length of the plastic mechanism, as implied in the work equation, allows the strength of the honeycombed concrete to be included in a rational manner. The method if later it can be proven as valid would provide the assessing engineers with very simple analytical tools in assessing the shear strength of honeycombed beams. Furthermore it provides the solution at the ultimate stage, which is more appropriate for assessment. For the latter two reasons, the plastic approach will be adopted in this study rather than the modified compression field theory.

The application of a plasticity method to the problem under study needs analytical work to be developed together with experimental verifications. Apparently, the most difficult problem is to consider the amount of contribution from the normal and honeycombed concretes. The logical way forward is to use the 'average' strength, but question arises on how to average them. Problems may also arise as how to accommodate the strength of both concretes in computing the effective concrete strength. All this will be developed by comparing the proposed theoretical consideration to the results of the experimental work.

The review also shows that shear in concrete beams is a complex subject. Its behaviour and strength depend on many variables. In such a situation, the current study has to limit the number of variables. The variables to be studied must be carefully chosen so that the present study can provide an optimum contribution to the current shear knowledge.

It seems appropriate that, for this particular study the major variable would be the locations of honeycombed zone within the high shear zone. The effects caused by the variation of both normal and honeycombed concrete strengths will also be examined. Other variables should include a study on the effects of different sizes of honeycombed zones on shear behaviour and strength. The problems of a honeycombed zone formed at an angle to the vertical plane of concrete beams are usually found in construction joints due to a poor construction practice. This could be another important problems which can be simulated and examined experimentally and analytically in this study.

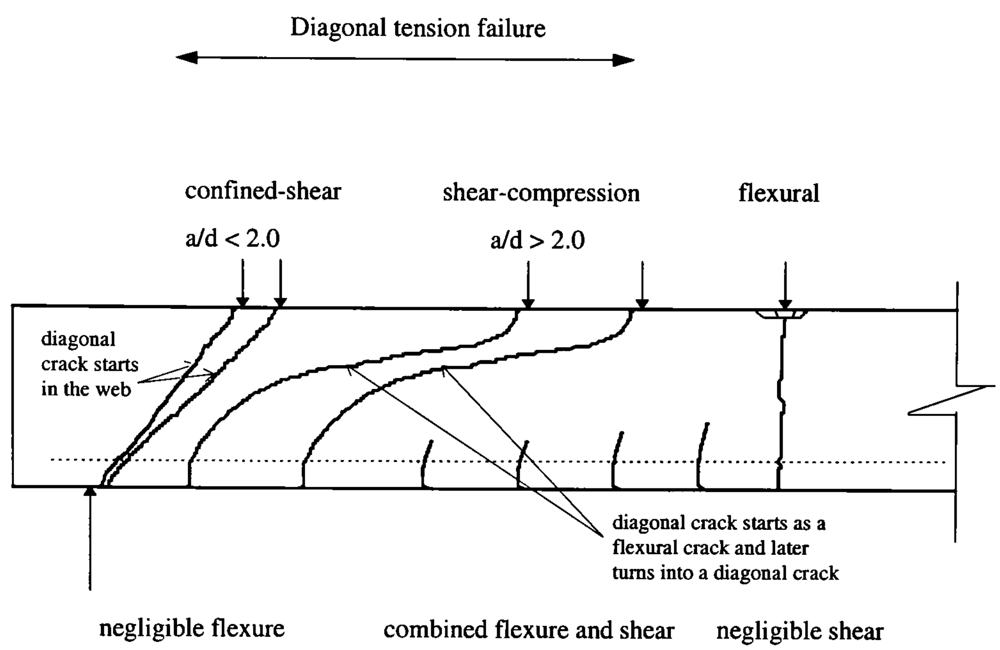


FIGURE 2.1 Modes of Shear Failure

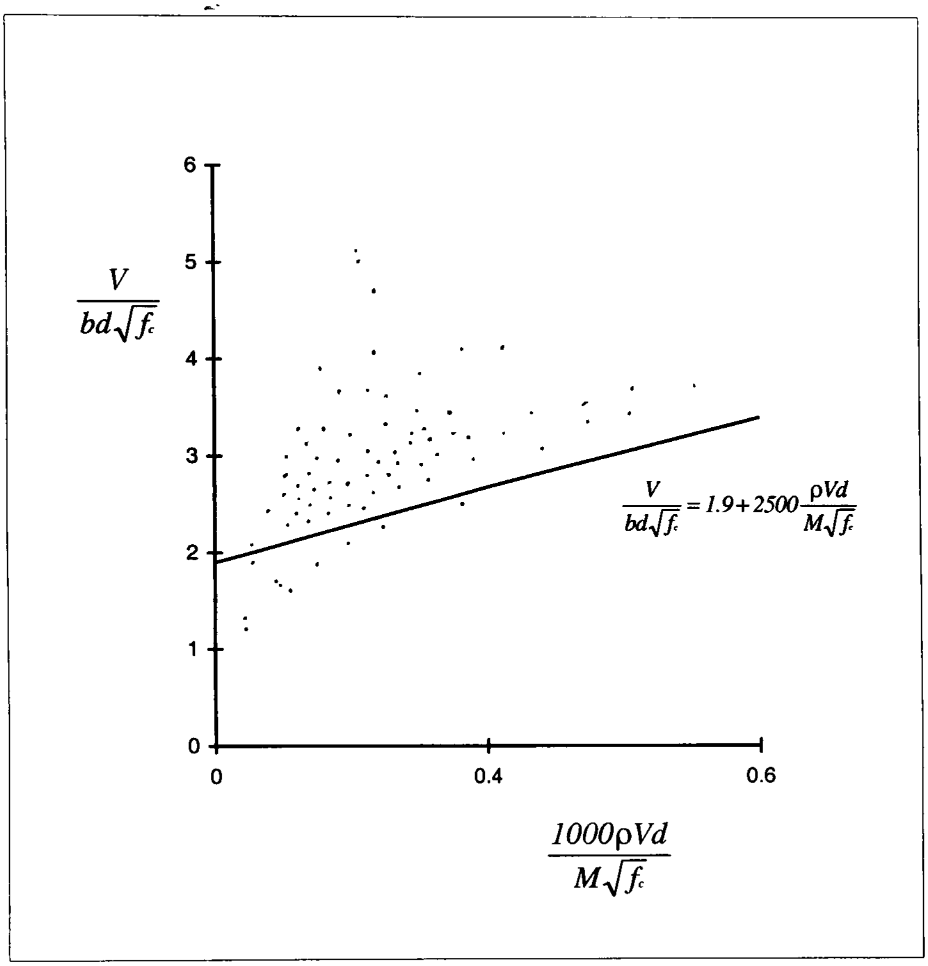
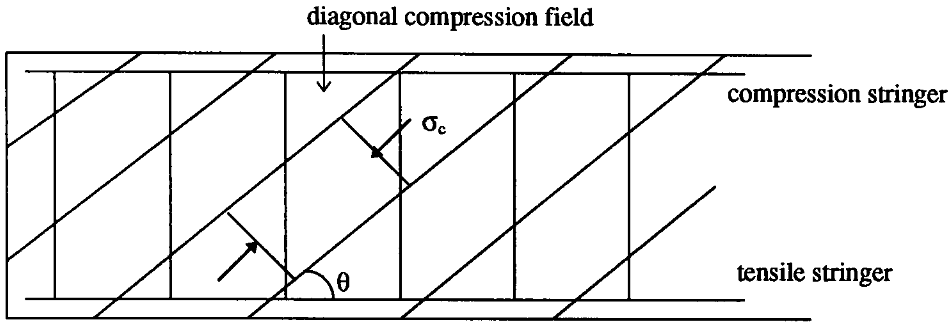


FIGURE 2.2 Plot of ACI cracking shear formula compared with test data (from Zutty)

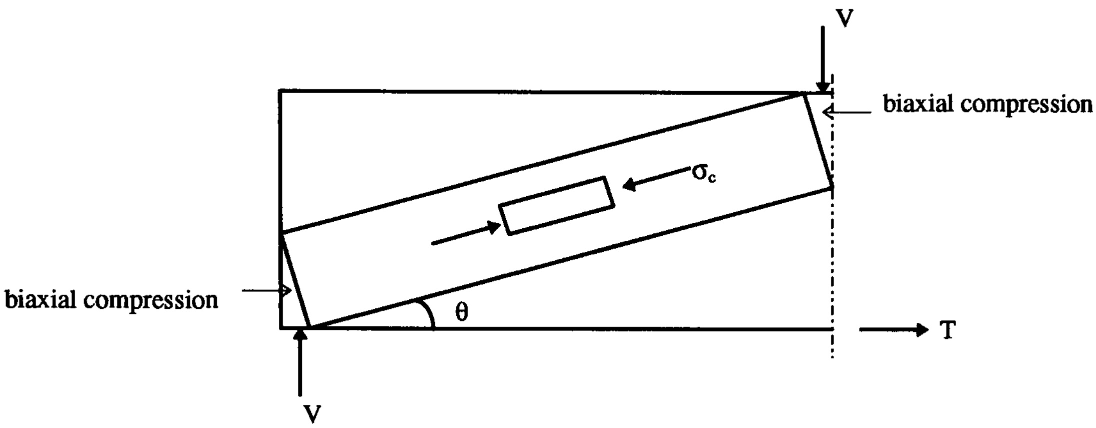


σ_c = uniaxial compressive stress in the concrete

θ = angle of inclination of compression field

Beam with shear reinforcement
-diagonal compression stress field in the web

FIGURE 2.3(a) Lower bound solution



Beam without shear reinforcement

FIGURE 2.3(b) Lower bound solution

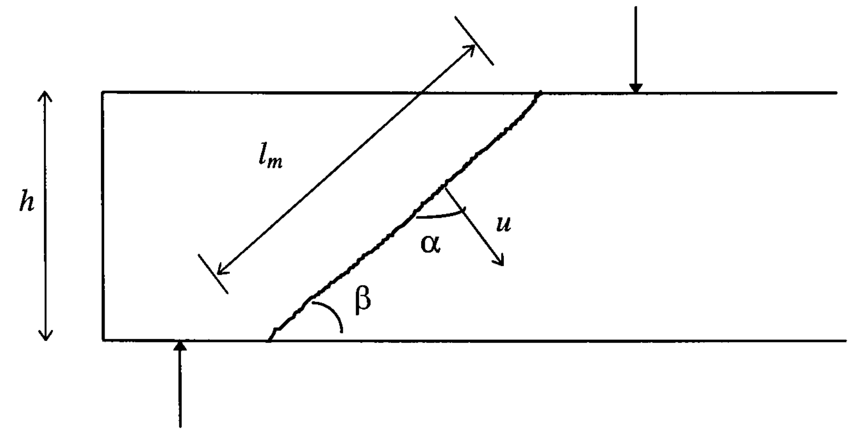


FIGURE 2.4(a) Plastic failure mechanism

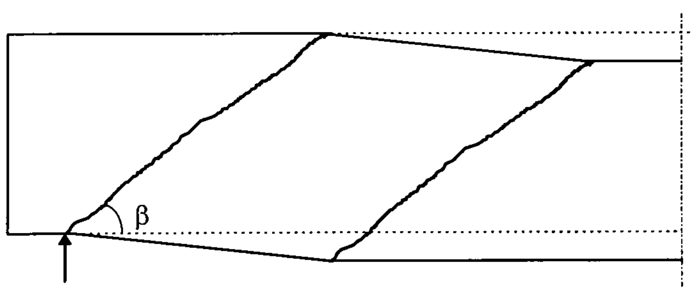


FIGURE 2.4(b) Alternative shear failure mechanism

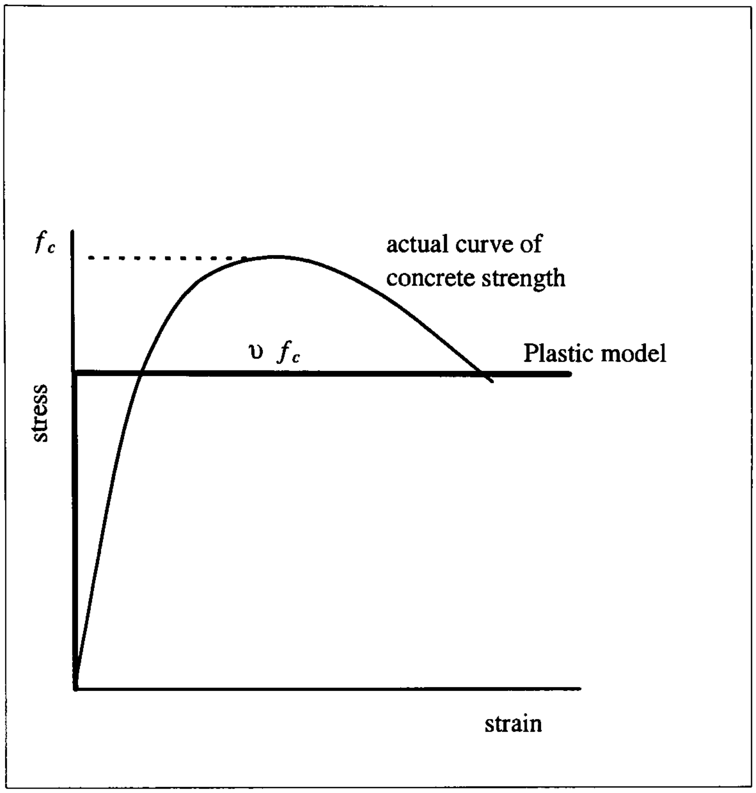


FIGURE 2.5 Typical curve of concrete behaviour and the plastic model

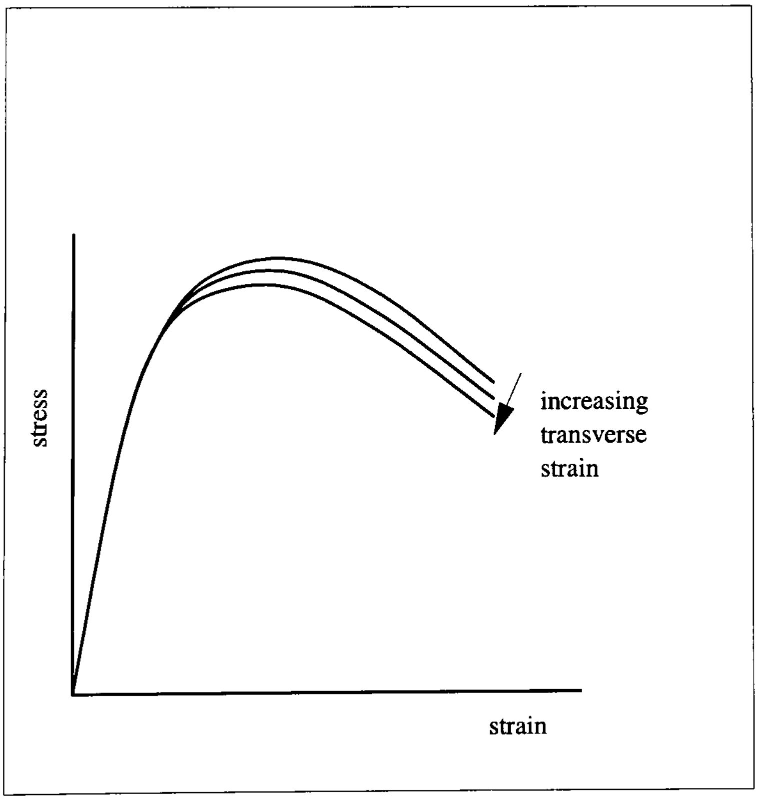


FIGURE 2.6 Typical concrete strength curve and the effect of increasing transverse strain

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CHAPTER 3

EXPERIMENTAL WORK

3.1 INTRODUCTION

Four series of tests were carried out in the current study, namely series 1A and 1B and series 2A and 2B. All the tests were carried out on rectangular beams. The dimensions of the beams were 100 mm wide and 200 mm deep with the total length of 2.435 m. In all the 4 series, a total of 36 beams were cast and tested. In series 1A and 1B, 8 and 9 beams were cast and tested respectively. For series 2A, 12 beams were cast and tested and in series 2B another 7 beams were cast and tested.

The main variable of series 1A and 1B tests was the location of the honeycombed zones within the high shear region of the beam. The size and the shape of the honeycombed zones, load position, shear span, the distance between supports and the amount of longitudinal reinforcement were kept constant for all the beams in both series. Series 1A dealt with a relatively high strength of normal concrete and series 1B dealt with a relatively normal range of concrete strength, with the average cube strengths of normal concrete of about 50.5 N/mm^2 and 33.5 N/mm^2 respectively. The strength of the honeycombed concrete was also fixed at the average of 23.4 N/mm^2 and 11.9 N/mm^2 throughout all beams in series 1A and 1B respectively.

Series 2A and 2B consisted of tests to verify certain observations made in series 1A and 1B tests respectively and also to further investigate the effect of certain parameters. The average strengths of the normal concrete in series 2A and 2B were 47.2 N/mm^2 and 35.6 N/mm^2 respectively. The average strengths of the honeycombed concrete in series 2A and 2B were 13.1 N/mm^2 and 13.0 N/mm^2 respectively. In most cases the location of honeycombed zone in 2A and 2B series was kept at the centre of the high shear region as this was the location that generally exhibited the maximum effect.

Included in series 2A were tests to examine the method used to simulate honeycombed zones in a concrete beam; to examine the effect of the size of a honeycombed zone on the shear behaviour; to examine the effect of the change in the ratio of the honeycombed to the normal concrete strengths; to examine the effect of the change in the shear span ratio; to study the behaviour of a beam with a honeycombed zone simulating the construction joint; and also included tests to examine the shear behaviour of honeycombed beams with shear reinforcement. Except for the study of the change in the strength ratio, in which 3 locations of honeycombed zone were examined, other studies were carried out for the honeycombed zone located at the centre of the high shear region.

In series 2B, the tests carried out were to study the following: the effect of the honeycombed zone present at an angle to the vertical plane, simulating the problems at a construction joint; the effect of a larger size of honeycombed zone; honeycombed beams with shear reinforcement; and lastly honeycombed beams with a bigger shear span ratio.

3.2 CONCRETE MIX

3.2.1 Materials

The ordinary Portland cement used throughout these investigations was supplied by Rugby Cement. Based on the size of the beams it was decided that 10 mm aggregate would be used for both the normal and honeycombed mixes. The aggregate was supplied by ARC Limited taken from Weeford Pit, Sutton Coldfield. The aggregate was graded according to BS 812, Part 103, 1985, and the result of the sieve analysis is given in **Table**

3.1. The sand used was medium grading and supplied by ARC Limited. The results of the sieve analysis of sand are given in **Table 3.2**.

High yield steel bars of 12 mm diameter were used as bottom longitudinal reinforcement in all beams. Samples of the steel were tested and the values of the elastic modulus, E_s, the yield strength, f_y and the ultimate strength, f_u were 193.4 kN/mm^2, 497 N/mm^2 and 594 N/mm^2 respectively. For the shear reinforcement, mild steel bars of 3 mm diameter were used. The elastic modulus, E_{sv}, the yield strength, f_{yv} and the ultimate strength, f_{uv} of the shear reinforcement were 217.5 kN/mm^2, 523.4 kN/mm^2 and 621.1 kN/mm^2 respectively. Two high yield steel bars of 8 mm diameter were provided at the top of the beam to hold the shear links for beams tested with shear reinforcement. The elastic modulus, E_{st}, the yield strength, f_{yt} and the ultimate strength, f_{ut} of this 8 mm diameter bar were 211.5 kN/mm^2, 537.1 N/mm^2 and 624.5 N/mm^2 respectively.

3.2.2 Normal Concrete Mix

The proportion of the materials used for the normal concrete mix of series 1A, 1B, 2A and 2B beams are as shown in **Table 3.3**. The strength of normal concrete for all the series of tests represents typical concrete used in real construction.

3.2.3 Honeycombed Concrete

The low strength of the honeycombed concrete can be associated with the large voids that exist within the mix. In the real structures honeycombed zones may form due to inadequate compaction and/or a lack of fine materials. In this investigation the degree of honeycombing was quantified in terms of strength. Two methods of simulating honeycombed concrete were investigated.

At the initial stage of the experiments, polystyrene balls of 5 mm average diameter were used to form voids. The polystyrene was mixed with the normal concrete mix occupying 50 percent of its volume. The cube strength was found to be reduced to about 50 percent. However, it was found that the polystyrene balls were very light and volatile and extremely difficult to handle. It was also difficult to accurately quantify the volume of polystyrene balls. Another problem encountered with the polystyrene ball was that the balls tended to float to the surface even with a minimum compaction applied to the mix.

Another method to simulate the honeycombed zone was to use an uncompacted no-fines mix. The portion of the sand in the normal mix was replaced with 10 mm aggregate. The mix produced had large voids uniformly distributed through its mass, and thus yielded a low strength mix. The no-fines mix needed a correct amount of water. Excessive amounts of water yielded a paste which was too fluid and which flowed off the aggregate particles, reducing cohesion in the upper portion of the specimen and filling voids in the lower part. Too little water led to a paste which did not coat the aggregate particles completely, and resulted in insufficient adhesion between particles so that proper compaction could not be achieved. It was recommended that hand compaction using a rod should be sufficient for this type of mix(57). The proportion of cement, water and aggregate for honeycombed mixes in all series are shown in **Table 3.3**.

From the cube tests it was found that the cube strength of the hand-compacted no-fines mix could be reduced to as low as 30% of its respective normal mix. However, the cubes produced from the mix had rough surfaces and this caused some concern to be expressed on the consistency of the results of cube strength. However, results from tests, to be discussed in **Section 4.2** of **Chapter 4**, indicated that consistency could be achieved. It was then decided that the no-fines mix would be used to simulate honeycombed concrete. Henceforth, concrete cast using this mix is referred to as honeycombed concrete.

3.3 THE PREPARATION OF BEAMS AND CONTROL SPECIMENS

3.3.1 The Descriptions of the Beams

All the concrete beams cast and tested in this study were rectangular in cross-section. The nominal width, height and length of all the beams were 100 mm, 200 mm and 2.435 m respectively. The dimensions chosen generally represent a typical range of concrete beams in terms of its breadth to depth ratio, used in real construction. Another reason why such dimensions were chosen was that the existing moulds available in the laboratory could be used for casting.

Two steel bars of 12 mm diameter were provided as longitudinal reinforcement at the bottom of each beam. Except for beams 2A-8 and 2A-9 and beams 2B-3 and 2B-4, all the other beams studied had no shear reinforcement. The anchorage of the reinforcement in all beams was provided by an extra length of the bar equivalent to at least 12 times the diameter of the bar beyond the point of support; thus, in such case hooks were not required. The concrete cover was 15 mm. All the beams were deliberately designed to fail in shear.

Figures 3.1 and **3.2** show the location of honeycombed zones in the high shear region and the reference identification of each beam for all series of tests. In all cases the honeycombed zone occupied the whole width of the beam. **Figure 3.1** shows the honeycombed zones that were square in shape and the beam's reference number in the table under the diagram. Except for beams 2A-7 and 2B-2, the size of the honeycombed zone in all the other beams was 60 x 60 mm. This dimension was about one third of the effective depth of the beam. For beams 2A-7 and 2B-2, the size of the honeycombed zone was enlarged to 90 x 90 mm. The size was about half of the effective depth of the beam. They were cast in order to examine the effect of the change in the size of the honeycombed zone on the shear capacity of the beam.

For the purpose of convenient discussions throughout this thesis, the table in **Figure 3.1** identifies the locations of the honeycombed zone in a grid form. Vertically the three locations are identified as top(T), middle(M) and bottom(B). Horizontally the locations are identified as support(S), middle(M) and load(L). For each beam the location of the honeycombed zone is identified with a code. For example for beam 1A-3, its identification code is [MS], the honeycombed zone is located vertically at the middle of the shear region and horizontally near the support.

Figure 3.2 shows the narrow honeycombed zone inclined to the vertical plane in beams 2A-6 and 2B-1. The width of the honeycombed strip was 30 mm and inclined at 45° to the longitudinal axis. This was to simulate problems of a construction joint.

The control beam, without a honeycombed zone, for all honeycombed beams in series 1A and beams 2A-1, 2A-2, 2A-3, 2A-6 and 2A-7 in series 2A was beam 1A-1. Beam 1B-2 was a control beam for all honeycombed beams in series 1B and beams 2B-1 and 2B-2 in series 2B. Beams 2A-4 and beams 2B-5 were the control beams for beams 2A-5 and 2B-6 respectively. They were the beams with a shear span of 630 mm. For beams with shear reinforcement, beams 2A-8 and 2B-3 were the controls for beams 2A-9 and 2B-4 respectively. For beams 2A-9 and 2B-4 the honeycombed zone was located at the centre of the high shear region.

For series 1A and 1B, the strength of the normal and honeycombed concretes were each kept constant throughout the tests. The only variable in both series was the location of the honeycombed zones within the high shear zone. The strength of the normal concrete in both series 2A and 2B were the same as series 1A and 1B respectively. The strength of the honeycombed concrete in series 2A and 2B was kept the same as in series 1B. This results in a lower ratio of strength of the honeycombed concrete to the normal concrete for series 2A compared to series 1A.

All the beams, except for beams 2A-4, 2A-5.1, 2A-5.2, 2B-5, 2B-6.1, and 2B-6.2, were prepared in such a way that a test could be done twice, once on each end. This could be carried out because those beams were tested with a short shear span. Those 6 beams that were exceptional were tested with a shear span of 630 mm, thus each of them could only be tested once.

There were three beams for 2A-1 [MM] series: beams 2A-1.1, 2A-1.2 and 2A-1.3, and they were cast simultaneously. The three beams were prepared in order to examine if there was any difference between a precast and cast in-situ honeycombed zone and also to compare the behaviour if the honeycombed zone was replaced by a void, simulating the most extreme situation. Beams 2A-1.1 and 2A-1.2 have an identical size and location of honeycombed zone i.e. both were identical to beam 1A-2 [MM], but were cast with two different methods of honeycombed concrete inclusion. Beam 2A-1.1 was cast with a normal inclusion method and beams 2A-1.2 used a precast honeycombed block. Details of casting methods will be discussed later. In beam 2A-1.3 the void section was located at the location of the honeycombed zone as in the other two beams.

Beams 2A-8, 2A-9 and beams 2B-3 and 2B-4 were beams with shear reinforcement. Two 8 mm diameter high yield steel bars were provided at the top of the beam in order to hold the shear reinforcement. Shear reinforcement was provided at 100 mm spacing using 3 mm diameter mild steel bar. Beams 2A-8 and 2B-3 were the control beams for beam 2A-9 and 2B-4 respectively and contained no honeycombed zone. **Figure 3.3** shows the details of the reinforcement arrangement in beams 2A-8 and 2A-9 and beams 2B-3 and 2B-4. The arrangement of shear reinforcement was made in such a way that the honeycombed zone could be placed in between the shear reinforcement.

3.3.2 Casting and Curing

In series 1A, 8 beams including a control beam with no honeycombed zone were cast in 4 pours and tested. In series 1B, originally 6 beams were cast in 3 pours and tested. Tests then needed to be repeated due to the inconsistent behaviour shown by some of the beams and some of them could only be tested at one end instead of two ends. The 3 extra beams were cast in pours 4 and 5. In pour 4 only one beam representing beam 1B-1 on one end and beam 1B-2 on the other was cast. In Pour 5, 2 beams, 1B-3 and 1B-4 were cast for tests to be repeated.

A total of 8 pours were carried out in series 2A and 2B, and 19 beams were cast. The beams however were not cast in sequence according to their series number. For example beams 2A-2 and 2A-3 were cast in pour 1 and beams 2A-1 series were cast in pour 5. The original pouring program was rescheduled because for beams 2A-1 to be cast, a special mould for precast honeycombed concrete needed to be fabricated and this took some time. As a result, pouring was continued with other beams. **Table 3.4** shows the listing of the pour number and beams cast for each series.

All beams were cast in pairs using steel moulds, except for beams 2A-1 series, 2A-4 and 2A-5 series, and 2B-5 and 2B-6 series in which 3 beams were cast simultaneously. Specially fabricated moulds made from a thin metal sheet were used to form the square honeycombed zone in the beam. At each time of pouring the special moulds were clamped at the designated positions to the steel mould so that their positions were secured throughout the pouring and compaction stage. In order to place the inclined honeycombed zone in beams 2A-6 and 2B-1, a pair of stiff 1.5 mm thick steel plates was fabricated so that they could be fitted to the steel mould and fixed at a designated angle.

At the time of casting, both the normal and the honeycombed mixes were simultaneously mixed in two separate mixers. Throughout the casting process, care was taken to avoid excessive compaction to the honeycombed zone. The compaction of normal mix was carried out using a small 1 inch diameter poker vibrator.

The sequence of pouring and the thickness of each layer depended on the position of the honeycombed zone. For beams without shear reinforcement and having a square honeycombed zone located at the bottom section of the beam, the honeycombed mix was first poured into the special mould to the level specified. A steel bar of 10 mm diameter was used to level the honeycombed mix. The normal mix was then poured into the steel mould and compacted in 3 layers up to the top level of the beam. The normal mix in the special mould that overlaid the honeycombed mix was compacted with mild compaction using the vibrator. Precautions were taken to avoid contact between the poker vibrator and the special honeycombed mould. Special care was also taken so as not to over-vibrate the concrete surrounding the special mould as it might affect the honeycombed concrete.

For all the other beams without shear links and with a square honeycombed zone, the normal mix was first poured and compacted up to the bottom level of the respective honeycombed zone. The honeycombed mix was then poured into the special moulds and leveled to the required level. After that, the normal mix was poured and compacted as for the above beams.

Once pouring and compaction were completed, the special moulds were unclamped and slowly pulled out from the main steel moulds, using a handle attached to them. In order to ensure that the honeycombed zone would not be disturbed, the special moulds were maintained in a vertical direction during the pulling out process. After that, a slow compaction was done using a trowel and the top surface of the beam was leveled. **Plates 3.1** to **3.4** show the special mould to place the square honeycombed zone, the sequence of casting a beam and the placement of honeycombed concrete.

For beams with an inclined honeycombed zone (beams 2A-6 and 2B-1), the honeycombed mix was first poured into the area confined by the two inclined stiff plates acting as formwork in three layers up to the top level of the beam. The honeycombed mix needed to be pushed into the mould with a steel bar as the formwork was quite narrow. The normal mix was then poured into the mould and compacted in three layers. Proper compaction was required and concrete needed to be pushed into the narrow area underneath the inclined honeycombed mould in order to ensure no voids formed in that area. Once pouring was completed, the stiff plates were unclamped and pulled out of the beam and the inclination of the plates were maintained at 45^{o} during the pulling out. **Plate 3.5** shows the arrangement of the mould.

For beams in 2A-1 series in which three beams were simultaneously cast and two different methods of honeycombed inclusion were used and one beam with a void, their casting is described here. No further description for the beam with the normal inclusion is required as it was as described above. In order to prepare for beam 2A-1.2 (with a precast honeycombed zone), two special steel moulds were fabricated in order to cast the precast honeycombed blocks with a dimension of 60 x 60 x 100 mm.

The casting of the precast honeycombed zone was done a day earlier before the casting of beams took place. Before the casting of beam 2A-1.2 began, the two precast honeycombed blocks were already demoulded from the special steel moulds. The exact locations of the precast honeycombed blocks were clearly marked on the steel mould. The normal mix was first poured up to the bottom level of the position of the honeycombed zone and compacted accordingly. The two precast blocks were then placed at their designated positions at both ends of the beam and secured in place by a wire which in turn tied to the steel mould. The normal mix was then poured and compacted as usual up to the top level of the beam. Since the position of the precast block could be reached by fingers, its exact location could be checked in order to ensure it was at the right position and was not moved during the compaction.

In preparing beam 2A-1.3 (beam with a void), polystyrene was cut to the size of honeycombed block, i.e 60 mm x 60 mm x 100 mm, with a little extra in length to ensure it could be tightly fitted into the steel mould. After placing the two polystyrene blocks into the mould at the designated positions, the normal mix was then poured into the mould in 3 layers and compacted. As for beam 2A-1.2, the position of the polystyrene could be checked by using fingers.

For beams with shear links and containing honeycombed zones, beams 2A-9 and 2B-4, the casting procedure is described below. As mentioned before the shear links were arranged in such a way that the special honeycombed mould could be inserted into the steel mould. Two gaps, one at each end at the designated locations were provided by the top bars. The special moulds were inserted into the steel mould through these gaps and securely clamped to the steel mould. The pouring was done the same as for the other beams. Once the honeycombed mix was placed into the mould, the normal mix poured and compacted up to the top level of the beam. However, at the location of honeycombed mould, the normal mix was poured just to cover the honeycombed concrete, but the gaps were not covered by the normal mix. The special moulds were pulled out and the gaps of the top bar were joined together using two pairs of short steel bar of the same diameter as the top bars. The normal mix was then poured to cover this area and compacted.

Once the pouring completed, all the beams and control specimens were covered with polythene sheets and left for 48 hours in the room temperature before demoulding was carried out. It was decided to demould the samples after 48 hours because there was a need to wait for the honeycombed concrete to be more intact.

After demoulding, all the specimens were placed on the floor underlaid by polythene sheets. Wet hessian covered the specimens and polythene sheets were used to prevent moisture from escaping. The specimens were allowed to cure at room temperature until one week before testing, at which time the wet hessian and polythene sheets were removed and all the specimens were allowed to dry before testing took place on the 28th day for series 1A and on the 21st day for series 1B, 2A and 2B.

3.3.3 Control Specimens

Control specimens comprising cubes, prisms and cylinders were cast together with the beams in each pour. The dimensions of all cubes were 100 mm. The dimensions of all prisms were 100x100 mm cross-section and 410 mm length. There were three types of cylinder moulds used. The first type was in inches: 4 inches (101.6 mm) diameter and 10 inches (254.0 mm) length. The second type was also in inches: 4 inches diameter and 8 inches (203.2 mm) length. The third was in mm: 100 mm diameter and 200 mm length.

For each type of mix of each pour, 3 cubes were cast in order to obtain the compressive cube strengths. Except for pours 4 and 5 of series 1B, in which only cubes were prepared, at least 2 cylinders and two prisms were cast for each type of mix for all the other pours in series 1A and 1B. The control specimens for the normal mix were compacted using the same 1 inch poker vibrator that was used to compact the beam specimens. For honeycombed specimens hand rodded compaction was applied. Visually it was checked that each particle was evenly coated with cement paste.

The values of elastic modulus were determined using cylinders, by loading the cylinders up to approximately 70% of their estimated crushing strength and the loads and the corresponding deformation were recorded. For the whole series of tests (1A, 1B, 2A and 2B), the elastic moduli were only obtained from series 1A and 1B. Deformations were measured using a 100 mm demec gauge. Studs were fixed at quarter points around the cylinders. After that, the cylinders either tested to obtain the cylinder splitting strength or cylinder compressive strength.

In series 2A and 2B, more data were required in order to establish the relationships between cube and cylinder compressive strengths for the normal and the honeycombed mixes. This was needed to be used in the plasticity analysis. Prisms and cylinders for splitting were also prepared in both 2A and 2B series in almost every pour.

Some of the honeycombed cubes were tested with MGA pads(58), and the effect of rate of loading on strength was also examined. The results are discussed in **Section 4.2** of **Chapter4**.

3.4 TESTING

3.4.1 Preparation of beams

A thin layer of white emulsion was applied to the beams for easy observation of the crack propagation. The use of emulsion could be the reason that the cracks could not be detected earlier as discussed in **Section 4.5** of **Chapter 4**.

3.4.2 Ultrasonic Pulse Velocity

Before applying the emulsion, ultrasonic pulse velocity through the beam width was measured to compare the velocity in the honeycombed and normal concretes. For beams in series 1A and 1B, measurements were taken at the middle of the honeycombed zone and also on the surrounding concrete at the top, bottom, right and left positions of the honeycombed zone.

For series 2A and 2B beams, a small exponential probe (**Plate 3.6**) was used to get more detailed pulse velocity readings around the honeycombed zone. Measurements were taken on selected points of a 15 mm grid around the area of the honeycombed zone in specimens 2A-1.1, 2A-2, 2A-3 and 2A-5. Those points are shown together with the velocities in **Figures 4.2(a)** and **(b)** in **Chapter 4**.

3.4.3 Test Set-up

The set-up of the tests was as shown in **Figure 3.4** and **Figure 3.5**. **Figure 3.4** shows the set-up of beams with 350 mm shear span. The distance between supports was 1.1 m and the short shear span was 0.35 m. At support A, the beam rested on 9 mm thick 50 mm wide steel plate, which then sat on a 37 mm diameter roller. The roller was then supported by a 19 mm thick and 75 mm wide steel plate. For support B, the upper and lower steel plates were of the same dimensions as in support A. In between the plates there was a half-roller. Both support systems rested on firm supports which comprised massive steel section. In between the loading jack and beam, there was a 35 mm diameter roller which rested between two 9 mm thick and 50 mm wide steel plates.

Figure 3.5 shows the set-up of beams with the shear span of 630 mm. The distance between supports was 1.9 m. The arrangements of both supports A and B were the same as described above.

The point load was applied through a jack with integral load cell. The jack was connected to a Mand Testing Machine. The jack was set to the 0-100 kN range and load could be monitored to an accuracy of 0.1 kN. Load increments were monitored by the control panel. Load control was used at the initial stage of testing up to the stage where beams showed a sign of reaching the ultimate failure, after which displacement control was used.

3.4.4 Experimental Monitoring

Deflections were measured using mechanical dial gauges with 0.01 mm accuracy. Two measurements were taken in series 1A and 1B, one under the point load and the other at the middle of the longer shear span. In series 2A and 2B, deflections were monitored at three points; the first point under the load, and the other two were at the supports at which deflections were measured at the top surface of the beams. The formation of the cracks were closely observed and ends of cracks were marked with corresponding loads.

Deflection readings were taken for each load increment which varied between 2 kN and 5 kN depending upon the anticipated failure loads for series 1A and 4 kN for series 1B, 2A and 2B, until the stage where the beams showed the sign of an ultimate failure. After that stage, load was continually increased until the beam failed. The failure mode was recorded after failure occurred.

Size (mm)	% passing
9.5	75.55
8.0	40.30
6.7	19.31
5.0	5.96
3.35	0.52

TABLE 3.1 Sieve analysis of 10 mm aggregate

Size (mm)	% passing
5.0	93.40
3.35	80.85
2.36	72.75
1.18	65.00
0.60	60.85
0.30	46.85

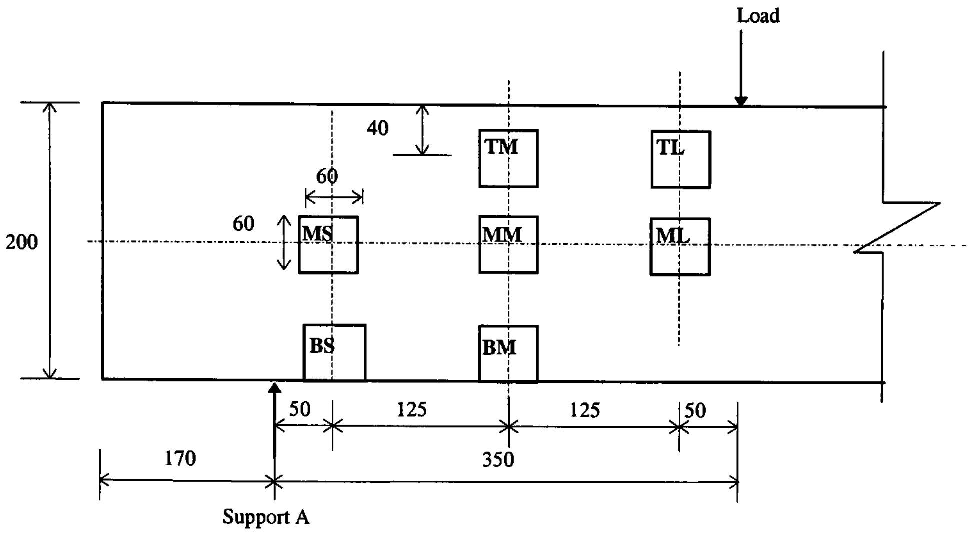
TABLE 3.2 Sieve analysis of sand

Materials	NORMAL		HONEYCOMB	
	Series 1A, 2A (kg)	Series 1B, 2B (kg)	Series 1A (kg)	Series 1B, 2A, 2B (kg)
Ordinary Portland cement	460	310	460	310
Water	230	176	230	176
10 mm aggregate	1092	1192	1680	1938
Sand	588	746	-	

TABLE 3.3 Material proportions of normal and honeycombed mixes (1 m^3)

Series No.	Pour No.	Beams cast
1A	1	1A-1, 1A-2
	2	1A-3, 1A-4
	3	1A-5, 1A-6
	4	1A-7, 1A-8
1B	1	1B-1, 1B-2
	2	1B-3, 1B-4
	3	1B-5, 1B-6
	4	1B-1R/1B-2R
	5	1B-3R, 1B-4R
2A, 2B	1	2A-2, 2A-3
	2	2A-4, 2A-5.1, 2A-5.2
	3	2A-8, 2A-9
	4	2B-3, 2B-4
	5	2A-1.1, 2A-1.2, 2A-1.3
	6	2B-5, 2B-6.1, 2B-6.2
	7	2B-1, 2B-2
	8	2A-6, 2A-7

TABLE 3.4 Series number, pour number and the beams cast



Positions	Beam No. for each series			
	Series 1A	Series 1B	Series 2A	Series 2B
MM	1A-2	1B-2	2A-1, 2A-5, 2A-7, 2A-9	2B-2, 2B-4, 2B-6
MS	1A-3	1B-3	2A-2	-
ML	1A-4	1B-4	2A-3	-
TM	1A-5	-	-	-
TL	1A-6	1B-5	-	
BS	1A-7	1B-6		
BM	1A-8			-

Notes: *-Beams 2A-5 and 2B-6 have their shear span of 630 mm*
-Beams 2A-7 and 2B-2: honeycombed zone is 90 x 90 mm
-Beams 2A-9 and 2B-4 contain shear reinforcement

FIGURE 3.1 The locations of honeycombed zone and the reference identification for beams with square honeycombed zone

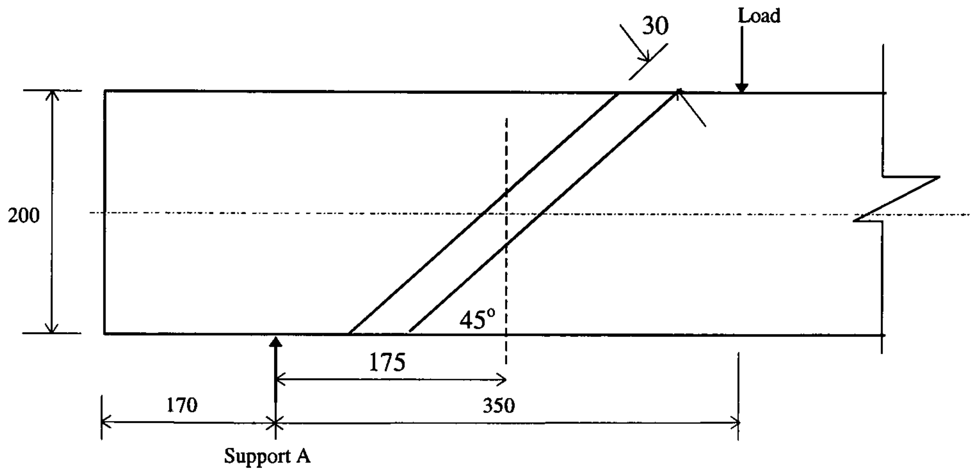


FIGURE 3.2 Honeycombed zone for beams 2A-6 and 2B-1

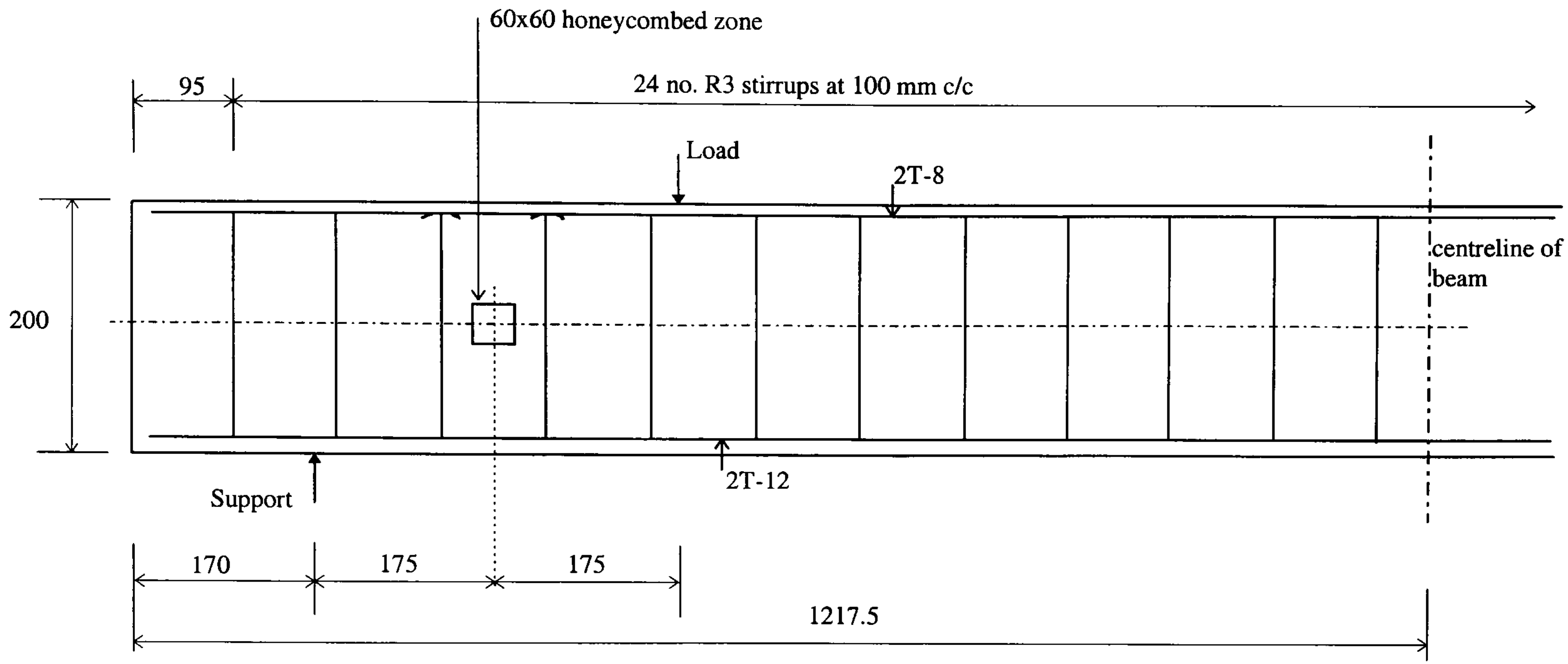


FIGURE 3.3 Details for beams 2A-8 and 2B-3 (without honeycombed zone) and beams 2A-9 and 2B-4 (with honeycombed zone)

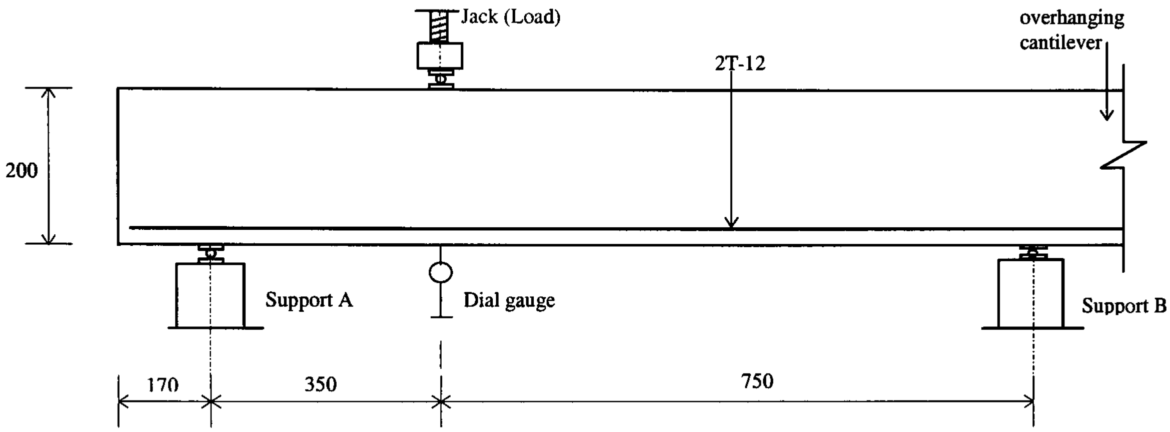


FIGURE 3.4 Test set-up (all beams except beams 2A-4, 2A-5, 2B-5 and 2B-6)

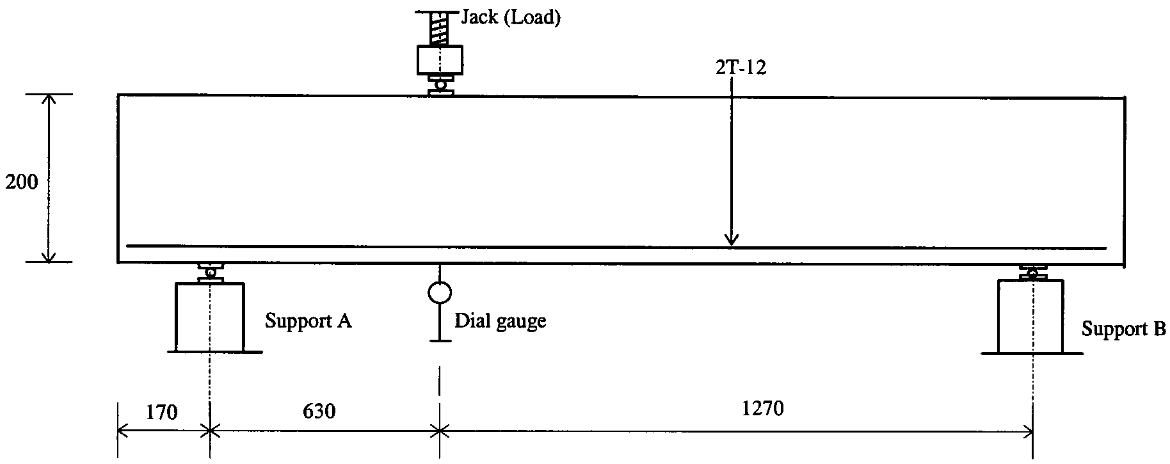


FIGURE 3.5 Test set-up for beams 2A-4, 2A-5, 2B-5 and 2B-6

Plate 3.1: The steel mould and the special mould for honeycombed zone

Plate 3.2: Honeycombed concrete mix poured into the special mould

Plate 3.3: The normal and honeycombed concrete mixes in the mould

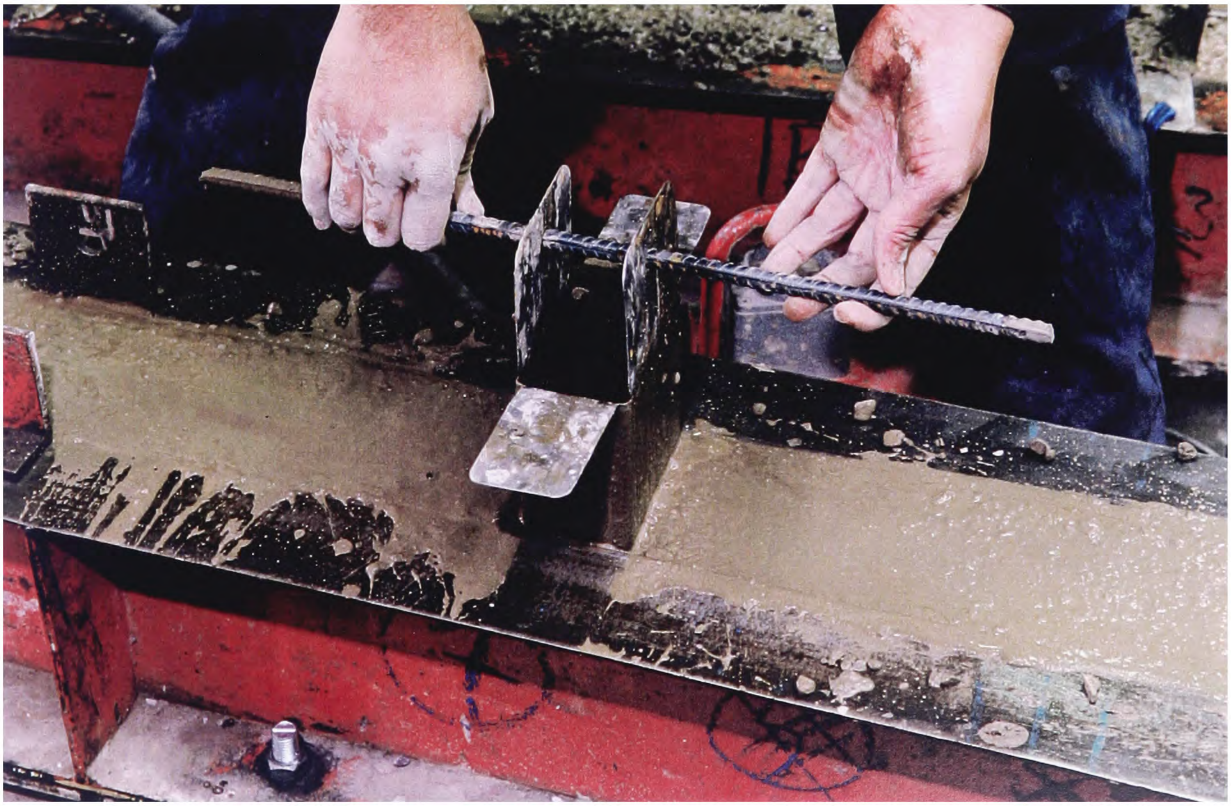


Plate 3.4: The special mould being pulled out from the steel mould

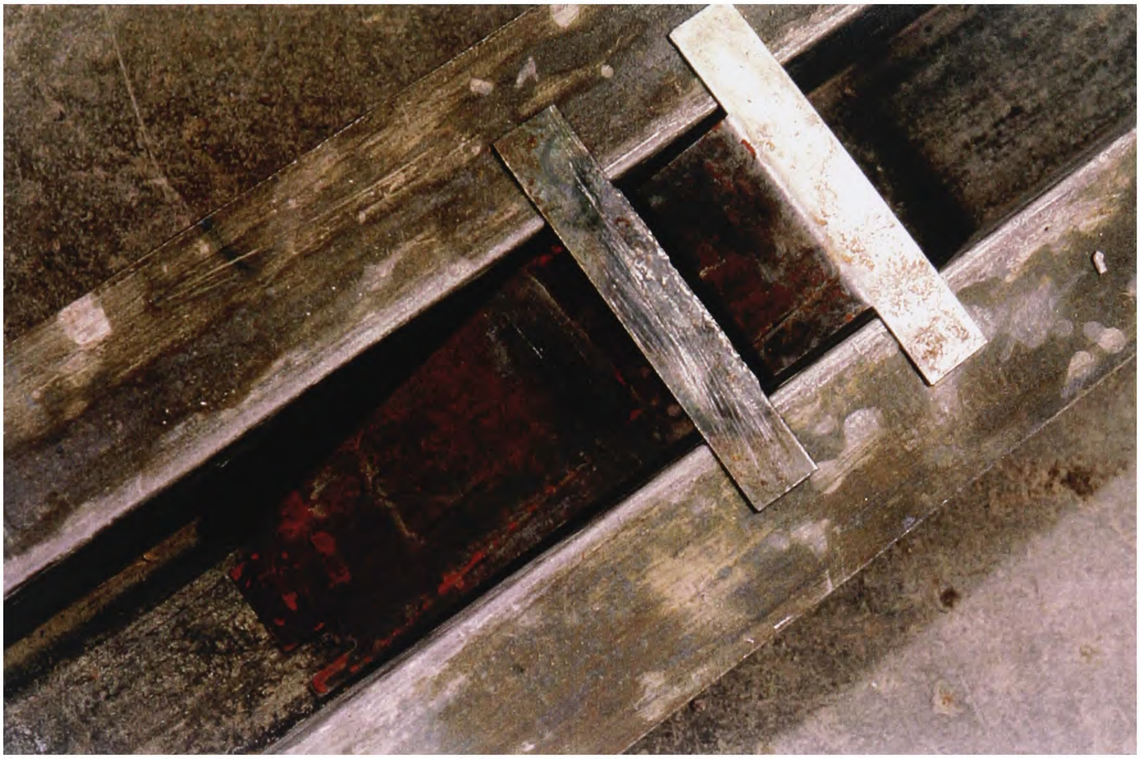


Plate 3.5: The special mould for construction joint

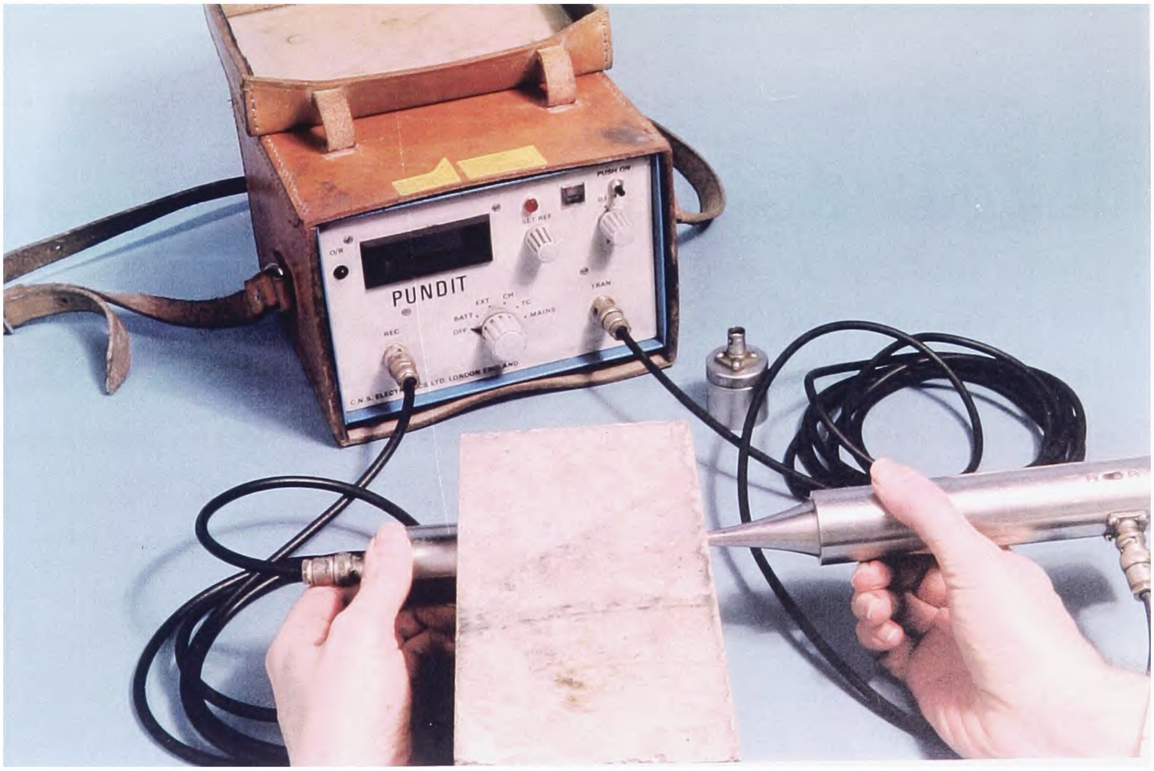


Plate 3.6: The exponential probe for measuring ultrasonic pulse velocity (UPV)

CHAPTER 4

EXPERIMENTAL RESULTS AND DISCUSSION

4.1 INTRODUCTION

The results of the experimental work are presented and discussed. The effects caused by the honeycombed zone on the shear behaviour and shear capacity of beams are examined and discussed. Close examinations are made of the various modes of shear behaviour displayed by the honeycombed beams and they are compared with non-honeycombed beams. The analysis of results are made in terms of diagonal cracking loads and the ultimate shear capacities. Comparisons are made with the control beams and also between honeycombed beams but with different locations of the honeycombed zone.

Other aspects of the test results are also discussed including the results of the properties of concrete obtained from control specimens such as cubes, cylinders and prisms, especially the data from the honeycombed specimens. Besides that, the results from the ultrasonic pulse velocity measurements meant to locate the honeycombed zone are also presented and discussed.

4.2 CONCRETE PROPERTIES

In general the properties of both normal and honeycombed concretes obtained from the control specimens were consistent and met their target values. There were however a few specimens especially of honeycombed concrete which produced inconsistent and strange results. These will be discussed further below.

All results of the control specimens are tabulated in **Table 4.1(a)**, **(b)**, **(c)** and **(d)** for series 1A, 2A, 1B and 2B respectively.

4.2.1 Cube Compressive Strength of the Normal Concrete

In all cases the cube strengths of the normal concrete were determined using test data of three cubes, except for pour number 4 of series 2B, in which only 2 cubes could be tested because the third cube was damaged during the curing stage. Examining all the data in every pour in **Tables 4.1(a)** to **(d)**, all of them are relatively consistent, except for the second cube of pour 4 in series 1B. The strength of that particular cube was quite low compared to the other two cubes. This could be due to inconsistent compaction, or due to improper curing. Note that compaction was carried out using a poker vibrator. At the time of curing there was a possibility that the particular cube was not properly covered with the wet hessian and the polythene sheet. However, the consistent trends showed by the data of other pours suggest that the results for pour 4 of series 2B could be acceptable.

The average compressive cube strength of the normal concrete, f_{cu}, for series 1A and 2A, were 50.5 N/mm^2 and 47.2 N/mm^2 respectively. The average strength for series 1B and 2B were 33.5 N/mm^2 and 35.6 N/mm^2 respectively. The results obtained were close to the target strengths of 50 N/mm^2 and 30 N/mm^2 respectively, which had been set before the experiment.

4.2.2 Cube Compressive Strength of the Honeycombed Concrete

As anticipated from the discussion in **Section 3.2.3** of **Chapter 3**, there were a few pours which produced inconsistent results of cube strength. However, generally the overall results were consistent and acceptable. The average strengths of the honeycombed concrete, f_{cuh} , for series 1A and 2A were 23.4 N/mm^2 and 13.1 N/mm^2 respectively. For series 1B and 2B the average strengths were 11.9 N/mm^2 and 13.0 N/mm^2 respectively. The strengths were close to the targets of 20 N/mm^2 for series 1A and 10 N/mm^2 for other series, which had been set before the experiment.

Note that the average strength for series 1A was based on the results of pours 2, 3 and 4. The data from pour 1 were very low and quite surprisingly all the three cubes in that particular pour produced consistently low results. All the honeycombed concrete cubes in this particular pour were tested using MGA pads. However the use of MGA pads was not the cause of the exceptionally low strength (to be discussed later in this section). The low strength could be due to inadequate compaction of the cubes. The acceptable results from the cylinders and also from the prisms of the same pour indicated that the mix was not mistakenly prepared.

There were cubes which were found to be very weak and brittle to the extent that their strength could not be recorded by the testing machine. These occurred in pour 3 of series 2A, pour 3 of series 1B and pour 7 of series 2B. As a result of that, the strengths of pour 3 of series 2A and pour 3 of series 1B, were based on one cube only. In pour 7 of series 2B, the average cube strength was determined based on results of the two cubes. In other pours variations occurred in the strength of cubes. The problems found could be attributed to the rough cube surface of the honeycombed concrete and the variations in the degree of compaction. Note that for the honeycombed mix only minimum hand compaction was applied. It could also be associated with the nature of the honeycombed mix which has been discussed in **Section 3.2.3** of **Chapter 3**.

In order to check the degree of consistency of compaction and the relationship between the strength and the weight, cubes from pour 1, 2 and 3 of series 2A and cubes from pour 4 of series 2B were weighed before testing. The average weight of all cubes in series 2A and series 2B were 1.8 kg.

The results showed that consistent compaction could be achieved in honeycombed concrete. Results from pour 3 of series 2A demonstrated that, for honeycombed concrete, without sand, there could be problems of adhesion of the aggregate. All the 3 cubes were more or less of the same mass, but 2 of the cubes had no strength recorded by the testing machine. This indicated that probably the cement paste was not evenly distributed within the mass of the honeycombed mix and resulted in a very weak concrete in one of the cubes.

The use of MGA pads was developed to eliminate friction between the platens of the testing machine and the cube(58). Initially it was used in the current experimental work for cubes in pour 1 of series 1A, and the results were as shown in **Table 4.1(a)**. The results of that particular test showed that the MGA pads might have caused the low strength of those cubes. In order to check those results, the MGA pads were again used in the testing of the honeycombed cubes of a trial mix. The results tabulated in **Table 4.2** showed that the MGA pads produced what was expected. The strength of cubes with the MGA pads was about 86% of the strength of cubes without the MGA pad. The results from the trial mix confirmed that the MGA pads were not the cause for the low strength of the honeycombed concrete in series 1A. Another cube in pour 2 of series 2A was also tested with the MGA pads and the results showed that its usage would not cause an adverse effect on the strength of the cube.

It was also initially thought that the rate of loading applied to the specimen might have had an effect on the strength of the honeycombed concrete. Cubes in pour 2, 3 and 4 of series 1A were subjected to various loading rates. The results showed that the difference in loading rate had no effect on the cube strength.

4.2.3 The Elastic Young's Modulus

The average initial modulus of elasticity, E_c, for both concretes were determined from a plot of stress-strain curves of their respective concrete cylinders which were obtained from series 1A and 1B tests only. The curves are shown in **Figure 4.1(a), (b), (c)** and **(d)**. The values of the initial moduli of elasticity are tabulated in **Table 4.3**. Note that the higher value in series 1A is consistent with the higher strength of honeycombed concrete.

4.2.4 Other Properties of Concrete

Other properties obtained from the experimental work were the flexural strength from prisms, the cylinder compressive strength and the cylinder splitting strength. Those properties were obtained from some of the pours. All the values are tabulated in **Table 4.1(a)**, **(b)**, **(c)** and **(d)**.

The flexural strengths of prisms were obtained based on 2 prisms from almost every pour. The results were consistent for the normal concrete. For the honeycombed concrete, in general the results for each pour were also relatively consistent.

Note the difference in the dimension of cylinders in series 2A and 2B. Most of the cylinder compressive strength data were obtained from series 2A and 2B. No data were available from series 1B, and 2 sets of data were obtained from series 1A. The test data showed that some variations occurred in the cylinder strength. It occurred both in the normal and honeycombed concretes. It is considered that this was caused by poor compaction of some of the large number of control specimens which had to be manufactured. Some results were ignored because they were too low to be considered. It is acknowledged that subjective judgement on what to ignore had to be made. This happened to the normal concrete in pour 1 and pour 3 of series 2A.

It should be noted that it is possible to compare the cylinder strength with the MGA cube strength only for pour 2 of series 2A. The strengths were 7.07 N/mm^2 and 12.1 N/mm^2 respectively.

For cylinder splitting tests, the results from the specimens of normal concrete showed that in general they were relatively consistent. For the honeycombed concrete, there were cases where only half of the specimen split during the test. As a result of that, they gave a very low splitting strength. Refer to pour 1 and 3 of series 1B. This could be attributed to the irregular surface of the specimen because of the nature of the honeycombed concrete.

4.2.5 Cube and Cylinder Strength Relationship

The relationship between the cube compressive strength and the cylinder compressive strength is required for the plastic analysis (to be discussed in **Chapters 5** and **6**). For the normal concrete, the relationship was established using the set of data in pours 1 and 2 of series 1A, pours 1, 2, 3 and 5 of series 2A, and pours 4, 6 and 7 of series 2B. The ratios of the cylinder strength to the cube strength from those sets of data varied between 0.68 and 0.86. The average ratio was 0.76.

For the honeycombed concrete, the sets of data from pour 2 of series 1A and pours 1, 2, 3 and 5 of series 2A were taken in order to get the cube-cylinder compressive strength relationship. The data from pour 4 of series 2B were ignored because the results of the honeycombed strength was greater for the cylinder and the cube strength was lower than for other pours. The range of cylinder to the cube strength ratios were from 0.52 to 0.84. The average ratio was found to be 0.66. The lower ratio for the honeycombed concrete is consistent with the finding of Malhotra(57), who suggested that it could be due to its relatively low shear strength.

4.3 ULTRASONIC PULSE VELOCITY (UPV)

The ultrasonic pulse velocity measurement for series 1A and 1B specimens are tabulated in **Table 4.4(a)** and **(b)**. The figures in the table show that the readings at the honeycombed zones, taken at the middle of the zone, for all the beams were consistently lower than the surrounding concretes. Neville(59) states that for the same element or in situations where materials are of the same type, the ultra-sonic pulse velocity reading can be used to indicate the level of concrete strength and/or the presence of voids. The readings in the table thus indicate that for all the beams prepared, concrete of lower strength and/or with voids, in this case a honeycombed zone, was present at the intended locations.

More detailed investigations of the exact location, size and the shape of the honeycombed zone using the ultrasonic pulse velocity measurement were carried out randomly on beams in series 2A. **Figure 4.2(a)** and **(b)** show the pulse velocity measurements in specimens 2A-2, 2A-3 and specimens 2A-1.1 and 2A-5 respectively. The values shown on the points of a 15 mm grid in each specimen are the corresponding ultrasonic pulse velocities measured at that location. The darkened lines shown in the diagram are the intended boundaries of the honeycombed zone. The velocities measurements taken showed that in general the velocities within the area of the honeycombed zone were lower than the surrounding concrete. The variations in the velocity measurement between the honeycombed area and the normal concrete indicated that the honeycombed zone were generally at the intended location. It also showed that the technique used to place the honeycombed zones seems to work. The measurement shown in beams 2A-2, at the top area of the central vertical lines, reflected that a poorly compacted normal concrete zone could be formed on top of the area of the honeycombed zone. This could occur because minimum compaction carried out in that particular area in order to avoid disturbance to the honeycombed zone.

Results from the ultrasonic pulse velocities show that it was possible by using a simple non-disruptive technique to reliably locate zones of poorly compacted concrete such as honeycombed zones.

4.4 THE RESULTS AND ANALYSIS OF THE EXPERIMENTAL DATA

For clarity in the following discussion, especially in order to be able to visualise the location of the honeycombed zone for any particular beam mentioned, the code locations as described in **Section 3.3.1** of **Chapter 3** are applied.

Also it should always be noted that the average strengths of normal concrete for beams in series 1A and 2A were 50.5 N/mm^2 and 47.2 N/mm^2 respectively. For series 1B and 2B the values were 33.5 N/mm^2 and 35.6 N/mm^2 respectively. The average strength of honeycombed concrete for beams in series 1A was 23.4 N/mm^2. For series 2A, 1B and 2B the average strengths were 13.1 N/mm^2, 11.9 N/mm^2 and 13.0 N/mm^2 respectively.

In terms of the ratio of the honeycombed to the normal concrete strength, for beams in series 1A and 2A the average ratios were 0.46 and 0.28 respectively. For series 1B and 2B the average ratios were 0.36 and 0.37 respectively.

4.4.1 General description of the behaviour of beams

For all series of the experiments, the test beams generally produced the expected results, except for a few specimens in series 1A, which, at the ultimate load, failed in shear but with the evidence of a torsional effect. This phenomenon of a torsional effect was observed when the beams failed in either the short shear span or in the long shear span. It was noticed during the test that, this occurred because the particular specimens were not properly levelled on their supports. This was later rectified. After the rectification, one specimen of series 1A and 4 specimens in series 1B, failed in shear on the long shear span and no evidence of torsional effects was observed. This phenomenon will be discussed in **Section 4.7.1**.

No data were available for specimens 1A-7(a) and 1B-2(b) because mistakes occurred while loading was being applied to those specimens. Tests could not be carried out on specimens 1B-3(b) and 1B-4(b)R, because the portion of those specimens that was intended to be supported in the second test was damaged while tests were carried out on their other first ends.

No major problem was encountered for series 2A and 2B tests. All specimens could be tested as planned and provided the required data. There were specimens in these 2 series which ultimately failed on the long shear span.

Table 4.5(a) to **4.5(d)** give the summary of behaviour of the specimens in each series observed in the tests. The diagonal cracking load shown in the table is the load that caused the diagonal cracking, which was thought would cause an ultimate failure, to propagate within the central zone of the neutral axis of the beam. The statement in the parenthesis in the column 'first flexural crack', below the first flexural cracking load, indicates the severity of the development of flexural cracking until the beam reached ultimate failure.

The plots of crack mapping for each specimen are shown in **Figures 4.3(a), (b), (c)** and **(d)** for series 1A, 2A, 1B, and 2B respectively. In those figures the values shown along the cracks are the loads in kN when the crack formed to the position of the value.

In general, all the short shear span specimens, with a shear span ratio of 2.0, of all series behaved as typical short and medium span beams designed to fail in shear. Those specimens with a shear span ratio of 3.5 in series 2A and 2B demonstrated a typical behaviour of a long shear span beam in shear, in which they failed almost immediately after the formation of the diagonal crack from a flexural crack. However, the unpredictable nature of shear behaviour was also observed. There were identical specimens which behaved differently.

Two types of typical diagonal crack formation were observed: an independently formed diagonal crack; and as an extension of a flexural crack. Due to the presence of the honeycombed zone, some specimens failed immediately after the formation of a diagonal crack, and some had a relatively high reserve of strength before reaching the ultimate failure. They either failed through shear-tension, a failure of the reinforcement anchorage, shear-compression or a combination of them and in some cases specimens failed due to the buckling of the top concrete 'arch'. The effect of the aggregate-interlock action was also observed in the form of local crushing at failure in some of the specimens.

The results from the tests showed that the presence of a honeycombed zone at certain locations within the high shear zone of a beam without shear reinforcement could significantly affect its shear behaviour. At certain locations honeycombed concrete could modify the path and the profile of the diagonal crack, accelerate its formation, determine the mode of failure and could reduce the ultimate shear capacity of the beam.

With regard to the method of placing the honeycombed zone into the beam, it was found that the technique used throughout the current experimental work was acceptable. Throughout the tests there was no evidence that the mode of behaviour of a honeycombed beam at any stage was determined or influenced by the line separating the honeycombed zone and the normal concrete. With another method in which a precast honeycombed zone was placed in the beam, it was observed that the zone separating the two concretes influenced the mode of behaviour. There was evidence (to be discussed in **Section 4.6.1**) that the crack path was influenced by the discontinuity of the two concretes. With the first method, such behaviour was not observed.

It was also found that the behaviour exhibited by the beams with a honeycombed zone had no similarity at all with beams with a void. The mode of behaviour of both beams with a void can be seen from specimens 2A-1.3. It was thought earlier in the current study that a honeycombed beam could possibly be treated in the same way as beam with an opening. This and various aspects observed in the current experimental work will be discussed in detail in the following sections.

4.5 FLEXURAL BEHAVIOUR

Figures 4.4(a) and **(b)** show the plots of observed versus predicted flexural cracking loads for series 1A and 2A and series 1B and 2B respectively. The test values of the flexural cracking loads are as listed in **Tables 4.5(a)** to **(d)**. The predicted loads were calculated using the modulus of rupture obtained from 100 x 100 x 410 mm prisms, the values are shown in **Tables 4.1(a)** to **(d)**. Note that there were specimens which did not have the modulus of rupture from the test. In such situations, the modulus of rupture was obtained based on the cube strength of that particular pour using an average factor derived from the data available from other pours within that series. The factor was derived based on the relationship usually recommended by the design codes in which the tensile strength of concrete is assumed to be a function of the square root of its compressive strength.

Generally the observed flexural cracking loads are higher than the predicted values. From all the specimens in all series only 4 specimens have an observed cracking load very close to the predicted load. For series 2A, the point shown in the figure close to the line of equality is the data from specimen 2A-3(b)[ML]. In **Figure 4.4(b)** the three points on the line of equality represent data from specimens 1B-1(a), 1B-4(a)[ML] and 2B-5, a control beam with a shear span ratio of 3.5. The reason for the higher observed loads was that the crack could not be seen with the naked eye when it was initially formed.

Figures 4.5(a) to **(d)** show the load-deflection curves of all the specimens tested for all series. For series 1A, the curve of each honeycombed specimen is plotted together with control specimen 1A-1(b) in order to demonstrate the effect of honeycombed concrete. The curve for specimen 1A-1(a), another control specimen, showed the effect of the beam initially bedding in and twisting, which caused the beam to fail with evidence of a torsional effect. The same effect can also be seen on the load-deflection curve of specimen 1A-3(a)[MS], which also failed with evidence of a torsional effect.

For beams in series 2A, except for beams 2A-4 and 2A-5, beams with a shear span ratio of 3.5 and beams 2A-8 and 2A-9, with shear reinforcement, all the curves of the honeycombed beams are each plotted with the curve of specimen 1A-1(b) and they are presented in **Figure 4.5(b)**. Beam 2A-4 , a control is plotted with the honeycombed beam of 2A-5. Beams 2A-8, a control and 2A-9, a beam with a honeycombed zone, are compared together in **Figure 4.5(b)**.

The curves show that, in some of the beams the stiffnesses were significantly reduced once the diagonal crack formed. This occurred in beams 1A-2[MM], 1A-3[MS], 1A-4[ML], and 1A-6[TL] of series 1A. For series 2A this occurred in: specimens 2A-1.3, a beam with a void; 2A-3(a)[ML]; 2A-6, a beam with a construction joint; and specimen 2A-7(a)[MM]. It was found that those specimens possessed a relatively low reserve of strength. The behaviour shown by specimens 2A-1.3, a beam with a void, is to be expected. Looking at the relatively low ultimate capacity of specimens 1A-5(a)[TM] and 1A-7[BS], they probably would have behaved similarly if more data were available for the curves to be extended. However, the dial gauges had been removed at a lower load.

The phenomenon shown by the curves of specimens 2A-3(b)[ML] and 2A-7(b)[MM] was probably due to the fact that, after the formation of the diagonal crack, their stiffnesses were reduced. However, at one stage the beam might have turned into a strong tie-arch structure, with a combination of a large area of a compression concrete 'arch' and a strong anchorage resistance. This phenomenon of behaviour will be discussed further in **Section 4.7.1.**

In series 1A, beams which failed with a high ultimate load, including those beams which failed on the long shear span, exhibited that the presence of a honeycombed zone did not affect their flexural stiffness after the diagonal crack had formed. In series 2A, for beam 2A-1.1[MM], although both of its specimens failed at a relatively not very high load, the formation of the diagonal crack had no effect on the flexural stiffness. It was also found that for a long shear span beam, the load-deflection curves of the control specimen, 2A-4, and specimens with a honeycombed zone, 2A-5 were almost identical. The flexural stiffness of those beams reduced as soon as a flexural crack formed at about 12 kN. After that no further sign of reduction of flexural stiffness occurred when the diagonal crack formed in the beams. For beams with shear reinforcement it was found that the flexural behaviour of the control, 2A-8, and the beam with a honeycombed zone, 2A-9, were identical.

For series 1B, all the control specimens produced curves of almost the same character. So, the curve of 1B-1(b) was chosen in order to compare the flexural behaviour of the honeycombed beams of series 1B and 2B. They are shown in **Figures 4.5(c)** and **(d)** respectively.

In series 1B, the control specimens 1B-1(a) and 1B-1R showed the same phenomenon. These two specimens ultimately failed at relatively lower loads than the other control specimen, 1B-1(b). The curve of specimen 1B-1(b), which possessed a high reserve of strength, did not change even after a diagonal crack had formed. The other specimens in series 1B that showed the same pattern of behaviour, with a low reserve of strength, were 1B-5[TL] and 1B-6[BS]. It could be assumed that specimens 1B-2R[MM], 1B-3(a)R[MS] and 1B-3(b)R[MS] would have shown the same behaviour if the deflection data up to the ultimate failure had been recorded. However, the dial gauges had to be removed to prevent them from damage. For specimens which had a high reserve of strength, including specimens which failed on the long shear span, the formation of a diagonal crack did not reduce their flexural stiffnesses; for example as shown by specimens 1B-4[ML].

The presence of a bigger honeycombed zone such as in beam 2B-2[MM], with a zone of 90x90 mm had no effect on the flexural stiffness. As in series 2A, beams with shear reinforcement and beams with a long shear span showed no indication that the presence of honeycombed concrete affected their flexural stiffnesses. In series 2B, the flexural stiffness of the beam with a construction joint, 2B-1(a), seemed to be affected when compared with the control beam.

The above observations showed that the presence of a honeycombed zone in the shear zone of the beam had no direct and obvious effects on the flexural stiffness of the beam. The only cause that altered the beam deflection was the formation of the diagonal crack in certain beams as mentioned above. It was observed that the character of the diagonal crack that affects the flexural stiffness and is then most likely to lead to an early failure was the one that appears as a straight line and formed at a shallow angle. The diagonal crack in beam 1A-2 and shown in **Figure 4.3(a)** is a typical example of such a crack. This crack is different from the curved diagonal crack which had no effect on the flexural stiffness of the beam. An example of the curved diagonal crack can be seen in beam 1A-1 in **Figure 4.3(a)**.

The size of the honeycombed zone probably was not that significant to cause any disturbance to the flexural behaviour of the beams. Its presence even at the most critical section of the compressive zone, as in beams 1A-6[TL] and 1B-5[TL], did not have any flexural effect. Also, its presence at the bottom of the beam did not affect the overall bonding between reinforcement and concrete.

In practice this lack of effect on stiffness of a honeycombed zone should be emphasised to the assessing engineer. Load testing is a technique used for assessment. Flexural stiffness is easily measured and in many situations it is used to indicate the level of safety of a particular structure. From these tests it has been shown that a honeycombed zone, which later will be shown can seriously affect the shear capacity of a particular structure, cannot be detected from a measure of the flexural stiffness of the beam.

4.6 DIAGONAL CRACKING

As mentioned in **Section 2.2.2** of **Chapter 2**, diagonal cracking is one of the important parameters in the study of concrete beams in shear. In the following sections the discussion is divided into two: first the discussion on the formation of the diagonal cracking and second the discussion on the diagonal cracking load. The modes of diagonal cracking formation in the honeycombed beams are examined and comparisons are made not only with the control but also to compare the mode of formation of diagonal cracking with different locations of a honeycombed zone. Comparisons are made between the diagonal cracking load in the control and in the honeycombed beams and also between honeycombed beams with different locations of a honeycombed zone.

4.6.1 The Formation of Diagonal Cracking

The mode of formation of diagonal cracks and their corresponding loads in each specimen of all series of tests are shown in **Tables 4.5(a)** to **(d)**. The crack mapping can be referred to in **Figures 4.3(a)** to **(d)** for series 1A, 2A, 1B and 2B respectively. Generally, modes of diagonal cracking formation for identical beams were more consistent in series 1A and 2A specimens. In series 1B, inconsistencies in the mode of formation of the diagonal crack were exhibited by the control specimens together with beams 1B-3 and 1B-4 of series 1B and beams 2B-3 and 2B-4 of series 2B. The reasons for these are discussed in the following section.

It will be difficult to theoretically predict the mode of formation of the diagonal cracking in the beams. What can possibly be done is to carry out an elastic method of prediction. This simple approach will be far from accurate as it takes no account of the actual cracked behaviour in the shear zone. Thus, at this stage of discussion no attempt was made to predict the formation of diagonal cracking and its corresponding load.

4.6.1.1 Beams With a Shear Span Ratio of 2.0

The observations made from the tests showed that, for beams with a shear span ratio of 2.0, shear could be transferred through a concrete compressive strut which extended between the support and the point of loading. This was in agreement with many previous tests on shear. However, as also shown in the previous tests, many parameters are involved in determining the mode of shear transfer. In this investigation, it appears that the presence of honeycombed concrete and the strength of the normal concrete influenced the formation of diagonal cracking.

It was observed that a critical diagonal crack could be initiated either due to the splitting of the concrete compressive strut once the principal tensile stress exceeded the tensile strength of the concrete, or due to the extension of the flexural crack. Which of these occurs depends on the properties of the concrete and the flexural stiffness of the beam. Crack formation occurs under a complex shear and bending interaction within the shear zone.

In the control beams of series 1A, it was the flexural crack that first initiated and was then more dominant in the development of the diagonal crack. This was evidenced through the curved profile of the crack. In series 1B, the principal tensile stress in the strut was first to exceed the concrete tensile strength in the two control specimens, and thus initiated the formation of diagonal cracking in those specimens. The behaviour showed by the other control specimen indicates that, sometimes, shear behaviour could be difficult to predict, due to the close and complex interactions of bending and shear.

Note that although the compressive strengths of the normal concrete in series 1A and 2A were significantly higher than in series 1B and 2B, their respective flexural and splitting tensile strengths were relatively close to each other. It was these two parameters that could determine the mode of diagonal cracking, either flexurally or independently formed. This explains why it was difficult to predict which strength component would first be exceeded by the shear force and consequently dominate the mode of shear behaviour in the control beam. Under a complex stress interaction and in a complex cracked concrete environment either can be easily preceded by the other. The presence of a honeycombed zone could result in an even more complex stress distribution within the shear zone.

The tests carried out proved that zones of honeycombed concrete located along the potential path of the concrete compressive strut would accelerate or alter the mode of formation of the diagonal crack. In series 1A, the effects were clearly observed. Both ends of beam 1A-2[MM], initiated their diagonal cracks independently. This also occurred in specimen 1A-7(b)[BS]. A close examination of the crack in the specimen after failure shows that the crack profile was of a character of an independently formed diagonal crack, although observation during the test showed the crack was flexurally formed. Although a flexural crack initiated the diagonal crack in specimen 1A-6[TL], the crack however developed rapidly and its profile was not as curved as in the control specimens. This indicates that the flexural crack just triggered a crack that was about to be formed through the splitting of the concrete strut within the honeycombed zone. Note that in beam 1A-6, the honeycombed zone was located at the upper end of the potential strut path.

For series 1B, since the formation of a diagonal crack in the specimens without a honeycombed zone was controlled by the compressive strut, no change would be expected in the mode of formation of the diagonal crack if a honeycombed zone were present within the potential path of the compressive strut. This was shown by beams 1B-5[TL] and 1B-6[BS]. Although diagonal cracks were observed to form flexurally in specimens 1B-2(a) and 1B-2R, a close examination of the diagonal crack after the tests suggests that they were actually independently formed. Those cracks had a profile of a diagonal crack dominated by the splitting of the compressive strut, although the influence of the flexural crack could also be seen.

Further tests in series 2A and 2B showed the same effect of the honeycombed zone located along the potential path of the compressive strut on the formation of diagonal cracking. Specimens of beam 2A-1.1[MM], and specimens of beam 2A-7[MM], with a 90 x 90 mm honeycombed zone, i.e. about half the effective depth of the beam, had an independently formed diagonal crack. In series 2B, a diagonal crack was independently formed in both specimens of beam 2B-2[MM], with a 90 x 90 mm honeycombed zone.

In series 1A, a honeycombed zone located at other locations within the shear zone also affected the mechanism of shear transfer. This effect could be seen at all the locations of the honeycombed zone investigated. The diagonal crack formed independently in both ends of beam 1A-3[MS], although the effect of torsion was observed in specimen 1A-3(a)[MS]. The same independent cracking occurred in beams 1A-4[ML], and 1A-5[TM], with evidence of torsion found in specimen 1A-4(a)[TM]. The crack propagation and the crack profile shown by beams 1A-3, 1A-4 and 1A-5, showed that, although a honeycombed zone might not be located within the potential path of the concrete strut, it could be involved in the redistribution of stresses within the shear zone. For beam 1A-8[BM], the behaviour showed by both of its specimens clearly indicated that the honeycombed zone accelerated the formation of flexural cracking which in turn extended to initiate an early development of diagonal cracking.

The effect of the honeycombed zone as observed in series 1A was also observed in series 2A, in which a honeycombed zone located not in the path of compressive strut affected the mode of diagonal cracking formation. The diagonal crack in both specimens of beam 2A-2[MS], identical to beam 1A-3 in terms of the location of a honeycombed zone, and one specimen of beam 2A-3[ML], identical to beam 1A-4, was formed independently. In specimen 2A-3(b)[ML], the profile of the diagonal cracking suggests that the split in the compressive strut dominated its propagation although the crack was triggered flexurally.

For series 1B, the observations made seem to suggest that there was no significant effect of honeycombed zone located outside the path of the compressive strut on the shear behaviour of the beam. For beams 1B-3[MS] and 1B-4[ML], variations occurred in the modes of formation of the diagonal cracking. A flexural crack initiated the diagonal cracking in Specimen 1B-3(a). For specimen 1B-3(a)R, the crack shows that the crack could have been initiated flexurally. The pattern of the crack developed in specimen 1B-3(b)R was almost similar to specimen 1B-3(a)R, but was independently developed. However, the rapid development and the profile of the crack seem to suggest that it could be possible for flexure and shear acting together to develop the crack as happened in the control beams. The behaviour exhibited by specimens 1B-4(a) and 1B-4(a)R, in which the diagonal crack was flexurally developed, seems to suggest that the honeycombed zone might slightly alter the flexural stiffness of the specimen.

The inconsistent behaviour in series 1B beams could be attributed to the fact that, the shear transfer mechanisms were more interdependent; which one initiates the diagonal crack will depend on the relative magnitude of the shear forces carried by each, and their resistances.

The effects of various individual parameters on the formation of the diagonal crack are now considered.

(a) The Effect of the Strength of Honeycombed Zone

This effect can be examined by comparing beam 1A-2[MM] and beam 2A-1.1[MM], beam 1A-3[MS] and beam 2A-2[MS], and beam 1A-4[ML] and beam 2A-3[ML]. Each particular pair of beams had about the same strength of normal concrete but the strength of the honeycombed concrete in series 2A was lower than in series 1A. There was no clear evidence to suggest that the difference in the honeycombed concrete strength resulted in a different mode of diagonal crack formation. Although a difference was spotted between beams 1A-4 and 2A-3, with the former having an independently formed crack and the latter a flexurally formed crack, their respective crack profiles however, indicated a similarity, in which the crack propagation was dominated by the split in the compressive strut.

(b) The Effect of the Size of Honeycombed Zone

The comparison is made between a 60 x 60 mm honeycombed zone, about a third of the effective depth of the beam, to a 90 x 90 mm honeycombed zone, about half of the effective depth. Comparing the behaviour shown by beams 1A-2[MM] in series 1A and beam 2A-7[MM] in series 2A, and also comparing beam 1B-2 of series 1B and beam 2B-2 of series 2B, it was found that, in terms of the diagonal cracking formation there was no evidence to show that the bigger honeycombed zone inflicted a more significant influence on the shear behaviour of the beam. Examining closely each specimen of those beams, they had about the same pattern of diagonal cracking and each specimen showed the same typical variation regardless of the size of the honeycombed zone.

(c) Beams With Shear Reinforcement

The observations made in the tests showed that the control beams with shear reinforcement exhibited the same pattern of diagonal cracking formation in series 1A and 1B beams. Both specimens of beam 2A-8 developed their diagonal crack from a flexural crack, similar to the control beams in 1A series. Whereas in series 2B, it was the compressive strut that triggered a critical diagonal crack in one of the specimens of beam 2B-3, an almost similar trend was shown by the control beams in series 1B. The profile of the diagonal crack in specimen 2A-9(a), seems to be dominated by the independently formed diagonal crack rather than a flexurally formed diagonal crack. In specimen 2A-9(b), the crack profile extending into the honeycombed zone indicates its influence on the crack formation.

In specimen 2B-4(b), the diagonal cracking formed independently. In specimen 2B-4(a), it was found that the diagonal crack that led to the ultimate failure was independently formed. The phenomenon showed by the honeycombed beams indicates that the shear reinforcement had no significant influence in preventing the effect caused by the honeycombed zone in terms of the mode of formation of the diagonal cracking, although later it will be shown that the shear reinforcement significantly influenced the magnitude of the load at which the diagonal crack could be seen. This is what one would expect because the shear reinforcement controls the opening of the diagonal crack.

(d) Beams With a Precast Honeycombed Zone

The effect of the precast honeycombed zone can be examined from the crack mapping of specimens 2A-1.2 shown in **Figure 4.3(b).** It was found that the crack pattern of both beams could be differentiated from all other honeycombed beams. With the normal inclusion method, in which a fresh honeycombed concrete mix was placed into the beam using a special mould, the propagation of the diagonal cracking seems to be unaffected by the line of discontinuity that separated the normal concrete and the honeycombed zone. In specimen 2A-1.2(a), it was clear that the path of the diagonal cracking seemed to 'avoid' the precast honeycombed zone. In specimen 2A-1.2(b), the diagonal crack propagation was found diverted from the honeycombed zone, although later it failed with the honeycombed zone crushed. This could be due to the existence of a discontinuous layer between the normal concrete and the honeycombed zone, across which force could not be transferred effectively.

This suggests that the precast inclusion method is not an appropriate method to simulate a honeycombed zone. It creates a zone of discontinuity between the normal concrete and the honeycombed zone, which does not accurately simulate the honeycombed problem in concrete beams.

(e) Beams With a Void Zone

The test was carried out on beam 2A-1.3. The void was of the same size as the honeycombed zone with 60 x 60 mm dimension. Referring to the crack map of specimens 2A-1.3 in **Figure 4.3(b)**, it was clear that the idea of treating a honeycombed beam similar to a beam with an opening was inaccurate. A honeycombed zone provides a medium for the transfer of shear forces, and thus contributes to the shear capacity of the beam. However this did not happen in the beam with a void.

(f) Beams With a Construction Joint

This is another case of study included in the current research. The observations showed that the diagonal cracking in all specimens tested formed along the line of the honeycombed zone that simulated the joint. This was different from the ordinary honeycombed beam and shall be treated differently.

4.6.1.2 Beams With A Shear Span Ratio of 3.5

For beams with a shear span ratio of 3.5 the results obtained agreed with the tests of other researchers. The modes of diagonal cracking formation were more influenced by the flexural crack rather than the compressive strut. As a result of that, the influence of the honeycombed zone on beams is less significant. Note that only one location of honeycombed zone, i.e. at the centre of shear zone was investigated. A honeycombed zone at the soffit may have a more significant effect on the shear of the beam.

From the tests in series 2A and 2B, all beams except for beam 2B-5, a control beam, the diagonal crack was formed through the extension of a flexural crack. Again the different behaviour exhibited by beam 2B-5 could be explained as in **Section 4.6.1** and by examining the profile of the crack, it was clear that it was more influenced by the flexural effect.

4.6.2 Diagonal Cracking Load

The analysis of the diagonal cracking load is carried out in two stages. The first stage is a general analysis in which a plot of diagonal cracking load is presented and comparisons made between honeycombed and control beams as well as between honeycombed beams with different locations of a honeycombed zone. The second stage is to analyse the effects of the honeycombed zone on the magnitude of the diagonal cracking load in comparison to the control beams.

4.6.2.1 General Analysis

In order to examine the overall results of diagonal cracking loads, a plot of diagonal cracking shear force for each specimen, V_c, versus the strength of normal concrete, f_{cu}, was produced. The plots for series 1A and 1B tests are shown in **Figure 4.6(a)** and **(b)** respectively. From **Figure 4.6(a)** the overall effects of the honeycombed zone on the diagonal cracking shear in series 1A and 1B beams can be observed. A detail comparison of results from each specimen can be made by referring to **Figure 4.6(b)**. Comparisons between series 1A and 2A tests and between series 1B and 2B can be examined in the plot in **Figure 4.7(a)** and **(b)** respectively.

The plots in **Figure 4.6(a)** demonstrate that for the control beams in series 1A and 1B, the diagonal cracking shear forces were quite significantly increased with the increase in the concrete strength. From **Figure 4.6(a)** and **(b)**, generally it can be seen that the effect of the honeycombed zone on diagonal cracking shear force was more significant in series 1A-beams. For series 1A beams, except for specimen 1A-5(a)[TM], all the other honeycombed beams had a lower diagonal cracking shear force than the control beam. For series 1B, both specimens of beam 1B-5[TL] and two out of three specimens of beam 1B-3[MS], had a higher diagonal cracking shear force than the control specimen.

It is interesting to note from the plots that the magnitude of reduction in the diagonal cracking shear force were more significant in series 1A beams than in series 1B for the same locations of a honeycombed zone. For example, note that the average diagonal cracking shear in beam 1A-2[MM] is lower than in beam 1B-2[MM]. This was consistent with the previous discussion, in which, the honeycombed beams in series 1A were more affected in their mode of diagonal cracking formation compared to series 1B beams.

Comparisons between the results of series 1A and series 2A tests can be seen in **Figure 4.7(a)**. The two lowest points in **Figure 4.7(a)** represent data from specimens 2A-1.3, beam with a void. The point at the top in the plot represents the diagonal cracking load of specimen 2A-1.2(b), the beam with a precast honeycombed zone. Another specimen of the beam, 2A-1.2(a), had its diagonal cracking shear force lower than in specimen 2A-1.2(b), but its data point is still in the region equivalent to the control beams. The other three points in the top region belong to beams with shear reinforcement, beams 2A-8 and 2A-9. The other point was the data from specimen 2A-1.1(a)[MM], which demonstrates the typical variation that is always found in shear test data. Data from 2A-1.1(b)[MM] specimen was well located together in the region of beam 1A-2. The diagonal cracking shears of other honeycombed beams with a shear span ratio of 2.0 in series 2A were below the control beam. The diagonal cracking shear of the control beam with a shear span ratio of 3.5, beam 2A-4, was significantly less than the control beam with a shear span ratio of 2.0. With a honeycombed zone introduced, the diagonal cracking load for beams with a shear span ratio of 3.5, beam 2A-5[MM] was further reduced, although not very substantially.

From **Figure 4.7(b)**, the lowest point in the plot was from beam 2B-1, a beam with a construction joint. Note that in series 2A, the diagonal cracking shear force of the same beam with a construction joint was not the lowest of series 1A and 2A. This will be explained later in **Section 4.6.2.2**. The top point in the plot was from specimen 2B-3(a), the control specimen of a beam with shear reinforcement. The results obtained from other beams with shear reinforcement showed a great variation. One of them, specimen 2B-

4(b), with a honeycombed zone had a low diagonal cracking shear force. The data points representing the diagonal cracking shear force of the control and the honeycombed beams with a shear span ratio of 3.5, seems to be located at the appropriate spot, and no great variation was found between the control and the honeycombed beams.

4.6.2.2 Detail Analysis of Diagonal Cracking Load

The diagonal cracking loads in **Table 4.5(a)** to **(d)** were averaged and normalised against the average compressive strength of the normal concrete of each series considered. These were done according to the relationship given in BS 8110, in which the diagonal cracking shear stress of a concrete beam is related to the cube root of the concrete compressive strength. The values for beams in series 1A and 1B are tabulated in **Table 4.6(a)** and **(b)** respectively. Note that for series 1A, the average compressive strength of the normal concrete is 50.5 N/mm^2. For series 1B, the average is 33.5 N/mm^2. Also shown in the table is the ratio of the average diagonal cracking load in the honeycombed specimens to the control specimens. **Figures 4.8(a)** and **(b)** illustrate the ratio more clearly according to the location of the honeycombed zone.

Honeycombed beams in series 2A and series 2B tests were compared to their respective control beams in series 1A and 1B. Comparisons were also made with the relevant honeycombed beams in series 1A and 1B respectively. In order to analyse the diagonal cracking loads of beams listed in **Table 4.7(a)** and **(b)**, the average strength, on which the normalisation was based, was obtained using the strength of each beam considered in the group. In **Table 4.7(a)** the average strength used for the normalisation of the diagonal cracking shear of those beams listed was 48.4 N/mm^2. In **Table 4.7(b)** the average strength was 34.6 N/mm^2. The average normalised diagonal cracking loads for beams with a shear span ratio of 3.5 and beams with shear reinforcement for series 2A and 2B are given in **Table 4.8**.

The ratios of the diagonal cracking load of honeycombed to the control for beams with a shear span ratio of 2.0 and without shear reinforcement in series 2A and 2B are presented in **Figure 4.9(a)** and **(b)** respectively. The ratio for the beam with a void is also included in **Figure 4.9(a)** for comparison.

(a) Beams With a Shear Span Ratio of 2.0

From series 1A tests, it was found that a honeycombed zone located along the potential path of the compressive strut not only modified the mode of the diagonal cracking formation, but also reduced the diagonal cracking load compared to the control specimens. The most substantial reduction occurred when a honeycombed zone was located in the middle of the shear zone as shown by beam 1A-2[MM], with the ratio of 0.64. A less substantial reduction occurred when a honeycombed zone was located at the upper portion of the potential path of the strut as in beam 1A-6[TL], with the ratio of 0.82. The least reduction occurred when a honeycombed zone was located at the lower section as in beam 1A-7[BS], with the ratio of 0.9.

In beams 1A-3[MS] and 1A-4[ML], the diagonal cracking load was quite substantially reduced, with the ratios being 0.79 and 0.71 respectively. Note that both beams had an independently formed diagonal crack. In beam 1A-5[TM], the average reduction was not very substantial. However in specimen 1A-5(b), the reduction was more substantial. Of all the locations of the honeycombed zone investigated, its presence at the bottom middle section of the shear zone as in beam 1A-8[BM] caused the most substantial reduction of the diagonal cracking load with the ratio of 0.56. This occurred because the weak honeycombed concrete in this region resulted in premature flexural cracking from which the diagonal crack developed.

For series 1B, although the honeycombed zone located along the potential path of concrete compressive strut did not change the mode of diagonal cracking formation, it was however found that the crack was formed at a lower load at two locations compared to the control. The lowest ratio of 0.79 occurred when a honeycombed zone was located at the middle section of the path of the potential strut, as in beam 1B-2[MM]. The reduction was not very significant with a ratio of only 0.91, when a honeycombed zone was at the lower section of the potential strut path, beam 1B-6[BS]. When a honeycombed zone was located at the upper section of the potential strut path, beam 1B-5[TL], the beam developed its diagonal crack at a higher load than the control beams with the ratio of 1.11. The behaviour shown by beam 1B-5 indicated that the shear force only significantly acts to initiate the diagonal cracking when it interacted with the tensile stress which exists at the lower section of the beam. In this situation the honeycombed zone at the upper section did not contribute to the diagonal crack formation. This could also be the reason for beam 1A-6[TL] of series 1A having a higher ratio of diagonal cracking load to the control beam, compared to beam 1A-2[MM].

The insignificant effect of a honeycombed zone located as in specimen 1B-3[MS] on the diagonal cracking formation was also shown by its effect on the load. The ratio was 1.01, which was almost similar to the control specimens. In beam 1B-4[ML], a honeycombed zone caused the crack to be formed earlier than the control specimen with the ratio of 0.82.

The ratios for beams 2A-1.1[MM], 2A-2[MS] and 2A-3[ML] in **Figure 4.9(a)** show the same effect of the honeycombed zone on the diagonal cracking as already found for beams in series 1A. The results indicated the consistencies of the experimental data. Although the magnitude of reduction of the diagonal cracking load varied, the pattern of reduction was consistent.

The effects of various parameters on the diagonal cracking load of the honeycombed beams are now considered.

(b) The Effect of the Strength of Honeycombed Zone

From **Figure 4.9(a)**, it can be seen that the strength of honeycombed concrete had no effect on the diagonal cracking load. Comparing the ratios between beams 1A-2[MM] and 2A-1.1[MM], and between beams 1A-3[MS] and 2A-2[MS], and between 1A-4[ML] and 2A-3[ML] no relationship between the honeycombed strength and the diagonal cracking load can be derived. It was in fact quite strange that the ratio for those beams of series 2A are higher than for those in series 1A. This suggests that the diagonal cracking load was governed by the more brittle nature of the normal concrete rather than by the strength of the honeycombed concrete.

(c) The Effect of the Size of Honeycombed Zone

Comparisons between the ratios in beam 2A-7[MM] and beam 1A-2[MM] in **Figure 4.9(a)** show that the size of the honeycombed zone had no effect on the magnitude of the diagonal cracking load. Note that the size of the honeycombed zone for beam 2A-7 was 90 x 90 mm compared to 60 x 60 mm in beam 1A-2. In fact the ratio for beam 2A-7, with a bigger zone of honeycombed and a lower strength, with the value of 0.71 is higher than the ratio found in beam 1A-2.

From **Figure 4.9(b)** the ratio of the diagonal cracking load of beam 2B-2[MM], with a honeycombed zone of 90 x 90 mm, is 0.98. The ratio is close to unity. This indicated the consistent trend that was found in series 1B tests that the effect of the honeycombed zone on the diagonal cracking load was not very significant in comparison to series 1A and 2A beams. Comparing the ratio for beams 1B-2 and 2B-2, as found in series 1A and 2A, the size of the honeycombed zone had no influence on the diagonal cracking load.

(d) Beams With A Shear Span Ratio of 3.5

The ratio of the diagonal cracking load for beams with a shear span ratio of 3.5 for series 2A and 2B are presented in **Figure 4.10(a)**. The ratios in series 2A and 2B, given by beams 2A-5 and 2B-6 were 0.88 and 1.07 respectively. The results show a consistent trend: a significant effect caused by a honeycombed zone in beams with a higher strength of normal concrete as occurred in series 1A and 2A, and less significant effect in beams with a lower strength of normal concrete, as found in series 1B and 2B. If the ratio of beam 2A-5 is compared with the ratio of beam 1A-2 and 2A-1.1 (refer **to Figure 4.9(a)**) they show that for beam with a long shear span, the effect of a honeycombed zone was less significant. This can be attributed to the fact that for a long shear span beam, the shear is more influenced by the flexure, rather than by the compressive strut in the shear zone.

(e) Beams With Shear Reinforcement

The ratio of the diagonal cracking load for beams with shear reinforcement for series 2A and 2B are presented in **Figure 4.10(b)**. The data from the experiment did not give conclusive results. For beam 2A-9, with the ratio of 1.11, it indicates that the presence of the honeycombed zone had no effect on the diagonal cracking load. However for beam 2B-4, the effect of a honeycombed zone was very significant with a ratio of 0.64. Examining **Figure 4.3(d),** the crack mapping of beam 2B-4, this phenomenon can probably be explained. In specimen 2B-4(b), an independent diagonal crack formed at a load of 24 kN. However in specimen 2B-4(a), the diagonal cracking load recorded was based on the flexurally formed diagonal crack which it was later found was not the crack that led to the ultimate failure. The crack that led to the failure was formed independently at a very high load. If that particular load was taken, the average diagonal cracking load for beam 2B-4 would be higher.

However from the results it was clear that the honeycombed zone could influence the mode of diagonal cracking formation, but shear reinforcement as anticipated, played a very substantial role in confining the effect.

(f) Beams With a Precast Honeycombed Zone

The ratio for beam 2A-1.2 of 1.07 as shown in **Figure 4.9(a)** was higher than in the control, providing further evidence that the precast honeycombed zone could not be used to simulate honeycombed problem in the concrete beam. As mentioned earlier the existence of a discontinuity zone separating the zone of precast honeycombed and normal concrete caused the critical diagonal crack formation to be delayed.

(g) Beams With a Void

Examining the mode of diagonal cracking in beam 2A-1.3, the ratio obtained of 0.44 is to be expected. The value was very low and not compatible with any honeycombed beams in the current study. This provided evidence that the mode of shear behaviour of the beam with a void was incompatible with the honeycombed beams.

(h) Beams With a Construction Joint

For beams with a joint, the ratio for beams 2A-6 and 2B-1 are 0.66 and 0.33 respectively. The inconsistency in the ratio could be attributed to the difference in the degree of compaction in both beams. It happened during the pouring of beam 2B-1 that the workability of the normal concrete mix was poor. This caused difficulties in carrying out the compaction. As a result of that the normal concrete did not flow into the narrow space between the honeycombed mix and the wall of the mould as happened in beam 2A-6. After demoulding, the zone of honeycombed in beam 2B-1 could be seen, in contrast to beam 2A-6 in which the surface of the beam was smooth. This might be the reason for the low ratio in beam 2B-1. The poor workability might also have caused the honeycombed mix in beam 2B-1 to receive a lesser amount of indirect impact compared to beam 2A-6.

4.7 ULTIMATE FAILURES

In the following section, the mode of the ultimate failures observed in the current tests are examined and the effects of the presence of a honeycombed zone on the mode of failure are discussed. Also discussed are the failure loads and an analysis is carried out with regard to the ratio of the ultimate load of honeycombed beams to the control. Also included is the analysis of the reserve of strength of honeycombed beams, which is defined as the percentage of load that the beam can take after the formation of the diagonal cracking, to reach the ultimate failure.

4.7.1 Behaviour Prior to and At Failure

Tables 4.5(a) to **(d)** show the summary of the mode of ultimate failure for each specimen. At the ultimate stage, most of the specimens in series 1A, regardless of the mode of diagonal cracking formation, failed in shear-compression. For beam 1A-2[MM], one of its ends failed ultimately in shear-tension and crushing of concrete occurred along the crack mainly at the honeycombed zone. This could be attributed to the flat angle of the diagonal crack. The second specimen failed through the buckling of the top concrete 'arch'. Beam 1A-7[BS] failed in shear-tension, thus indicating that the presence of a honeycombed zone in the tension zone near to the support had weakened the anchorage resistance of the reinforcement.

In series 2A beams, most honeycombed beams which failed in the short span, failed in shear-compression. Specimen 2A-1.1(a)[MM] failed in shear-compression but was accompanied by the buckling of the top compressive 'arch'. In specimen 2A-1.1(b)[MM], the honeycombed region crushed accompanying the shear-compression failure. Beams with a void failed due to the failure of the top chord and in the anchorage. Beam with a honeycombed zone simulating a joint, beam 2A-6, failed by the buckling of the top arch. All the control and the honeycombed beams with a shear span ratio of 3.5 failed by the buckling of the top 'arch' and an anchorage failure.

For series 1B, the shallow angle of diagonal cracking caused specimen 1B-1(a) to fail in shear tension. Beam 1B-6[BS] failed ultimately in shear-tension with the same reason as described above for beam 1A-7[BS]. Other specimens either failed in shear-compression or failed on the long shear span.

In series 2B, beam 2B-2[MM] failed by the buckling of the top compressive 'arch' but the failure in specimen 2B-2(b) was also accompanied by crushing along the crack. For beam with shear reinforcement without a honeycombed zone, specimen 2B-3(a) failed in shear compression and specimen 2B-3(b) failed in the long shear span. For the honeycombed beam with shear reinforcement, one specimen failed in shear compression, and another specimen failed by the buckling of the top 'arch' and the crushing of concrete at the region of honeycombed zone. As in series 2A, beams with a shear span ratio of 3.5 failed by the buckling of the top 'arch' and accompanied by the anchorage failure. Beams with a joint failed by the buckling of top 'arch', similar to their identical beams in series 2A.

In all series, every specimen without shear reinforcement that failed in the long shear span had a similar profile of diagonal cracks in their short span. Each of them had a steep, flexurally developed diagonal crack which initiated at the middle of the shear span. From the failure mode shown by all the beams that failed in the long shear span, the causes of such behaviour can be explained as follows. The formation of a steep flexurally developed diagonal cracking turned the beam into a strong tie-arch structure. The structural resistance was provided by the large area of concrete in the top compressive 'arch' and the long reinforcement anchorage. As the load increased, the shear force on the long shear span increased, until it reached the limit of the tensile strength of concrete. At the same time, the resistance provided by the short shear span however exceeded the shear force in the short span associated with the limit of tensile strength of concrete in the long shear span. This led to the failure in the long shear span, as the formation of the diagonal cracking was immediately followed by the ultimate failure.

4.7.2 Ultimate Loads

Tables 4.5(a) to **(d)** show the failure load for each specimen of all series of tests. The plots of the ultimate shear load versus the normal concrete compressive strength of each specimen of series 1A and 1B are shown in **Figures 4.11(a)** and **(b)** respectively. The overall effects of the honeycombed zone on the ultimate capacity of beams are shown in **Figure 4.11(a)**. A more detailed study of the effects on each honeycombed beam can be found from **Figure 4.11(b)**.

The plots of the ultimate shear versus the compressive strength of beams in series 2A are presented together with results from series 1A in **Figure 4.12(a)**. Results from series 2B tests are plotted together with results from series 1B and are shown in **Figure 4.12(b)**.

The plots in **Figure 4.11(a)** demonstrate that for beams without a honeycombed zone the ultimate shear capacity increases with the increase in concrete strength. From **Figure 4.11(b)** it can be seen that, ultimately, a few identical specimens in both series produced scattered results. This occurred because those identical specimens had different modes of ultimate failure or it was due to the different degree of arching effect. For example, the high capacity in specimen 1A-8(b)[BM] was due to the failure which occurred on the long shear span. Specimen 1A-5(b)[TM] has a steep diagonal crack which leads to a higher degree of arching effect as compared to specimen 1A-5(a)[TM].

In series 1B, the results from specimen 1B-1(b) were quite scattered from the other 2 control specimens because it failed on the long shear span. Specimens 1B-2(a)[MM] and 1B-2R[MM] also showed scattered results, and this occurred because their modes of failure were different, a shear compression mode in the former and, in the latter, it was due to the buckling of the top 'arch'. The results of the other specimens were not that scattered, although some identical specimens developed a different mode of diagonal cracking.

From the plots it can be seen that the magnitudes of the reduction of the ultimate capacity due to a honeycombed zone were generally more significant in series 1A beams. It is interesting to observe that, at certain locations investigated, with the same position of a honeycombed zone, beams in series 1A possessed a lower ultimate capacity than beams in series 1B. This phenomenon occurred in beams 1A-2[MM] and 1B-2[MM], beams 1A-3[MS] and 1B-3[MS] and in beams 1A-4[ML] and 1B-4[ML].

It was also observed that for beams 1A-6[TL] and 1B-5[TL], and also beams 1A-7[BS] and 1B-6[BS], both pairs had a honeycombed zone at the same location respectively, their ultimate capacity appeared quite close to each other regardless of the difference in their normal concrete strength. Note that both beams 1A-6[TL] and 1B-5[TL] failed due to the crushing of concrete in the honeycombed zone, and both beams 1A-7[BS] and 1B-6[BS] failed due to the failure of reinforcement anchorage.

With regard to the results of series 2A tests, from the plots in **Figure 4.12(a)** it can be seen that except for beams with shear reinforcement and beams 2A-1.2[MM], with a precast honeycombed zone, 2A-2(a)[MS], and 2A-3[ML], all the other honeycombed beams failed at a lower load than the control beam of series 1A. Some identical specimens failed at different magnitudes of ultimate loads. Except for beams 2A-2[MS] and 2A-7[MM], the magnitude of the differences in the other beams were not very substantial. As for beams in series 1A and 1B, these could be attributed to the difference in the pattern of the formation of diagonal cracking. By examining closely the crack pattern of specimens 2A-2(a) and 2A-2(b) as shown in **Figure 4.3(b)**, it is found that the crack profile in the former is steeper and curved, while in the latter the crack path is slightly shallower. The same pattern of diagonal cracks can be observed on specimens 2A-7(a)[MM] and 2A-7(b)[MM]. The more steeper and curved crack path occurred in specimen 2A-7(b) which led to a higher ultimate load.

For beams with shear reinforcement, both the control and the honeycombed beams possessed very high ultimate capacities. Results from each specimen were very close to each other, although one specimen of each beam failed in the long shear span. This type of behaviour is to be expected since the presence of shear reinforcement reduces the influence of concrete and hence of a honeycombed zone.

For beams with a shear span ratio of 3.5, their ultimate capacities were very close, with no significant difference between the control and the honeycombed beams. The lowest ultimate loads of all beams in series 1A and 2A occurred in beams with a void, beam 2A-1.3, and also in beams with a joint, beam 2A-6.

For the beam with a precast honeycombed zone, its ultimate capacity is very close to the control. It is now clear that the precast honeycombed zone cannot be used to simulate a honeycombed zone. The results obtained from the formation of the diagonal cracking, its diagonal cracking load and up to the ultimate failure, were very close to the beam without a honeycombed zone. These were in contrast to the behaviour shown by beams with a cast in-situ honeycombed zone.

The overall picture of the ultimate loads of beams in series 2B can be seen in **Figure 4.12(b)**. The highest ultimate shear loads occurred in beams with shear reinforcement. Except for beam 2B-4(b), the three other specimens had very close values to each other. The lowest ultimate loads occurred in beam 2B-1, the beam with a joint. The ultimate loads in beams with a shear span ratio of 3.5 were very close to each other.

For a more detailed analysis, the average ultimate loads for each beam in series 1A and 1B were normalised against the average normal concrete strength of each series and presented in **Tables 4.9(a)** and **(b)**. The calculation of the average values only considered the specimens which failed in the short shear span. Each test of the specimens of beam 1B-4, however, failed in the long shear span.

In order to investigate the effect of the honeycombed zone, two parameters can be evaluated: the ratio of ultimate loads in honeycombed specimens to the control specimens; and the percentage of reserve of strength a specimen possesses after the diagonal crack formed. These values for series 1A and 1B tests are presented in **Figures 4.13(a)** and **(b)** respectively.

For series 2A and 2B, the normalisation of the ultimate loads was carried out based on the average strength obtained from all specimens in the group considered. In **Table 4.10(a)**, the average strength used for the normalisation was 48.4 N/mm^2. In **Table 4.10(b)**, the normalisation was based on the average strength of 34.6 N/mm^2. Note that the difference in the average strength is the reason for the difference between the percentage of reserve of strength quoted for beams 1A-1, 1A-2, 1A-3 and 1A-4 in **Table 4.9(a)** and in **Table 4.10(a)**, and for beams 1B-1 and 1B-2 in **Table 4.9(a)** and in **Table 4.10(b)**. In **Tables 4.9(a)** and **(b)**, the normalisation was based on the average strength of 50.5 N/mm^2 and 33.5 N/mm^2 respectively. The results for beams with a shear span ratio of 3.5 and beams with shear reinforcement are tabulated in **Table 4.11**. In their case no normalisation was required as their respective normal concrete strength was the same. For example, beams 2A-4, the control, and 2A-5, the honeycombed beam, were from the same mix. The same applies for other pairs of beams in **Table 4.11**.

4.7.2.1 Beams With a Shear Span Ratio of 2.0

As shown in **Table 4.9(a),** for series 1A, the ultimate load of the control beam was the highest in the series. A honeycombed zone located at all locations investigated caused a reduction in the ultimate shear capacity of the beam. As shown in the table and also from

Figure 4.13(a), the highest ratio of ultimate load of a honeycombed beam to the control beam occurred in beam 1A-8[BM] with the ratio of 0.84. The lowest ultimate capacity occurred when the honeycombed zone was located at the central area of the shear span, beam 1A-2[MM], with the ratio of only 0.47. The honeycombed zone also significantly reduced the ultimate capacity of beams at two other locations. This occurred in beams 1A-3[MS] and 1A-7[BS] which both have the ratio of 0.49. At other locations the ratios are between 0.64 and 0.78.

In terms of reserve of strength, for series 1A, the control specimen possessed the value of 92.6%. For specimens with a honeycombed zone, the reserves of strength were in the range of 4.6% to 188.5%. The extraordinarily high value in beam 1A-8 occurred because the presence of honeycombed concrete accelerated the formation of the diagonal crack. However with a steep diagonal crack the beam turned into a strong tie-arch structure and could resist further substantial shear force before failure. The low value of only 4.6% which occurred in beam 1A-7[BS], was due to the weak bonding and tensile properties of the honeycombed zone which resulted in the significant reduction of the anchorage resistance.

Observations of series 1A showed that the mode of the formation and the profile of the diagonal crack could to some extent determine the behaviour prior to failure, the mode of failure and the ultimate shear capacity of the beams. It was observed that, generally, a steep diagonal crack leads to a higher ultimate load compared to a shallow diagonal crack. A steep slope was observed in beams 1A-1, a control, 1A-4[ML], 1A-5[TM], 1A-6[TL], and 1A-8[BM]. Other beams had a shallow diagonal crack. Beams with a steep slope also possessed a higher reserve of strength. Beam 1A-6[TL] is exceptional, because the presence of a honeycombed zone at the top compression zone caused early crushing of the concrete.

From the values in **Table 4.9(b)**, and also shown in **Figure 4.13(b)**, in series 1B tests, the ultimate loads of honeycombed beams were not much different from the control beam. The lowest ratio occurred in beam 1B-6[BS], with the value of 0.80. This again demonstrates the significant effect of the honeycombed zone in weakening the anchorage resistance of the reinforcement. When the honeycombed zone was located at the central area of the shear span, beam 1B-2[MM], the ratio was 0.89, which shows that a honeycombed zone located at the central shear span can be critical to the shear capacity of a reinforced concrete beam. At other locations the values seem to suggest that honeycombed concrete did not affect the ultimate shear capacity of the beam in comparison to the control.

Taking the average values, the reserve of strength in the control beam of series 1B was only 43.9%. This was low because two of the control specimens formed their diagonal crack independently with a shallow angle. The other control specimen with a steep flexurally formed diagonal crack had a reserve strength of 93.8%. For the honeycombed beams, the low reserve of strength only occurred in beams 1B-5[TL] and 1B-6[BS] with the values of 38.7% and 26.7% respectively. The reasons for these are the same as given for beams 1A-6[TL] and 1A-7[BS] respectively. For beams 1B-2[MM] and 1B-3[MS], the reserves of strength are 62.2% and 62.9% respectively. The highest reserve of strength occurred in beam 1B-4[ML] with the value of 105.0%.

It should be remembered that, in the above analysis, the ultimate failure loads were normalised against the normal concrete compressive strength. This normalisation appears to be acceptable for cases where specimens failed ultimately by crushing of the concrete, which occurred in most of the specimens in this investigation. As already mentioned earlier, there were specimens which ultimately failed due to splitting of concrete along the reinforcement anchorage. The anchorage failure is related to the tensile strength of concrete. In order to appropriately compare the results of those beams, they should be normalised against the tensile strength of concrete, which is normally related to the square root of compressive strength. This was done for specimens 1A-2(a), 1A-7, 1B-1(a) and 1B-6 which failed in the reinforcement anchorage. The modifications to the figures in **Table 4.9(a)** and **(b)** were however very insignificant.

(a) The Effect of the Strength of Honeycombed Zone

This effect can be examined by comparing the values for beams 1A-2[MM] and 2A-1.1[MM], between beams 1A-3[MS] and 2A-2[MS] and between beams 1A-4[ML] and 2A-3[ML]. Their values are presented in **Table 4.10(a)** and **Figure 4.14(a)**. From the values of the ratio of the ultimate load of honeycombed beams to the control, there was no indication that the difference in the strength of the honeycombed zone had influenced the ultimate strength of those beams. In fact the results show a contrast. All beams in series 2A have a higher ratio than their respective beams in series 1A, while in fact the strength of the honeycombed concrete in beams in series 2A was lower.

In terms of the reserve of strength, the difference in the values between respective beams in series 1A and 2A, were due to the difference in the mode of the diagonal cracking path. The substantial difference between beams 1A-3[MS] and 2A-2[MS] was caused by the fact that a more shallow diagonal crack occurred in beam 1A-3. The same phenomenon could be observed in beams 1A-2[MM] and 2A-1.1[MM]. Comparing the cracking path of beams 1A-4[ML] and 2A-3[ML], they were similar and led to the same value of reserve of strength.

The results indicate that the variation that occurred in the shear behaviour and strength were greater than the potential effect caused by the difference in the strength of the honeycombed concretes studied.

(b) The Effect of the Size of Honeycombed Zone

This effect can be examined by comparing the values in beams 1A-2 and 2A-1.1 with values in beam 2A-7, and between beam 1B-2 and beam 2B-2, which are presented in **Tables 4.10(a)** and **(b)** and in **Figures 4.14(a)** and **(b)**. In series 1A and 2A beams, the ratio for the beam with a bigger honeycombed zone, 2A-7, is bigger than the other two beams with a smaller honeycombed zone, beams 1A-2 and 2A-1.1. In series 1B and 2B, the ratio of beams 1B-2 and 2B-2 are about the same value.

In terms of the reserve of strength, again the value in beam 2A-7 of 92.6% is substantially bigger than in beams 1A-2 and 2A-1.1. The percentage is even greater than in the control, with the value of 87.7%. This occurred because one specimen of beam 2A-7, specimen 2A-7(b), developed quite a steep and curved diagonal crack and resulted in a very high ultimate load. If only specimen 2A-7(a) is considered, the reserve of strength of beam 2A-7 would be about 65%. With regard to the greater percentage of beam 2A-7 compared to the control, it was due to the fact that the percentage obtained was based on the diagonal cracking load. As shown in **Table 4.10**, the ultimate load of beam 2A-7 was 58.5 kN compared to 80.4 kN in the control. However the diagonal cracking load in beam 2A-7 was 30.4 kN compared to 42.8 kN in the control. Thus, it should be clear that the percentage of reserve of strength cannot be used to compare the shear capacity between beams. It only measures the ability of the beam to sustain load after the formation of diagonal cracking.

The results from beams 1B-2, with the value of 66.4%, and 2B-2, with the value of 39.2%, may indicate the effect of the bigger honeycombed zone in beam 2B-2. However as mentioned above their strength ratios are about the same.

The results again indicate that the magnitude of the difference in the size of the honeycombed zone in the current study might not be that significant in comparison to the variation that is always found to occur in shear tests.

(c) Beams With Shear Reinforcement

The results for beams with shear reinforcement are given in **Table 4.11** and shown, according to the location of the honeycombed zone, in **Figure 4.15(b)**. The results give clear evidence that, although the honeycombed zone could modify the formation of the diagonal cracking as discussed in **Section 4.6.1**, the presence of shear reinforcement could prevent the subsequent adverse effect caused by the honeycombed zone. In terms of the ratio, in beam 2A-9, the value is 1.06, indicating that there was no reduction of the ultimate capacity of the honeycombed beam compared to the control. In beam 2B-4, the ratio is slightly lower with a value of 0.84. It was found that in beam 2B-4, specimen 2B-4(b) failed at quite a low ultimate load. This occurred because from the beginning the diagonal cracking was formed independently.

Examining the values of reserve of strength, the significant effect of shear reinforcement in confining the effect caused by the honeycombed zone can be seen in beam 2B-4. The average percentage of reserve of strength in beam 2B-4 was 152.9%. This shows that the honeycombed beam with shear reinforcement could take a substantial amount of load although the diagonal crack formed early as a result of the presence of a honeycombed zone. The percentage of strength in beams 2A-8, 2A-9 and 2B-3, with the values of 84.5%, 84.2% and 91.6% respectively, indicate that, ultimately, a honeycombed zone had no influence on the ultimate shear capacity of beams.

(d) Beams With a Precast Honeycombed Zone

Comparison between the ratios of ultimate load of beams 1A-2 and 2A-1.1 with beam 2A-1.2 as shown in **Figure 4.14(a)** clearly show that there was a significant difference in the ultimate shear capacity of beams with a precast honeycombed zone and beams with a cast in-situ honeycombed zone. The ratios in beams 1A-2 and 2A-1.1 are 0.47 and 0.65 respectively compared to 0.98 in beam 2A-1.2. In terms of reserve of strength, the value in beam 2A-1.2 is 70.9% compared to only 37.6% and 56.1% in beams 1A-2 and 2A-1.1 respectively.

The results obviously indicate that the mode of shear behaviour for the beams with a precast honeycombed zone were stronger than beams with a cast in-situ honeycombed zone. It should be emphasised that the precast honeycombed zone and the normal concrete were effectively bonded and acted compositely to resist the shear force. The shear behaviour in both specimens show the evidence very clearly. The nature of the rough surfaces of precast honeycombed block helped to improve the bonding. However, as explained earlier the transfer of forces seems to be diverted. It was observed that this happened due to a discontinuity layer which existed between the precast honeycombed and the normal concrete.

It was found from this comparison that the cast in-situ technique was a more appropriate technique to simulate a honeycombed zone. No discontinuity zone formed which can separate the two concretes. The forces were transferred without any sign of discontinuity occurring. After all, in the actual situation, the honeycombed zone would normally be formed within the normal concrete.

(e) Beams With a Void

From **Figure 4.14(a)**, the values for beam 2A-1.3, beam with a void, are shown. The ratio of the ultimate load in beam 2A-1.3 to the control is only 0.38. This indicate the low ultimate shear capacity of the voided beam compared to beams with a honeycombed zone. From the early stage up to ultimate it is clear that there was no compatibility between a beam with a void and a beam with a honeycombed zone. The relatively high value of reserve of strength of 62.2% does not reflect any form of compatibility with the honeycombed beams. It merely shows that the beam had quite a substantial reserve of strength. Note however that the beam formed its diagonal crack at a load of only 44% of the control beam, the second lowest of all tests in the current study. The lowest occurred in a beam with a joint.

(f) Beams With a Construction Joint

The values for beams 2A-6 and 2B-1, with a joint are shown in **Figures 4.14(a)** and **(b)**. For beams 2A-6 and 2B-1, their ratios of ultimate load to the respective control beams are 0.39 and 0.37 respectively. The values are very low, beyond the range of values found in the normal honeycombed beams. The beam therefore needs to be treated as a special case. The substantial difference in the values of the reserve of strength again was due to the difference in the degree of compaction as described earlier in **Section 4.6.2.2.**

4.7.2.2 Beams With a Shear Span Ratio of 3.5

The ratios of the ultimate load of the honeycombed beam to the control and the percentage of reserve of strength of beams with a shear span ratio of 3.5 are shown in **Figure 4.15(a)**. The ratios of 0.95 in beam 2A-5 and 0.92 in beam 2B-6 show that the honeycombed zone caused a slight reduction in the ultimate load of the beams. Variations in the percentage of reserve of strength between the control and honeycombed beams in series 2A and 2B make it difficult to say that the honeycombed beams were more brittle than the control. In series 2A, the control beam was more brittle, whereas in series 2B the honeycombed beam was more brittle.

4.8 CONCLUSIONS

From the current experimental work that has been carried out, the following general conclusions can be made:

1. It was observed that the flexural stiffnesses of the honeycombed beams were not affected by the presence of the honeycombed zone. This indicates that, when a honeycombed zone is present in the high shear zone, the reliance on the flexural

stiffness alone, as often adopted when load testing is used, as an aid in assessment work can lead to unsafe assessment results.

2. Despite all the variations found from the experimental work of the current study, the overall results provide a clear indication that a honeycombed zone present within the high shear region can affect the shear capacity of the beam. The degree of the effects varies and it seems that the major parameter that determine the magnitude of the effect is the location of the honeycombed zone within the high shear zone.

3. The effect of honeycombed concrete can take the form of accelerating the formation of diagonal shear cracking and modifying the mode of diagonal shear cracking formation. This consequently leads to the modification of the mode of ultimate failure and reduces the ultimate shear capacity of the beam. The details of this are presented in the conclusions in **Chapter 7**.

4. The effect of a honeycombed zone is more significant in beams with a shear span ratio of 2.0. For beams with a shear span of 3.5, the effect is small.

5. The presence of shear reinforcement in a concrete beam can be very effective in mitigating the adverse effect caused by the honeycombed zone.

6. The results show that the effect caused by the honeycombed zone is more critical in beams with a high strength of normal concrete. This is shown by the more significant effect observed in series 1A and 2A tests compared to series 1B and 2B tests.

7. The strength of the honeycombed concrete is not an important factor in determining the degree of its effect. From the results of beams in series 1A and 2A tests, it is clear that the reduction in the shear capacity of the beam is not proportional to the strength of the honeycombed concrete. This implies that the brittleness of the normal concrete is more significant in determining the mode of shear behaviour although a honeycombed zone can cause an adverse effect.

8. As far as the scope of the current experimental work is concerned, the size of the honeycombed zone was also insignificant. No indication was found which showed that a bigger honeycombed zone, the size of which was about half the effective depth of the beam can cause a more detrimental effect compared to the smaller honeycombed zone, with a size of about a third of the effective depth.

9. It is clear that the cast in-situ technique used throughout the tests is appropriate in simulating the honeycombed problem. No indication was found to suggest any existence of a zone of discontinuity between the zone of honeycombed concrete and the normal concrete. In contrast a pre-cast honeycombed zone was found to introduce a zone of discontinuity which can disturb the transfer of forces within the shear zone.

10. The tests show no evidence of any form of compatibility between a honeycombed beam and a beam with a void to simulate the honeycombed zone at any stage of their behaviour. Consequently the possibility of considering an analytical method for a beam with an opening to be applied to a honeycombed beam is unacceptable.

11. For beams with a joint, it was found that their behaviour was not similar to beams with a honeycombed zone. They need to be treated separately.

SERIES 1A	CUBE TESTS cube no.			Avg	Cube Strength	PRISMS prism no.		Avg	Mod. of Rupture	CYLINDER CRUSH cylinder no.			Avg	Cylinder Strength	CYLINDER SPLITTING		Avg	Splitting Strength
	1	2	3	(kN)	(N/mm^2)	1	2	(kN)	(N/mm^2)	1	2	3	(kN)	(N/mm^2)	1	2	(kN)	(N/mm^2)
POUR 1																		
normal	443	472	452	455.7	45.6	13.81	12.28	13.045	5.35	302	240	280	274.0	33.80				
honeycomb	41	31	40	37.3	3.7	7.01	4.98	6.00	2.46	160.3	90.3	-	125.3	15.46				
POUR 2																		
normal	554	525	535	538.0	53.8	14.77	11.57	13.17	5.40	300	300	-	300.0	37.00				
honeycomb	161	265	253	226.3	22.6	6.21	5.99	6.10	2.50	100.5	96.7	90.4	95.9	11.82				
load/min	75	75	75															
POUR 3																		
normal	471	476	502	483.0	48.3	11.08	10.81	10.945	4.49						141.3	168.6	155.0	3.82
honeycomb	199	244	203	215.4	21.5	7.11	6.25	6.68	2.74						43.9	55.8	49.9	1.54
load/min	150	75	100															
POUR 4																		
normal	536	548	543	542.3	54.2	14.64	12.73	13.685	5.61						149.8	127.9	138.9	3.43
honeycomb	300	265	220	261.5	26.2	6.27	8.19	7.23	2.96						68.7	56.4	62.6	1.93
load/min	75	150	150															

Notes:
1. All normal concrete cubes tested at 150 kN/min
2. Unless indicated otherwise the rate of loading of honeycombing cubes is at 150 kN/min
3. Honeycombing cubes in Pour 1 tested using MGA pad

Notes:
Dimensions of prism

Length (mm) 410
Height (mm) 100
Width (mm) 100

Notes:
Dimension of cylinder

Diameter (mm) 101.6
Length, normal (mm) 254.0
h/comb (mm) 203.2

Notes:
Dimension of cylinder

Diameter (mm) 101.6
Length, normal (mm) 254.0
h/comb (mm) 203.2

TABLE 4.1(a) Test results of control specimens (Series 1A)

SERIES 2A	CUBE TESTS cube no. 1	2	3	Avg (kN)	Cube Strength (N/mm^2)	PRISMS prism no. 1	2	Avg (kN)	Mod. of Rupture (N/mm^2)	CYLINDER CRUSH cylinder no. 1	2	Avg (kN)	Cylinder Strength (N/mm^2)	CYLINDER SPLITTING cylinder no. 1	2	Avg (kN)	Splitting Strength (N/mm^2)
POUR 1																	
normal	551	541	527	539.7	54.0	14.25	12.43	13.34	5.47	174.3	289.0	-	36.80	149.9*	113.4	-	3.65
honeycomb	149	141	135	141.3	14.1	6.39	6.13	6.26	2.57	61.5	63.9	62.7	7.98	71.5*	61.2*	66.4	1.64
mass (g)	1917.5	1879.5	2010.5	-	-												
POUR 2																	
normal	521	522	534	525.5	52.6	9.49	9.61	9.55	3.92	321.0*	337.0	-	41.25	138.8*	127.3*	133.1	3.28
honeycomb	103	121*	165	129.5	13.0	3.44	3.77	3.61	1.48	66.9	44.2	55.6	7.07	56.2	26.5	41.4	1.32
mass (g)	1909.5	1881.5	1989.0	-	-												
POUR 3																	
normal	394	383	442	406.3	40.6	8.60	7.74	8.17	3.35	149.9	232.0	-	29.54	118.9*	91.0*	105.0	2.59
honeycomb	-	-	111	111.0	11.1	4.85	3.79	4.32	1.77	70.0	76.0	73.0	9.29	46.1*	39.1	-	1.19
mass (g)	1712.5	1716.0	1732.0	-	-												
POUR 5																	
normal	420	420	430	423.2	42.3	11.39	10.81	11.10	4.55	293.0	278.0	285.5	36.35	128.5*	97.7	-	3.14
honeycomb(1)	100	97	117	104.4	10.4	5.95	3.46	4.71	1.93	78.2	58.3	68.3	8.69	64.6*	45.7*	55.2	1.36
honeycomb(2)	142	110	95	115.7	11.6												
POUR 8																	
normal	448	473	467	462.5	46.3									116.2	90.0	103.1	3.28
honeycomb	188	168	195	183.7	18.4												
mass (g)																	

__Notes:__
1. All cubes tested at 150 kN/min
*2. Cubes marked with * were tested using MGA pad*
3. Weight is for honeycombed cubes or cylinders
4. Honeycomb (2) in pour 5 refer to honeycomb prepared for precast inclusion of series 2A-1 beams

__Notes:__
Dimensions of prism

Length (mm)	410
Height (mm)	100
Width (mm)	100

__Notes:__
Dimension of cylinder
*Marked with **

Diameter (mm)	101.6
Length (mm)	254.0

Others

Diameter (mm)	100
Length (mm)	200

__Notes:__
Dimension of cylinder
*Marked with **

Diameter (mm)	101.6
Length, (mm)	254.0

Others

Diameter (mm)	100
Length (mm)	200

TABLE 4.1(b) Test results of control specimens (Series 2A)

SERIES 1B	CUBE TESTS cube no.			Avg	Cube Strength	PRISMS prism no.		Avg	Mod. of Rupture	CYLINDER SPLITTING cylinder no.		Avg	Splitting Strength
	1	2	3	(kN)	(N/mm^2)	1	2	(kN)	(N/mm^2)	1	2	(kN)	(N/mm^2)
POUR 1													
normal	378	353	363	364.5	36.4	10.83	10.71	10.77	4.42	141.9	134.3	138.1	3.41
honeycomb	90	83	102	91.8	9.2	4.36	4.36	4.36	1.79	33.3	32.7	33.0	0.81
POUR 2													
normal	394	442	381	405.9	40.6	9.06	9.91	9.49	3.89	143.3	136.4	139.9	3.45
honeycomb	138	135	135	135.7	13.6	4.49	4.21	4.35	1.78	68.7	52.3	60.5	1.49
POUR 3													
normal	308	280	307	298.3	29.8	9.45	8.75	9.10	3.73	113.1	83.8	98.5	2.43
honeycomb	91	-	-	91.0	9.1	3.85	4.2	4.03	1.65	43.8	34.8	39.3	0.97
POUR 4													
normal	354	284	363	333.9	33.4								
honeycomb	134	123	168	141.3	14.1								
POUR 5													
normal	277	275	272	274.7	27.5								
honeycomb	126	157	119	134.1	13.4								

Notes:
1. All cubes tested at 150 kN/min

Notes:
Dimensions of prism

Length (mm)	410
Height (mm)	100
Width (mm)	100

Notes:
Dimension of cylinder

Diameter (mm)	101.6
Length (mm)	254.0

- Pour 1 and 3, all honeycombed cylinders half-split

TABLE 4.1(c) Test results of control specimens (Series 1B)

SERIES 2B	CUBE TESTS cube no.			Avg	Cube Strength	PRISMS prism no.		Avg	Mod. of Rupture	CYLINDER CRUSH cylinder no.		Avg	Cylinder Strength	CYLINDER SPLITTING cylinder no.		Avg	Splitting Strength
	1	2	3	(kN)	(N/mm^2)	1	2	(kN)	(N/mm^2)	1	2	(kN)	(N/mm^2)	1	2	(kN)	(N/mm^2)
POUR 4																	
normal	338	322	-	329.8	33.0	13.82	12.06	12.94	5.31	163.6	221.0	192.3	24.48	142.6*	79.6	-	3.03
honeycomb	80	75	113	89.2	8.9	5.73	5.44	5.59	2.29	82.08	101.7	91.9	11.70	55.3*	63.8*	59.6	1.47
mass (g)	1765.5	1725.5	1819.0	-	-												
POUR 6																	
normal	414	376	400	396.7	39.7	14.35	10.04	12.20	5.00	270.0	263.0	266.5	33.93	94.6	105.1	99.9	3.18
honeycomb	123	172	150	148.3	14.8												
mass (g)																	
POUR 7																	
normal	341	341	339	340.0	34.0	14.83	14.27	14.55	5.97	217.0	182.2	199.6	25.41	90.0	93.5	91.8	2.92
honeycomb	-	159.7	148	153.9	15.4												
mass (g)																	

Notes:
1. All cubes tested at 150 kN/min
2. Weight is for honeycombed cubes or cylinders

Notes:
Dimensions of prism

Length (mm)	410
Height (mm)	100
Width (mm)	100

Notes:
Dimension of cylinder
*Marked with **

Diameter (mm)	101.6
Length (mm)	254.0

Others

Diameter (mm)	100
Length (mm)	200

Notes:
Dimension of cylinder
*Marked with **

Diameter (mm)	101.6
Length, (mm)	254.0

Others

Diameter (mm)	100
Length (mm)	200

TABLE 4.1(d) Test results of control specimens (Series 2B)

Remark		cube no 1	2	3	Average (kN)	cube strength (N/mm^2)
cubes tested using MGA pads, load applied at 150 kN/min		87.6	77.3	75.4	80.1	8.01
	mass (g)	1885	1923	1881		
cubes tested without MGA pads, at 150 kN/min		87.7	96.5	94.5	92.9	9.29
	mass (g)	1854	1876	1879		

TABLE 4.2 Results from a trial mix of honeycombed concrete, comparing the use of MGA pads

Test series	Average initial Young's Modulus of concrete (kN/mm^2)	
	Normal	Honeycombed
series 1A	28.0	16.0
series 2A	28.0	10.5
series 1B	22.0	10.5
series 2B	22.0	10.5

TABLE 4.3 Concrete Young's modulus

Beam	Positions (km/s)				
	Honeycomb spot	Top	Bottom	Right	Left
1A-2(a)	3.70	4.31	4.17	4.13	4.31
1A-2(b)	3.75	4.52	4.31	4.29	4.33
1A-3(a)	3.89	4.26	4.44	4.35	4.31
1A-3(b)	4.02	4.18	4.27	4.29	4.29
1A-4(a)	4.10	4.37	4.42	4.33	4.35
1A-4(b)	4.20	4.42	4.42	4.37	4.44
1A-5(a)	4.05		4.24	4.20	4.20
1A-5(b)	4.02		4.33	4.26	4.26
1A-6(a)	3.98		4.29	4.26	4.27
1A-6(b)	3.98		4.18	4.18	4.22
1A-7(a)	3.29	4.26		4.42	3.92
1A-7(b)	2.81	3.70		4.18	4.24
1A-8(a)	3.48	4.29		4.22	4.37
1A-8(b)	4.02	4.22	-	4.35	4.46

TABLE 4.4 (a) Ultrasonic pulse velocity of series 1A beams

Beam	Positions (km/s)				
	Honeycomb spot	Top	Bottom	Right	Left
1B-2(a)	3.89	4.24	4.29	4.33	4.35
1B-2(b)	3.91	4.37	4.29	4.33	4.35
1B-2R	3.77	4.33	4.24	4.15	4.33
1B-3(a)	4.02	4.52	4.41	4.26	4.37
1B-3(b)	4.03	4.29	4.37	4.37	4.33
1B-3(a)R	3.97	4.35	4.33	4.41	4.37
1B-3(b)R	3.94	4.31	4.27	4.33	4.33
1B-4(a)	3.92	4.33	4.33	4.29	4.31
1B-4(b)	3.98	4.37	4.35	4.35	4.35
1B-4(a)R	3.79	4.35	4.37	4.24	4.33
1B-4(b)R	3.92	4.39	4.29	4.26	4.22
1B-5(a)	3.70		4.17	4.22	4.26
1B-5(b)	3.73		4.24	4.39	4.24
1B-6(a)	3.86	4.29		4.26	4.37
1B-6(b)	3.83	4.31	-	4.33	4.33

TABLE 4.4 (b) Ultrasonic pulse velocity of series 1B beams

Beam	First flexural crack (kN) (development)	Diagonal cracking load (kN)	Diagonal cracking formation	Ultimate load (kN)	Mode of ultimate failure
1A-1(a)	26.0 (significant)	44.0	flexural	68.2	torsion on longer span
1A-1(b)	25.0 (significant)	40.0	flexural	75.7	shear-compression
1A-2(a)	20.0 (not significant)	30 0	independent	35.0	shear-tension/ crushing along crack
1A-2(b)	24.0 (not significant)	24.0	independent	36.3	shear-tension/ tension at the top-arch (buckling)
1A-3(a)	25.0 (quite significant)	36.0	independent	36.0	torsion on shorter span
1A-3(b)	25.0 (quite significant)	34.0	independent	44.0	shear-compression
1A-4(a)	20.0 (quite significant)	35.0	independent	38.4	torsion on shorter span
1A-4(b)	20.0 (quite significant)	28.0	independent	57.0	shear-tension/ shear-compression
1A-5(a)	20.0 (significant)	45.0	independent	50.0	tension at the top-arch (buckling)
1A-5(b)	20.0 (significant)	30.0	independent	75.0	shear-compression/ crushing along crack
1A-6(a)	25.0 (significant)	35.0	flexural	51.0	shear compression
1A-6(b)	20.0 (significant)	35.0	flexural	51.8	shear compression
1A-7(a)	- (not significant)				shear-compression/ shear tension/ crushing along crack
1A-7(b)	25.0 (quite significant)	40.0	flexural	44.0	shear-tension/ crushing along crack
1A-8(a)	20 (significant)	25.0	flexural	65.5	shear compression/ crushing along crack
1A-8(b)	20 (significant)	25.0	flexural	86.0	shear failure on the longer shear span

TABLE 4.5(a) Summary of behaviour of beams (Series 1A)

Beam	First flexural crack(kN) (development)	Diagonal cracking load(kN)	Diagonal cracking formation	Ultimate load (kN)	Mode of ultimate failure
1B-1(a)	12.0 (significant)	40.0	independent	56.5	shear-tension
1B-1(b)	24.0 (significant)	36.0	flexural	69.8	shear-compression/ shear-tension/ crushing along crack, on the long shear span
1B-1 (Repeat)	24.0 (significant)	36 0	independent	53.5	shear-compression
1B-2(a)	20.0 (significant)	28 0	flexural	56.6	shear-compression
1B-2(b)	Data not available as mistake occured during load application				
1B-2 (Repeat)	24.0 (quite significant)	31.0	flexural	42.5	buckling of the top arch
1B-3(a)	20.0 (significant)	36.0	flexural	71.0	shear-tension on the long shear span
1B-3(b)	Test could not be done as the other half of the beam damaged				
1B-3(a) (Repeat)	20.0 (significant)	36.0	independent	53.4	shear-compression
1B-3(b) (Repeat)	20.0 (quite significant)	36.0	independent	45.3	shear-compression
1B-4(a)	12.0 (significant)	32.0	flexural	63.8	shear-compression on long shear span
1B-4(b)	20.0 (not significant)	32.0	independent		shear-compression
1B-4(a) (Repeat)	16.0 (significant)	28.0	flexural	57.5	shear-compression on long shear span
1B-4(b) (Repeat)	Test could not be done as the other half of the beam damaged				
1B-5(a)	20.0 (quite significant)	40.0	independent	54.6	shear-compression
1B-5(b)	16.0 (quite significant)	38.5	independent	46.1	shear-compression/ crushing along crack
1B-6(a)	16.0 (significant)	32.0	independent	39.2	shear-tension
1B-6(b)	16.0 (quite significant)	32.0	independent	35.8	shear-tension

TABLE 4.5(b) Summary of behaviour of beams (Series 1B)

Beam	First flexural crack (kN) (development)	Diagonal cracking		Ultimate load (kN)	Mode of ultimate failure
		load (kN)	formation		
2A-1.1(a)	16.0 (quite significant)	40.0	independent	50.9	shear-compression (buckling of top arch)
2A-1.1(b)	16.0 (quite significant)	24.0	independent	40.5	shear-compression and crush near honeycombing
2A-1.2(a)	20.0 (significant)	40.0	flexural	66.4	shear-compression
2A-1.2(b)	16.0 (significant)	48.0	flexural	71.0	crush at honeycombing, tension at top arch
2A-1.3(a)	20.0 (not significant)	20.0	independent	26.1	top chord failure and anchorage
2A-1.3(b)	16.0 (not significant)	16.0	independent	27.2	top chord failure and anchorage
2A-2(a)	24.0 (significant)	36.0	independent	73.7	shear-compression
2A-2(b)	20.0 (significant)	36.0	independent	54.5	shear-compression
2A-3(a)	20.0 (quite significant)	40.0	flexural	68.1	shear-compression
2A-3(b)	16.0 (significant)	40.0	flexural	77.1	shear-compression
2A-4 (control)	12.0 (significant)	32.0	flexural	35.6	buckling of top arch and anchorage failure
2A-5.1	12.0 (significant)	28.0	flexural	36.8	buckling of top arch and anchorage failure
2A-5.2	12.0 (significant)	28.0	flexural	30.5	buckling of top arch and anchorage failure
2A-6(a)	24.0 (only 2 cracks)	28.0	independent	32.0	buckling of top arch
2A-6(b)	20.0 (only 2 cracks)	28.0	independent	28.0	buckling of top arch
2A-7(a)	24.0 (quite significant)	28.0	independent	46.0	shear-compression
2A-7(b)	20.0 (quite significant)	32.0	independent	66.0	shear-compression
2A-8(a) (control)	16.0 (significant)	40.0	flexural	70.1	shear-compression
2A-8(b) (control)	16.0 (significant)	36.0	flexural	84.9	fail on long shear span
2A-9(a)	12.0 (quite significant)	40.0	flexural	74.0	shear-compression
2A-9(b)	16.0 (significant)	44.0	flexural	88.9	fail on long shear span

TABLE 4.5(c) Summary of behaviour of beams (Series 2A)

Beam	First flexural crack (kN) (development)	Diagonal cracking		Ultimate load (kN)	Mode of ultimate failure
		load (kN)	formation		
2B-1(a)		12.0	independent	19.5	buckling of top arch
2B-1(b)	-	12.0	independent	20.1	buckling of top arch
2B-2(a)	20.0 (quite significant)	36.0	independent	52.0	buckling of top arch
2B-2(b)	20.0 (quite significant)	36.0	independent	47.0	buckling of top arch and crush along diagonal crack
2B-3(a) (control)	20.0 (significant)	48.0	independent	84.3	shear-compression
2B-3(b) (control)	20.0 (significant)	40.0	flexural	85.8	fail on long shear span
2B-4(a)	20.0 (significant)	32.0	flexural	85.9	shear-compression
2B-4(b)	20.0 (quite significant)	24 0	independent	55.7	buckling at top arch, and crushing at honeycombed zone
2B-5 (control)	8.0 (significant)	28.0	independent	42.0	buckling of top arch and anchorage failure
2B-6.1	12.0 (significant)	28.0	flexural	39.1	buckling of top arch
2B-6.2	12.0 (significant)	32.0	flexural	37.9	buckling of top arch and anchorage failure

TABLE 4.5(d) Summary of behaviour of beams (Series 2B)

Beam	Average normalised diagonal cracking load (kN)	The ratio of diagonal cracking load of honeycombed to the control beam
1A-1	43.5	-
1A-2	27.9	0.64
1A-3	34.3	0.79
1A-4	30.8	0.71
1A-5	38.1	0.88
1A-6	35.5	0.82
1A-7	39.1	0.90
1A-8	24.4	0.56

TABLE 4.6(a) Normalised diagonal cracking loads (Series 1A)

Beam	Average normalised diagonal cracking load (kN)	The ratio of diagonal cracking load of honeycombed to the control beam
1B-1	36.7	-
1B-2	29.1	0.79
1B-3	36.9	1.01
1B-4	30.0	0.82
1B-5	40.8	1.11
1B-6	33.3	0.91

TABLE 4.6(b) Normalised diagonal cracking loads (Series 1B)

Beam	Average normalised diagonal cracking load (kN)	The ratio of diagonal cracking load of honeycombed to the control beam
1A-1	42.8	
1A-2	27.5	0.64
1A-3	33.8	0.79
1A-4	30.4	0.71
2A-1.1	33.5	0.78
2A-1.2	46.0	1.07
2A-1.3	18.8	0.44
2A-2	34.7	0.81
2A-3	38.6	0.90
2A-6	28.4	0.66
2A-7	30.4	0.71

TABLE 4.7(a) Normalised diagonal cracking loads (Series 1A and 2A)

Beam	Average normalised diagonal cracking load (kN)	The ratio of diagonal cracking load of honeycombed to the control beam
1B-1	36.9	-
1B-2	29.4	0.79
2B-1	12.1	0.33
2B-2	36.2	0.98

TABLE 4.7(b) Normalised diagonal cracking loads (Series 1B and 2B)

Beam	Average normalised diagonal cracking load (kN)	The ratio of diagonal cracking load of honeycombed to the control beam
***a/d*=3.5**		
2A-4	32.0	
2A-5	28.0	0.88
2B-5	28.0	-
2B-6	30.0	1.07
beam with shear links		
2A-8	38.0	
2A-9	42.0	1.11
2B-3	44.0	
2B-4	28.0	0.64

TABLE 4.8 Normalised diagonal cracking loads, beams with *a/d*=3.5, and beams with shear reinforcement (series 2A and 2B)

Beam	Normalised Ultimate load (kN)	Ratio of ultimate load honeycombed/control	percentage of reserve strength
1A-1	83.8	-	92.6
1A-2	39.5	0.47	41.6
1A-3	41.3	0.49	20.4
1A-4	53.5	0.64	73.7
1A-5	65.4	0.78	71.7
1A-6	53.8	0.64	51.5
1A-7	40.9	0.49	4.6
1A-8	70.4	0.84	188.5

(a) Series 1A

Beam	Normalised Ultimate load (kN)	Ratio of ultimate load honeycombed/control	percentage of reserve strength
1B-1	52.8		43.9
1B-2	47.2	0.89	62.2
1B-3	60.1	1.14	62.9
1B-4	61.5 (L)	1.17	105.0
1B-5	56.6	1.07	38.7
1B-6	42.2	0.80	26.7

Notes: (L): Failure on long shear span

(b) Series 1B

TABLE 4.9 Normalised ultimate load

Beam	Normalised Ultimate load (kN)	Ratio of ultimate load honeycombed/control	percentage of reserve strength
1A-1	80.4		87.7
1A-2	37.8	0.47	37.6
1A-3	39.6	0.49	17.1
1A-4	51.3	0.64	68.7
2A-1.1	52.3	0.65	56.1
2A-1.2	78.6	0.98	70.9
2A-1.3	30.5	0.38	62.2
2A-2	57.5	0.72	65.6
2A-3	65.1	0.81	68.6
2A-6	31.4	0.39	10.4
2A-7	58.5	0.73	92.6

(**a**) Series 1A and 2A

Beam	Normalised Ultimate load (kN)	Ratio of ultimate load honeycombed/control	percentage of reserve strength
1B-1	54.6		47.9
1B-2	48.9	0.90	66.4
2B-1	20.2	0.37	66.5
2B-2	50.4	0.92	39.2

(**b**) Series 1B and 2B

TABLE 4.10 Normalised ultimate load (Series 1A/2A and 1B/2B)

Beam	Normalised Ultimate load (kN)	Ratio of ultimate load honeycombed/control	percentage of reserve strength
***a/d* = 3.5**			
2A-4	35.6	-	11.3
2A-5	33.7	0.95	20.2
2B-5	42.0	-	50.0
2B-6	38.5	0.92	28.3
with shear links			
2A-8	70.1	-	84.5
2A-9	74.0	1.06	84.2
2B-3	84.3	-	91.6
2B-4	70.8	0.84	152.9

TABLE 4.11 Normalised ultimate load (Series 1A/2A and 1B/2B), beam with *a/d*=3.5 and beam with shear reinforcement

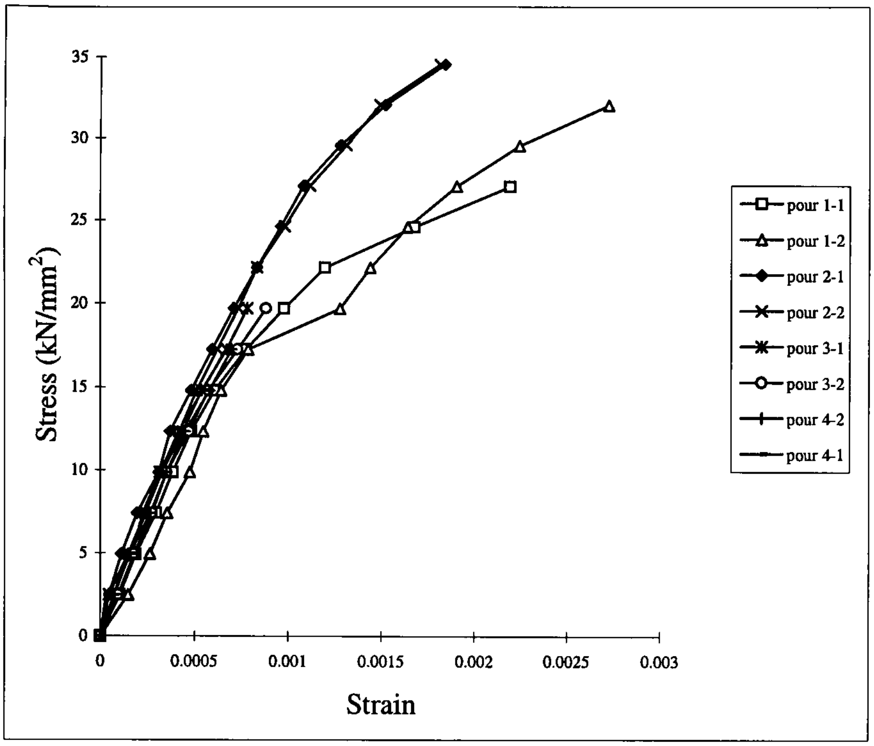


Figure 4.1(a) The stress-strain curves of normal concrete (Series 1A and 2A)

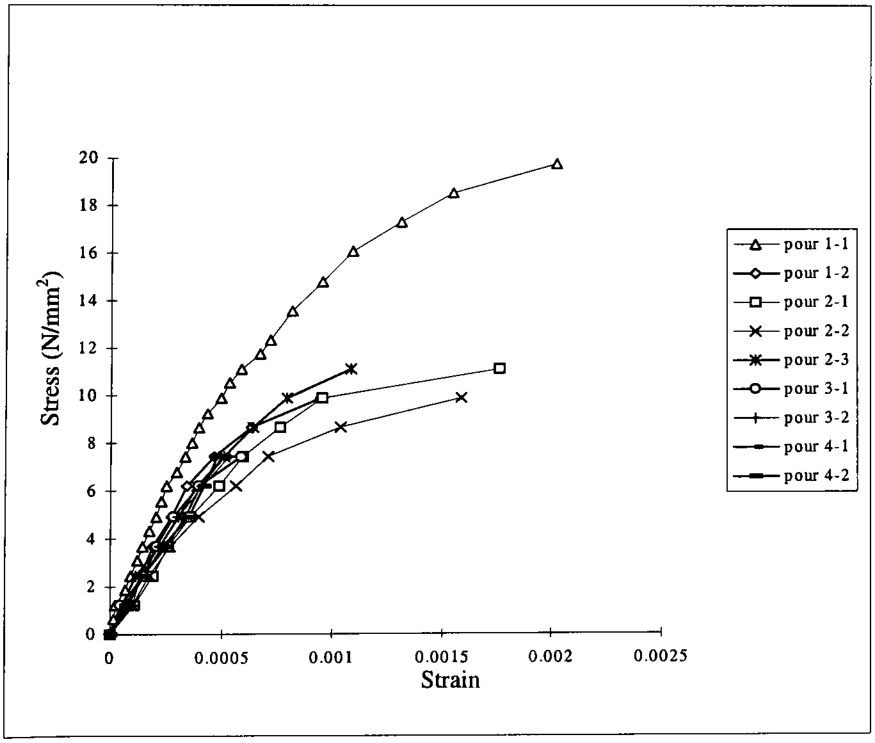


Figure 4.1(b) The stress-strain curves of honeycombed concrete (Series 1A)

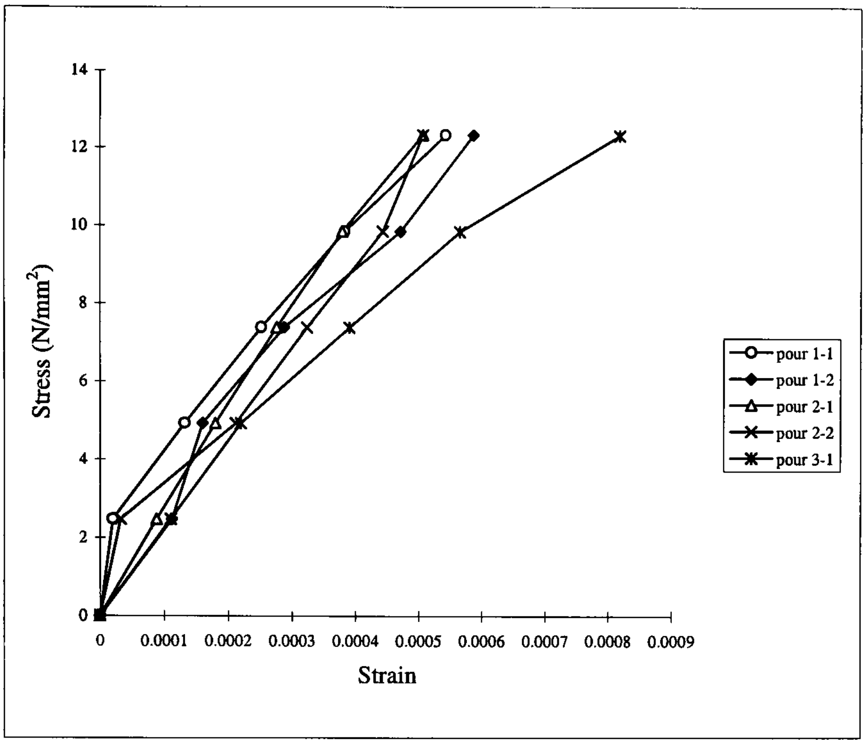


Figure 4.1(c) The stress-strain curves of normal concrete (Series 1B and 2B)

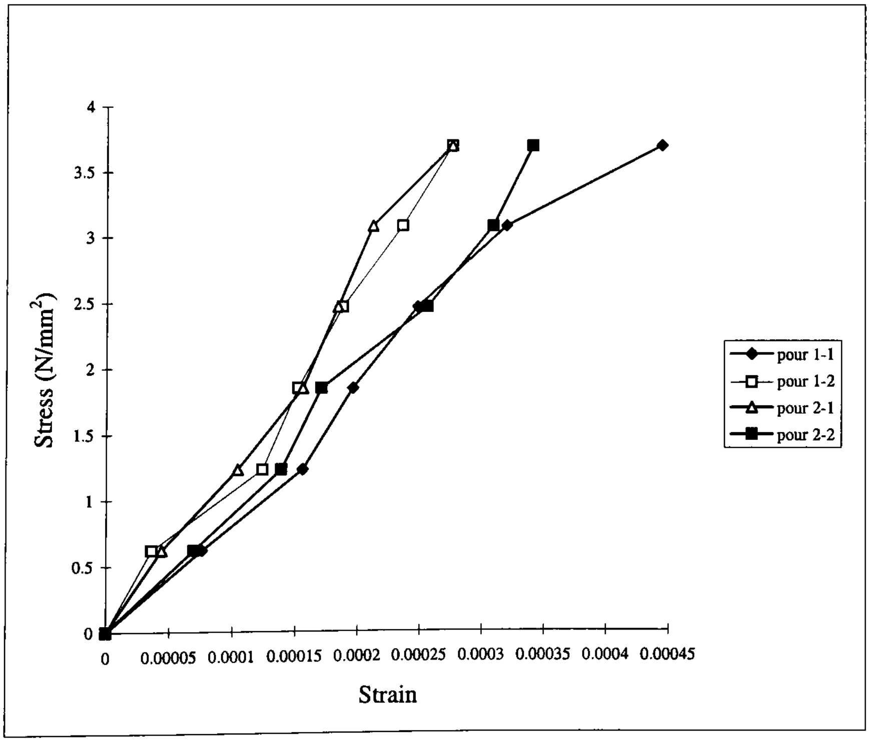


Figure 4.1(d) The stress-strain curves of honeycombed concrete (Series 1B, 2A and 2B)

				3.94				
	4.29			3.92			4.41	
		4.18		4.15		4.12		
			4.08		4.12			
4.27	4.35	4.00		4.10		4.18	4.24	4.42
			4.17		4.27			
		4.26		4.18		4.26		
	4.26			4.17			4.42	
				4.29				

Beam 2A-2

				4.33				
	4.15			4.24			4.29	
		4.00		4.08		4.27		
			4.05		4.12			
4.26	4.26	4.02		3.94		4.05	4.27	4.20
			4.08		4.10			
		4.27		4.10		3.88		
	4.31			4.07			4.37	
				4.18				

Beam 2A-3

FIGURE 4.2(a) Ultrasonic pulse velocity (in km/s) of Beams 2A-2 and 2A-3 at the area around a honeycombed zone measured on 15 mm grids. The dark lines indicate the boundary of honeycombed zone.

4.42

				4.50			
				4.46			
				4.42			
				3.91			
4.48	4.44	4.46	3.76	3.61	3.92	4.46	4.42
				3.89			
				3.82			
				3.80			
				4.37			

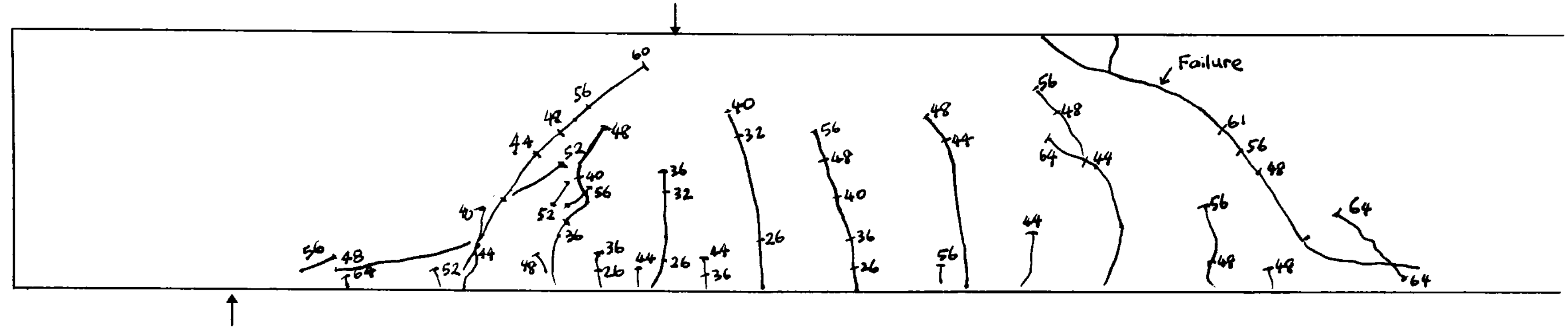
Beam 2A-1.1

4.35

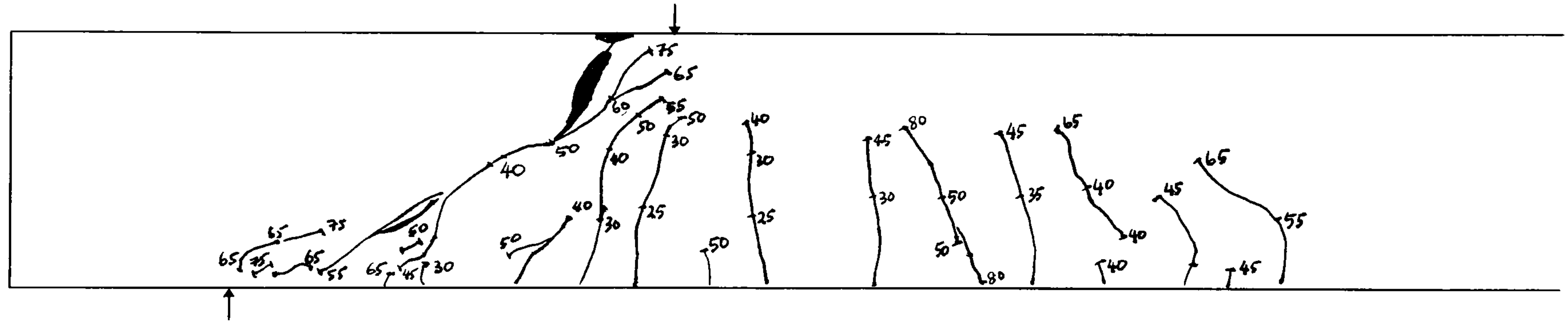
				4.35			
	4.33			4.44			4.42
		4.33		4.39		4.20	
			4.00		3.98		
4.48	4.44	4.24		3.50		3.98	4.41
			3.77		3.94		
		4.20		4.13		4.15	
	4.29			4.26			4.39
				4.37			

Beam 2A-5

FIGURE 4.2(b) Ultrasonic pulse velocity (in km/s) of Beams 2A-1.1 and 2A-5 at the area around a honeycombed zone measured on 15 mm grids. The dark lines indicate the boundary of honeycombed zone.

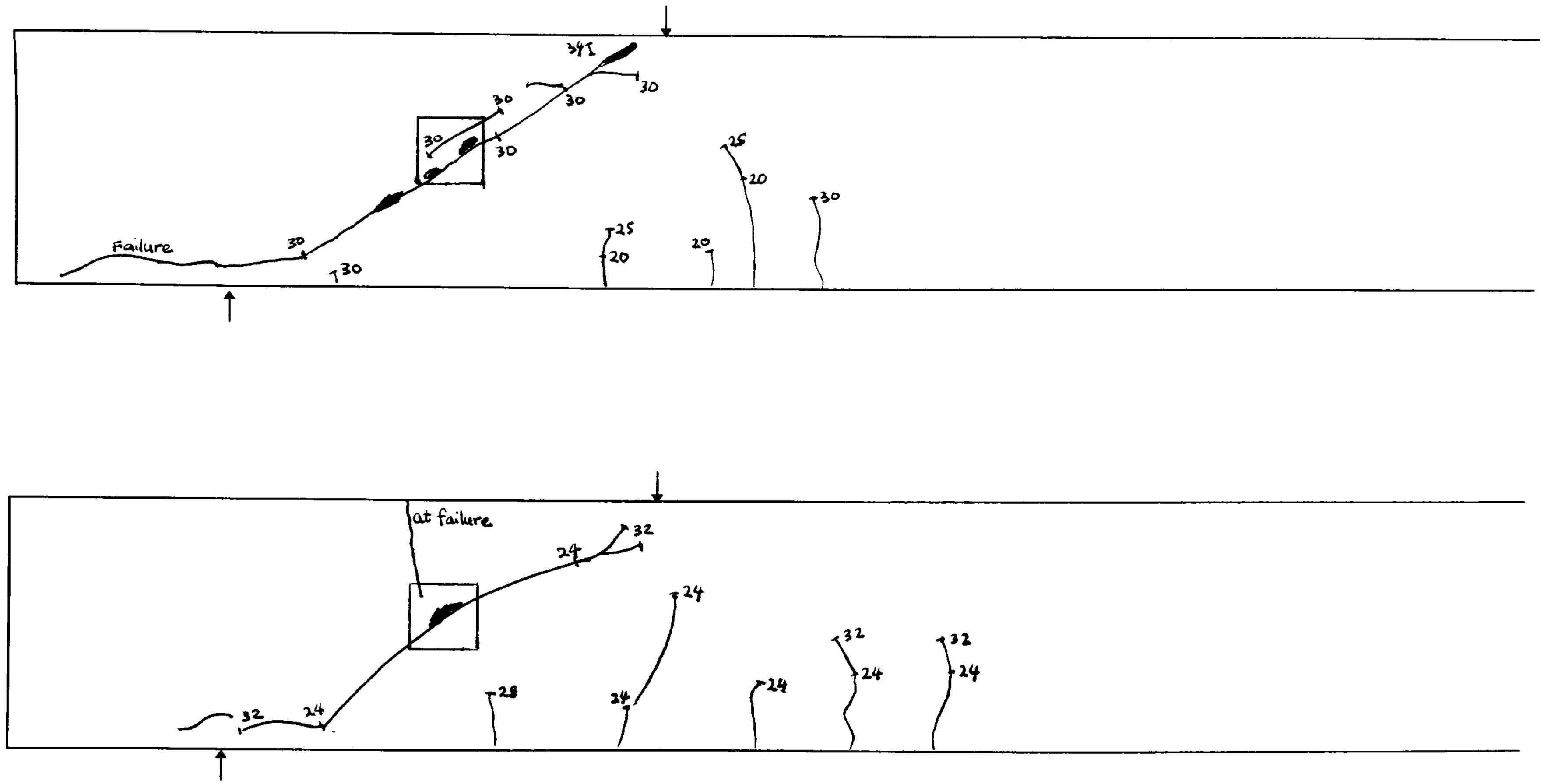


1A-1(a)



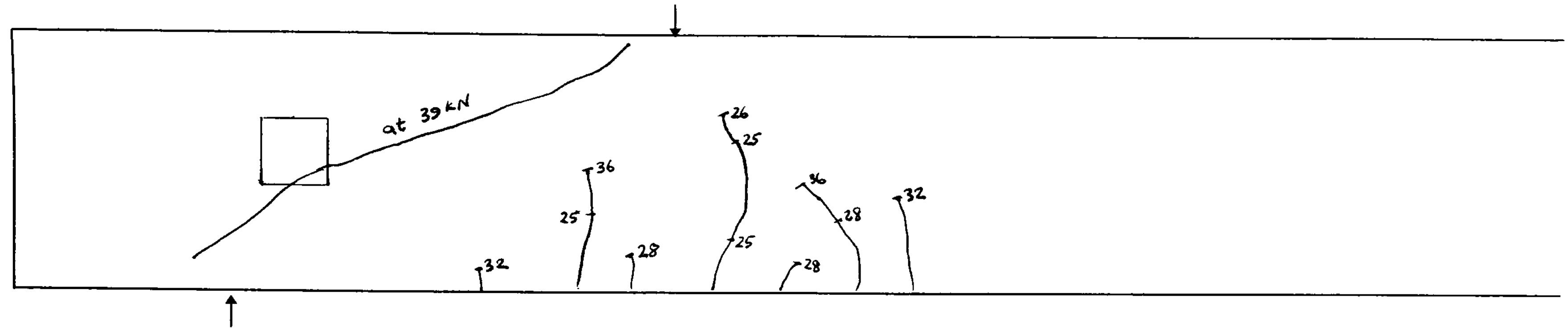
1A-1(b)

FIGURE 4.3(a)

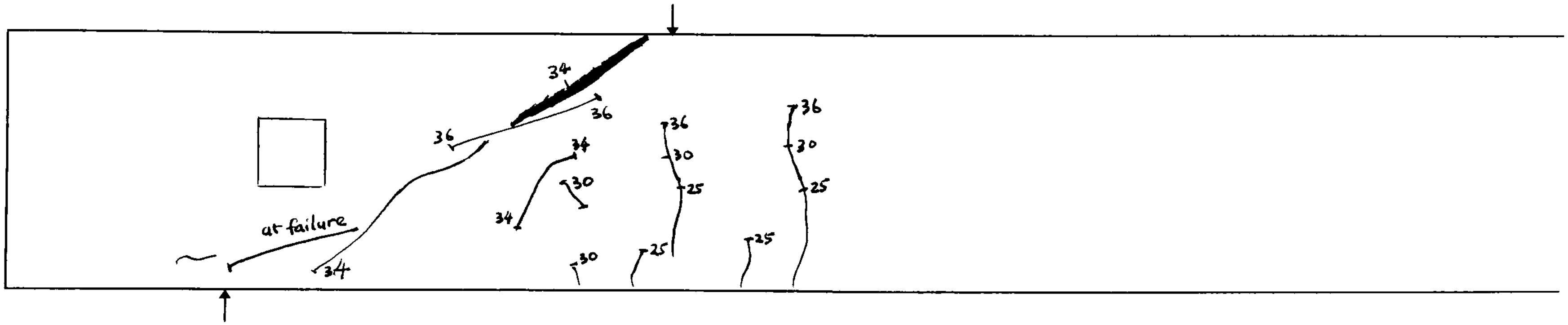


1A-2(b)

FIGURE 4.3(a)- contd

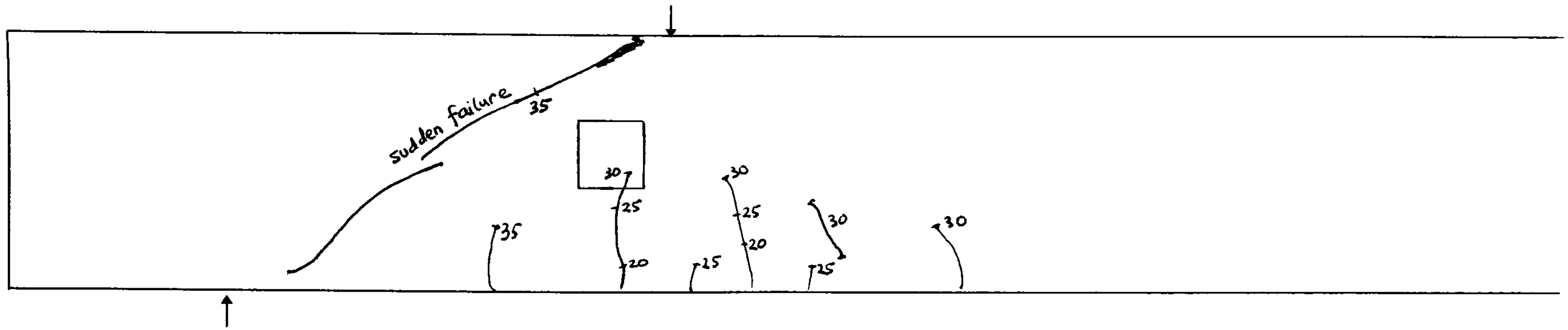


1A-3(a)

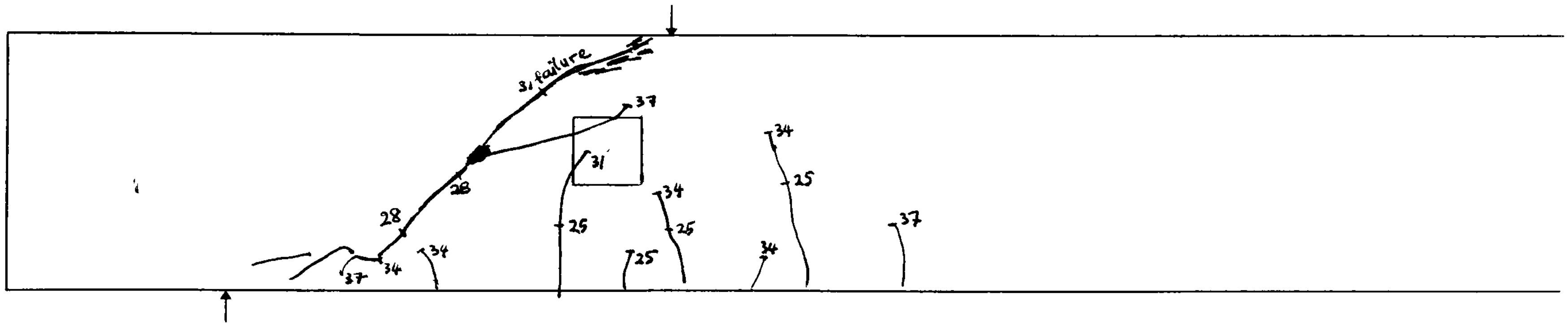


1A-3(b)

FIGURE 4.3(a)- contd

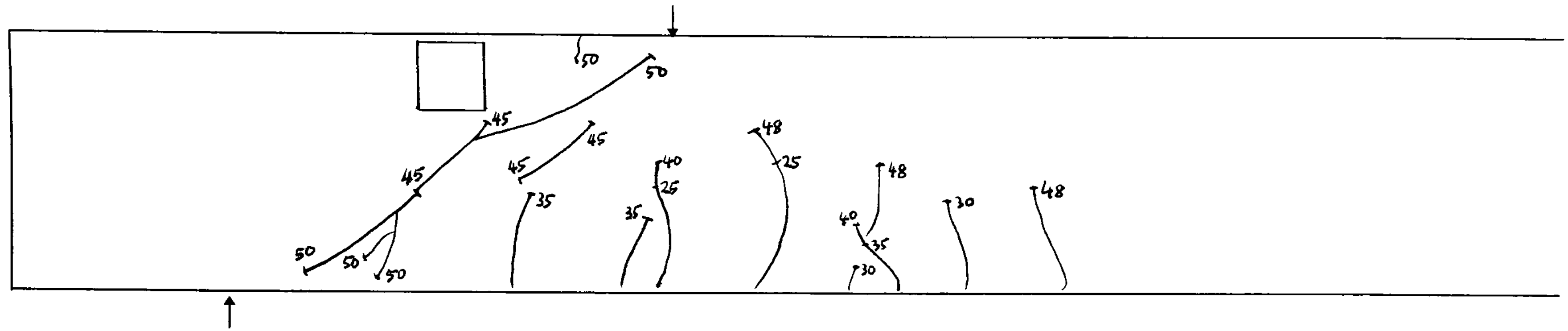


1A-4(a)

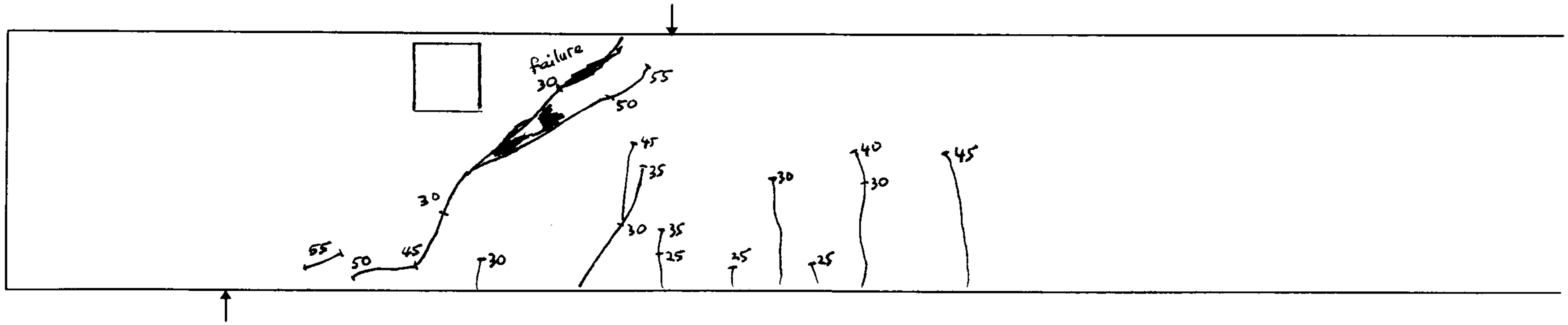


1A-4(b)

FIGURE 4.3(a)- contd

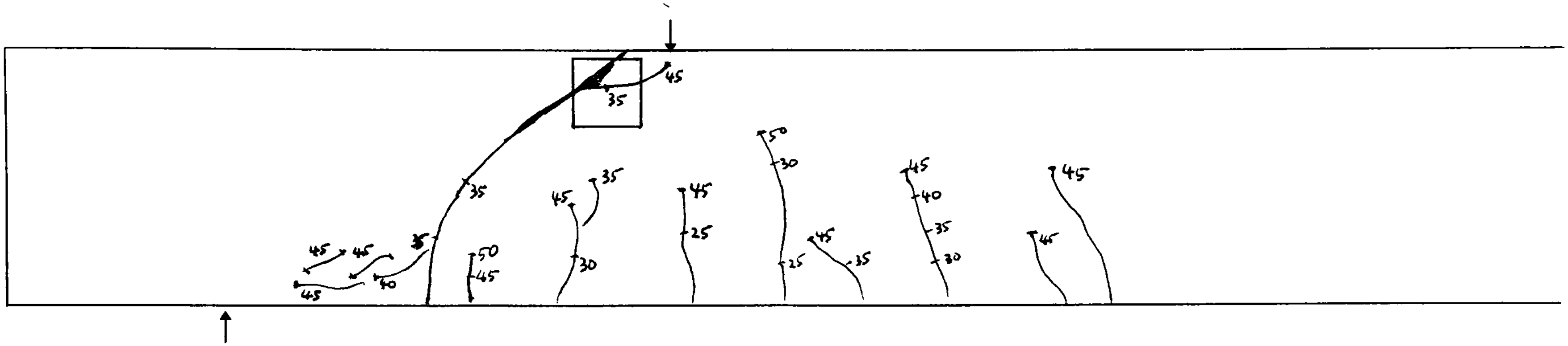


1A-5(a)

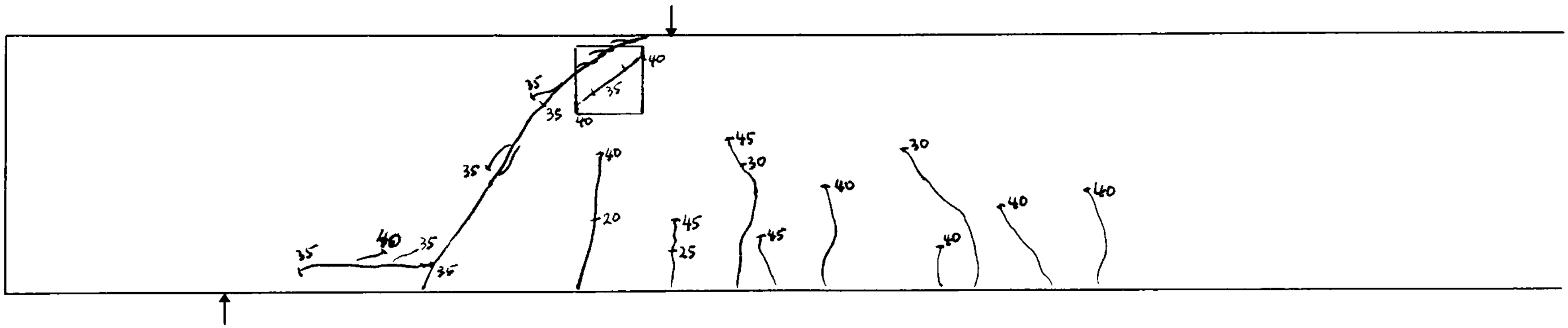


1A-5(b)

FIGURE 4.3(a)- contd

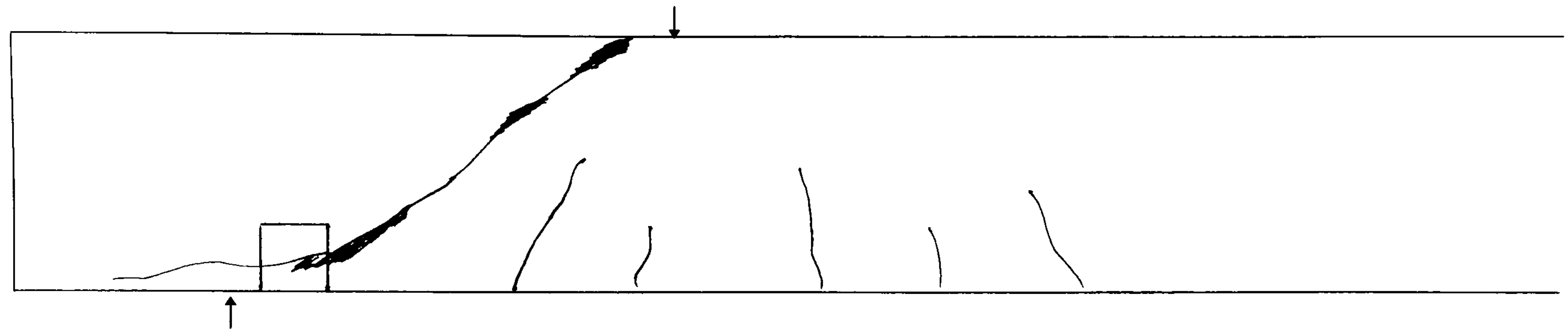


1A-6(a)

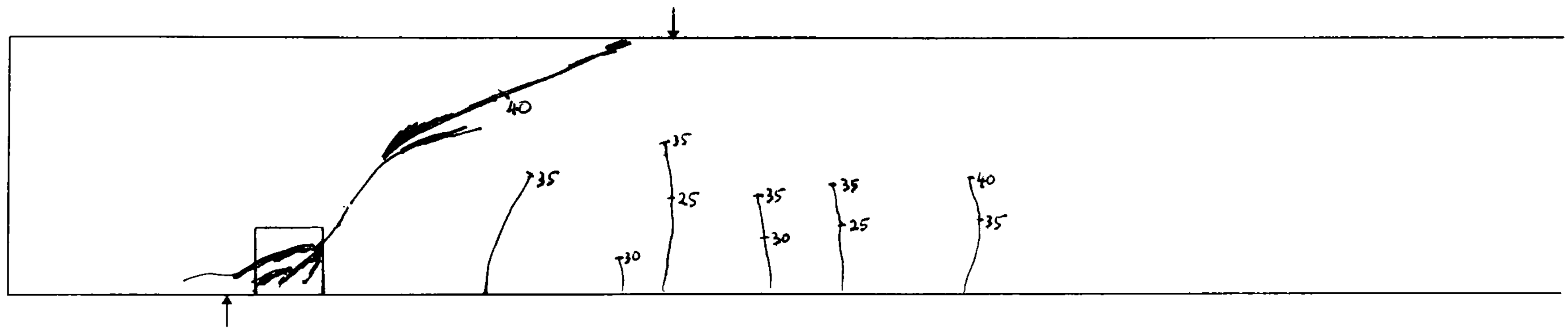


1A-6(b)

FIGURE 4.3(a)- contd

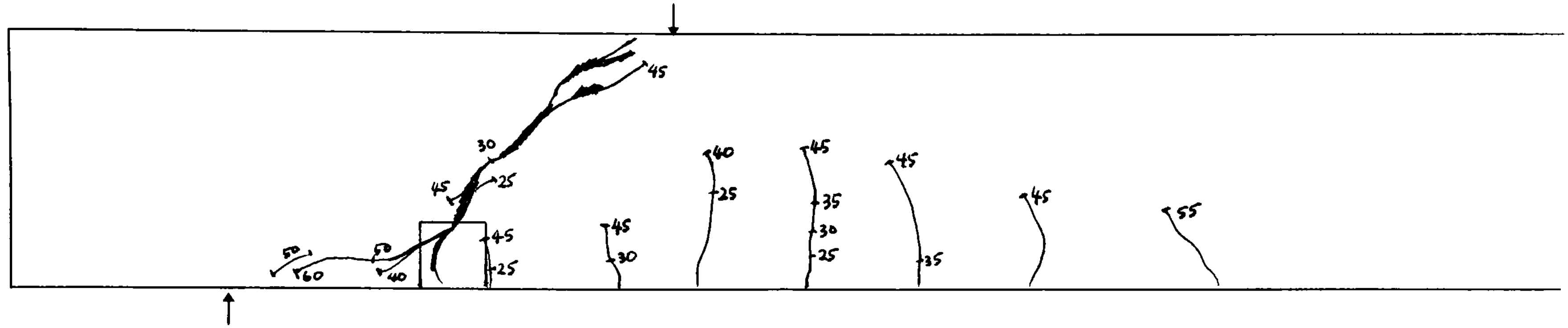


1A-7(a)

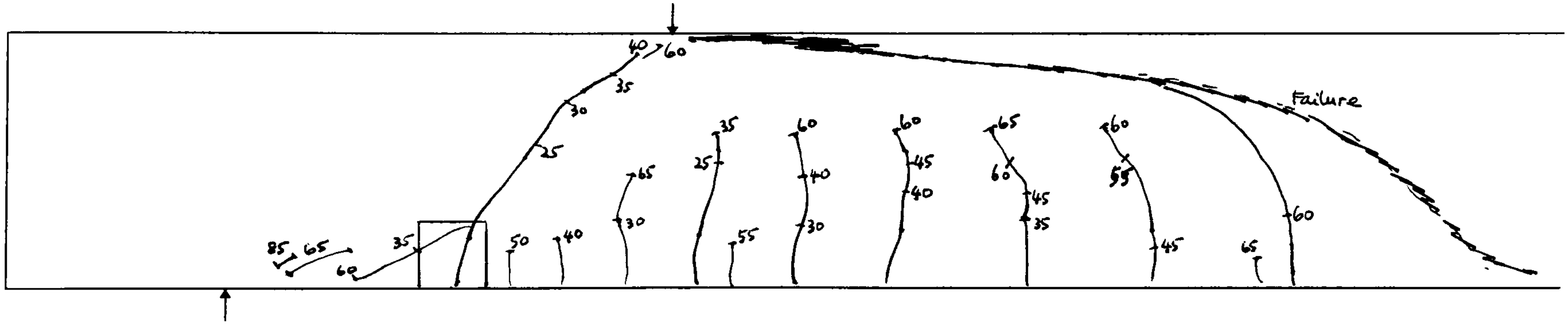


1A-(7(b)

FIGURE 4.3(a)- contd

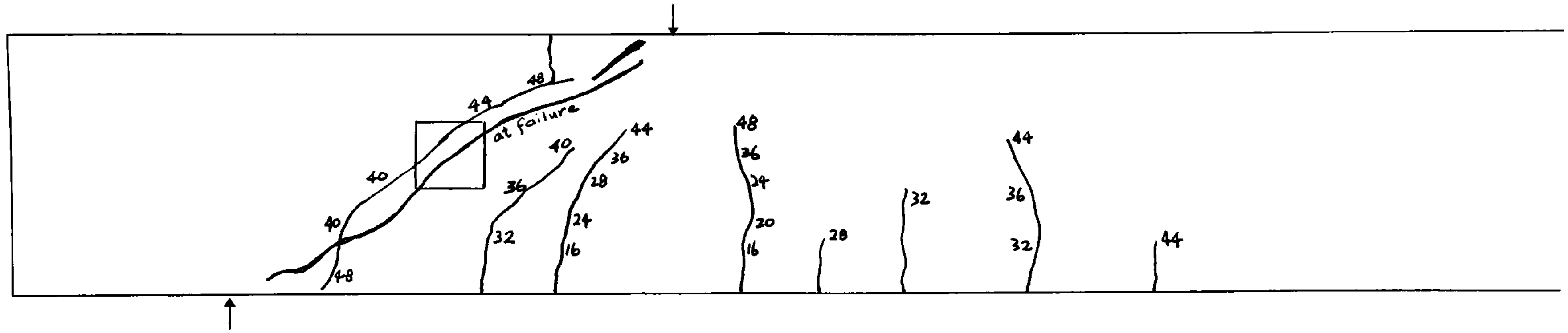


1A-8(a)

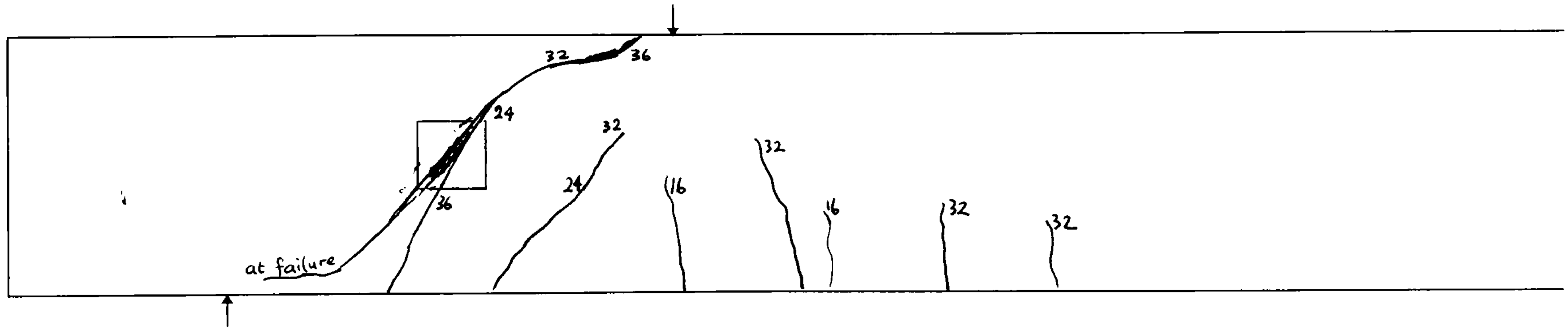


1A-8(b)

FIGURE 4.3(a)- contd

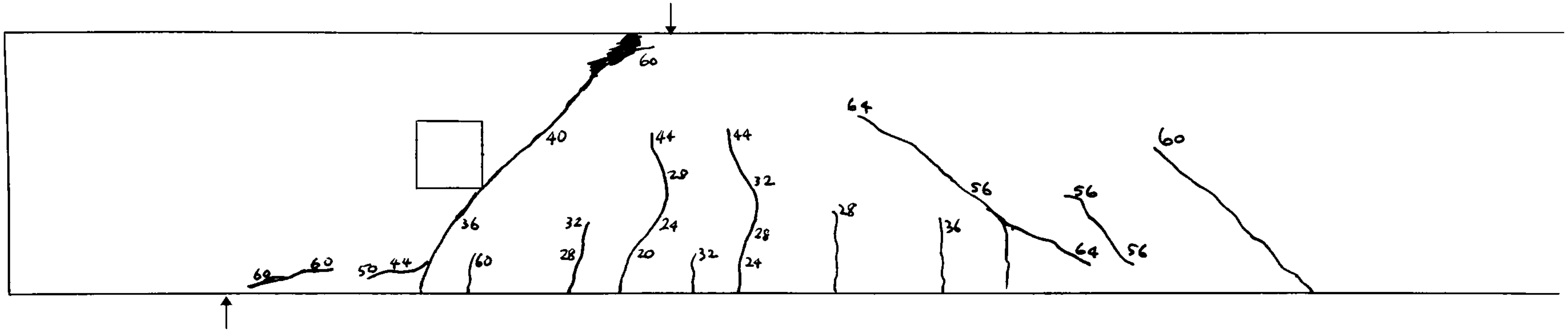


2A-1.1(a)

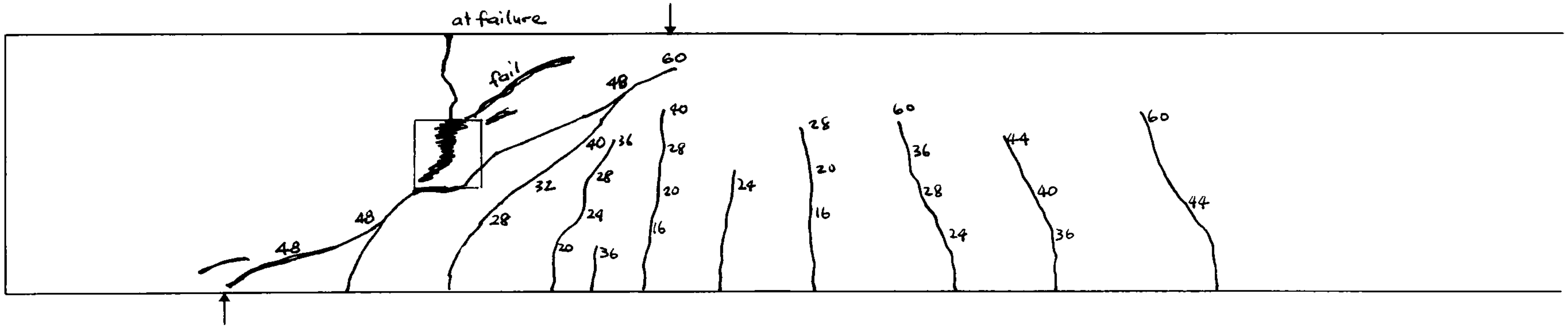


2A-1.1(b)

FIGURE 4.3(b)

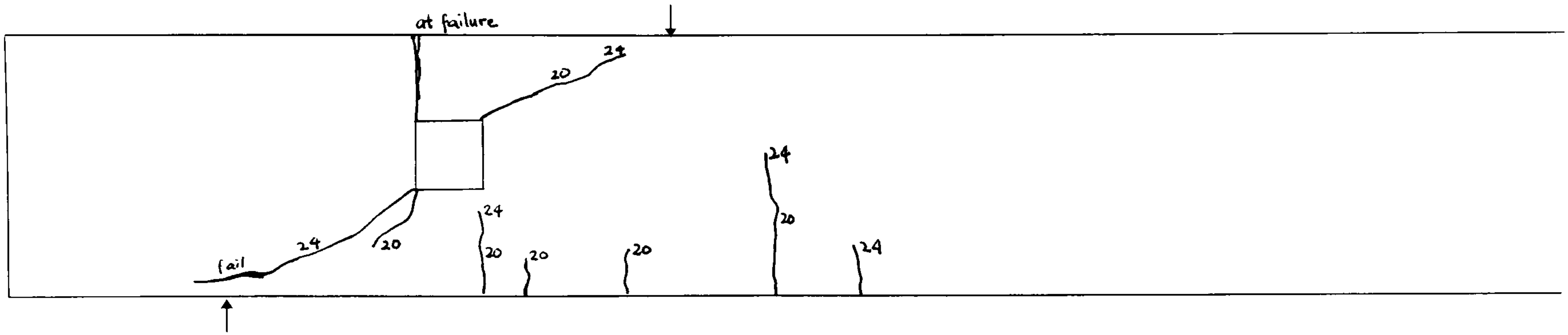


2A-1.2(a)

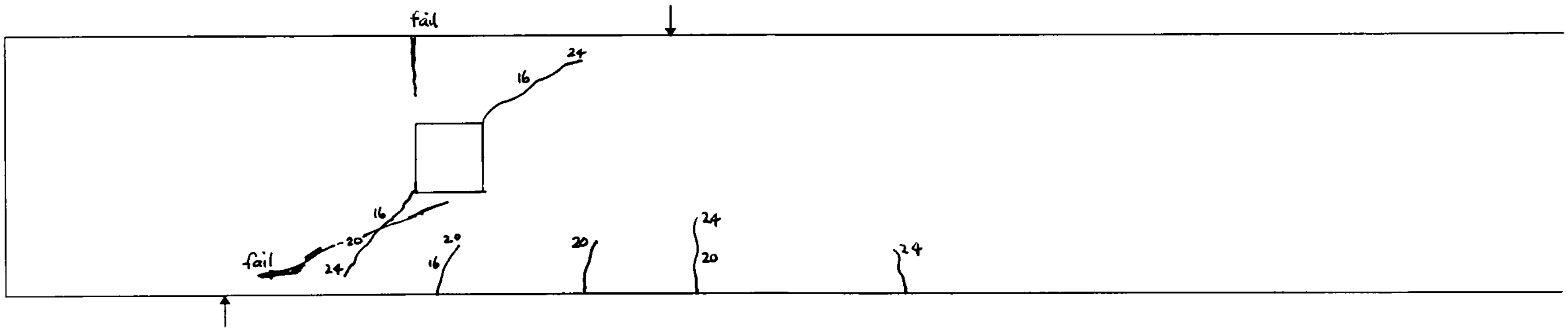


2A-1.2(b)

FIGURE 4.3(b)- contd

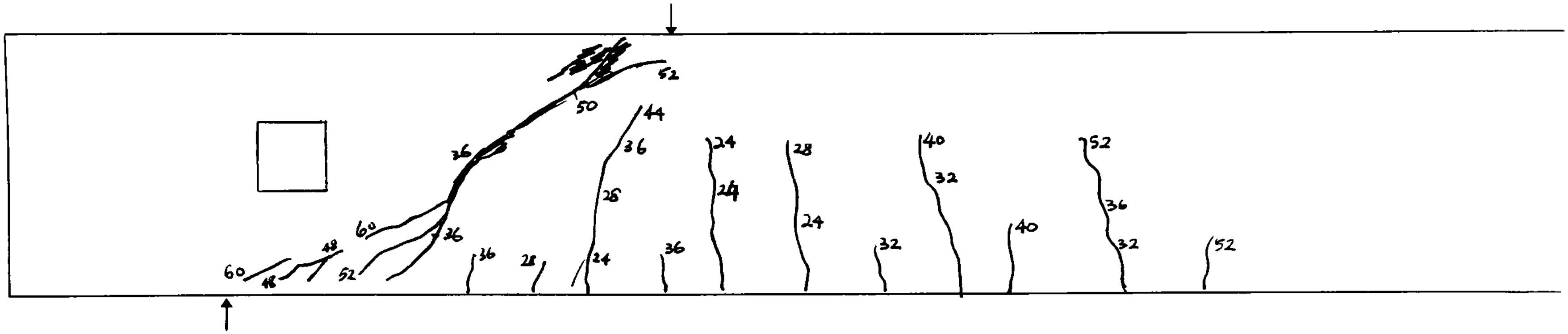


2A-1.3(a)

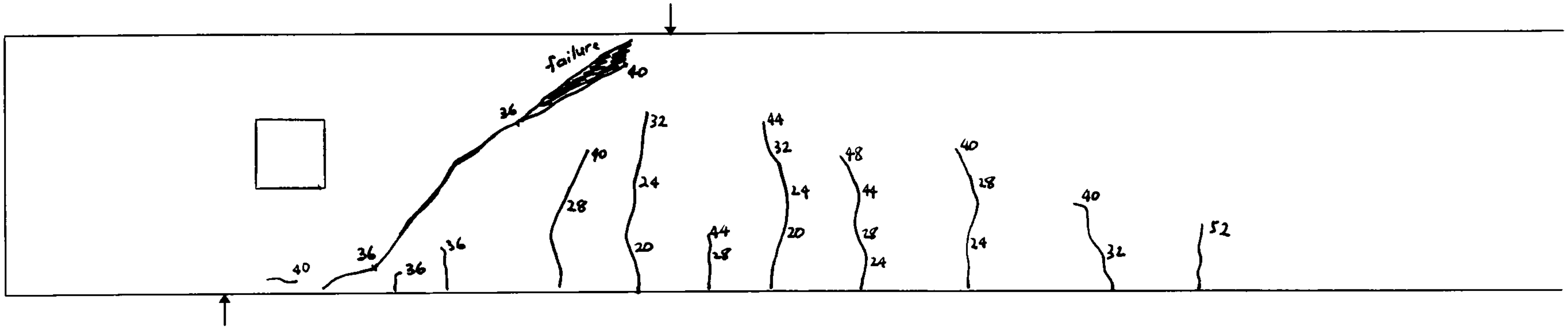


2A-1.3(b)

FIGURE 4.3(b)- contd

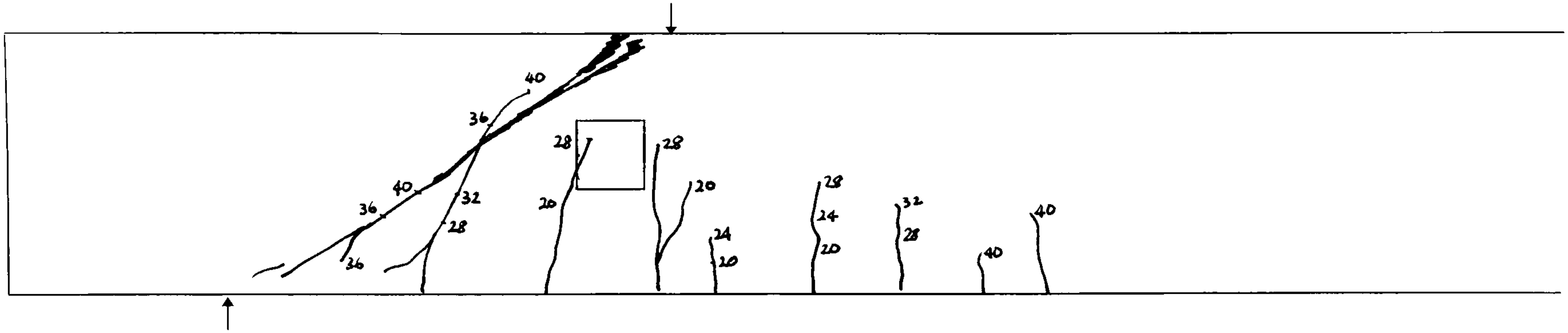


2A-2(a)

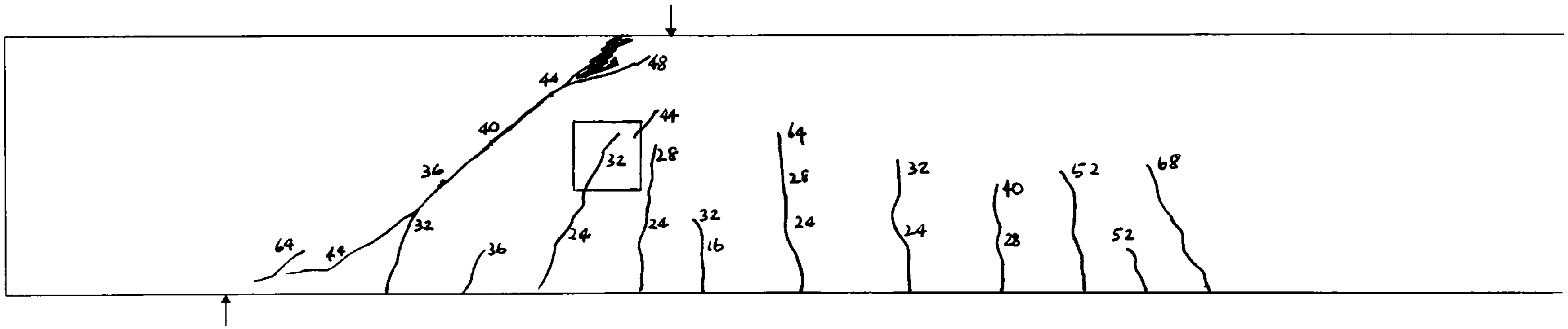


2A-2(b)

FIGURE 4.3(b)- contd

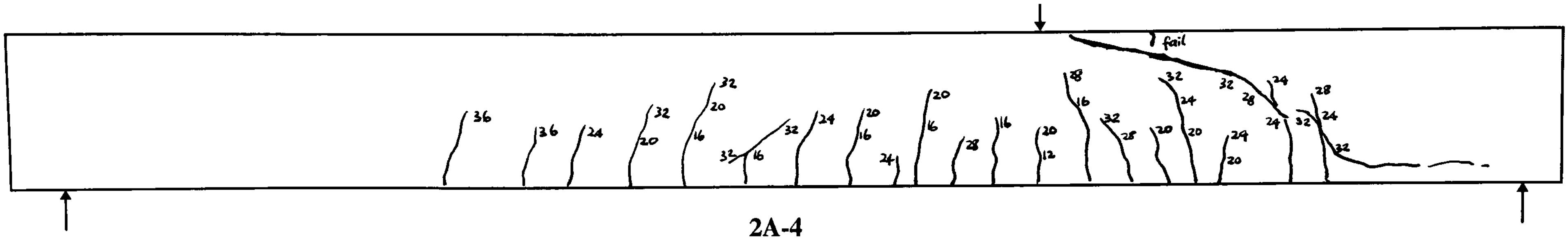


2A-3(a)

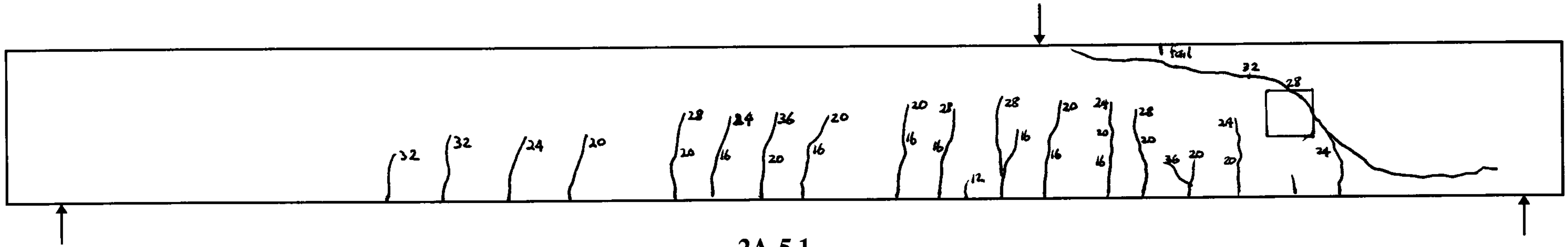


2A-3(b)

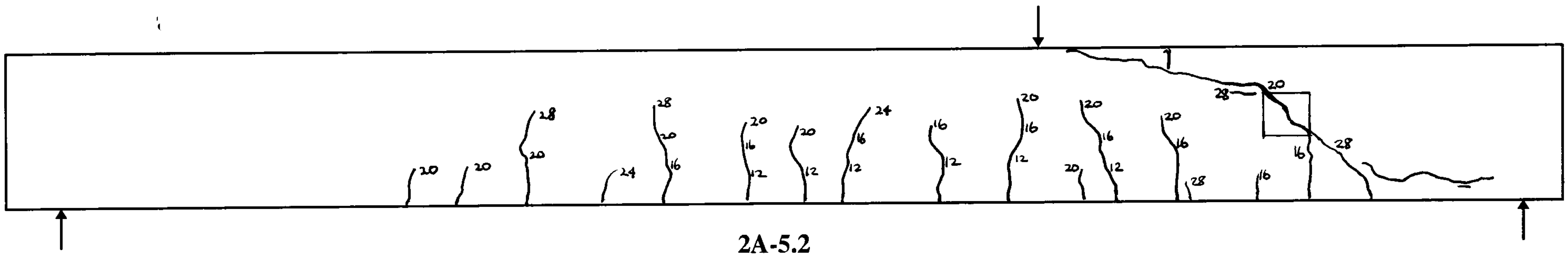
FIGURE 4.3(b)-contd



2A-4

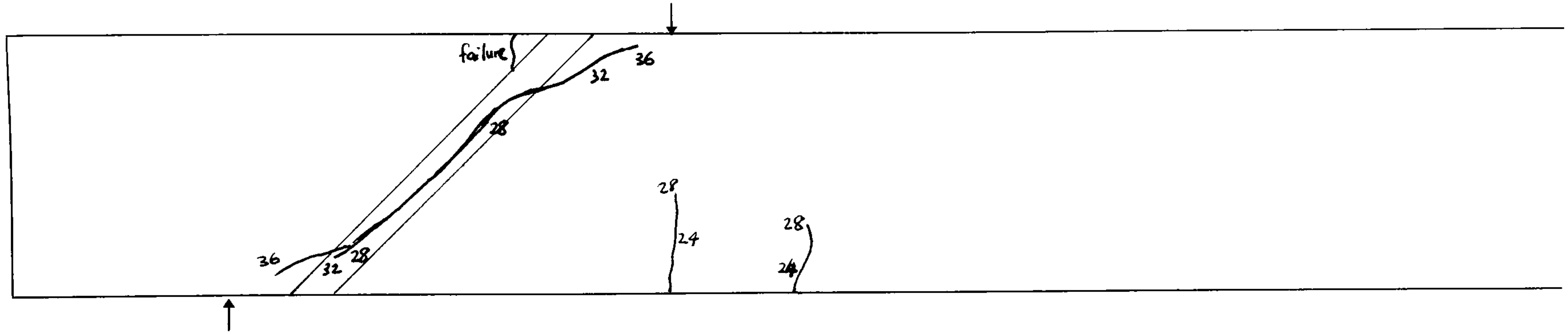


2A-5.1

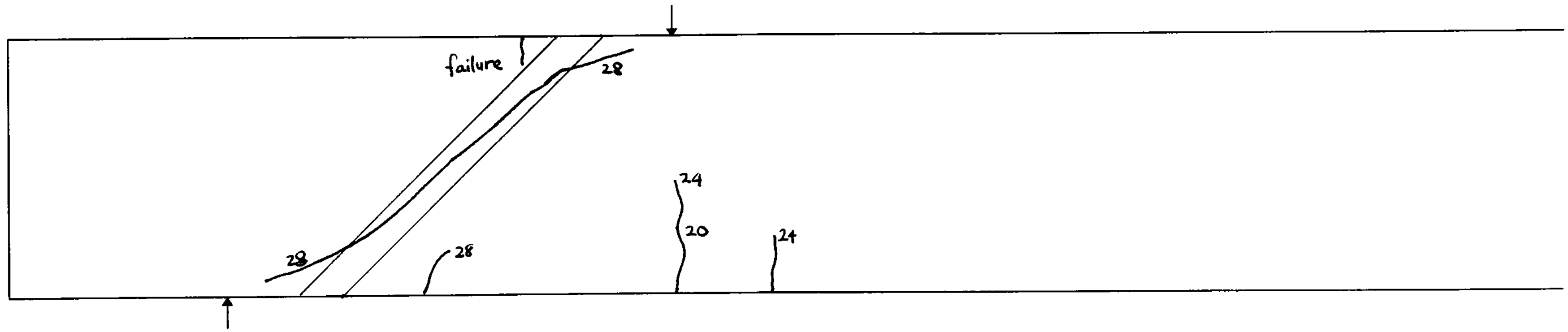


2A-5.2

FIGURE 4.3(b)- contd

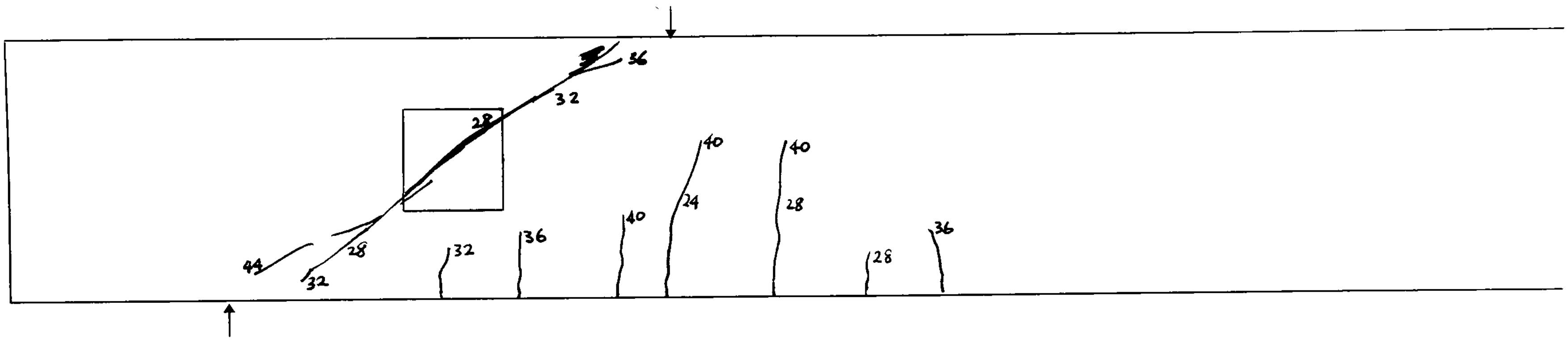


2A-6(a)

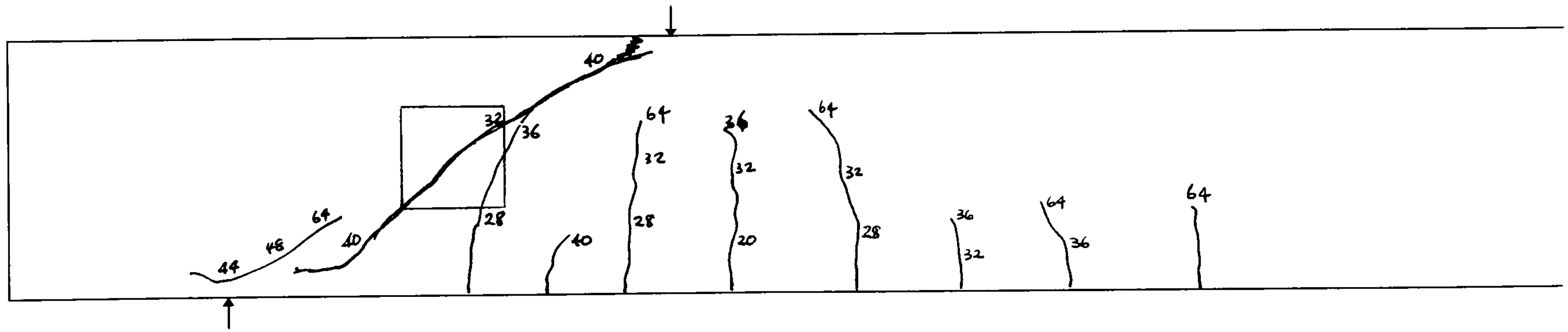


2A-6(b)

FIGURE 4.3(b)-contd

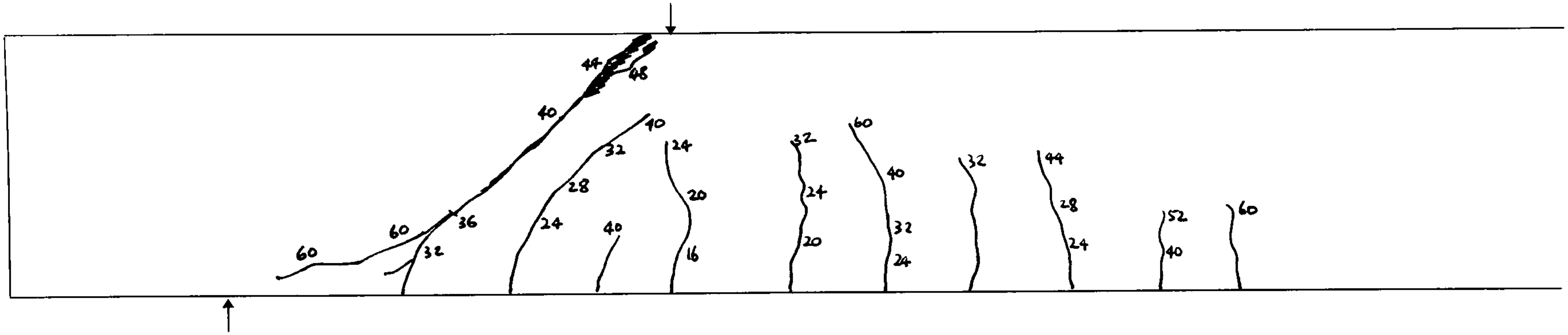


2A-7(a)

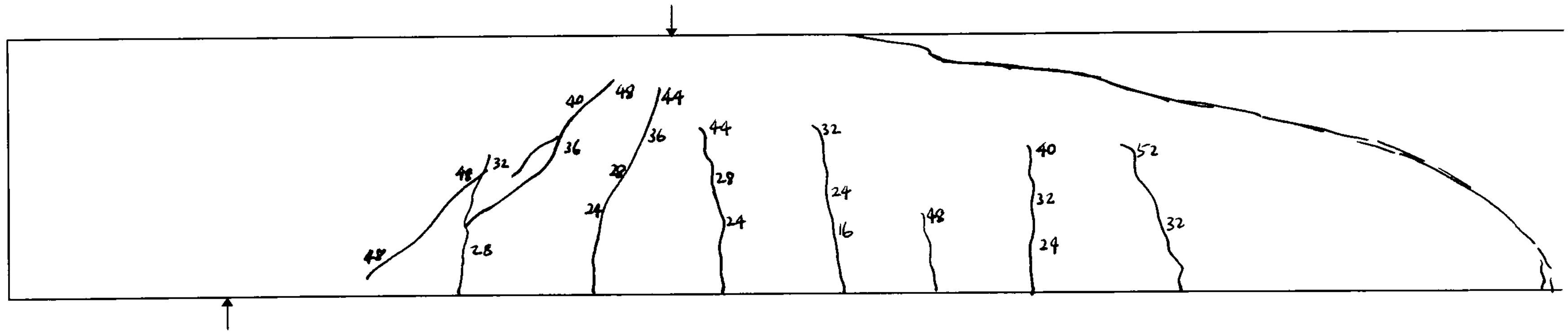


2A-7(b)

FIGURE 4.3(b)- contd

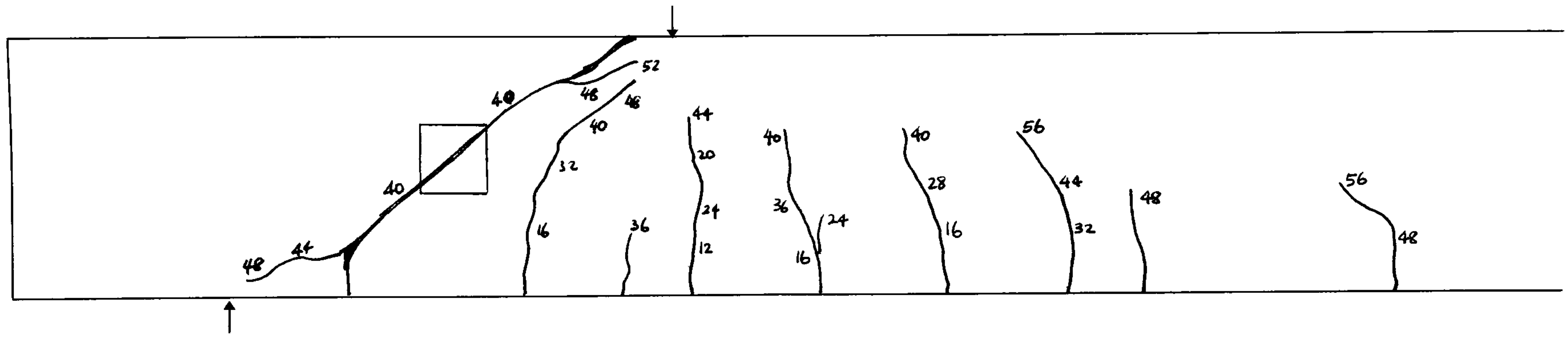


2A-8(a)

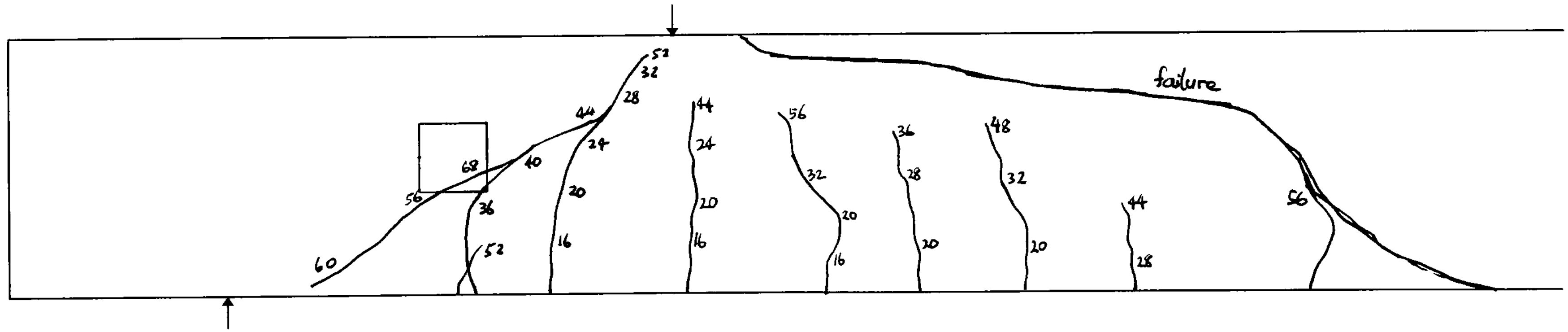


2A-8(b)

FIGURE 4.3(b)- contd

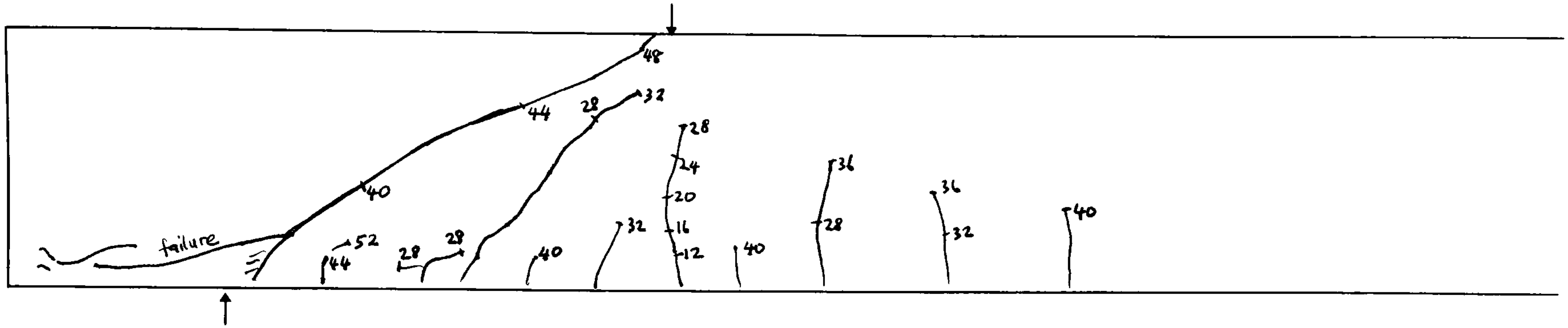


2A-9(a)

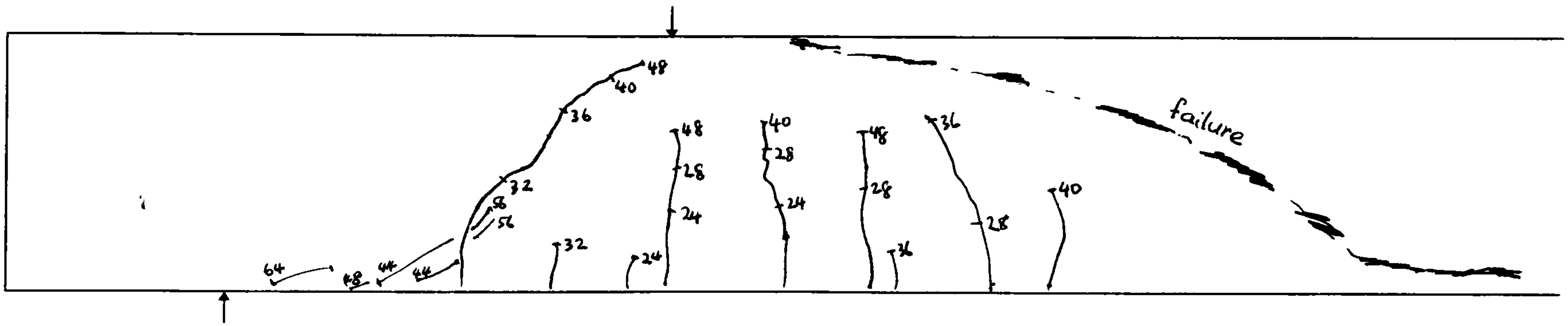


2A-9(b)

FIGURE 4.3(b)-contd

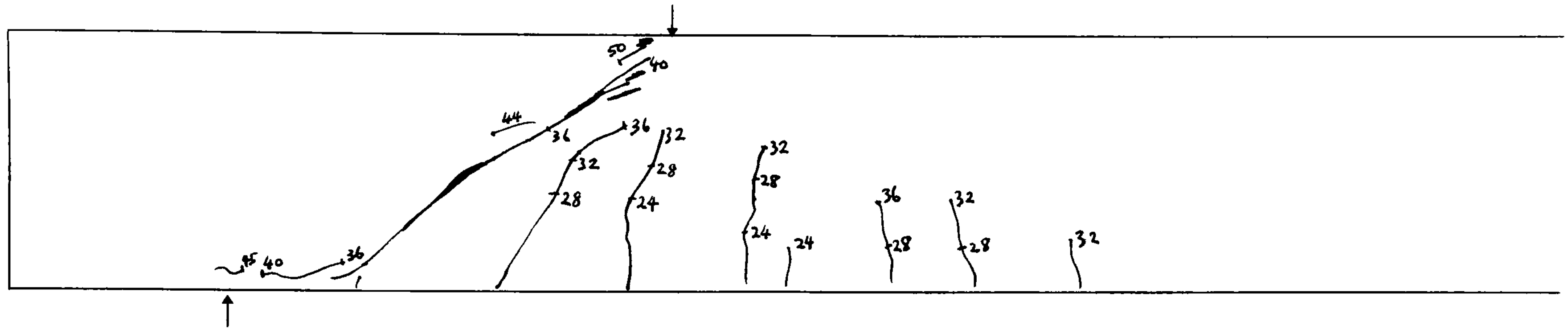


1B-1(a)

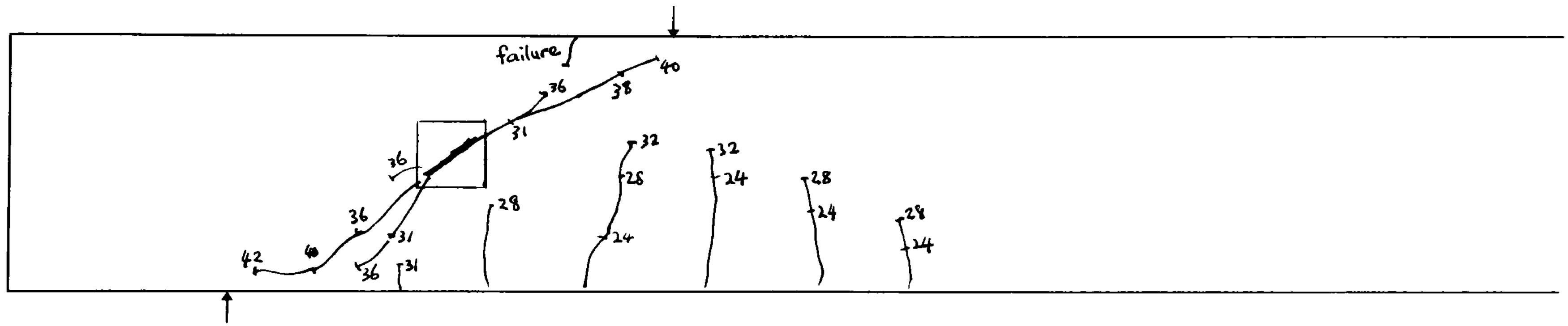


1B-I(b)

FIGURE 4.3(c)

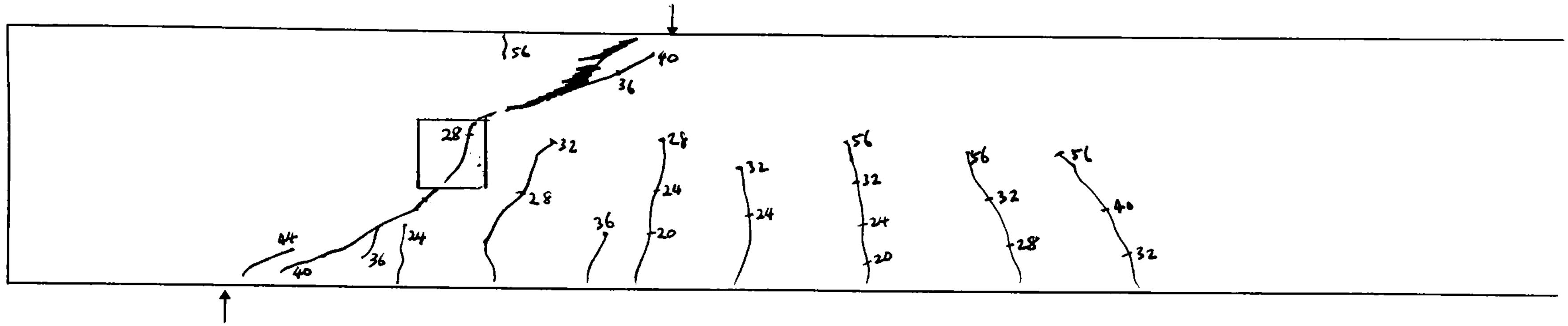


1B-1(R)

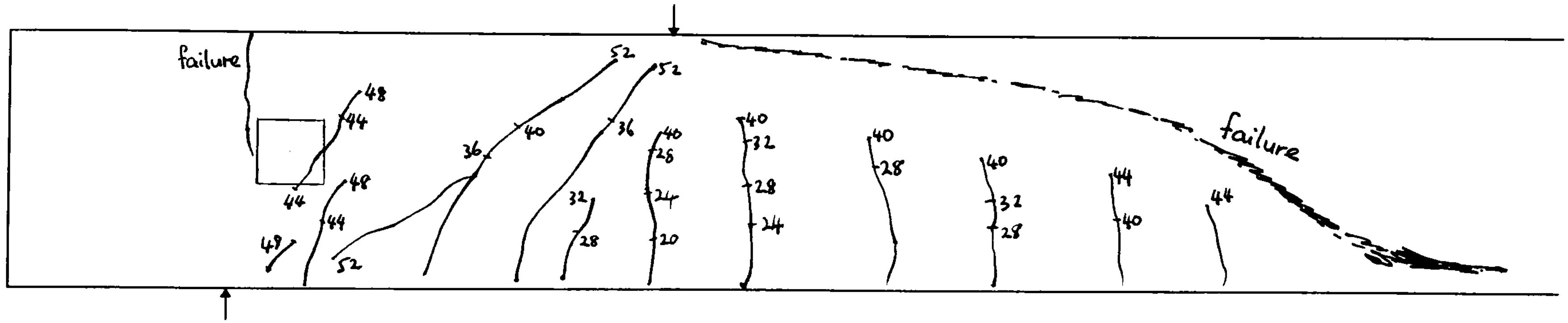


1B-2(R)

FIGURE 4.3(c)- contd

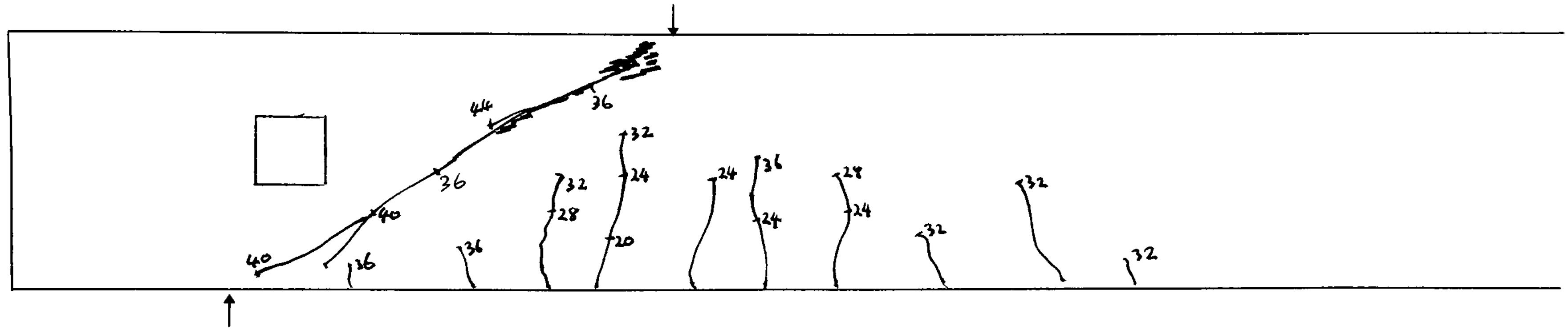


1B-2(a)

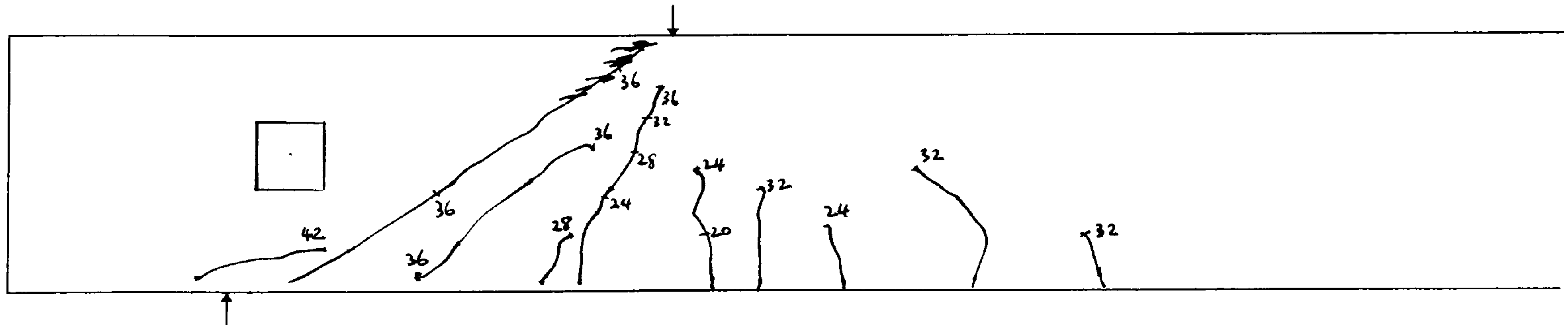


1B-3(a)

FIGURE 4.3(c)- contd

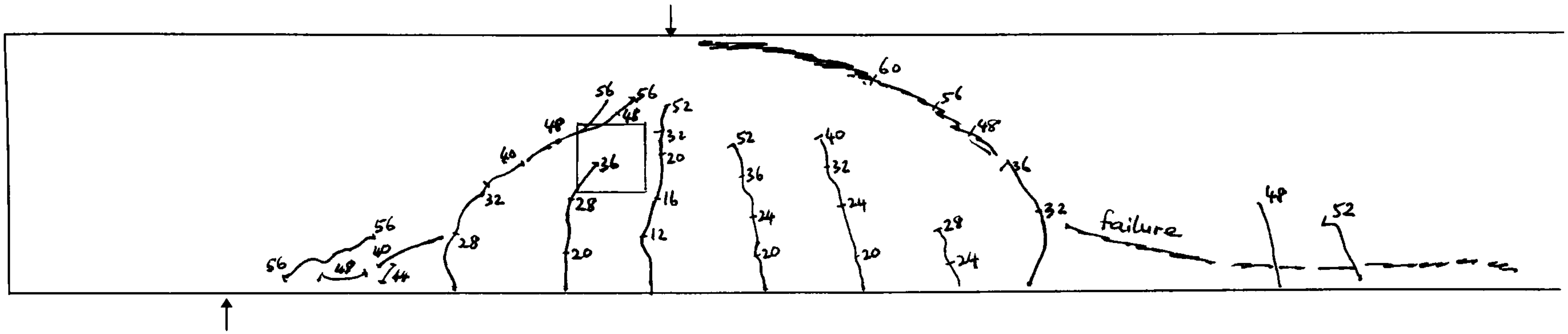


1B-3(a)R

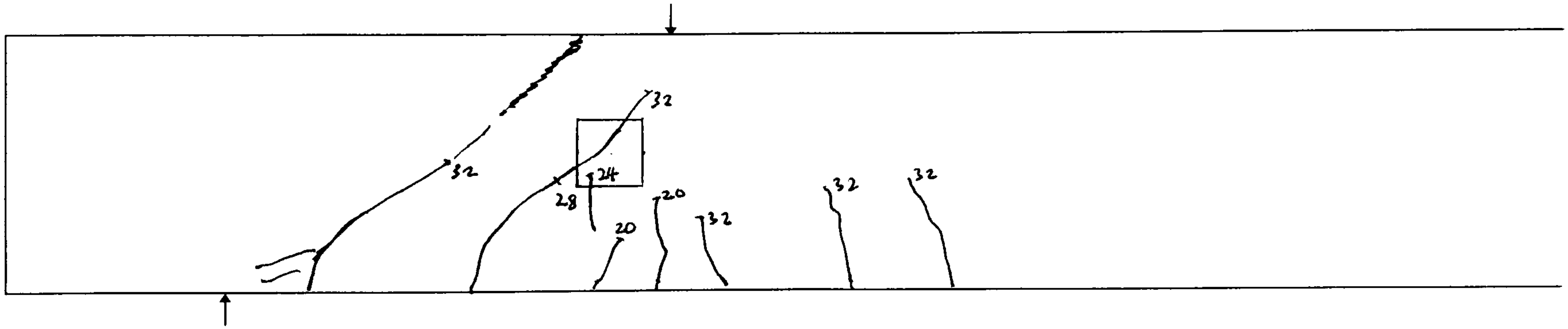


1B-3(b)R

FIGURE 4.3(c)- contd

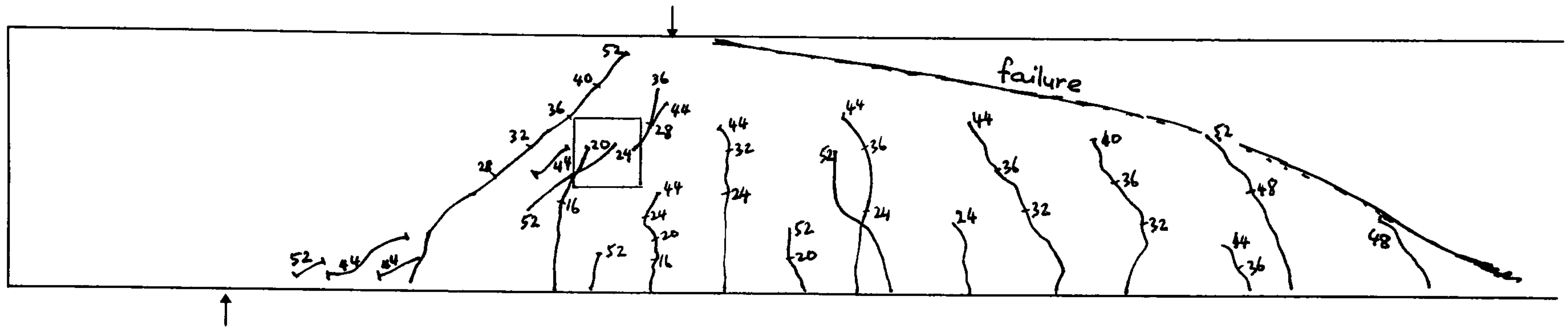


1B-4(a)



1B-4(b)

FIGURE 4.3(c)-contd



1B-4(a)R

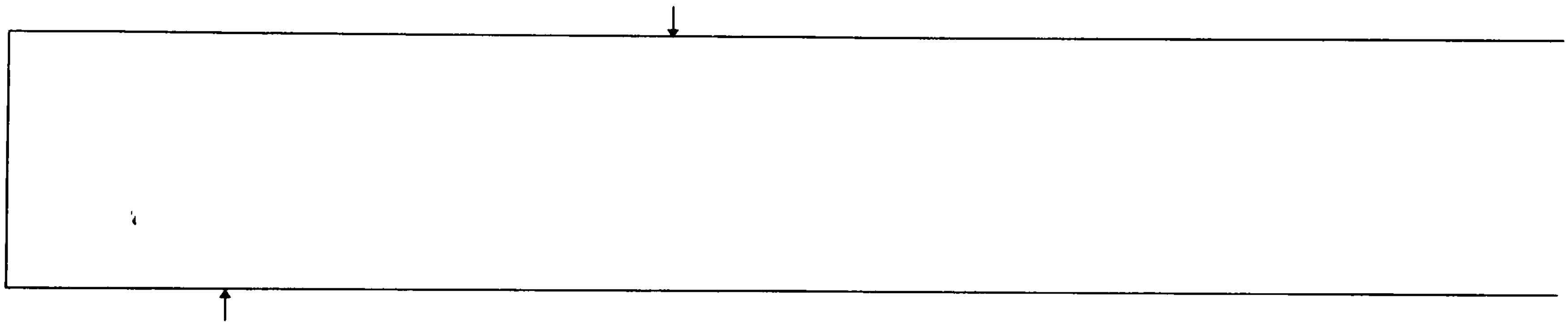
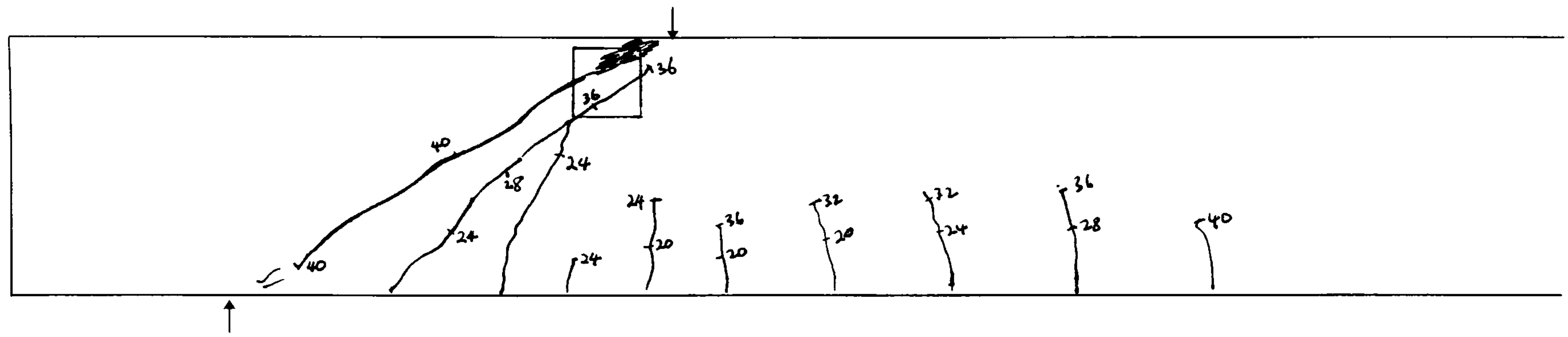
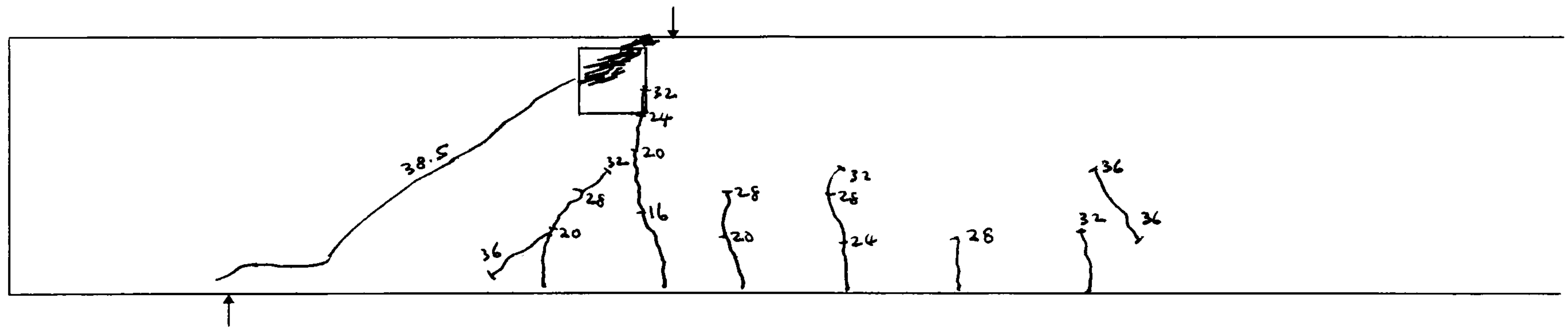


FIGURE 4.3(c)- contd

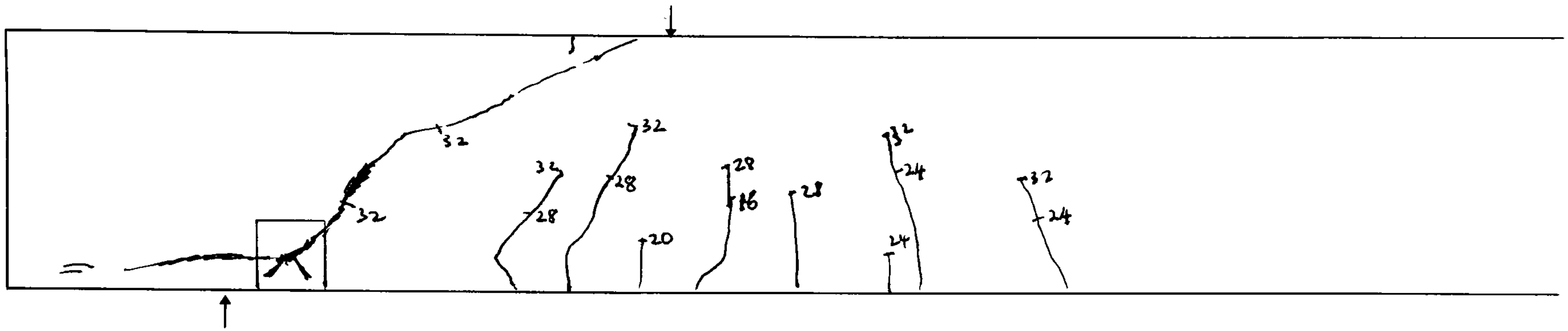


1B-5(a)

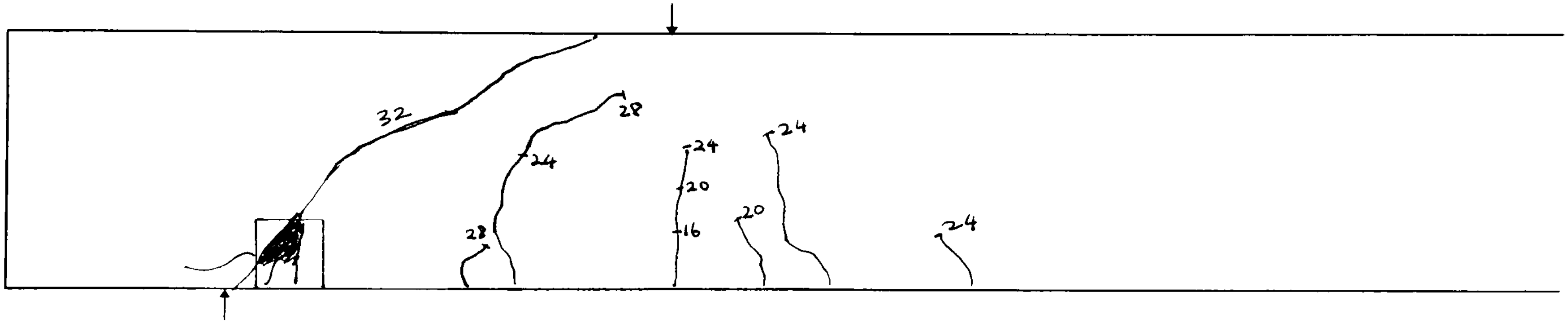


1B-5(b)

FIGURE 4.3(c)-contd

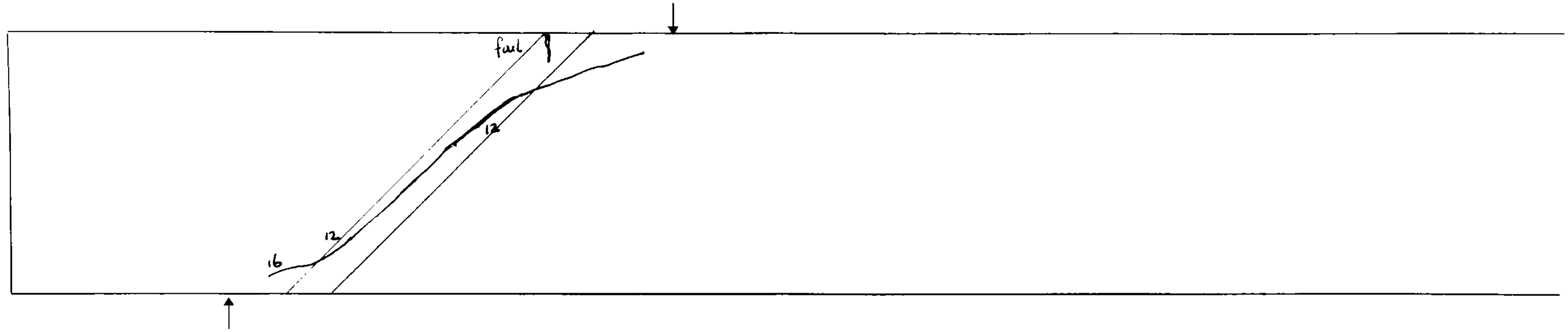


1B-6(a)

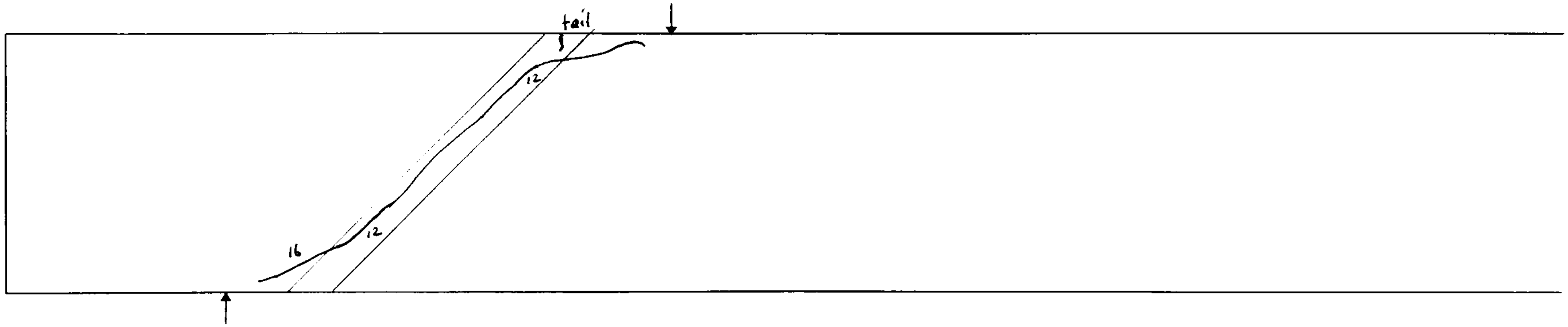


1B-6(b)

FIGURE 4.3(c)- contd

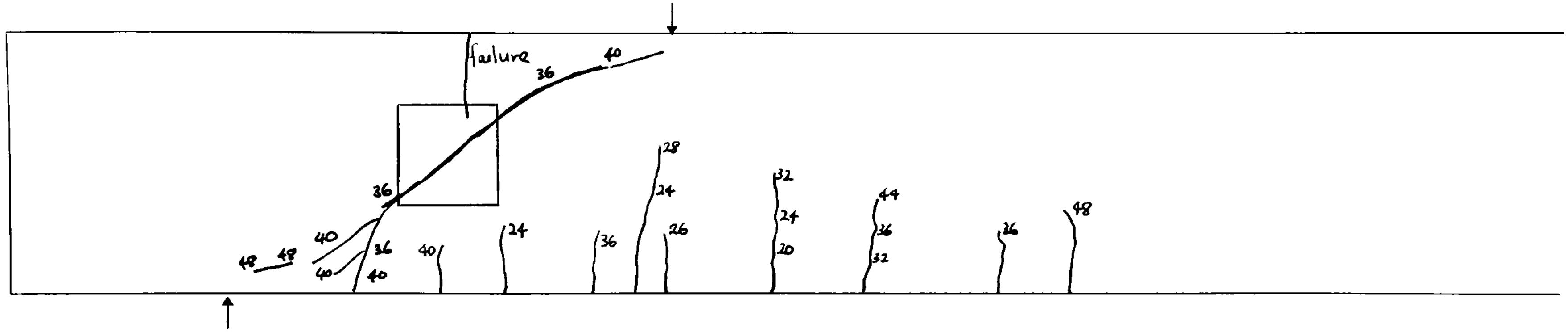


2B-1(a)

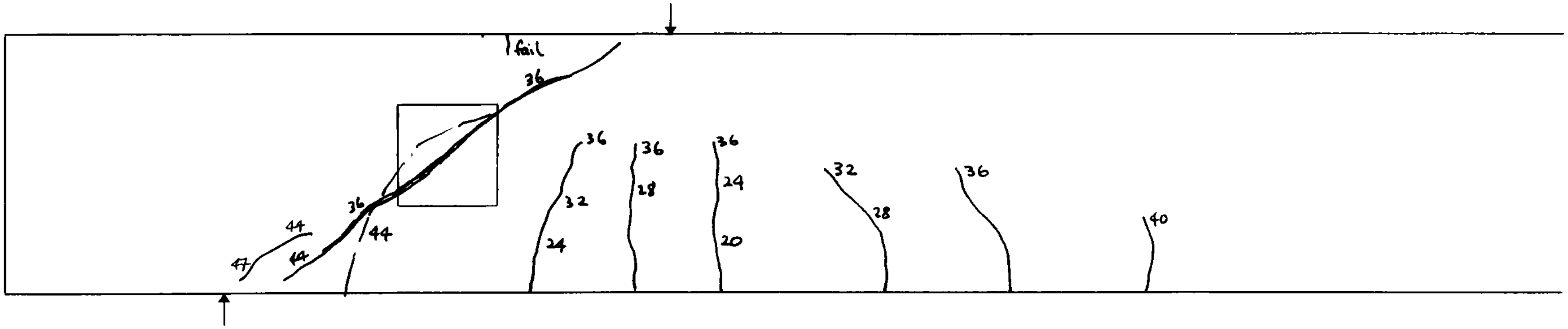


2B-1(b)

FIGURE 4.3(d)

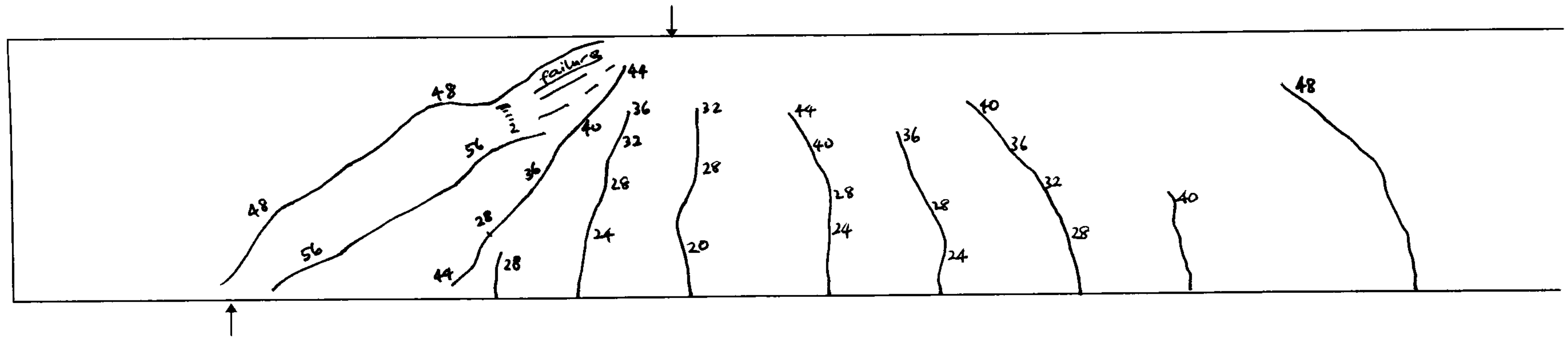


2B-2(a)

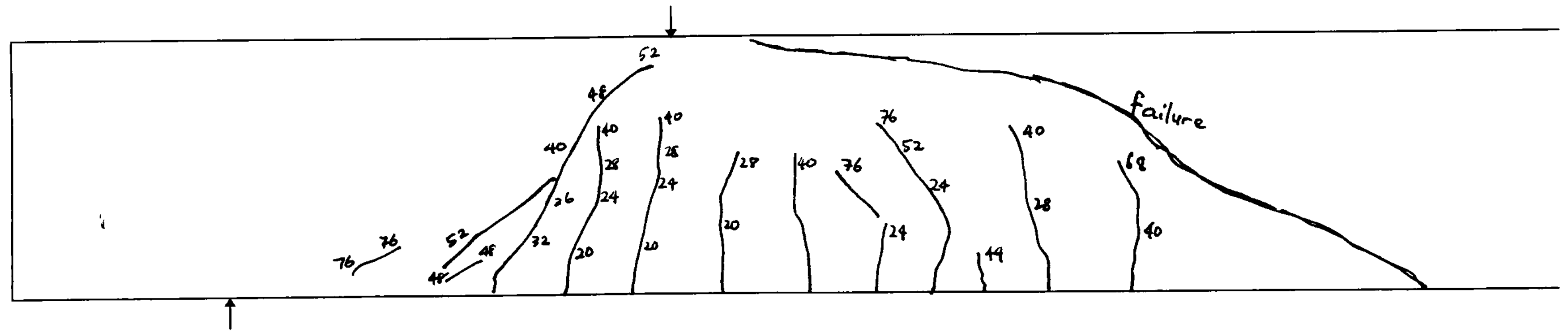


2B-2(b)

FIGURE 4.3(d)- contd

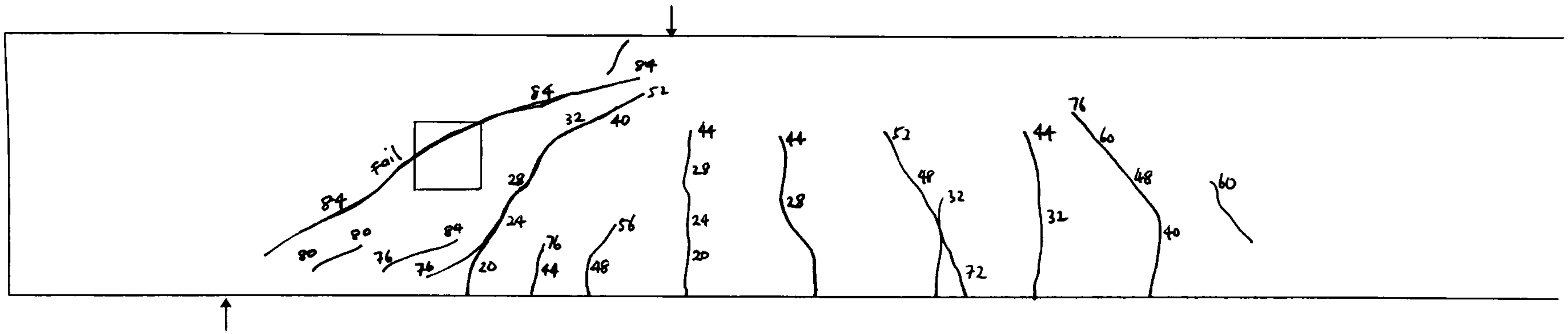


2B-3(a)

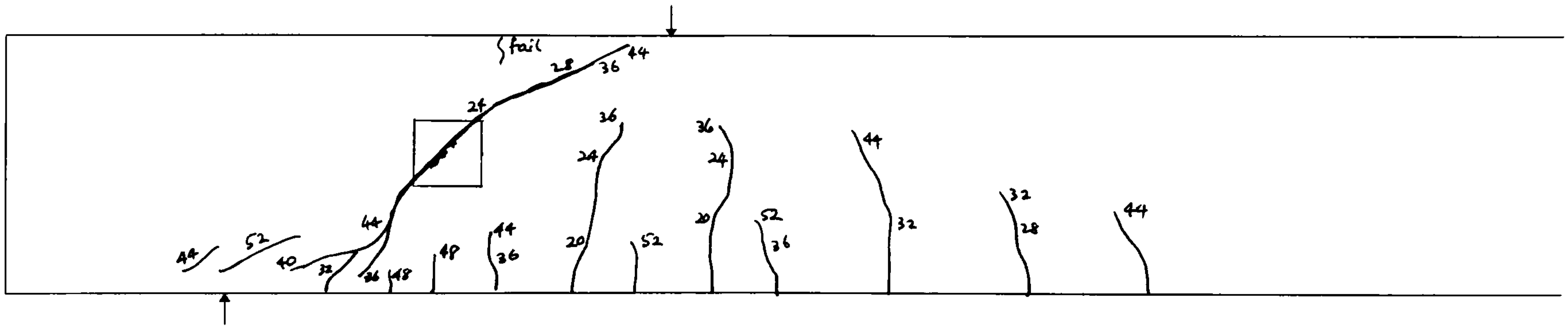


2B-3(b)

FIGURE 4.3(d)- contd

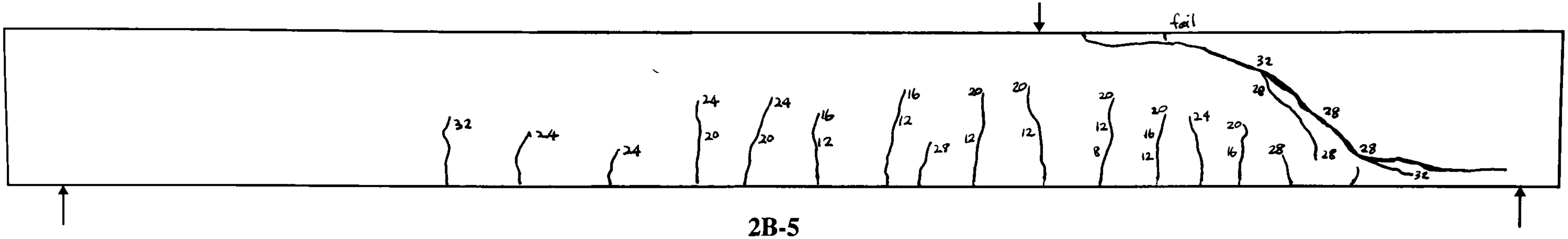


2B-4(a)

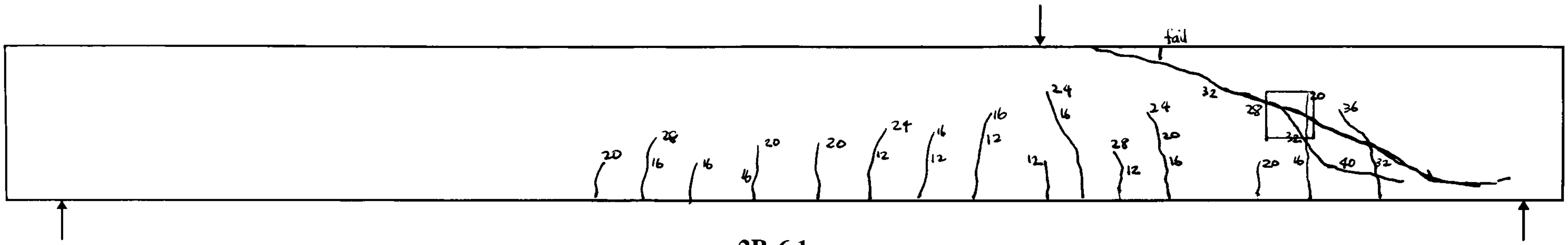


2B-4(b)

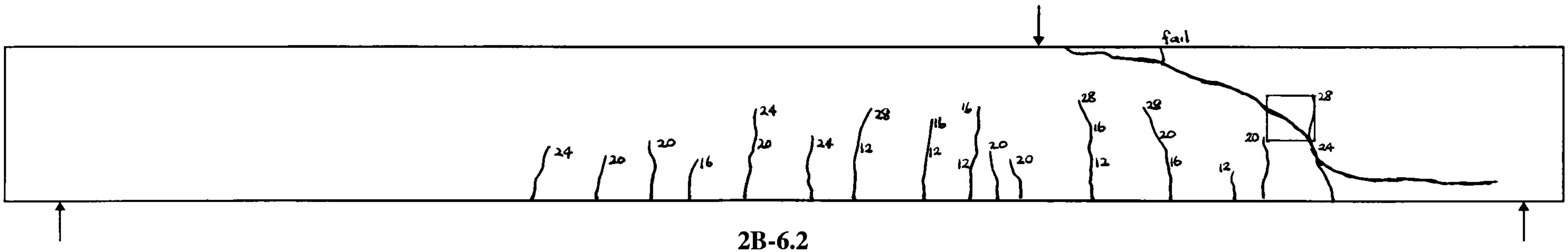
FIGURE 4.3(d)- contd



2B-5

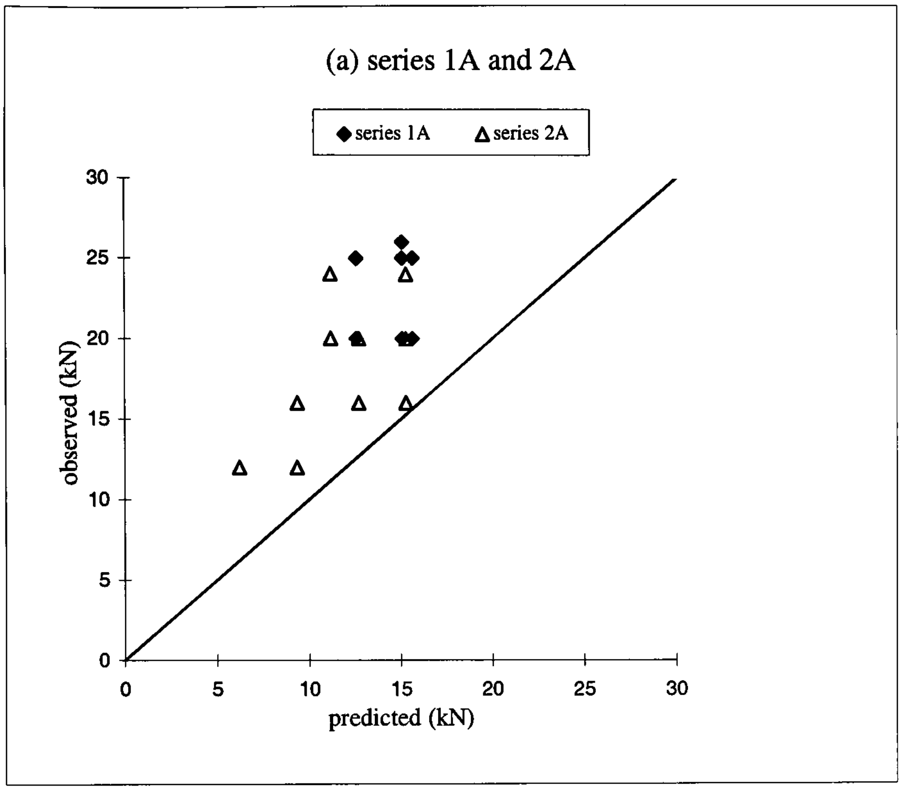


2B-6.1



2B-6.2

FIGURE 4.3(d)- contd



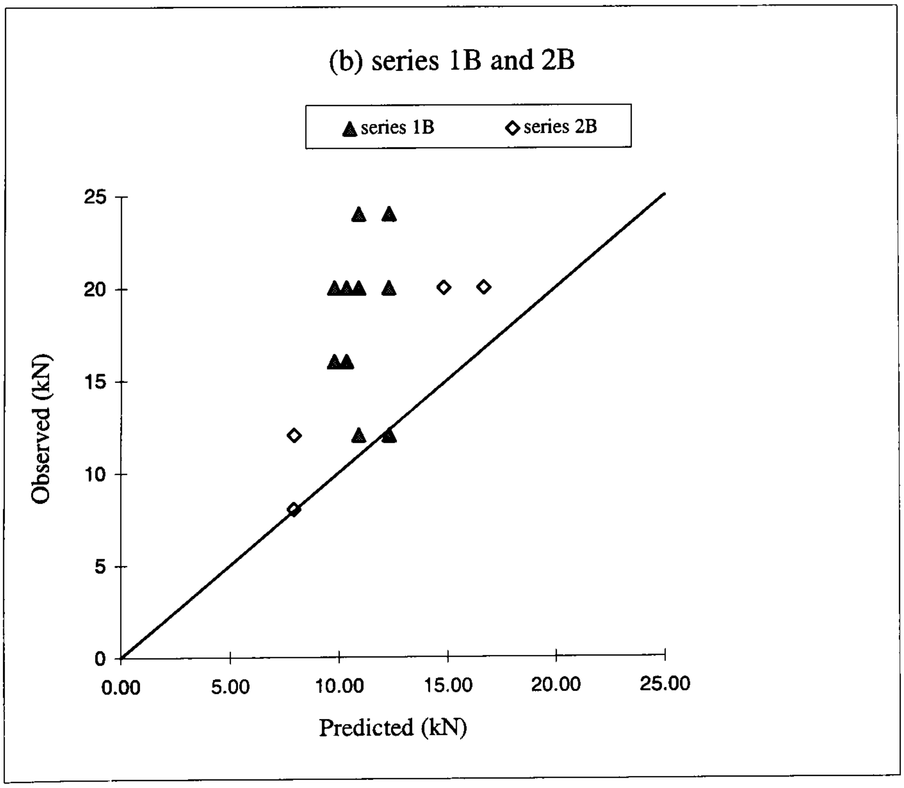


FIGURE 4.4 Flexural cracking load, Observed vs Predicted

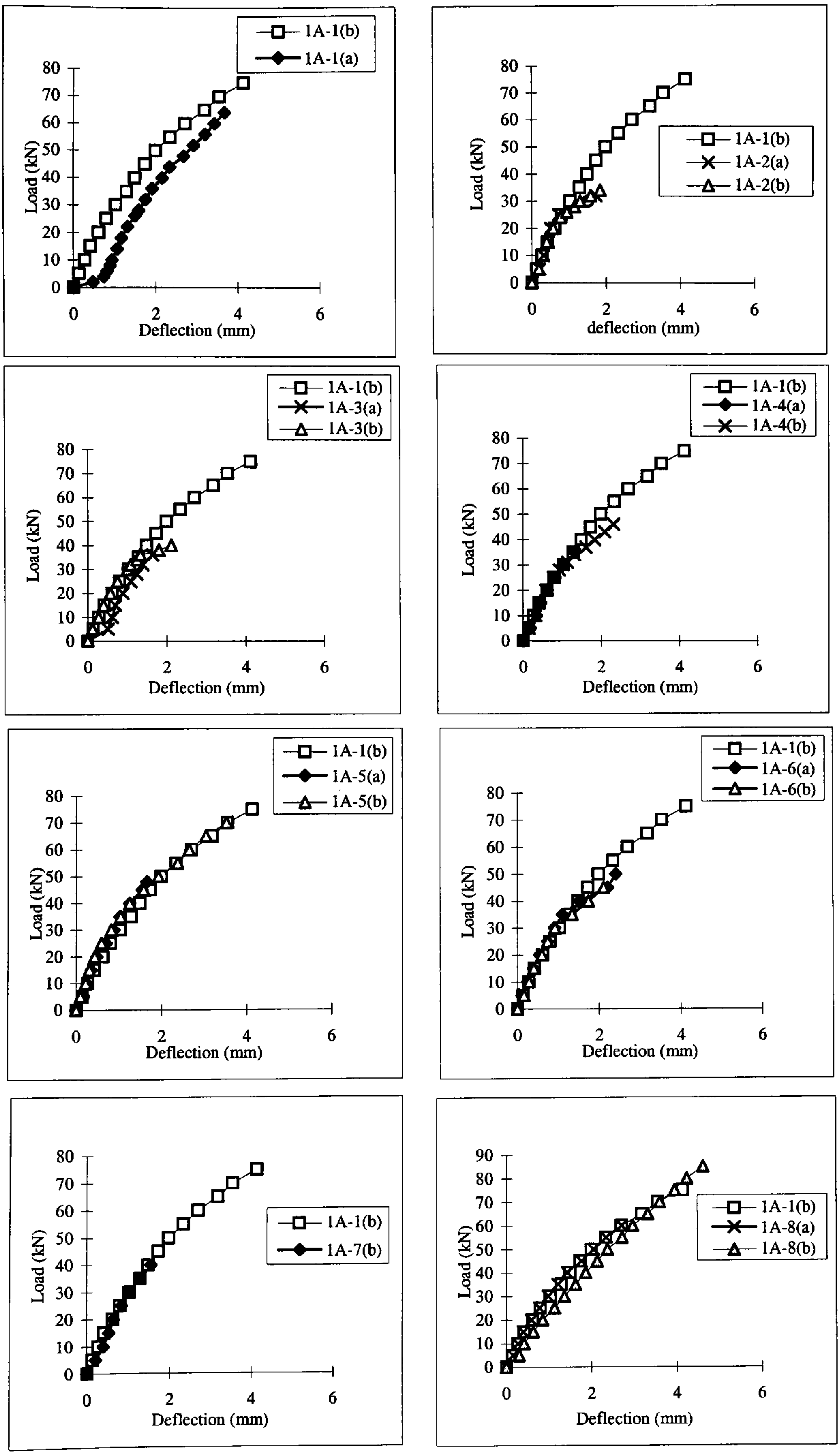


FIGURE 4.5(a) Load-Deflection curves (Series 1A)

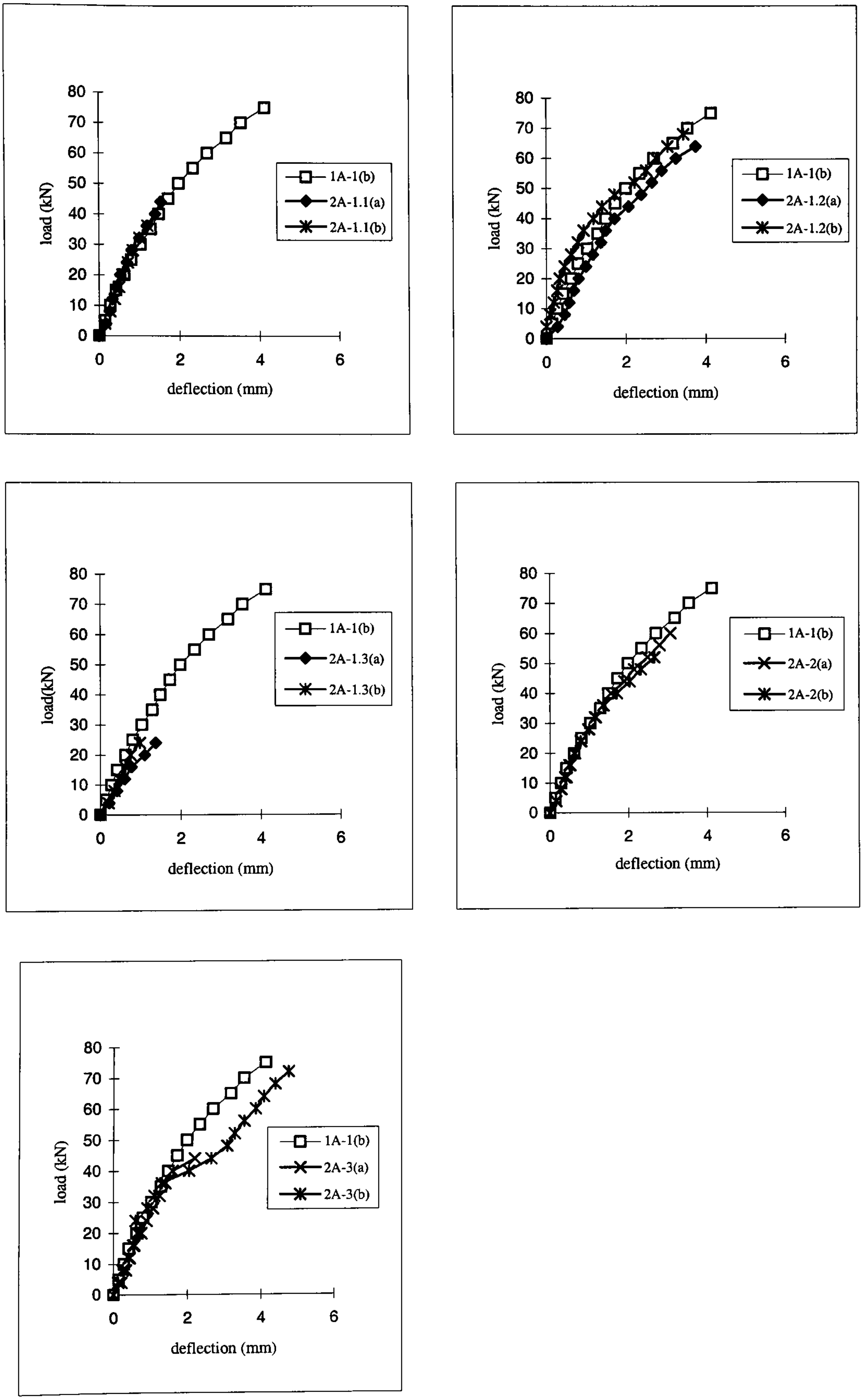


FIGURE 4.5(b) Load-Deflection curves (Series 2A)

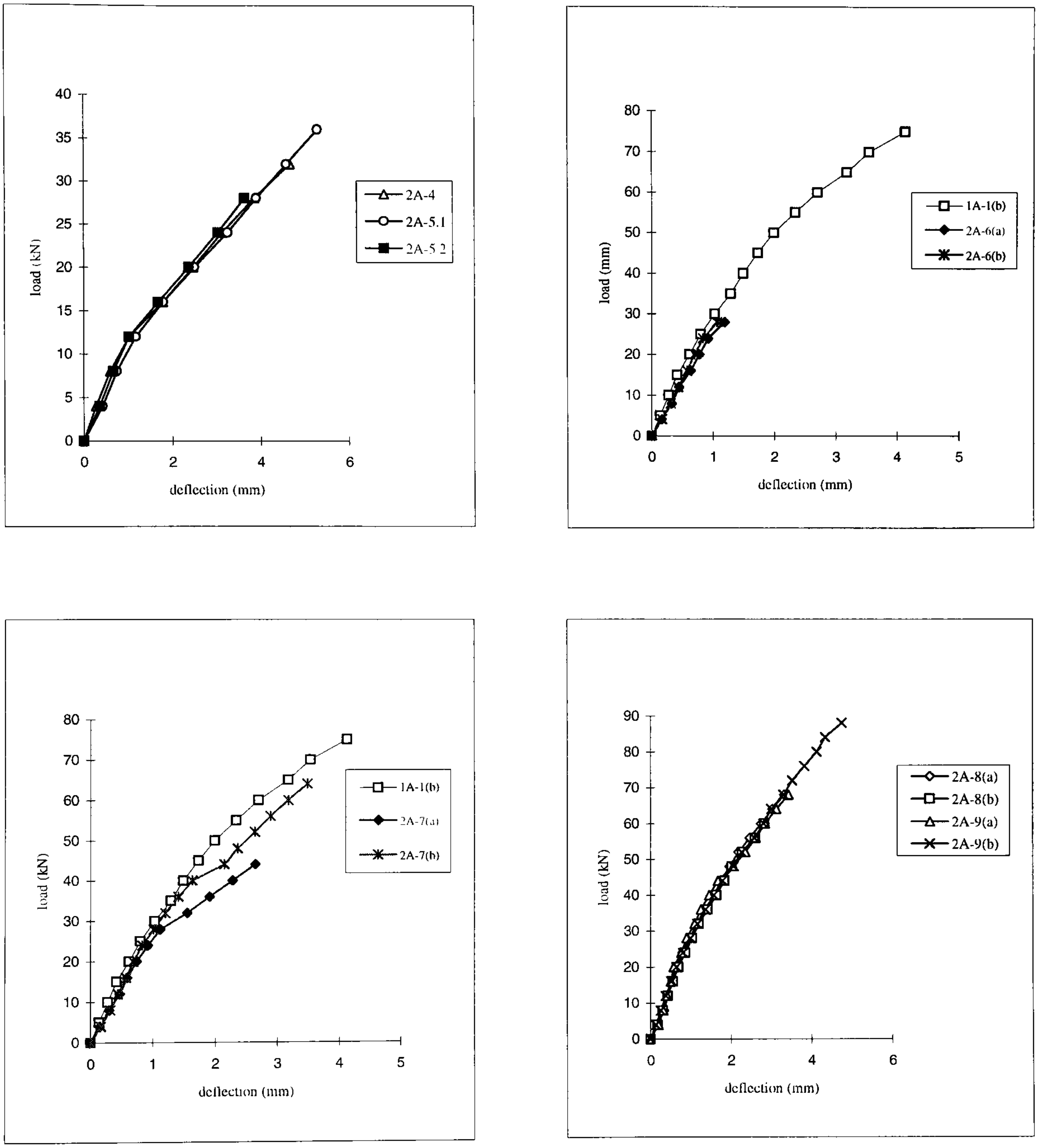


FIGURE 4.5(b) Load-Deflection curves (Series 2A-contd)

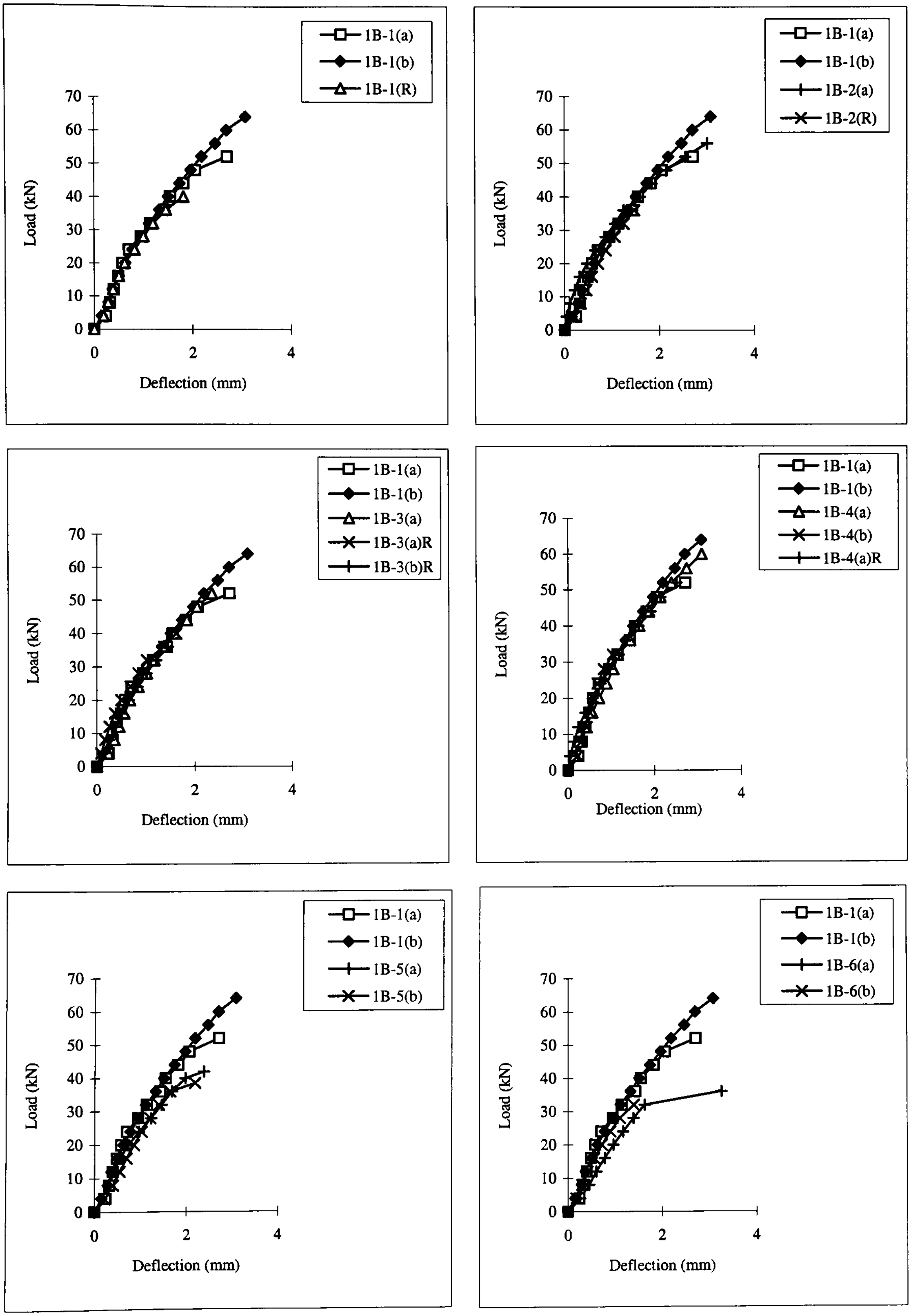


FIGURE 4.5(c) Load-Deflection curves (Series 1B)

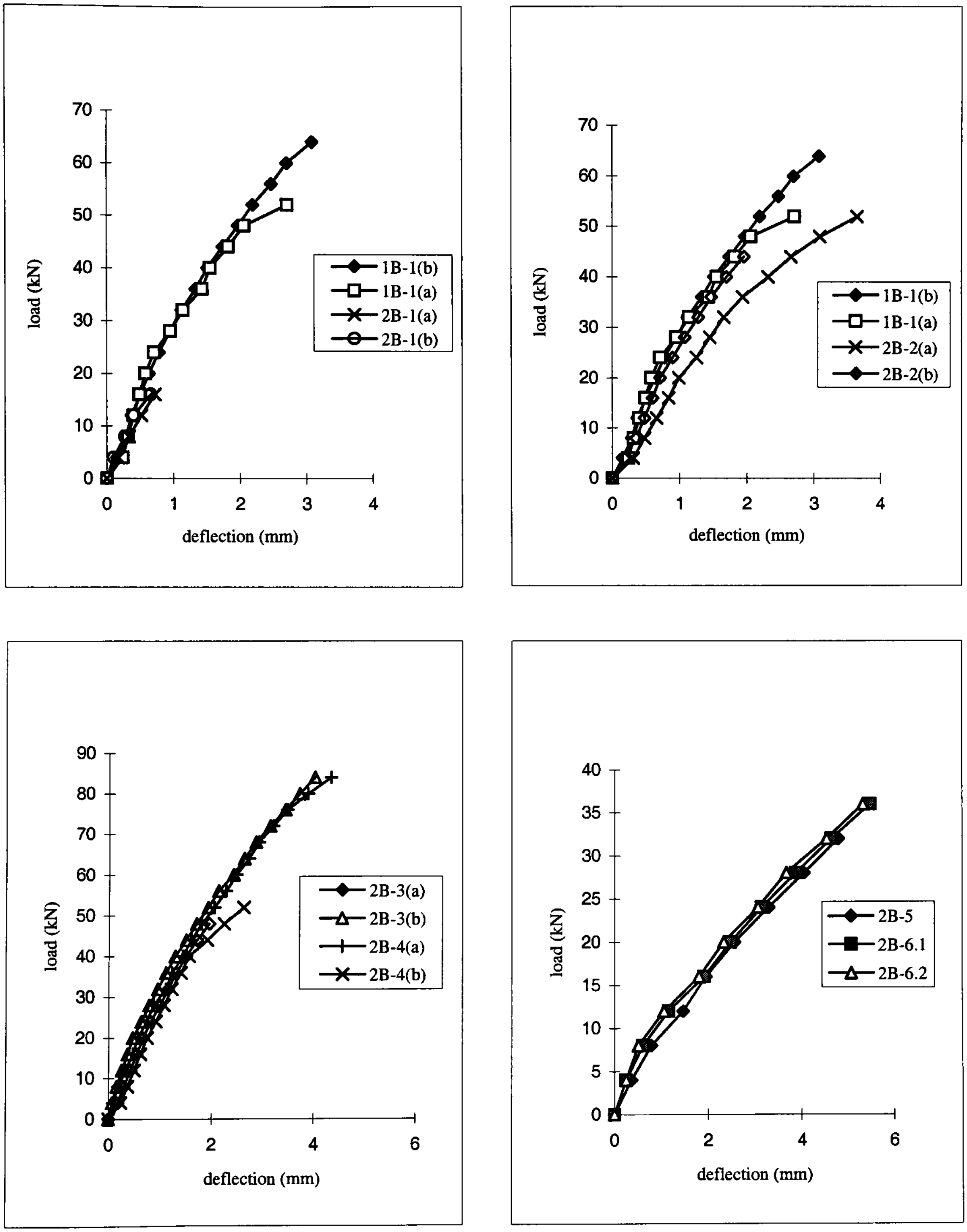


FIGURE 4.5 (d) Load-Deflection curves (Series 2B)

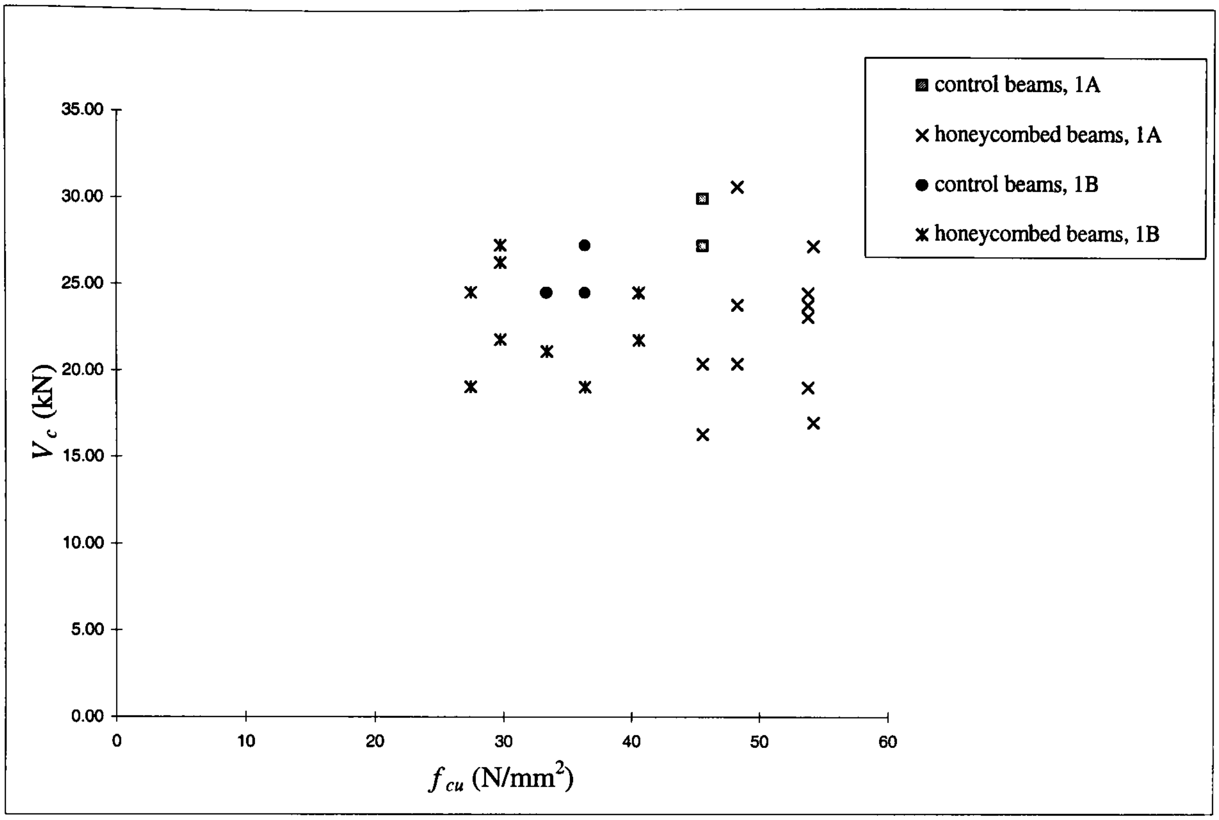


FIGURE 4.6(a) V_c versus normal concrete strength, f_{cu} (Series 1A and 1B)

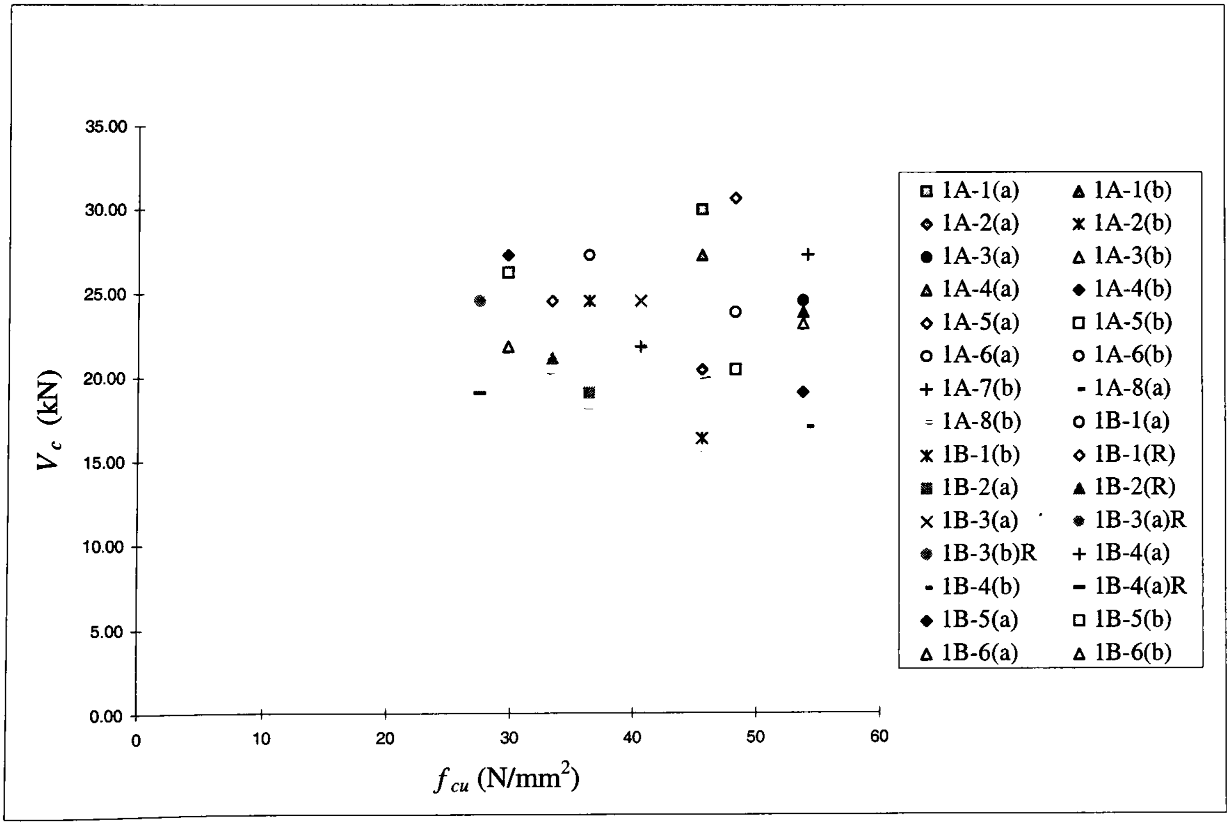


FIGURE 4.6(b) V_c versus normal concrete strength, f_{cu} (Series 1A and 1B)

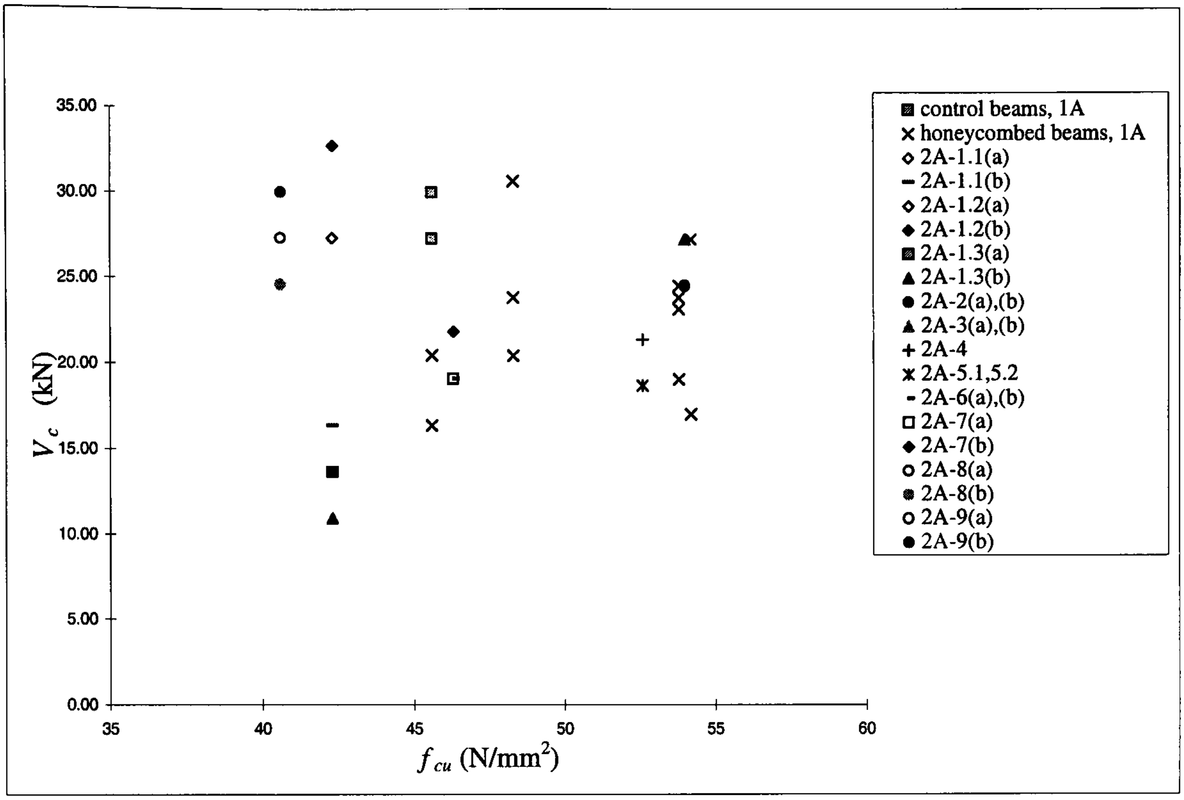


FIGURE 4.7(a) V_c versus normal concrete strength, f_{cu} (Series 1A and 2A)

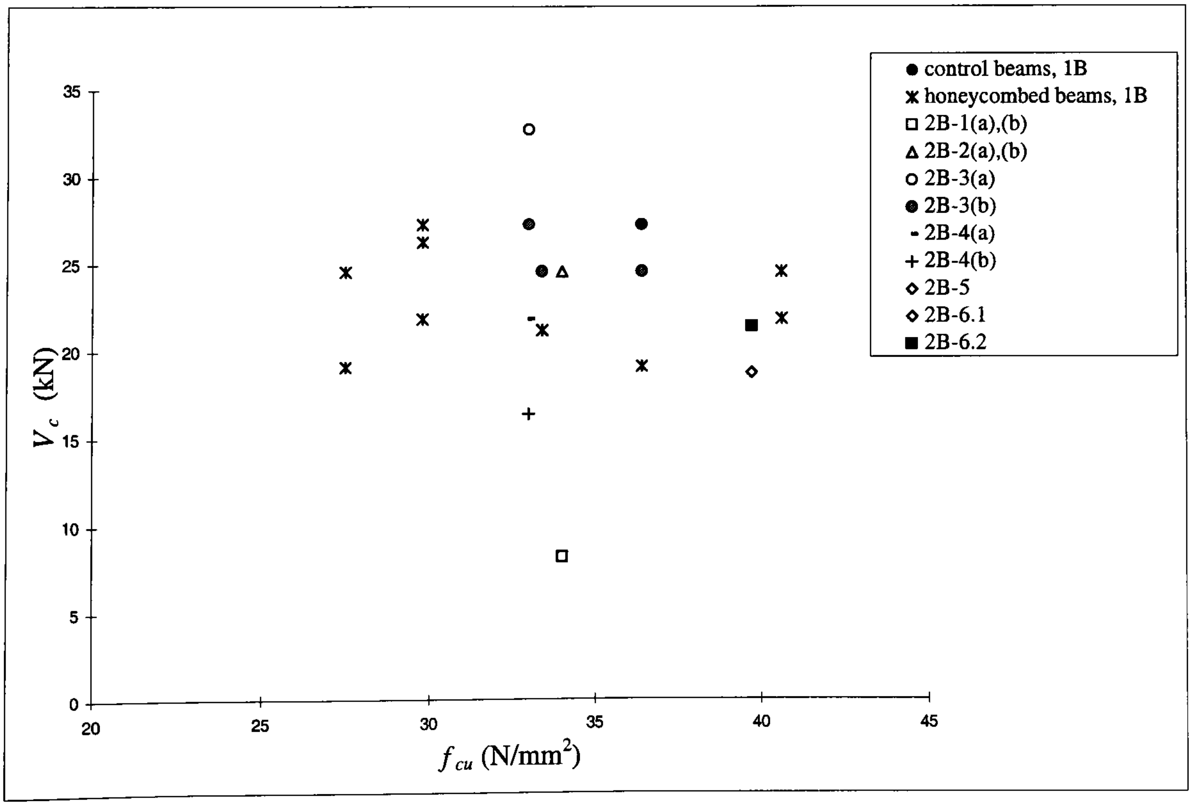


FIGURE 4.7(b) V_c versus normal concrete strength, f_{cu} (Series 1B and 2B)

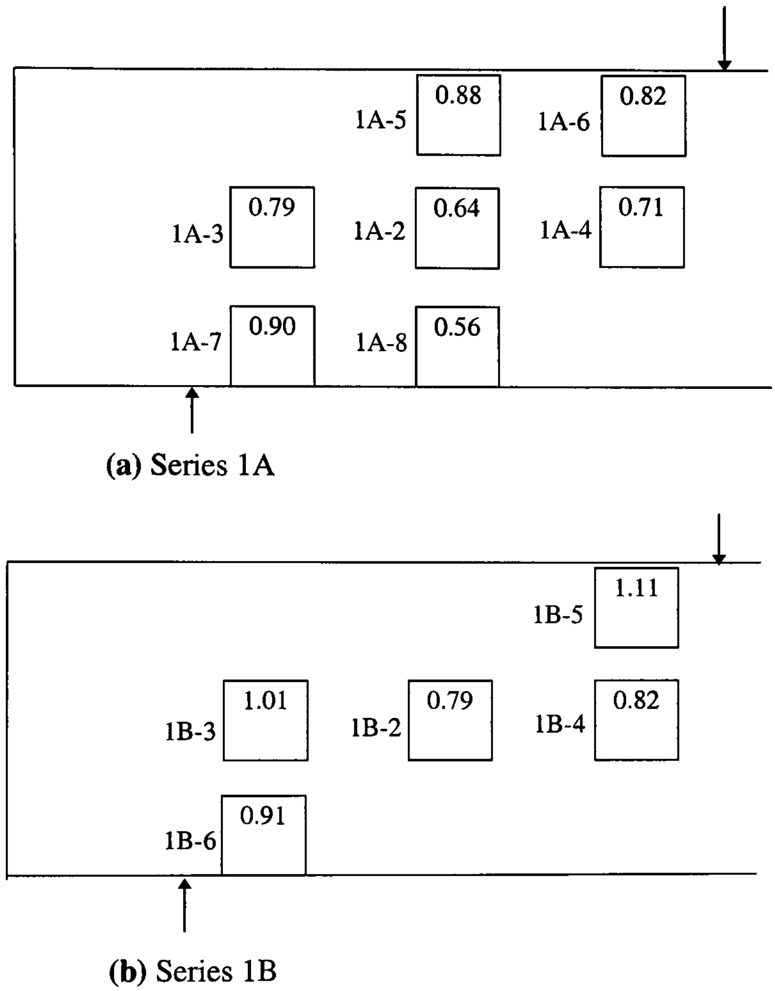
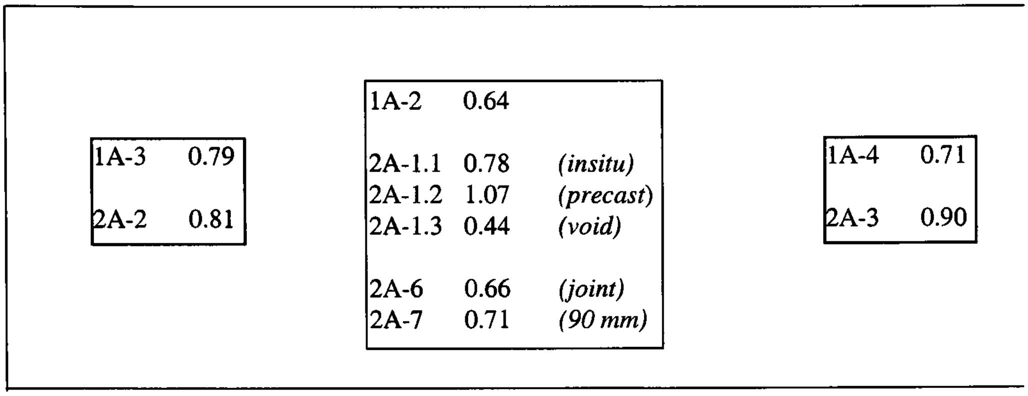
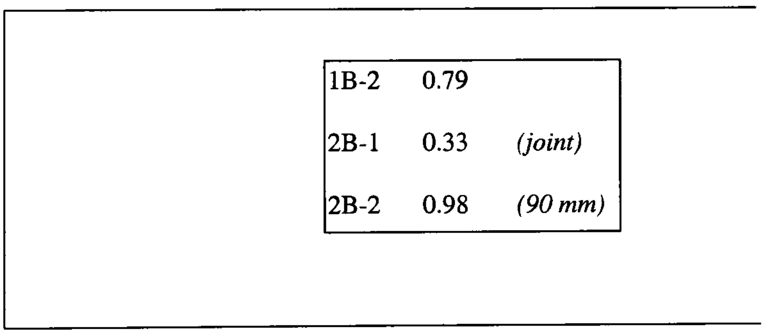


FIGURE 4.8 The ratio of diagonal cracking load, honeycombed beam to control beam



(a) Series 1A/2A



(b) Series 1B/2B

FIGURE 4.9 The ratio of diagonal cracking load, honeycombed beam to control beam

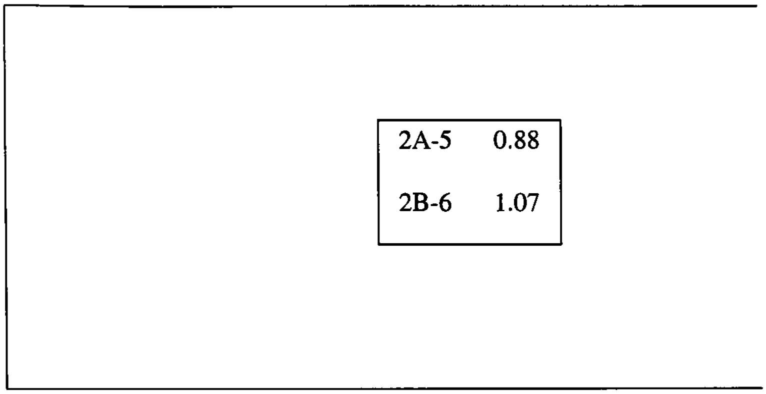


FIGURE 4.10(a) The ratio of diagonal cracking load, honeycombed beam to control beam, with ***a/d*= 3.5** (Series 2A and 2B)

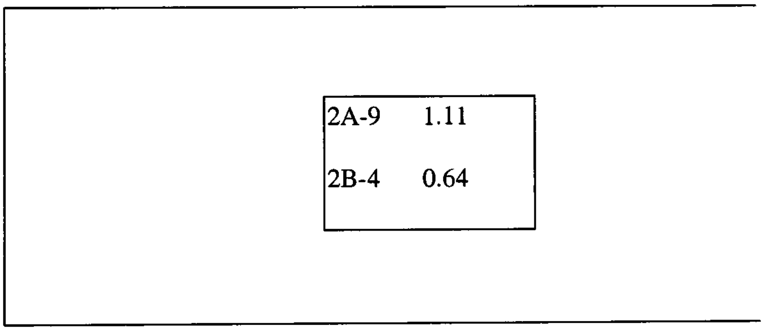


FIGURE 4.10(b) The ratio of diagonal cracking load, honeycombed beam to control beam, beam with shear reinforcement (Series 2A and 2B)

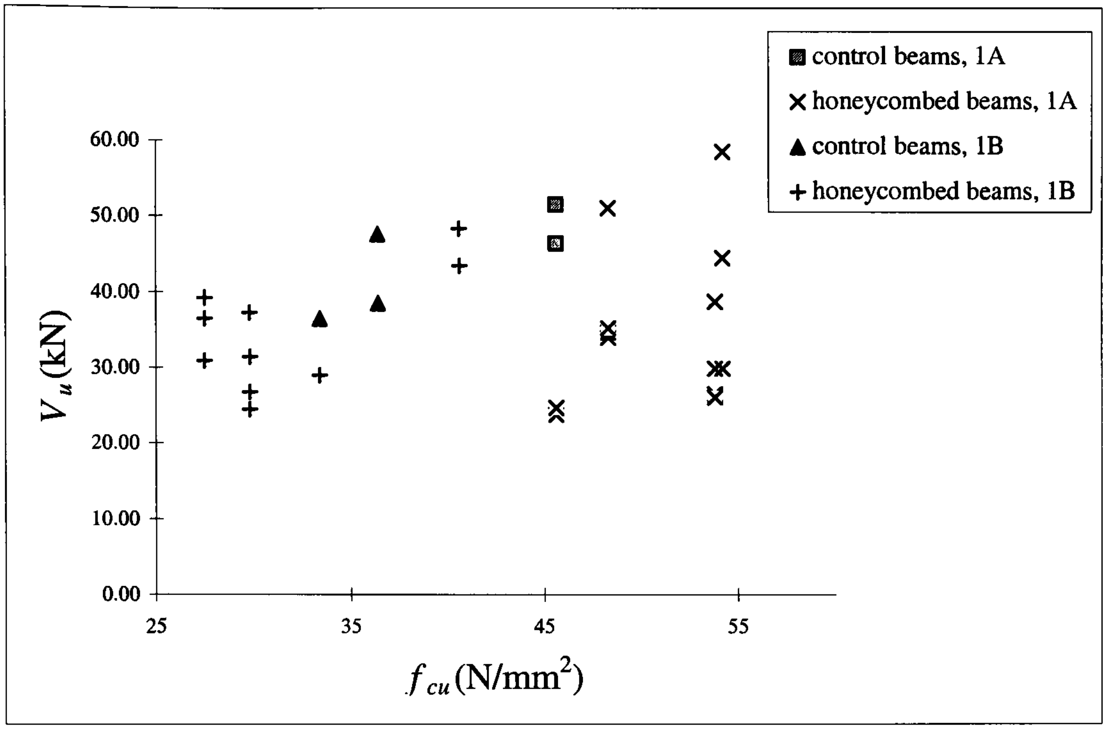


FIGURE 4.11(a) The plots of ultimate shear versus the compressive strength of normal concrete (Series 1A and 1B)

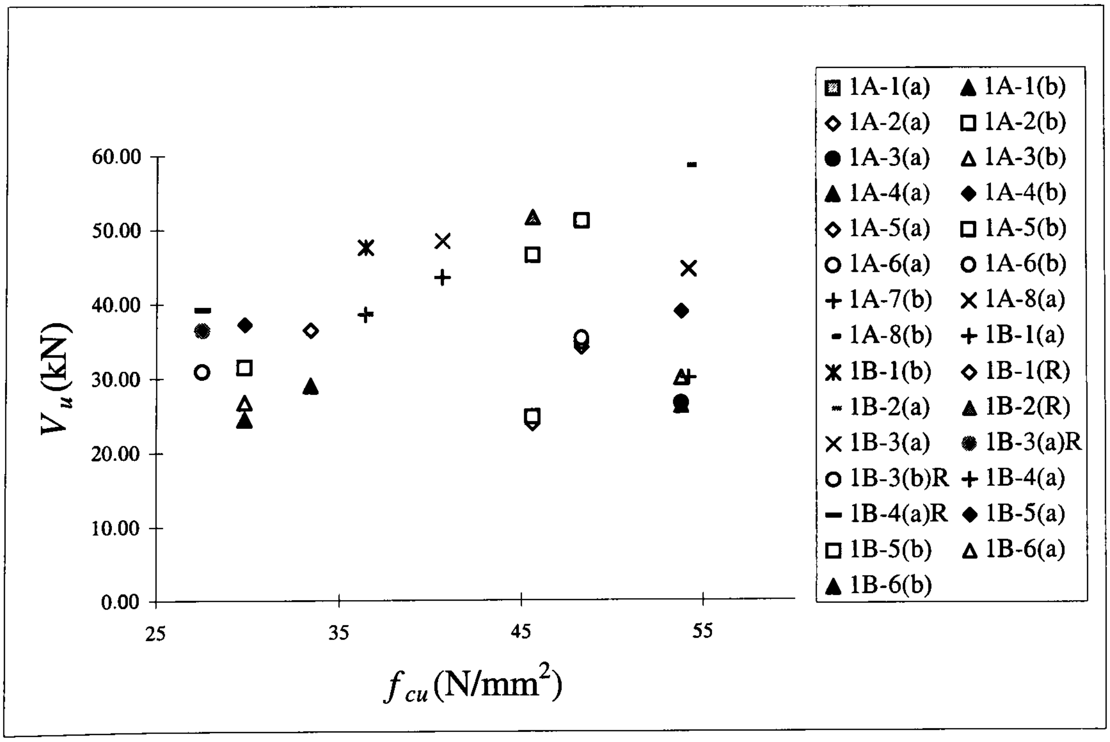


FIGURE 4.11(b) The plots of ultimate shear versus the compressive strength of normal concrete (Series 1A and 1B)

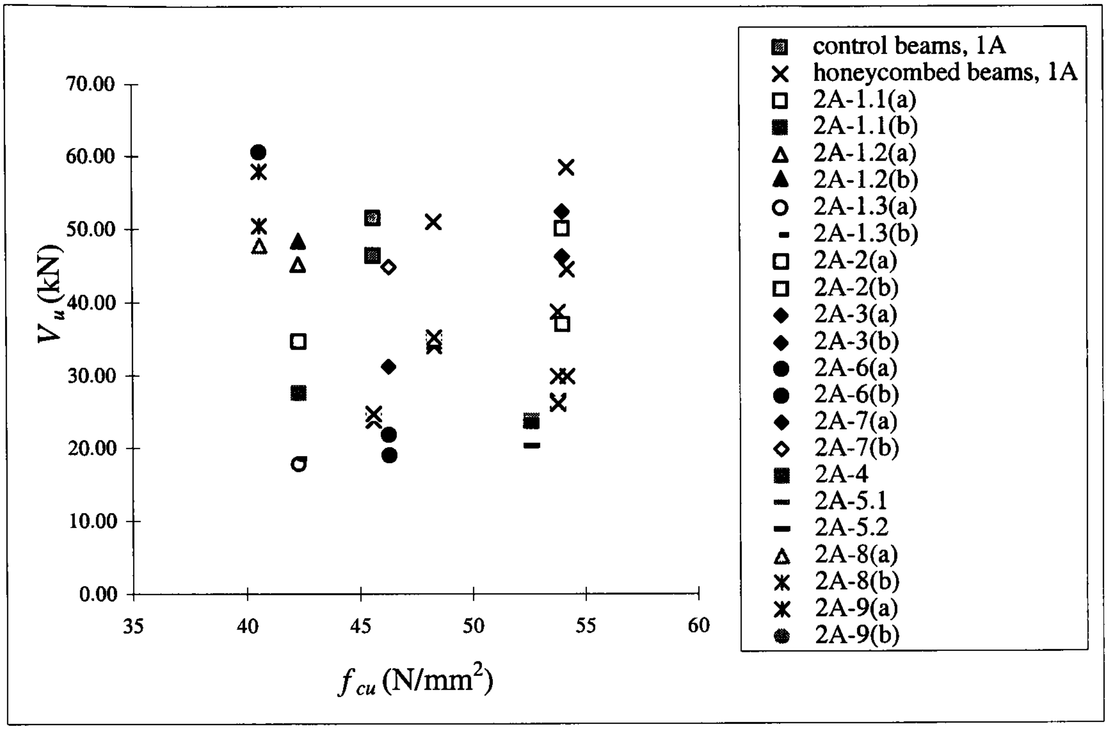


FIGURE 4.12(a) The plots of ultimate shear versus the strength normal concrete (Series 1A and 2A)

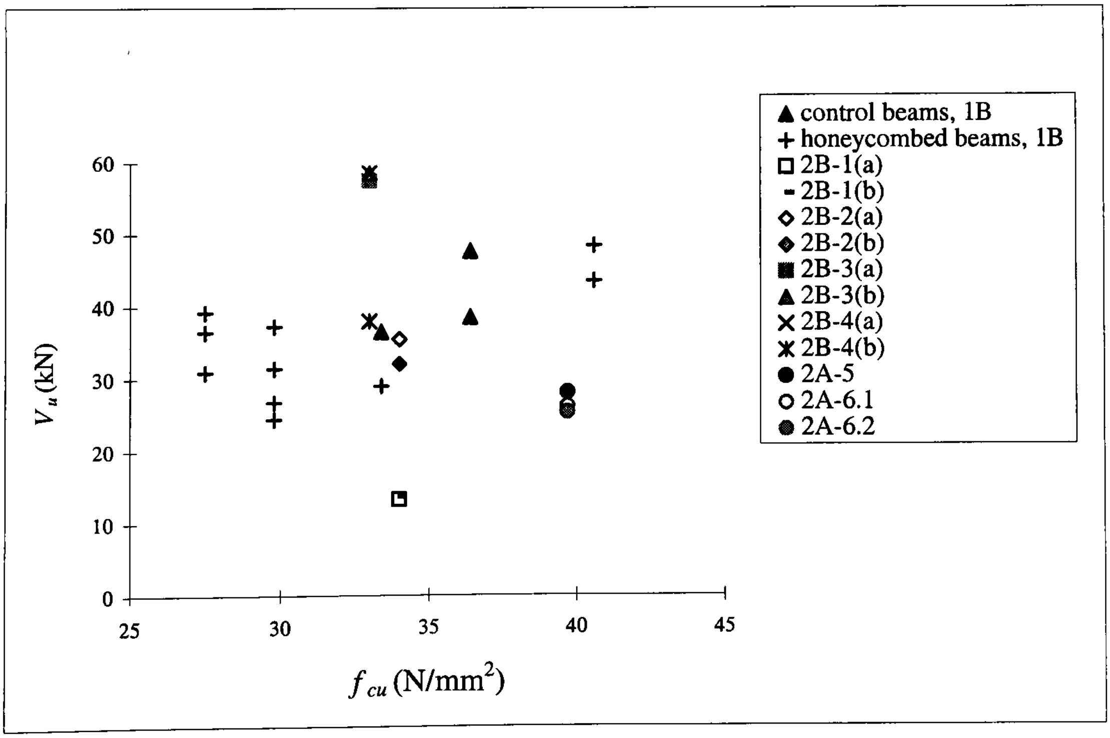


FIGURE 4.12(b) The plots of ultimate shear versus the strength normal concrete (Series 1B and 2B)

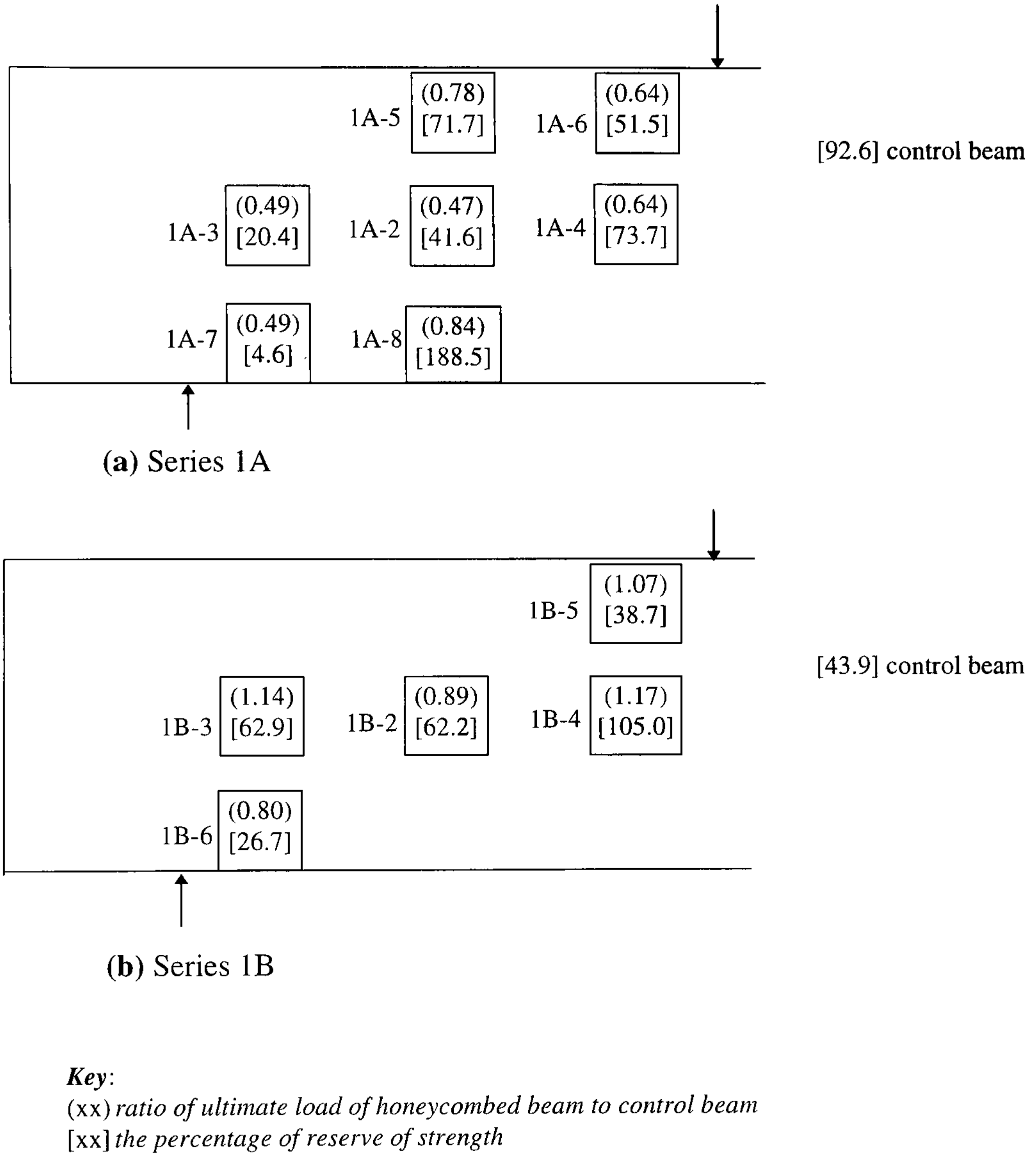


FIGURE 4.13 The ratio of ultimate load, honeycombed to control beam; and the percentage of reserve of strength

1A-1 *(control)*
[87.7]

1A-3	(0.49) [17.1]	
2A-2	(0.72) [65.6]	

1A-2	(0.47),[37.6]	
2A-1.1	(0.65),[56.1]	*(insitu)*
2A-1.2	(0.98),[70.9]	*(precast)*
2A-1.3	(0.38),[62.2]	*(void)*
2A-6	(0.39),[10.4]	*(joint)*
2A-7	(0.73),[92.6)	*(90 mm)*

1A-4	(0.64) [68.7]
2A-3	(0.81) [68.6]

(a) Series 1A/2A

1B-1 *(control)*
[47.9]

1B-2	(0.90),[66.4]	
2B-1	(0.37),[66.5]	*(joint)*
2B-2	(0.92),[39.2]	*(90 mm)*

(b) Series 1B/2B

***key*:**
(xx) *ratio of ultimate load of honeycombed to control beam*
[xx] *the percentage of reserve of strength*

FIGURE 4.14 The ratio of ultimate load, honeycombed to control beam, and the percentage of reserve of strength

2A-4	[11.3] *(control)*
2A-5	(0.95), [20.2]
2B-5	[50.0] *(control)*
2B-6	(0.92), [28.3]

(a) Beam with a/d= 3.5

2A-8	[84.5] *(control)*
2A-9	(1.06), [84.2]
2B-3	[91.6] *(control)*
2B-4	(0.84), [152.9]

(b) Beams with shear reinforcement

key:
(xx) *ratio of ultimate load of honeycombed to control beam*
[xx] *the percentage of reserve of strength*

FIGURE 4.15 The ratio of ultimate load, honeycombed to control beam, and the percentage of reserve of strength beams with a/d=3.5 and beams with shear reinforcement (Series 2A and 2B)

CHAPTER 5

THE THEORY OF PLASTICITY FOR HONEYCOMBED BEAM AND BEAM WITH CONSTRUCTION JOINT

5.1 INTRODUCTION

The upper bound plasticity theory can be utilised and provides a rational analytical tool to assess the shear capacity of honeycombed beams. For many years it has been successfully applied in the analysis of the shear capacity of reinforced concrete structures. Although assumptions made in this theory with regard to the behaviour of materials, especially concrete, seem very far from what is normally observed in concrete and although it is necessary to adopt an effectiveness factor determined from experiments, the theory, however, is in general increasingly accepted by researchers. In recent years the development of the plasticity theory to deal with the assessment of shear in concrete structural members has been regarded as at par with the more rational and realistic flexural theory. Combined with the advanced and more rational flexural theory it brings some uniformity of realism in the assessment of concrete structures(60).

This chapter describes the extension of the existing upper-bound plasticity theory for predicting the shear capacity of an ordinary concrete beam in order that it can be used as an assessment tool in predicting the shear capacity of honeycombed beams. The work is divided into two: first the extension of the existing theory of plasticity to predict the shear capacity of a beam with a honeycombed zone; and second, the extension of plastic analysis to predict the shear capacity of a concrete beam with an inclined construction joint. With regard to the latter, analytical work is carried out in order to study the effect of a construction joint to the shear capacity of a beam. The influence of various parameters such as the angle of inclination of the joint, the strength of the joint, the amount of longitudinal reinforcement and the shear span on the shear capacity are included in the study. The results and discussion are presented in **Section 5.4.1**.

5.2 THE BACKGROUND OF THE THEORY

A complete theoretical background of the plasticity theory can be found in the literature and some of them are listed in the references (43),(44),(45),(60),(61). As already mentioned in **Section 2.3.3.1** and **Section 2.4.2.3** of **Chapter 2**, a number of assumptions are required with regard to the behaviour of the materials, especially concrete, in formulating this theory.

Essentially the following are the assumptions that are required(44):

(a) The concrete is rigid, perfectly plastic with the modified Coulomb failure criterion as yield condition and the associated flow rule (normality condition). The tensile strength is neglected. The compressive strength is the strength obtained from a concrete cylinder.

(b) The reinforcement steel is assumed as rigid, perfectly plastic and unable to resist lateral forces. It carries tension forces only and any dowel effects are ignored.

(c) Any elastic deformations and work-hardening effect are neglected and unlimited ductility is assumed.

In the above assumptions, concrete is assumed to behave according to the modified Coulomb material. According to that criterion, failure in concrete subjected to loads can take place in two forms: sliding failure and separation failure. A pure sliding failure occurs when the shear stress acting on a plane exceeds the sliding resistance of the concrete and the motion is parallel to the rupture surface. However, due to the roughness of the concrete sliding surface, dilatancy occurs and there is a normal component in addition to the parallel component. A separation failure occurs when tensile stress exceeds the separation resistance of the concrete and the motion is perpendicular to the rupture surface. These failures can take place in concrete in combination.

With regard to concrete ductility, as already mentioned in **Chapter 2,** and as shown in **Figure 2.8**, concrete has a limited ductility and as a result of that in order for the theory to yield a good prediction of shear capacity of beams, the compressive strength of concrete obtained from a cylinder needs to be multiplied by a reduction factor called an effectiveness factor. This effectiveness factor needs to be determined from test data.

5.2.1 Upper-bound Plasticity Approach

In plastic theory two solutions are possible, lower-bound and upper-bound approaches. In the following development of the theoretical work, the upper-bound approach will be used as it is more appropriate in dealing with the asssessment of an existing structure. The upper-bound plasticity theorem describes that 'if various geometrically possible strain fields or failure mechanisms are considered, the work equation can be used to find values of the load-carrying capacity that are greater than or equal to the true one'(44).

The theorem thus implies that the prediction of the load capacity of a structural member can be unsafe. However it has been shown theoretically that, using both lower and upper-bound approaches in predicting a shear capacity of reinforced concrete beam leads to the same unique solution. Comparisons made between theoretical and experimental results, certainly with the inclusion of the effectiveness factor, also showed that a good correlation and safe prediction can be achieved. Thus it should be emphasised that the use of the upper-bound solution, although theoretically unsafe, tends to be safe in practice.

5.2.2 The Equation of Internal Work

In the upper-bound plasticity theory the solution is based on the external work done by the load and the internal work done by the displacement of the plastic mechanism. The following derivation of the internal plastic work equations is based upon that of Nielsen(44) and Jensen(61).

Consider a volume bounded by two parallel planes at a distance δ apart. The diagram is shown in **Figure 5.1**. Assume that there is a plane, homogeneous strain field occurring in the narrow volume, and that parts I and II outside the volume move as rigid bodies in the (*n-t*) plane. The strains, expressed in terms of the displacement u and the angle α which the displacement vector forms with the *t*-axis, are:

$$\varepsilon_n = \frac{u \sin\alpha}{\delta}, \qquad \varepsilon_t = 0, \qquad \gamma_{nt} = \frac{u \cos\alpha}{\delta}$$

It has been shown that, the internal work done, W_I , per unit length, in the line of discontinuity or normally known as yield line measured in the direction of the *t*-axis is,

$$W_I = ub\left[\frac{(1-\sin\alpha)}{2} f_c + \frac{\sin\alpha - \sin\varphi}{1-\sin\varphi} f_t\right] \tag{5.1}$$

where,

f_c	=	compressive strength of concrete cylinder
u	=	the displacement
b	=	the dimension of the body measured perpendicular to the (n,t)- plane
φ	=	angle of internal friction
f_t	=	tensile strength of concrete

For the compressive strength of concrete, as already mentioned, concrete is not truly plastic, thus, an effectiveness factor needs to be applied to the concrete strength, f_c. It has been shown(44),(45),(61) that equation (5.1), in its application to evaluate the shear strength in beams and for a joint, can be further simplified by neglecting the tensile strength of concrete, thus $f_t = 0$. This will lead to conservative results for beam. In joints, it is also conservative to ignore the tensile strength of concrete(62).

The internal plastic work equation now can be expressed as:

$$W_I = ub\left[\frac{(1-\sin\alpha)}{2}f_c\right] \tag{5.2}$$

In all the following works, equation (5.2) which is a simpler version of the internal plastic work equation will be used.

5.3 PLASTICITY THEORY FOR SHEAR IN HONEYCOMBED CONCRETE BEAMS

The theory described below is an extension from Nielsen's plasticity theory for shear in reinforced concrete beams(43),(44),(45).

5.3.1 Normal Concrete Beams Without Shear Reinforcement

Consider a beam without a honeycombed zone as shown in **Figure 5.2**. The width and the height of the beam are b and h respectively. The beam is subjected to a point load, P and the reaction at the support of the section considered is V. The shear span is a. A plastic failure mechanism is assumed to form as a straight line, at an angle β to the horizontal axis, and the displacement, u takes place at angle α to the yield line. For the mechanism shown, the shear capacity, V, in the shear span considered is given by the following work equation:

$$V.u\,sin(\alpha+\beta)=u\left[\frac{(1-sin\alpha)}{2}f_c\right]\frac{bh}{sin\beta}-A_sf_y.u\,cos(\alpha+\beta) \qquad (5.3)$$

where

f_c	=	cylinder compressive strength of concrete
b, h	=	width and height of the beam
A_s	=	the area of the longitudinal reinforcement
f_y	=	the yield strength of longitudinal reinforcement

Introducing the effectiveness factor, ν, into the equation, and eliminating u, the equation becomes,

$$V\,sin(\alpha+\beta)=\frac{1}{2}\nu f_c(1-sin\alpha)\frac{bh}{sin\beta}-A_sf_y\,cos(\alpha+\beta) \qquad (5.4)$$

Following Nielsen et al(43), and defining the average shear stress, τ and the degree of longitudinal reinforcement, Φ as follow,

$$\tau = \frac{V}{bh}$$

$$\Phi = \frac{A_s f_y}{bhf_c}$$

equation (5.4) can now be written in the following form,

$$\frac{\tau}{f_c} = \frac{\nu(1 - sin\alpha) - 2\Phi\, sin\beta . cos(\alpha + \beta)}{2\, sin\beta . sin(\alpha + \beta)} \tag{5.5}$$

Nielsen et al(43) found that the critical plastic yield line, which gives the minimum shear capacity in concrete beams without shear reinforcement, extends from the point of loading to the point of support. Thus, β in equation (5.5) is no longer a variable, and $cot\,\beta$ can be replaced by *a/h.*

Minimising the above equation with respect to α, it can be shown that, the lowest upper-bound value of the shear capacity of concrete beam without shear reinforcement is,

$$\frac{\tau}{f_c} = \frac{\nu}{2}\left(\sqrt{cot^2\beta + \frac{4\Phi(\nu - \Phi)}{\nu^2}} - cot\beta\right) \tag{5.6}$$

valid for, $\Phi \leq \frac{\nu}{2}$

and,

$$\frac{\tau}{f_c} = \frac{\nu}{2}\left(\sqrt{cot^2\beta + 1} - cot\beta\right) \tag{5.7}$$

valid for, $\Phi \geq \frac{\nu}{2}$

The angle α can be obtained by the following equation:

$$cos(\alpha + \beta) = -\frac{1}{\nu}(\nu - 2\Phi).sin\beta \tag{5.8}$$

5.3.2 Beam Without Shear Reinforcement and With a Honeycombed Zone

The above solution cannot be applied to a honeycombed beam without first ensuring that the critical yield line in the honeycombed beam studied also extends from the point of loading to the point of support. With a honeycombed zone present, it is premature to assume that the critical yield line remains the same as in the beam without a honeycombed zone, without examining it analytically. The presence of a honeycombed zone may results in a critical yield line, which gives the minimum shear capacity, passing through the honeycombed zone but it may not necessarily extend from the point of loading to the point of support. This needs to be checked for all cases of honeycombed beams.

Equation (5.4) can also be written in the following form, replacing *h/sin* β with l_m , the length of the yield line,

$$V.sin(\alpha + \beta) = \frac{1}{2}.\nu.b(1 - sin\alpha)f_c.l_m - A_sf_y.cos(\alpha + \beta) \tag{5.9}$$

Consider now a concrete beam without shear reinforcement but with a honeycombed zone present in the shear zone as shown in **Figure 5.3**. Assume that the failure mechanism is now passing through the area of the honeycombed zone for a length denoted as l_h. l_c is the length of the mechanism which passes through the normal concrete. In the failure mechanism, now there are two types of concrete strength that need to be considered. If the effectiveness factor of the normal concrete and the honeycombed concrete each are denoted as ν_c and ν_h, and the concrete cylinder strengths are denoted as f_c and f_{ch} respectively, from equation (5.9), V can be expressed as follows:

$$V.sin(\alpha+\beta)=\frac{1}{2}b(1-sin\alpha)(\nu_c f_c.\,l_c+\nu_h.\,f_{ch}.\,l_h)-A_s f_y.\,cos(\alpha+\beta) \tag{5.10}$$

The problem now is to solve the equation for the minimum V. In the equation above, the minimum value of V depends on the angle β, which determines the lengths of l_c and l_h, which then will influence the contribution of normal and honeycombed concretes to the shear capacity of the beam. The other variable is α. An analytical solution cannot be found because of the interdependence of the various parameters in a non-continuous manner.

It is possible to solve the above equation by a numerical approach. However, the solution will be significantly simplified if the α variable can be removed. This can be done by assuming that the angle $(\alpha+\beta)$ is equal to 90^{o}. This assumes that the displacement, u is in the vertical direction, and thus the contribution of the longitudinal reinforcement is ignored.

Equation (5.10) is now reduced to,

$$V=\frac{1}{2}b(1-cos\beta)[\nu_c f_c.\,l_c+\nu_h f_{ch} l_h] \tag{5.11}$$

V is evaluated using a trial value of β. Another parameter that needs to be specified is the distance of the yield line from the support, x, as shown in Figure 5.3. For various values of β and x, the contributions of normal and honeycombed concretes are then evaluated. A FORTRAN program is used in order to solve for the minimum V. This is discussed in **Section 5.3.4**.

It should be emphasised that the assumption regarding the contribution of longitudinal reinforcement is temporarily required to simplify the solution with the FORTRAN programming described in the section below. It will be shown later in **Section 6.3.2.2** of **Chapter 6** that, by using this approach to explore the critical yield line in the honeycombed beams, it was found that for all the honeycombed beams studied, the critical yield line extends from the point of loading to the point of support. Thus, it is similar to Nielsen at. al(43).

As a result, equations (5.6) and (5.7), (discussed in **Section 5.3.1**) can be used in beams with a honeycombed zone and the effect of the longitudinal reinforcement can be taken into account. This will be presented and discussed in **Section 6.3.2.3** of **Chapter 6**.

5.3.3 The Strength of Concrete and The Effectiveness Factor

In equation (5.11) the properties of the normal and honeycombed concretes and the effectiveness factors for both concretes must be determined before V can be obtained. The empirical expressions proposed by Nielsen et al(45) are used to evaluate the effectiveness factors. As appeared in **Section 2.4.2.3** of **Chapter 2**, the effectiveness factor, ν, can be obtained from the following:

$$\nu = f_1(f_c) f_2(h) f_3(\rho) f_4\left(a/h\right) \tag{5.12}$$

in which,

$$f_1(f_c) = 3.5/\sqrt{f_c}$$

$$f_2(h) = 0.27\left(1 + 1/\sqrt{h}\right)$$

$$f_3(\rho) = 0.15\rho + 0.58$$

$$f_4\left(\frac{a}{h}\right) = 1.0 + 0.17\left(\frac{a}{h} - 2.6\right)^2$$

Where,

f_c	=	compressive strength of concrete cylinder
a	=	shear span
h	=	overall depth of beam section
ρ	=	the percentage of the longitudinal steel

It is clear that the above expression which has been determined from experiments depends on various factors. In order to be used for a honeycombed concrete beam, the simplest way is to evaluate the effectiveness factor separately for normal and honeycombed concretes based on their respective compressive strengths. This approach seems very inappropriate because the factor not only depends on the strength of concrete, but is influenced by the amount of steel reinforcement, the size of the beam and the shear span ratio. Those factors are not acting independently.

With regard to the effect of concrete strength on the effectiveness factor, the expression takes into account that the stronger is the concrete the more brittle it is. A concrete with higher strength produces a lower effectiveness factor and vice versa. Suggestions of using the strength of honeycombed concrete to evaluate the effectiveness factor for the whole beam may lead to a higher value of effectiveness factor which implies that the beam is less brittle in shear. It can also result in a higher shear capacity prediction if the effectiveness factor evaluated based on the honeycombed strength is used with the strength of normal concrete in the work equation. This may not give an accurate prediction of the behaviour and the shear capacity of honeycombed beams.

It may be appropriate to consider using the 'weighted average' of strength of both concretes, taken according to the lengths to which they contribute to the failure mechanism. This average strength can be used to evaluate the effectiveness factor and the shear capacity. If it is necessary to bring the effectiveness factor to the lowest value, probably the strength of the normal concrete can be used to obtain the effectiveness factor, and the 'weighted average' strength used to evaluate the shear strength.

Considering the mechanism as shown in **Figure 5.3**, the 'weighted average' strength, f_{cav} is given as,

$$f_{cav} = \frac{f_c l_c + f_{ch} l_h}{l_m} \tag{5.13}$$

where, $l_c = l_m - l_h$

The behaviour and the shear capacity obtained from the experimental work need to be observed and the effects of the presence of a honeycombed zone must be examined before the suitable selection of concrete strength is made. This will be discussed in **Section 6.3.2.1** of **Chapter 6** where comparisons are made between the experimental results and the theoretical predictions.

5.3.4 FORTRAN Programming For Evaluating Shear Force

Two separate programs were written, one to evaluate shear in beams without a honeycombed zone, called SHEAR 1, and the other for beams with a honeycombed zone, called SHEAR 2. The listing of the programs can be referred to in the **Appendix**.

Both programs are written in FORTRAN 77 language. SHEAR 1 enables the evaluation of shear capacity to be repeated for different strengths of concrete. SHEAR 2 enables the evaluation of the shear capacity of beams with a honeycombed zone using equation (5.11). A straight line plastic mechanism can be assumed to form at any distance, x, as shown in **Figure 5.3**, and at any angle within the shear span considered. For each assumed mechanism the program can evaluate the 'weighted average' strength of concrete. The effectiveness factor using equation (5.12) can then be evaluated using any concrete strength which can be specified in the program. The shear capacity of the beam can be obtained using the specified concrete strength. Using the program, the lowest value of shear capacity and the corresponding plastic failure mechanism can be obtained.

The program can handle only a honeycombed zone of rectangular or square shape. The size of the beam, the shear span, and the strength of both normal and honeycombed concretes can be specified as input data.

5.3.5 Honeycombed Beams With Shear Reinforcement

For a normal concrete beam with shear reinforcement, the best upper-bound shear strength can be obtained by the following general expression as given by Nielsen(45),

$$\frac{\tau}{f_c} = 2\sqrt{\Phi(1-\Phi)}.\sqrt{\psi(1-\psi)} \tag{5.14}$$

where,

$$\psi = \frac{A_{sv}.f_{yv}}{s_v.b.f_c} \tag{5.15}$$

ψ is the shear reinforcement degree. A_{sv} and f_{yv} are the area and the yield strength of shear reinforcement respectively. s_v is the spacing of the shear reinforcement. All other terms are as defined before. Note that the effectiveness factor is not yet included in the equation.

It should be noted that if $\Phi \geq 1/2$, the displacement of the yield line is vertical and it can be shown that it is necessary to replace $\sqrt{\Phi(1-\Phi)}$ in the equation by $1/2$. Hence,

$$\frac{\tau}{f_c} = \sqrt{\psi(1-\psi)} \tag{5.16}$$

Also if $\psi \geq 1/2$, then the shear reinforcement does not yield and it can be shown that it is necessary to replace $\sqrt{\psi(1-\psi)}$ by *1/2*.

Equation (5.14) is however valid only if,

$$\frac{h}{a} \leq tan\,\beta \leq \infty \tag{5.17}$$

where tan β can be obtained by,

$$tan\,\beta = \frac{1}{\sqrt{\Phi(1-\Phi)}} \cdot \frac{\sqrt{\psi(1-\psi)}}{1-2\psi} \tag{5.18}$$

The condition set above means that the yield line must be within the points of loading and support.

If tan β is smaller than *h/a*, then the shear strength can be obtained using the following equation,

$$\frac{\tau}{f_c} = \frac{\nu}{2}\left(\sqrt{\left(\frac{a}{h}\right)^2 + \frac{4\Phi(\nu-\Phi)}{\nu^2}} - \frac{a}{h}\right) + \nu\psi\,\frac{a}{h} \tag{5.19}$$

Equation (5.19) is identical to equation (5.6) except that it contains an extra term representing the contribution from the shear reinforcement, and cot β is replaced by *a/h*. The yield line extends from the point of loading to the point of support. Note that the effectiveness factor is included in equation (5.19). As for equation (5.6), the term $4\Phi(\nu - \Phi)/\nu^2$ is equal to unity, if $\Phi \geq \nu/2$.

For a honeycombed beam, the effectiveness factor will be evaluated based on the strength of the normal concrete and the value of f_c in equations (5.14) and (5.19) is taken from the 'weighted average' strength, f_{cav}. This approach is consistent as applied to honeycombed beams without shear reinforcement.

As given by Nielsen et al (45), for beams with shear reinforcement, the effectiveness factor can be obtained with the following relationship,

$$\nu = 0.7 - \frac{f_c}{200} \tag{5.20}$$

5.4 PLASTICITY THEORY FOR SHEAR IN BEAM WITH A CONSTRUCTION JOINT

There is an application of the theory of plasticity which has been developed specifically to evaluate the shear capacity of joints in concrete members(44),(45)(61). In the current study, the existing plasticity theory is applied to predict the shear capacity of a joint in a reinforced concrete beam. The term joint in this study refers to a narrow honeycombed zone at an angle to the horizontal axis of the beam and extending throughout the depth and the width of the beam. This simulates a construction joint which can occur as a result of poor construction practice. It is known to occur, particularly, in hot-climate countries.

Consider a concrete beam without shear reinforcement and containing a narrow honeycombed zone, inclined with an angle β with respect to the horizontal axis, located in the shear zone, with the shear span of a. Assume that the beam is subjected to shear force, V, and a line of plastic mechanism forms along the honeycombed zone. This was the mechanism observed in the test reported in **Section 4.6.1.2** of **Chapter 4**. Assume that the displacement of the plastic mechanism, u takes place at an angle, α to the mechanism. Refer to the diagram shown in **Figure 5.4**.

From the mechanism that forms along the joint, the work equation can be written in the following form(45),

$$V.u\,sin(\alpha+\beta)=u\left[\frac{(1-sin\alpha)}{2}f_c\right]\frac{bh}{sin\beta}-A_sf_y.u\,cos(\alpha+\beta) \qquad (5.21)$$

where,

f_c = the cylinder strength of concrete at the joint

The above equation is identical to the expression derived by Nielsen and Braestrup(43) for beams without shear reinforcement. Minimising against α, the solution to equation (5.21) for a joint at an angle β will be the same as equations (5.6) and (5.7).

The shallowest line of plastic failure mechanism extends from the support to the point of loading, for which $cot\ \beta$ is equal to (a/h). Hence, in the above solution $cot\ \beta$ cannot be greater than (a/h). For joints, the effectiveness factor can be taken as 0.45(45),(61).

A theoretical prediction of the shear capacity of joints at various angles of inclination can be carried out using the above solution. It should be clear that in the actual prediction of the capacity of joints, the minimum value of shear capacity can be obtained if a mechanism forms at an angle shown in **Figure 5.5**.

The above solution is incomplete as it is unable to predict the shear capacity of a beam if a joint is formed at an angle less than the angle of the line connecting the point of loading and the support as shown in **Figure 5.6**. Note that the line connecting those two points is the plastic mechanism that will produce a minimum shear capacity of a beam. The solution for this particular case can be derived as follows.

Consider a shear zone in a beam subjected to a shear force of V. Assume that a narrow honeycombed zone simulating a joint exists as shown in **Figure 5.6**. At failure a possible plastic mechanism is as shown, where two separate yield lines may form, one in the normal concrete and the other along the joint, but joining together. The length of mechanism in the joint is denoted as l_j, and it must not be beyond the line of loading. The length of the plastic mechanism in the normal concrete is denoted as l_n.

The angle of the joint is β and assuming that the mechanism in the normal concrete forms an angle γ with the vertical axis. The displacement, u, forms angles of α and θ with the mechanisms in the normal concrete and in the joint respectively. The tensile strength of the joint is ignored. With regard to the effectiveness factor, its value is calculated based on the normal concrete strength, because the more brittle behaviour of the normal concrete will govern the failure mode of the combined mechanism. This is consistent with the approach for beams with a honeycombed zone. They are treated in the same way (refer to **Section 6.3.2.1** of **Chapter 6**).

From the diagram, and assuming that the concrete strength of the joint is the strength of honeycombed concrete, f_{ch} the following work equation can be written,

$$Vu\cos(\alpha-\gamma)=\frac{1}{2}\nu b.u\left[f_c(1-\sin\alpha)l_n+f_{ch}(1-\sin\theta)l_j\right]+A_sf_y.u\sin(\alpha-\gamma) \qquad (5.22)$$

The equation then can be rewritten as,

$$V=\frac{1}{2}\frac{\nu b}{\cos(\alpha-\gamma)}\left[f_c(1-\sin\alpha)l_n+f_{ch}(1-\sin\theta)l_j\right]+A_sf_y.\tan(\alpha-\gamma) \qquad (5.23)$$

From the geometrical consideration, it can be found that,

$$tan\gamma = \frac{a - l_j . cos\beta}{h - l_j sin\beta} \tag{5.24}$$

$$l_n = \sqrt{(a - l_j . cos\beta)^2 + (h - l_j . sin\beta)^2} \tag{5.25}$$

$$\theta = 90 + \alpha - \beta - \gamma \tag{5.26}$$

The limits for α and θ are:

$$\gamma \leq \alpha \leq 90$$

$$90 - \beta \leq \theta \leq 90$$

At failure the plastic mechanism can be a combination of a failure line in the normal concrete and in the joint; and l_j together with α and θ are the variables for any given value of β.

There will be cases where joints are at a certain angle below which the failure of the beam will be governed by the failure in the normal concrete, and will not involve the joint. This occurs when the plastic mechanism only occurs in the normal concrete extending from the point of loading to the support. By using equation (5.23) that angle can be found.

In the following section, equation (5.23) together with equations (5.6) and (5.7) will be used to study the shear capacity of reinforced concrete beams with a joint present at various angles, with different concrete strength of the joint, different shear span ratio and with different amount of longitudinal reinforcement.

5.4.1 The Theoretical Prediction of Shear Capacity of Beam With a Construction Joint

In this section results of the theoretical prediction of shear capacity of reinforced concrete beams without shear reinforcement and containing a joint are presented and discussed.

The variables in this analysis are the strength of the honeycombed concrete in the joint, the angle of the inclination of the joint, the amount of longitudinal reinforcement and the shear span. Honeycombed strengths included in this study are 10, 15 and 20 N/mm^2. The angles of the inclination are in the range of 25° to 85°. The amounts of longitudinal reinforcement are 0.2, 0.5 and 1.0 %. For the shear span, shear span ratios, *a/h,* of 1.0, 1.5 and 2.5 are examined. The yield stress of steel is taken from the current tests, and the value is 497 N/mm^2. The area of steel, A_s, is determined based on 2 number 12 mm diameter bars of longitudinal reinforcement which is 226 mm^2 as used in the beam of the current study.

Note that all the concrete strengths mentioned in this section are referred to as cube strengths unless stated otherwise. In order to be used in the plastic analysis, the cube strengths are to be converted to cylinder strength by the factors given in **Section 4.2.5** in **Chapter 4**; 0.76 for normal concrete and 0.66 for honeycombed concrete.

5.4.2 The Effect of the Strength of Normal Concrete

The initial analysis shows that concrete strength does not have a significant effect on the shear capacity of beams without a joint, except for beams with a high amount of longitudinal reinforcement. The initial analysis of calculating the shear capacity of beams without a joint is done using equations (5.6) since in all cases studied, the degree of reinforcement, calculated based on the strength of the normal concrete, resulted in

$\Phi \leq \nu/2$. The value of *cot* β in the equation is replaced with *a/h*. The effectiveness factor is evaluated using equation (5.12).

Figure 5.7 shows the plot of shear capacity of control beams versus the strength of normal concrete for different shear span ratio and different amount of longitudinal reinforcement. The strengths studied are in the range of 30 to 50 N/mm^2. For beams with a longitudinal reinforcement percentage of 0.2% the biggest difference of shear strength occurs in beams with a shear span ratio of 1.0. The difference in shear capacity between beams with 30 N/mm^2 and 50 N/mm^2 concrete strength is only 3%. The largest difference of shear capacity occurs in beams with a shear span ratio of 1.0 with the percentage of longitudinal reinforcement of 1.0%. The difference is 13.5%. The insignificant effect of concrete strength is due to the fact that a concrete with higher strength is more brittle, and consequently results in a lower effectiveness factor. It has been demonstrated in tests that the increase in the strength of concrete beam would not result in the same increase in the shear capacity(63),(64). It has also been observed that a shear failure in a high strength concrete beam is more abrupt, signifying the high brittleness of the concrete(63),(64).

The bigger difference in beams having a high percentage of longitudinal reinforcement, compared to beams with a low percentage, is due to more of the shear strength being provided by a larger amount of steel. Note that in the case of this study, since $\Phi \leq \nu/2$, calculated using the strength of normal concrete, the reinforcement does contribute to the shear capacity of the beam. The influence of the longitudinal reinforcement can be explained as follows. According to equation (5.8), for a given β, α depends on the value of Φ. In the range of $\Phi \leq \nu/2$, as Φ decreases, α will increase. This results in more work being done in the steel as more displacement takes place in it, and this causes the influence of the concrete on the shear capacity to be less significant.

The following analysis is carried out in terms of the ratio of shear capacity of the joint to the shear capacity of the beam, $V_{joint}/V_{control}$. From the above investigation, it is found that, although not very significant, the higher is the strength of the normal concrete, the higher shear capacity it is. For any given joint, the ratio of shear capacity will be lower if the shear capacity of the beam with a higher concrete strength is taken. Thus, the following analyses are carried out based on control beams with a strength of concrete of 50 N/mm^2. The resulting strength ratio when applied to a beam with a lower strength of concrete will be conservative.

5.4.3 The Shear Capacity of Beams With Construction Joint

As already mentioned, to evaluate the shear capacity of a joint, V_{joint}, equations (5.6) and (5.7) are used. Both equations can be used directly. At a given β, the shear capacity of a joint of different concrete strengths can be evaluated for different a/h values and percentages of longitudinal reinforcement. The concrete strength of the joints studied are 10, 15 and 20 N/mm^2. The a/h values are 1.0, 1.5 and 2.5 and the longitudinal reinforcement is in the range of 0.2, 0.5 and 1.0%.

For equation (5.23), for a given β, l_j and α are variables. The minimum work done must be determined by a numerical iteration. $V_{control}$ is defined as the shear capacity of a beam without a joint, for which the plastic mechanism forms from the point of loading to the support.

Figure 5.8 shows the relationships between $V_{joint}/V_{control}$ and the angle of the joint, β, for a/h values of 1.0, 1.5 and 2.5 and the values of $100A_s/bh$ of 0.2, 0.5 and 1.0. The strengths of the concrete in the joint are 10, 15 and 20 N/mm^2, and the strength of the normal concrete of the beam is 50 N/mm^2.

In the following discussion, the effects of various parameters on the shear capacity of beams with a joint are discussed.

5.4.3.1 The Effect of the Strength of the Joint Concrete

The influence of the strength of concrete in the joint on its shear capacity is significant. This is clearly shown in **Figure 5.8**. In terms of their shear capacity, at $100A_s/bh$ of 1.0, and for any given angle of inclination, the increase in shear strength is proportional to the increase in the joint strength. This can be easily understood as, when evaluating the shear capacity of joints, the effectiveness factor is fixed at 0.45. When the value of $100A_s/bh$ is 1.0, the degree of longitudinal reinforcement, $\Phi \geq \nu/2$, thus the shear capacity is directly proportional to concrete strength. Note that in evaluating Φ for joints, it is calculated based on the strength of the honeycombed concrete. For cases where, $\Phi \leq \nu/2$, the difference in shear capacity will also depend on the amount of longitudinal reinforcement. However the difference in shear capacity is still very significant due to the fixed value of the effectiveness factor regardless of the strength of the joint.

It should be noted that the fixed value of the effectiveness factor of 0.45 for joints is based on far less test data than is the variable value for beam shear.

5.4.3.2 The Effect of the Shear Span

In terms of the shear span, for a short shear span beam, failure can occur in a joint although its angle is steep. For example for a/h of 1.0, and $100A_s/bh$ of 0.2, failure will occur in a joint if its angle is below about 70° for f_{ch} of 10 N/mm^2 and below about 60° for f_{ch} of 20 N/mm^2 (refer to **Figure 5.8**).

For a long shear span beam, for example, for a/h of 2.5 (refer to **Figure 5.8**), there are tendencies that the failure of the beam will be governed by the control except for beams with a joint at quite a flat angle. The angle of the critical mechanism in the control beam for a/h of 2.5 is about 22°. Referring to **Figure 5.8**, for a joint with a concrete strength, f_{ch}, of 10 N/mm^2, the failure will only occur in the joint if the inclination of the joint is at

35^o or below. For joints with a higher concrete strength, it needs to be at a more shallow angle for a joint to govern the failure.

This can be explained as follows. For a long shear span beam the shear capacity of the control beam is very low, compared to a short beam. The failure in the joint will only take place if the strength of the joint is very low and it forms at a shallow angle. For a short beam, it possesses a high shear strength, and a joint present, even though at a steep angle, can lead to a failure of the beam through the joint.

5.4.3.3 The Effect of the Longitudinal Reinforcement

From the graphs in **Figure 5.8**, it is clear that the percentage of the longitudinal reinforcement has a significant influence on the shear capacity of the beams and joints. Note that in evaluating the shear capacity of the control beam, in all cases, $\Phi \leq \nu/2$, so the effect of reinforcement is present for all ranges of $100A_s/bh$ studied (see equation (5.6)).

In evaluating the shear capacity of a joint, for a joint with a concrete strength of up to 15 N/mm^2, the effect of reinforcement only occurs when its percentage is below 0.5. There is no increase in the shear capacity of a joint for $100A_s/bh$ values of 0.5 and above. This occurs because once $\Phi \geq \nu/2$, the displacement of the plastic yield line, u, will be in the vertical direction and the longitudinal reinforcement will no longer contribute to the shear capacity of the joint. When the strength of the joint increases to 20 N/mm^2, an increase in the reinforcement percentage up to 1.0 %, which results in $\Phi \leq \nu/2$, will increase the shear capacity of the joint.

From all the graphs they show that the increase in $100A_s/bh$ can lead to a failure in the joint rather than in the control beam. For example, for beams with a/h of 1.0, as $100A_s/bh$ increases to 1.0, and regardless of the strength of the joint concrete, its presence within the shear zone at any angle up to as steep as 85° will result in a joint failure. For beams with *a/h* of 1.5 and at $100A_s/bh$ of 1.0, a failure will only be governed by the control beam, if the strength of the joint concrete is 20 N/mm^2 and the angle of the joint is below 80°. For a longer shear span beam, for example with *a/h* of 2.5, a joint with a low concrete strength will govern the failure regardless of its angle. As the strength increases, for example at f_{ch} of 20 N/mm^2, the failure in a joint occurs at angles below 55°.

In the range of $100A_s/bh$ studied, the increase in the amount of longitudinal reinforcement will result in the increase of shear capacity in the control beam. However the increase in the shear strength of the joint is not proportional to the increase in the control for the reason already explained above. As a result a higher amount of reinforcement in the beam does not carry any advantage if a joint is present, since the failure will be governed by the joint.

5.4.3.4 The Effect of the Inclination of the Joint

This is a straight-forward relationship as long as the angle of the joint is greater than that of the line joining the point of loading and the support. A steeper joint needs more work to fail it, thus the shear capacity is higher, and a flatter joint needs less work, thus leading to a lower shear capacity. A joint with a flat angle present in a beam may lead to a joint failure, compared to a steeper joint. However these effects also depend on other parameters as can be seen from the graphs in **Figure 5.8** and the previous discussion. For example, for a short beam, even with an almost vertical joint, the failure still can be governed by a joint.

In this theoretical study, the effects of a joint which extend beyond the point of loading are also examined. Joints with angle of inclination as flat as 25° with shear span ratios, *a/h,* of 1.0 and 1.5 are investigated. In a normal situation a joint flatter than 25° is very unlikely to exist. Using equation (5.23), the effect of such joints can be seen in Graphs

(a) to (f) of **Figure 5.8**. As already mentioned, in using equation (5.23), the effectiveness factor is evaluated based on equation (5.12) and the strength of normal concrete is used.

It is interesting to note that as the angle of the joint decreases below a certain value, the beam will fail without the participation of the joint. In other words a joint present at less than a certain angle of inclination in the beam will not cause the beam to fail early, but the capacity of the beam in conventional beam shear will govern the failure. In that situation the mechanism of plastic failure will only form in normal concrete. This happens even when the strength of the concrete in the joint is very low. Joints with an angle greater than the above, but lower than the angle of the line joining the point of loading and support will produce a failure mechanism consisting of a plastic mechanism in the joint and in the normal concrete. In that circumstance their capacity is lower than the control.

The value of that critical angle depends also on other parameters. From **Figure 5.8** it can be seen that for a given a/h value, the angle is shallower as $100A_s/bh$ increases. For example for a/h value of 1.0, and for $100A_s/bh$ value of 1.0, the angle is about 33°, compared to 42° for $100A_s/bh$ of 0.2. With regard to the effect of shear span on this angle, as the shear span increases the angle will get shallower. This can be seen by comparing Graphs (d) to (f) with Graphs (a) to (c) of **Figure 5.8**. For example from Graph (a), where a/h is 1.0, the angle is 42°, compared to an angle between 30° to 32° when a/h is 1.5.

5.4.4 The Summary of the Behaviour of Reinforced Concrete Beam with a Joint in Shear Failure

From the above theoretical study, a single graph depicting various failure modes of a beam with a construction joint at various angles of inclination can be prepared as a summary. The graph, shear capacity versus the angle of inclination of the joint, β, is shown in **Figure 5.9**. β_b is the angle of the line joining the point of loading and the support. β_I in the plot is the angle below which a joint, with the angle of inclination of less than β_b, will cause the beam failure to be governed by the mechanism in the normal concrete only. β_{II} is the angle of a joint with the angle of inclination greater than β_b, above which a shear failure is governed by the beam.

In the graph plotted, a failure mode can be classified into 4 zones. The shear capacity of a beam with a joint inclined at an angle up to β_I will be governed by a mechanism in which a yield line forms in the normal concrete only. This is shown as zone A in the diagram. A joint present in a beam at angles between β_I and β_b will cause a shear failure in which a mechanism is a combination of yield lines formed in the normal concrete and in the joint. This mode is shown by zone B in the diagram. As the angle of a joint increases between β_b and β_{II} failure occurs when a mechanism forms in the joint. This is in zone C. Zone D represents a beam with a joint greater than β_{II} and failure is governed by the shear strength of the beam. From the diagram, a beam will posses a lowest shear capacity when a joint is present at an angle equal to β_b.

Figure 5.10 shows the sketch of a typical plastic yield line according to the failure mode for each zone. The mode of failures summarised in the diagram will only be of concern for short to medium shear span beams. According to this study it is only for *a/h* is in the range of 1.0 to 1.5 that there is the possibility of any of the four modes occurring. However, as already discussed, depending on other factors such as the amount of longitudinal reinforcement and the strength of the joint, a failure mode in zone D may not occur for a beam with a short shear span. Refer to **Figure 5.8**.

For a long shear span beam, according to this study, at *a/h* of 2.5 and above, only the failure modes in zones C and D should be of concern. This is because to involve the failure modes in zones A and B, the joint in the beam must be at a very shallow angle, which in practice is very unlikely to exist.

5.5 CONCLUSIONS

A plasticity theory has been extended to be used as a tool in assessing the shear capacity of beams with a honeycombed zone. The appropriate method of treating the effectiveness factor remains to be decided and this will be discussed further in **Section 6.3.2.1** of **Chapter 6**, where comparisons will be made between test data and the analytical prediction.

A plasticity analytical tool is also developed which can be used to assess the shear capacity of beams with a construction joint. The theoretical predictions presented are useful information for the assessment. A summary of the theoretical shear behaviour of a beam with a joint is also presented. In order to validate the entire set of results, a significant number of tests will be required. Within the scope of the present study it has been possible to carry out the four tests which were described in **Section 4.6.1.2** and **Section 4.7.2.1** in **Chapter 4**. The comparisons of the test data with the theoretical prediction will be discussed in **Section 6.5** of **Chapter 6**.

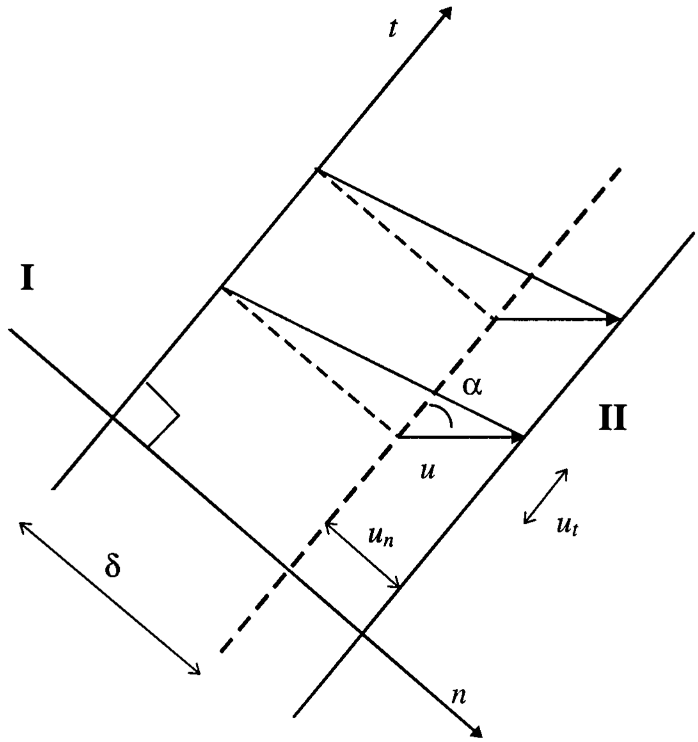


FIGURE 5.1 Displacement zone between two rigid parts

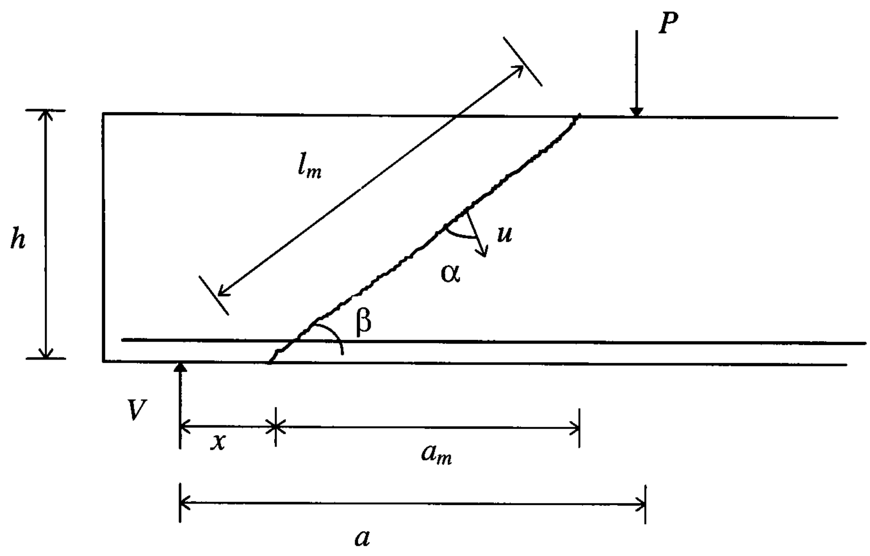


FIGURE 5.2 Plastic failure mechanism, beam without honeycombed zone

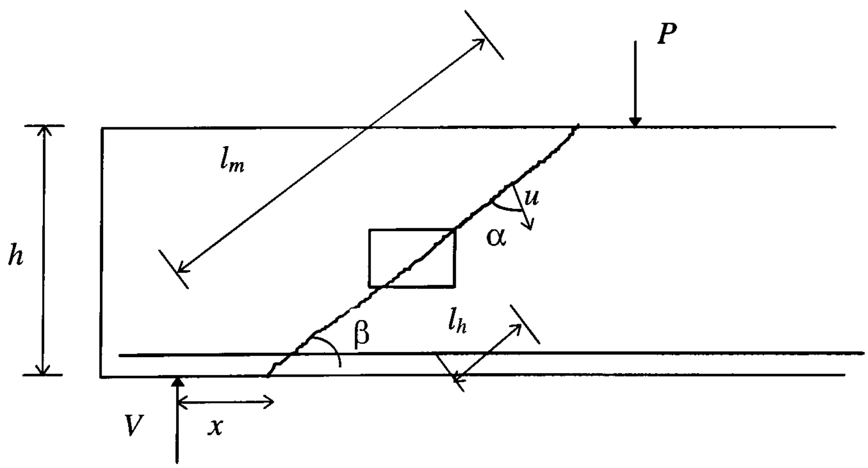


FIGURE 5.3 Plastic failure mechanism, beam with honeycombed zone

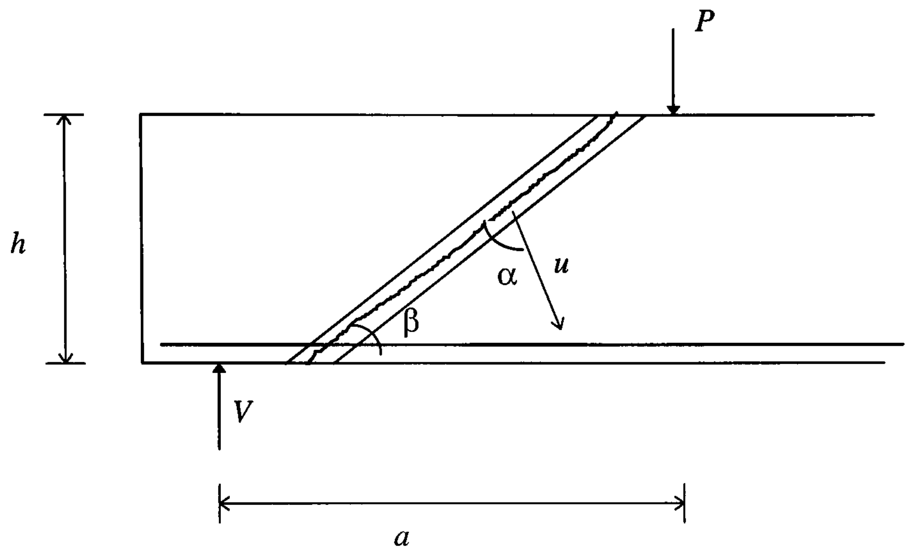


FIGURE 5.4 Plastic failure mechanism, the joint in a beam

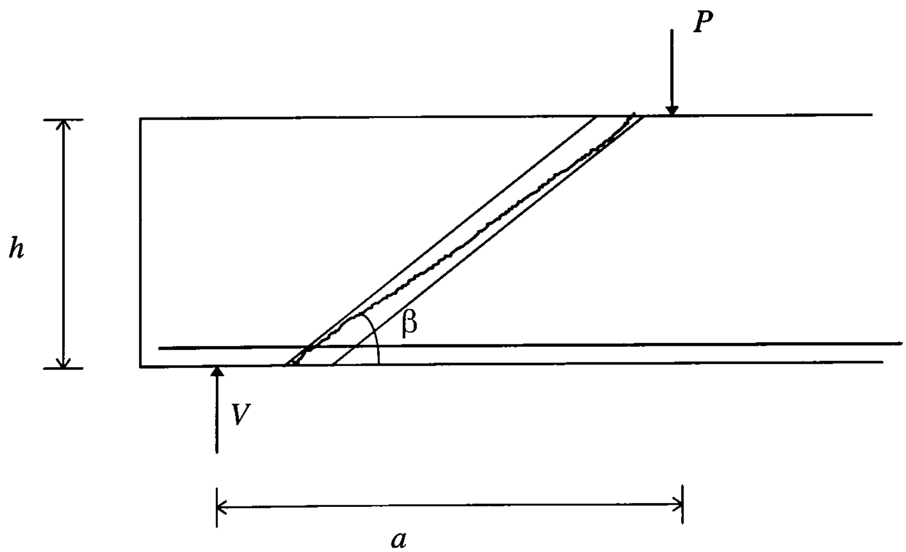


FIGURE 5.5 Plastic failure mechanism formed at more flat angle in joint

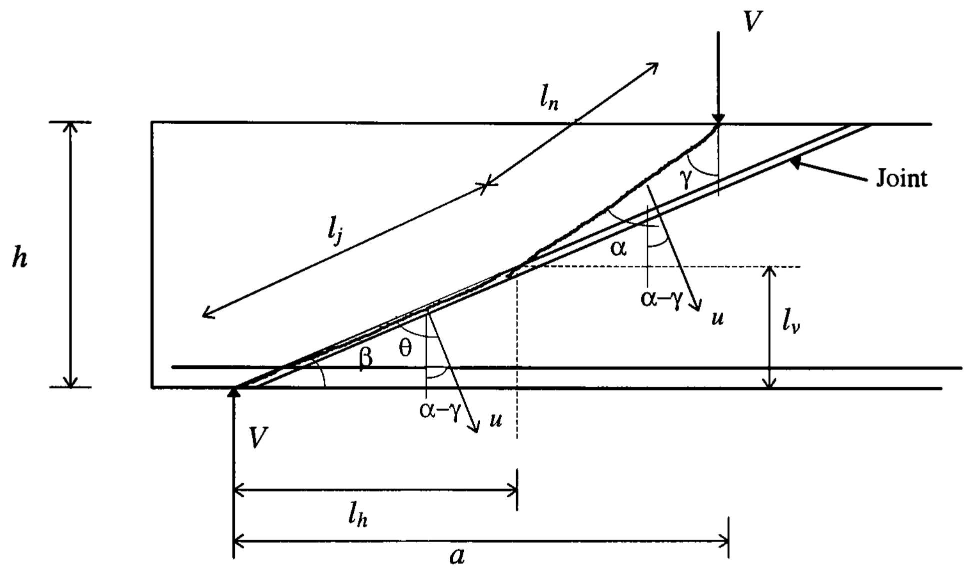


FIGURE 5.6 Plastic failure mechanism, the joint in a beam

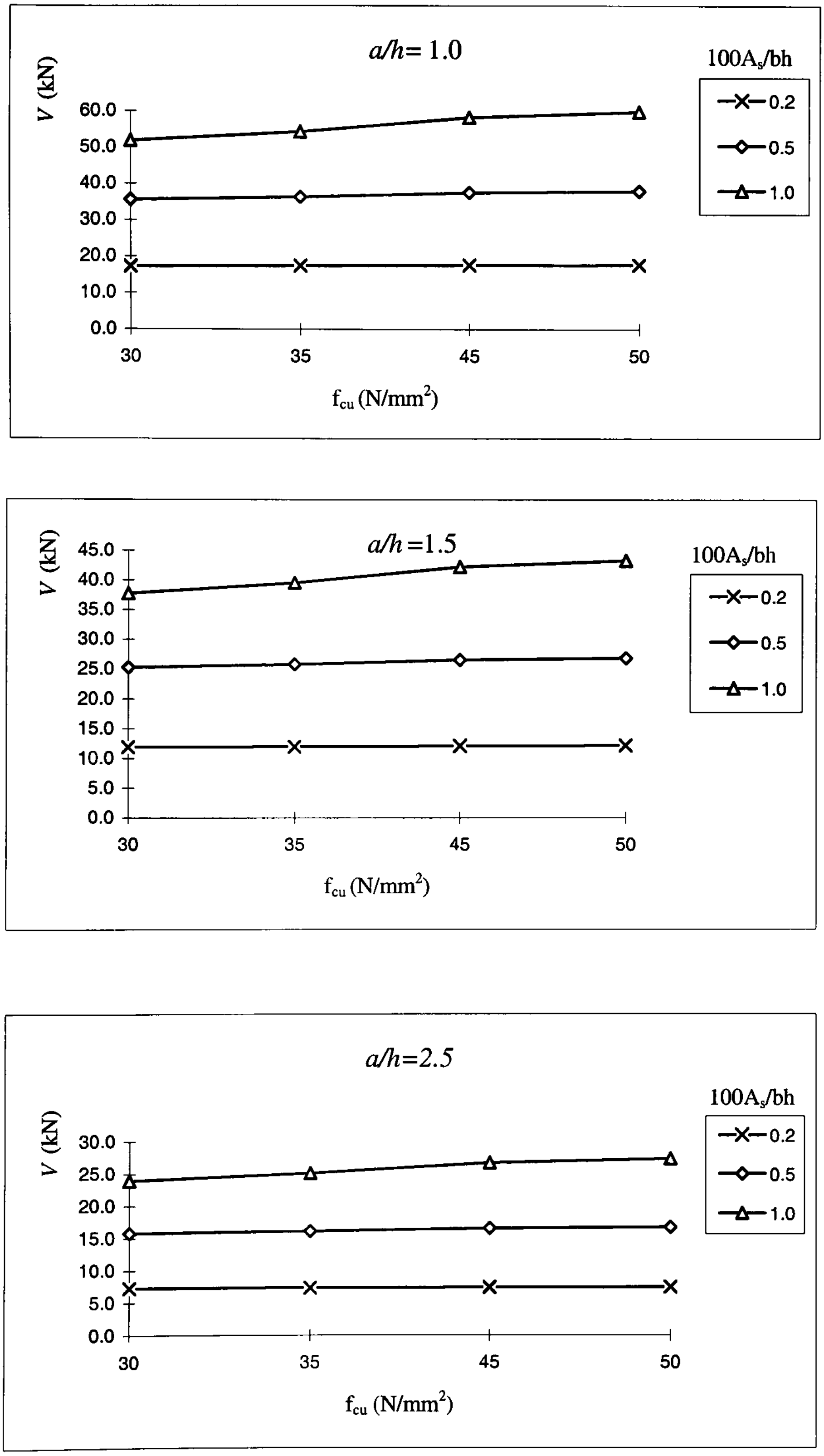


FIGURE 5.7 Shear strength vs concrete strength of beams without joint for different *a/h* and the percentage of longitudinal reinforcement

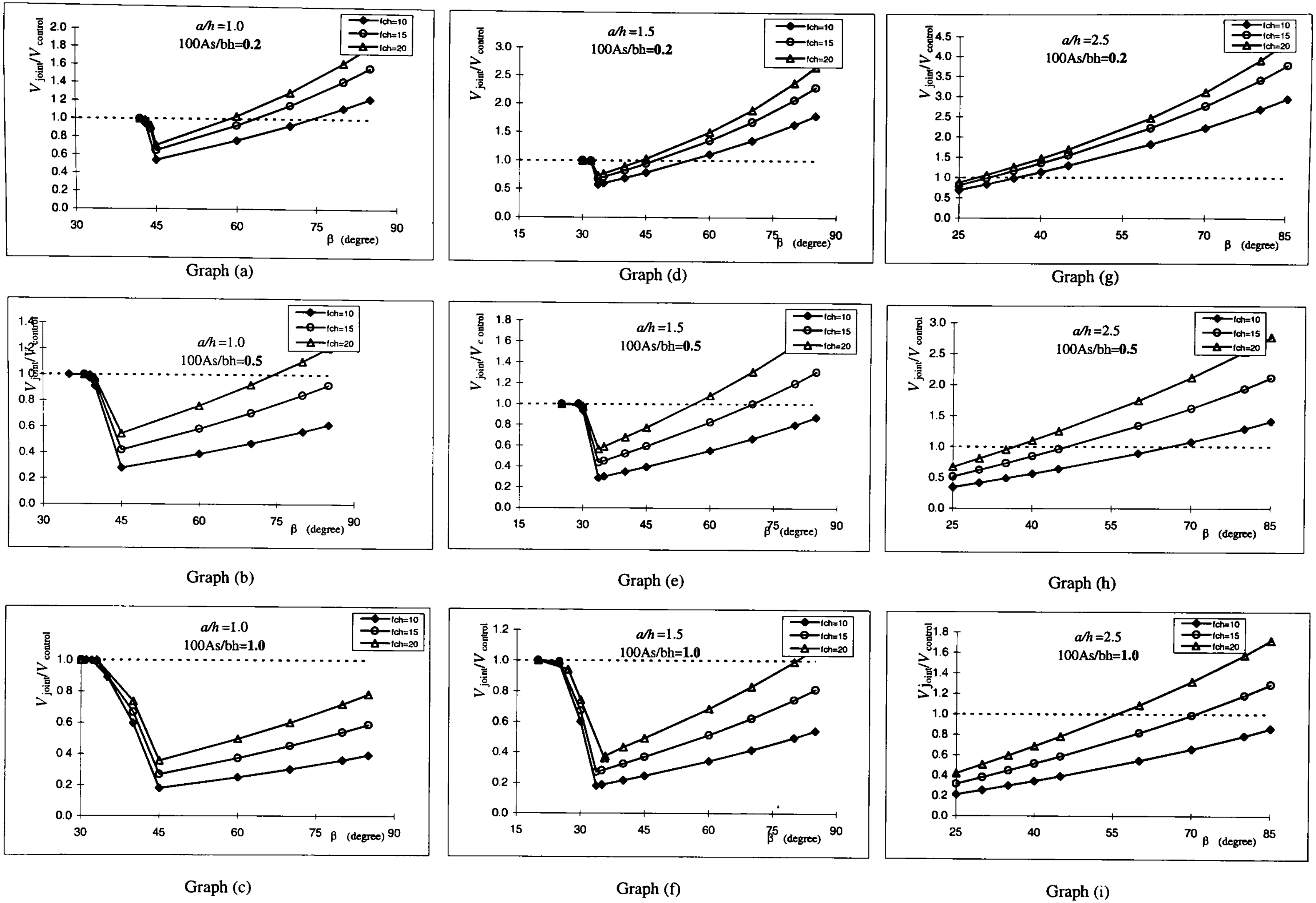


FIGURE 5.8 $V_{joint}/V_{control}$ vs β for different a/h ratio and $100A_s/bd$

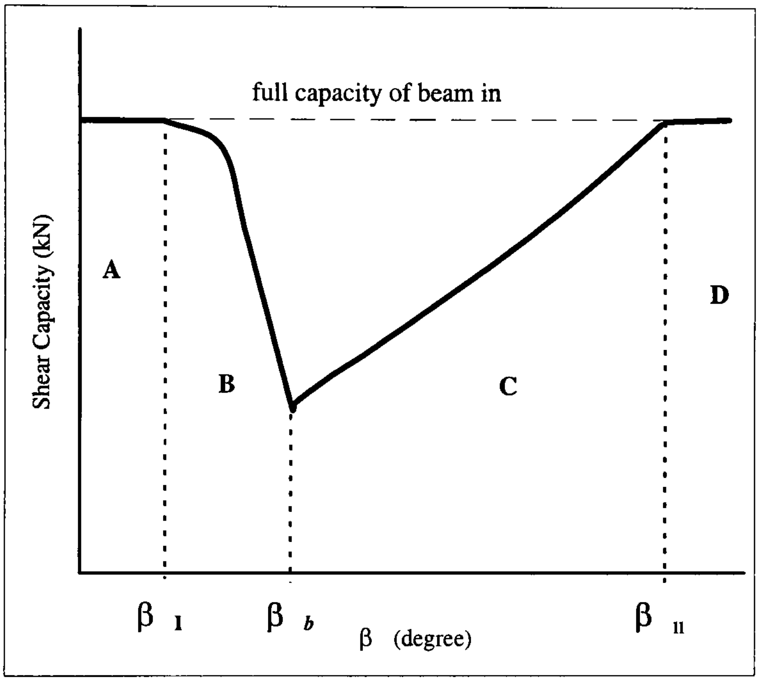


FIGURE 5.9 The mode of failure of a beam with a construction joint

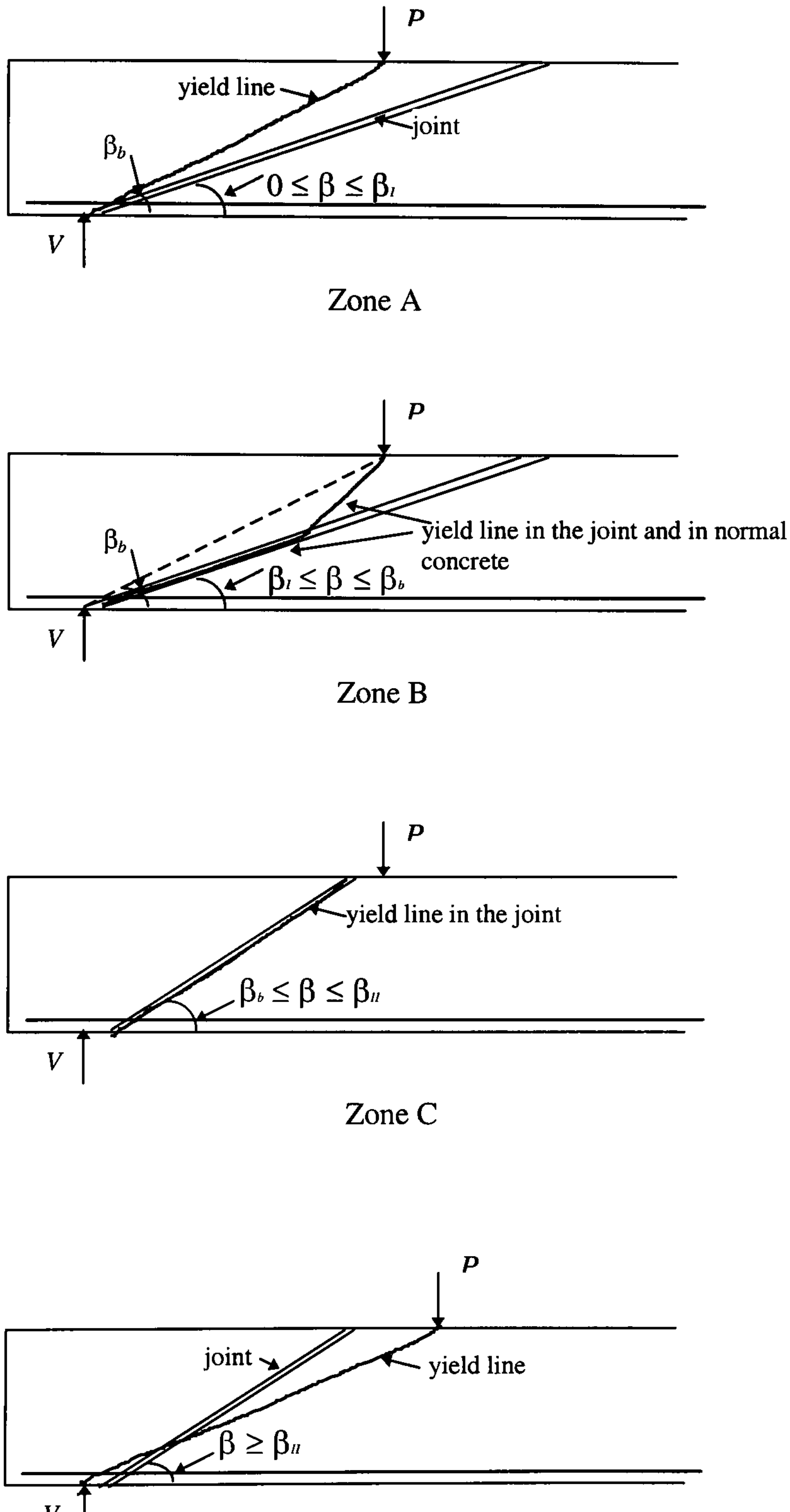


FIGURE 5.10 Various modes of failure for beam with a joint

CHAPTER 6

THE COMPARISONS BETWEEN ANALYTICAL METHODS AND EXPERIMENTAL RESULTS

6.1 INTRODUCTION

In this chapter the shear capacities of honeycombed beams, obtained from tests are compared with the predictions made by various analytical tools. The plasticity methods as described in **Chapter 5**, together with BS 8110 and BD 44/95, are used and their predictions are compared with the experimental results. Condition factors derived from the statistical analysis of test data are proposed to be included in the plastic analysis when it is used for the assessment of shear capacity of honeycombed beams without shear reinforcement.

For reinforced concrete beams with a joint, theoretical predictions of their failure loads are compared with the results obtained from test data.

Prior to that, the stress distribution within the high shear area was examined by using linear elastic finite element analysis. The purpose of this analysis is to examine, theoretically, if the presence of a honeycombed zone causes a significant shifting of stress redistribution or elastic stress concentration in the shear zone.

6.2 THE ELASTIC STRESS DISTRIBUTIONS WITHIN THE HIGH SHEAR ZONE- THE FINITE ELEMENT ANALYSIS

The purpose of using this linear elastic finite element analysis was to examine the distribution of stresses within the high shear area when a honeycombed zone present in the zone. This is to theoretically check any development and/or shifting of stress concentration that can affect the formation of flexural and diagonal cracks in the beam. It was observed in the present tests that, in some of the honeycombed beams, notably beams in series 1A and also in series 2A in which a honeycombed zone was located along the elastic neutral axis, the development of cracks seems to be influenced by some forms of stress concentration. Diagonal cracks in beams 1A-3[MS] and 2A-2[MS], both with the honeycombed zone located along the neutral axis near to the support, and beam 1A-4[ML] with the honeycombed zone located along the neutral axis near to the loading point, formed in the centre of the shear span. This suggests that a high stress concentration might have shifted to the central area of the shear zone because of the presence of the honeycombed zone. This needs to be investigated theoretically.

The analysis was carried out using LUSAS Finite Element Analysis package, Version 12. Two types of two-dimensional plane stress element were used to model the whole beam. The elements were an eight noded 2-D continuum element, used to model the concrete, and a three noded bar element, used to model the steel reinforcement, and each was known as QPM8 and BAR3 respectively in the package.

For linear elastic analysis, the QPM8 element requires the thickness of the beam as geometrical properties input, and the Young's modulus and Poisson's ratio of concrete as material properties input. BAR3 element requires the total cross-sectional area of the steel bars in the beam as its geometrical properties input, and Young's modulus and Poisson's ratio as input to the material properties. The values of Poisson's ratio of normal and honeycombing concretes and steel reinforcement were assumed as 0.2 and 0.3 respectively. All other values were taken from the experiment. The Young's modulus of the steel was 193.4 kN/mm^2. The moduli for the normal and honeycombed concretes were 28 kN/mm^2 and 16 kN/mm^2 respectively.

Four beams were analysed: three honeycombed beams, each with a honeycombed zone along the neutral axis but at different positions, and one control beam. **Figure 6.1** shows the meshes of the control beam and also for the honeycombed beam with a honeycombed zone at the centre of the shear area in element number 19. For the other two honeycombed beams with a honeycombed zone along the neutral axis but near the support, and the other near to the loading point, meshes are slightly modified. They are shown in **Figures 6.2** and **6.3.**

In the analysis, two values of Young's modulus were used to model the honeycombed zones. First, a honeycombed zone was modelled with a very small value of Young's modulus of 0.1% of the modulus of the normal concrete, and then the actual value was used. With regard to the load, all beams were analysed with a point load of 10 kN. This load was chosen, because this analysis was trying to detect a stress concentration prior to cracking, which means that the materials were still in the linear elastic region. Based on the experimental observations of series 1A and 1B in the current study, at 10 kN load, no flexural or diagonal cracking was observed to have formed in the shear area. Principal tensile and compressive stresses are taken from the output of the analysis for examination.

The principal tensile and compressive stress distributions obtained from the analyses using the actual Young's modulus of the honeycombed concrete are presented in **Figures 6.4, 6.5, 6.6** and **6.7**. Negative sign is for compression. The values shown are the nodal average stresses.

Close examination of the results show that no significant shifting of stress concentration occurs in the central area of the shear span due to the presence of a honeycombed zone.

As expected when using a finite element analysis, of course there were zones where stress concentration occurs. When the actual values of Young's modulus of honeycombed concrete were used the stress concentration at the boundaries were not that significant. For a beam with a honeycombed zone near the support and also a beam with a honeycombed zone near to the loading point, there was no indication from the analysis that the stress concentration had shifted to the central area of the shear span, which could have caused the formation of a diagonal crack in that region, as observed in the experiment.

From the analysis, and the comparison made with the experimental observations, it is found that a simple theoretical prediction of shear transfer, such as this linear elastic approach is very unlikely to yield any good correlation. A more complicated stress analysis is probably needed. This will require a more detailed model of the materials and a non-linear analysis. However, it is beyond the scope of the current study.

6.3 PREDICTION OF THE SHEAR CAPACITY OF HONEYCOMBED BEAMS USING AN UPPER-BOUND PLASTICITY METHOD

The method described in **Section 5.3** of **Chapter 5** is used to evaluate the shear capacity of beams with honeycombed zone located in the high shear zone. Also discussed is the use of Nielsen and Braestrup equations (described in **Sections 5.3.1** and **5.3.2** of **Chapter 5**) to evaluate the shear capacity of honeycombed beams. A clear definition of various parameters are explained below before proceeding with further discussions.

6.3.1 Parameters and Dimensions

In using the plasticity analysis, the dimension of the shear span of a beam is determined as proposed by Nielsen(45). Following that of Nielsen, the shear span is taken as the internal distance measured from the edge of the load platen to the edge of the support platen. Referring to the **Figures 3.4** and **3.5** in **Chapter 3**, and taking into account that the width of the platens at the support and at the loading point are each 50 mm, the shear spans of beams in **Figure 3.4** and **Figure 3.5** are thus 300 mm and 580 mm respectively.

The plasticity methods in this study refer to the concrete compressive strength as the strength obtained from cylinders. As described in **Section 4.2.5** of **Chapter 4**, it was found from the experimental work in this current study that the cylinder strength of normal concrete was about 76% of the strength obtained from cubes. For honeycombed concrete it was about 66%. These relationships will be used in the following plastic analysis.

The properties of the longitudinal reinforcement used are as given in **Section 3.2.1** of **Chapter 3**, in which the yield stress of the 12 mm bottom steel bar, f_y is 497 N/mm^2. The area of steel, A_s in all beams is 226 mm^2. The area of steel is based on 2 number 12 mm diameter steel bar provided at the bottom of the beam as described in **Section 3.3.1** of **Chapter 3**.

For the shear reinforcement, as described in **Sections 3.2.1** and **3.3.1** of **Chapter 3**, 3 mm diameter bar at 100 mm spacing was provided in the beam. The values of the area of shear reinforcement, A_{sv} and the yield stress, f_{yv} used in evaluating the shear capacity of beams with shear reinforcement are 14 mm^2 and 523 N/mm^2 respectively.

6.3.2 The Prediction of Shear Capacity of Honeycombed Beams Without Shear Reinforcement

Using the approach and equations described in **Section 5.3** of **Chapter 5**, the prediction of the shear capacity of control beams without a honeycombed zone, and beams with a honeycombed zone are carried out. Before discussing and comparing the prediction results with the test data, the selection of the concrete strength to be used in evaluating the effectiveness factor is first presented below. Also discussed is the use of equations (5.6) and (5.7), Nielsen and Braestrup equations, to evaluate the shear strength of honeycombed beams.

6.3.2.1 The Strength of Concretes to Evaluate the Effectiveness Factor and the Shear Strength

After trying a few values with regard to the strength of concretes by using the program SHEAR 2, it was decided that the effectiveness factor for the honeycombed beams will be evaluated based on that for the strength of normal concrete. This applies to all cases of the honeycombed beams, regardless of whether the critical yield line, that is the yield line that gives the lowest shear strength, passes through the honeycombed zone or not. Note that the evaluation of the effectiveness factor in SHEAR 2 was carried out using equation (5.12). The normal strength concrete was used because it produces the lowest value of the effectiveness factor. Another reason for that selection is that overall consistencies can be seen when the prediction results are compared with the experimental results.

For example, as shown in **Table 6.1(a)** and **(b)** the shear ratio of the experimental result to the theoretical prediction seems to agree with the behaviour of beams observed in the tests. The lower ratios of test to theoretical failure loads that occur in beams in series 1A and the higher ratio in series 1B are consistent with the observation made in the test which has been described in **Section 4.7** of **Chapter 4**. It was observed that, for example beams 1A-2[MM] failed at a very low load compared to their control, whereas beams 1B-2[MM] failed at the average load almost close to their controls. Prediction using the effectiveness factor determined based on the normal concrete strength produces the same pattern.

Initially, the effectiveness factor was evaluated using the strength of the honeycombed concrete for cases where the critical plastic yield line passes through a honeycombed zone. However, results showed that the prediction was higher as a result of the higher effectiveness factor due to the low concrete strength used, and overall consistencies with test data could not be achieved. It gave a close prediction for series 1A test, but predicted a much higher load for beams in series 2A, 1B and 2B. The use of a 'weighted average' strength was also tried and the results produced were not as low as if the normal strength were used.

The shear strength was evaluated using the 'weighted average' strength for beams where the critical plastic failure mechanism passes through both the normal and the honeycombed zone. By using SHEAR 2, this was done automatically. If the lowest shear is given by a failure mechanism that does not pass through the honeycombed zone, SHEAR 2 takes that into account and the shear strength will be evaluated using the strength of normal concrete alone. The use of a 'weighted average' strength in calculating the shear strength in cases where plastic failure mechanism passes through the two concretes was done after the same consideration as above was made on the test observations, and consistencies in the overall results could be achieved.

6.3.2.2 Theoretical Prediction Using SHEAR 1 and SHEAR 2

The results of the shear prediction using SHEAR 1 and SHEAR 2 are shown in **Table 6.1(a)** for series 1A and 2A beams and in **Table 6.1(b)** for series 1B and 2B beams. All beams are included. The beam with a void instead of honeycombed concrete in the shear zone (beam 2A-1.3) is also included for comparison. For this beam the strength of the honeycombed concrete was put as zero in evaluating its shear capacity using the SHEAR 2. Note that both mean and characteristic values of shear capacity are tabulated and these will be discussed in **Section 6.3.3**.

Results from SHEAR 2 show that in all cases under the present study, regardless of the location and the size of honeycombed zones, and the strength of normal and honeycombed concretes, the critical plastic yield line forms along the line that joins the edge of the load platen to the edge of the support platen. This conforms to the Nielsen and Braestrup(43) solution that for beams without shear reinforcement the lowest shear capacity is given by the above yield line.

A point to be noted from the above results is that, with regard to the plasticity method of analysis and in relation to the current study, only at certain locations, will a honeycombed zone theoretically affect the shear capacity. A honeycombed zone located in such a position that the line connecting the point of loading and the support passes through its zone will have its effect on the shear capacity of the beam. At other locations, since the line of critical yield line does not pass through the honeycombed zone, the shear capacity will be evaluated based on the normal concrete. Therefore, theoretically a honeycombed zone present not within the line of critical yield line will not affect the shear capacity of a beam. However, it will be seen later in the following section that the test results show that a honeycombed zone not within the line of critical yield line can affect the shear capacity of the beam. As a result of that a condition factor will be required to be applied to the existing plastic method, in order for it to give a safe prediction of the shear capacity of a honeycombed beam.

It should also be noted that as mentioned in **Sections 5.3.2** and **5.3.4** of **Chapter 5**, SHEAR 1 and SHEAR 2 were developed by ignoring the effect of the longitudinal reinforcement. Since the solution obtained using SHEAR 2 conforms to the solution proposed by Nielsen and Braestrup with regard to the yield line formation, thus the equation proposed by Nielsen and Braestrup can be directly used to evaluate the shear strength of honeycombed beams. With that solution the effect of the longitudinal reinforcement on the shear capacity of honeycombed beams can be taken into account.

6.3.2.3 Theoretical Shear Prediction of Honeycombed Beams Using Nielsen and Braestrup Equations

Equations (5.6) and (5.7), given in **Section 5.3.1** of **Chapter 5** can be used depending on the amount of the longitudinal reinforcement. All beams in the current study have a reinforcement degree, $\Phi \leq \nu/2$, so equation (5.6) should be used in which the longitudinal reinforcement contributes to the shear strength of the beam. The effectiveness factor was evaluated using equation (5.12) based on the strength of the normal concrete. Shear capacity was evaluated using the 'weighted average' strength which could be determined from SHEAR 2 for cases where the plastic mechanism passed through both the normal and honeycombing concrete. Note that the effectiveness factor and the 'weighted average' for each beam were obtained using SHEAR 2 and are given in **Tables 6.1(a)** and **(b)**.

The results of the prediction for all beams are tabulated in **Table 6.2**. Note that as in **Tables 6.1(a)** and **(b)**, both mean and characteristic shear strengths are tabulated and these will be discussed in **Section 6.3.3**.

Comparing the shear strengths obtained from SHEAR 2 (**Tables 6.1**) and Nielsen and Braestrup equation (**Tables 6.2**), the differences are not very significant. However, the latter gives a lower shear capacity compared to the former. This occurs because the contribution of the longitudinal reinforcement was taken into account. Since $\Phi \leq \nu/2$, the displacement of the yield line will be no longer vertical. It will be at an angle to the vertical axis. As a result of that, less work is done compared to if the displacement is in the vertical direction. Physically, what happens is that the amount of longitudinal reinforcement in the beam studied is insufficient to ensure that the plastic mechanism has a displacement in the vertical direction only. This causes the displacement to be at a greater angle to the yield line. As a result, the contribution of concrete to the shear capacity become less significant. This explains the lower shear capacity predicted by Nielsen's method compared to the predictions made by SHEAR 1 and SHEAR 2 .

A statistical analysis based on the mean values of shear strength prediction shows that the mean shear ratio of experiment to the theory, using SHEAR 2 for beams without a honeycombed zone is 1.01. The coefficient of variation is 0.141. Using Nielsen and Braestrup, the mean value of the shear ratio is 1.07, with the coefficient of variation of 0.129. The mean shear ratio obtained from SHEAR 2 is closer, but the coefficient of variation obtained from Nielsen and Braestrup is smaller. Henceforth, all the comparisons of test data will be made with the Nielsen and Braestrup's prediction.

6.3.2.4 Comparisons Between Theoretical Prediction and Experimental Results

The discussion in this section refers to the mean shear prediction values in **Table 6.2**. **Figure 6.8** are the plots of the shear capacity obtained from the experiment versus the mean predicted values for series 1A and 2A and series 1B and 2B. Greater variation between test data and the prediction in series 1A and 2A, and much smaller variation in series 1B and 2B are consistent with the experimental observations. As discussed in **Section 4.7.2** of **Chapter 4**, it was observed that a more significant reduction of shear capacity occurred in honeycombed beams in series 1A and 2A compared to series 1B and 2B. Note that series 1A and 2A were beams with a high strength of normal concrete and beams in series 1B and 2B were of a lower strength of normal concrete.

The predictions for each of the 2 control specimens in series 1A, beams 1A-1 and 1 control beam in series 2A, beam 2A-4 are very close to the test values, with ratios of shear capacity of the test to the theoretical are of 1.04, 1.15 and 1.02 respectively. Note that the shear span of the control beam in series 1A is 300 mm and for beam 2A-4, it is 580 mm. For beams in series 1B, 2 out of 3 specimens with a shear span of 300 mm have a predicted values less than the test results and the shear ratio of the test to the theoretical prediction are 0.92 and 0.90. Note that both specimens failed on the short shear span. The third specimen, which failed on the longer shear span has a ratio of 1.14. The control beam in series 2B, beam 2B-5 with a shear span of 580 mm has a ratio of 1.33.

As stated before, the mean ratio of the test to the prediction for all the control beams is 1.07 with coefficient of variation of 0.129. This coefficient of variation is within that given by Nielsen(45), which is 0.145. The value of 0.145 was obtained from test data of ordinary concrete beams which have been used to determine the expression for the effectiveness factor. The results obtained from the current test, although statistically not very significant due to the small number of test data, indicate that as far as the normal concrete beam is concerned the plastic analysis is able to give a good prediction of shear capacity and the current test results are within the scatter of results obtained when the theory is applied to other experimental data.

The theoretical prediction for beams with a honeycombed zone show that in general lower predictions are obtained in series 1A and 2A compared to beams in series 1B and 2B, in which closer predictions are obtained. However they seems to be consistent if test data are examined. For example, in general, honeycombed beams in series 1A and 2A failed at significantly lower loads compared to control beams. In series 1B and 2B, the reduction in the shear capacity of beams with a honeycombed zone was not that significant (refer to the discussion in **Section 4.7.2** of **Chapter 4**). In the following discussion, the location of a honeycombed zone in any particular specimen mentioned can be identified with the code location.

In series 1A, the theory seems not able to cope with beams with a honeycombed zone at all locations studied, except for beams with a honeycombed zone located at the middle top and middle bottom of the shear zone, beams 1A-5[TM] and 1A-8[BM]. The average ratios of the test to the prediction for the 2 specimens in each of these beams are 0.93 and

1.10 respectively. At other locations the ratio of the test to the theory is as low as 0.60 which occurs in specimen 1A-2(a)[MM], in which a honeycombed zone was located at the middle of shear zone. At other locations, the ratios are between 0.64, in specimen 1A-3(b)[MS], and 0.84, in specimen 1A-6(b)[TL]. The lowest ratios that occur in specimens 1A-3(a)[MS] and 1A-4(a)[ML] cannot be considered since they failed with the effect of torsion (as discussed in **Section 4.7.1** of **Chapter 4**).

In series 2A, beams with a shear span of 580 mm are well predicted by the theory. The ratio of the shear capacity of the test to the theoretical prediction for the control beam, beam 2A-4 and the average ratio of the honeycombed beams, beams 2A-5.1 and 2A-5.2, are 1.02 and 1.05 respectively. The predictions for other honeycombed beams are relatively close to the test results. The average shear ratio of the test to theoretical for beams 2A-1.1[MM], 2A-2[MS] and 2A-3[ML] are 0.84, 0.93 and 1.06 respectively.

It is clear that the plastic theory cannot be used to predict a beam with a void, as its prediction is very high compared to the test results. The ratios for the 2 specimens are 0.51 and 0.53. The mode of failure assumed by the plasticity method in this study is not valid and this agrees with what was observed in the test as described in **Section 4.7** of **Chapter 4**.

In series 1B and 2B, generally the prediction of the shear capacity of honeycombed beams by the plastic theory are very close. Except for beams 1B-6[BS] with the average shear ratio of two specimens of 0.76, the average shear ratio of other specimens with a shear span of 300 mm are in the range of 0.95 to 1.02. For honeycombed beams with a shear span of 580 mm, the predicted values are higher than the test. The ratio for the control beam (2B-5) is 1.33 and the average ratio of the two honeycombed beams is 1.31.

It is interesting to note that in cases where the theoretical prediction takes into account the presence of a honeycombed zone, the prediction is still far from the test value for some of the beams. This occurs especially in beams in series 1A and 2A. Refer to the mean shear ratios in beams 1A-2[MM], 1A-6[TL], 1A-7[BS], 2A-1.1[MM] and 2A-

7[MM]. In series 1B and 2B this can be seen in beam 1B-6. In those beams, where the honeycombed zone was located along the line of the critical yield line, the 'weighted average' takes into account the strength of the honeycombed concrete. This indicates that, although the honeycombed strength is taken into account, the theory is still insufficient and a condition factor is required.

Also interesting to note is the fact that for honeycombed beams where the theoretical prediction takes no account of the presence of the honeycombing, the actual test results show that the honeycombing does affect the shear capacity of the beam. This effect can be seen in beams in series 1A and 2A, beams 1A-3[MS], 1A-4[ML], 2A-2[MS] and 2A-3[ML], and also in beam 1B-3[MS] in series 1B. This indicates that the use of the existing plastic theory to evaluate the shear capacity of honeycombed beams, regardless of the location of the honeycombed zone, needs a reduction factor which can be considered as a condition factor reflecting the poor quality of a honeycombed beam. This will be discussed in the following section.

6.3.2.5 Discussion of Results and the Proposal of Condition Factors

From the discussion in **Section 6.3.2.4**, it is clear that from the results of prediction of the honeycombed beams in series 1A and 2A, the plasticity theory needs a modification before it can be a reliable assessment tool for shear in a honeycombed beam. In series 1B and 2B, the results show that not all of the honeycombed beams require modifications. A more clear picture of the shear capacity ratio of the test to the prediction for each location of the honeycombed zone is presented in **Figure 6.9**. The values given are the average ratio at each location, calculated separately for series 1A and 2A and for series 1B and 2B. Also shown in the figure are values if all the series are combined. Note that the calculation ignores the test data for honeycombed beams which failed with the effect of torsion and also beams with a precast honeycombed zone and beams with a void.

From the figure, if honeycombed beams in series 1A and 2A alone are considered, only beams with a honeycombed zone at the middle of the bottom of the shear zone do not require modification to the present plastic solution. For series 1B and 2B, 2 out of 5 locations of the honeycombed zone studied do not require modification. Those locations are at the central section of the shear zone and at the section along the neutral axis of the beam near the point of loading. If all beams are considered together, all honeycombed beams require a modification to the present plastic solution in order for the solution to provide a safe assessment.

A statistical analysis is carried out on the test data in order to determine the condition factor in a more systematic manner. Although the number of test data is statistically not very significant, this is the only sensible approach. This will provide a more general solution to the problem under study.

Two approaches can be considered. The first approach is to consider honeycombed beams in series 1A and 2A as one separate group and in series 1B and 2B as another group. A condition factor is then determined separately for each group. The second approach is to consider all honeycombed beams in all the series and a single condition factor is determined for all the honeycombed beams. Both approaches will be considered here.

Consider now beams in series 1A and 2A only. Note that in this statistical analysis, honeycombed beams which failed with the effect of torsion are neglected. Beams with a precast honeycombed zone and beams with a void are also neglected. Hence, for series 1A and 2A, specimens 1A-3(a), 1A-4(a), 2A-1.2(a) and (b), and 2A-1.3(a) and (b) are ignored. For the rest of the specimens which consist of 21 test data, the mean shear ratio of the test to the theoretical prediction based on the mean values is 0.905. The standard deviation is 0.193 and the coefficient of variation is 0.213. The mean ratio less than unity implies that for all cases of honeycombed beams considered in series 1A and 2A, the prediction by the present solution is inadequate. The variation of the test results obtained in the current study is greater than that given by Nielsen which is 0.145. This indicates that beams with a honeycombed zone have a greater variation in shear capacity than the variation in the ordinary concrete beams. Therefore the present plastic solution which has been developed for the ordinary concrete beams need to be modified in order to be applicable to honeycombed beams.

It seems that the sensible way to modify the present plastic solution is to modify the effectiveness factor. Three approaches can be examined. The first approach is to bring up the mean shear ratio of 0.905 to unity. In order to do that, it must be multiplied by a condition factor of 1.1 or an increase of 10%. In order to increase the shear ratio, the predicted shear must be reduced and this can be achieved by reducing the effectiveness factor. For the condition factor to be statistically valid and safe, the values of the characteristic shear ratio must be examined. The existing characteristic effectiveness factor is divided by the condition factor determined above and this will reduce the characteristic shear prediction. This will lead to an increase in the characteristic shear ratio. If by increasing the characteristic shear ratio results in only 5% of the characteristic shear ratio of the test data below unity, the 1.1 condition factor can be accepted.

The characteristic ratios are given in **Table 6.2**. They are obtained from the characteristic shear prediction. Converting the mean effectiveness factor to characteristic values, the corresponding characteristic shear capacity prediction can be found. The experimental shear capacity is divided by the characteristic shear capacity to give the characteristic shear ratio. Using the coefficient of variation of 0.145 and the characteristic value as defined in BS 8110, the conversion from mean to characteristic value is carried out by multiplying equation (5.12) with a factor of 0.762.

From 21 test data considered in series 1A and 2A, it can only allow one ratio to be below unity. After reducing the characteristic effectiveness factor by 1.1, it is found that the characteristic shear ratio of 6 test data are below unity, thus the condition factor of 1.1 cannot be accepted. The new shear ratio based on the condition factor of 1.1 is tabulated in **Table 6.3**, and those ratios below unity are shown in italic.

Another approach is to allow a greater variation in the characteristic effectiveness factor. Instead of using the coefficient of variation of 0.145 to convert the mean to characteristic effectiveness factor, the value obtained from the test of 0.213 is used. This leads to a lower characteristic effectiveness factor, by a condition factor of 1.17. However, it is found that there are 4 shear ratios below unity. Refer to **Table 6.3**. Thus, the condition factor determined cannot be accepted.

The only option now is to further reduce the characteristic effectiveness factor. This can be done by a trial condition factor. It is found that a condition factor of 1.34 applied to the characteristic effectiveness factor results in only one shear ratio below unity. Thus, the condition factor derived is acceptable.

For series 1B and 2B, from 15 test data considered, the mean shear ratio of the experiment to the theory based on the mean effectiveness factor is 0.995. The standard deviation is 0.169, and the coefficient of variation of 0.170. The mean ratio is very close to unity, but the coefficient of variation is greater than that for the ordinary concrete beams. Thus a condition factor is required. As for the above, three approaches are examined. However for a small number of data, which consist of only 15 test data, 5% of data means not even one ratio from the data can be below unity. The first two approaches as above, with a condition factor of 1.01 and 1.06 respectively, results in one characteristic ratio below unity. The trial approach, with a condition factor of 1.08, all the characteristic ratios are greater than unity. Thus, for series 1B and 2B, the condition factor of 1.08 can be accepted.

If all the test data in series 1A and 2A and 1B and 2B are combined, the mean shear ratio is 0.942. The standard deviation is 0.189 and the coefficient of variation is 0.201. For 36 number of test data, it can allow 2, a round-off of 1.8, shear ratios to be below unity. The first approach which brings up the mean ratio to 1.0, results in 8 test data below unity. If a greater variation is allowed, with a condition factor of 1.14, it is found that 4 ratios of test data are below unity. Again, only by a trial approach, with a condition factor of 1.33, results in 2 ratios from the test data being below unity.

With a small number of test data, it is quite difficult to draw a conclusive and reliable condition factor. The great variation of results notably in series 1A and 2A and different pattern of variation in test data between series 1A and 2A and series 1B and 2B, makes the task even more difficult. From the current study, two options can be considered. The first option is to use a separate condition factor for each series. From the above, the condition factors for series 1A and 2A and series 1B and 2B of 1.34 and 1.08 respectively can be proposed. They are about 25.4% and 7.4% reductions from the existing plastic solution respectively. The second option is to use a single condition factor of 1.33 or a reduction of about 24.8%.

It is clear that a single condition factor is more conservative for some of the honeycombed beams studied, since it is derived from more data. However it is probably unnecessarily far too conservative when applied to series 1B and 2B beams. Note that there is almost no difference between the condition factor proposed for the whole series and for series 1A and 2A. This indicates that series 1A and 2A beams cause the high condition factor for the whole range of honeycombed beams. From the test data for beams in series 1B and 2B, the prediction by the existing plastic solution is very close. Only one beam with a honeycombed zone at the bottom section near the support causes the coefficient of variation to stretch to 0.170. Otherwise the coefficient of variation is significantly smaller. Therefore it is quite safe, although the number of data is small, for beams in series 1B and 2B to have a separate condition factor. Note that the condition factors derived are applicable for beams with a honeycombed zone at all locations within the high shear zone, and also for different shear span ratios, a/d of 2.0 and 3.5. Note that the shear span ratios are based on the shear span measured centre to centre of 350 and 630 mm respectively.

It seems however that a single reduction factor determined from all four series of tests will be more reliable in view that it is based on more data. It is also more conservative. Therefore a reduction factor of 1.33 is proposed to be used in the assessment of a honeycombed beam.

6.3.3 The Prediction of Shear Capacity of Honeycombed Beams With Shear Reinforcement

The evaluation of the values of Φ and ψ show that in all cases both are less than $\nu \leq 1/2$. The calculation of the angle β using equation (5.18) shows that for all cases, tan β is smaller than *h/a*. As a result of that the shear strength of the beams must be evaluated using equation (5.19). Equations (5.18) and (5.19) are given in **Section 5.3.5** of **Chapter 5**.

For beams with a honeycombed zone, as adopted in beams without shear reinforcement, the effectiveness factor is evaluated using equation (5.20) based on the strength of the normal concrete. The 'weighted average' strength is used in equation (5.19). All other values as required in equations (5.19) and (5.20) are taken from the experimental data and are given in **Section 6.3.1**. The results of the predicted values and the comparison with the test data for beams in series 2A and 2B are presented in **Table 6.4**.

Generally the predicted shear capacities are close to the experimental results for both control and honeycombed beams in both series. Note that the normal concrete in series 2A was stronger. Although specimens 2A-8(b) and 2B-3(b), the control beams and specimen 2A-9(b), a honeycombed beam, failed in the long shear span, these is not reflected in the ratios obtained. Except for specimen 2B-4(b) with a ratio of 0.82, the other honeycombed specimens are safely predicted, with ratios between 1.03 and 1.27. This indicates that the proposed approach in which the 'weighted average' strength is used in calculating the shear strength and the strength of normal concrete is used in evaluating the effectiveness factor can be adopted for a honeycombed beam with shear reinforcement. The ratio found in specimen 2B-4(b) may indicate the typical variation that always occur in shear tests. However it may also indicate the significant effect that a honeycombed zone can cause on a beam. Note that it was observed in the test that the diagonal crack formed independently in specimen 2B-4(b).

From all the honeycombed specimens the average ratio is 1.09. This suggests that the proposed approach is safe in assessing a honeycombed beam with shear reinforcement. However, it should be noted that this conclusion is based on only 4 numbers of test data. Further tests are required to thoroughly investigate the problem, for example if a honeycombed zone occurs at other locations than at the centre of shear region.

6.4 COMPARISONS OF THE EXPERIMENTAL RESULTS WITH THE PREDICTIONS OF BS 8110 AND BD 44/95

BS 8110(19), a design code for concrete structures and BD 44/95(1), an assessment code for concrete bridges are used to predict the shear capacity of honeycombed beams. The expressions in both documents essentially predict the diagonal cracking shear of a beam. In this study, the predictions given by BS 8110 and BD 44/95 are compared with the diagonal cracking shear and also with the ultimate shear obtained from tests. In using the expressions from both documents the partial factor of safety is set to unity.

In normal circumstances, it is quite normal practice for an assessing engineer to use the lowest strength of concrete obtained from a site investigation in evaluating the structural capacity of a beam. In this study, the predictions of shear capacity of honeycombed beams are made using both the strength of the normal concrete and the strength of the honeycombed concrete. Therefore, comparison can be made with the actual capacity obtained from tests and this can indicate how conservative or critical the approach is.

6.4.1 Shear Assessment Formulae in BS 8110 and BD 44/95

From BS 8110, the expression to predict a diagonal cracking shear, V_c in concrete beam is given as,

$$V_c = 0.79.\sqrt[4]{\frac{400}{d}}.\sqrt[3]{\frac{100A_s}{bd}}.bd \tag{6.1}$$

where,

d	=	effective depth of beam
A_s	=	the area of longitudinal steel

According to BS 8110, the above expression is valid for concrete with a strength of 25 N/mm^2. If the concrete strength is greater than 25 N/mm^2 , equation (6.1) must be multiplied by,

$$\sqrt[3]{\frac{f_{cu}}{25}} \tag{6.2}$$

where,

f_{cu}	=	the cube strength of concrete

In the current study, all the normal concretes have a strength greater than 25 N/mm^2, thus in calculating the diagonal cracking shear using the normal concrete strength, equation (6.1) must be multiplied by equation (6.2).

Also in all cases of this study, the strength of the honeycombed concrete is below 25 N/mm^2. In order to be consistent, the evaluation of the diagonal cracking shear of a honeycombed beam using the strength of the honeycombed concrete by equation (6.1), must also be multiplied with equation (6.2).

From BD 44/95, the diagonal cracking shear can be predicted by,

$$V_c = 0.24.\sqrt[4]{\frac{500}{d}}.\sqrt[3]{\frac{100A_s}{bd}}.\sqrt[3]{f_{cu}}.bd \tag{6.3}$$

For beams with shear reinforcement, BS 8110 and BD 44/95 give the same truss analogy expression to evaluate the shear capacity provided by the shear reinforcement:

$$V_s = A_{sv}.f_{yv}.\frac{d}{s_v} \tag{6.4}$$

In the following prediction, the effective depth of the beam, *d*, is taken as 179 mm. This is based on 15 mm concrete cover used in the experiment and 12 mm longitudinal steel bar. The area of the longitudinal steel, A_s is 226 mm^2. For beams with shear reinforcement, the area, A_{sv} is 14 mm^2 and the spacing, s_v is 100 mm. Unlike in the plastic method, the shear span dimension mentioned in this section will refer to the distance measured from the centre-line of the support to the centre-line of the loading point.

Note that it will be found in the following discussion that there are variations in the results between nominally identical beams and specimens. In many situations the variations are due to the variation observed in the shear behaviour of the beams in the tests as discussed in **Chapter 4**. For example identical specimens developed different modes of diagonal cracking which lead to different diagonal cracking loads. This eventually might resulted in the different mode of behaviour and load at the ultimate stage. In discussing the comparison, if any particular variation is due to the variation observed in the test, further explanation needs to be referred to the appropriate section of **Chapter 4**.

6.4.2 Beams Without Shear Reinforcement

The diagonal cracking shear calculated for all beams without shear reinforcement using BS 8110 and BD 44/95 and the summary of results are tabulated in **Table 6.5(a)** for beams in series 1A and 2A and **Table 6.5(b)** for series 1B and 2B beams. Presented in the tables are ratios of the experimental diagonal cracking shear to the diagonal cracking shear predicted by BS 8110 and BD 44/95 using both the strength of the normal concrete and the strength of the honeycombed concrete. The ratios of the ultimate shear obtained from test to the predicted shear by both expressions are also included in the tables.

All beams tested in series 1A are included. For series 2A, except for beams with a precast honeycombed zone and beams with a void, all other beams are included for comparisons. The purpose of the inclusion of beam 2A-6, where the honeycombed zone simulates a construction joint, is to compare the results when beams in such a condition are predicted by an ordinary shear assessment method. For series 1B and 2B, all beams are included. As in series 2A, the beam with a construction joint is also examined.

6.4.2.1 Diagonal Cracking Shear

The following discussions are carried out according to the method of prediction: BS 8110 and BD 44/95. The predictions obtained are compared with the diagonal cracking shear found from tests.

(a) BS 8110

For beams without a honeycombed zone, it seems that BS 8110 provides a conservative prediction for beams with a shear span of 350 mm in all series. The average ratio of the test to the predicted diagonal cracking shear in series 1A and in series 1B are 1.25 and

1.21 respectively. However, for beams with a shear span of 630 mm, the prediction of the diagonal cracking shear for the control beams in both series 2A and 2B are lower than the actual. The ratio of the test to the predicted value for beam 2A-4 of series 2A and beam 2B-5 of series 2B are 0.89 and 0.86 respectively. Results show that the difference in the normal concrete strength between series 1A and 2A and series 1B and 2B, does not seem to significantly affect the predictions of the diagonal cracking shear

Using the strength of the normal concrete in predicting the diagonal cracking shear results in some of the honeycombed beams in series 1A and 2A being under-predicted by BS 8110. The locations at which a honeycombed zone need to be treated with extra caution are at the central area of the shear zone, beams 1A-2, 2A-1.1, 2A-7, along the longitudinal axis of the beam but near to the loading point beam 1A-4[ML], and at the bottom middle of the shear zone, beam 1A-8[BM]. The shear span of those beams is 350 mm.

The most critical is for beam 1A-8, with an average ratio of the test to the predicted shear of 0.7. As mentioned in **Section 4.6** of **Chapter 4**, a honeycombed zone in beam 1A-8 accelerates the formation of a flexural crack which subsequently triggers an early formation of a diagonal crack. The average ratio of the test to the predicted shear for beams with a shear span of 350 mm and with a honeycombed zone at the central area, is 0.89. Note that this average ratio includes a beam with a honeycombed zone of 90 x 90 mm, beam 2A-7. If the specimen with a high ratio of shear, specimen 2A-1.1(a) with the ratio of 1.23, is ignored the average ratio is 0.83. For beam 1A-4[ML], with a honeycombed zone along the neutral axis and near to the loading point, the average ratio from the two specimens is 0.89. For the other honeycombed beams except the beam with a construction joint, their ratios indicate that the BS 8110 can be a sufficient assessment tool even though the strength of normal concrete is used in the assessment. The average ratio of those beams are between 0.99 to 1.13.

Note the difference in the ratio between beam 1A-4 and beam 2A-3. BS 8110 safely predicts the diagonal cracking shear of beam 2A-3, but not for beam 1A-4. Both are identical with regard to the location of the honeycombed zone, and have about the same level of strength of normal concrete, but have a different strength of honeycombed concrete. As mentioned and discussed in **Chapter 4**, this is the example of the sort of variation which can be expected in shear and must be carefully observed.

For honeycombed beams with a shear span of 350 mm in series 1B and 2B, in general it is found that assessing the diagonal cracking shear using the strength of normal concrete will result in a safe prediction. This applies to all honeycombed beams. However a little attention is required when a honeycombed zone is located at the central area and along the neutral axis of the beam, near to the loading point, since as shown in **Table 6.5(b)**, specimen 1B-2(a)[MM] and all specimens of beam 1B-4[ML] have a ratio of the test to the predicted of 0.90 and 0.99 respectively.

For beams with a honeycombed zone at the central section but with a shear span of 630 mm, since BS 8110 is unable to adequately predict the diagonal cracking shear of the control beam, it is thus not expected to give a safe prediction for a honeycombed beam. Using the normal concrete strength, the average ratio for beam 2A-5 and beam 2B-6 are 0.78 and 0.92 respectively.

It is clear that the diagonal cracking shear of a beam with a construction joint cannot be assessed using BS 8110. The actual diagonal cracking shear for beams in series 2A, beam 2A-6 and in series 2B, beam 2B-1 are only 83% and 40% respectively of the predicted shear when the strength of the normal concrete is used.

If the diagonal cracking shears of honeycombed beams are assessed using the strength of the honeycombed concrete, it is found that for many honeycombed beams, especially in series 1B and 2B the results are very conservative. In some circumstances this may result in uneconomic assessment and rehabilitation.

In series 1A the ratios are found to be as high as 1.73 which occurred in specimen 1A-5(a)[TM], with a honeycombed zone at the middle top of the shear zone. The only specimens which are unsafely predicted are specimen 1A-2(b), with a ratio of 0.89 and 1A-8 with a ratio of 0.9. The other specimens in series 1A are safely predicted with ratios between 1.06 to 1.44. In series 2A, all beams including beams with a shear span of 630 mm and beams with a joint are well predicted by BS 8110 using the strength of the honeycombed concrete. The ratios are between 1.13 and 1.96.

In series 1B and 2B, all beams examined, including beams with a shear span of 630 mm are safely predicted by BS 8110. The only exception is the beam with a construction joint in series 2B. Except for beam 2B-1, beam with a joint, with a ratio of 0.51, the other honeycombed beams have ratios between 1.19, which occurs in beams with a shear span of 630 mm, to as high as 2.04.

This study shows that except for a honeycombed zone in the centre and at the middle bottom of the shear zone, and beam with a joint, all other locations of honeycombed zone are well predicted when the strength of the honeycombed concrete is used. Note that the lower the strength of honeycombed concrete, the more conservative an assessment is produced. This, for example is shown by beams 2A-1.1[MM], 2A-2[MS] and 2A-3[ML]. This occurs because the predicted shear depends on the values of the strength of concrete, while the test results show that in general the strength of honeycombed beam did not have a significant effect on the shear capacity of the beam. This indicates that in practice it may happens that a value of lower strength of concrete found from an isolated spot and used in evaluating the shear capacity leads to unnecessarily very conservative results. Note also that the assessment formulae do not account for the size of the honeycombed zone. With regard to this, it may happens in the real assessment that a low strength of concrete found from a spot which is insignificant in size as far as the shear capacity is concerned, leads to unrealistic and highly conservative assessment results.

There are however, honeycombed beams which need to be treated carefully even when the lower strength is used to predict their diagonal cracking shear, as shown in **Table 6.5(a)**, for beam 1A-2[MM] and the more critical beam 1A-8[BM]. However as for beam

1A-8, it will later be shown that the unsafe prediction of diagonal cracking shear will not be very significant as at the ultimate stage, the beam behaviour is safely predicted.

(b) BD 44/95

The more conservative shear assessment produced by BD 44/95, is due to the fact that BD 44/95 is formulated based on the lower bound test data. All variations including variation in concrete quality due to for example poor compaction are taken into account. In contrast, BS 8110 is apparently based on the mean value of test data, thus some of the worst variations that occur are left out. However as can be seen below the prediction using BD 44/95 is still insufficient for some of the honeycombed beams.

With a honeycombed zone at certain locations in the high shear region, using the strength of the normal concrete to predict the cracking shear load can still result in unsafe assessment. These can be seen from the ratios in beams 1A-2[MM], 1A-4(b)[ML], 1A-5(b)[TM], 1A-8[BM], 2A-1.1(b)[MM], 2A-5[MM], and 2A-7(a)[MM]. The ratios of test to the predicted values in those beams are between 0.75 and 0.95. In series 1B and 2B, except for specimen 1B-2(a)[MM], and specimen 2B-5 and 2B-6.1, beams with a shear span of 630 mm, and also beams with a construction joint, all beams are safely predicted by BD 44/95 using the strength of the normal concrete.

Using the strength of the honeycombed concrete leads to a conservative assessment. There are however honeycombed beams, with a honeycombed zone at the centre of shear region and also when the honeycombed zone at the bottom middle of the shear zone, which the prediction by BD 44/95 is unsafe. This can be seen in specimens 1A-2(b)[MM] and 1A-8[BM], with ratios of 0.95 and 0.96 respectively. In series 2B only beams with a construction joint cannot be adequately predicted by BD 44/95 using the strength of the honeycombed concrete.

6.4.2.2 Ultimate Shear

The more significant parameter in assessment is the ultimate shear. It will be shown in the following discussion that, in some cases, the diagonal cracking shear does not carry any significance to the safety of the beam. This occurs because the formation of diagonal cracking does not necessarily in every case lead to a sudden failure. In fact in many cases, the honeycombed beams studied can possess a much higher load before it fails. In the following, the ultimate loads obtained from tests are compared to the shear predicted by BS 8110 and BD 44/95. The evaluations will be based on both normal and honeycombed concrete strengths.

(a) BS 8110

Examining the values of the ratio of the ultimate shear obtained from the test to the predicted shear using the strength of the normal concrete, it is clear that except for beams with a construction joint and beams with a shear span of 630 mm in series 2A, beams 2A-5.1 and 2A-5.2, of which the average ratio is 0.94, all the other honeycombed beams will be safe if BS 8110 is used in the ultimate shear assessment. The lowest ratio occurs in beam 1A-2[MM] with an average ratio of 1.06. As mentioned in **Section 4.7** of **Chapter 4**, the phenomenon shown by beams 2A-5 is typical for a longer shear span beam in which the formation of a diagonal crack will be immediately followed by ultimate failure.

Note the ratio for beam 1A-8. While its diagonal cracking shear is under-predicted, on average, the actual ultimate shear of the beam is 2.14 times greater than the BS 8110 prediction. This indicates that the effect of the honeycombed zone in accelerating the diagonal crack will not necessarily lead to an early failure. Note also that even beams which failed with the effect of torsion; specimens 1A-3(a) and 1A-4(a), can be safely assessed by BS 8110.

The use of the honeycombed strength results in a far more conservative assessment. Honeycombed beams with a shear span of 630 mm in series 2A are now safe with the average ratio of 1.5. From the overall results, one can imagine how unrealistic the assessment results can possibly be. In practice it is always the case that regardless of the location and the size of the low strength zone, the assessment is carried out based on the lowest strength found from the investigation. Adopting such an approach, the actual shear capacity as shown in this study can be up to 3 times the predicted value.

However, for the beam with a joint in series 2B, even though the strength of the honeycombed concrete is used, the BS 8110 is still unable to provide a safe prediction.

(b) BD 44/95

Generally predicting the ultimate shear of honeycombed beams using BD 44/95 results in a conservative assessment. The exceptional is for the honeycombed beam with a shear span of 630 mm. This is shown by specimen 2A-5.2. When the strength of normal concrete is used the ratio is 0.91. For beams with a construction joint, the prediction of the ultimate shear failures using BD 44/95 are unsafe. These are shown by beam 2B-1 in series 2B, with an average ratio of 0.69 and specimens 2A-6 in series 2A with an average ratio of 0.95.

For other beams, using the strength of the normal concrete, the lowest ratio of the test ultimate shear to the predicted is 1.09 occurs in specimen 2A-5.1. This is a honeycombed beam with a shear span of 630 mm. For honeycombed beams with a shear span of 350 mm, the lowest ratio is 1.08, in specimen 1A-3(a)[MS]. This specimen failed with the effect of torsion. The prediction can be as high as 2.58 times the test value as occur in specimen 1A-8(b)[BM].

Using the strength of the honeycombed concrete, brings the predictions generally far more conservative. The lowest ratio which occurs in specimen 1A-2(a)[MM] is 1.39. The highest ratio is as high as 3.62 occurs in specimen 2A-3(b)[ML]. Beams with a construction joint in series 2A are also safely predicted. However in series 2B, the prediction is unsafe.

6.4.3 Beams With Shear Reinforcement

The results are tabulated in **Table 6.6**. Presented in the table are both the ratios of the cracking shear and the ratio of the ultimate shear. Note that only the theoretical ultimate shears are tabulated. The theoretical cracking shear can be obtained by subtracting the respective ultimate shear with a constant value of 13.2 kN, which is obtained from equation (6.4). The value is constant because for all beams the shear reinforcement was the same.

6.4.3.1 Diagonal Cracking Shear

With regard to the ratio of the cracking shear in the control beams for beams in series 2A, using BS 8110, the average of the 2 specimens is 1.18. Note that for beams without shear reinforcement in series 1A, the average ratio of the cracking shear is 1.25. The values are quite close to each other, indicating that the shear reinforcement is not that significant in preventing the formation of the diagonal cracking. For series 1B, the average ratio is 1.21, compared to the average ratio for beams in series 2B of 1.47. The values are not very far from each other.

Using either the strength of the normal concrete or the strength of the honeycombed, BS 8110 predicted a safe diagonal cracking for all honeycombed specimens with the exception of specimen 2B-4(b) with a ratio of 0.8. As expected, BD 44/95 generally predicted more conservative values, with the exception of beam 2B-4(b), with a ratio of 0.84. Although BS 8110 and BD 44/95 produce unsafe predictions for specimen 2B-4(b), for beams with shear reinforcement it is more appropriate to compare the ratio of ultimate load, in which the contribution from the shear reinforcement is taken into account.

6.4.3.2 Ultimate Load

Using either the strength of the normal concrete or the strength of the honeycombed, both BS 8110 and BD 44/95 give a safe prediction for beams with and without a honeycombed zone. As usual, a more conservative prediction is given by using the honeycombed concrete strength. With BD 44/95, the use of the honeycombed concrete strength provides further conservatism to the results. For series 2A specimens the average ratio is 2.09. A lower ratio is given by specimens in series 2B, with a value of 1.89. As mentioned earlier, the relatively low result shown by specimen 2B-4(b), could be attributed to the effect caused by the honeycombed zone.

6.5 COMPARISONS OF EXPERIMENTAL RESULTS TO THE PLASTIC PREDICTION OF SHEAR CAPACITY IN BEAMS WITH A CONSTRUCTION JOINT

Based on the experimental results of 4 specimens, a comparison is made with the theoretical prediction. Those specimens are 2A-6(a), 2A-6(b), 2B-1(a) and 2B-1(b). The theoretical prediction is evaluated using equations (5.6) or (5.7) depending on the amount of longitudinal reinforcement. Only the ultimate shear can be evaluated and compared.

The effectiveness factor, ν, is taken as 0.45, and this has been explained in **Section 5.4** of **Chapter 5**. The critical yield line is as shown in **Figure 5.5** of **Chapter 5**. Based on the angle of inclination of the construction joint in the test, which is 45°, and the thickness of the joint of 30 mm, $cot\ \beta$ can be worked out and is found to be 1.21. Evaluating the value of Φ, based on the area of longitudinal steel, A_s of 226 mm^2, the yield stress of 497 N/mm^2, and the breadth and depth, b and h of 100 mm and 200 mm respectively, and the honeycombed cylinder strength for series 2A and 2B of 12.14 N/mm^2 and 10.16 N/mm^2 respectively, the values of Φ, for series 2A and 2B are 0.463 and 0.553 respectively. They are greater than $\nu/2$, based on ν of 0.45. Hence equation (5.7) is to be used to evaluate the shear capacity of the joint.

The predictions of the ultimate shear of all beams tested in the current study are tabulated in **Table 6.7**. The ratios of the test data to the predicted value are also calculated. As expected the results vary between each specimen. For each specimen in each series, their ratios are quite consistent. In series 2A, the prediction is very close to the test results. However, in series 2B, higher prediction is made by the theory. Note that it was observed in the test that the failure modes were as assumed by the plastic method.

There are uncertainties with regard to the effectiveness factor which is taken as 0.45. However the closeness of the predicted results to the test data indicate that the value of 0.45 is not far away. Probably a better value can be obtained if more tests are carried out to study this problem.

6.6 CONCLUSIONS

A general conclusion is presented below. A more detailed conclusion will be presented later in **Chapter 7**.

It has been shown that the elastic stress development within the area of high shear when a honeycombed zone is present cannot be predicted by the linear elastic finite element analysis. This proves that the stress distribution within the shear zone is highly indeterminate, and it requires a more detailed model for materials, and non-linear analysis to consider it further. The results of the finite element analysis indicate that a truly rational analysis to predict a shear behaviour is still to be reached. It just confirms the problems that have been encountered in numerous shear research works. The presence of a honeycombed zone adding more problems to be solved.

The plastic analysis with the proposed condition factors can provide an alternative solution in predicting the shear capacity of honeycombed beams without shear reinforcement. However as noted, more tests are required for a more reliable condition factor to be proposed. The significant difference in the condition factor derived from series 1A and 2A and series 1B and 2B is an indicator that variation in shear is inevitable.

For honeycombed beams with shear reinforcement, generally the existing plastic analysis, with a 'weighted average' strength approach can be safely used without a need of introducing a condition factor. However the number of test data is insufficient in order to give a conclusive general guidance.

The use of BS 8110 and BD 44/95 in the shear assessment of honeycombed beams with and without shear reinforcement needs to be carried out carefully. When the strength of the normal concrete is used in evaluating the shear capacity, the current study shows that the strength of a beam with a honeycombed zone at the central zone of a shear span of a beam could be under-predicted. This also can happen even when the strength of the honeycombed concrete is used.

However, the results of this study also indicate that, without a proper knowledge of the magnitude of the effect of a honeycombed zone on the shear capacity of a beam, the use of BS 8110 and BD 44/95, can lead to a highly conservative assessment result. The degree of conservatism can be further increased if the strength of the weakest concrete in a beam is used without giving any consideration to the location where it is found and the size of the low strength concrete zone. From this study, the results can provide a general guidance which can be adopted in dealing with the assessment of shear in a short shear span beam.

BD 44/95 tends to provide a more conservative assessment compared to BS 8110. This is understandable as the former is formulated on the basis of a lower bound to test data rather than the mean and consequently it takes more account of structural deficiencies. Evaluating the shear capacity of honeycombed beams with BD 44/95 and using the strength of the honeycombed concrete may result in an unrealistic and uneconomic decision for a particular structure.

It is also found that the use of ultimate shear rather than the diagonal cracking shear is more appropriate in the assessment. From the current study, it shows that the formation of diagonal cracking is not necessarily followed by an immediate shear failure. The clear example is exhibited by beam 1A-8, with a honeycombed zone at the middle bottom of the span. For all the honeycombed beams in this study including beams with a shear span ratio of 3.5, BS 8110 and BD 44/95 are reliable to be used to assess their ultimate shear capacity.

For a beam with a construction joint, the ordinary assessment method is insufficient to predict its shear capacity. It needs to be treated as a special case.

Remark	Beam	f_{cu} (N/mm^2)	f_{cuh} (N/mm^2)	Average f_{cav} (N/mm^2)	Ultimate load (kN)	Ult. shear experiment (kN)	Ultimate shear theory (kN)		Shear ratio experiment/theory		Effectiveness factor	
							mean	charac	mean	charac	mean	charac
control	1A-1(a)	45.6		34.7	68.2	46.5	49.26	37.54	0.94	1.24	0.469	0.357
control	1A-1(b)	45.6		34.7	75.7	51.6	49.26	37.54	1.05	1.38	0.469	0.357
	1A-2(a)	45.6	23.5	30.8	35.0	23.9	43.88	33.44	0.54	0.71	0.469	0.357
	1A-2(b)	45.6	23.5	30.8	36.3	24.8	43.88	33.44	0.56	0.74	0.469	0.357
torsion	1A-3(a)	53.8	22.6	40.9	36.0	24.5	53.50	40.77	0.46	0.60	0.432	0.329
	1A-3(b)	53.8	22.6	40.9	44.0	30.0	53.50	40.77	0.56	0.74	0.432	0.329
torsion	1A-4(a)	53.8	22.6	40.9	38.4	26.2	53.50	40.77	0.49	0.64	0.432	0.329
	1A-4(b)	53.8	22.6	40.9	57.0	38.9	53.50	40.77	0.73	0.95	0.432	0.329
	1A-5(a)	48.3	21.5	36.7	50.0	34.1	50.69	38.63	0.67	0.88	0.456	0.347
	1A-5(b)	48.3	21.5	36.7	75.0	51.1	50.69	38.63	1.01	1.32	0.456	0.347
	1A-6(a)	48.3	21.5	33.7	51.0	34.8	46.55	35.47	0.75	0.98	0.456	0.347
	1A-6(b)	48.3	21.5	33.7	51.8	35.3	46.55	35.47	0.76	1.00	0.456	0.347
	1A-7(b)	54.2	26.2	36.9	44.0	30.0	48.09	36.64	0.62	0.82	0.431	0.328
	1A-8(a)	54.2	26.2	41.2	65.5	44.7	53.70	40.92	0.83	1.09	0.431	0.328
	1A-8(b)	54.2	26.2	41.2	86.0	58.6	53.70	40.92	1.09	1.43	0.431	0.328
insitu	2A-1.1(a)	42.3	10.4	27.2	50.9	34.7	40.07	30.53	0.87	1.14	0.487	0.371
insitu	2A-1.1(b)	42.3	10.4	27.2	40.5	27.6	40.07	30.53	0.69	0.90	0.487	0.371
precast	2A-1.2(a)	42.3	11.6	27.3	66.4	45.3	40.32	30.72	1.12	1.47	0.487	0.371
precast	2A-1.2(b)	42.3	11.6	27.3	71.0	48.4	40.32	30.72	1.20	1.58	0.487	0.371
void	2A-1.3(a)	42.3	-	25.7	26.1	17.8	37.95	28.92	0.47	0.62	0.487	0.371
void	2A-1.3(b)	42.3	-	25.7	27.2	18.5	37.95	28.92	0.49	0.64	0.487	0.371
	2A-2(a)	54.0	14.1	41.0	73.7	50.3	53.60	40.84	0.94	1.23	0.431	0.328
	2A-2(b)	54.0	14.1	41.0	54.5	37.2	53.60	40.84	0.69	0.91	0.431	0.328
	2A-3(a)	54.0	14.1	41.0	68.1	46.4	53.60	40.84	0.87	1.14	0.431	0.328
	2A-3(b)	54.0	14.1	41.0	77.1	52.6	53.60	40.84	0.98	1.29	0.431	0.328
a/d=3.5	2A-4	52.6	-	40.0	35.6	23.8	24.66	18.79	0.96	1.27	0.368	0.280
a/d=3.5	2A-5.1	52.6	13.0	36.7	36.8	24.6	22.65	17.26	1.09	1.43	0.368	0.280
a/d=3.5	2A-5.2	52.6	13.0	36.7	30.5	20.4	22.65	17.26	0.90	1.18	0.368	0.280
90 mm	2A-7(a)	46.3	18.4	28.3	46.0	31.4	39.88	30.39	0.79	1.03	0.466	0.355
90 mm	2A-7(b)	46.3	18.4	28.3	66.0	45.0	39.88	30.39	1.13	1.48	0.466	0.355

TABLE 6.1(a) The results of theoretical prediction of shear capacity (SHEAR 1 and SHEAR 2) and comparisons with the test data (series 1A and 2A)

Remark	Beam	f_{cu} (N/mm^2)	f_{cuh} (N/mm^2)	Average f_{cav} (N/mm^2)	Ultimate load (kN)	Ult. shear experiment (kN)	Ultimate shear theory (kN)		Shear ratio experiment/theory		Effectiveness factor	
							mean	charac	mean	charac	mean	charac
control	1B-1(a)	36.4		27.7	56.5	38.5	44.01	33.54	0.88	1.15	0.525	0.400
control	1B-1(b)	36.4		27.7	69.8	47.6	44.01	33.54	1.08	1.42	0.525	0.400
control	1B-1(R)	33.4		25.4	53.5	36.5	42.16	32.13	0.87	1.14	0.549	0.418
	1B-2(a)	36.4	9.2	23.4	56.6	38.6	37.22	28.36	1.04	1.36	0.525	0.400
	1B-2(R)	33.4	14.1	22.2	42.5	29.0	36.82	28.06	0.79	1.03	0.549	0.418
	1B-3(a)	40.6	13.6	30.9	71.0	48.4	46.48	35.42	1.04	1.37	0.497	0.379
	1B-3(a)R	27.5	13.4	20.9	53.4	36.4	38.25	29.15	0.95	1.25	0.604	0.460
	1B-3(b)R	27.5	13.4	20.9	45.3	30.9	38.25	29.15	0.81	1.06	0.604	0.460
	1B-4(a)	40.6	13.6	30.9	63.8	43.5	46.48	35.42	0.94	1.23	0.497	0.379
	1B-4(a)R	27.5	13.4	20.9	57.5	39.2	38.25	29.15	1.02	1.35	0.604	0.460
	1B-5(a)	29.8	9.1	20.4	54.6	37.2	35.92	27.37	1.04	1.36	0.581	0.443
	1B-5(b)	29.8	9.1	20.4	46.1	31.4	35.92	27.37	0.88	1.15	0.581	0.443
	1B-6(a)	29.8	9.1	19.6	39.2	26.7	34.46	26.26	0.78	1.02	0.581	0.443
	1B-6(b)	29.8	9.1	19.6	35.8	24.4	34.46	26.26	0.71	0.93	0.581	0.443
90 mm	2B-2(a)	34.0	15.4	21.1	52.0	35.5	34.79	26.51	1.02	1.34	0.544	0.415
90 mm	2B-2(b)	34.0	15.4	21.1	47.0	32.0	34.79	26.51	0.92	1.21	0.544	0.415
a/d=3.5	2B-5	39.7	-	30.2	42.0	28.1	21.42	16.32	1.31	1.72	0.424	0.323
a/d=3.5	2B-6.1	39.7	14.8	28.1	39.1	26.1	19.92	15.18	1.31	1.72	0.424	0.323
a/d=3.5	2B-6.2	39.7	14.8	28.1	37.9	25.3	19.92	15.18	1.27	1.67	0.424	0.323

TABLE 6.1(b) The results of theoretical prediction of shear capacity (SHEAR 1 and SHEAR 2) and comparisons with the test data (series 1B and 2B)

Remark	Beam	f_{cu} (N/mm^2)	f_{cuh} (N/mm^2)	Average f_{cav} (N/mm^2)	Ult. shear experiment (kN)	Ultimate shear theory (kN)		Shear ratio experiment/theory	
						mean	charac	mean	charac
control	1A-1(a)	45.6		34.7	46.5	44.88	37.2	1.04	1.25
control	1A-1(b)	45.6		34.7	51.6	44.88	37.2	1.15	1.39
	1A-2(a)	45.6	23.5	30.8	23.9	39.88	33.06	0.60	0.72
	1A-2(b)	45.6	23.5	30.8	24.8	39.88	33.06	0.62	0.75
torsion	1A-3(a)	53.8	22.6	40.9	24.5	46.93	39.74	0.52	0.62
	1A-3(b)	53.8	22.6	40.9	30.0	46.93	39.74	0.64	0.76
torsion	1A-4(a)	53.8	22.6	40.9	26.2	46.93	39.74	0.56	0.66
	1A-4(b)	53.8	22.6	40.9	38.9	46.93	39.74	0.83	0.98
	1A-5(a)	48.3	21.5	36.7	34.1	45.60	38.11	0.75	0.90
	1A-5(b)	48.3	21.5	36.7	51.1	45.60	38.11	1.12	1.34
	1A-6(a)	48.3	21.5	33.7	34.8	41.88	34.99	0.83	0.99
	1A-6(b)	48.3	21.5	33.7	35.3	41.88	34.99	0.84	1.01
	1A-7(b)	54.2	26.2	36.9	30.0	42.14	35.72	0.71	0.84
	1A-8(a)	54.2	26.2	41.2	44.7	47.05	39.89	0.95	1.12
	1A-8(b)	54.2	26.2	41.2	58.6	47.05	39.89	1.25	1.47
insitu	2A-1.1(a)	42.3	10.4	27.2	34.7	37.16	30.47	0.93	1.14
insitu	2A-1.1(b)	42.3	10.4	27.2	27.6	37.16	30.47	0.74	0.91
precast	2A-1.2(a)	42.3	11.6	27.3	45.3	37.29	30.58	1.21	1.48
precast	2A-1.2(b)	42.3	11.6	27.3	48.4	37.29	30.58	1.30	1.58
void	2A-1.3(a)	42.3		25.7	17.8	35.11	28.79	0.51	0.62
void	2A-1.3(b)	42.3		25.7	18.5	35.11	28.79	0.53	0.64
	2A-2(a)	54.0	14.1	41.0	50.3	46.91	39.73	1.07	1.27
	2A-2(b)	54.0	14.1	41.0	37.2	46.91	39.73	0.79	0.94
	2A-3(a)	54.0	14.1	41.0	46.4	46.91	39.73	0.99	1.17
	2A-3(b)	54.0	14.1	41.0	52.6	46.91	39.73	1.12	1.32
a/d=3.5	2A-4	52.6		40.0	23.8	23.31	18.78	1.02	1.27
a/d=3.5	2A-5.1	52.6	13.0	36.7	24.6	21.40	17.25	1.15	1.43
a/d=3.5	2A-5.2	52.6	13.0	36.7	20.4	21.40	17.25	0.95	1.18
90 mm	2A-7(a)	46.3	18.4	28.3	31.4	36.27	30.14	0.87	1.04
90 mm	2A-7(b)	46.3	18.4	28.3	45.0	36.27	30.14	1.24	1.49
control	1B-1(a)	36.4		27.7	38.5	41.90	33.51	0.92	1.15
control	1B-1(b)	36.4		27.7	47.6	41.90	33.51	1.14	1.42
	1B-1(R)	33.4		25.4	36.5	40.74	32.15	0.90	1.13
	1B-2(a)	36.4	9.2	23.4	38.6	35.44	28.34	1.09	1.36
	1B-2(R)	33.4	14.1	22.2	29.0	35.63	28.12	0.81	1.03
	1B-3(a)	40.6	13.6	30.9	48.4	43.43	35.38	1.11	1.37
	1B-3(a)R	27.5	13.4	20.9	36.4	37.80	29.12	0.96	1.25
	1B-3(b)R	27.5	13.4	20.9	30.9	37.80	29.12	0.82	1.06
	1B-4(a)	40.6	13.6	30.9	43.5	43.43	35.38	1.00	1.23
	1B-4(a)R	27.5	13.4	20.9	39.2	37.80	29.12	1.04	1.35
	1B-5(a)	29.8	9.1	20.4	37.2	35.18	27.35	1.06	1.36
	1B-5(b)	29.8	9.1	20.4	31.4	35.18	27.35	0.89	1.15
	1B-6(a)	29.8	9.1	19.6	26.7	33.80	26.27	0.79	1.02
	1B-6(b)	29.8	9.1	19.6	24.4	33.80	26.27	0.72	0.93
90 mm	2B-2(a)	34.0	15.4	21.1	35.5	33.46	26.48	1.06	1.34
90 mm	2B-2(b)	34.0	15.4	21.1	32.0	33.46	26.48	0.96	1.21
a/d=3.5	2B-5	39.7		30.2	28.1	21.13	16.34	1.33	1.72
a/d=3.5	2B-6.1	39.7	14.8	28.1	26.1	19.68	15.21	1.33	1.72
a/d=3.5	2B-6.2	39.7	14.8	28.1	25.3	19.68	15.21	1.29	1.67

TABLE 6.2 The results of theoretical prediction of shear capacity (Nielsen et al) and comparisons with the test data (all series)

Remark	Beam	Consider each series separately			Consider all series together		
		shear ratio, exp/theory (charac) Condition factor of			shear ratio, exp/theory (charac) Condition factor of		
		1.10	1.17	1.34	1.06	1.14	1.33
	1A-2(a)	*0.79*	*0.84*	1.00	*0.76*	*0.82*	*0.99*
	1A-2(b)	*0.82*	*0.87*	1.04	*0.79*	*0.85*	1.03
	1A-3(b)	*0.82*	*0.86*	*0.99*	*0.79*	*0.84*	*0.99*
	1A-4(b)	1.06	1.12	1.29	1.02	1.09	1.28
	1A-5(a)	*0.97*	1.03	1.22	*0.94*	1.00	1.21
	1A-5(b)	1.46	1.55	1.83	1.41	1.51	1.81
	1A-6(a)	1.08	1.15	1.36	1.04	1.12	1.34
	1A-6(b)	1.10	1.17	1.38	1.06	1.13	1.36
	1A-7(b)	*0.91*	*0.96*	1.11	*0.88*	*0.93*	1.10
	1A-8(a)	1.21	1.28	1.48	1.17	1.24	1.46
	1A-8(b)	1.59	1.68	1.94	1.54	1.63	1.92
	2A-1.1(a)	1.25	1.34	1.62	1.20	1.30	1.60
	2A-1.1(b)	*0.99*	1.07	1.29	*0.96*	1.03	1.28
	2A-2(a)	1.36	1.44	1.67	1.32	1.41	1.66
	2A-2(b)	1.01	1.07	1.24	*0.98*	1.04	1.23
	2A-3(a)	1.26	1.33	1.55	1.22	1.30	1.53
	2A-3(b)	1.43	1.51	1.75	1.38	1.47	1.73
a/d=3.5	2A-5.1	1.59	1.72	2.16	1.52	1.65	2.13
a/d=3.5	2A-5.2	1.31	1.43	1.79	1.26	1.37	1.76
90 mm	2A-7(a)	1.13	1.21	1.44	1.10	1.17	1.42
90 mm	2A-7(b)	1.63	1.74	2.06	1.57	1.68	2.04
		Condition factor of			Consider all series together		
		1.01	1.06	1.08			
	1B-2(a)	1.38	1.44	1.47	1.44	1.55	1.81
	1B-2(R)	1.04	1.09	1.11	1.09	1.17	1.37
	1B-3(a)	1.38	1.44	1.48	1.45	1.57	1.96
	1B-3(a)R	1.26	1.32	1.35	1.33	1.42	1.66
	1B-3(b)R	1.07	1.12	1.15	1.12	1.21	1.41
	1B-4(a)	1.24	1.30	1.33	1.3	1.41	1.76
	1B-4(a)R	1.36	1.42	1.45	1.43	1.53	1.79
	1B-5(a)	1.37	1.44	1.47	1.44	1.55	1.81
	1B-5(b)	1.16	1.21	1.24	1.22	1.31	1.53
	1B-6(a)	1.03	1.08	1.10	1.08	1.16	1.35
	1B-6(b)	*0.94*	*0.98*	1.00	*0.98*	1.06	1.24
90 mm	2B-2(a)	1.35	1.41	1.45	1.42	1.52	1.78
90 mm	2B-2(b)	1.22	1.28	1.31	1.28	1.38	1.61
a/d=3.5	2B-6.1	1.74	1.82	1.86	1.82	1.95	2.28
a/d=3.5	2B-6.2	1.68	1.76	1.80	1.77	1.89	2.21

TABLE 6.3 Characteristic shear ratio for different condition factors (all series)

Beam	Normal concrete strength f_{cu} (N/mm^2)	Honeycombed strength f_{cuh} (N/mm^2)	Weighted average f_{cav} (N/mm^2)	Ultimate load, experiment (kN)	Ultimate shear, experiment (kN)	Ultimate shear, theory (kN)	Ratio of ultimate shear exp/theory
2A-8(a)	40.6	-	30.9	70.1	47.8	58.0	0.82
2A-8(b)	40.6	-	30.9	84.9	57.9	58.0	1.00
2A-9(a)	40.6	11.1	26.2	74.0	50.5	49.2	1.03
2A-9(b)	40.6	11.1	26.2	88.9	60.6	49.2	1.23
2B-3(a)	33.0	-	25.1	84.3	57.5	54.3	1.06
2B-3(b)	33.0	-	25.1	85.8	58.5	54.3	1.08
2B-4(a)	33.0	8.9	21.3	85.9	58.6	46.1	1.27
2B-4(b)	33.0	8.9	21.3	55.7	38.0	46.1	0.82

TABLE 6.4 Predicted ultimate shear for beams with shear reinforcement- Plastic theory

Beam	normal strength f_{cu} (N/mm²)	h.comb strength f_{cuh} (N/mm²)	cracking shear V_c exp (kN)	ultimate shear V_u exp (kN)	shear V_c BS 8110 use f_{cu} (kN)	shear V_c BS 8110 use f_{cuh} (kN)	shear V_c BD44/95 use f_{cu} (kN)	shear V_c BD44/95 use f_{cuh} (kN)	ratio of V_c exp/BS 8110 use f_{cu}	ratio of V_c exp/BS 8110 use f_{cuh}	V_u/V_c exp/ BS 8110 use f_{cu}	V_u/V_c exp/ BS 8110 use f_{cuh}	ratio of V_c exp/ BD44/95 use f_{cu}	ratio of V_c exp/ BD44/95 use f_{cuh}	V_u/V_c exp/ BD44/95 use f_{cu}	V_u/V_c exp/ BD44/95 use f_{cuh}
1A-1(a)	45.6	-	30.0	46.5	22.83	-	21.45		1.31	-	2.04	-	1.40	-	2.17	-
1A-1(b)			27.3	51.6	22.83	-	21.45		1.19	-	2.26	-	1.27	-	2.41	-
1A-2(a)	45.6	23.50	20.5	23.8	22.83	18.31	21.45	17.19	0.90	1.12	1.04	1.30	0.95	1.19	1.11	1.39
1A-2(b)			16.4	24.8	22.83	18.31	21.45	17.19	0.72	0.89	1.08	1.35	0.76	0.95	1.15	1.44
1A-3(a)	53.8	22.60	24.5	24.5	24.13	18.07	22.66	16.97	1.02	1.36	1.02	1.36	1.08	1.45	1.08	1.45
1A-3(b)			23.2	30.0	24.13	18.07	22.66	16.97	0.96	1.28	1.24	1.66	1.02	1.37	1.32	1.77
1A-4(a)	53.8	22.60	23.9	26.2	24.13	18.07	22.66	16.97	0.99	1.32	1.09	1.45	1.05	1.41	1.16	1.54
1A-4(b)			19.1	38.9	24.13	18.07	22.66	16.97	0.79	1.06	1.61	2.15	0.84	1.12	1.72	2.29
1A-5(a)	48.3	21.50	30.7	34.1	23.27	17.77	21.86	16.69	1.32	1.73	1.46	1.92	1.40	1.84	1.56	2.04
1A-5(b)			20.5	51.1	23.27	17.77	21.86	16.69	0.88	1.15	2.20	2.88	0.94	1.23	2.34	3.06
1A-6(a)	48.3	21.50	23.9	34.8	23.27	17.77	21.86	16.69	1.03	1.34	1.49	1.96	1.09	1.43	1.59	2.08
1A-6(b)			23.9	35.3	23.27	17.77	21.86	16.69	1.03	1.34	1.52	1.99	1.09	1.43	1.62	2.12
1A-7(b)	54.2	26.2	27.3	30.0	24.19	18.98	22.72	17.83	1.13	1.44	1.24	1.58	1.20	1.53	1.32	1.68
1A-8(a)	54.2	26.2	17.0	44.7	24.19	18.98	22.72	17.83	0.70	0.90	1.85	2.35	0.75	0.96	1.97	2.50
1A-8(b)			17.0	58.6	24.19	18.98	22.72	17.83	0.70	0.90	2.42	3.09	0.75	0.96	2.58	3.29
2A-1.1(a)	42.3	10.4	27.3	34.7	22.27	13.95	20.92	13.11	1.23	1.96	1.56	2.49	1.30	2.08	1.66	2.65
2A-1.1(b)			16.4	27.6	22.27	13.95	20.92	13.11	0.74	1.17	1.24	1.98	0.78	1.25	1.32	2.11
2A-2(a)	54.0	14.1	24.6	50.3	24.16	15.44	22.69	14.51	1.02	1.59	2.08	3.26	1.08	1.69	2.21	3.46
2A-2(b)			24.6	37.2	24.16	15.44	22.69	14.51	1.02	1.59	1.54	2.41	1.08	1.69	1.64	2.56
2A-3(a)	54.0	14.1	27.3	46.4	24.16	15.44	22.69	14.51	1.13	1.77	1.92	3.01	1.20	1.88	2.05	3.20
2A-3(b)			27.3	52.6	24.16	15.44	22.69	14.51	1.13	1.77	2.18	3.41	1.20	1.88	2.32	3.62
2A-4	52.6	-	21.4	23.8	23.95	-	22.49		0.89	-	0.99	-	0.95	-	1.06	-
2A-5.1		13.0	18.7	24.6	23.95	15.03	22.49	14.12	0.78	1.25	1.03	1.64	0.83	1.33	1.09	1.74
2A-5.2			18.7	20.4	23.95	15.03	22.49	14.12	0.78	1.25	0.85	1.36	0.83	1.33	0.91	1.44
2A-6(a)	46.3	18.4	19.1	21.8	22.95	16.87	21.55	15.85	0.83	1.13	0.95	1.29	0.89	1.20	1.01	1.38
2A-6(b)			19.1	19.1	22.95	16.87	21.55	15.85	0.83	1.13	0.83	1.13	0.89	1.20	0.89	1.20
2A-7(a)	46.3	18.4	19.1	31.4	22.95	16.87	21.55	15.85	0.83	1.13	1.37	1.86	0.89	1.20	1.46	1.98
2A-7(b)			21.8	45.0	22.95	16.87	21.55	15.85	0.95	1.29	1.96	2.67	1.01	1.38	2.09	2.84

TABLE 6.5(a) Comparisons of shear from test data and predictions by BS 8110 and BD 44/95 using f_{cu} and f_{cuh} (series 1A and 2A)

Beam	normal strength f_{cu} (N/mm²)	h.comb strength f_{cuh} (N/mm²)	cracking shear V_c exp (kN)	ultimate shear V_u exp (kN)	shear V_c BS 8110 use f_{cu} (kN)	shear V_c BS 8110 use f_{cuh} (kN)	shear V_c BD44/95 use f_{cu} (kN)	shear V_c BD44/95 use f_{cuh} (kN)	ratio of V_c exp/BS 8110 use f_{cu}	ratio of V_c exp/BS 8110 use f_{cuh}	V_u/V_c exp/ BS 8110 use f_{cu}	V_u/V_c exp/ BS 8110 use f_{cuh}	ratio of V_c exp/ BD44/95 use f_{cu}	ratio of V_c exp/ BD44/95 use f_{cuh}	V_u/V_c exp/ BD44/95 use f_{cu}	V_u/V_c exp/ BD44/95 use f_{cuh}
1B-1(a)	36.4	-	27.3	38.5	21.18	-	19.89	-	1.29	-	1.82	-	1.37	-	1.94	-
1B-1(b)		-	24.5	47.6	21.18	-	19.89	-	1.16	-	2.25	-	1.23	-	2.39	-
1B-1R	33.4	-	24.5	36.5	20.58	-	19.33	-	1.19	-	1.77	-	1.27	-	1.89	-
1B-2(a)	36.4	9.2	19.1	38.6	21.18	13.39	19.89	12.58	0.90	1.43	1.82	2.88	0.96	1.52	1.94	3.07
1B-2R	33.4	14.1	21.1	29.0	20.58	15.44	19.33	14.50	1.03	1.37	1.41	1.88	1.09	1.46	1.50	2.00
1B-3(a)	40.6	13.6	24.5	48.4	21.96	15.25	20.63	14.33	1.12	1.61	2.20	3.17	1.19	1.71	2.35	3.38
1B-3(a)R	27.5	13.4	24.5	36.4	19.29	15.18	18.12	14.26	1.27	1.62	1.89	2.40	1.35	1.72	2.01	2.55
1B-3(b)R			24.5	30.9	19.29	15.18	18.12	14.26	1.27	1.62	1.60	2.03	1.35	1.72	1.70	2.17
1B-4(a)	40.6	13.6	21.8	43.5	21.96	15.25	20.63	14.33	0.99	1.43	1.98	2.85	1.06	1.52	2.11	3.04
1B-4(b)			21.8	-	21.96	15.25	20.63	14.33	0.99	1.43	-	-	1.06	1.52	-	-
1B-4(a)R	27.5	13.4	19.1	39.2	19.29	15.18	18.12	14.26	0.99	1.26	2.03	2.58	1.05	1.34	2.16	2.75
1B-5(a)	29.8	9.1	27.3	37.2	19.81	13.34	18.61	12.53	1.38	2.04	1.88	2.79	1.47	2.18	2.00	2.97
1B-5(b)			26.3	31.4	19.81	13.34	18.61	12.53	1.32	1.97	1.59	2.36	1.41	2.09	1.69	2.51
1B-6(a)	29.8	9.1	21.8	26.7	19.81	13.34	18.61	12.53	1.10	1.64	1.35	2.00	1.17	1.74	1.44	2.13
1B-6(b)			21.8	24.4	19.81	13.34	18.61	12.53	1.10	1.64	1.23	1.83	1.17	1.74	1.31	1.95
2B-1(a)	34.0	15.4	8.2	13.3	20.70	15.90	19.45	14.93	0.40	0.51	0.64	0.84	0.42	0.55	0.68	0.89
2B-1(b)			8.2	13.7	20.70	15.90	19.45	14.93	0.40	0.51	0.66	0.86	0.42	0.55	0.70	0.92
2B-2(a)	34.0	15.4	24.5	35.5	20.70	15.90	19.45	14.93	1.19	1.54	1.71	2.23	1.26	1.64	1.82	2.37
2B-2(b)			24.5	32.0	20.70	15.90	19.45	14.93	1.19	1.54	1.55	2.02	1.26	1.64	1.65	2.15
2B-5	39.7	-	18.7	28.1	21.80	-	20.48	-	0.86	-	1.29	-	0.91	-	1.37	-
2B-6.1		14.8	18.7	26.1	21.80	15.69	20.48	14.74	0.86	1.19	1.20	1.67	0.91	1.27	1.28	1.77
2B-6.2			21.4	25.3	21.80	15.69	20.48	14.74	0.98	1.36	1.16	1.61	1.04	1.45	1.24	1.72

TABLE 6.5(b) Comparisons of shear from test data and predictions by BS 8110 and BD 44/95 using f_{cu} and f_{cuh} (series 1B and 2B)

Beam	Normal concrete strength f_{cu} (N/mm^2)	Honey-combed strength f_{cuh} (N/mm^2)	cracking shear, exp (kN)	Ult shear, exp (kN)	shear BS 8110 use f_{cu} (kN)	shear BS 8110 use f_{cuh} (kN)	shear BD 44/95 use f_{cu} (kN)	shear BD 44/95 use f_{cuh} (kN)	CRACKING				ULTIMATE			
									ratio exp/ BS 8110 use f_{cu}	ratio exp/ BS 8110 use f_{cuh}	ratio exp/ BD 44/95 use f_{cu}	ratio exp/ BD 44/95 use f_{cuh}	ratio exp/ BS 8110 use f_{cu}	ratio exp/ BS 8110 use f_{cuh}	ratio exp/ BD 44/95 use f_{cu}	ratio exp/ BD 44/95 use f_{cuh}
2A-8(a)	40.6	-	27.3	47.8	35.2	-	33.9	-	1.24	-	1.31	-	1.36	-	1.41	-
2A-8(b)	40.6	-	24.6	57.9	35.2	-	33.9	-	1.12	-	1.19	-	1.64	-	1.71	-
2A-9(a)	40.6	11.1	27.3	50.5	35.2	27.5	33.9	26.6	1.24	1.91	1.31	2.04	1.43	1.84	1.49	1.90
2A-9(b)	40.6	11.1	30.0	60.6	35.2	27.5	33.9	26.6	1.36	2.10	1.45	2.24	1.72	2.20	1.79	2.28
2B-3(a)	33.0	-	32.7	57.5	33.7	-	32.5	-	1.60	-	1.69	-	1.71	-	1.77	-
2B-3(b)	33.0	-	27.3	58.5	33.7	-	32.5	-	1.33	-	1.41	-	1.74	-	1.80	-
2B-4(a)	33.0	8.9	21.8	58.6	33.7	26.4	32.5	25.6	1.06	1.65	1.13	1.75	1.74	2.22	1.80	2.29
2B-4(b)	33.0	8.9	16.4	38.0	33.7	26.4	32.5	25.6	0.80	1.24	0.84	1.32	1.13	1.44	1.17	1.48

TABLE 6.6 Beams with shear reinforcement-comparisons with BS 8110 and BD 44/95

Beam	Ultimate load, experiment (kN)	Ultimate shear, experiment (kN)	Ultimate shear, theory (kN)	Ratio of ultimate shear exp/theory
2A-6(a)	32.0	21.8	19.7	1.11
2A-6(b)	28.0	19.1	19.7	0.97
2B-1(a)	19.5	13.3	16.5	0.81
2B-1(b)	20.1	13.7	16.5	0.83

TABLE 6.7 Predicted ultimate shear for beams with a construction joint, Plastic theory

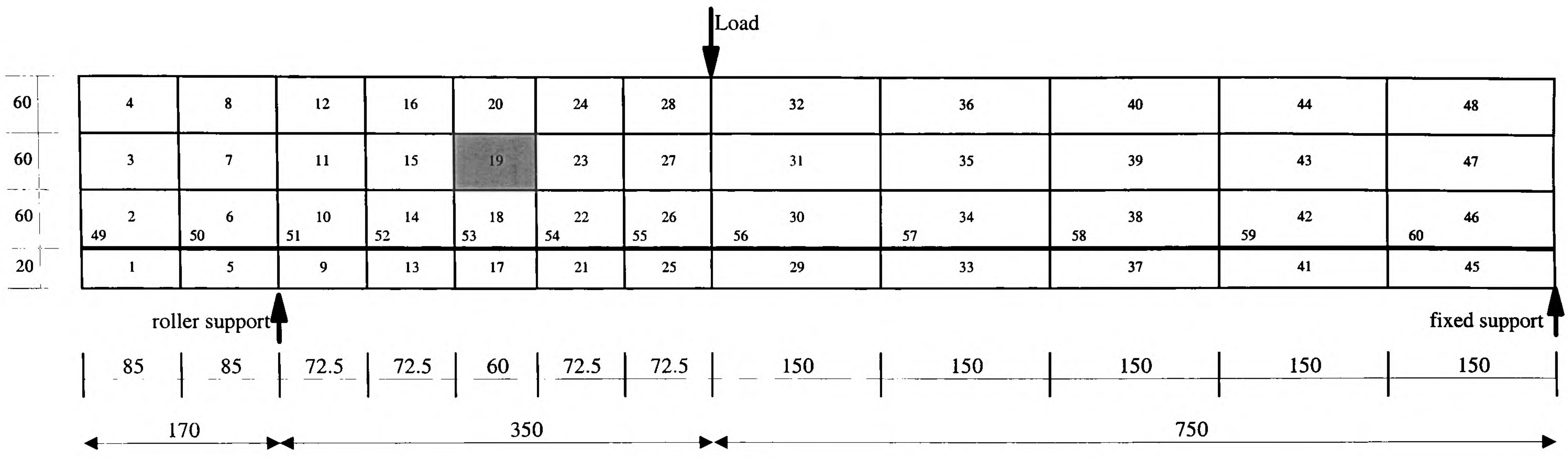


FIGURE 6.1 Finite element meshes for control beam and beam with a honeycombed zone at the centre of high shear zone, element type QPM8 for plate element and BAR3 for steel reinforcement

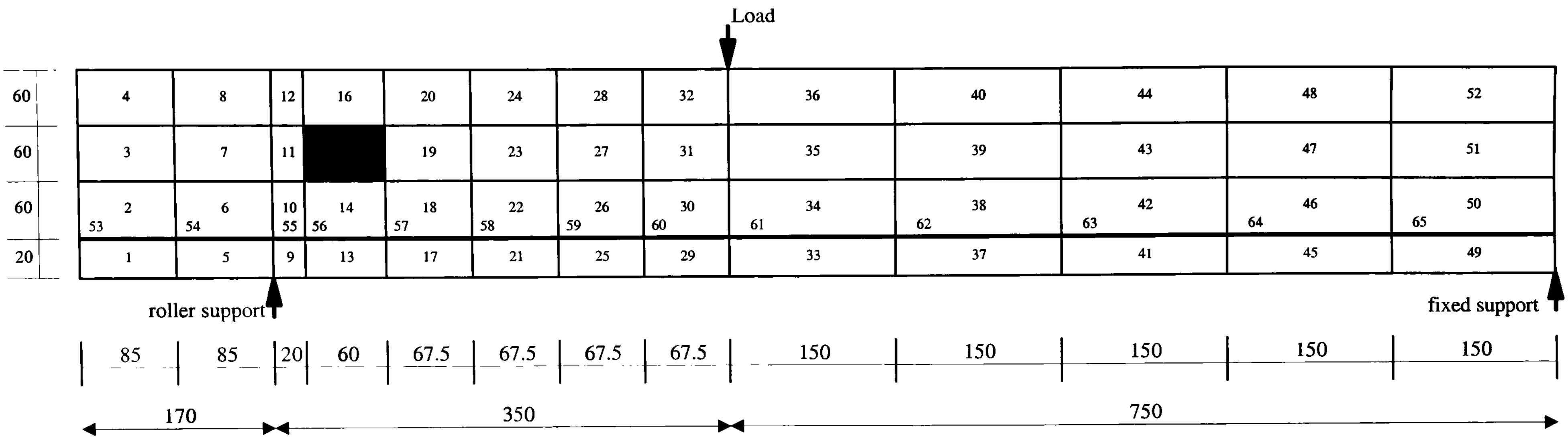


FIGURE 6.2 Finite element meshes for beam with a honeycombed zone along the neutral axis near to the support

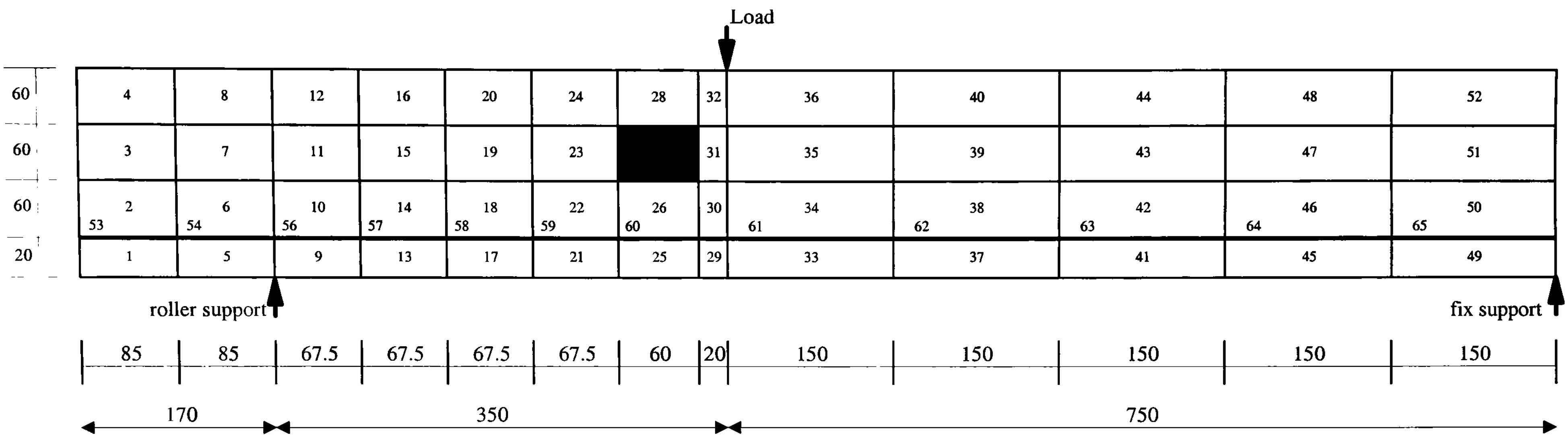


FIGURE 6.3 Finite element meshes for beam with a honeycombed zone along the neutral axis near to the loading point

0.023	0.001	0.006	0.010	0.013	-0.040	0.000	0.080	0.530	0.530	-0.840
0.070	-0.330	-0.605	-0.960	-1.325	-1.610	-1.530	-2.150	-2.440	-3.400	-5.580
0.560		0.080		0.060		0.080		0.175		-1.140
-0.180		-0.555		-1.015		-1.395		-1.855		-3.100
0.125	0.200	0.270	0.250	0.243	0.200	0.220	0.110	0.180	0.040	-0.690
-0.310	-0.465	-0.620	-0.730	-0.835	-0.920	-1.058	-1.215	-1.640	-1.310	-1.140
0.210		0.390		0.385		0.355		0.275		-0.180
-0.430		-0.605		-0.600		-0.680		-0.900		-0.540
0.310	0.405	0.558	0.560	0.603	0.630	0.640	0.645	0.683	0.565	0.440
-0.580	-0.560	-0.635	-0.520	-0.425	-0.430	-0.395	-0.450	-0.475	-0.385	-0.355
0.280		0.605		0.765		0.970		1.195		1.280
-0.940		-0.335		-0.215		-0.180		-0.195		-0.140
0.240	0.235	0.363	0.405	0.528	0.615	0.713	0.825	0.928	0.995	2.160
-1.320	-0.385	-0.048	-0.005	-0.080	-0.040	-0.043	-0.020	-0.025	-0.010	-0.010

FIGURE 6.4 Principal tensile and compressive stresses, beam without a honeycombed zone

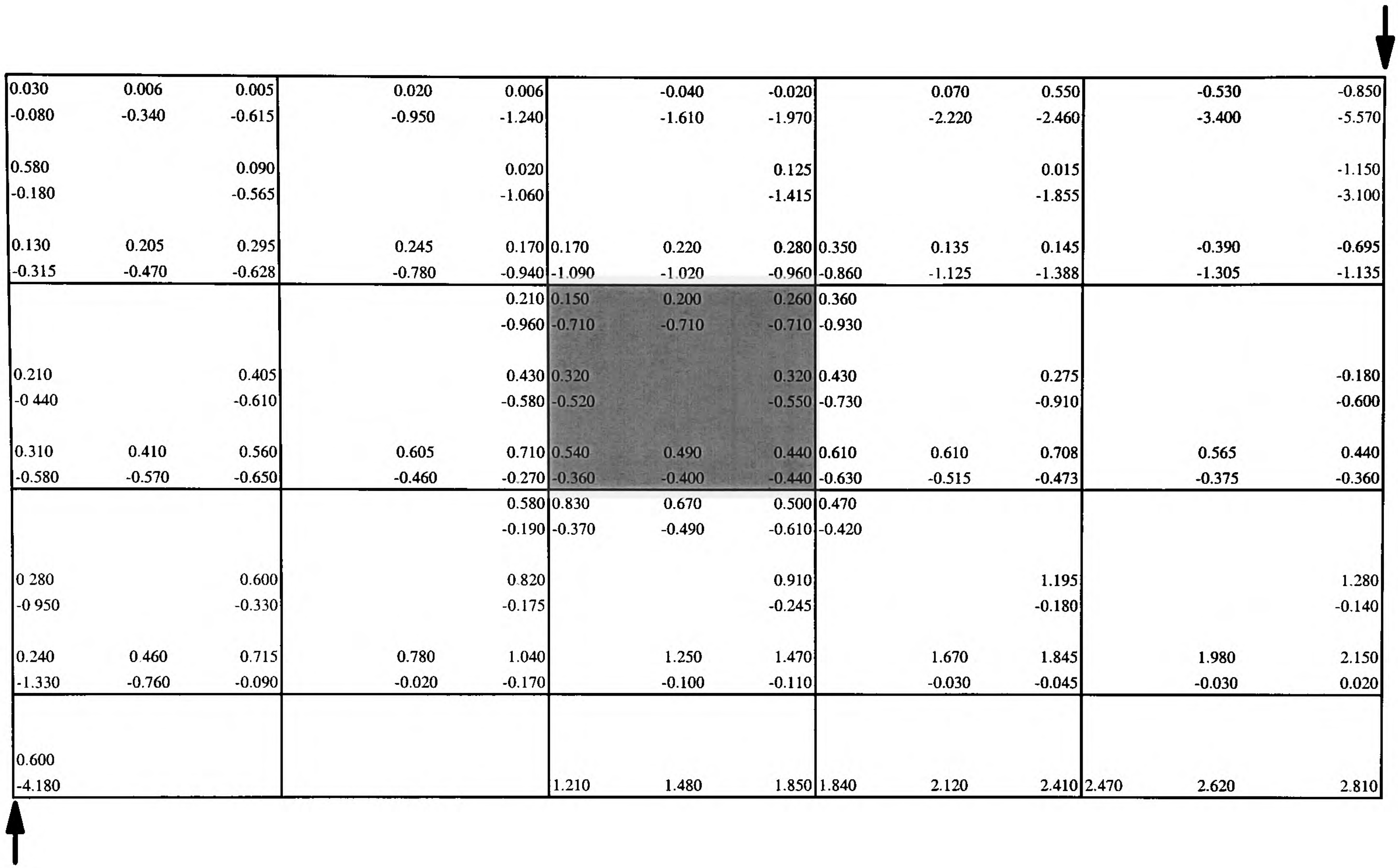


FIGURE 6.5 Principal tensile and compressive stresses, beam with a honeycombed zone at the central of high shear area

0.010	0.020	0.030		0.020	-0.002		0.000	0.025		-0.030	-0.020		0.090	0.535		-0.560	-0.860
-0.060	-0.100	-0.130		-0.440	-0.785		-1.060	-1.355		-1.660	-1.920		-2.200	-2.495		-3.480	-5.690
0.030		0.050			0.110			0.045			0.075			0.160			-1.170
-0.180		-0.315			-0.595			-1.030			-1.460			-1.895			-3.130
0.075	0.085	0.100	0.120	0.250	0.390	0.290	0.275	0.215		0.190	0.220		0.100	0.153		-0.410	-0.650
-0.300	-0.430	-0.510	-0.570	-0.580	-0.590	-0.470	-0.680	-0.855		-0.945	-1.098		-1.265	-1.678		-1.295	-1.170
		0.090	0.100	0.200	0.300	0.400											
		-0.590	-0.430	-0.430	-0.430	-0.530											
0.200		0.220	0.240		0.340	0.420		0.390			0.350			0.250			-0.180
-0.480		-0.460	-0.400		-0.520	-0.620		-0.620			-0.700			-0.910			-0.590
0.375	0.400	0.400	0.390	0.380	0.390	0.470	0.520	0.628		0.635	0.650		0.645	0.680		0.560	0.430
-0.380	-0.535	-0.400	-0.400	-0.500	-0.620	-0.720	-0.555	-0.448		-0.425	-0.408		-0.450	-0.473		-0.380	-0.355
		0.500	0.590	0.530	0.520	0.330											
		-0.480	-0.500	-0.630	-0.800	-0.610											
0.360		0.505			0.545			0.760			1.000			1.205			1.270
-0.880		-0.870			-0.350			-0.185			-0.160			-0.190			-0.140
0.450	0.280	0.545		0.540	0.710		0.870	1.045		1.270	1.485		1.700	1.880		2.000	2.160
-1.300	-1.200	-1.310		-0.470	-0.041		-0.070	-0.075		-0.080	-0.060		-0.040	-0.045		-0.020	0.020
1.180	0.290	1.950															
-8.430	-3.250	-0.600						1.150	1.280	1.550	1.830	1.860	2.140	2.410	2.510	2.640	2.780

FIGURE 6.6 Principal tensile and compressive stresses, beam with a honeycombed zone near to the support

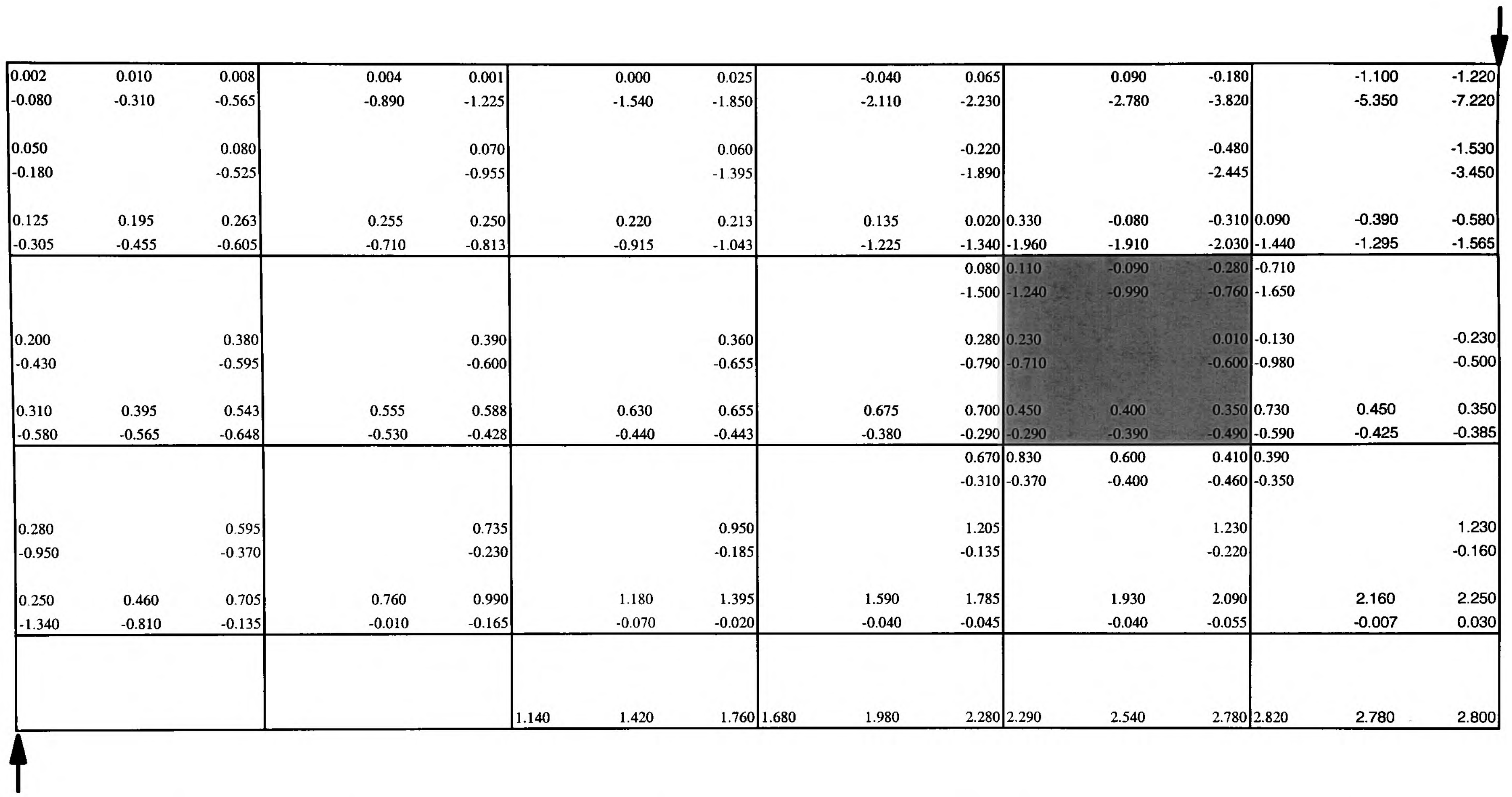
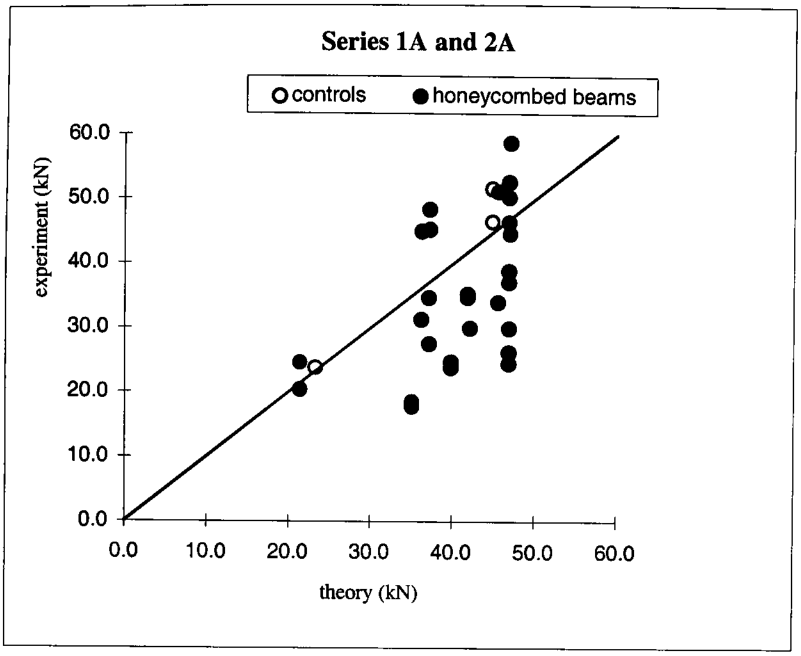


FIGURE 6.7 Principal tensile and compressive stresses, beam with a honeycombed zone near to the loading point



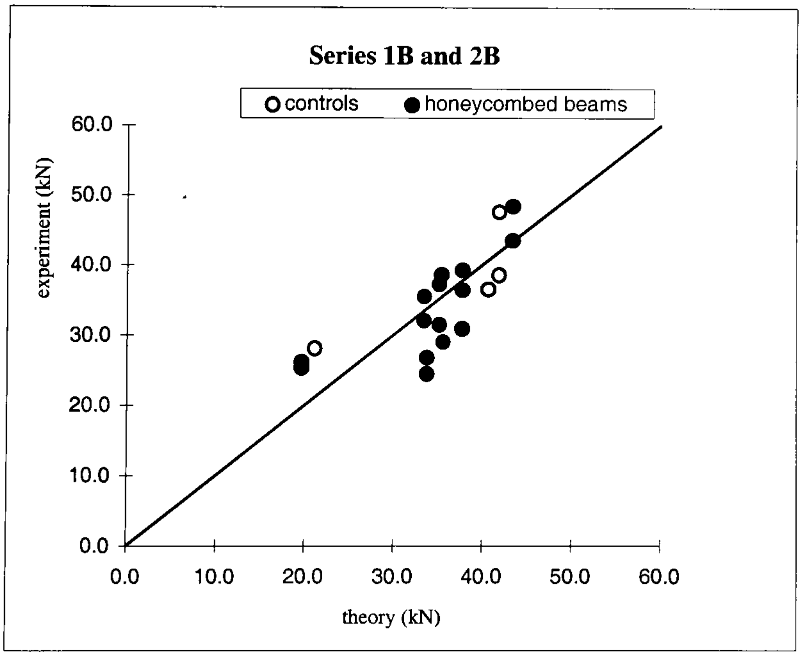
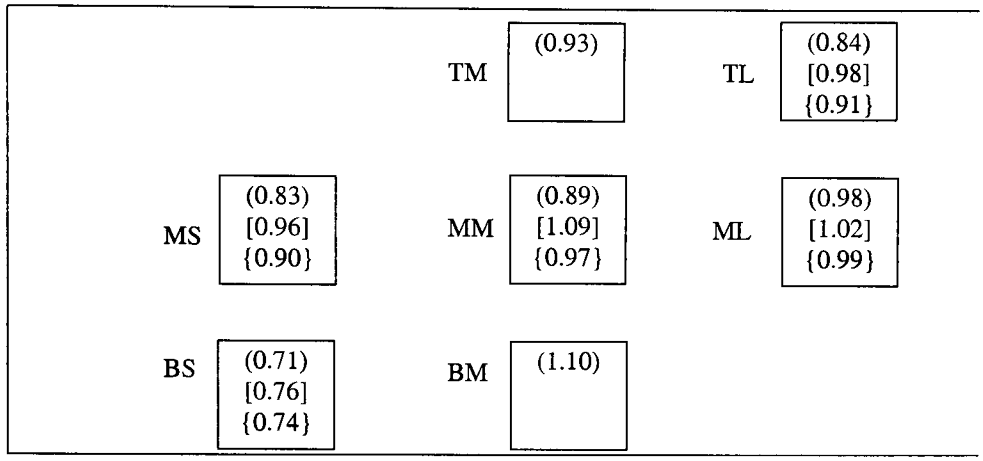


FIGURE 6.8 Ultimate shear, experimental vs plasticity theory



Notes:

(xxx)	series 1A and 2A
[xxx]	series 1B and 2B
{xxx}	combination of all series

FIGURE 6.9 The average ratio of shear strength (experiment to plasticity theory) for honeycombed beams at each location of honeycombed zone

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

7.1 CONCLUSIONS

7.1.1 The effects of a honeycombed zone on shear in concrete beams

1. Despite all the variations in the test results, the experimental work clearly demonstrates that a honeycombed zone can affect the shear behaviour in a concrete beam. One or both of the following can be affected: the formation of the diagonal crack; and the ultimate shear capacity. There are locations of the honeycombed zone at which the effects are critical for one or all of: the formation of diagonal cracking; the diagonal cracking load; and the ultimate shear capacity. There are also locations where a honeycombed zone only affects the formation of the diagonal cracking but not the ultimate capacity and vice versa.

2. It is clear that the cast in-situ technique used throughout the tests is appropriate in simulating the honeycombed problem. No indication was found to suggest any existence of a zone of discontinuity between the zone of honeycombed concrete and the normal concrete. In contrast a pre-cast honeycombed zone was found to introduce a zone of discontinuity which can disturb the transfer of forces within the shear zone.

3. The effects are of a different magnitude for the 4 series of beams tested, with generally, series 1A and 2A beams demonstrating a more significant effect of the presence of a honeycombed zone compared to beams of series 1B and 2B. This occurs both in terms of the formation of diagonal cracking and the ultimate shear capacity. The series 1A and 2A tests have a higher strength of normal concrete, with an average of 50.5 N/mm^2 and 47.2 N/mm^2 respectively. For series 1B and 2B beams, the average strength of the normal concrete are 33.5 N/mm^2 and 35.6 N/mm^2 respectively. Series 1A has a higher honeycombed concrete strength of 23.4 N/mm^2. The honeycombed concrete strength in series 2A, 1B and 2B are 13.1 N/mm^2, 11.9 N/mm^2 and 13.0 N/mm^2 respectively. The average ratio of the honeycombed to the normal concrete strength for series 1A and 2A are 0.46 and 0.28 respectively. For series 1B and 2B the average ratios are 0.36 and 0.37 respectively.

4. From the above point, beams in series 1A can be categorised as beams with a high strength of normal concrete and a medium strength of honeycombed concrete. Series 2A consists of beams with a high strength of normal concrete with a low strength of honeycombed concrete. Beams in series 1B and 2B can be categorised as beams of medium strength of normal concrete and low strength of honeycombed concrete. In the following points of the conclusions this qualitative description will be used.

5. Honeycombed concrete, irrespective of its strength, has a greater effect on shear behaviour when the strength of the normal concrete is high. This can be attributed to the fact that concrete of higher strength is more brittle. With a weak concrete present in the high shear zone, a more adverse effect results.

6. With the exception of a void, which simulates the lowest possible strength of honeycombed concrete, there is no significant difference in the effect on shear behaviour of beams with a high strength of normal concrete but with different level of strength of honeycombed concrete of medium and low strengths. This shows that within the range of relative strength of the honeycombed concrete studied in this current work, (i.e medium and low strengths), the strength of the honeycombed

concrete has no significant effect in influencing the magnitude of the effect on shear behaviour.

7. Test results show that within the scope of the current work, the size of the honeycombed zone up to approximately one-half of the beam effective depth is also insignificant in determining the magnitude of the effect on shear capacity of a beam.

8. The effect of a honeycombed zone is significant in beams with a shear span ratio of 2.0. For beams with a shear span of 3.5, the effect is small. As a result of that the following conclusions refer to beams with a shear span ratio of 2.0, unless stated otherwise.

9. The presence of shear reinforcement in a concrete beam can be very effective in mitigating the adverse effect caused by the honeycombed zone.

10. The flexural stiffness of a beam with a honeycombed zone is not affected. The reduction in the stiffness only occurs once the diagonal crack has formed. This however only occurs in beams which have a low reserve of strength. The reserve of strength is the percentage of load that the beam can sustain after the formation of diagonal cracking before it reaches the ultimate failure. For beams with high reserve of strength the above phenomenon never occurs. Their stiffnesses are unchanged until they reach the ultimate failure. This indicates that the measurement of flexural stiffness in a load test for assessment, will not be able to detect the presence of a honeycombed zone in the high shear region.

11. The mode and the magnitude of the effect on shear are influenced by the position of the honeycombed zone and also the original shear transfer mechanism that takes place in a beam without a honeycombed zone.

12. A zone of honeycombed concrete at the centre of a shear region, which is at the mid-point of the potential concrete compressive strut, has the greatest influence on shear

behaviour. For beams with high strength of normal concrete and with medium strength of honeycombed concrete, the diagonal cracking load when the honeycombed zone is about a third of the effective depth is only 64% of the control. For beams with a high strength of normal concrete but with a low strength of honeycombed concrete the value is about 78% of the control. The values show the insignificant effect of the strength of the honeycombed concrete on the shear behaviour. In terms of ultimate capacity, the shear strengths are only in the order of 47% of the control in the former and 65% in the latter case.

13.For a beam with medium strength of normal concrete and low strength of honeycombed concrete, the value for the diagonal cracking load is 79% of the control. The ultimate shear strength is about 89% of the control.

14. For a honeycombed zone of about half of the effective depth and located at the centre of shear region, the percentage of diagonal cracking load relative to the control is 71% for beams with a high strength of normal concrete, and 98% in beams with medium strength of normal concrete respectively. Note that both have a low strength of honeycombed concrete. Their ultimate shear strengths are 73% and 92% of the control respectively.

15.The observations showed that, irrespective of the strength of the honeycombed concrete, beams with a high strength of normal concrete and with a honeycombed zone at the centre of shear region can also change the mode of diagonal cracking from flexurally formed to independently formed diagonal cracking and results in a brittle ultimate failure.

16.A honeycombed zone at the bottom tension zone near the support, can divert the path of diagonal cracking close to the support, and a brittle failure can occur through the anchorage. For beams with a high strength of normal concrete and a medium strength of honeycombed concrete, the reserve of strengths is as low as 4.6%. For beams with medium strength of normal concrete and with a low strength of honeycombed concrete the reserve of strength is 26.7%. Their respective ultimate shear strengths compared to the control are found to be only 49% in the former and 80% in the latter case.

17. A zone of honeycombed concrete at the top close to the point of loading, which is at the upper end of the potential compressive strut, only reduces the diagonal cracking load of beams with a high strength of normal concrete and medium strength of honeycombed concrete. At failure, the reduction in the ultimate load compared to the control beam is 36%. This beam and also a beam with a medium strength of normal concrete and a low strength failed due to crushing in the honeycombed zone and their reserve of strengths are in the order of 51.5% and 38.7% respectively.

18. A honeycombed zone in the bottom tension zone at the mid-shear span position can prematurely accelerate the formation of a flexural crack which can initiate the early formation of diagonal cracking. The tests demonstrated that the early diagonal cracking would not however pose any immediate danger to the safety of the beam as it possesses a very high reserve of strength before reaching the ultimate failure. The reserve of strength is 188.5%. The beam has a high strength of normal concrete and a medium strength of honeycombed concrete.

19. Honeycombed zones at other locations in beams with a high strength of normal concrete have quite a significant influence both on the diagonal cracking load and the ultimate load, irrespective of the strength of the honeycombed concrete. For the beam with a medium strength of honeycombed concrete, a honeycombed zone located along the neutral axis close to the support reduces the diagonal cracking load and the ultimate load by up to 21% and 51% respectively. For beam with a low strength of honeycombed concrete, the reductions are 19% and 28% respectively. The honeycombed zone can also cause a brittle failure with a reserve of strength of only 20.4% for a beam in the former category, but with a higher value of 65.6% in the latter category. The values indicate that, although the zone of honeycombed concrete is far from the critical shear zone, the effects can be very significant.

20.Also along the neutral axis, but close to the loading point, a honeycombed zone reduces the diagonal cracking load and the ultimate load by 29% and 36% respectively for the beams with a medium strength of honeycombed concrete and 10% and 19% respectively for beams with a low strength of honeycombed concrete. However, the failure of the beams were not particularly brittle, with values reserve of strength of 73.7% and 68.6% in the former and latter respectively. The effect of a honeycombed zone is about in the same magnitude when it is located at the top middle of shear span.

21.For beams with medium strength of normal concrete the effects of a honeycombed zone at other locations are not significant except that it reduces the diagonal cracking load by only 18% when it is located along the neutral axis, close to the loading point.

7.1.2 BS 8110 and BD 44/95 Predictions

1. Using the strength of the normal concrete, BS 8110 may not be able to adequately predict the diagonal cracking load of honeycombed beams with a shear span ratio of 2.0, especially beams with a higher strength of normal concrete, when the honeycombed zone is located along the longitudinal axis and at the bottom middle of shear span. If the evaluation is carried out using the strength of the honeycombed concrete, two locations are unsafe, at the centre of the shear region and at the bottom middle of the shear span.

2. For beams with a shear span of 3.5, using the normal concrete strength, BS 8110 predicts lower diagonal cracking loads for both the control and the honeycombed beams. Using the strength of the honeycombed concrete safe predictions can be obtained for all honeycombed beams.

3. The results indicate that for beams with a shear span of 2.0, a safe and realistic assessment can be obtained if it is based on the ultimate shear capacity rather than the diagonal cracking load capacity.

4. BD 44/95 can provide a safe assessment for all beams, with the exception for beams with a honeycombed zone at the centre of shear span and also at the bottom middle section of the shear region.

5. However, using BD 44/95 may also lead to a high degree of conservatism in the assessment especially if the strength of the honeycombed concrete is used. In practice this may lead to unnecessary rehabilitation work on existing structures. Generally, using the strength of normal concrete to evaluate the ultimate capacity will be safe, except when a honeycombed zone is at the centre of the shear region for beam with a shear span ratio of 3.5.

6. For honeycombed beams with shear reinforcement, BS 8110 and BD 44/95 can safely predict the ultimate shear capacity.

7.1.3 Analytical Results

1. The elastic finite element analysis is unable to give a satisfactory prediction of the distribution of stresses within the shear zone. The stress pattern in a beam with a honeycombed zone as observed in the tests cannot be predicted by the method. This can be anticipated as it has been found that the transfer of shear in concrete beams involves a complex mechanism.

2. The existing plastic analysis can be modified to take account of a honeycombed zone by using a 'weighted average' strength for the concrete but basing the concrete

effectiveness factor on the normal concrete strength, and applying a condition factor of 1.33.

3. The proposed modification to the plastic analysis can be an alternative solution in assessing a honeycombed beam without shear reinforcement. If the ratio of ultimate shear capacity of the test to the proposed modified analytical method is averaged for all honeycombed beams without shear reinforcement the value is 1.57. Comparing the ultimate shear capacity of the honeycombed beams with the values predicted by BS 8110, using the strength of the normal concrete, the average ratio is also 1.57. If the honeycombed concrete strength is used the average ratio is 2.19. The values given by BD 44/95 are 1.67 and 2.36 respectively.

4. The values above show that the proposed modified plastic analytical solution gives a more realistic prediction. Note that the average values are calculated based on beams of all series of tests. If beams with a high strength of normal concrete and beams with a medium strength of normal concrete are evaluated separately, a plastic solution with a more appropriate safety margin can be produced.

5. For honeycombed beams with shear reinforcement, the proposed approach in which the normal concrete strength is used to evaluate the effectiveness factor and a 'weighted average' strength is used to evaluate the shear strength using the existing plastic method is adequate without any need to apply a condition factor. However this is based only on beams with honeycombed zone at the centre of the shear region.

7.1.4 Beams With A Construction Joint

1. Beams with a honeycombed zone simulating a poor construction joint fail in shear along the line of the joint. They need to be treated as special cases.

2. The proposed method of plastic analysis predicts strengths which are very close to the test results.

3. An analytical study of the shear behaviour of beams with a construction joint have identified various modes of possible shear failure, which provides valuable assistance to the assessing engineer. However, further tests are required for validation.

7.2 RECOMMENDATIONS FOR FUTURE WORK

It should be emphasised that no report of a similar nature of work has been found in the literature searched. As such the current work is not expected to provide a complete and satisfactory solution. Shear in general is a complex phenomenon, and it has been shown in numerous previous research works that no single general solution can be formulated which produces a close prediction without having a factor calibrated from test data.

A clear result from the current work is that a honeycombed zone located in the high shear region can cause an adverse effect on the shear capacity of the beam. Thus, assessing engineers need more information from test data, to assist them to make a realistic and appropriate assessment.

Following the results of the current work, future work should address the following:

1. The current work is mainly devoted to study the effect on beams with a shear span ratio of 2.0. Although tests on specimens with a shear span ratio of 3.5 showed no significant effect, there is a need to study the effect on beams with for example a shear span ratio of 2.5 or 3.0. Also it may be worth investigating beams with a shear span less than 2.0. This because unless evidence is found from the experimental work, it is difficult to predict the behaviour by interpolating or extrapolating from the current tests. Note that for the beam with a shear span ratio of 3.5, the current tests only

investigated one location of the honeycombed zone. Probably other locations of the honeycombed zone, especially at the bottom middle of the shear span, need to be examined.

2. The current work is insufficient to provide a conclusive guidance as to which locations of a honeycombed zone need to be treated very seriously and at which locations it can be ignored. Further tests should be able to produce a map in which the critical location can be marked and would provide useful assistance to assessing engineers.

3. A more detailed analysis on the existing data could be carried out. This would include the possibility of analysis in more detail of the effect of the honeycombed zone according to its location for all series of beams, and taking into account the failure modes observed.

4. Investigations should continue in order to be certain on the lower and upper limits of honeycombed strength, in relation to the strength of the normal concrete, at which it can cause an adverse effect on the shear capacity of a beam.

5. Another area of further work is the possibility of using the beam stiffness and/or crack length as parameters to evaluate the shear strength of a honeycombed beam.

6. Further investigation should be carried out to study the effect of other shapes of honeycombed zone which includes for example, a circular honeycombed zone. Such a shape would result in lower stress concentrations than those induced by the square zone used in the current study.

7. The optimum size of a honeycombed zone which can cause an adverse effect should also be investigated.

8. A combination of more test results can be used to calibrate with the plastic analysis. Probably a condition factor can be determined based on more variables, such as the strength ratio of normal and honeycombed concrete, different shear span ratios, and different dimensions of the cross section of a beam. The location of a honeycombed zone can also be included. This certainly requires a large number of tests.

9. Further experimentation to study the effect of a uniformly distributed load (UDL) instead of a point load should also be carried out, because the type of loading influences the development of the failure mode after cracking and the strength reserve between crack formation and the maximum load.

10. For beams with shear reinforcement there is a need to examine cases such as when a honeycombed zone is located at other locations than at the centre of shear region.

11. Another area for future work is to verify experimentally the mode of shear behaviour in a beam with a construction joint produced by the analytical work.

12. A more detailed non-linear finite element analysis can be used to verify the stress distribution within the shear region due to the presence of a honeycombed zone. However, it should be noted that a correct model of behaviour of honeycombed concrete needs to be determined.

13. The analytical methods such as the modified compression field theory can probably give a good solution provided that the honeycombed concrete can be modelled accurately. Further research is worthwhile.

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Plate A-1 Honeycombed concrete prism

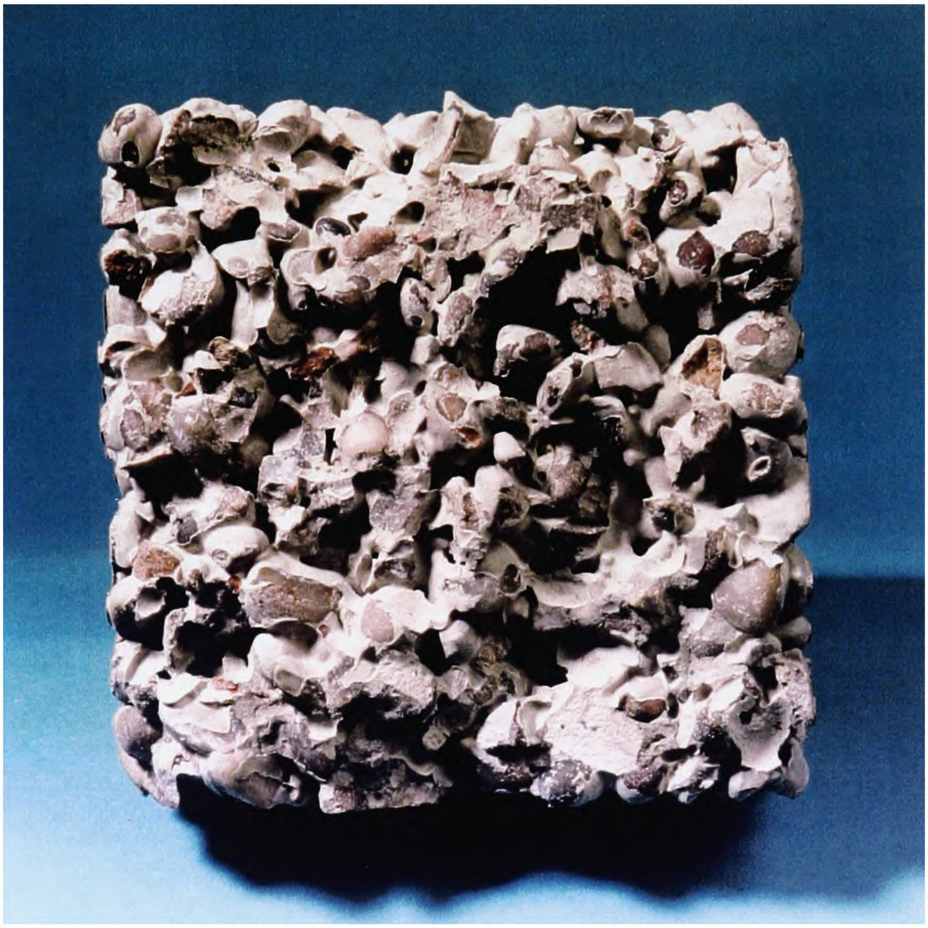

Plate A-2: Voids in honeycombed prism-close-up view

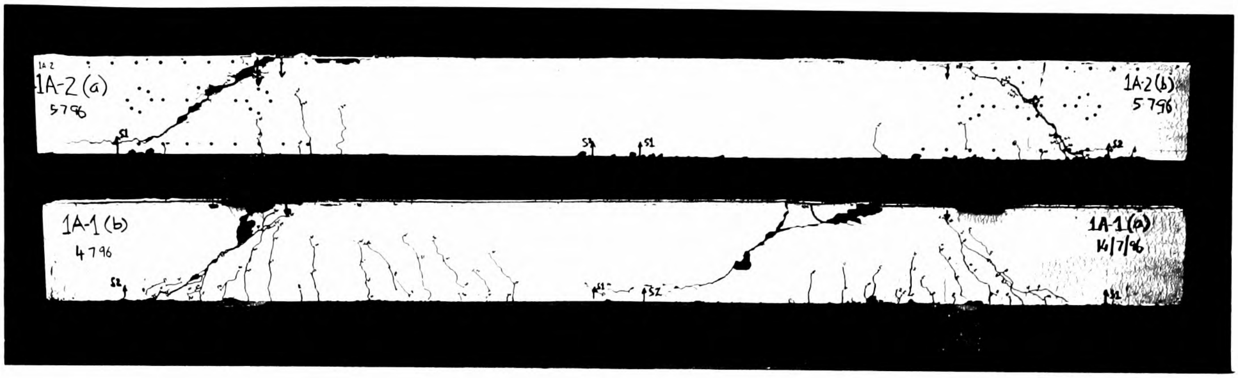

Plate B-1 Beams 1A-1 and 1A-2

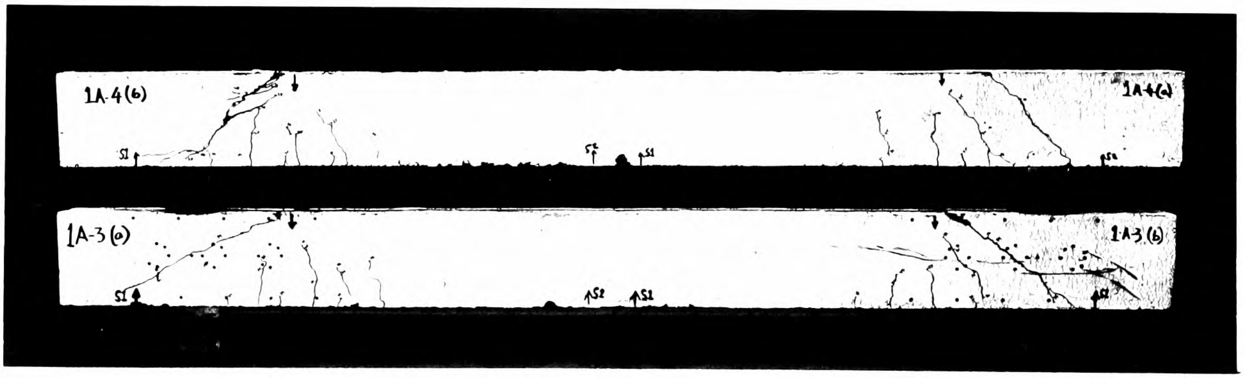

Plate B-2 Beams 1A-3 and 1A-4

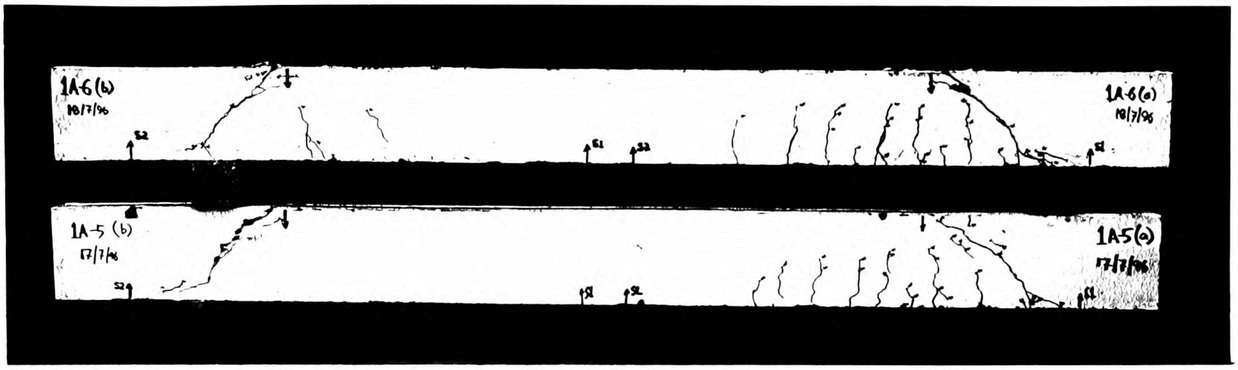

Plate B-3 Beams 1A-5 and 1A-6

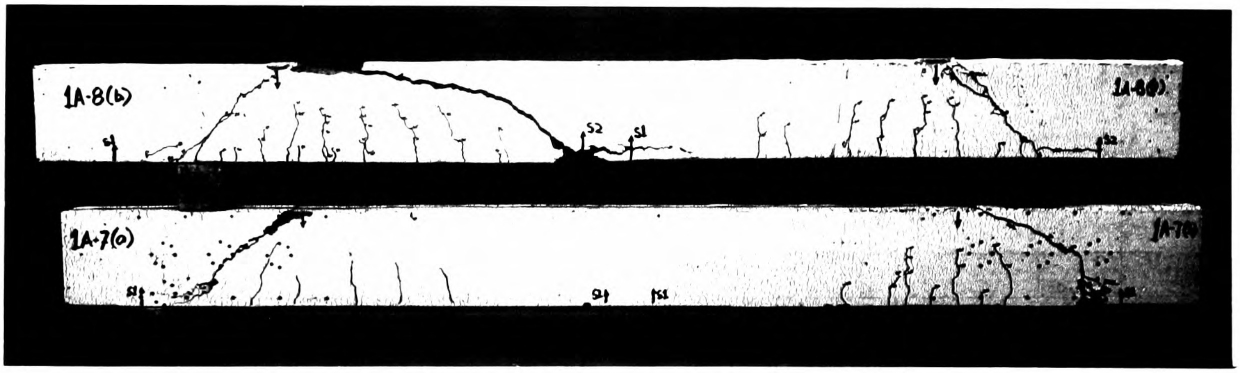

Plate B-4 Beams 1A-7 and 1A-8

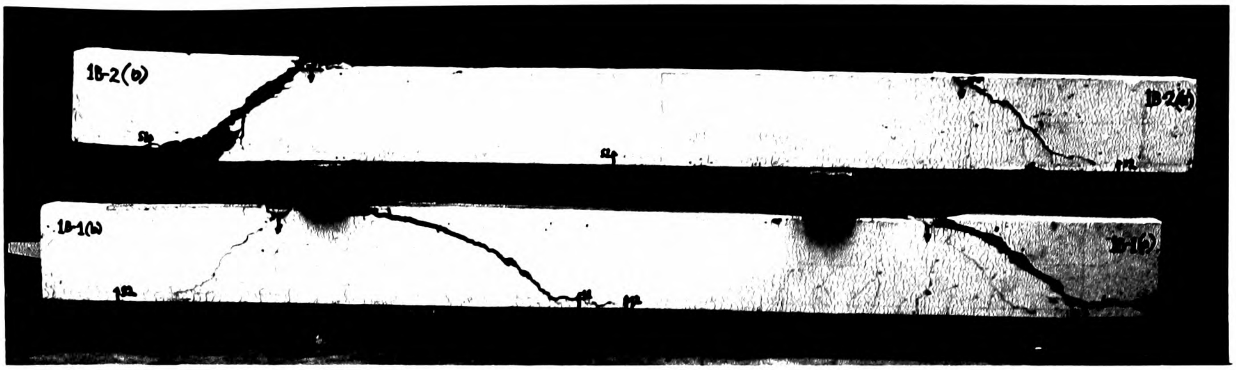

Plate B-5 Beams 1B-1 and 1B-2

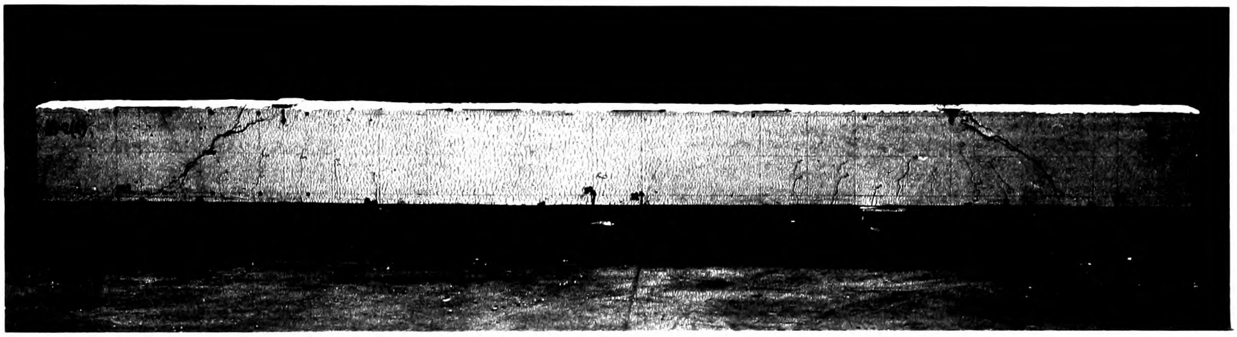

Plate B-6 Beams 1B-1(R) and 1B-2(R)

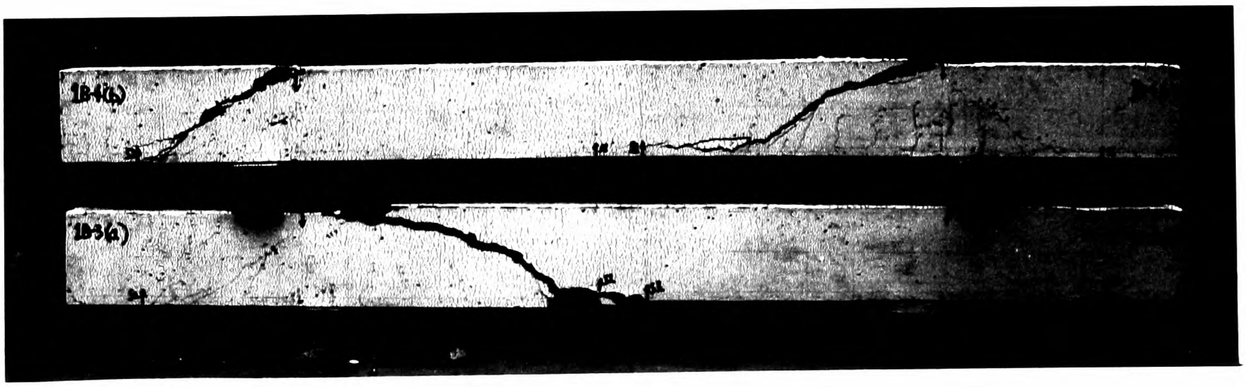

Plate B-7 Beams 1B-3 and 1B-4

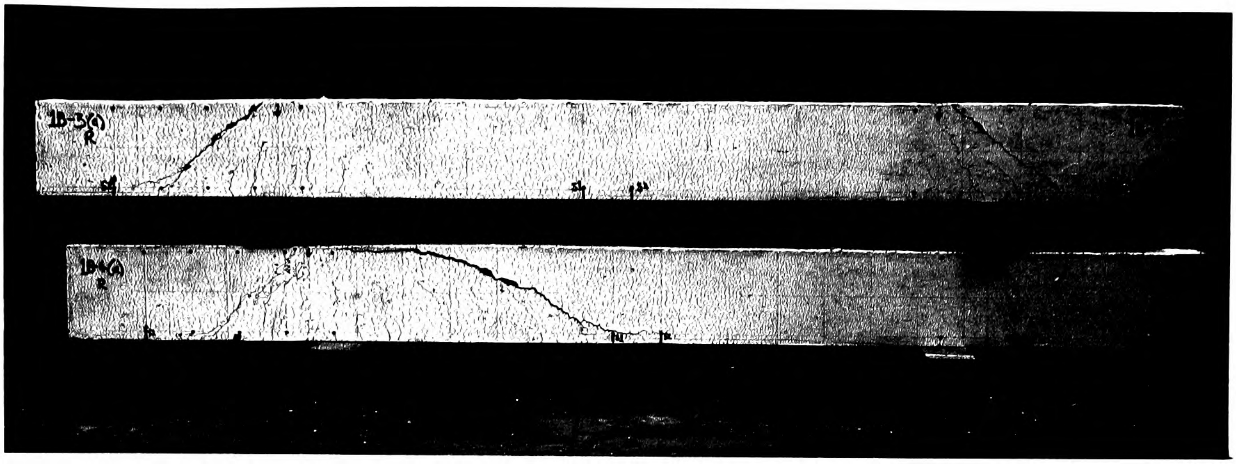

Plate B-8 Beams 1B-3(R) and 1B-4(R)

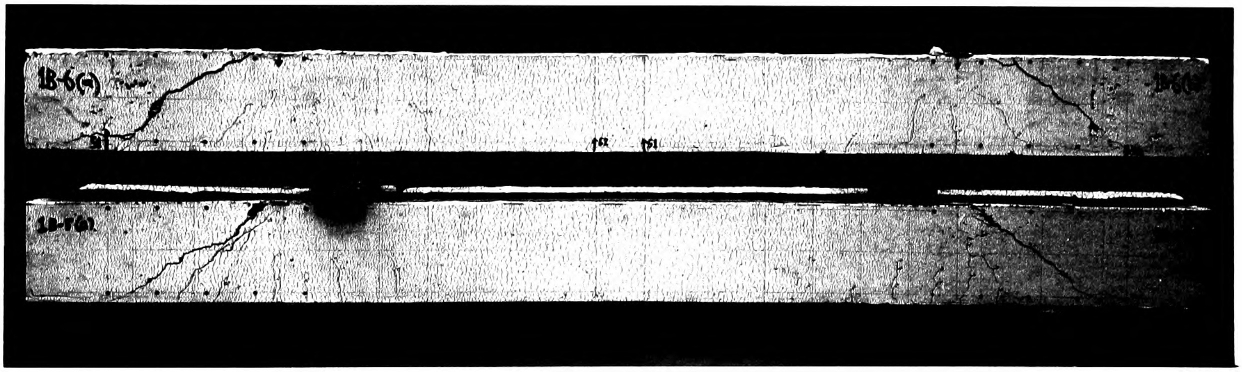

Plate B-9 Beams 1B-5 and 1B-6

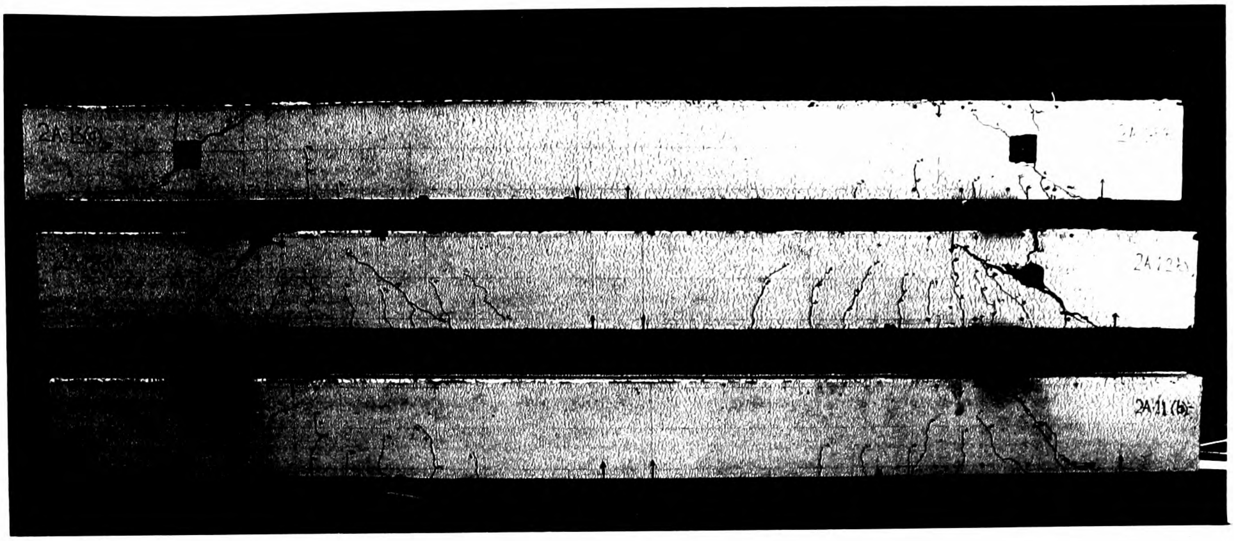

Plate B-10 Beams 2A-1.1, 2A-1.2 and 2A-1.3

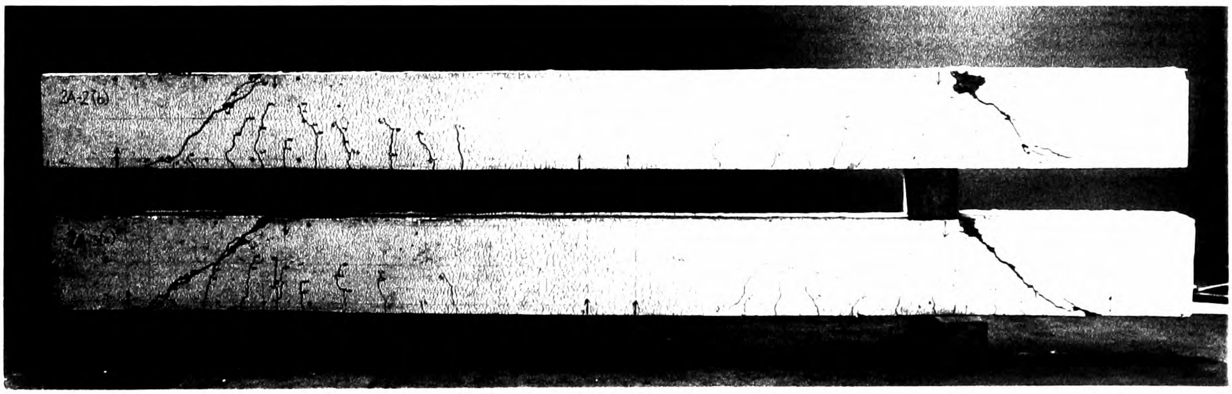

Plate B-11 Beams 2A-2 and 2A-3

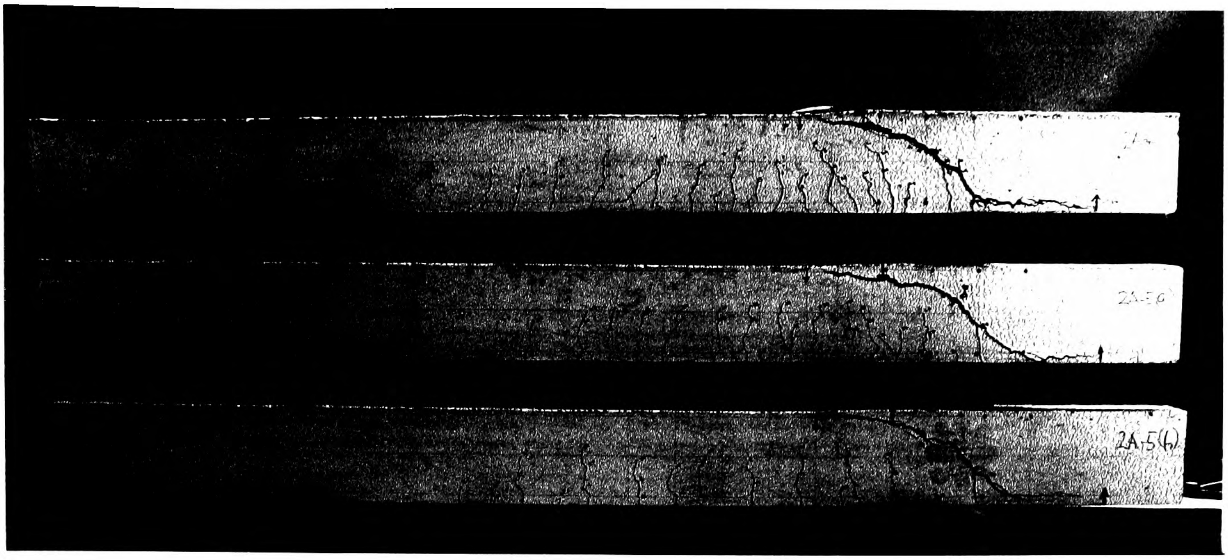

Plate B-12 Beams 2A-4 and 2A-5

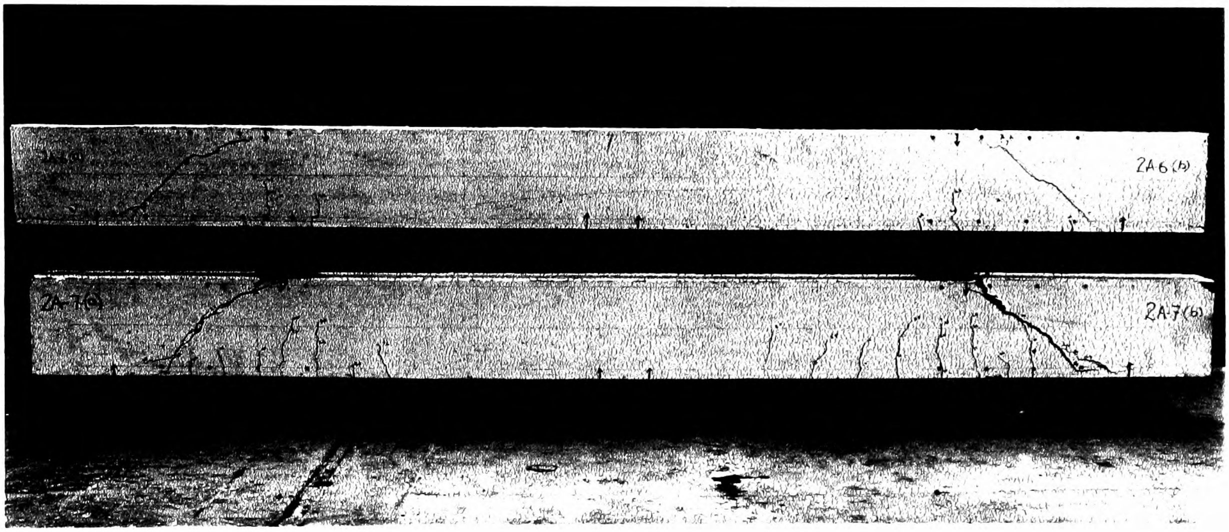

Plate B-13 Beams 2A-6 and 2A-7

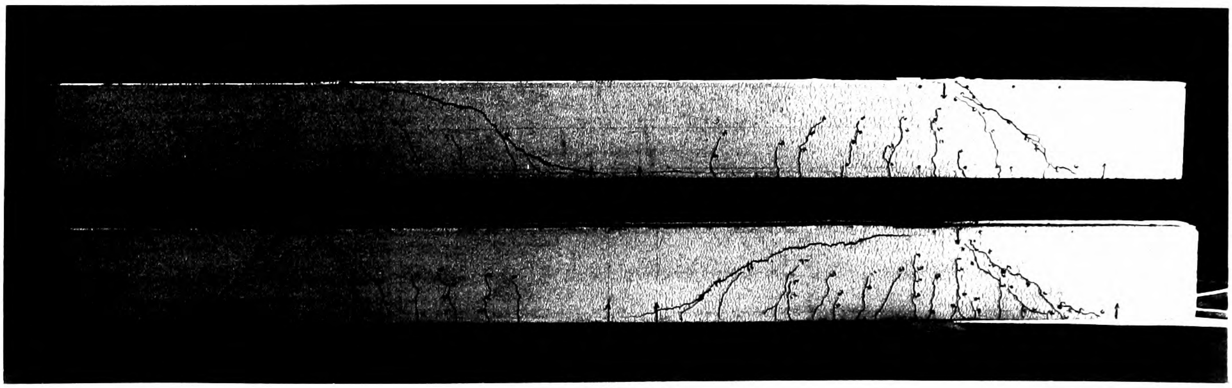

Plate B-14 Beams 2A-8 and 2A-9

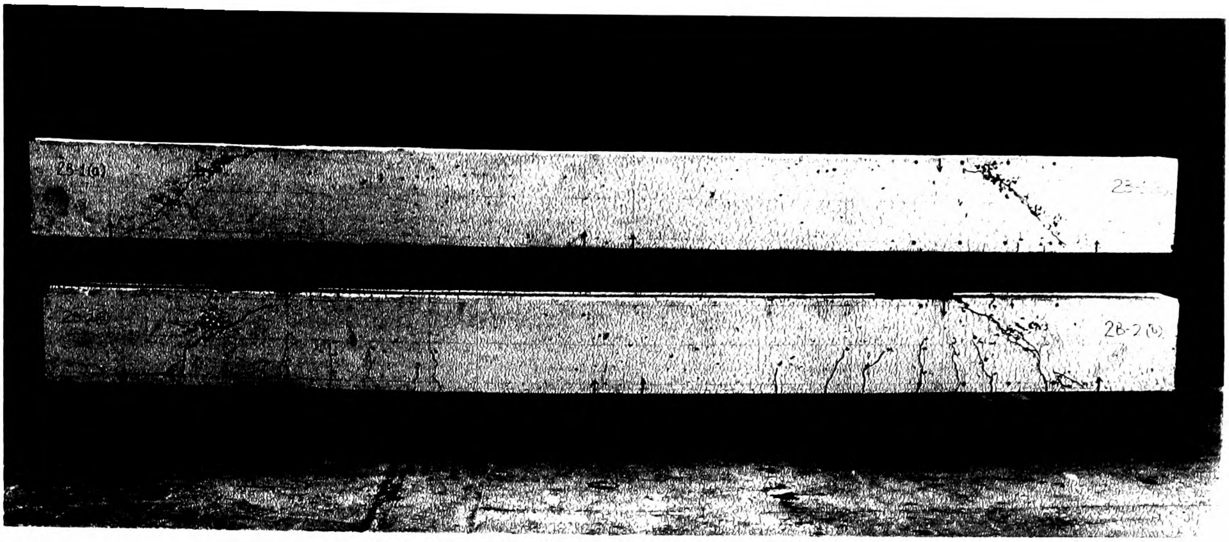

Plate B-15 Beams 2B-1 and 2B-2

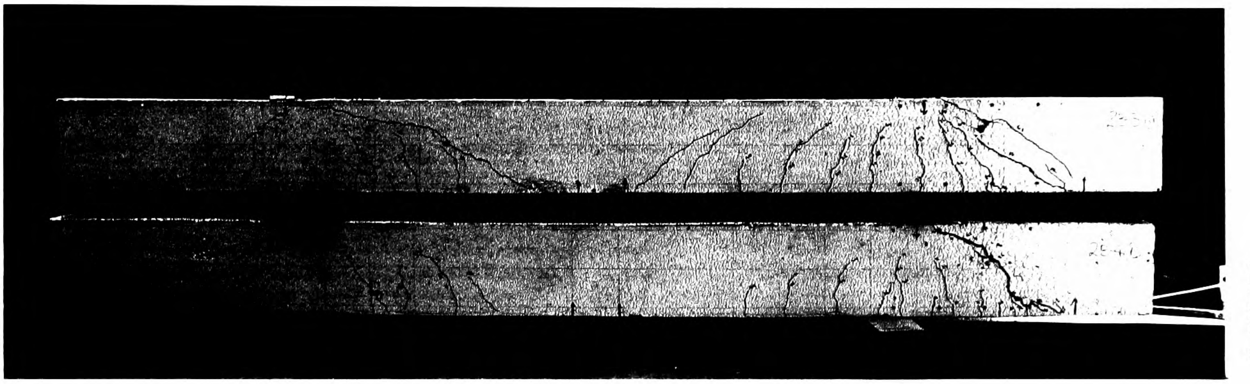

Plate B-16 Beams 2B-3 and 2B-4

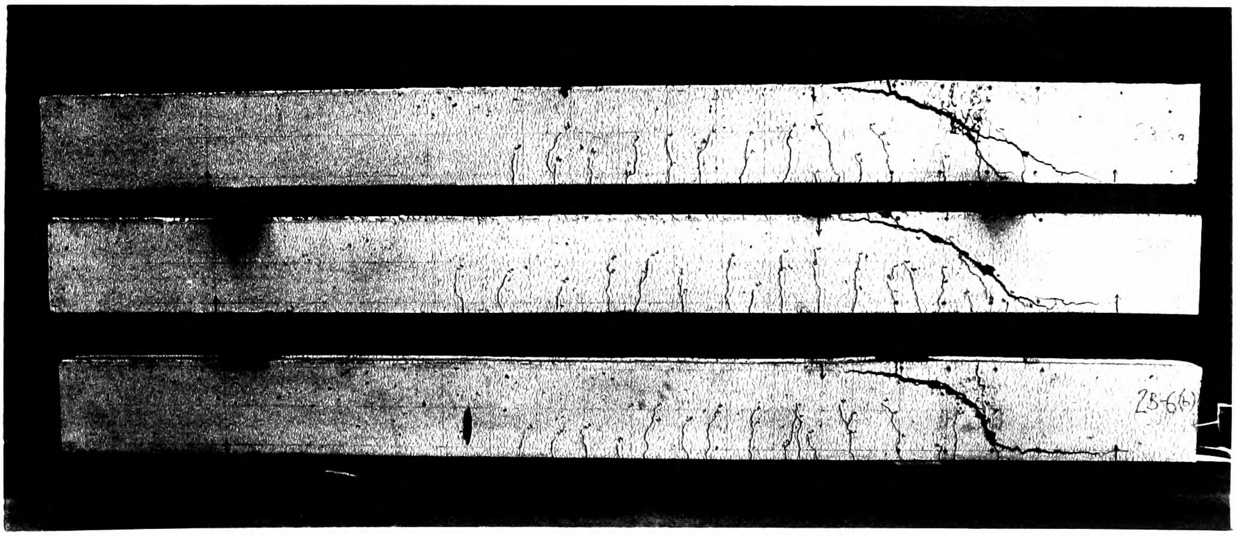

Plate B-17 Beams 2B-5 and 2B-6

```
C                         SHEAR 1
C
C************************************************************
C                   PROGRAM PLASTIC ANALYSIS
C EVALUATE SHEAR IN BEAM WITHOUT HONEYCOMBED ZONE
C************************************************************
C
      REAL BETAL(20),DBETAL(20)

      COMMON/C1/WIDTH,B,SSPAN,FCC,RHO,HEIGHT,
     * EFF,SHEAR,XH(20),XHI

      OPEN (UNIT=2,FILE='CONT-1.DAT')
      OPEN (UNIT=3,FILE='CONT-1.OUT')

C****** READ BEAM DIMENSION AND HONEYCOMBING LOCATION

      READ (2,100) SPAN,HEIGHT,WIDTH,SSPAN
 100  FORMAT (F6.1,1X,3(1X,F5.1))

C*******READ CONCRETE CUBE STRENGTH, REINFORCEMENT
C     AND CONVERSION FACTOR (CUBE TO CYLINDER)

      READ (2,101) FCU,CFC,AS
 101  FORMAT (F4.1,1X,F4.2,1X,F6.1)

C******EQUIVALENT CONCRETE CYLINDER STRENGTH, RHO

      FCC=CFC*FCU
      RHO=100*AS/(WIDTH*HEIGHT)

      WRITE (3,200) SPAN,HEIGHT,WIDTH,SSPAN,FCU,CFC,AS,FCC,RHO

 200  FORMAT (/,2X,'SPAN(MM)',8X,F6.1,/,2X,'HEIGHT(MM)',6X,F5.1,/,2X,
     * 'WIDTH(MM)',7X,F5.1,/,2X,'SHEAR SPAN(MM)',2X,F5.1,//,2X,
     * 'FCU(N/MM2)',5X,F4.1,/,2X,'CFC'13X,F4.2,/,2X,'AS(MM2)',
     * 9X,F6.1,//,2X,'FCC(N/MM2)',5X,F4.1,/,2X,
     * 'RHO(%)',10X,F4.2)
C
C NUMBER OF POINTS CONSIDERED,NP
      READ (2,110) NP
 110  FORMAT(I3)
      WRITE (3,210) NP
 210  FORMAT (/,2X,'NUMBER OF POINTS',3X,I3)

C READ XH(I)

      DO 10 I=1,NP
      READ (2,120) XH(I)
 120  FORMAT (F5.1)
      WRITE (3,121) XH(I)
 121  FORMAT (F5.1)
 10   CONTINUE

C  EVALUATE BETA L IN RADIAN

      DO 20 I=1,NP
```

```
      BETAL(I)=ATAN(HEIGHT/(SSPAN-XH(I)))

C BETA IN DEGREE

      IF (BETAL(I).GT.0) THEN
      DBETAL(I)=BETAL(I)*45.0/ATAN(1.0)
      ELSE
      DBETAL(I)=180.0+BETAL(I)*45.0/ATAN(1.0)
      END IF


      WRITE (3,230) XH(I),DBETAL(I)
 230  FORMAT (/,6X,'FOR X=',F5.1,//,2X,'BETA L',6X,F6.2)

20    CONTINUE

C EVALUATE SHEAR (AT POSITIONS, XH, BETA L)
      WRITE (3,245)
 245  FORMAT (/,4X,'BETA L')
      DO 70 I=1,NP
         B=BETAL(I)
         XHI=XH(I)
         CALL EVSH

 70   CONTINUE
      END

      SUBROUTINE EVSH
C--------------------------------
      COMMON/C1/WIDTH,B,SSPAN,FCC,RHO,HEIGHT,
     * EFF,SHEAR,XH(20),XHI

         XLM=HEIGHT/SIN(B)

C*******EFF FACTOR***

      F1=3.5/SQRT(FCC)
      F2=0.27*(1+1/SQRT(HEIGHT*0.001))
      F3=0.15*RHO+0.58
      F4=1.0+0.17*((SSPAN/HEIGHT-2.6)**2)

      EFF=F1*F2*F3*F4

      SHEAR=0.5*WIDTH*(1-COS(B))*(EFF*FCC*XLM)*0.001
      BETA=B*45.0/ATAN(1.0)
      WRITE (3,300) XHI,BETA,FCC,EFF,SHEAR
 300  FORMAT (/,2X,F5.1,4X,F5.2,12X,F4.1,10X,F5.3,9X,F6.2)
      RETURN
      END
```

```
C                         SHEAR 2
C
C**********************************************
C                 PROGRAM PLASTIC ANALYSIS
C            FOR BEAM WITH HONEYCOMBED ZONE
C          USING AVERAGE STRENGTH FOR STRENGTH
C USING NORMAL CONCRETE FOR EFFECTIVENESS FACTOR
C**********************************************
C
      REAL BETAA(20),BETAB(20),BETAC(20),BETAD(20),BETAL(20),
     * DBETAA(20),DBETAB(20),DBETAC(20),DBETAD(20),DBETAL(20),
     * VC(20)
      COMMON/C1/WIDTH,B,SSPAN,FCC,FHC,RHO,HEIGHT,HV2,HH1,AVFC,
     * EFFAV,SHEAR,HVC,HHC,HH2,HV1,XH(20),VCI,XHI

      OPEN (UNIT=2,FILE='PLAS11-2.DAT')
      OPEN (UNIT=3,FILE='PLAS11-2.OUT')

C****** READ BEAM DIMENSION AND HONEYCOMBING LOCATION

      READ (2,100) SPAN,HEIGHT,WIDTH,SSPAN,HV1,HV2,HH1,HH2
 100  FORMAT (F6.1,1X,7(1X,F5.1))

      HVC=HV2-HV1
      HHC=HH2-HH1

C*******READ CONCRETE CUBE STRENGTH, REINFORCEMENT
C     AND CONVERSION FACTOR (CUBE TO CYLINDER)

      READ (2,101) FCU,FCH,CFC,CFH,AS
 101  FORMAT (F4.1,1X,F4.1,1X,F4.2,1X,F4.2,1X,F6.1)

C******EQUIVALENT CONCRETE CYLINDER STRENGTH, RHO

      FCC=CFC*FCU
      FHC=CFH*FCH
      RHO=100*AS/(WIDTH*HEIGHT)

      WRITE (3,200) SPAN,HEIGHT,WIDTH,SSPAN,HV1,HV2,HH1,HH2,FCU,FCH,
     * CFC,CFH,AS,FCC,FHC,RHO,HVC,HHC

 200  FORMAT (/,2X,'SPAN(MM)',8X,F6.1,/,2X,'HEIGHT(MM)',6X,F5.1,/,2X,
     * 'WIDTH(MM)',7X,F5.1,/,2X,'SHEAR SPAN(MM)',2X,F5.1,/,2X,'HV1(MM)',
     * 10X,F5.1,/,2X,'HV2(MM)',10X,F5.1,/,2X,'HH1(MM)',
     * 10X,F5.1,/,2X,'HH2(MM)',10X,F5.1,//,2X,'FCU(N/MM2)',5X,F4.1,/,
     * 2X,'FCH(N/MM2)',5X,F4.1,/,2X,'CFC'13X,F4.2,/,2X,'CFH',13X,F4.2
     * /,2X,'AS(MM2)',9X,F6.1,//,2X,'FCC(N/MM2)',5X,F4.1,/,2X,
     * 'FHC(N/MM2)',5X,F4.1,/,2X,'RHO(%)',10X,F4.2,//,2X,'HVC(MM)',10X,
     * F5.1,/,2X,'HHC(MM)',10X,F5.1)
C
C NUMBER OF POINTS CONSIDERED,NP
      READ (2,110) NP
 110  FORMAT(I3)
      WRITE (3,210) NP
 210  FORMAT (/,2X,'NUMBER OF POINTS',3X,I3)

C READ XH(I)
```

```
      DO 10 I=1,NP
      READ (2,120) XH(I)
 120   FORMAT (F5.1)
      WRITE (3,121) XH(I)
 121   FORMAT (F5.1)
 10    CONTINUE

C  EVALUATE BETA A, BETA B, BETA C, BETA D, BETA L IN RADIAN

      DO 20 I=1,NP

      BETAA(I)=ATAN(HV2/(HH1-XH(I)))
      BETAB(I)=ATAN(HV2/(HH2-XH(I)))
      BETAC(I)=ATAN(HV1/(HH1-XH(I)))
      BETAD(I)=ATAN(HV1/(HH2-XH(I)))
      BETAL(I)=ATAN(HEIGHT/(SSPAN-XH(I)))

C BETA IN DEGREE

      IF (BETAA(I).GE.0) THEN
      DBETAA(I)=BETAA(I)*45.0/ATAN(1.0)
      ELSE
      DBETAA(I)=180.0+BETAA(I)*45.0/ATAN(1.0)
      END IF
      IF (BETAB(I).GE.0) THEN
      DBETAB(I)=BETAB(I)*45.0/ATAN(1.0)
      ELSE
      DBETAB(I)=180.0+BETAB(I)*45.0/ATAN(1.0)
      END IF
      IF (BETAC(I).GE.0) THEN
      DBETAC(I)=BETAC(I)*45.0/ATAN(1.0)
      ELSE
      DBETAC(I)=180.0+BETAC(I)*45.0/ATAN(1.0)
      END IF
      IF (BETAD(I).GE.0) THEN
      DBETAD(I)=BETAD(I)*45.0/ATAN(1.0)
      ELSE
      DBETAD(I)=180.0+BETAD(I)*45.0/ATAN(1.0)
      END IF
      IF (BETAL(I).GE.0) THEN
      DBETAL(I)=BETAL(I)*45.0/ATAN(1.0)
      ELSE
      DBETAL(I)=180.0+BETAL(I)*45.0/ATAN(1.0)
      END IF


      WRITE (3,230) XH(I),DBETAA(I),DBETAB(I),DBETAC(I),
     * DBETAD(I),DBETAL(I)
 230   FORMAT (/,6X,'FOR X=',F5.1,//,2X,'BETA A',6X,F6.2,/,
     * 2X,'BETA B',6X,F6.2,/,2X,'BETA C',6X,F6.2,/,2X,
     * 'BETA D',6X,F6.2,/,2X,'BETA L',6X,F6.2)

 20    CONTINUE

C EVALUATE SHEAR (AT POSITIONS, XH, BETA A)
      WRITE (3,240)
 240   FORMAT (/,4X,'BETA A')
```

```
      DO 30 I=1,NP

      IF (DBETAA(I).LE.DBETAL(I)) GO TO 30

      IF (DBETAA(I).LT.90.0) THEN
           B=BETAA(I)
           XHI=XH(I)
           VC(I)=HV2-(HH1-XH(I))*TAN(B)
           VCI=VC(I)
           CALL EVSH1
      END IF
 30   CONTINUE

C EVALUATE SHEAR (AT POSITIONS, XH BETA B)
      WRITE (3,241)
 241  FORMAT (/,4X,'BETA B')

      DO 40 I=1,NP
           B=BETAB(I)
           XHI=XH(I)

      IF (DBETAB(I).LE.DBETAL(I)) GO TO 40
      IF (DBETAB(I).GE.90.0) GO TO 40

      IF (HV1.EQ.0.0) GO TO 400

      IF (DBETAB(I).GT.DBETAC(I)) THEN
            VC(I)=HV2-(HH1-XH(I))*TAN(B)
            VCI=VC(I)
            CALL EVSH1
      ELSE
           CALL EVSH3
      END IF
      GO TO 40
 400  IF (XH(I).LE.HH1) THEN
           VC(I)=HV2-(HH1-XH(I))*TAN(B)
           VCI=VC(I)
           CALL EVSH1
      ELSE
           CALL EVSH3
      END IF

 40   CONTINUE

C EVALUATE SHEAR (AT POSITIONS, XH, BETA C)
      WRITE (3,243)
 243  FORMAT (/,4X,'BETA C')

      DO 50 I=1,NP
           B=BETAC(I)
           XHI=XH(I)

      IF (HV1.EQ.0.0) GO TO 50
      IF (DBETAC(I).LE.DBETAL(I)) GO TO 50
      IF (DBETAC(I).GT.90.0) GO TO 50

      IF (DBETAC(I).LE.DBETAB(I)) THEN

           CALL EVSH2
```

```
      ELSE
           CALL EVSH3
      END IF
 50   CONTINUE
C EVALUATE SHEAR (AT POSITIONS, XH, BETA L)
      WRITE (3,245)
 245  FORMAT (/,4X,'BETA L')
      DO 70 I=1,NP
           B=BETAL(I)
           XHI=XH(I)
      IF (HV1.EQ.0.0) GO TO 500
C      IF (DBETAL(I).LE.DBETAD(I)) GO TO 70

      IF (DBETAL(I).GE.DBETAA(I)) THEN
           VCI=0
           CALL EVSH1
      ELSE IF (DBETAL(I).LE.DBETAD(I)) THEN
           VCI=0
           CALL EVSH1
      ELSE
           GO TO 600
      END IF
      GO TO 70
 500  IF (XH(I).GE.HH2) GO TO 70
      IF (DBETAL(I).GE.DBETAA(I)) THEN
           VCI=0
           CALL EVSH1
      ELSE IF (DBETAL(I).LE.DBETAD(I)) THEN
           VCI=0
           CALL EVSH1
      ELSE

           IF (XH(I).LE.HH1) THEN

                IF (DBETAL(I).LE.DBETAB(I)) THEN
                     CALL EVSH2
                ELSE
                     VC(I)=HV2-(HH1-XH(I))*TAN(B)
                     VCI=VC(I)
                     CALL EVSH1
                END IF
           ELSE
                IF (DBETAL(I).GE.DBETAB(I)) THEN
                     CALL EVSH3
                ELSE
                     VC(I)=(HH2-XH(I))*TAN(B)-HV1
                     VCI=VC(I)
                     CALL EVSH1
                END IF
           END IF
      END IF
      GO TO 70
600   IF (DBETAL(I).GE.DBETAC(I)) THEN
           IF (DBETAL(I).GE.DBETAB(I)) THEN
                VC(I)=HV2-(HH1-XH(I))*TAN(B)
                VCI=VC(I)

                CALL EVSH1
           ELSE
```

```
            CALL EVSH2
         END IF
      ELSE
         IF (DBETAL(I).GE.DBETAB(I)) THEN
            CALL EVSH3
         ELSE
            VC(I)=(HH2-XH(I))*TAN(B)-HV1
            VCI=VC(I)
            CALL EVSH1

         END IF
      END IF
 70   CONTINUE

C EVALUATE BETA INTERMEDIATE
C     BETAAB=(BETAA+BETAB)/2
C     BETABL=(BETAB+BETAL)/2


C     WRITE (3,203)
C 203   FORMAT (/,2X,'BETA',6X,'AVG CONCR STRENGTH',2X,'EF FAC',
C    * 9X,'SHEAR')

      RETURN
      END

      SUBROUTINE EVSH1
C---------------------------------
      COMMON/C1/WIDTH,B,SSPAN,FCC,FHC,RHO,HEIGHT,HV2,HH1,AVFC,
     * EFFAV,SHEAR,HVC,HHC,HH2,HV1,XH(20),VCI,XHI

         XLM=HEIGHT/SIN(B)
         XLH=VCI/SIN(B)
         XLC=XLM-XLH

C*******AVERAGE CONCRETE STRENGTH, AVFC***

      AVFC=(FCC*(XLC)+FHC*(XLH))/(XLC+XLH)

C*******EFF FACTOR***

      F1=3.5/SQRT(FCC)
      F2=0.27*(1+1/SQRT(HEIGHT*0.001))
      F3=0.15*RHO+0.58
      F4=1.0+0.17*((SSPAN/HEIGHT-2.6)**2)

      EFFAV=F1*F2*F3*F4

      SHEAR=0.5*WIDTH*(1-COS(B))*(EFFAV*AVFC*XLM)*0.001
      BETA=B*45.0/ATAN(1.0)
      WRITE (3,300) XHI,BETA, AVFC,EFFAV,SHEAR
 300  FORMAT (/,2X,F5.1,4X,F5.2,12X,F4.1,10X,F5.3,9X,F6.2)
      RETURN
      END

      SUBROUTINE EVSH2
C---------------------------------
      COMMON/C1/WIDTH,B,SSPAN,FCC,FHC,RHO,HEIGHT,HV2,HH1,AVFC,
     * EFFAV,SHEAR,HVC,HHC,HH2,HV1,XH(20),VCI,XHI
```

```
          XLM=HEIGHT/SIN(B)
          XLH=HHC/COS(B)
          XLC=XLM-XLH

C*******AVERAGE CONCRETE STRENGTH, AVFC***

      AVFC=(FCC*(XLC)+FHC*(XLH))/(XLC+XLH)

C*******EFF FACTOR***

      F1=3.5/SQRT(FCC)
      F2=0.27*(1+1/SQRT(HEIGHT*0.001))
      F3=0.15*RHO+0.58
      F4=1.0+0.17*((SSPAN/HEIGHT-2.6)**2)

      EFFAV=F1*F2*F3*F4

      SHEAR=0.5*WIDTH*(1-COS(B))*(EFFAV*AVFC*XLM)*0.001
      BETA=B*45.0/ATAN(1.0)
      WRITE (3,301) XHI,BETA,AVFC,EFFAV,SHEAR
301   FORMAT (/,2X,F5.1,4X,F5.2,12X,F4.1,10X,F5.3,9X,F6.2)
      RETURN
      END

      SUBROUTINE EVSH3
C--------------------------------
      COMMON/C1/WIDTH,B,SSPAN,FCC,FHC,RHO,HEIGHT,HV2,HH1,AVFC,
     * EFFAV,SHEAR,HVC,HHC,HH2,HV1,XH(20),VCI,XHI

          XLM=HEIGHT/SIN(B)
          XLH=HVC/SIN(B)
          XLC=XLM-XLH

C*******AVERAGE CONCRETE STRENGTH, AVFC***

      AVFC=(FCC*(XLC)+FHC*(XLH))/(XLC+XLH)

C*******EFF FACTOR***

      F1=3.5/SQRT(FCC)
      F2=0.27*(1+1/SQRT(HEIGHT*0.001))
      F3=0.15*RHO+0.58
      F4=1.0+0.17*((SSPAN/HEIGHT-2.6)**2)

      EFFAV=F1*F2*F3*F4

      SHEAR=0.5*WIDTH*(1-COS(B))*(EFFAV*AVFC*XLM)*0.001
      BETA=B*45.0/ATAN(1.0)
      WRITE (3,302) XHI,BETA, AVFC,EFFAV,SHEAR
302   FORMAT (/,2X,F5.1,4X,F5.2,12X,F4.1,10X,F5.3,9X,F6.2)
      RETURN
      END
```