Preliminary study on shear strength of reinforced concrete beams by nonlinear analysis.

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PRELIMINARY STUDY ON SHEAR STRENGTH OF REINFORCED CONCRETE BEAMS BY NONLINEAR ANALYSIS

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This thesis is presented as partial requirements for the award of the Degree of Master of Philosophy of the

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DECLARATION

This work has not been previously accepted for any degree and is not being submitted in candidature for any degree.

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ABSTRACT

The exact prediction of load carrying capacity of reinforced concrete beam with their shear failure, and crack pattern has not been yet clearly defined although lot of research were conducted. However, as more and more knowledge and understanding is gained through continuing experimental and analytical studies, a day may come when a satisfactory design procedure may be developed by using nonlinear finite element analysis.

The objectives of this study were to investigate the influence of following factors on shear strength of reinforced concrete beam in turn to study shear failure behaviour, and load carrying capacity of beams by using nonlinear finite element analysis:

(i) Concrete strength

(ii) Main longitudinal reinforcement ratio

(iii) Shear span –effective depth ratio

(iv) Size of beam

In this research work, beams of different cross sections, main longitudinal reinforcement ratios, and concrete compressive strengths were investigated by considering different load cases.

From this research work it's concluded that, shear strength of reinforced concrete beams increased with increase of concrete compressive strength for (lesser shear span-effective depth ratio), and tensile reinforcement ratio. Also nominal shear stress of reinforced concrete beams increased with the increase of tensile reinforcement ratio and decreased with the increase in beam size.

NOTATION LIST

Symbol	Title	Units
b	Width of beam	mm
d	Effective depth of beam	mm
f_{cu}	Characteristic strength of concrete cube	N/mm²
E_c	Concrete elastic modulus	N/mm²
ν	Poisson's ratio	
γ_m	Material partial safety factor	
f'_c	Compressive strength of concrete	N/mm²
h	Overall depth of beam	mm
D	Displacement of beam	mm
p	Tensile reinforcement ratio	
S_c	Shear capacity	KN
T_s	Tensile stress	N/mm²
C_s	Compressive stress	N/mm²
S_s	Shear stress in XY plane	N/mm²
ν	Nominal shear stress	N/mm²
A_{sv}	Area of two legged stirrups	mm²
S_v	Spacing of shear reinforcement	mm
F_{yv}	Yield stress of shear steel	N/mm²
a	Shear span	mm

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CHAPTER 1
INTRODUCTION

1.1 Aim of the work

The aim of the research work is to study the shear failure behaviour of reinforced concrete beam with various combinations of compressive strength of concrete, shear span –depth ratio, longitudinal reinforcement ratio, different beam sizes, and provided same shear reinforcement ratio in all different 1beam sizes.

For this purpose ANSYS Package which is finite element software has been used to simulate shear failure behaviour of reinforced concrete beam.

Another aim of this research work is to predict the load carrying capacity of a reinforced concrete beam by the finite element method.

1.2 Layout of Thesis

This chapter deals with objectives of this research project, discussion about shear strength of reinforced concrete, application of finite element analysis in structural engineering field with their different software packages.

Chapter 2 deals with the literature review about shear strength of reinforced concrete beam by using Ansys program, this in turn gives description of their method of analysis.

Chapter 3 introduces the method of modelling of reinforced concrete beam, in turn dealt with element type, material properties like concrete, steel, and cross section of reinforced concrete beam.

Chapter 4 deals with the shear stress affected by variable parameters like shear span-effective depth ratio, tensile reinforcement ratio, concrete compressive strength, beam size by using nonlinear finite element analysis.

Chapter 5 processes the Ansys graphic results obtained from numerical modelling of reinforced concrete beam under two point loading.

Chapter 6 draws discussion and conclusions about the research work.

1.3 Investigation about shear strength

The ultimate shear strength is defined as the total and complete failure of the members due to the shear and diagonal tension phenomena. The shear strength of concrete members has generated a lot of research and debate since from turn of the 20th century until now these design procedures for concrete members are bases which primarily depends on experimental results to somewhat than purely theoretical findings. Even though extensive work has been conducted on the subject, the effects of the main variables on shear strength of concrete members are yet to be clearly defined and accepted in the prevailing literatures.

The following are the list of experimental work conducted by various investigators on the shear strength of high performance and normal strength concrete beams:

K.G.Moody, I.M.Viest, R.C.Elstner, and Hognestad (1954), concluded the following points:

(i) The simple beams which were tested with one or more concentrated loads in turn failed in shear after one more diagonal tension crack formed in the region of maximum shear.

(ii) The magnitude of the load causing the formation of initial diagonal tension cracks depend primarily on the dimensions of the cross section and on the strength of the concrete

(iii) The magnitude of the failure loads depended primarily on the dimensions of the cross section, on the amount of longitudinal reinforcement, on the amount of web reinforcement, on the strength of concrete, and on the length of shear span.

The ratio of the ultimate load to the load at first cracking decreased as the ratio of the shear span/effective depth increased.

The following conclusions drawn by R.C.Elstner, K.G. Moody, I.M. Viest, and E. Hognestad (1955),

(i) The magnitude of the cracking load depend primarily on the dimensions of the cross section and on the strength of concrete but practically independent of web reinforcement

(ii) The magnitude of the ultimate load depended clearly on the amount and type of web reinforcement

(iii) The magnitude of the ultimate load increased with increasing amount of web reinforcement and was higher for beams with diagonal stirrups than for beams with vertical stirrups

Phil M. Ferguson and Farid N.Matloob (1959) made the following observation about the shear strength of beams as follows:

(i) Cutting off bars in tension zone will bring about some complication in web or shear stress which will usually lower in shear strength of the member. At cut off point of part of tensile reinforcement, there is a sudden increase in the total tension which results in increased shear stress at that point and corresponding reduction in diagonal tension strength of the beam.

(ii) As higher strength steels are used in turn high f_s values are used, there is some indication that the shear strength (with bar cut off) may be further reduced

(iii) Bending bars instead of cutting them off will largely or completely nullify these ill results and may even increase shear strength above that for full length straight bars

(iv) It is hazardous to cut off bars in a tension zone unless closely spaced stirrups are provided at the cut off point

(v) Even though the trend today is to eliminate bent bars, this practice seems to eliminate some sources of strength and to introduce some sources of weakness.

The following conclusions drawn by Boris Bresler and A.C.Scordelis (1963),

(i) The shear strength of reinforced concrete beams could be increased with small amounts of stirrup reinforcement provided the stirrups are spaced d/2 apart or closer

(ii) The multilayered arrangement of tensile reinforcement appears to be increase shear resistance of reinforced concrete beams.

The studies such as those done by Taub and Neville (1960) and Kani (1966) had highlighted the importance of a/d ratio on the shear strength of reinforced concrete beam.

Vecchio and Collins (1982), in their research stated the following points:

(i) Spalled web thickness within the confinement of the stirrups was considered to be effective for shear.

(ii) Normal compressive stress increased the shear resistance of reinforced concrete but normal tensile stress had the opposite effect.

K.N.Smith and A.S.Vantsiotis (1982) carried out experiments on reinforced concrete beams under two equal symmetrically placed point loads. Test results indicated that, web reinforcement produces no effect on formation of inclined cracks and seems to moderately affect ultimate shear strength. In addition to vertical web reinforcement improves ultimate shear strength of deep beams. However, additional horizontal web reinforcement had little or no influence on ultimate shear strength. In addition previous conclusions, considerable increase in load carrying capacity was observed with increasing concrete strength and decreasing shear span –effective depth ratio.

From previous studies by Kani (1966) and Elzanaty et al (1986) concluded that the tensile reinforcement ratio has considerable effect on the shear strength of reinforced concrete beam.

The following conclusions drawn by Elzanaty, Nilson and Slate (1986),

(i) The code was more conservative at greater concrete compressive strength
(ii) Shear strength of beams increased with greater concrete compressive strength

As for Johnson and Ramierez (1989), they stated the following points:

(i) The shear force transferred to the stirrups during diagonal tension cracking was greater for higher compressive concrete strength and caused stirrups to yield, rupture, and carry greater shear force

(ii) The shear contribution from the shear reinforcement was found to decrease with increasing concrete compressive strength for beams with minimum amount of shear reinforcement.

In the research of Ganwei and Nilson (1990),

(i) The experimental shear capacities of the reinforced concrete beams were only 60-70% of the prediction from the plasticity theory.

Sarsam and Al-musawi (1992) concluded the following points:

(i) Both the ACI and Canadian codes were conservative as concerned to shear strength value predictions
(ii) The size or depth factor did not have a significant effect on the shear strength of beams with shear reinforcement

(iii) The increment of concrete compressive strength up to 80 MPa did not reduce the safety factor (ratio of test shear strength /predicted shear strength) for the ACI code predictions.

Watanable (1993), the following conclusions drawn by

(i) For beams with concrete compressive strengths up to 110 MPa, Nielsen's truss or the AIJ(Japanese) code method could be used to predict the shear strength provide that the effective concrete strength was taken as $\text{v}f_c{}' = 1.7\ f$

(ii) The ACI code gave over conservative predictions of shear strengths for beams with large amount of shear reinforcement.

Xie, Ahmad, Yu, Hino and Chung (1994), concluded the following points:

(i) with beams of a/d ratio of 3, the shear ductility index (area under load-deflection curve) was not significantly influenced by an increase in concrete compression strength

Thirugnana Sundralingam, Sanjayan and Hollins (1995), confirmed the following points:

(i) the diagonal cracking shear force was not influenced by the stirrup spacing

(ii) the ACI 318-89 code predictions were conservative for their beams

(iii) Crack widths were smaller in beams with shear reinforcement compared to those beams without shear reinforcement.

As for conclusions drawn by Kriski and Loov (1996),

(i) The shear strength of a beam with concrete compressive strength up to about 80 MPa could be assumed to vary with square root of f'_c. This is due to fact that, most of the ACI prefer tensile strength of concrete for their mix design, that is why shear strength of concrete links to square root of f'_c.

As for Dino Angelakos (1999), their objective was to investigate the influence of concrete strength and main longitudinal reinforcement ratio on the shear strength of large lightly reinforced members with or without shear reinforcement.

The following conclusions resulted from this study:

(i) The average ratio of the experimental shear failure load to the predicted shear failure load for the ACI method and the General method was 0.74.

(ii) The beam specimens constructed with 1% longitudinal reinforcement without shear reinforcement, and concrete strengths of 20 MPa, 32 MPa, 38 MPa, 65 MPa, and 80 MPa had essentially the same ultimate shear capacity in turn no benefit was realized in the shear capacity for the higher strength concrete beams. In fact the beam specimen with 80 MPa concrete had the lowest shear capacity.

(iii) The effectiveness of high strength concrete proved to be beneficial only when transverse reinforcement was used.

1.4 Finite element Analysis Application in Structural engineering

The finite element method is a powerful numerical technique for analysing structures like nuclear power stations, linear and nonlinear (material, geometric, and boundary conditions) linear analysis of concrete structures, design of building frames, bridges, slabs and walls which subject to some unusual loading situation, assessment of stress arising out of thermal effects, fluid mechanics, electromagnetic, geotechnical engineering.

For the usage of finite element analysis different investigators use their own software or commercial software like FEPACS1, MSC/NASTRAN, DENA, and ANSYS.

In turn these commercial finite element software faced major difficulty because of strain softening behaviour of concrete once it is yielded which leads to inadequate in strain softening behaviour of concrete. This is because these software offer only the traditional nonlinear solution techniques like Newton-Raphson, modified Newton-Raphson methods which in turn can not handle the nonlinear post yielding analysis of members made of materials like concrete , soil , and rock which exhibit strain softening behaviours after their yielding.

CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

The literature on shear behaviour of reinforced concrete beams is very extensive as it extends back to the turn of the twentieth century. This is because of shear design procedures are generally considered to be unsatisfactory in spite of the considerable efforts that are continuously made to revise them. However, it is possible to predict the realistic material behaviour by the advancement of nonlinear finite element analysis technique. In order to achieve such good prediction of behaviour (like failure mode, strength, and cracking pattern etc) of any type of concrete or steel structures in turn to develop a successful nonlinear analysis program requires three major ingredients like:

(i) A realistic material model
(ii) An efficient discretization technique
(iii) An efficient and reliable solution algorithm.

2.2 Numerical modelling of concrete in general

The application of the finite element analysis for the study of reinforced concrete beams was first reported by Ngo and Scordelis (1967). In their research work linear elastic analysis were performed on beams with predefined crack patterns to determine principal stresses in concrete, stresses in steel reinforcement, and bond stresses. However accurate determination of the displacements and the internal stresses in a reinforced concrete structure throughout its load history is made difficult.

Again by Nilson (1968) extended the method of analysis to consider cracking by nodal separation to introduce nonlinear material properties and a nonlinear bond-slip relation into the analysis (quadrilateral plane stress finite element were used).

Franklin (1970) advanced the capability of analytical method by applying a variable stiffness approach to include the effect of tensile cracking and nonlinear material behaviour (an incremental iterative method was used for the nonlinear analysis).

The "initial stress" method proposed by Zienkiewicz et al (1969) was adopted by Valliappan and Doolan (1972) to include the effect of elasto-plastic behaviour of concrete and steel and progressive cracking of concrete in the study of reinforced concrete structures.

Suidan and Schnobrich (1973) also studied behaviour of beams by using three dimensional elements with 20-node rectangular isoparametric elements.

Oral Buyukozturk (1977) considered a generalised Mohr-Coulomb behaviour for the yield and failure of concrete under combined stress in the nonlinear analysis of reinforced concrete structures.

The nonlinear finite element analysis system is rarely used in practice for the analysis of concrete structures even though several systems have been developed up to this date. The main reason appears to be lack of accurate numerical description of material behaviour characteristics. This is due to nonlinear stress-strain relationships and a material such as concrete that to exhibit changes in behaviour with time because of gain in strength and creep which in turn both related to the chemical process between the cement and the water. Concrete has a very low strength in tension which leads to crack at a very early stage of loading and fracture of concrete cracking in tension and crushing in compression, reinforcement yield add more complexity. The NLFEA systems are based on standard numerical techniques by Choleski or frontal solution, iterative procedures based on New-Raphson method.

The problem of developing a good material model for reinforced concrete is probably the most difficult task. Even though this material has been tested experimentally and in turn it is phenomenological behaviour is well understood, it has proved difficult to cast knowledge into mathematical forms that could be used in computations for analysis to obtain accurate solution. The classical elasticity and plasticity theory could be sufficiently accurate found application particularly when the concrete structure fails in cracking of concrete and yielding of reinforcement bars. Whereas plasticity theory for concrete is sufficiently accurate for uniaxial stress and two dimensional states of stress with proportional loading. But for complex loading histories and three dimensional states of stress it has been these theories became inadequate. In addition to these theories , one more theory called internal variable theories were to be more powerful in describing materials in which several rheological phenomena take place simultaneously (like gradual deterioration).

One such material model is Endochronic theory which has been adapted to concrete behaviour by Bazant (1976). The term “ Endochronic” means that in which case nonlinearity of the material is introduced by means of an intrinsic time parameter which is an independent scalar variable whose increments depends on both time increments and deformation increments. From this endochronic theory (Valanis) who was first to (for metals) predicts strain hardening, unloading diagrams, effect of pretwist on axial behaviour, contraction of hysteresis loops in cyclic loading, and the effect of strain rate. In order to model nonlinear behaviour of concrete in turn needs three major extensions like:

(i) Hydrostatic pressure sensitivity of inelastic strain

(ii) Inelastic dilatancy

(iii) Strain softening tendency at high stress

C. S. Krishnamoorthy and A. Panneerselvam (1977), have presented a computer program FEPACS1 for nonlinear finite element analysis of reinforced concrete framed structures in which finite element formulation for reinforcement in any orientation in computing the element stiffness was explained. From their research work in turn concluded that, the algorithm presented is general in nature so as to develop the computer program for nonlinear analysis of any reinforced concrete structures.

A .Arnesen, S .I. SØrensen and P. G. Bergan (1979), their research work was to develop two different computer programs for nonlinear analysis of reinforced concrete structures. The first program handles plane stress problems (beams, corbels) and second program is developed for analysis of plates (square plates and shells with geometric nonlinear) and shells.

The following are conclusions drawn by their research work:

(i) the plane stress program is efficient and inexpensive in use
(ii) with this program , both geometric and the material nonlinearities are well predicted
(iii) The Endochronic theory could be implemented in a finite element formulation which also considers cracking and yielding of reinforcement bars.

Michael D.Kotsovos (1984), finite element analysis of under and over reinforced concrete beams subjected to two point loading indicates that placing shear reinforcement in the middle rather than in the shear span results in both higher load carrying capacity and ductility when the shear span to effective depth ratio is between 1.0 and 2.50.

The following conclusions were to be drawn from their research work:

(i) the nonlinear finite element analysis of under and over reinforced concrete beams with a/d < 2.50 subjected to two loading indicated that placing shear reinforcement within the middle span rather than the shear span results in a significant improvement of both load carrying capacity and ductility of beams.

(ii) whereas in case of a/d > 2.50, failure of the beams is due to branching of the diagonal crack within the shear span towards the compressive zone of the middle span and not due to crushing of the compressive region of the loading point .

(iii) It has also been found that collapse of the beams always occurs before the compressive strength of concrete is exceeded any where within the beams.

(iv) Even in the compressive zone concrete fails under combined compressive and tensile stresses in turn these tensile stresses results from the interaction of adjacent concrete elements subjected to different states of stress.

In this research work, isoparametric elements were used to model both concrete and steel. For each a/d and as, four types of reinforced concrete beams containing different arrangements of shear reinforcement were investigated:

Type A – without shear reinforcement

Type B – shear reinforcement within the shear span only

Type C – shear reinforcement within the middle span only

Type D – shear reinforcement throughout the span

Strength characteristics:

When a/d = 2.27, Type C carries a max load carrying capacity as when compared to Type B irrespective of tensile reinforcement .This behaviour supports the proposal that collapse of these beams is caused by failure of the compressive zone of the middle span under compression –tension stress conditions and not by failure of the region of the loading point under wholly compressive state of stress. Thus using shear reinforcement in the middle span prevents failure of middle span rather than delaying the occurrence of the diagonal crack within the shear span.

However, a/d > 2.5, sustain a higher load when the shear reinforcement is placed within the shear span rather than the middle span. But a/d > 2.5 are know to collapse due to failure of their shear span .It is also concluded from the program that, placing of shear reinforcement throughout the span generally improved the loading carrying capacity of the beams .

Michael D.Kotsovos (1986) describes an investigation into the causes of shear failure of reinforced concrete beams subjected to two point loading with shear span-effective depth ratio > 2.50. A finite element analysis of beams with various arrangements of stirrups has shown that the predicted behaviour is incompatible with the concept of shear capacity of critical sections that forms the basis of current shear design procedures.

The beams were subjected to two point loading with shear span values equal to 300 to 400 mm. and corresponding a/d ratio equal to 3.3 and 4.30.

For each a/d and as, four types of reinforced concrete beams containing different arrangement of stirrup were to be investigated:

Type A – without shear reinforcement

Type B – shear reinforcement throughout the span

Type C – shear reinforcement within the region of the shear span between cross section at the support and that at a distance of 200 mm from the support

Type D – shear reinforcement throughout the span except in the regions reinforced with shear reinforcement in Type C beams

In all cases the shear reinforcement had a cross sectional area of 16.09 mm^2 with a spacing of 50 mm and yield stress of 417 N/mm^2.

From Strength characteristics:

On the basis of the concept of shear capacity of a critical section, beams A, C, and D must have a similar load carrying capacity where shear force is constant. Whereas beam D has a load carrying capacity significantly greater than that of beam A and while beams A and C have same load carrying capacity in most cases. Furthermore, the load carrying capacity of beam D is essentially equal to that of beam B. Whereas on the basis of the concept of the compressive force path (1968), a beam without shear reinforcement fails due to the development of tensile stresses within the compressive zone.

S .Y. A .Ma and I .M. May (1986),

The finite element method has been used to analyse reinforced concrete structures from zero load up to collapse in which modified Newton-Raphson method has been used as the basic solution procedure. In this paper, the performance of various accelerators are discussed and compared with each other by Crisfield (1984).

The following conclusions drawn from their work:

(i) The use of the displacement criterion in the procedures with loading incrementation gives false pictures of the behaviour of the structures at their post collapse stage.

(ii) The results obtained lend support to the findings of Crisfield that accelerators used were reliable and lead to a significant improvement in the rate of convergence of the modified Newton-Raphson method.

A. Ranjbaran and M .E. Phipps (1994) have done research work on finite element program for the nonlinear stress analysis of two dimensional problems by considering both metallic and reinforced concrete structures. For this analysis, software package called DENA were to be employed. In this study an embedded model was used. In their program , equation of systems are solved by using an incremental iterative approach in which nonlinear solution is achieved by a series of successive linear solutions and adjustment of material constants in such a way that at the final stage the new constitute equation is satisfied .

P .Bhatt and M. Abdel Kader (1995), predicted shear strength of rectangular reinforced concrete beams by using a single material model for concrete. The objective of their work was to suggest a material model to achieve the criteria for good lower bound predictions but at the same time predict responses in agreement with the observed behaviour.

In this study the best prediction was for beams with shear reinforcement and a small spacing stirrups

Sreekanta Das and Muhammad N .S. Hadi (1996),

Performed nonlinear finite element analysis of reinforced concrete members using MSC/NASTRAN. In turn MSC/NASTRAN offers many advanced solution techniques like Crisfield's arc-length method, Risk's arc-length method, and modified Risk's arc-length method and thus these methods can handle the strain softening behaviour adequately.

One of the major advantages in modelling RC members in MSC/NASTRAN is that it offers gap element. The gap element can be made suitable to simulate cracks in the concrete and additional property of friction in the gap element is assigned to be negligible/zero.

One of the Crisfield's arc-length methods is therefore used to pass the unstable region of the stress-strain curve successfully and also in the stain softening region to avoid the numerical complexities associated with the nonlinear post yielding analysis of concrete members.

MSC/NASTRAN offers many special features like one of them is SUBCASE , which provides the options of applying the total load in different load steps and every load step with different number of load increments , different number of iterations for each load increments , and other controlling parameters of nonlinear solution process .

Another special feature RESTART, which is used to restart the solution process from the last converged solution which is saved by MSC/NASTRAN automatically. It is concluded that any RC members subjected to uni-axial stress condition could be analysed using MSC/NASTRAN version 68.2 very successfully.

The finite element code Ansys version 5.3 has been used in which, a simply supported reinforced concrete beam subjected to uniformly distributed loading has been analysed without transverse reinforcement.

From their work the following points were to be concluded:

(i) Only nonlinear stress-strain relations for concrete in compression have made it possible to reach the ultimate load and determine the entire load-deflection diagram.

Tanijun Wang, Thomas T.C, and Hsu (2001),

Performed nonlinear finite element analysis to various type of reinforced concrete members using a new set of constitute models established in the fixed angle softened truss model. A computer code FEAPRC (in turn take care of the four important characteristics of cracked reinforced concrete like softening effect of concrete in compression, tension stiffening effect by concrete in tension, average stress-strain curve of steel bars embedded in concrete, rational shear modulus of concrete) was developed for reinforced concrete structures by modifying the general purpose program FEAP.

From their work the following points were to be concluded:

(i) By this code (FEAPRC), it is possible to predict the behaviour of reinforced concrete members.

(ii) The behaviour of beams, shear panels, and framed shear walls predicted by FEAPRC was found to agree very well with the observed behaviour.

2.3 Reasons for Caution about NLFEA

(i) Diversity of theoretical approaches

A number of rather diverse approaches exist for nonlinear finite element analysis (NLFEA) modelling of reinforced concrete structures like those available are: models built on nonlinear elasticity, plasticity, fracture mechanics, damage continuum mechanics, endochronic theory or other hybrid formulations. Cracking can be modelled discretely or using smeared crack approaches, in which smeared crack approaches can range from fully rotating crack models to fixed crack models, to multiple non orthogonal crack models, to hybrid crack models. In general some approaches place heavy emphasis on classical mechanics formulations, other draw more heavily on empirical data and phenomenological models.

(ii) **Diversity of behaviour models**

The reinforced concrete structures particularly in their crack steels are dominated in their behaviour by a number of second order mechanisms and influencing factors. Depending on the particular details and conditions prevailing , a structures strength , deflection ductility and failure mode may be affected by mechanisms such as : compression softening due to transverse cracking, tension, stiffening, tension softening, aggregate inter lock and crack shear slip, rebar dowel action, rebar compression buckling, scale effects, creep, and shrinkage. Thus, which could be users of a NLFEA software must be aware of what mechanisms are likely to be significant are included in the analysis model.

(iii) **Incompability of models and approaches**

The formulation and calibration of a concrete behaviour model is often dependent on the particular analysis methodology being used. As a result some models can not be randomly transplanted from one analysis approaches to another or freely combined with other models. As an example of Vecchio and Collins (1986), Okamur and Maekawa (1991), conducted that in, Vecchio and Collins formulations of slip overestimations strength and slightly under- estimates ductility whereas in case Maekawa formulation slightly under estimates strength and slightly overestimates ductility.

(i) Experience required

- The use of NLFEA for modelling and analysis of reinforced concrete structures requires a certain amount of experience and expertise.
- The proper decisions must be made with respect to modelling of the structure and selection of material behaviour models which will have significant impact on the results obtained
- The proper decisions must be made regarding mesh layout, type of element used, representation of reinforcement details, support conditions, method of loading, convergence criteria, and selection of material behaviour model will provide divergence of results.

2.4 ANSYS Overview

The finite element package used in this research work to simulate behaviour of shear failure of reinforced concrete beam was ANSYS (7.1) version. The software package ANSYS employed in various other simulation works like simulation of response of solid structures to various types of loading such as mechanical, thermal, static, dynamic, and electromagnetic. This software ANSYS was developed by Swanson Analysis System Inc in 1970's as a general purpose program in turn it could be employed in field such as structural, mechanical, electrical, thermal, fluid, and biomedical.
ANSYS program could be run in two ways as by writing batch files or by directly interacting through graphical user interface (GUI). The following are the most general way of input data into the ANSYS software:

(i) Define element type
(ii) Define element real constants
(iii) Define material properties
(iv) Creation of geometrical model
(v) By applying loads obtain solution
(vi) Review the results for accurate solution

The format of the ANSYS program is written in the following way :

/Prep 7 ---------------------------Pre-processor

Element type

Element real constant

Material properties

Geometrical model creation

Apply boundary condition ---- Solution processor

Apply loads

Analysis type

Review results -------------------Postprocessor

The ANSYS program contains element library which comprised of 150 different element types with each element having a reference number and prefix that identifies the element group. By selecting the element type in turn mention their degree of freedom and whether the element is two or three dimensional one. Next step is to mention the element real constant which are the properties depend on element type. Another step is to specify material properties where the solution is linear or nonlinear. ANSYS provides two methods of creating a model:

(i) Solid modelling

(ii) Direct generation

In the case of solid modelling, user is to describe the geometrical shape of the model in turn ANSYS automatically meshes this geometry with nodes and elements in which user could be able to control size and shape of these elements. While in the case of direct generation method, user is to specify the location of each node and connectivity of each element. Further analysis type is defined, loads were to be applied and then solution is initiated. ANSYS program contains different analysis types like static, transient, harmonic, modal, spectrum, buckling, and sub structuring. With the selection of appropriate type of analysis type then loads were to be applied. Also ANSYS program contains the following types of loads like: DoF constraints, forces, body loads, surface loads, inertia loads, and coupled field loads. In order to review the results postprocessor needed to obtain appropriate solution. The ANSYS program comprises of two types of post processor :(i) Post 1, which is a general post processor in which it allows to review results at specific load step and sub step. (ii) Post 26, which is time history post processor in turn it allows to review particular result item with respect to time frequency.

Research Brief: The main thrust of this research work was to study the shear failure behaviour of reinforced concrete beams by nonlinear analysis of finite element method. The variable parameters considered were: shear span-effective depth ratio, compressive strength of concrete, tensile reinforcement ratio, same shear reinforcement ratio in all different beams size. In the literature review about shear failure behaviour of reinforced concrete beam subjected to two point loading by M.D.Kotsovos (1984, and1986) , in which their main intension was to study the behaviour of reinforced concrete beams with a shear to depth ratio between 1.0 and 2.5 and greater than 2.5 . When $a/d > 2.5$, they concluded the following points:

From Strength characteristics: The load carrying capacity of beam type D (stirrups throughout the span except in the region at a distance 200 mm from the support) is essentially equal to that of beam type B (which is reinforced stirrups throughout the shear span).

From Deformation characteristics: The beams B and D should exhibit ductile behaviour since in turn they were failed in flexure.

From Fracture characteristics: The placing of shear reinforcement throughout the span of beam, reduce both the amount of inclined cracking with the shear span and allowed the beam to attain its flexural capacity.

Thus from their finite element analysis of beams with various arrangements of stirrups has shown that the predicted behaviour is incompatible with the concept of shear capacity of critical sections which forms the basis of current shear design procedures.

When shear span-effective depth ratio (a/d) = 1 & 2.5, concluded the following points:
From Strength characteristics: The load carrying capacity of beam type D (stirrups throughout the span except in the region at a distance 200 mm from the support) is essentially equal to that of beam type B (which is reinforced stirrups throughout the shear span).

From Deformation characteristics: The beams B and D should exhibit ductile behaviour since in turn they were failed in flexure.

From Fracture characteristics: The placing of shear reinforcement throughout the span of beam, reduce both the amount of inclined cracking with the shear span and allowed the beam to attain its flexural capacity.

Thus from their research work by using finite element analysis of under and over reinforced concrete beams subjected to two point loading indicated that placing shear reinforcement in the middle rather than in the shear span results in both higher load carrying capacity and ductility when shear span to depth ratio is between 1 and 2.5.

Whereas in the case of a/d = between 1.0 and 2.5, they concluded the following points:
From Strength characteristics : if a/d =2.27, the load carrying capacity of beam type C (shear reinforcement within middle span) is higher than that of the beam type B(shear reinforcement within shear span) irrespective of the amount of tension reinforcement .

From Fracture characteristics : The presence of shear reinforcement throughout the beam type D, both delayed the extension of diagonal cracking within the shear span towards the loading point and helped the compressive zone of the middle span to sustain a substantially larger amount of cracking .

In the present research work five different beam sizes of same span length of 1000 mm were considered with equidistant shear reinforcement for the entire span and kept shear reinforcement ratio constant in all these beam sizes. The cross sectional areas of the tension reinforcement were (8mmø) 201.06 mm², (14mmø) 615.75 mm², (16mmø) 804.24 mm². The different concrete compressive strength, and yield stress of tensile reinforcement and shear reinforcement considered were 16.75, 20.10, 26.80, 33.50, 40.20 N/mm², and 460, 250 N/mm². The beams were subjected to two point loading with values of shear span (a) equal to 200, 250, 300, and 350 mm respectively. In all five sizes of beam, the stirrups had a cross sectional area of (6mm ø) 28.27 mm² with spacing of 100 mm.

2.5 Method of solution

For this research work, ANSYS program uses Newton-Raphson method for updating the modified stiffness. This method recalculates the stiffness matrix for each iteration within the load step. Once the convergence tolerance is reached, the solution could be continued to the next step.

2.6 Selection of Element type

An eight nodded solid element Solid 65 was used for the numerical modelling of concrete which has three degrees of freedom at each node. Also a three dimensional Link 8 was used for the numerical modelling of the steel reinforcement in which this element has three degrees of freedom at each node.

2.7 Cross sections of Reinforced Concrete Beam

The cross sectional dimensions of beams (120mmx150mm), longitudinal reinforcement (8mmø), shear reinforcement with their spacing (100mm) as shown in Fig.1, and concrete cube strength of 40 N/mm² concern in turn it was chosen from previous research work by C. S. Chin (2002). But, in order to study the behaviour of shear failure of reinforced concrete beam concern in turn consider different beam size, longitudinal reinforcement , concrete compressive strength , a/d ratio , and by keeping same shear reinforcement ratio in all different beam sizes. The Fig.1 represents cross section of reinforced concrete beam with loading arrangement.

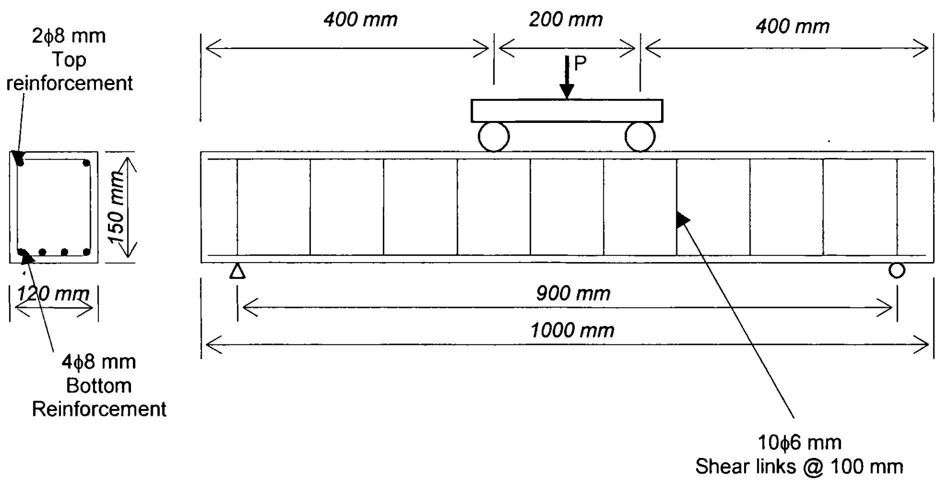

Fig.1 Cross sections of Reinforced Concrete Beam with arrangement of Loading

CHAPTER 3

NUMERICAL MODELLING OF REINFORCED CONCRETE BEAMS

3.1 Introduction

The aim of this project work is to present an analytical description of the numerical modelling of five different sizes of reinforced concrete beams in which beams were subjected to two point loading conditions.

3.2 Modelling of RC beam under two-point loading

For this research work concern, the geometric dimensions of reinforced concrete beams were considered as per C.S. Chin research project (2002). The main thrust of the research work is to study shear failure behaviour of beams under two point loading conditions by using nonlinear finite element analysis. The different variable parameters were: shear span-effective depth of beam, compressive strength of concrete, and tension steel ratio with same shear reinforcement ratio in all different sizes of beam in turn to predicting load carrying capacity of reinforced concrete beams.

3.3 Element type

In the present work, an eight node solid element Solid 65 was used which has got three degrees of freedom at each node, it is capable of cracking, and crushing and it could be used for both linear as well as nonlinear problems. A three dimensional spar element Link8 was used for numerical modelling of the steel reinforcement. Link8 is a uniaxial tension-compression element with three degrees of freedom at each node: translations in the nodal X, Y, and Z directions and it is capable of problems like plasticity and large deflections. Fig.2 and Fig.3 represent Solid 65 element and Link 8 element.

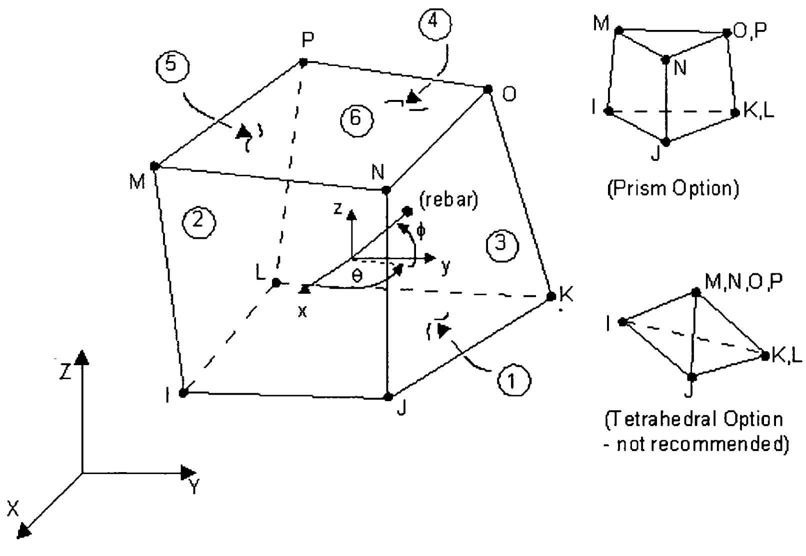

Fig.2 Solid 65 element

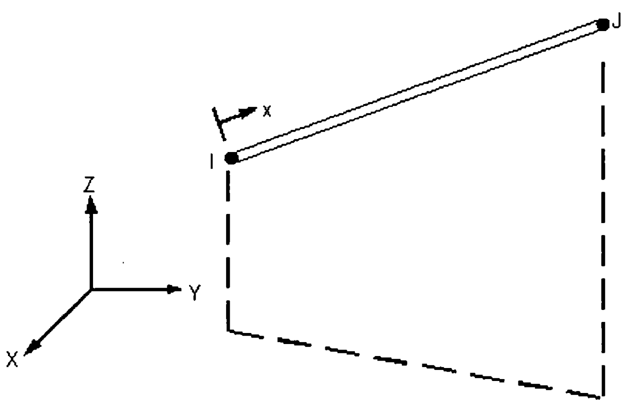

Fig.3 Link 8 element

3.4 Material properties

3.4.1 Concrete

The numerical modelling is one of the most important areas for finite element analysis. The accuracy in the modelling of geometry, loads, material properties, boundary conditions, and other structural properties are needed for numerical idealisation of the actual structure. An accurate determination of the displacements and the internal stresses in a reinforced concrete structure in its entire load history is made difficult due to following factors like:

(i) nonhomogenity of the member cross sections
(ii) nonlinear material constitute relationship
(iii) inelastic behaviour of concrete requiring adequate yield and failure criteria under biaxial (multiaxial) states of stress
(iv) profound influence of progressive cracking and continuing changing topology
(v) bond-slip characteristic between concrete and steel
(vi) Influence of creep and shrinkage.

In compression region, the stress-strain curve is linear elastic up to 30% of the maximum compressive strength in turn after concrete attains peak load. As a result of descending nature of stress-strain curve, crushing occurs at an ultimate strain .Whereas in tension, the stress-strain curve is linear elastic up to maximum tensile strength and after the concrete cracks in turn strength decreases gradually to zero value by M.Y.H.Bangash (1989).

For the numerical modelling of concrete material by using Ansys program following material properties were to consider:

For this project work, it was decided to use as initial elastic modulus from the first point of the curve and then compare it with formulae given by BS8110-Part1:1997
$E_c = 5500 \sqrt{f_c}$

Also for this research work poisons ratio taken as 0.15. The two input strength parameters like un-axial compressive (16.75 N/mm^2) and tensile strength (3 N/mm^2) were to be considered to define a failure surface for the concrete. The uni-axial compressive strength is to control the crushing of the model and uni-axial tensile strength is to control cracking of the model. The shear transfer coefficient indicates conditions of the crack face and typical shear transfer coefficients range from 0-1 in which 0 represents smooth crack (complete loss of shear transfer) and 1 represents a rough crack (no loss of shear transfer) .

The shear transfer coefficients used for this research work are in case of open crack (0.05) and closed crack (0.2). The proceeding table represents concrete material data for element Solid 65 is presented.

Table 1: Solid 65 Concrete material Data

Const	Meaning
1	Shear transfer coefficients for an open crack.
2	Shear transfer coefficients for a closed crack.
3	Uniaxial tensile cracking stress.
4	Uniaxial crushing stress positive.
5	Biaxial crushing stress positive.
6	Ambient hydrostatic stress state for use with constants 7 and 8.
7	Biaxial crushing stress positive under the ambient hydrostatic stress stat (constant 6).
8	Uniaxial crushing stress positive under the ambient hydrostatic stress state (constant 6).
9	Stiffness multiplier for cracked tensile condition, used if Keyopt (7) = 1(default to 0.6).

The simplified stress-strain curve used in this was constructed from six points and values of maximum stress and maximum strain were according to BS 8110-1:1997.

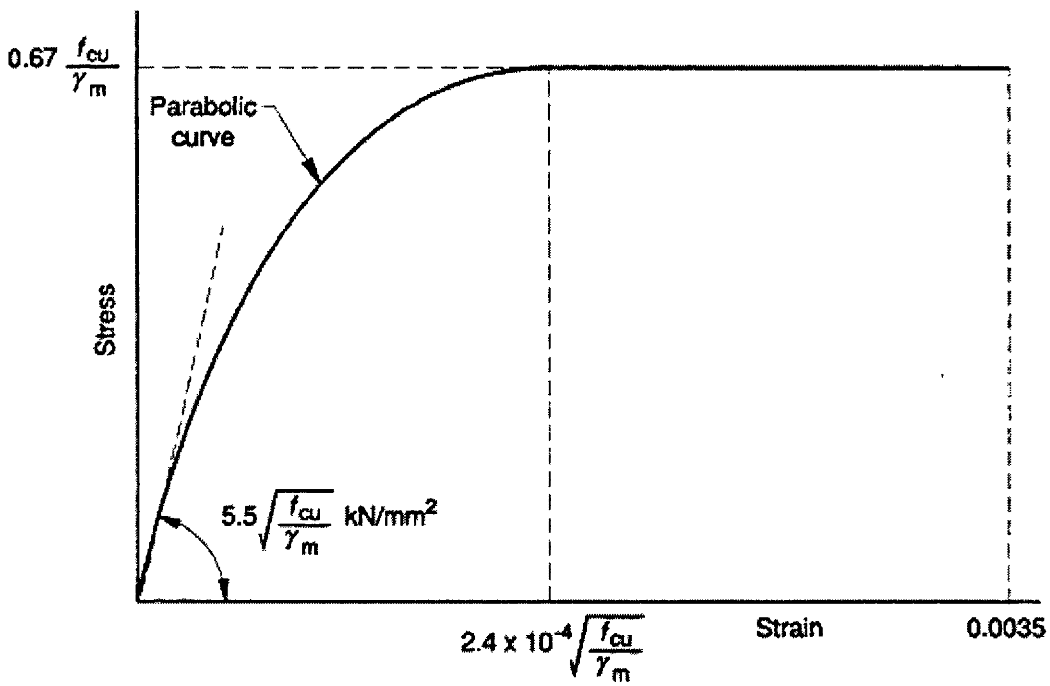

Fig.4 Short term design stress-strain curve of normal weight concrete

As with reference to Fig.4 it shows variation of stress-strain curve of normal weight concrete in which concrete shows plasticity behaviour that is, as the load is applied the ratio between stress and strains is approximately linear at first and the concrete behaves almost as an elastic material with a full recovery of displacement if the load is removed. Then after the curve is no longer linear and concrete behaves more and more as a plastic material. If the load were removed during the plastic range the recovery would no longer be complete and a permanent deformation would remain.

For this study, smeared cracking model was used and by increasing this smeared coefficient in turn leads to increase of stiffness of the element. The smeared crack model assumes that reinforcement is uniformly spread throughout the concrete element in a defined region of the finite element mesh. Also smeared crack model which is capable of properly combining crack formation and the nonlinear behaviour of the concrete between the cracks and of handling secondary cracking owing to rotation of the principal stress axis after primary crack formation. Similarly smeared crack model is a numerical model for capturing the flexural modes of failure of reinforced concrete systems and it is capable of accurately predicts the deflection and shear strength of a reinforced concrete system for a given load.

3.4.2 Steel Reinforcement

The steel reinforcement used for the modelling in the beams in this was assumed to be an elastic perfectly plastic material. The stress-strain relationship as proposed by BS8110-1:1997 which is used in this study. As with reference to Fig.5 it shows variation of stress-strain curve for reinforcement.

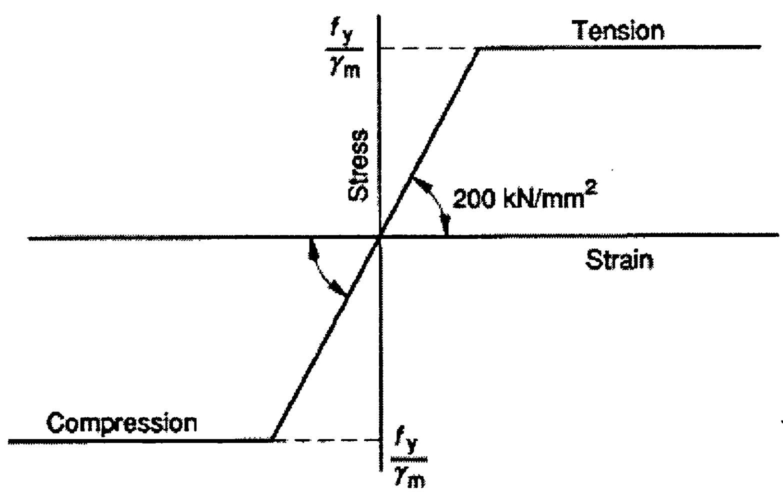

Fig.5 Stress-strain curve for reinforcement

3.4.3 Specimen Details

The main thrust of this research work was to study the effects of variable parameters like a/d ratio, concrete compressive strength, tensile reinforcement ratio, and beam size on the shear strength of reinforced concrete beam under two point loading condition by using nonlinear finite element analysis. In this research work, five different beam sizes were considered like sizes from (75x100; 100x125; 120x150; 150x200; 200x300) mm , different a/d ratios considered were range from 0.75-4.11, and concrete compressive strength range from 25-60 N/mm² . In this work, the span length considered was 1000mm for all types of beams and same shear reinforcement was considered for all sizes of beams. All five sizes of beams were simply supported in the direction of the load and there is no complication to specify boundary conditions.

As concern to specimen details, Table 2 gives complete detailed description of various types of beam which is to be used in this present research work and their reinforcement specification.

Table 2 - Cross Sections of Reinforced Concrete Beams

Dimensions (mm)	75x100 Beam type 1	100x125 Beam type 2	120x150 Beam type 3	150x200 Beam type 4	200x300 Beam type 5
Stirrups	6 mmø	6 mmø	6 mmø	6 mmø	6 mmø
Spacing (mm)	100	100	100	100	100
Yield Stress (MPa)	250	250	250	250	250
Long. Steel Yield Stress (MPa)	2-8 mmø(T); 4-8 mmø(B) 460	2-8 mmø(T); 4-8 mmø(B) 460	2-8 mmø(T); 4-8 mmø(B)	2-8mmø(T); 4-8 mmø(B) 460	2-8 mmø(T); 4-8 mmø(B) 460
Long. Steel Yield Stress (MPa)	2-14 mmø(T); 4-14mmø(B) 460	2-14 mmø(T); 4-14mmø(B) 460	2-14 mmø(T); 4-14 mmø(B) 460	2-14mmø(T); 4-14mmø(B) 460	2-14mmø(T); 4-14mmø(B) 460
Long. Steel Yield Stress (MPa)	2-16 mmø(T); 4-16mmø(B) 460	2-16 mmø(T); 4-16mmø(B) 460	2-16 mmø(T); 4-16 mmø(B) 460	2-16mmø(T); 4-16mmø(B) 460	2-16mmø(T); 4-16mmø(B) 460

3.4.4 Nonlinear Solution and Results

In the nonlinear analysis, the total load which is applied to a finite element model is divided into a series of load increments called as load steps. After the completion of each these load step, the stiffness matrix of the model is recalculated before proceeding to the next load increment.

The method of solution considered in this Ansys program was Newton-Rapson method which in turn updates the model's stiffness.

CHAPTER 4

SHEAR STRESS AFFECTED BY VARIABLE PARAMETERS

The objective of this study were to investigate the influence of the following variables : concrete compressive strength, main longitudinal reinforcement ratio, shear span to effective depth ratio, and beam size on the shear strength of reinforced concrete beam in turn to study shear failure behaviour, and load carrying capacity of beams by using nonlinear finite element analysis .

As concern to specimen details refer Table 2 which gives complete detailed description of various types of beam which is to be used in this present research work and their reinforcement specification. The Table 3 to 7 represents the comparison of mid-span deflections obtained by ansys with varying shear span-effective depth ratio, concrete compressive strength, tensile reinforcement ratio, and beam size.

Beam type 1

Table 3 – Comparison of mid-span Deflections with different a/d ratio (Beam type 1)

p = 0.03, a/d	f'_c, N/mm²	S_c, KN	D, mm	p = 0.096, a/d	f'_c, N/mm²	S_c, KN	D, mm	p = 0.13, a/d	f'_c, N/mm²	S_c, KN	D, mm
2.4	16.8	26	2.3	2.4	16.8	30	1.5	2.4	16.8	28	1.1
2.9	16.8	22	2.4	2.9	16.8	28	1.6	2.9	16.8	28	1.3
3.5	16.8	22	2.8	3.5	16.8	26	1.5	3.5	16.8	28	1.5
4.1	16.8	18	2.4	4.1	16.8	22	1.3	4.1	16.8	26	1.6
2.4	20.1	30	2.66	2.4	20.1	34	1.8	2.4	20.1	34	1.3
2.9	20.1	26	2.85	2.9	20.1	32	2.1	2.9	20.1	32	1.5
3.5	20.1	24	3.05	3.5	20.1	32	1.8	3.5	20.1	32	1.7
4.1	20.1	24	3.48	4.1	20.1	26	1.5	4.1	20.1	28	1.6
2.4	26.8	40	3.38	2.4	26.8	42	2.2	2.4	26.8	44	1.5
2.9	26.8	34	3.53	2.9	26.8	38	2.1	2.9	26.8	40	1.8
3.5	26.8	30	3.64	3.5	26.8	36	1.9	3.5	26.8	38	1.9
4.1	26.8	28	3.81	4.1	26.8	34	1.8	4.1	26.8	36	2.0
2.4	33.5	34	3.62	2.4	33.5	52	2.7	2.4	33.5	56	1.9
2.9	33.5	34	4.05	2.9	33.5	48	2.7	2.9	33.5	52	2.2
3.5	33.5	40	3.97	3.5	33.5	46	2.4	3.5	33.5	48	2.4.
4.1	33.5	44	4.55	4.1	33.5	42	2.2	4.1	33.5	42	2.3
2.4	40.2	50	4.01	2.4	40.2	56	2.9	2.4	40.2	56	1.9
2.9	40.2	44	4.32	2.9	40.2	54	2.7	2.9	40.2	54	2.2
3.5	40.2	38	4.31	3.5	40.2	48	2.6	3.5	40.2	52	2.5
4.1	40.2	36	4.61	4.1	40.2	46	2.2	4.1	40.2	48	2.6

From Table 3 for Beam type 1, the following observations were made by considering different a/d ratio, tensile reinforcement ratio, concrete compressive strength, and beam size by keeping same shear reinforcement ratio with even spaced of their stirrups.

For small a/d = 2.4, ρ = 0.031 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm², the observed increased total shear capacity were to be 26, 30, 40, 34, and 50 KN which in turn indicates that concrete compressive strength has it's influence on increased (92.30%) of total shear capacity and observed displacements were to be lower (14.96%) as when compared with higher a/d ratio's displacement value i.e. in the range of 2.30, 2.66, 3.38, 3.62, and 4.01 mm .

For increased a/d = 4.11, ρ = 0.031 with different f'_c = 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², the observed decreased or increased total shear capacity were to be 18, 24, 28, 44, and 36 KN which indicate that concrete compressive strength has less effect on increment (100%) of total shear capacity and it is observed displacements were to be in the increased order of 2.40, 3.48, 3.81, 4.55, and 4.61 mm.

From Table 3, it is noticed that for small a/d = 2.4, with increased ρ = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm², the total shear capacity were increased from 30, 34, 42, 52, and 56 KN which indicates that the tensile reinforcement ratio has it's effect on the increased (86.67%) of total shear capacity and observed displacements were to be in the range of 1.50, 1.80, 2.21, 2.72, and 2.89 mm .

For increased a/d = 4.11, ρ = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm², the increased and in one case decreased total shear capacity observed were to be 22, 26, 34, 42, and 46 KN which in turn indicates that even though for increased a/d ratio and increased ρ ratio which has little influence on increment (109%) of total shear capacity and observed displacements were to be 1.31, 1.52, 1.76, 2.15, and 2.22 mm .

Further more from Table 3 again for less a/d = 2.4, ρ = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm², there is a tendency of increase in total shear capacity were to be observed as 28, 34, 44, 56, and 56 KN which indicates that tensile reinforcement ratio has some effect on the increased (100%) of total shear capacity as when compared to two previous tensile reinforcement ratio (i.e. 0.031, and 0.096) and correspondingly observed very small displacements were to be 1.01, 1.25, 1.52, 1.95, and 1.90 mm and which in turn tensile reinforcement ratio has it's effect on decreased mid span deflection .

For increased a/d = 4.11, p = 0.126 with different f'_c = 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², the observed decreased total shear capacity were to be 26, 28, 36, 42, and 48 KN which indicates that even though with increased tensile reinforcement ratio and with increased concrete compressive strength has less effect on increment (84.61%) of total shear capacity and observed displacements were to be 1.55, 1.62, 1.98, 2.30, and 2.60 mm.

Beam type 2

Table 4 – Comparison of mid-span Deflections with different a/d ratio (Beam type 2)

p = 0.03, a/d	f'_c, N/mm²	S_c, KN	D, mm	p = 0.096, a/d	f'_c, N/mm²	S_c, KN	D, mm	p = 0.13, a/d	f'_c, N/mm²	S_c, KN	D, mm
2	16.8	38	2.1	2	16.8	46	1.3	2	16.8	46	1.1
2.5	16.8	30	1.9	2.5	16.8	44	1.5	2.5	16.8	46	1.4
3	16.8	26	1.9	3	16.8	36	1.4	3	16.8	38	1.3
3.5	16.8	32	3.1	3.5	16.8	36	1.6	3.5	16.8	36	1.4
2	20.1	44	2.4	2	20.1	54	1.5	2	20.1	56	1.4
2.5	20.1	42	2.9	2.5	20.1	54	1.9	2.5	20.1	54	1.6
3	20.1	36	2.8	3	20.1	44	1.8	3	20.1	52	1.9
3.5	20.1	34	3.1	3.5	20.1	44	2.1	3.5	20.1	50	2.1
2	26.8	58	2.9	2	26.8	72	1.9	2	26.8	68	1.5
2.5	26.8	52	3.3	2.5	26.8	68	2.2	2.5	26.8	60	1.6
3	26.8	42	3.0	3	26.8	54	2.0	3	26.8	60	2.0
3.5	26.8	42	3.5	3.5	26.8	50	2.2	3.5	26.8	54	2.0
2	33.5	74	3.8	2	33.5	78	2.1	2	33.5	84	1.9
2.5	33.5	62	3.9	2.5	33.5	70	2.3	2.5	33.5	76	2.1
3	33.5	52	3.8	3	33.5	62	2.3	3	33.5	64	2.1
3.5	33.5	50	4.2	3.5	33.5	54	2.3	3.5	33.5	60	2.2
2	40.2	86	4.3	2	40.2	96	2.5	2	40.2	96	2.1
2.5	40.2	76	4.7	2.5	40.2	82	2.6	2.5	40.2	92	2.5
3	40.2	62	4.4	3	40.2	74	2.7	3	40.2	72	2.3
3.5	40.2	56	4.5	3.5	40.2	66	2.7	3.5	40.2	70	2.5

From Table 4 for Beam type 2, the following observations were to be made by considering different a/d ratio, tensile reinforcement ratio, concrete compressive strength, and beam size.

For less a/d = 1.98, p = 0.031 with different f'_c = 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², the observed increased total shear capacity were to be 38, 44, 58, 74, and 86 KN which indicates that the beam size has more influence on the increment (126.31%) of total shear capacity and observed displacements were to be 2.10, 2.40, 2.97, 3.86, and 4.34 mm.

For increased a/d = 3.46, p = 0.031 with different f'_c = 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², the observed decrease total shear capacity were to be 32, 34, 42, 50, and 56 KN which is observed to be more (75%) as when compared to Beam type 1 with (p = 0.031, and a/d = 4.11) and correspondingly observed displacements were to be 3.10, 3.10, 3.56, 4.24, and 4.50 mm.

From Table 4 for lesser a/d = 1.98, p = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , there is a tendency of increase (11.63%) to be observed in the total shear capacity as 46, 54, 72, 78 and 96 KN as when compared to (a/d = 1.98, p = 0.031) and also this increment in total shear capacity is far more (71.45%) as when compared to Beam type 1 (a/d = 2.35, p = 0.096) and observed decreased displacements were to be 1.35, 1.57, 1.96, 2.12, and 2.55 mm as when compared to (a/d = 1.98, p = 0.031).

Similarly for increased a/d = 3.46, p = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , the observed decreased total shear capacity were to be as 36, 44, 50, 54, and 66 KN which is to be more (17.86%) as when compared to (a/d = 3.46, p = 0.031) and also observed to more (43.47%) as when compared to Beam type 1 with (a/d = 4.11, p = 0.096) and observed displacements were to be 1.69, 2.1, 2.21, 2.34, and 2.74 mm which is to be low as when compared to (a/d = 3.46, p = 0.031) which in turn more as when compared to Beam type 1 with (a/d = 3.46, p = 0.031) .

Similarly for lesser a/d = 1.98, p = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , the observed total shear capacity to be same in some case and show little bit more increase in total shear capacity as when compared with (a/d = 1.98, p = 0.096) and their values were to be 46, 56, 68, 84 and 96 KN but this shear capacity observed were to be more (71.43%) as when compared to Beam type 1 (a/d = 2.35, p = 0.126) which in turn indicates the effect of beam size and observed displacements were to be 1.16, 1.41, 1.57, 1.98, and 2.18 mm which is to be low as when compared to (a/d = 1.98, p = 0.096) .

Also with increased a/d = 3.46, p = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , the observed decreased total shear capacity were to be 36, 50, 54, 60, and 70 KN but this value is more (45.83%) as when compared to Beam type 1 with (a/d = 4.11, p = 0.126) and observed displacements were to be 1.44, 2.10, 2, 2.27, and 2.51 mm which is to be low in almost all a/d ratio's as when compared to (a/d = 3.46, p = 0.031) .

Beam type 3

Table 5 – Comparison of mid-span Deflections with different a/d ratio (Beam type 3)

p = 0.03, a/d	f'$_c$, N/mm²	S$_c$, KN	D, mm	p = 0.096, a/d	f'$_c$, N/mm²	S$_c$, KN	D, mm	p = 0.13, a/d	f'$_c$, N/mm²	S$_c$, KN	D, mm
1.5	16.8	42	1.09	1.5	16.8	44	0.5	1.5	16.8	48	0.56
1.9	16.8	44	1.46	1.9	16.8	42	0.7	1.9	16.8	44	0.63
2.3	16.8	44	1.77	2.3	16.8	42	0.8	2.3	16.8	42	0.72
2.7	16.8	46	2.18	2.7	16.8	44	1.0	2.7	16.8	44	0.87
1.5	20.1	58	1.57	1.5	20.1	50	0.6	1.5	20.1	54	0.64
1.9	20.1	58	2.00	1.9	20.1	52	0.8	1.9	20.1	54	0.79
2.3	20.1	54	2.23	2.3	20.1	58	1.2	2.3	20.1	60	1.09
2.7	20.1	54	2.59	2.7	20.1	58	1.3	2.7	20.1	60	1.25
1.5	26.8	66	1.63	1.5	26.8	72	0.9	1.5	26.8	70	0.78
1.9	26.8	66	2.10	1.9	26.8	70	1.1	1.9	26.8	70	0.97
2.3	26.8	66	2.53	2.3	26.8	66	1.2	2.3	26.8	66	1.09
2.7	26.8	66	2.93	2.7	26.8	66	1.4	2.7	26.8	70	1.33
1.5	33.5	90	2.29	1.5	33.5	92	1.2	1.5	33.5	94	1.09
1.9	33.5	90	2.92	1.9	33.5	90	1.5	1.9	33.5	90	1.29
2.3	33.5	84	3.25	2.3	33.5	84	1.6	2.3	33.5	88	1.49
2.7	33.5	78	3.43	2.7	33.5	84	1.8	2.7	33.5	84	1.60
1.5	40.2	96	2.35	1.5	40.2	108	1.4	1.5	40.2	110	1.25
1.9	40.2	106	3.39	1.9	40.2	102	1.6	1.9	40.2	104	1.47
2.3	40.2	96	3.61	2.3	40.2	98	1.9	2.3	40.2	98	1.64
2.7	40.2	92	4.0	2.7	40.2	94	2.1	2.7	40.2	96	1.83

From Table 5 for Beam type 3 with lesser a/d = 1.53, p = 0.031 with different f'$_c$ = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , the observed increased total shear capacity were to be 42, 58, 66, 90, and 96 KN which is more (11.63%) and decreased displacements were to be noted as 1.10, 1.57, 1.63, 2.29, and 2.35 mm as when compared to Beam type 2 with (a/d = 1.98, p = 0.031) .

For increased a/d = 2.69, p = 0.031 with different f'$_c$ = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , it's observed that total shear capacity were to be 46, 54, 66, 78, and 92 decreased (4.35%) as when compared with (a/d = 1.53, p = 0.031), but this observed shear capacity is more (64.29%) as when compared to Beam type 2 with (a/d = 3.46, p = 0.031) and observed displacements were to be 2.18, 2.59, 2.93, 3.43, and 4.01 mm which appears to be low as when compared to Beam type 2 with (a/d = 3.46, p = 0.031) .

Again from Table 5 for Beam type 3 for small a/d = 1.53, p = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm^2 , shows there is an increased (12.50%) tendency in the total shear capacity were to be observed as 44, 50, 72, 92, and 108 KN as when compared with (a/d = 1.53, p = 0.031) which in turn indicates there is an influence of tensile reinforcement ratio on this incremental shear capacity but this total shear capacity was less in some cases but increased more (12.50%) as when compared to Beam type 2 with (a/d = 1.98, p = 0.096) and observed displacements were to be 0.59, 0.67, 0.96, 1.26, and 1.42 mm which is to be less as when compared with (a/d = 1.53, p = 0.031) and which is also less as compared to Beam type 2 with (a/d = 1.98, p = 0.096) .

Also for increased a/d = 2.69, p = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm^2 , there is decreased (14.89%) tendency of total shear capacity were to be observed as 44, 58, 66, 84, and 94 KN as when compared with (a/d = 1.53, p = 0.096) and this observed total shear capacity was more (42.42%) as when compared to Beam type 2 with (a/d = 3.46, p = 0.096) and observed displacements were to be 1.01, 1.39, 1.45, 1.85, and 2.1 mm which is to be less deflected as when compared with (a/d = 2.69, p = 0.031) .

From Table 5 for less a/d = 1.53, p = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm^2 , the observed increased total shear capacity were to be 48, 54, 70, 94, and 110 KN and their displacements as 0.56, 0.64, 0.78, 1.09, and 1.25 mm which is to be less as when compared to (a/d = 1.53, p = 0.096) and in turn total shear capacity is more (1.85%) as when compared to (a/d= 1.53, p = 0.096) but also this shear capacity was observed to be more (14.58%) as when compared to Beam type 2 with (a/d = 1.98, p = 0.126) .

Similarly for increased a/d = 2.69 , p = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50 and 40.20 N/mm^2 , the observed decreased total shear capacity were to be 44, 60, 70, 84, and 96 KN with their displacements were to be 0.87, 1.25, 1.33, 1.60 and 1.83 mm . This total shear capacity was found to be more (2.13%) as when compared to (a/d = 2.69, p = 0.096) and this obtained total shear capacity was found to be more (37.14%) as compared with Beam type 2 with (a/d = 3.46, p = 0.126) .

Beam type 4

Table 6 – Comparison of mid-span Deflections with different a/d ratio (Beam type 4)

p = 0.03, a/d	f'c, N/mm²	Sc, KN	D, mm	p = 0.096, a/d	f'c, N/mm²	Sc, KN	D, mm	p = 0.13, a/d	f'c, N/mm²	Sc, KN	D, mm
1.1	16.8	56	0.6	1.1	16.8	56	0.30	1.1	16.8	58	0.2
1.4	16.8	58	0.8	1.4	16.8	62	0.47	1.4	16.8	60	0.4
1.7	16.8	64	1.1	1.7	16.8	62	0.57	1.7	16.8	62	0.4
2	16.8	64	1.3	2	16.8	68	0.79	2	16.8	66	0.6
1.1	20.1	66	0.7	1.1	20.1	66	0.41	1.1	20.1	68	0.3
1.4	20.1	82	1.3	1.4	20.1	68	0.54	1.4	20.1	68	0.4
1.7	20.1	78	1.4	1.7	20.1	68	0.66	1.7	20.1	70	0.5
2	20.1	80	1.7	2	20.1	82	0.96	2	20.1	84	0.8
1.1	26.8	96	1.0	1.1	26.8	102	0.64	1.1	26.8	100	0.5
1.4	26.8	88	1.2	1.4	26.8	96	0.75	1.4	26.8	100	0.6
1.7	26.8	88	1.4	1.7	26.8	86	0.79	1.7	26.8	98	0.8
2	26.8	88	1.7	2	26.8	88	0.93	2	26.8	88	0.8
1.1	33.5	122	1.3	1.1	33.5	120	0.77	1.1	33.5	116	0.6
1.4	33.5	122	1.7	1.4	33.5	122	0.98	1.4	33.5	116	0.8
1.7	33.5	112	1.8	1.7	33.5	122	1.17	1.7	33.5	116	0.9
2	33.5	118	2.3	2	33.5	106	1.14	2	33.5	116	1.1
1.1	40.2	140	1.5	1.1	40.2	142	0.91	1.1	40.2	146	0.8
1.4	40.2	138	1.9	1.4	40.2	144	1.14	1.4	40.2	142	0.9
1.7	40.2	130	2.1	1.7	40.2	136	1.29	1.7	40.2	140	1.1
2	40.2	144	2.8	2	40.2	136	1.47	2	40.2	134	1.2

From Table 6 (Beam type 4) for lesser a/d = 1.14, p = 0.031 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , the observed increased total shear capacity were to be (56, 66, 96, 122, and 140 KN) and their displacements were to be 0.63, 0.75, 1.05, 1.37, and 1.53 mm, which indicates that for lesser a/d ratio and increased beam size in turn achieve more total shear capacity with increase in concrete compressive strength . This increased total shear capacity was found to be more (45.83%) as when compared to Beam type 3 with (a/d = 1.53, p = 0.031) and corresponding displacements were found to be less as when compared to Beam type 3 with (a/d = 1.53, p =0.031) .

Similarly for increased a/d = 2, p =0.031 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , the observed total shear capacity were to be varied from 64, 80, 88, 118, and 144 KN with their displacements were to be 1.35, 1.75, 1.71, 2.37, and 2.89 mm . The observed total shear capacity were found to be more (56.52%) as when compared to Beam type 3 with (a/d = 2.69, p = 0.031). Also observed displacements were found to be less in some cases as when compared to (a/d = 2.69, p =0.031).

For Beam type 4 with increased a/d = 1.14, p = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm^2 , the observed total shear capacity were to be slightly increased or decreased with their magnitude as 56, 66, 102, 120, and 142 KN with displacements as 0.30, 0.41, 0.64, 0.77, and 0.91 mm . The observed total shear capacity were found to be less or less same as when compared to (a/d = 1.14, p = 0.031) but their displacements were found to be less as when compared to (a/d = 1.14, p = 0.031), but this shear capacity was more (31.48%) as when compared to Beam type 3 with (a/d = 1.53, p = 0.096).

With increased a/d = 2, p = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm^2 , the observed total shear capacity were to be 68, 82, 88, 106, and 136 KN with their displacements were to be 0.79, 0.96, 0.93, 1.14, and 1.47 mm. The observed shear capacity was found to be more (44.68%) as when compared to Beam type 3 with (a/d = 2.69, p = 0.096), and also observed displacements were found to be lesser as when compared to Beam type 3 with (a/d = 2.69, p = 0.096).

For Beam type 4 with a/d = 1.14, p = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm^2 , the observed total shear capacity were found to be(58, 68, 100, 116, and 146 KN) little bit increased as when compared to (a/d = 1.14, p = 0.096) and their displacements were to be 0.27, 0.37, 0.54, 0.64, and 0.82 mm which is also found to be lesser as when compared to (a/d = 1.14, p = 0.096) . But the total shear capacity was found to be more (32.72%) when compared to Beam type 3 with (a/d = 1.53, p = 0.126).

When a/d = 2, p = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm^2 , the observed total shear capacity were to be 66, 84, 88, 116, and 134 KN which is found to be more (39.58%) as when compared to Beam type 3 with (a/d = 2.69, p = 0.126) and their displacements were to be 0.65, 0.84, 0.81, 1.10, and 1.25 mm which is to be less as when compared with (a/d =2, p = 0.096).

Beam type 5

Table 7 – Comparison of mid-span Deflections with different a/d ratio (Beam type 5)

p = 0.03, a/d	f'c, N/mm²	Sc, KN	D, mm	p = 0.096, a/d	f'c, N/mm²	Sc, KN	D, mm	p = 0.13, a/d	f'c, N/mm²	Sc, KN	D, mm
0.8	16.8	100	0.3	0.8	16.8	94	0.1	0.8	16.8	94	0.12
0.9	16.8	100	0.4	0.9	16.8	98	0.2	0.9	16.8	98	0.23
1.1	16.8	106	0.6	1.1	16.8	100	0.3	1.1	16.8	100	0.29
1.3	16.8	108	0.7	1.3	16.8	100	0.3	1.3	16.8	102	0.36
0.8	20.1	110	0.3	0.8	20.1	112	0.2	0.8	20.1	112	0.19
0.9	20.1	108	0.4	0.9	20.1	114	0.3	0.9	20.1	114	0.27
1.1	20.1	128	0.7	1.1	20.1	114	0.3	1.1	20.1	114	0.33
1.3	20.1	130	0.9	1.3	20.1	128	0.5	1.3	20.1	116	0.41
0.8	26.8	142	0.4	0.8	26.8	146	0.3	0.8	26.8	150	0.31
0.9	26.8	148	0.6	0.9	26.8	148	0.4	0.9	26.8	154	0.39
1.1	26.8	146	0.7	1.1	26.8	152	0.5	1.1	26.8	152	0.47
1.3	26.8	170	1.1	1.3	26.8	150	0.5	1.3	26.8	150	0.53
0.8	33.5	170	0.5	0.8	33.5	168	0.3	0.8	33.5	174	0.36
0.9	33.5	174	0.7	0.9	33.5	174	0.5	0.9	33.5	170	0.44
1.1	33.5	174	0.9	1.1	33.5	176	0.6	1.1	33.5	174	0.56
1.3	33.5	206	1.3	1.3	33.5	178	0.7	1.3	33.5	180	0.66
0.8	40.2	210	0.7	0.8	40.2	196	0.4	0.8	40.2	198	0.42
0.9	40.2	200	0.8	0.9	40.2	202	0.5	0.9	40.2	206	0.54
1.1	40.2	204	1.0	1.1	40.2	208	0.7	1.1	40.2	204	0.65
1.3	40.2	200	1.2	1.3	40.2	206	0.8	1.3	40.2	210	0.7

From Table 7 (Beam type 5) For lesser a/d = 0.75, p = 0.031 with different f'c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , the observed total shear capacity were to be 100, 110, 142, 170, and 210 KN and their displacements were to be 0.33, 0.37, 0.47, 0.58, and 0.72 mm . The observed shear capacity was found to be more (50%) as compared to Beam type 4 with (a/d = 1.14, p = 0.031) and the observed displacements were also lesser as compared to Beam type 4 with (a/d = 1.14, p = 0.031) which in turn indicates that the beam size has more effect on the increment of total shear capacity and also one more factor is to be considered in this type of beam i.e. Beam type 5 with a/d = 0.75, which is very small as when compared to all other types of Beam .

Also for a/d = 1.32, p = 0.031 with different f'c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm² , the observed total shear capacity were found to be 108, 130, 170, 206, and 200 KN with their displacements were to be 0.75, 0.92, 1,12, 1.36, and 1.22 mm . The observed shear capacity was more (38.89%) as when compared to Beam type 4 with (a/d = 2, p = 0.031) but their observed displacements were to be less compared to Beam type 4 with (a/d = 2, p = 0.031).

For lesser a/d = 0.75, p = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm², the observed total shear capacity were to be 94, 112, 146, 168, and 196 KN and their displacements were to be 0.17, 0.23, 0.32, 0.38, and 0.46 mm . The observed shear capacity was found to be varied little bit (7.14%) with (a/d = 0.75, p = 0.031) .The observed displacements were found to very less as compared to with (a/d = 0.75, p = 0.031). But the observed shear capacity was found to be more (38%) as when compared to Beam type 4 with (a/d = 1.14, p = 0.096).

Similarly with a/d = 1.32, p = 0.096 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm², the observed total shear capacity were to be 100, 128, 150, 178, and 206 KN and their displacements were to be 0.39, 0.56, 0.59, 0.73, and 0.83 mm . The observed shear capacity was found to be more (51.47%) as compared to Beam type 4 with (a/d = 2, p = 0.096) .The observed displacements were found to less as compared to Beam type 4 with (a/d = 2, p = 0.096).

For a/d = 0.75, p = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm², the observed total shear capacity were to be 94, 112, 150, 174, and 198 KN and their displacements were to be 0.12, 0.19, 0.31, 0.36, and 0.42 mm . The observed shear capacity was found to be increased (1.02%) as when compared with (a/d = 0.75, p = 0.096) .The observed displacements were found to very less as when compared to Beam type 4 with (a/d = 1.14, p = 0.126). The observed shear capacity was found to be more (35.61%) as compared to Beam type 4 with (a/d = 1.14, p = 0.126) .

For a/d = 1.32, p = 0.126 with different f'_c = 16.75 , 20.10, 26.80, 33.50, and 40.20 N/mm², the observed total shear capacity was 102, 116, 150, 180, and 210 KN and their displacements were to be 0.36, 0.41, 0.53, 0.66, and 0.78 mm . The observed shear capacity was found to be more (56.71%) as compared to Beam type 4 with (a/d = 2, p = 0.126) .The observed displacements were found to lesser as when compared to Beam type 4 with (a/d = 2, p = 0.126).

The variation of Nominal shear stress versus effective shear span to depth ratio by considering different combinations of concrete compressive strength , tensile reinforcement ratio, a/d ratio , different beam size were to be represented from figures 6 to 20 .

Beam type 1

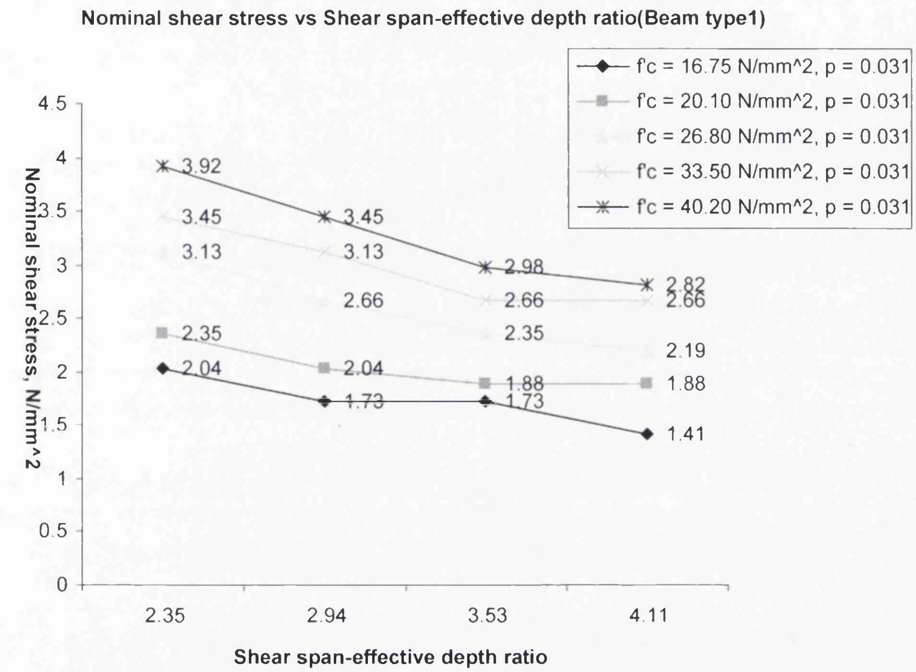

Fig.6 Effect of Shear span-effective depth ratio on Nominal shear stress

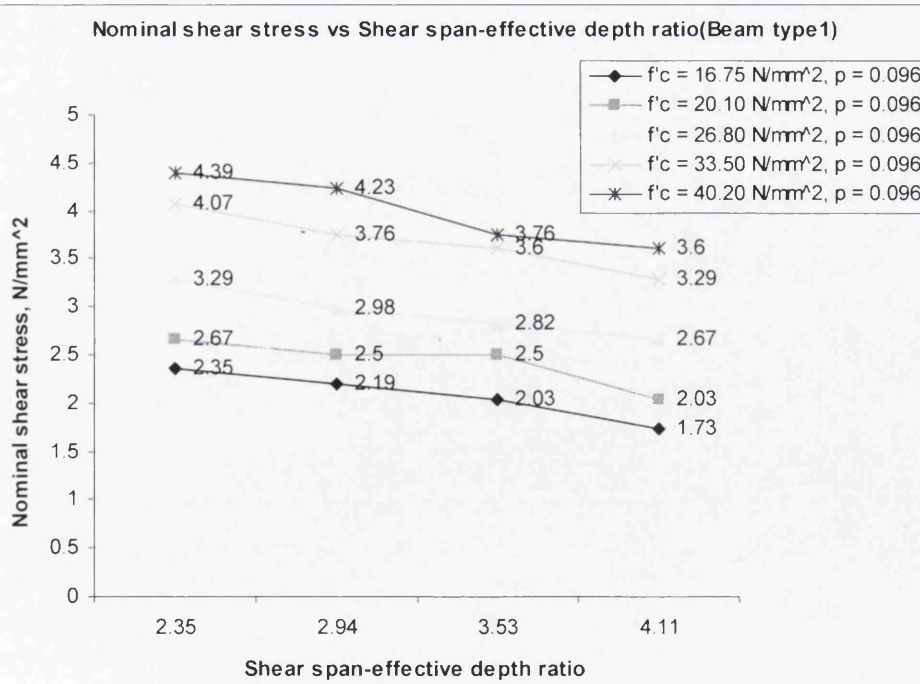

Fig.7 Effect of Shear span-effective depth ratio on Nominal shear stress

Fig. 6 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(ρ) of 0.031 and shear span-effective depth ratio(a/d) of 2.35, 2.94, 3.53, and 4.11. It can be observed that for lesser the a/d ratio of 2.35 more the nominal shear stresses 2.04, 2.35, 3.13, 3.45, and 3.92 N/mm² could be achieved with the increase in concrete compressive strength. It can also observe that for large a/d ratio of 4.11 lesser the nominal shear stresses 1.41, 1.88, 2.19, 2.66, 2.82 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 7 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(ρ) of 0.096 and shear span-effective depth ratio(a/d) of 2.35, 2.94, 3.53, and 4.11. It can be observed that for lesser the a/d ratio of 2.35 more the nominal shear stresses 2.35, 2.67, 3.29, 4.07, and 4.39 N/mm² could be achieved with the increase in concrete compressive strength and tensile reinforcement ratio. It can also observe that for large a/d ratio of 4.11 lesser the nominal shear stresses 1.73, 2.03, 2.67, 3.29, and 3.60 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 8 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(ρ) of 0.126 and shear span-effective depth ratio(a/d) of 2.35, 2.94, 3.53, and 4.11. It can be observed that for lesser the a/d ratio of 2.35 more the nominal shear stresses 2.19, 2.67, 3.45, 4.39, and 4.39 N/mm² could be achieved with the increase in concrete compressive strength. It can also observe that for large a/d ratio of 4.11 lesser the nominal shear stresses 2.03, 2.19, 2.82, 3.29, and 3.67 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio. But contrary to previous observations, in this case tensile reinforcement ratio has no effect on the improvement of the nominal shear stress.

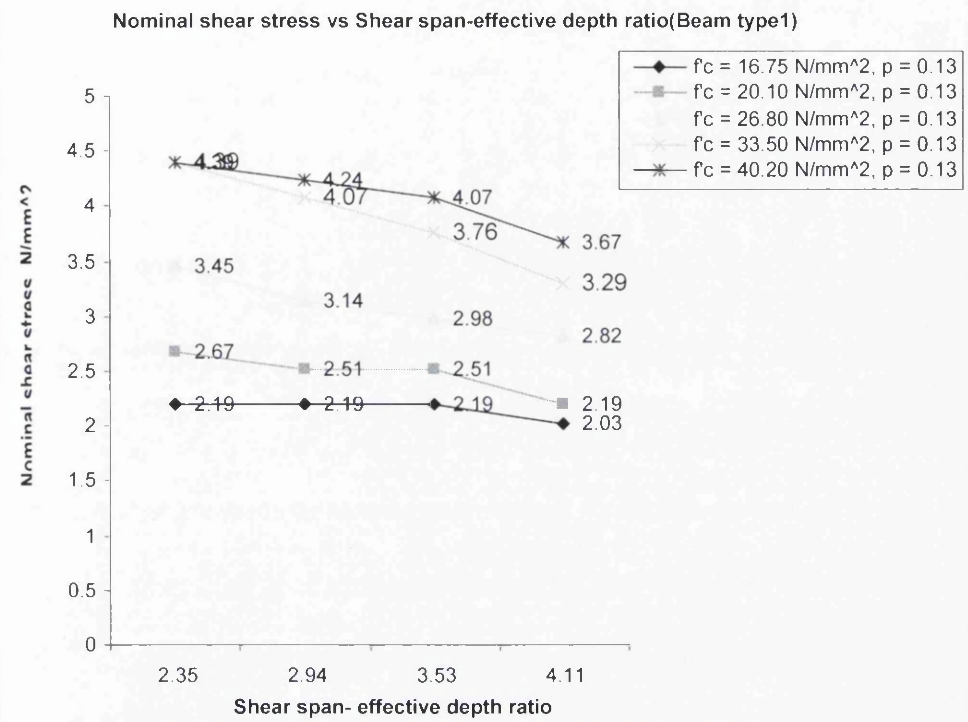

Fig.8 Effect of Shear span-effective depth ratio on Nominal shear stress

Beam type 2

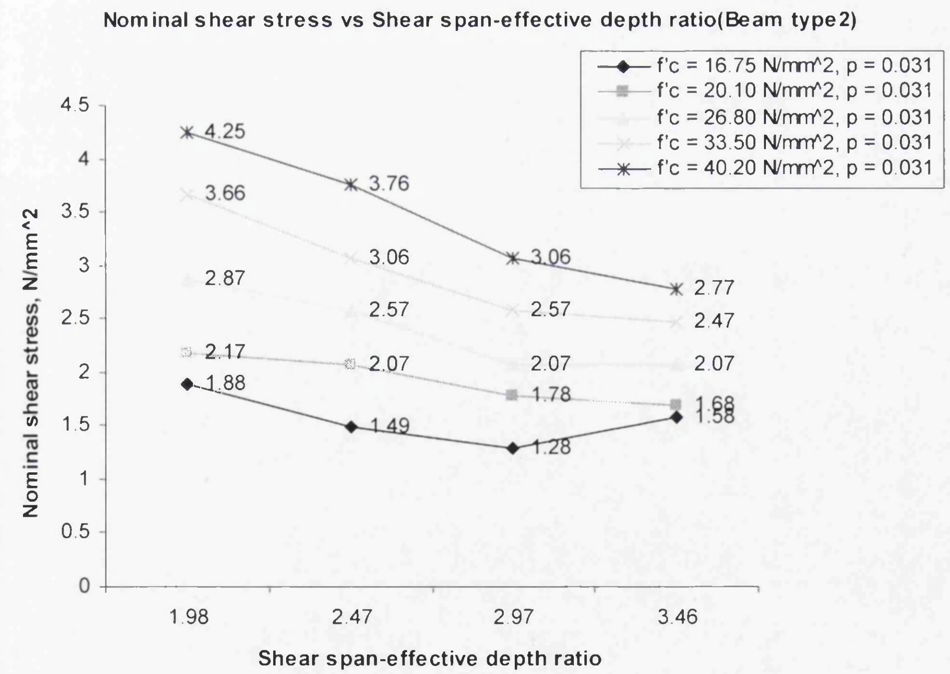

Fig.9 Effect of Shear span-effective depth ratio on Nominal shear stress

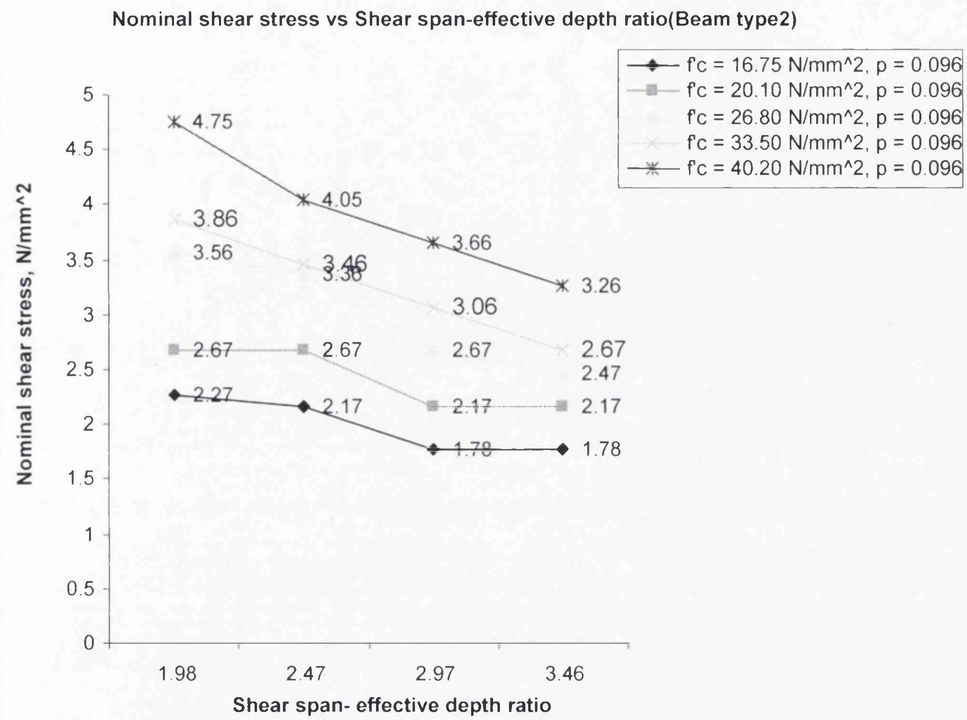

Fig.10 Effect of Shear span-effective depth ratio on Nominal shear stress

Fig. 9 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.031 and shear span-effective depth ratio(a/d) of 1.98, 2.47, 2.97, and 3.46. It can be observed that for lesser the a/d ratio of 1.98 more the nominal shear stresses 1.88, 2.17, 2.87, 3.66, and 4.25 N/mm² could be achieved with the increase in concrete compressive strength and beam size. It can also observe that for large a/d ratio of 3.46 lesser the nominal shear stresses 1.58, 1.68, 2.07, 2.47, and 2.77 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 10 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.096 and shear span-effective depth ratio(a/d) of 1.98, 2.47, 2.97, and 3.46. It can be observed that for lesser the a/d ratio of 1.98 more the nominal shear stresses 2.27, 2.67, 3.56, 3.86, and 4.75 N/mm² could be achieved with the increase in concrete compressive strength and tensile reinforcement ratio. It can also observe that for large a/d ratio of 3.46 lesser the nominal shear stresses 1.78, 2.17, 2.47, 2.67, 3.26 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 11 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.126 and shear span-effective depth ratio(a/d) of 1.98, 2.47, 2.97, and 3.46. It can be observed that for lesser the a/d ratio of 1.98 more the nominal shear stresses 2.27, 2.77, 3.36, 4.16, and 4.75 N/mm² could be achieved but could not altered in the value of shear stress even though with the increase in tensile reinforcement ratio. It can also observe that for large a/d ratio of 3.46 lesser the nominal shear stresses 1.78, 2.47, 2.67, 2.97, 3.46 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

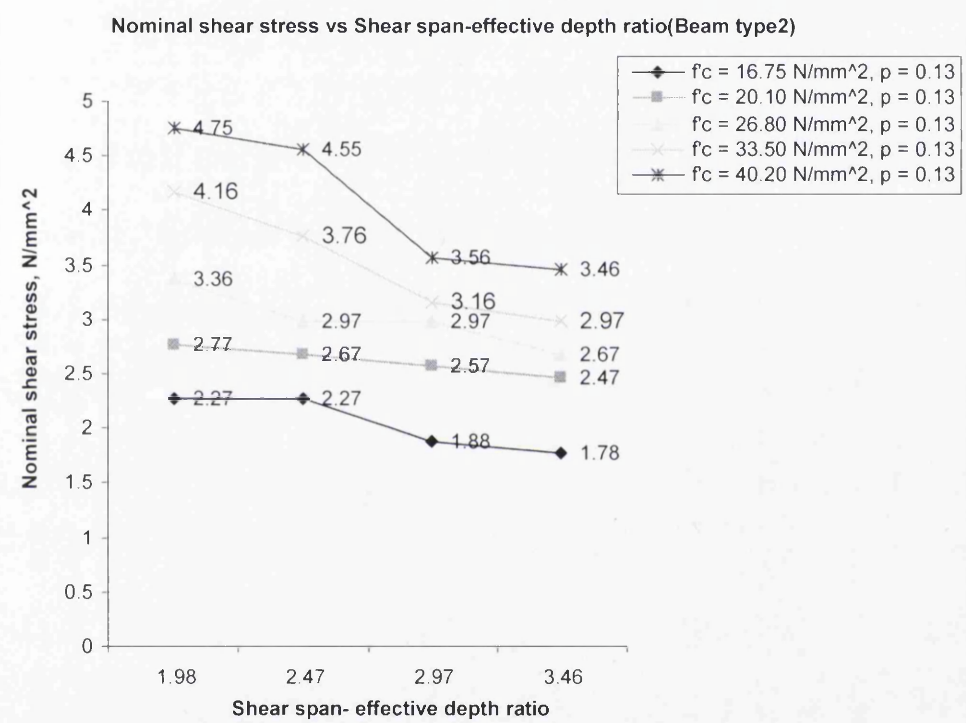

Fig.11 Effect of Shear span-effective depth ratio on Nominal shear stress

Beam type 3

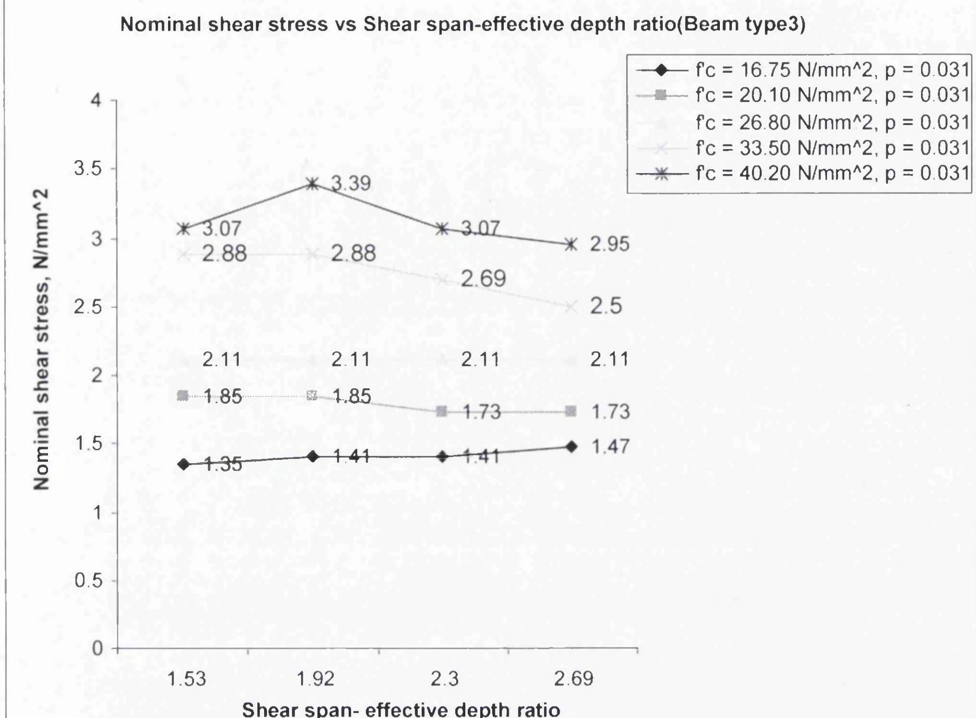

Fig.12 Effect of Shear span-effective depth ratio on Nominal shear stress

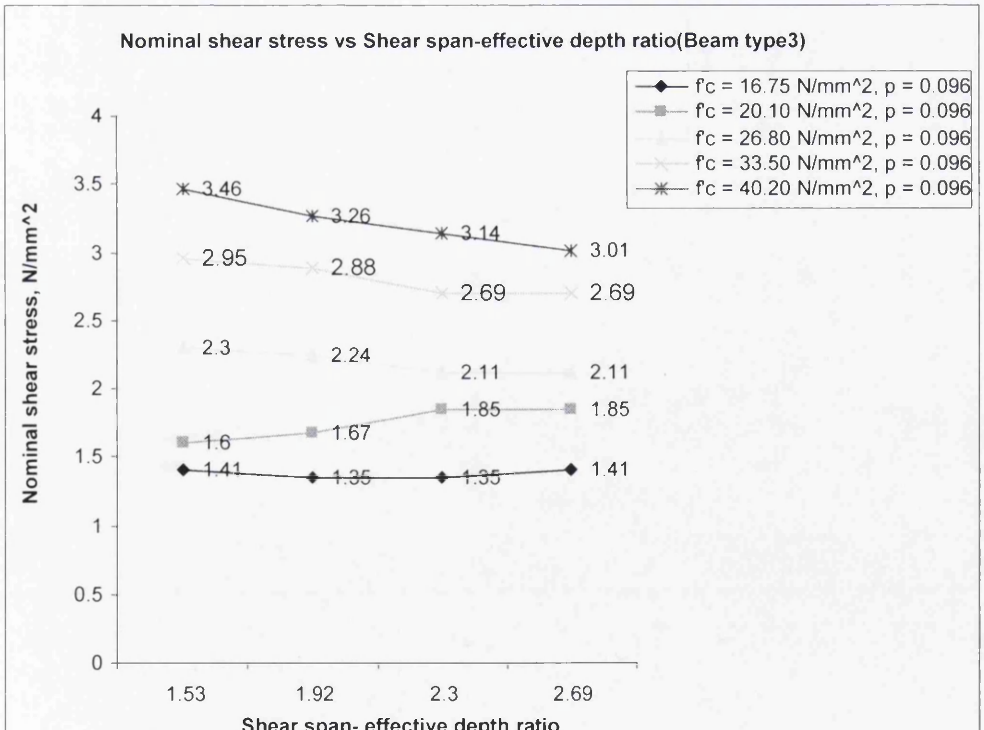

Fig.13 Effect of Shear span-effective depth ratio on Nominal shear stress

Fig. 12 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(ρ) of 0.031 and shear span-effective depth ratio(a/d) of 1.53, 1.92, 2.3, and 2.69. It can be observed that for lesser the a/d ratio of 1.53 more the nominal shear stresses 1.35, 1.85, 2.11, 2.88, and 3.07 N/mm² could be achieved with the increase in concrete compressive strength, but there has been no effect of increased beam size on the improvement of nominal shear stress. It can also observe that for large a/d ratio of 2.69 lesser the nominal shear stresses 1.47, 1.73, 2.11, 2.5, 2.95 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 13 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(ρ) of 0.096 and shear span-effective depth ratio(a/d) of 1.53, 1.92, 2.3, and 2.69. It can be observed that for lesser the a/d ratio of 1.53 more the nominal shear stresses 1.41, 1.6, 2.3, 2.95, and 3.46 N/mm² could be achieved with the increase in concrete compressive strength and tensile reinforcement ratio. It can also observe that for large a/d ratio of 2.69 lesser the nominal shear stresses 1.41, 1.85, 2.11, 2.69, and 3.01 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 14 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(ρ) of 0.126 and shear span-effective depth ratio(a/d) of 1.53, 1.92, 2.3, and 2.69. It can be observed that for lesser the a/d ratio of 1.53 more the nominal shear stresses 1.54, 1.73, 2.24, 3.01, and 3.52 N/mm² could be achieved, but could not be improved with the increase in concrete compressive strength and tensile reinforcement ratio. It can also observe that for large a/d ratio of 2.69 lesser the nominal shear stresses 1.41, 1.92, 2.24, 2.69, and 3.07 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

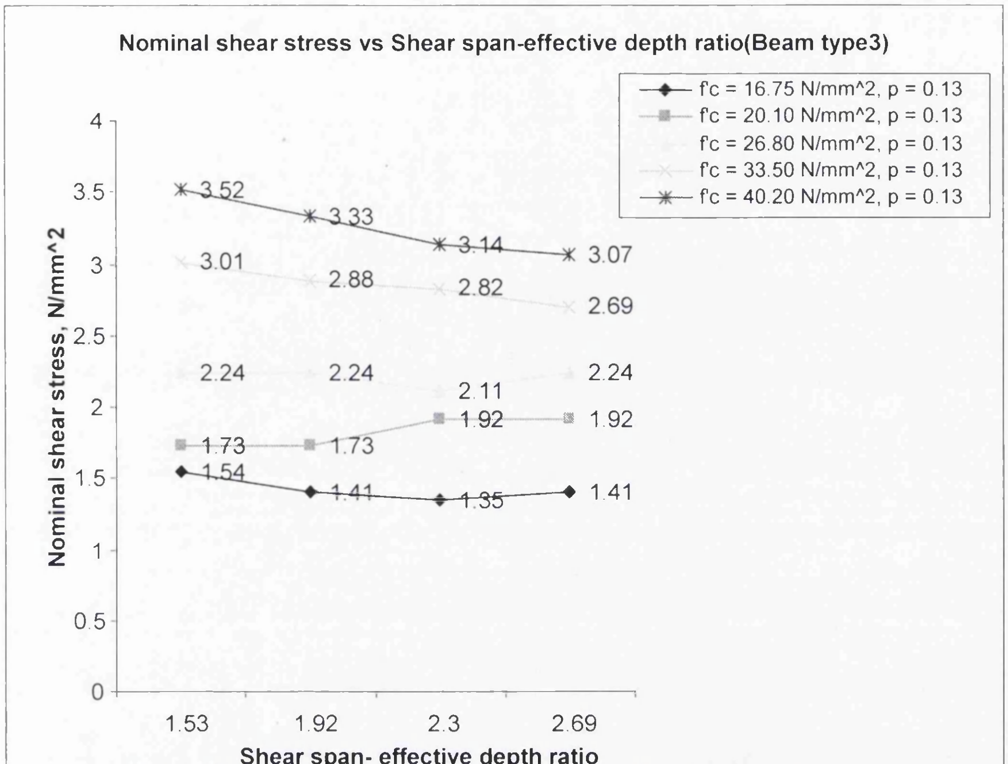

Fig.14 Effect of Shear span-effective depth ratio on Nominal shear stress

Beam type 4

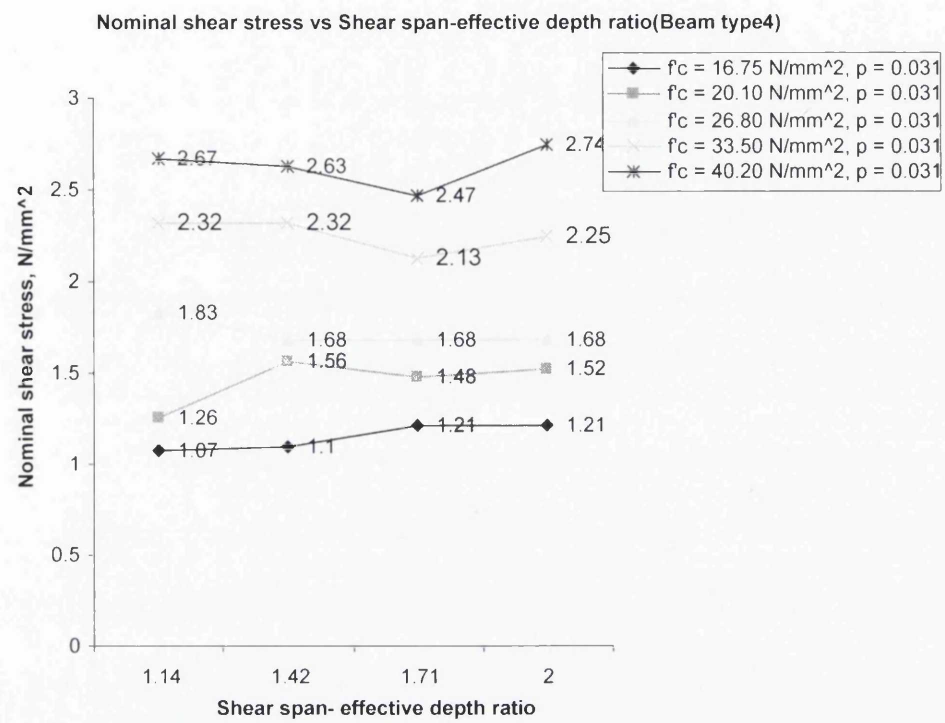

Fig.15 Effect of Shear span-effective depth ratio on Nominal shear stress

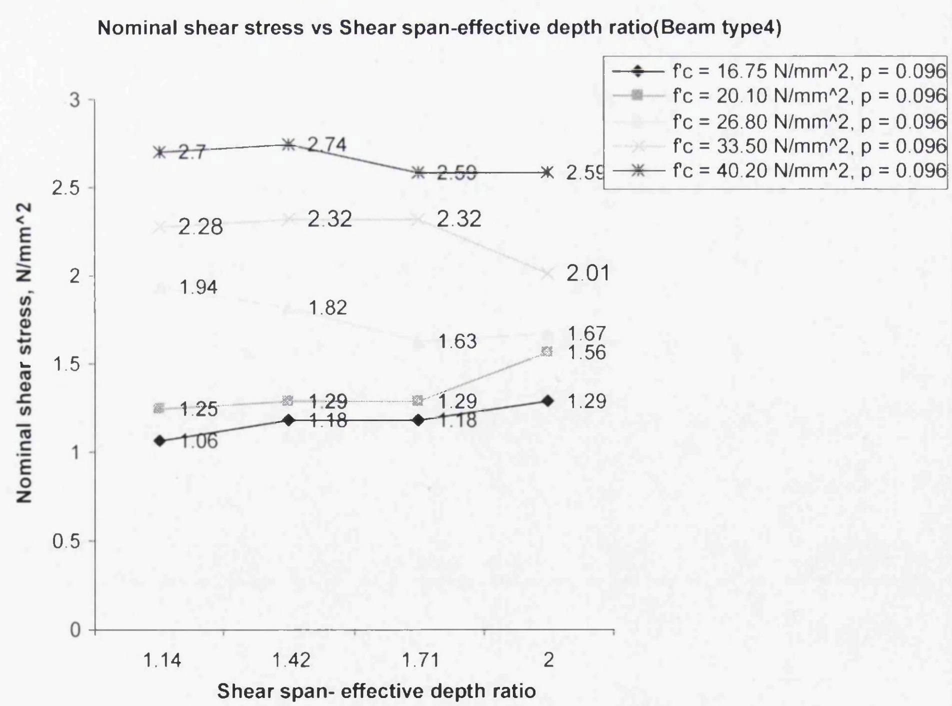

Fig.16 Effect of Shear span-effective depth ratio on Nominal shear stress

Fig. 15 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.031 and shear span-effective depth ratio(a/d) of 1.14, 1.42, 1.71, and 2. It can be observed that for lesser the a/d ratio of 1.14 more the nominal shear stresses 1.07, 1.26, 1.83, 2.32, and 2.67 N/mm² could be achieved with the increase in concrete compressive strength but there has been decrease in nominal shear stress value due to increased beam size. It can also observe that for large a/d ratio of 2 lesser the nominal shear stresses 1.21, 1.52, 1.68, 2.25, 2.74 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 16 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.096 and shear span-effective depth ratio(a/d) of 1.14, 1.42, 1.71, and 2. It can be observed that for lesser the a/d ratio of 1.14 more the nominal shear stresses 1.06, 1.25, 1.94, 2.28, and 2.7 N/mm² could be achieved with the increase in concrete compressive strength but there has been no change in the nominal stress value due to increase in tensile reinforcement ratio. It can also observe that for large a/d ratio of 2 lesser the nominal shear stresses 1.06, 1.25, 1.94, 2.28, 2.7 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 17 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.126 and shear span-effective depth ratio(a/d) of 1.14, 1.42, 1.71, and 2. It can be observed that for lesser the a/d ratio of 1.14 slightly more the nominal shear stresses 1.1, 1.29, 1.9, 2.2, and 2.78 N/mm² could be achieved with the increase in concrete compressive strength and tensile reinforcement ratio. It can also observe that for large a/d ratio of 2 lesser the nominal shear stresses 1.25, 1.68, 1.68, 2.29, 2.55 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

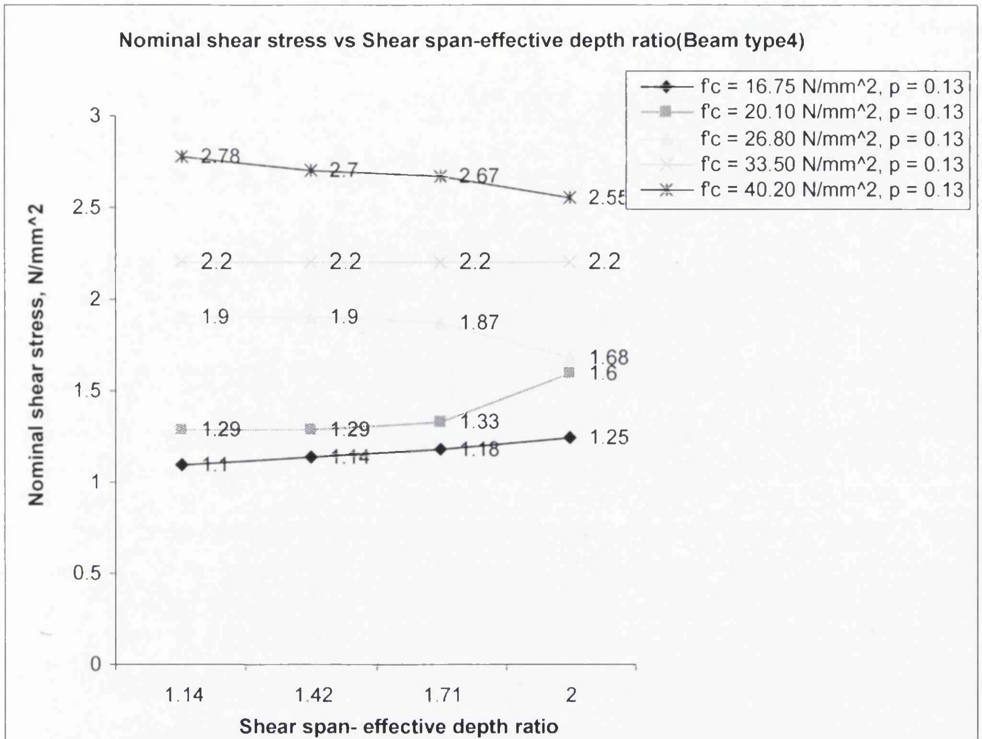

Fig.17 Effect of Shear span-effective depth ratio on Nominal shear stress

Beam type 5

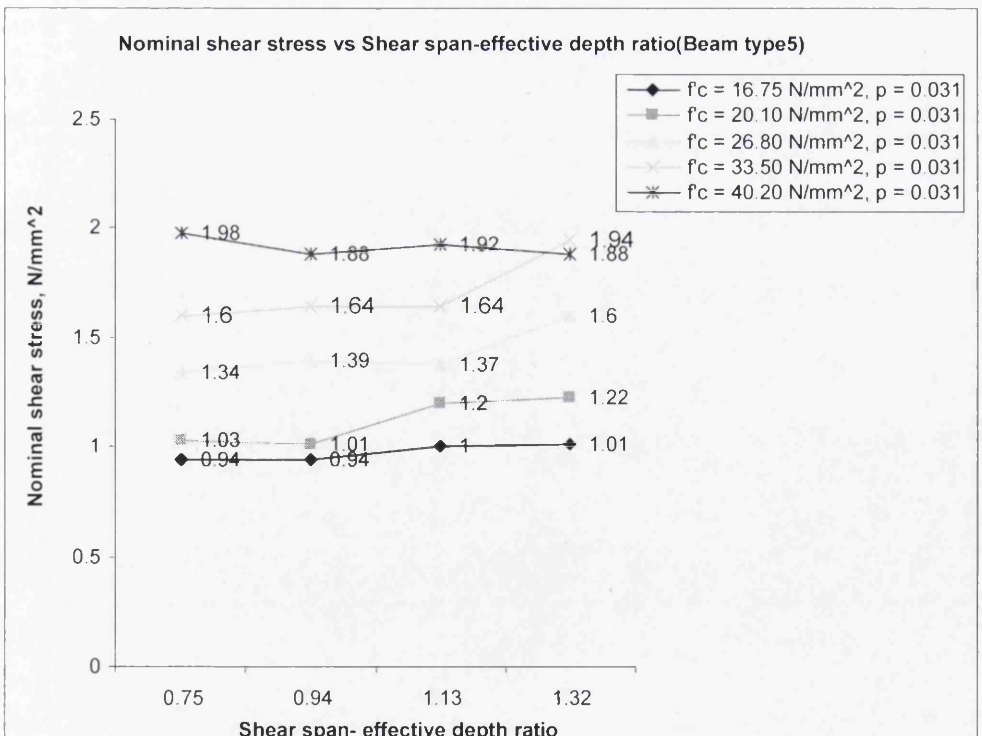

Fig.18 Effect of Shear span-effective depth ratio on Nominal shear stress

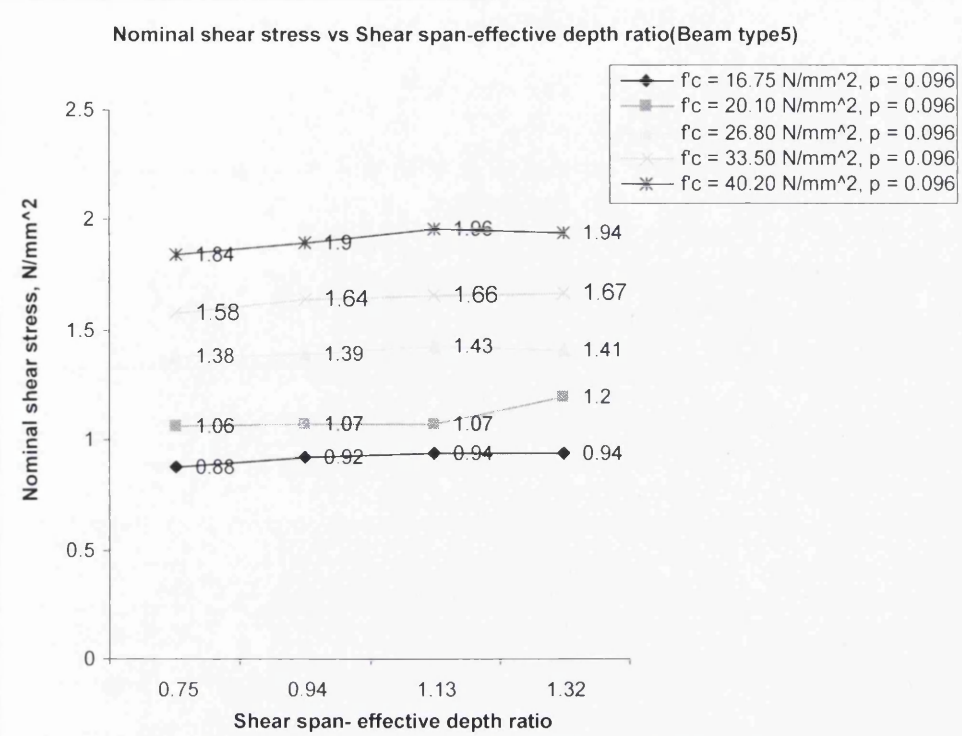

Fig.19 Effect of Shear span-effective depth ratio on Nominal shear stress

Fig. 18 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.031 and shear span-effective depth ratio(a/d) of 0.75, 0.94, 1.13, and 1.32. It can be observed that for lesser the a/d ratio of 0.75 more the nominal shear stresses 0.94, 1.03, 1.34, 1.6, and 1.98 N/mm² could be achieved with the increase in concrete compressive strength but there has been decrease in nominal shear stress value due to increased beam size. It can also observe that for large a/d ratio of 1.32 lesser the nominal shear stresses 1.01, 1.22, 1.6, 1.94, and 1.88 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 19 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.096 and shear span-effective depth ratio(a/d) of 0.75, 0.94, 1.13, and 1.32. It can be observed that for lesser the a/d ratio of 0.75 more the nominal shear stresses 0.88, 1.06, 1.38, 1.58, and 1.84 N/mm² could be achieved with the increase in concrete compressive strength but decreased with increased tensile reinforcement ratio. It can also observe that for large a/d ratio of 1.32 lesser the nominal shear stresses 0.94, 1.2, 1.41, 1.67, 1.94 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

Fig. 20 shows the effects of variation of nominal shear stress with shear span-effective depth ratio for different combinations of concrete compressive strength (f'_c) range from 16.75, 20.10, 26.80, 33.50, and 40.20 N/mm², tensile reinforcement ratio(p) of 0.126 and shear span-effective depth ratio(a/d) of 0.75, 0.94, 1.13, and 1.32. It can be observed that for lesser the a/d ratio of 0.75 slightly more the nominal shear stresses 0.88, 1.06, 1.41, 1.64, and 1.86 N/mm² could be achieved with the increase in concrete compressive strength and tensile reinforcement ratio. It can also observe that for large a/d ratio of 1.32 lesser the nominal shear stresses 0.96, 1.09, 1.41, 1.69, and 1.98 N/mm² could be achieved for respective concrete compressive strength and tensile reinforcement ratio.

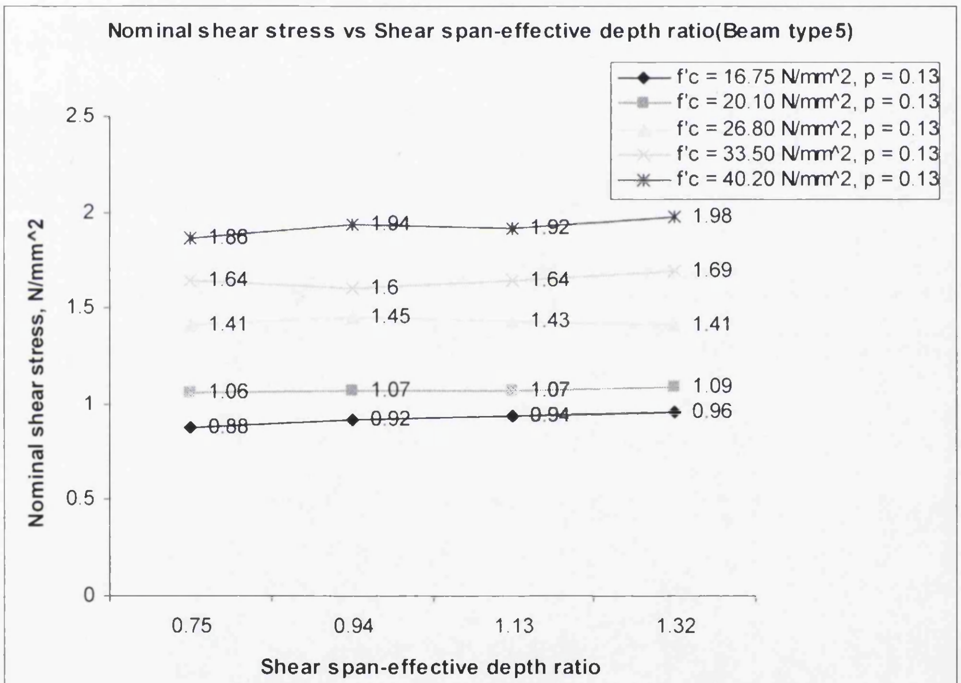

Fig.20 Effect of Shear span-effective depth ratio on Nominal shear stress

CHAPTER 5
ANSYS GRAPHIC RESULTS

The objective of the research work was to predict load carrying capacity, shear failure, and crack mode by using nonlinear analysis for various types of beam size. The following are the Ansys graphic results for various types of beam with their shear failure and crack mode.

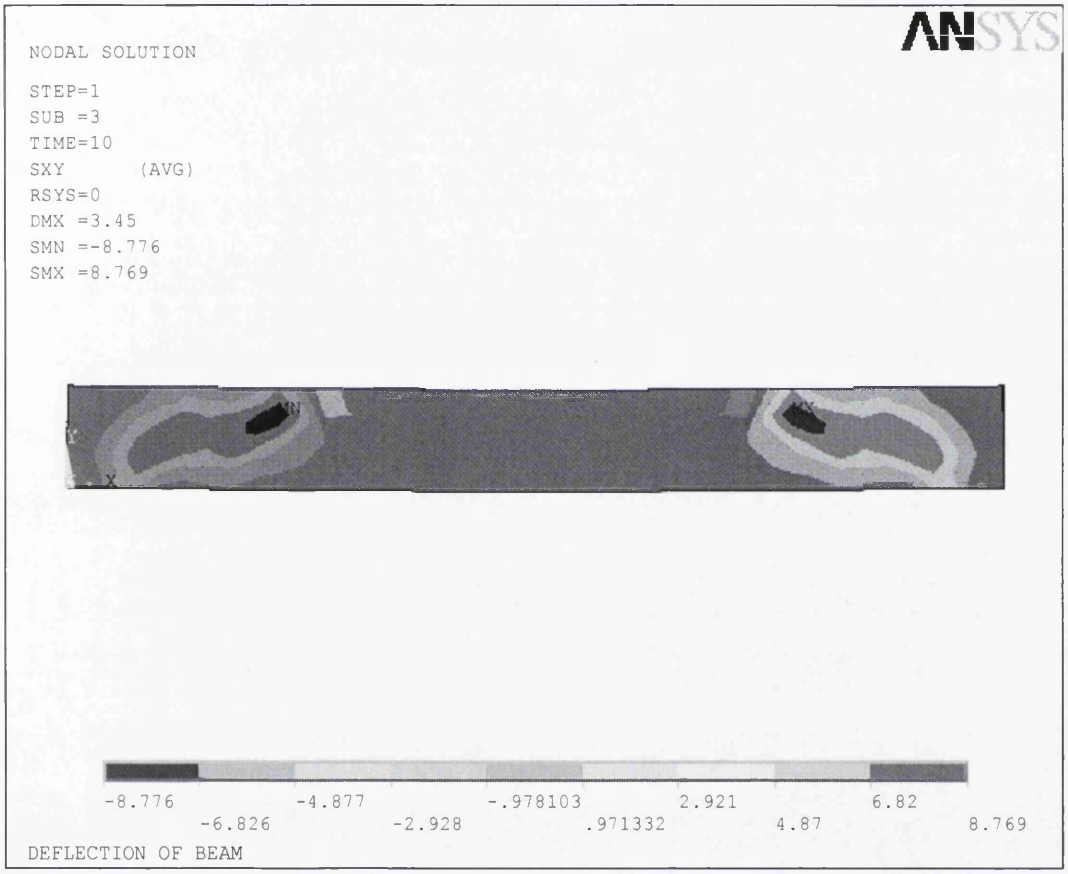

Fig. 21 Shear stress contour for Beam type 1

Fig .21 shows variation of shear stress contour for Beam type 1 with lower a/d ratio of 2.35, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm² and observed shear stress were to be 8.77 N/mm².

From Fig .6 for less a/d = 2.35, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 3.14 N/mm² which is more (2.67 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (3.92 N/mm²) as when compared to f'_c = 40.20 N/mm².

For increased a/d = 4.1, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 2.19 N/mm² which is less (3.45 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (2.82 N/mm²) as when compared to f'_c = 40.20 N/mm².

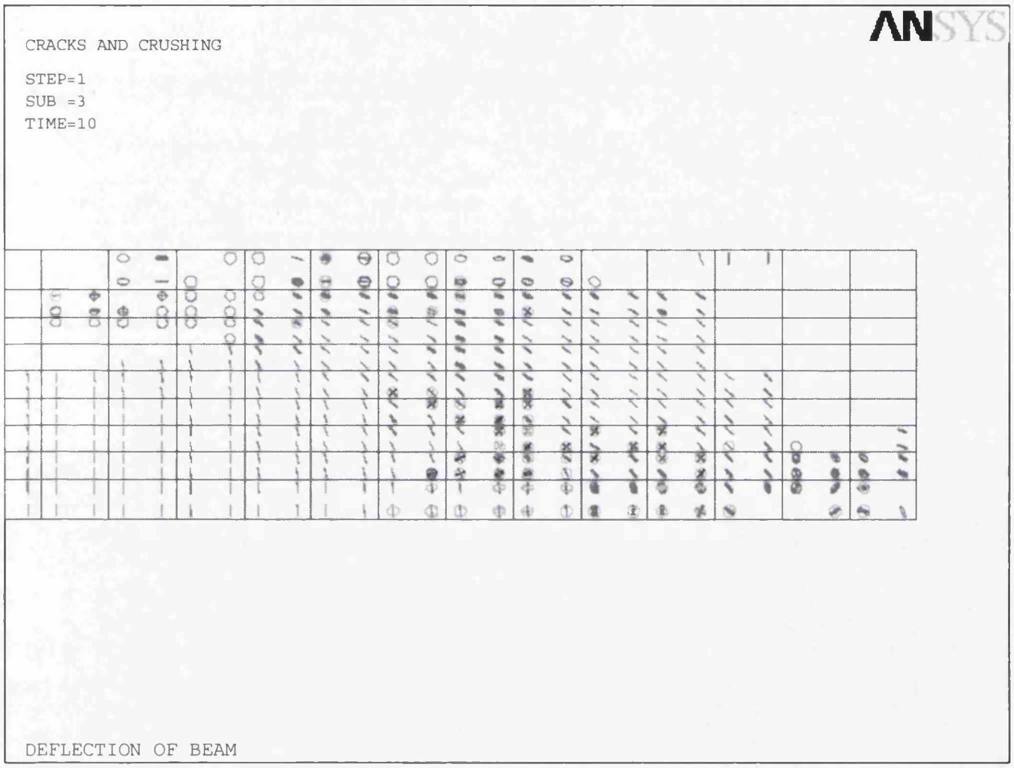

Fig. 22 Crack pattern for half portion of Beam type 1

Fig. 22 shows crack pattern for Beam type 1 with lower a/d ratio of 2.35, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm². In this case, reinforced concrete beam subjected predominantly to flexure failure in between shear span, rare appearance of third crack and crushing of concrete at the top, nearer to support as well as at the bottom side of beam. On the other side shear span subjected rarely to shear failure which slightly tends toward the support.

Fig. 23 Shear stress contour for Beam type 2

Fig .23 shows variation of shear stress contour for Beam type 2 with lower a/d ratio of 1.98, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm² and observed shear stress were to be 7.98 N/mm².

From Fig .9 for less a/d = 1.98, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 2.87 N/mm² which is less (3.66 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (4.25 N/mm²) as when compared to f'_c = 40.20 N/mm².

For increased a/d = 3.5, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 2.07 N/mm² which is less (2.51 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (2.77 N/mm²) as when compared to f'_c = 40.20 N/mm².

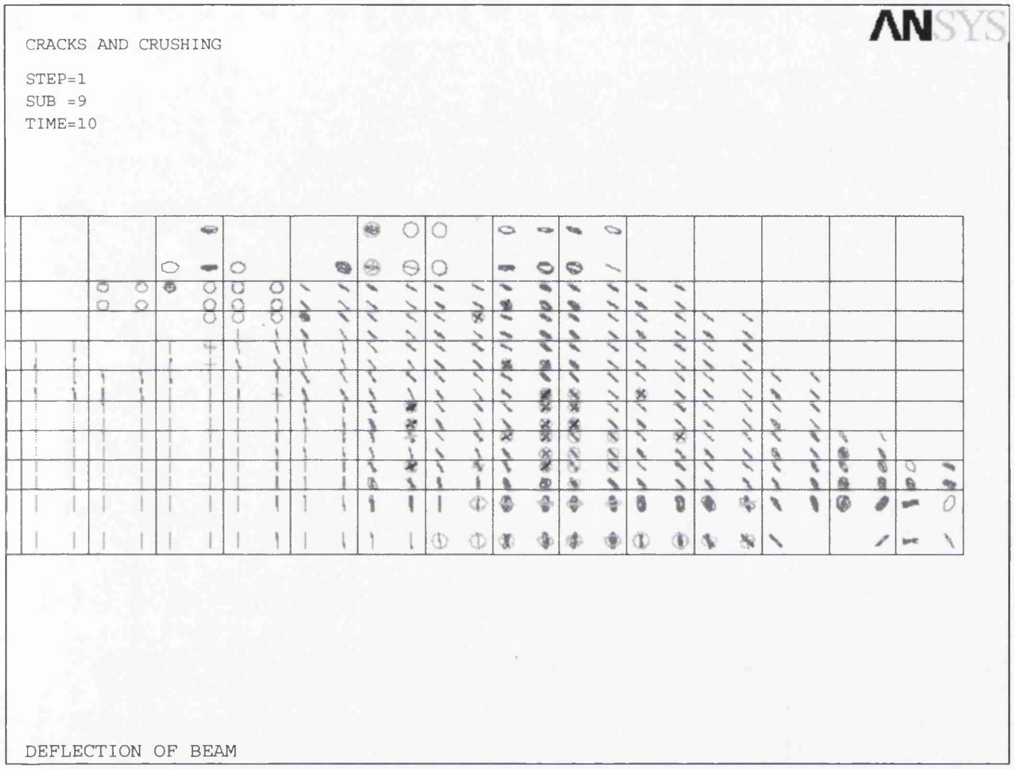

Fig. 24 Crack pattern for half portion of Beam type 2

Fig .24 shows crack pattern for Beam type 2 with lower a/d ratio of 1.98, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm². In this case reinforced concrete beam subjected to flexure failure with varying in their magnitude in between shear span and rarely uniform distribution of third crack at the bottom surface of beam nearer to support. On the other side shear span subjected to shear failure which is more inclined towards the support, rare appearance of third crack and crushing of concrete at top, middle, bottom side (nearer to support) of beam.

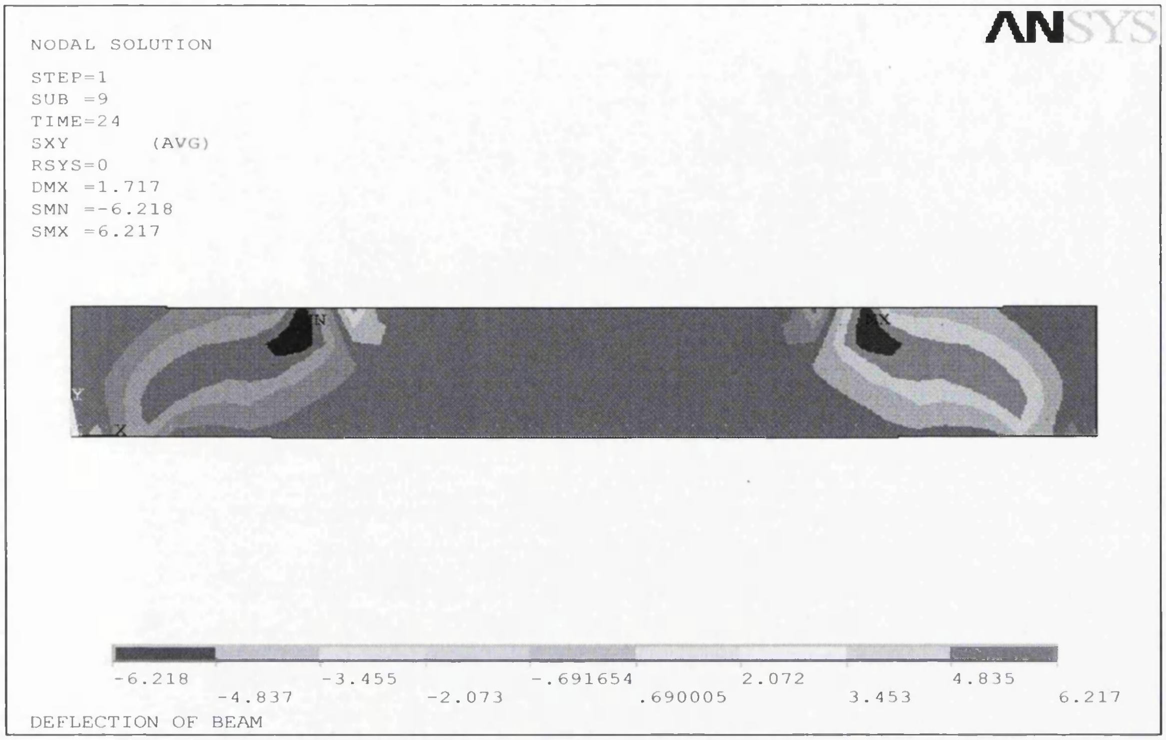

Fig. 25 Shear stress contour for Beam type 3

Fig. 25 shows variation of shear stress contour for Beam type 3 with lower a/d ratio of 1.53, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm² and observed shear stress were to be 6.2 N/mm².

From Fig .12 for less a/d = 1.53, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 2.11 N/mm² which is less (2.88 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (3.07 N/mm²) as when compared to f'_c = 40.20 N/mm².

For increased a/d = 2.7, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 2.11 N/mm² which is less (2.5 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (2.95 N/mm²) as when compared to f'_c = 40.20 N/mm².

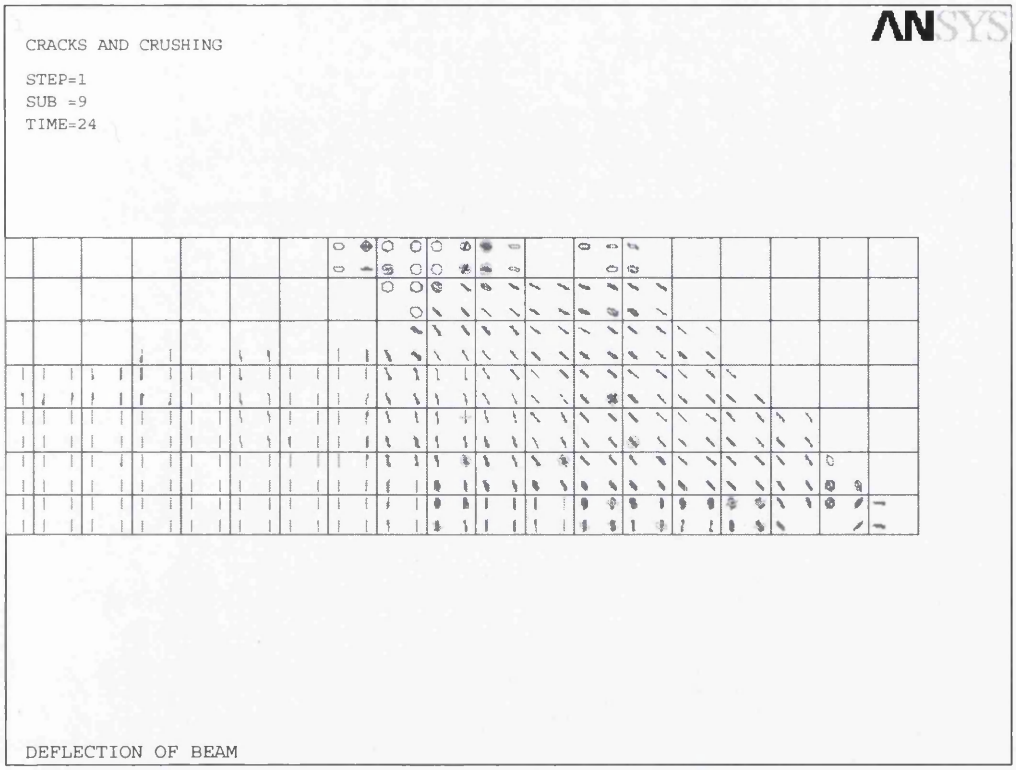

Fig. 26 Crack pattern for half portion of Beam type 3

Fig .26 shows crack pattern for Beam type 3 with lower a/d ratio of 1.53, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm². Here the reinforced concrete beam subjected to flexure failure in between shear span portion. On the other side shear span subjected to flexure as well as shear failure, rare appearance of third crack and crushing of concrete at top, bottom(nearer to support), middle side of beam.

Fig. 27 Shear stress contour for Beam type 4

Fig. 27 shows variation of shear stress contour for Beam type 4 with lower a/d ratio of 1.14, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm² and observed shear stress were to be 6.7 N/mm².

From Fig .15 for less a/d = 1.14, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 1.83 N/mm² which is less (2.32 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (2.67 N/mm²) as when compared to f'_c = 40.20 N/mm².

For increased a/d = 2, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 1.68 N/mm² which is less (2.25 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (2.74 N/mm²) as when compared to f'_c = 40.20 N/mm².

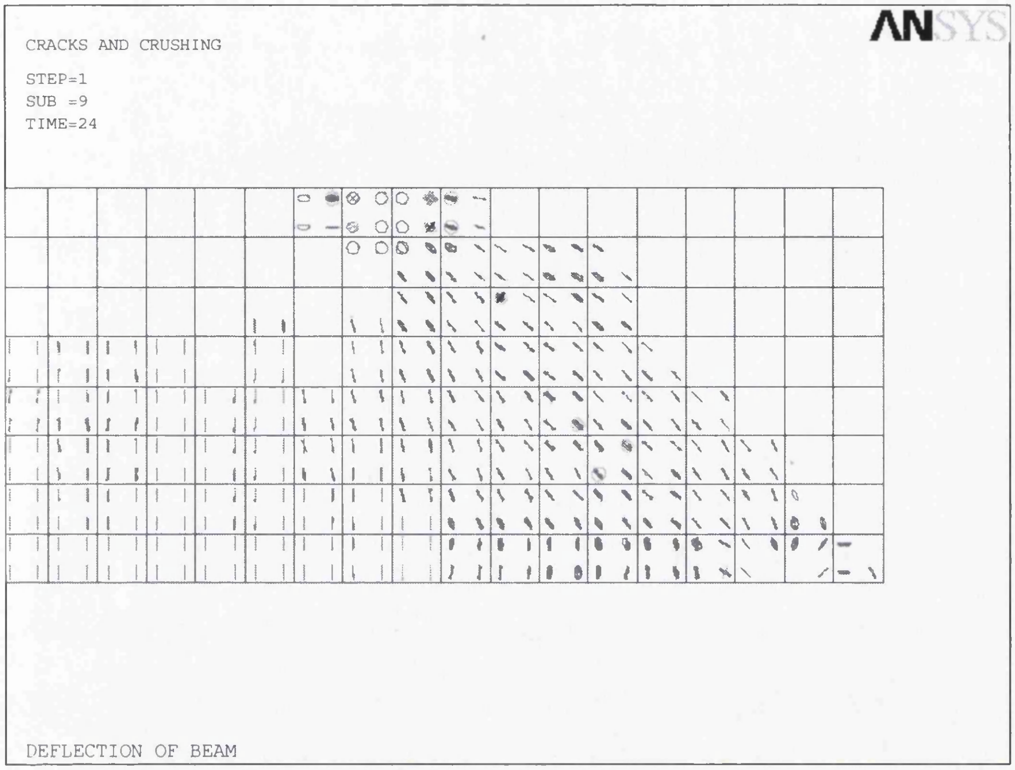

Fig. 28 Crack pattern for half portion of Beam type 4

Fig .28 shows crack pattern for Beam type 4 with lower a/d ratio of 1.14, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm². In this case reinforced concrete beam subjected to flexure in between shear span. On the other side shear span subjected to shear as flexure failure, rare appearance of concrete crushing and third crack at bottom (nearer to support), top, and middle side of beam.

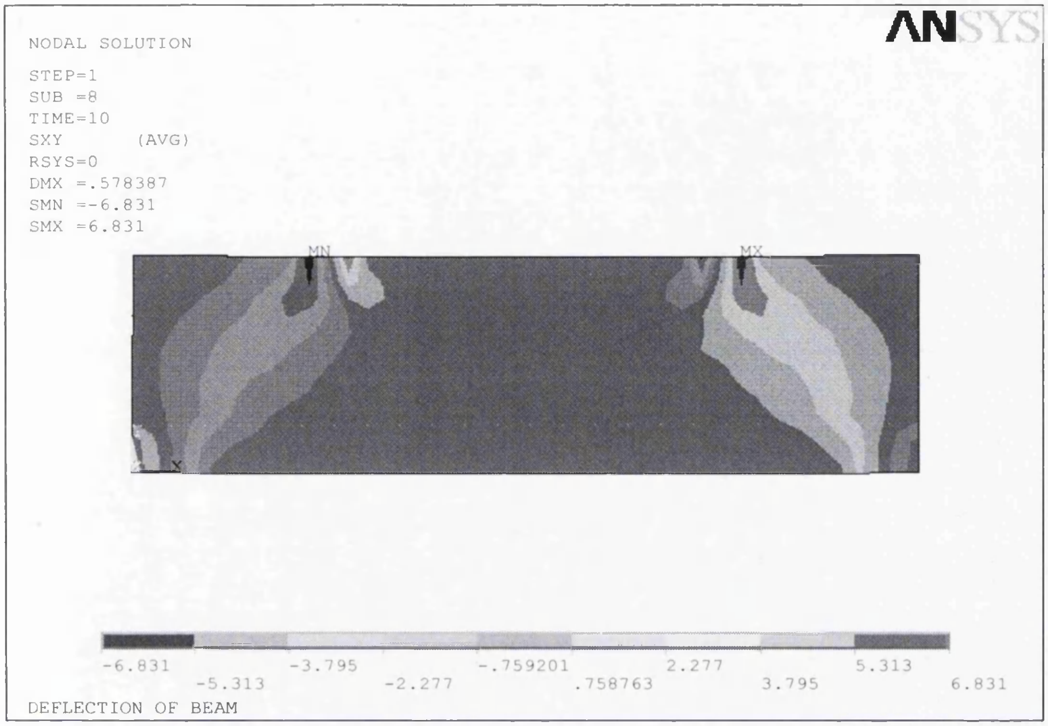

Fig. 29 Shear stress contour for Beam type 5

Fig .29 shows variation of shear stress contour for Beam type 5 with lower a/d ratio of 0.75, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm² and observed shear stress were to be 6.83 N/mm².

From Fig .18 for less a/d = 0.75, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 1.34 N/mm² which is less (1.6 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (1.98 N/mm²) as when compared to f'_c = 40.20 N/mm².

For increased a/d = 1.32, p = 0.031, and f'_c = 26.80 N/mm², the observed nominal shear stress were to be 1.6 N/mm² which is less (1.94 N/mm²) as when compared to f'_c = 33.50 N/mm², and also less (1.88 N/mm²) as when compared to f'_c = 40.20 N/mm².

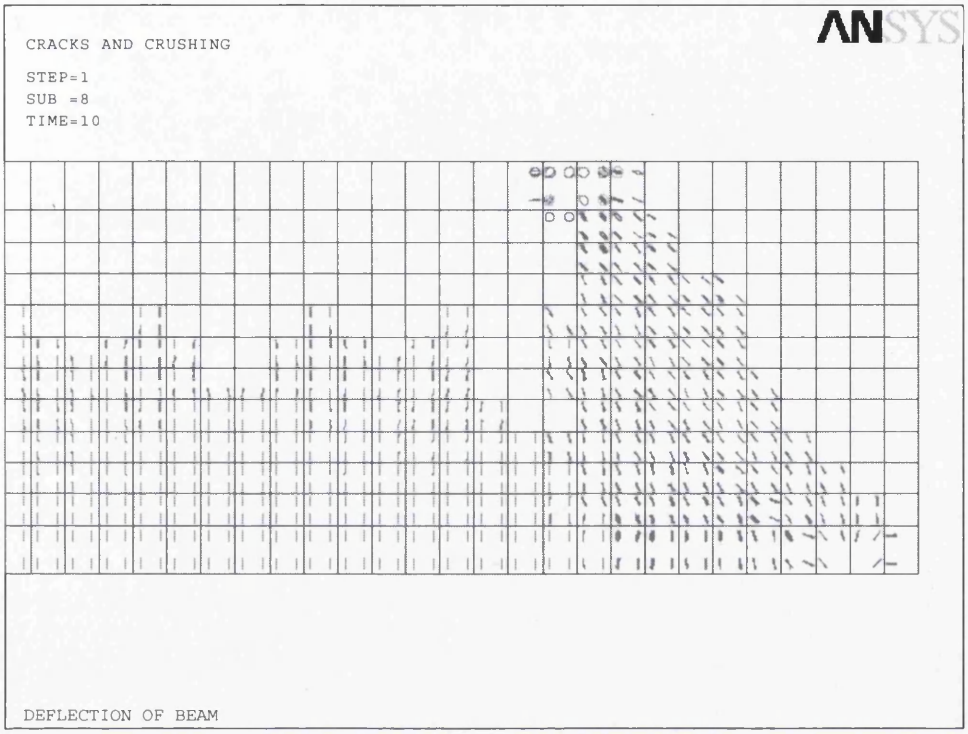

Fig. 30 Crack pattern for half portion of Beam type 5

Fig .30 shows crack pattern for Beam type 5 with lower a/d ratio of 0.75, tensile reinforcement ratio of 0.031, and concrete compressive strength of 26.80 N/mm². In this case reinforced concrete beam subjected to flexure failure (three fourth portions) of span. On the other side shear span subjected to shear (one third portions) as well as flexure failure, rare appearance of third crack and crushing of concrete at top side of beam.

CHAPTER 6

DISCUSSION ANDCONCLUSIONS

The following observations can be made regarding the effects of the prominent variables on the shear strength of reinforced concrete beam with uniform distribution of shear reinforcement for entire span length in all types of beam by using nonlinear finite element analysis:

1. The shear strength of reinforced concrete beam in case of (Beam type 1) increased with the increase of concrete compressive strength of about (26.80 N/mm²) for lesser shear span-effective depth ratio of 2.4 with different tensile reinforcement ratios of (0.031, 0.096, and 0.13). The corresponding increment in shear strength and displacement were to be in the order of 42.85%, 23.53%, 22.22% and 12.72% (decrease), 25.56% (increase), 30.26 %(decrease) as when compared with large shear span- effective depth ratio of (4.1) with the same respective tensile reinforcement ratio.

   The shear strength of reinforced concrete beam in case of (Beam type 2) increased with the increase of concrete compressive strength of about (26.80 N/mm²) for lesser shear span-effective depth ratio of 1.98 with different tensile reinforcement ratios of (0.031, 0.096, and 0.13). The corresponding increment in shear strength and decrement in displacement were to be in the order of 38.09%, 44.00%, 25.93% and 19.86%, 12.75%, 27.38% as when compared with large shear span- effective depth ratio of (3.5) with the same respective tensile reinforcement ratio.

   The shear strength of reinforced concrete beam in case of (Beam type 3) increased with the increase of concrete compressive strength of about (26.80 N/mm²) for lesser shear span-effective depth ratio of 1.53 with different tensile reinforcement ratios of (0.031, 0.096, and 0.13). The corresponding increment in shear strength and decrement in displacement were to be in the order of 0.00%, 9.10%, 0.00% and 79.95%, 51.04%, 70.51% as when compared with large shear span- effective depth ratio of (2.7) with the same respective tensile reinforcement ratio.

   The shear strength of reinforced concrete beam in case of (Beam type 4) increased with the increase of concrete compressive strength of about (26.80 N/mm²) for lesser shear span-effective depth ratio of 1.14 with different tensile reinforcement ratios of (0.031, 0.096, and 0.13). The corresponding increment in shear strength and displacement were to be in the order of 9.00%, 15.90%, 13.63% and 62.85% (decrease), 45.31% (decrease), 50% (increase) as when compared with large shear span to effective depth ratio of (2) with the same respective tensile reinforcement ratio.

The shear strength of reinforced concrete beam in case of (Beam type 5) decreased with the increase of concrete compressive strength of about (26.80 N/mm^2) for lesser shear span-effective depth ratio of 0.75 with different tensile reinforcement ratios of (0.031, 0.096, and 0.13). The corresponding decrement in shear strength and displacement were to be in the order of 19.71%, 2.74%, 0.00% and 138%, 84.37%, 70.96% as when compared with large shear span- effective depth ratio of (1.32) with the same respective tensile reinforcement ratio.

2. The shear strength of reinforced concrete beam increased with the increase of concrete compressive strength of about (26.80 N/mm^2) and beam size as in the case of (Beam type 2) for less shear span to effective depth ratio of 1.98 with different tensile reinforcement ratios of (0.031, 0.096, 0.13). The corresponding increment in shear strength and displacement were to be in the order of 45.00%, 71.42%, 54.54% and 13.80% (decrease), 12.75% (decrease), 3.28 %(decrease) as when compared to (Beam type 1 with small shear span- effective depth ratio of 2.4) with the same respective tensile reinforcement ratio.

   The shear strength of reinforced concrete beam increased with the increase of concrete compressive strength of about (26.80 N/mm^2) and beam size as in the case of (Beam type 3) for less shear span to effective depth ratio of 1.53 with different tensile reinforcement ratios of (0.031, 0.096, 0.13). The corresponding increment in shear strength and displacement were to be in the order of 13.79%, 0.0%, 2.94% and 82.20% (decrease), 104.16% (decrease), 101.28 %(decrease) as when compared to (Beam type 2 with small shear span- effective depth ratio of 1.98) with the same respective tensile reinforcement ratio.

   The shear strength of reinforced concrete beam increased with the increase of concrete compressive strength of about (26.80 N/mm^2) and beam size as in the case of (Beam type 4) for less shear span to effective depth ratio of 1.14 with different tensile reinforcement ratios of (0.031, 0.096, 0.13). The corresponding increment in shear strength and displacement were to be in the order of 45.45%, 41.67%, 42.85% and 55.23% (decrease), 50.0% (decrease), 44.44

%(decrease) as when compared to (Beam type 3 with small shear span- effective depth ratio of 1.53) with the same respective tensile reinforcement ratio.

The shear strength of reinforced concrete beam increased with the increase of concrete compressive strength of about (26.80 N/mm²) and beam size as in the case of (Beam type 5) for less shear span to effective depth ratio of 0.75 with different tensile reinforcement ratios of (0.031, 0.096, 0.13). The corresponding increment in shear strength and displacement were to be in the order of 47.91%, 43.13%, 50.0% and 123.40% (decrease), 100% (decrease), 74.19 %(decrease) as when compared to (Beam type 4 with small shear span- effective depth ratio of 1.14) with the same respective tensile reinforcement ratio.

3. The shear strength of a reinforced concrete beam increases with an increase in concrete strength, beam size, and tensile reinforcement for smaller shear span-effective depth ratio.

4. The shear stress of a reinforced concrete beam increases with an increase in tensile reinforcement and decreased with an increase in beam size.

5. The shear stress value of (2.11 N/mm²) obtained for Beam type 3 (f'_c of 26.80 N/mm², a/d ratio of 2.7, ρ ratio 0.031) by ansys result agrees very well with the experimental shear stress value of 2.58 N/mm² with (f'_c of 27.37 N/mm², a/d ratio of 2.57, ρ ratio 0.031).

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APPENDIX - A

Table 8 – Details of Beam type 1 with Ansys results (b = 75 mm, h = 100 mm, d = 85 mm)

a/d	Fyv, N/mm²	A_{sv}, mm²	Sv, mm	p	f'c, N/mm²	S_c, KN	C_s, N/mm²	T_s, N/mm²	S_s, N/mm²	v, N/mm²	S_c, cal KN
2.4	250	56.50	100	0.03	16.8	26	-18.92	2.95	5.60	2.03	23
2.9	250	56.50	100	0.03	16.8	22	-18.18	2.11	3.96	1.72	23
3.5	250	56.50	100	0.03	16.8	22	-21.16	2.14	4.17	1.72	23
4.1	250	56.50	100	0.03	16.8	18	-20.37	1.33	3.48	1.41	23
2.4	250	56.50	100	0.03	20.1	30	-21.60	3.54	6.26	2.35	24
2.9	250	56.50	100	0.03	20.1	26	-22.78	2.63	5.24	2.03	24
3.5	250	56.50	100	0.03	20.1	24	-24.12	2.33	5.26	1.88	24
4.1	250	56.50	100	0.03	20.1	24	-24.79	2.35	5.69	1.88	24
2.4	250	56.50	100	0.03	26.8	40	-30.95	1.62	8.77	3.13	25
2.9	250	56.50	100	0.03	26.8	34	-32.12	3.29	6.82	2.66	25
3.5	250	56.50	100	0.03	26.8	30	-32.87	2.30	6.97	2.35	25
4.1	250	56.50	100	0.03	26.8	28	-33.35	2.24	6.31	2.19	25
2.4	250	56.50	100	0.03	33.5	44	-36.75	1.57	9.38	3.45	26
2.9	250	56.50	100	0.03	33.5	40	-38.79	1.86	8.77	3.13	26
3.5	250	56.50	100	0.03	33.5	34	-39.20	2.95	7.50	2.66	26
4.1	250	56.50	100	0.03	33.5	34	-40.67	2.96	7.99	2.66	26
2.4	250	56.50	100	0.03	40.2	50	-43.06	1.74	10.76	3.92	27
2.9	250	56.50	100	0.03	40.2	44	-45.17	1.94	9.44	3.45	27
3.5	250	56.50	100	0.03	40.2	38	-46.47	3.14	8.52	2.98	27
4.1	250	56.50	100	0.03	40.2	36	-47.26	2.91	7.69	2.82	27
2.4	250	56.50	100	0.09	16.8	30	-15.98	2.12	5.84	2.35	28
2.9	250	56.50	100	0.09	16.8	28	-16.41	1.79	5.16	2.19	28
3.5	250	56.50	100	0.09	16.8	26	-16.30	1.82	4.94	2.03	28
4.1	250	56.50	100	0.09	16.8	22	-15.61	2.89	4.20	1.72	28
2.4	250	56.50	100	0.09	20.1	34	-17.57	1.46	8.68	2.66	29
2.9	250	56.50	100	0.09	20.1	32	-18.31	2.09	6.09	2.51	29
3.5	250	56.50	100	0.09	20.1	32	-20.17	2.10	6.48	2.51	29
4.1	250	56.50	100	0.09	20.1	26	-20.69	1.81	4.85	2.03	29
2.4	250	56.50	100	0.09	26.8	42	-24.10	2.04	8.02	3.29	31
2.9	250	56.50	100	0.09	26.8	38	-24.21	1.48	7.18	2.98	31
3.5	250	56.50	100	0.09	26.8	36	-26.63	1.79	7.59	2.82	31
4.1	250	56.50	100	0.09	26.8	34	-29.47	1.93	6.15	2.66	31
2.4	250	56.50	100	0.09	33.5	52	-31.07	1.51	11.51	4.07	32
2.9	250	56.50	100	0.09	33.5	48	-32.48	2.57	9.26	3.76	32
3.5	250	56.50	100	0.09	33.5	46	-35.00	1.60	9.54	3.60	32
4.1	250	56.50	100	0.09	33.5	42	-35.70	2.59	8.59	3.29	32
2.4	250	56.50	100	0.09	40.2	56	-34.28	1.94	11.18	4.39	33
2.9	250	56.50	100	0.09	40.2	54	-36.99	1.79	10.14	4.23	33
3.5	250	56.50	100	0.09	40.2	48	-37.87	2.52	10.01	3.76	33
4.1	250	56.50	100	0.09	40.2	46	-42.01	0.88	9.52	3.60	33
2.4	250	56.50	100	0.13	16.8	28	-14.36	2.22	5.54	2.19	29
2.9	250	56.50	100	0.13	16.8	28	-15.29	1.89	5.42	2.19	29
3.5	250	56.50	100	0.13	16.8	28	-16.89	1.90	5.81	2.19	29
4.1	250	56.50	100	0.13	16.8	26	-17.14	1.79	4.61	2.03	29
2.4	250	56.50	100	0.13	20.1	34	-18.27	2.00	7.37	2.66	30
2.9	250	56.50	100	0.13	20.1	32	-17.21	1.77	6.37	2.51	30
3.5	250	56.50	100	0.13	20.1	32	-18.28	1.02	6.52	2.51	30
4.1	250	56.50	100	0.13	20.1	28	-17.98	1.74	5.00	2.19	30
2.4	250	56.50	100	0.13	26.8	44	-23.52	2.41	9.53	3.45	32
2.9	250	56.50	100	0.13	26.8	40	-23.55	1.83	7.31	3.13	32
3.5	250	56.50	100	0.13	26.8	38	-26.60	1.06	7.93	2.98	32
4.1	250	56.50	100	0.13	26.8	36	-27.66	2.19	7.40	2.82	32
2.4	250	56.50	100	0.13	33.5	56	-31.21	1.57	10.9	4.39	34
2.9	250	56.50	100	0.13	33.5	52	-31.93	1.83	9.68	4.07	34
3.5	250	56.50	100	0.13	33.5	48	-31.66	2.68	9.74	3.76	34
4.1	250	56.50	100	0.13	33.5	42	-31.51	2.33	8.14	3.29	34
2.4	250	56.50	100	0.13	40.2	56	-32.64	1.74	11.25	4.39	35
2.9	250	56.50	100	0.13	40.2	54	-35.08	1.75	10.45	4.23	35
3.5	250	56.50	100	0.13	40.2	52	-36.60	1.53	10.31	4.07	35
4.1	250	56.50	100	0.13	40.2	48	-39.35	2.60	9.36	3.76	35

Table 9 – Details of Beam type 2 with Ansys results (b = 100 mm, h = 125 mm, d = 101 mm)

a/d	Fyv, N/mm²	A_{sv}, mm²	S_v, mm	p	f'$_c$, N/mm²	S_c, KN	C_s, N/mm²	T_s, N/mm²	S_s, N/mm²	v, N/mm²	S_c, cal KN
2	250	56.50	100	0.03	16.8	38	-18.78	1.87	4.91	1.88	28
2.5	250	56.50	100	0.03	16.8	30	-18.83	1.25	3.72	1.48	28
3	250	56.50	100	0.03	16.8	26	-18.94	1.10	3.47	1.28	28
3.5	250	56.50	100	0.03	16.8	32	-22.30	1.30	4.48	1.58	28
2	250	56.50	100	0.03	20.1	44	-22.40	2.11	5.88	2.17	29
2.5	250	56.50	100	0.03	20.1	42	-24.35	1.98	5.79	2.07	29
3	250	56.50	100	0.03	20.1	36	-24.25	1.52	4.97	1.78	29
3.5	250	56.50	100	0.03	20.1	34	-25.25	1.40	4.67	1.68	29
2	250	56.50	100	0.03	26.8	58	-31.05	2.34	7.98	2.87	31
2.5	250	56.50	100	0.03	26.8	52	-32.16	1.94	6.93	2.57	31
3	250	56.50	100	0.03	26.8	42	-31.76	1.59	5.43	2.07	31
3.5	250	56.50	100	0.03	26.8	42	-33.93	1.69	6.15	2.07	31
2	250	56.50	100	0.03	33.5	74	-39.93	0.91	9.32	3.66	32
2.5	250	56.50	100	0.03	33.5	62	-40.48	2.66	8.03	3.06	32
3	250	56.50	100	0.03	33.5	52	-39.54	1.92	7.06	2.57	32
3.5	250	56.50	100	0.03	33.5	50	-41.48	1.82	6.92	2.47	32
2	250	56.50	100	0.03	40.2	86	-47.96	3.19	11.49	4.25	33
2.5	250	56.50	100	0.03	40.2	76	-48.79	2.98	9.95	3.76	33
3	250	56.50	100	0.03	40.2	62	-48.38	2.27	8.91	3.06	33
3.5	250	56.50	100	0.03	40.2	56	-48.70	1.96	8.19	2.77	33
2	250	56.50	100	0.09	16.8	46	-16.51	1.85	5.55	2.27	35
2.5	250	56.50	100	0.09	16.8	44	-18.50	3.63	5.27	2.17	35
3	250	56.50	100	0.09	16.8	36	-17.68	2.35	4.66	1.78	35
3.5	250	56.50	100	0.09	16.8	36	-19.88	2.45	5.02	1.78	35
2	250	56.50	100	0.09	20.1	54	-19.26	1.85	6.12	2.67	36
2.5	250	56.50	100	0.09	20.1	54	-21.99	2.45	6.20	2.67	36
3	250	56.50	100	0.09	20.1	44	-21.00	3.04	5.86	2.17	36
3.5	250	56.50	100	0.09	20.1	44	-23.75	3.02	5.25	2.17	36
2	250	56.50	100	0.09	26.8	72	-27.96	1.61	8.87	3.56	38
2.5	250	56.50	100	0.09	26.8	68	-31.54	2.32	8.32	3.36	38
3	250	56.50	100	0.09	26.8	54	-28.84	3.44	7.02	2.67	38
3.5	250	56.50	100	0.09	26.8	50	-30.46	2.97	6.03	2.47	38
2	250	56.50	100	0.09	33.5	78	-32.77	1.06	9.51	3.86	40
2.5	250	56.50	100	0.09	33.5	70	-34.58	0.97	7.93	3.46	40
3	250	56.50	100	0.09	33.5	62	-34.25	1.91	8.31	3.06	40
3.5	250	56.50	100	0.09	33.5	54	-34.05	3.40	6.69	2.67	40
2	250	56.50	100	0.09	40.2	96	-40.29	1.36	11.56	4.75	42
2.5	250	56.50	100	0.09	40.2	82	-40.22	1.45	9.84	4.05	42
3	250	56.50	100	0.09	40.2	74	-41.90	1.44	9.73	3.66	42
3.5	250	56.50	100	0.09	40.2	66	-41.79	1.85	8.41	3.26	42
2	250	56.50	100	0.13	16.8	46	-16.01	1.95	5.65	2.27	37
2.5	250	56.50	100	0.13	16.8	46	-18.47	1.87	5.44	2.27	37
3	250	56.50	100	0.13	16.8	38	-17.45	3.18	4.76	1.88	37
3.5	250	56.50	100	0.13	16.8	36	-18.76	2.55	4.53	1.78	37
2	250	56.50	100	0.13	20.1	56	-19.85	2.16	6.86	2.77	38
2.5	250	56.50	100	0.13	20.1	54	-20.20	2.37	6.46	2.67	38
3	250	56.50	100	0.13	20.1	52	-22.42	2.05	6.57	2.57	38
3.5	250	56.50	100	0.13	20.1	50	-24.05	1.92	6.25	2.47	38
2	250	56.50	100	0.13	26.8	68	-26.08	1.61	8.28	3.36	41
2.5	250	56.50	100	0.13	26.8	60	-26.19	1.64	7.36	2.97	41
3	250	56.50	100	0.13	26.8	60	-26.48	1.68	7.58	2.97	41
3.5	250	56.50	100	0.13	26.8	54	-29.27	3.66	6.83	2.67	41
2	250	56.50	100	0.13	33.5	84	-32.43	2.40	10.30	4.15	43
2.5	250	56.50	100	0.13	33.5	76	-34.32	1.23	9.38	3.76	43
3	250	56.50	100	0.13	33.5	64	-33.46	1.95	7.78	3.16	43
3.5	250	56.50	100	0.13	33.5	60	-34.27	2.16	7.23	2.97	43
2	250	56.50	100	0.13	40.2	96	-37.92	1.01	11.51	4.75	44
2.5	250	56.50	100	0.13	40.2	92	-41.98	2.51	10.91	4.55	44
3	250	56.50	100	0.13	40.2	72	-37.40	0.99	9.02	3.56	44
3.5	250	56.50	100	0.13	40.2	70	-40.63	2.08	8.83	3.46	44

Table 10 - Details of Beam type 3 with Ansys results (b = 120 mm, h = 150 mm, d = 130 mm)

a/d	Fyv, N/mm²	A_{sv}, mm²	Sv, mm	p	f'$_c$, N/mm²	S_c, KN	C_s, N/mm²	T_s, N/mm²	S_s, N/mm²	v, N/mm²	S_c, cal KN
1.5	250	56.50	100	0.03	16.8	42	-14.28	2.32	3.73	1.34	36
1.9	250	56.50	100	0.03	16.8	44	-16.96	2.11	3.99	1.41	36
2.3	250	56.50	100	0.03	16.8	44	-18.36	1.99	3.77	1.41	36
2.7	250	56.50	100	0.03	16.8	46	-20.68	1.85	4.08	1.47	36
1.5	250	56.50	100	0.03	20.1	58	-18.86	2.04	5.30	1.85	37
1.9	250	56.50	100	0.03	20.1	58	-22.26	2.17	5.35	1.85	37
2.3	250	56.50	100	0.03	20.1	54	-21.70	1.71	5.23	1.73	37
2.7	250	56.50	100	0.03	20.1	54	-24.62	2.00	4.84	1.73	37
1.5	250	56.50	100	0.03	26.8	66	-22.89	2.21	6.21	2.11	39
1.9	250	56.50	100	0.03	26.8	66	-27.10	1.42	6.24	2.11	39
2.3	250	56.50	100	0.03	26.8	66	-30.31	1.40	6.66	2.11	39
2.7	250	56.50	100	0.03	26.8	66	-33.19	1.41	6.62	2.11	39
1.5	250	56.50	100	0.03	33.5	90	-31.05	2.57	8.41	2.88	41
1.9	250	56.50	100	0.03	33.5	90	-36.90	2.32	8.32	2.88	41
2.3	250	56.50	100	0.03	33.5	84	-39.60	2.20	8.39	2.69	41
2.7	250	56.50	100	0.03	33.5	78	-40.32	2.12	7.67	2.50	41
1.5	250	56.50	100	0.03	40.2	96	-33.76	2.57	9.05	3.07	42
1.9	250	56.50	100	0.03	40.2	106	-44.29	2.88	9.89	3.39	42
2.3	250	56.50	100	0.03	40.2	96	-47.16	2.64	9.54	3.07	42
2.7	250	56.50	100	0.03	40.2	92	-47.84	2.46	8.11	2.94	42
1.5	250	56.50	100	0.09	16.8	44	-11.90	1.97	3.69	1.41	44
1.9	250	56.50	100	0.09	16.8	42	-12.52	1.56	3.59	1.34	44
2.3	250	56.50	100	0.09	16.8	42	-13.75	1.27	3.48	1.34	44
2.7	250	56.50	100	0.09	16.8	44	-14.93	1.41	3.93	1.41	44
1.5	250	56.50	100	0.09	20.1	50	-14.06	1.82	3.98	1.60	46
1.9	250	56.50	100	0.09	20.1	52	-15.83	1.70	4.42	1.66	46
2.3	250	56.50	100	0.09	20.1	58	-19.04	2.25	5.77	1.85	46
2.7	250	56.50	100	0.09	20.1	58	-18.46	2.21	5.40	1.85	46
1.5	250	56.50	100	0.09	26.8	72	-19.88	2.52	6.57	2.30	48
1.9	250	56.50	100	0.09	26.8	70	-21.67	2.20	6.21	2.24	48
2.3	250	56.50	100	0.09	26.8	66	-23.60	2.22	6.36	2.11	48
2.7	250	56.50	100	0.09	26.8	66	-25.31	2.22	6.24	2.11	48
1.5	250	56.50	100	0.09	33.5	92	-26.62	1.48	8.19	2.94	51
1.9	250	56.50	100	0.09	33.5	90	-28.01	3.11	7.96	2.88	51
2.3	250	56.50	100	0.09	33.5	84	-29.48	2.80	8.01	2.69	51
2.7	250	56.50	100	0.09	33.5	84	-32.31	2.64	7.52	2.69	51
1.5	250	56.50	100	0.09	40.2	108	-29.49	1.63	9.62	3.46	53
1.9	250	56.50	100	0.09	40.2	102	-32.63	1.62	8.84	3.26	53
2.3	250	56.50	100	0.09	40.2	98	-35.24	3.26	8.91	3.14	53
2.7	250	56.50	100	0.09	40.2	94	-38.36	2.92	9.13	3.01	53
1.5	250	56.50	100	0.13	16.8	48	-12.35	1.67	3.89	1.53	46
1.9	250	56.50	100	0.13	16.8	44	-12.14	2.29	3.61	1.41	46
2.3	250	56.50	100	0.13	16.8	42	-13.23	1.35	3.57	1.34	46
2.7	250	56.50	100	0.13	16.8	44	-14.52	1.55	3.72	1.41	46
1.5	250	56.50	100	0.13	20.1	54	-14.74	2.44	4.51	1.73	48
1.9	250	56.50	100	0.13	20.1	54	-15.35	1.83	4.61	173	48
2.3	250	56.50	100	0.13	20.1	60	-17.29	2.74	5.56	1.92	48
2.7	250	56.50	100	0.13	20.1	60	-19.06	2.64	5.55	1.92	48
1.5	250	56.50	100	0.13	26.8	70	-18.56	2.57	6.24	2.24	51
1.9	250	56.50	100	0.13	26.8	70	-20.49	2.59	5.91	2.24	51
2.3	250	56.50	100	0.13	26.8	66	-21.43	2.63	5.96	2.11	51
2.7	250	56.50	100	0.13	26.8	70	-22.57	2.51	6.17	2.24	51
1.5	250	56.50	100	0.13	33.5	94	-25.50	2.36	8.31	3.01	54
1.9	250	56.50	100	0.13	33.5	90	-26.03	3.37	7.74	2.88	54
2.3	250	56.50	100	0.13	33.5	88	-27.60	3.20	7.91	2.82	54
2.7	250	56.50	100	0.13	33.5	84	-29.01	3.11	7.49	2.69	54
1.5	250	56.50	100	0.13	40.2	110	-30.99	1.68	9.80	3.52	56
1.9	250	56.50	100	0.13	40.2	104	-32.12	1.56	8.95	3.33	56
2.3	250	56.50	100	0.13	40.2	98	-32.70	3.31	9.45	3.14	56
2.7	250	56.50	100	0.13	40.2	96	-35.18	3.19	8.46	3.07	56

Table 11 - Details of Beam type 4 with Ansys results (b = 150 mm, h = 200 mm, d = 175 mm)

a/d	Fyv, N/mm²	A_{sv}, mm²	Sv, mm	p	f'c, N/mm²	S_c, KN	C_s, N/mm²	T_s, N/mm²	S_s, N/mm²	v, N/mm²	S_c, cal KN
1.1	250	56.50	100	0.03	16.8	56	-11.02	1.98	3.85	1.06	48
1.4	250	56.50	100	0.03	16.8	58	-13.46	2.10	3.81	1.10	48
1.7	250	56.50	100	0.03	16.8	64	-16.77	1.95	4.38	1.21	48
2.0	250	56.50	100	0.03	16.8	64	-19.14	1.93	3.70	1.21	48
1.1	250	56.50	100	0.03	20.1	66	-13.85	2.04	3.60	1.25	49
1.4	250	56.50	100	0.03	20.1	82	-18.28	1.86	5.34	1.56	49
1.7	250	56.50	100	0.03	20.1	78	-20.13	1.83	5.11	1.48	49
2.0	250	56.50	100	0.03	20.1	80	-22.63	2.14	5.58	1.52	49
1.1	250	56.50	100	0.03	26.8	96	-19.97	2.47	6.71	1.82	52
1.4	250	56.50	100	0.03	26.8	88	-21.92	2.02	4.85	1.67	52
1.7	250	56.50	100	0.03	26.8	88	-24.59	0.91	6.10	1.67	52
2.0	250	56.50	100	0.03	26.8	88	-26.46	1.77	6.28	167	52
1.1	250	56.50	100	0.03	33.5	122	-24.59	2.58	8.45	2.32	54
1.4	250	56.50	100	0.03	33.5	122	-28.35	1.62	8.18	2.32	54
1.7	250	56.50	100	0.03	33.5	112	-31.31	1.33	7.78	2.13	54
2.0	250	56.50	100	0.03	33.5	118	-35.99	1.42	8.29	2.24	54
1.1	250	56.50	100	0.03	40.2	140	-28.48	2.22	10.04	2.66	56
1.4	250	56.50	100	0.03	40.2	138	-32.68	1.67	9.60	2.62	56
1.7	250	56.50	100	0.03	40.2	130	-36.40	1.53	9.06	2.47	56
2.0	250	56.50	100	0.03	40.2	144	-45.93	1.86	9.23	2.74	56
1.1	250	56.50	100	0.09	16.8	56	-9.86	2.08	3.15	1.06	58
1.4	250	56.50	100	0.09	16.8	62	-11.01	2.32	4.02	1.18	58
1.7	250	56.50	100	0.09	16.8	62	-12.37	2.07	3.92	1.18	58
2.0	250	56.50	100	0.09	16.8	68	-14.53	1.66	4.40	1.29	58
1.1	250	56.50	100	0.09	20.1	66	-12.54	2.02	4.10	1.25	60
1.4	250	56.50	100	0.09	20.1	68	-12.89	1.88	4.48	1.29	60
1.7	250	56.50	100	0.09	20.1	68	-13.99	1.57	4.52	1.29	60
2.0	250	56.50	100	0.09	20.1	82	-17.32	1.50	5.19	1.56	60
1.1	250	56.50	100	0.09	26.8	102	-19.16	1.83	6.94	1.94	64
1.4	250	56.50	100	0.09	26.8	96	-18.56	1.36	6.22	1.82	64
1.7	250	56.50	100	0.09	26.8	86	-19.02	1.45	5.58	1.63	64
2.0	250	56.50	100	0.09	26.8	88	-20.97	2.00	4.81	1.67	64
1.1	250	56.50	100	0.09	33.5	120	-21.51	2.26	8.22	2.28	67
1.4	250	56.50	100	0.09	33.5	122	-23.37	1.82	8.04	2.32	67
1.7	250	56.50	100	0.09	33.5	122	-26.04	1.58	7.82	2.32	67
2.0	250	56.50	100	0.09	33.5	106	-26.95	1.42	6.16	2.01	67
1.1	250	56.50	100	0.09	40.2	142	-25.75	2.71	9.99	2.70	70
1.4	250	56.50	100	0.09	40.2	144	-28.82	2.55	9.72	274	70
1.7	250	56.50	100	0.09	40.2	136	-29.85	2.11	9.26	2.59	70
2.0	250	56.50	100	0.09	40.2	136	-32.62	1.83	8.97	2.59	70
1.1	250	56.50	100	0.13	16.8	58	-8.97	2.03	3.65	1.10	62
1.4	250	56.50	100	0.13	16.8	60	-10.86	1.61	4.04	1.14	62
1.7	250	56.50	100	0.13	16.8	62	-11.57	2.35	3.98	1.18	62
2.0	250	56.50	100	0.13	16.8	66	-13.64	2.36	4.10	1.25	62
1.1	250	56.50	100	0.13	20.1	68	-11.35	1.82	4.31	1.29	64
1.4	250	56.50	100	0.13	20.1	68	-12.31	2.51	4.38	1.29	64
1.7	250	56.50	100	0.13	20.1	70	-13.63	2.01	4.49	1.33	64
2.0	250	56.50	100	0.13	20.1	84	-17.08	1.56	5.17	1.60	64
1.1	250	56.50	100	0.13	26.8	100	-18.38	1.70	6.65	1.90	68
1.4	250	56.50	100	0.13	26.8	100	-19.22	2.54	6.71	1.90	68
1.7	250	56.50	100	0.13	26.8	98	-20.17	1.80	6.49	1.86	68
2.0	250	56.50	100	0.13	26.8	88	-20.76	1.67	5.20	167	68
1.1	250	56.50	100	0.13	33.5	116	-20.01	1.87	7.73	2.21	71
1.4	250	56.50	100	0.13	33.5	116	-21.41	2.35	7.55	2.21	71
1.7	250	56.50	100	0.13	33.5	116	-23.59	1.72	7.80	2.21	71
2.0	250	56.50	100	0.13	33.5	116	-26.15	1.72	7.57	2.21	71
1.1	250	56.50	100	0.13	40.2	146	-26.33	2.96	10.13	2.78	74
1.4	250	56.50	100	0.13	40.2	142	-26.91	2.76	9.83	2.70	74
1.7	250	56.50	100	0.13	40.2	140	-28.71	2.42	9.55	2.66	74
2.0	250	56.50	100	0.13	40.2	134	-30.29	1.96	9.10	2.55	74

Table 12 - Details of Beam type 5 with Ansys results (b = 200 mm, h = 300 mm, d = 265 mm)

a/d	Fyv, N/mm²	A_{sv}, mm²	Sv, mm	p	f 'c, N/mm²	S_c, KN	C_s, N/mm²	T_s, N/mm²	S_s, N/mm²	v , N/mm²	S_c, cal KN
0.8	250	56.50	100	0.03	16.8	100	-8.46	2.02	4.79	0.94	71
0.9	250	56.50	100	0.03	16.8	100	-10.3	2.38	4.81	0.94	71
1.1	250	56.50	100	0.03	16.8	106	-12.85	1.42	4.97	1.00	71
1.3	250	56.50	100	0.03	16.8	108	-13.47	2.38	4.85	1.01	71
0.8	250	56.50	100	0.03	20.1	110	-10.72	2.29	5.13	1.03	73
0.9	250	56.50	100	0.03	20.1	108	-11.98	2.11	5.21	1.01	73
1.1	250	56.50	100	0.03	20.1	128	-15.79	1.08	6.08	1.20	73
1.3	250	56.50	100	0.03	20.1	130	-17.36	1.30	5.92	1.22	73
0.8	250	56.50	100	0.03	26.8	142	-13.85	1.93	6.83	1.34	77
0.9	250	56.50	100	0.03	26.8	148	-17.27	2.13	7.30	1.39	77
1.1	250	56.50	100	0.03	26.8	146	-18.93	1.46	7.31	1.37	77
1.3	250	56.50	100	0.03	26.8	170	-24.04	1.52	7.90	1.60	77
0.8	250	56.50	100	0.03	33.5	170	-16.40	2.44	8.23	1.60	80
0.9	250	56.50	100	0.03	33.5	174	-19.85	2.36	8.74	1.64	80
1.1	250	56.50	100	0.03	33.5	174	-22.15	1.36	9.26	1.64	80
1.3	250	56.50	100	0.03	33.5	206	-29.47	1.15	9.73	1.94	80
0.8	250	56.50	100	0.03	40.2	210	-20.69	1.84	10.35	1.98	82
0.9	250	56.50	100	0.03	40.2	200	-23.84	1.63	10.02	1.88	82
1.1	250	56.50	100	0.03	40.2	204	-27.46	1.68	10.71	1.92	82
1.3	250	56.50	100	0.03	40.2	200	-30.08	1.74	10.81	1.88	82
0.8	250	56.50	100	0.09	16.8	94	-8.46	2.40	4.54	0.88	86
0.9	250	56.50	100	0.09	16.8	98	-8.21	2.25	4.59	0.92	86
1.1	250	56.50	100	0.09	16.8	100	-9.63	2.15	4.70	0.94	86
1.3	250	56.50	100	0.09	16.8	100	-10.48	1.91	4.94	0.94	86
0.8	250	56.50	100	0.09	20.1	112	-10.81	2.37	5.13	1.05	89
0.9	250	56.50	100	0.09	20.1	114	-11.48	1.93	5.64	1.07	89
1.1	250	56.50	100	0.09	20.1	114	-12.40	2.72	5.25	1.07	89
1.3	250	56.50	100	0.09	20.1	128	-14.60	1.92	5.96	1.20	89
0.8	250	56.50	100	0.09	26.8	146	-14.12	2.15	6.82	1.37	94
0.9	250	56.50	100	0.09	26.8	148	-15.82	2.20	7.11	1.39	94
1.1	250	56.50	100	0.09	26.8	152	-17.94	1.76	7.37	1.43	94
1.3	250	56.50	100	0.09	26.8	150	-18.16	1.69	7.28	1.41	94
0.8	250	56.50	100	0.09	33.5	168	-18.11	2.21	6.71	1.58	99
0.9	250	56.50	100	0.09	33.5	174	-20.58	1.66	8.08	1.64	99
1.1	250	56.50	100	0.09	33.5	176	-20.07	1.39	8.70	1.66	99
1.3	250	56.50	100	0.09	33.5	178	-20.98	1.87	8.72	1.67	99
0.8	250	56.50	100	0.09	40.2	196	-18.94	2.13	9.16	1.84	103
0.9	250	56.50	100	0.09	40.2	202	-21.59	1.66	9.72	1.90	103
1.1	250	56.50	100	0.09	40.2	208	-24.49	1.64	10.22	1.96	103
1.3	250	56.50	100	0.09	40.2	206	-24.67	1.36	10.30	1.94	103
0.8	250	56.50	100	0.13	16.8	94	-8.27	3.17	4.09	0.88	90
0.9	250	56.50	100	0.13	16.8	98	-8.00	2.00	4.59	0.92	90
1.1	250	56.50	100	0.13	16.8	100	-9.11	1.80	4.65	0.94	90
1.3	250	56.50	100	0.13	16.8	102	-10.17	1.66	5.06	0.96	90
0.8	250	56.50	100	0.13	20.1	112	-10.41	2.25	4.95	1.05	94
0.9	250	56.50	100	0.13	20.1	114	-11.34	2.33	5.14	1.07	94
1.1	250	56.50	100	0.13	20.1	114	-12.15	1.82	5.17	1.07	94
1.3	250	56.50	100	0.13	20.1	116	-12.80	1.82	5.23	1.09	94
0.8	250	56.50	100	0.13	26.8	150	-15.08	2.38	7.43	1.41	99
0.9	250	56.50	100	0.13	26.8	154	-16.45	3.34	7.38	1.45	99
1.1	250	56.50	100	0.13	26.8	152	-16.85	1.94	7.23	1.43	99
1.3	250	56.50	100	0.13	26.8	150	-17.07	1.89	7.17	1.41	99
0.8	250	56.50	100	0.13	33.5	174	-15.87	2.23	7.99	1.64	104
0.9	250	56.50	100	0.13	33.5	170	-16.34	1.95	7.92	1.60	104
1.1	250	56.50	100	0.13	33.5	174	-19.14	2.25	8.50	1.64	104
1.3	250	56.50	100	0.13	33.5	180	-20.58	2.19	8.74	1.69	104
0.8	250	56.50	100	0.13	40.2	198	-18.65	1.86	9.20	1.86	108
0.9	250	56.50	100	0.13	40.2	206	-20.11	2.34	9.88	1.94	108
1.1	250	56.50	100	0.13	40.2	204	-23.30	1.92	9.97	1.92	108
1.3	250	56.50	100	0.13	40.2	210	-26.09	1.29	10.49	1.98	108

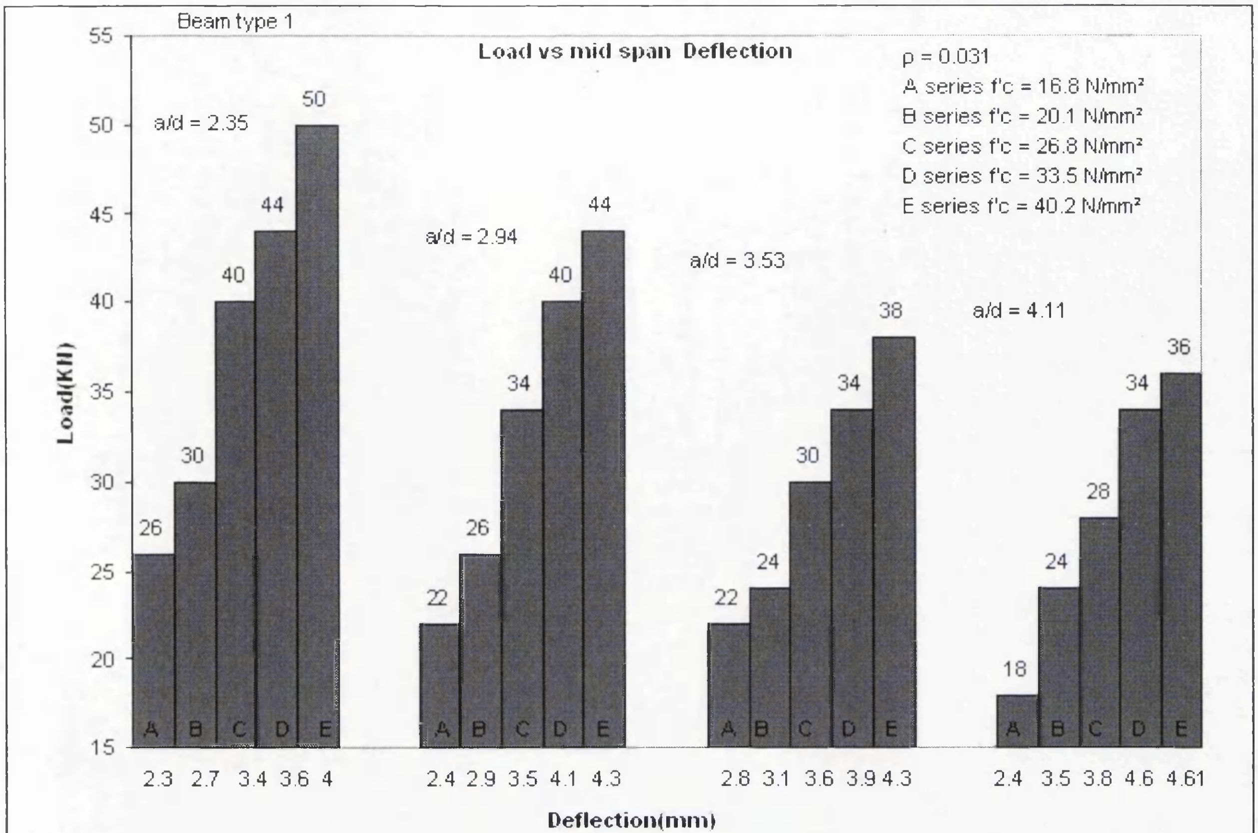

Fig. 31 Load -mid span deflection

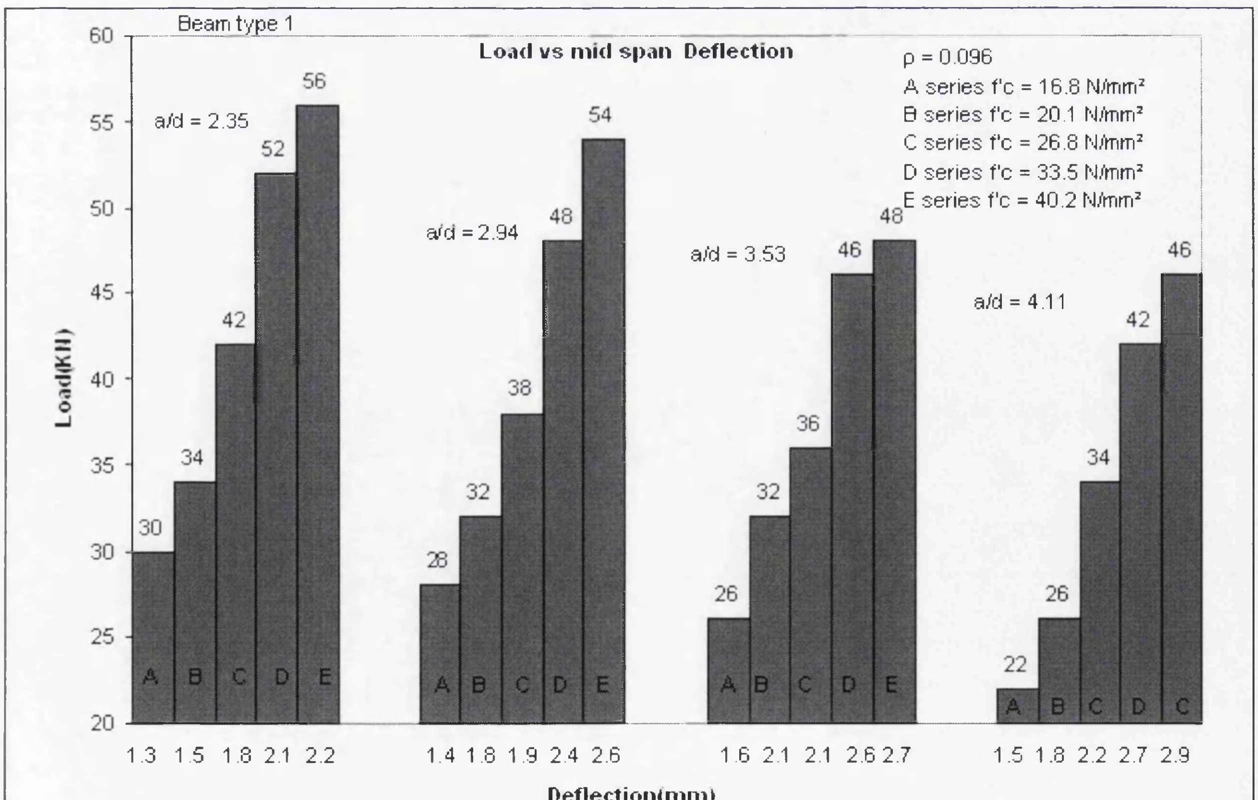

Fig. 32 Load -mid span deflection

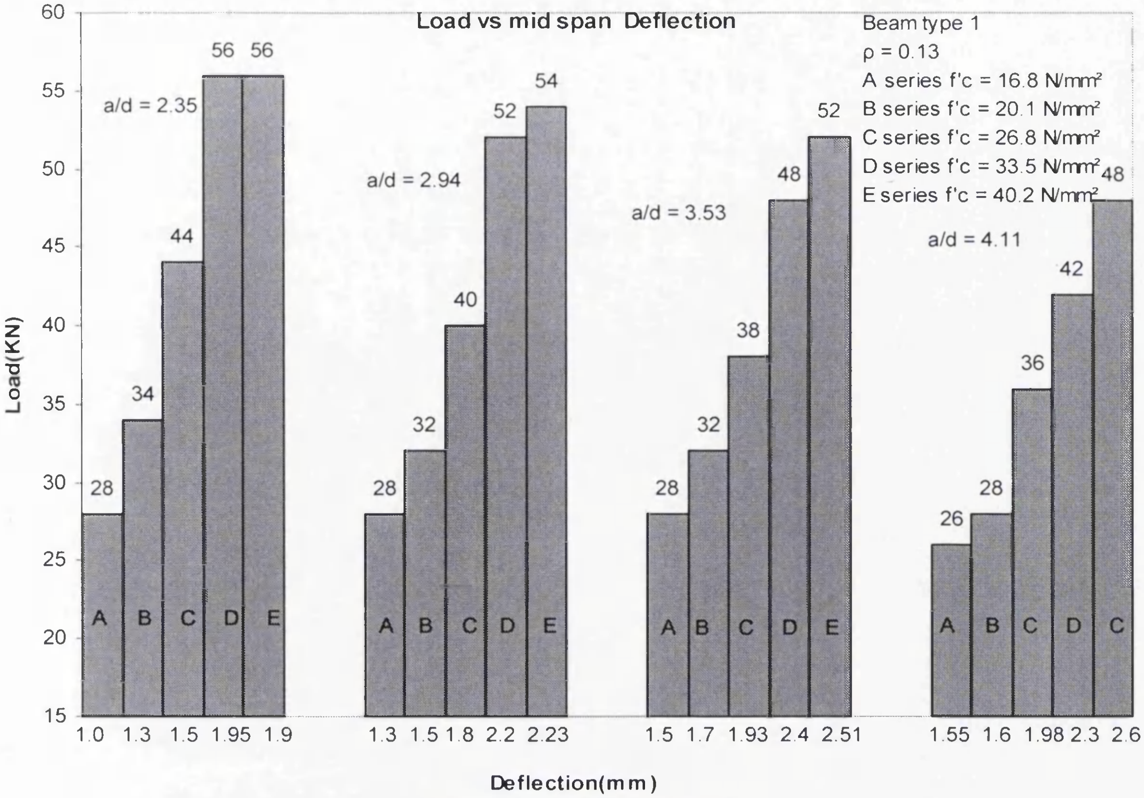

Fig. 33 Load-mid span deflection

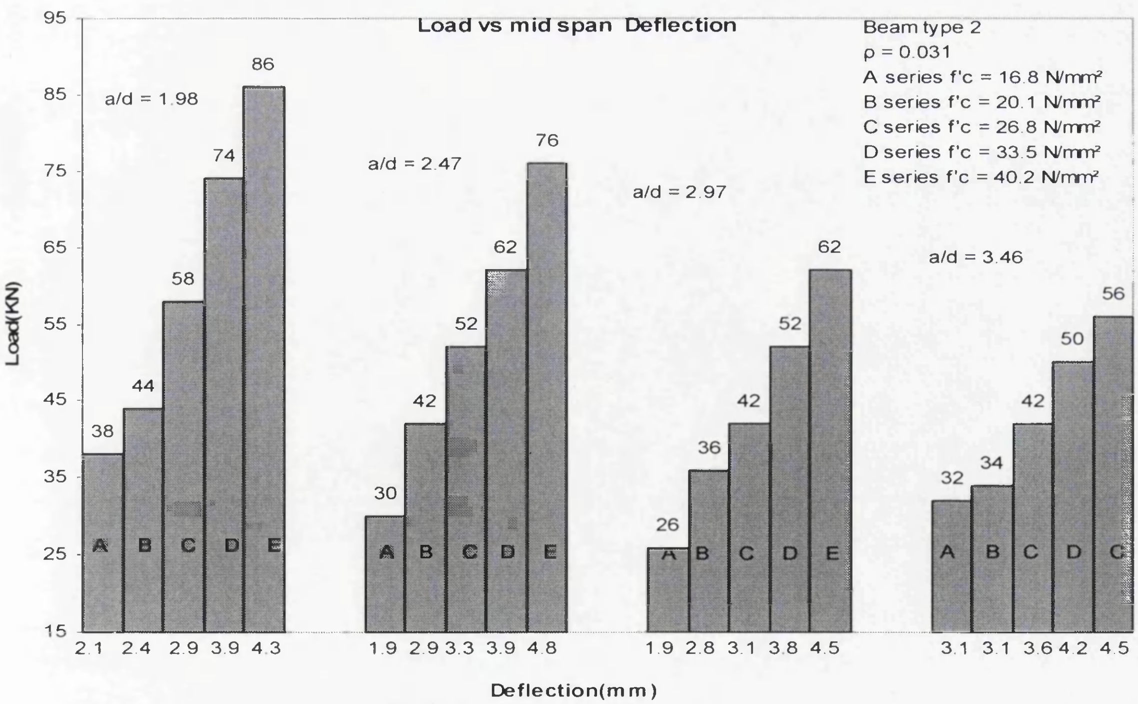

Fig. 34 Load-mid span deflection

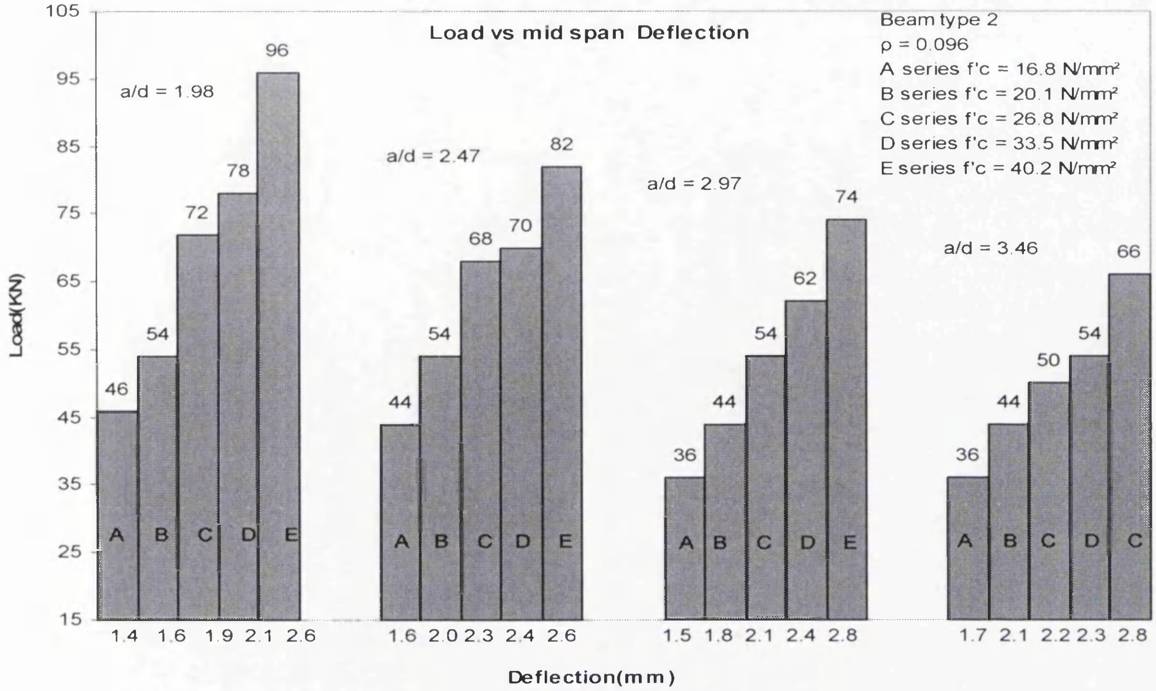

Fig. 35 Load-mid span deflection

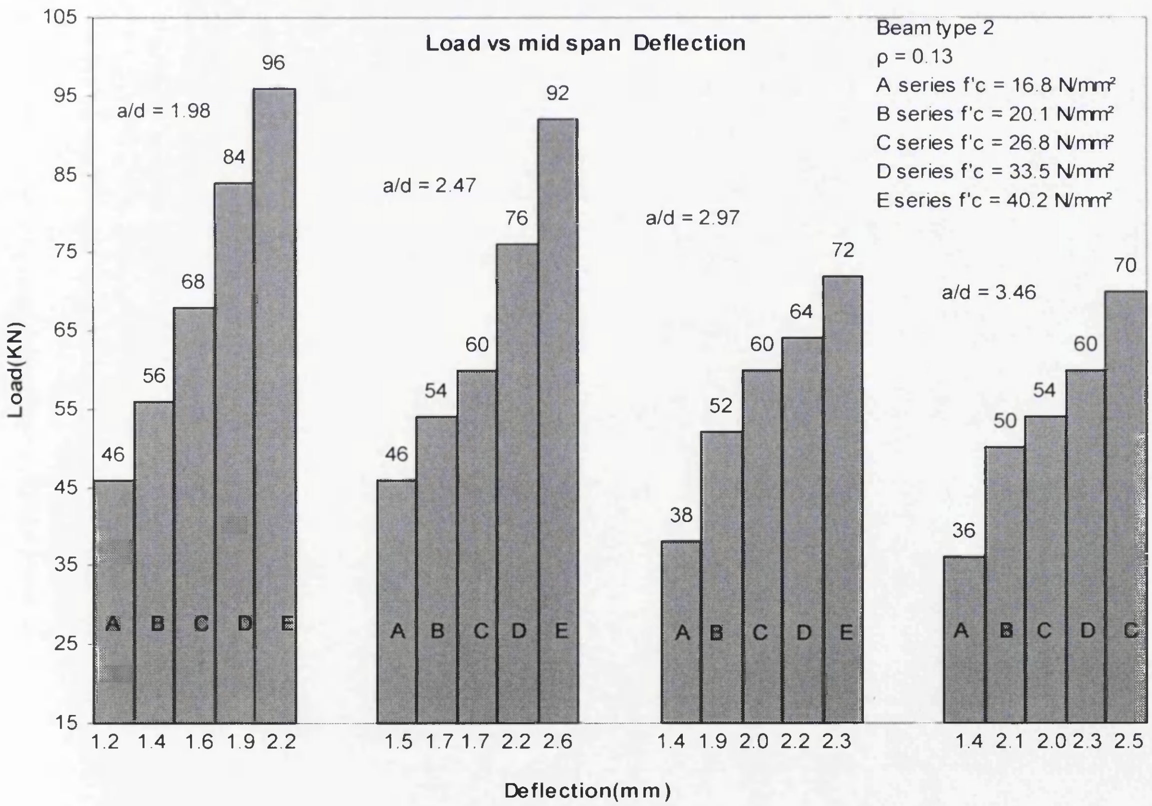

Fig. 36 Load-mid span deflection

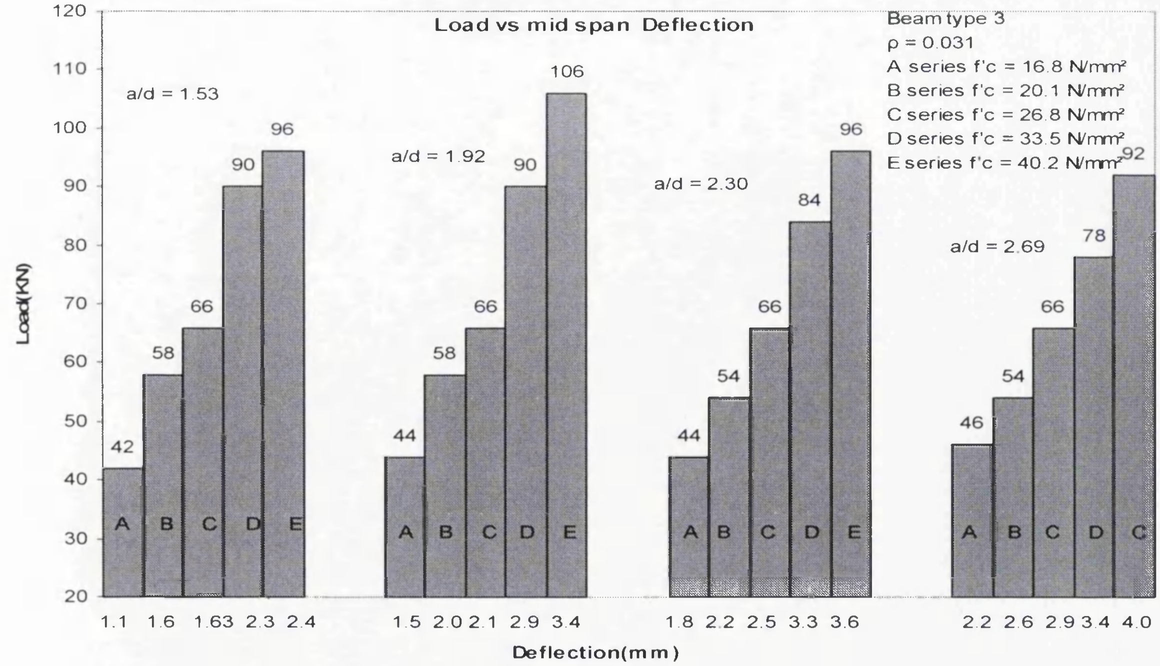

Fig. 37 Load-mid span deflection

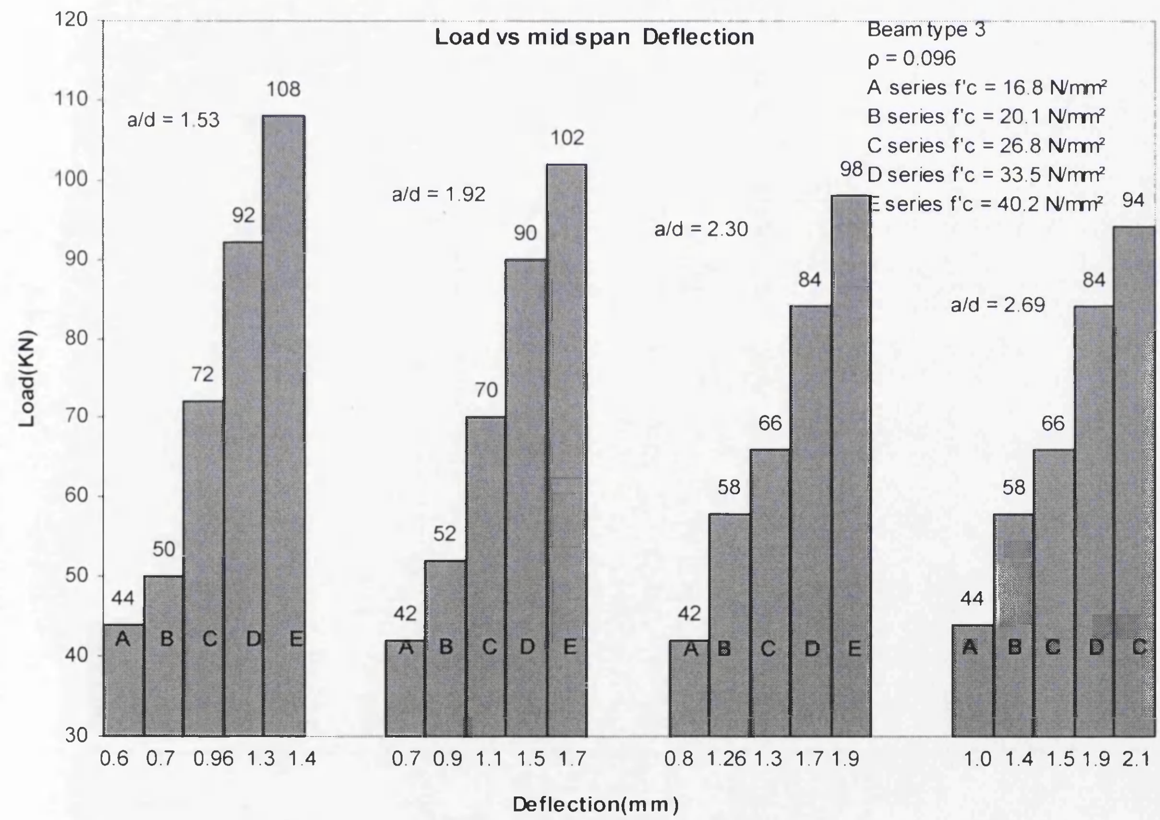

Fig. 38 Load-mid span deflection

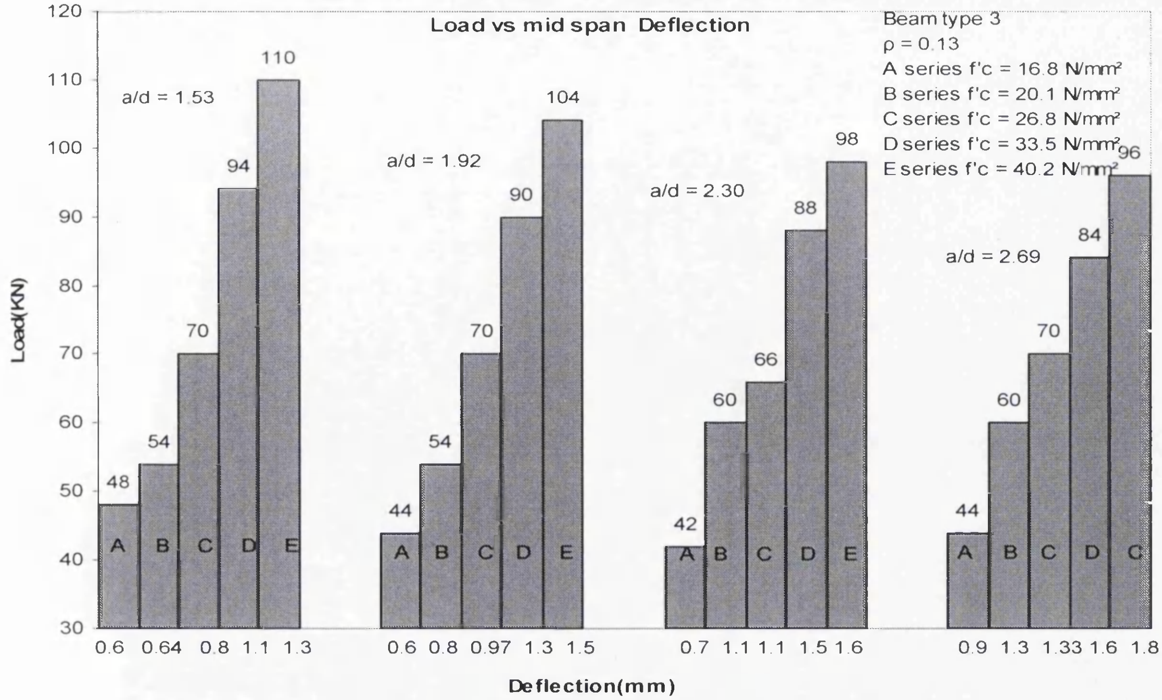

Fig. 39 Load-mid span deflection

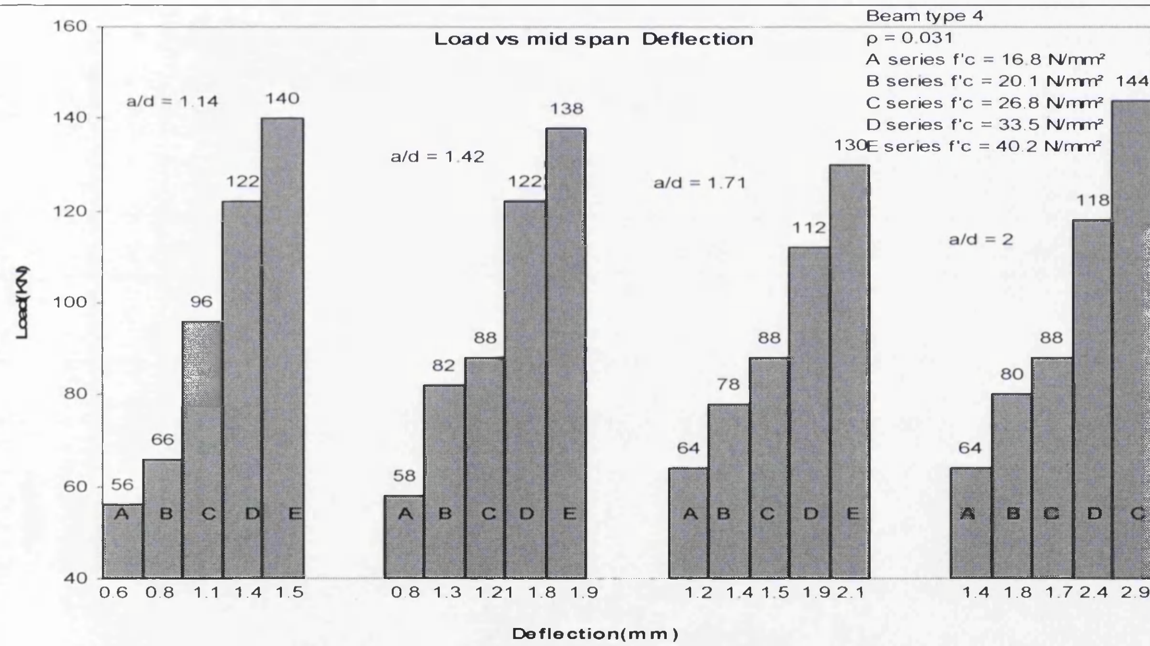

Fig. 40 Load-mid span deflection

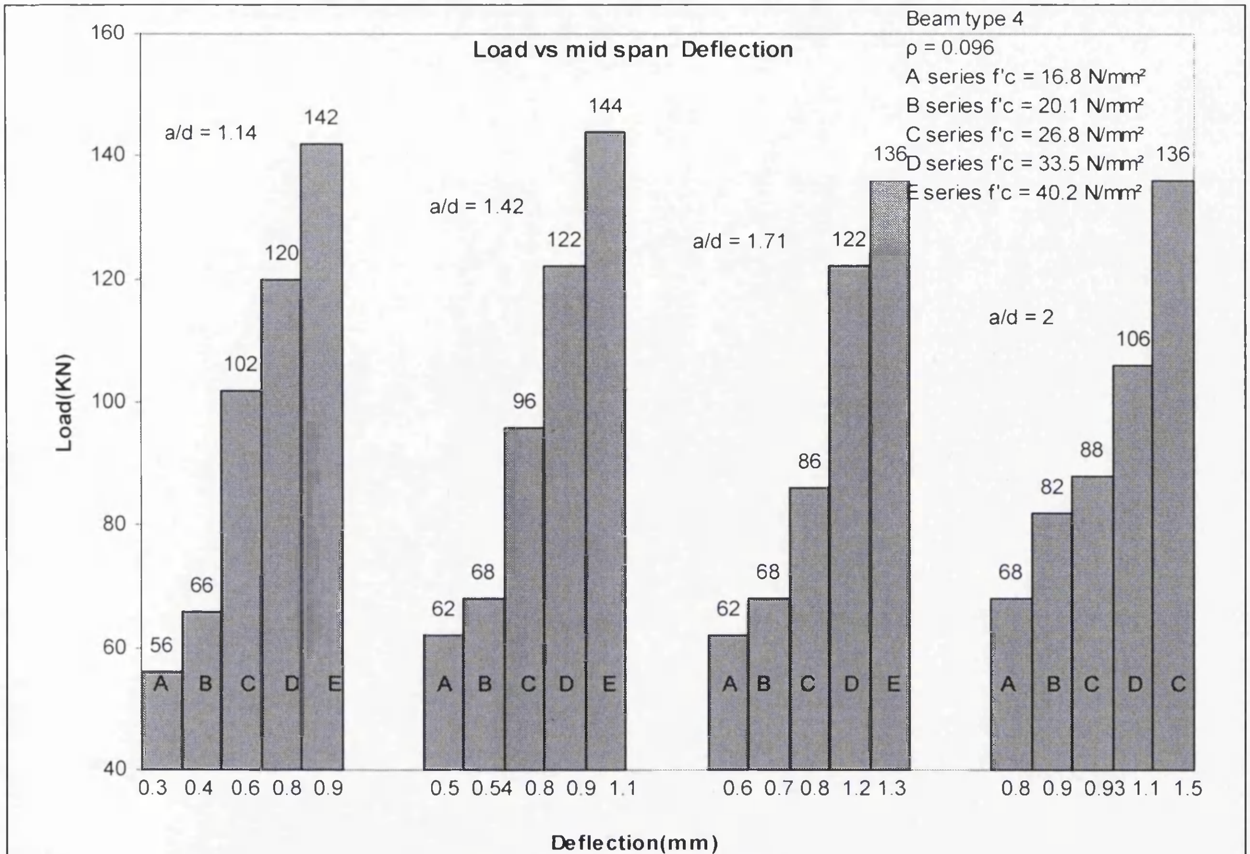

Fig. 41 Load-mid span deflection

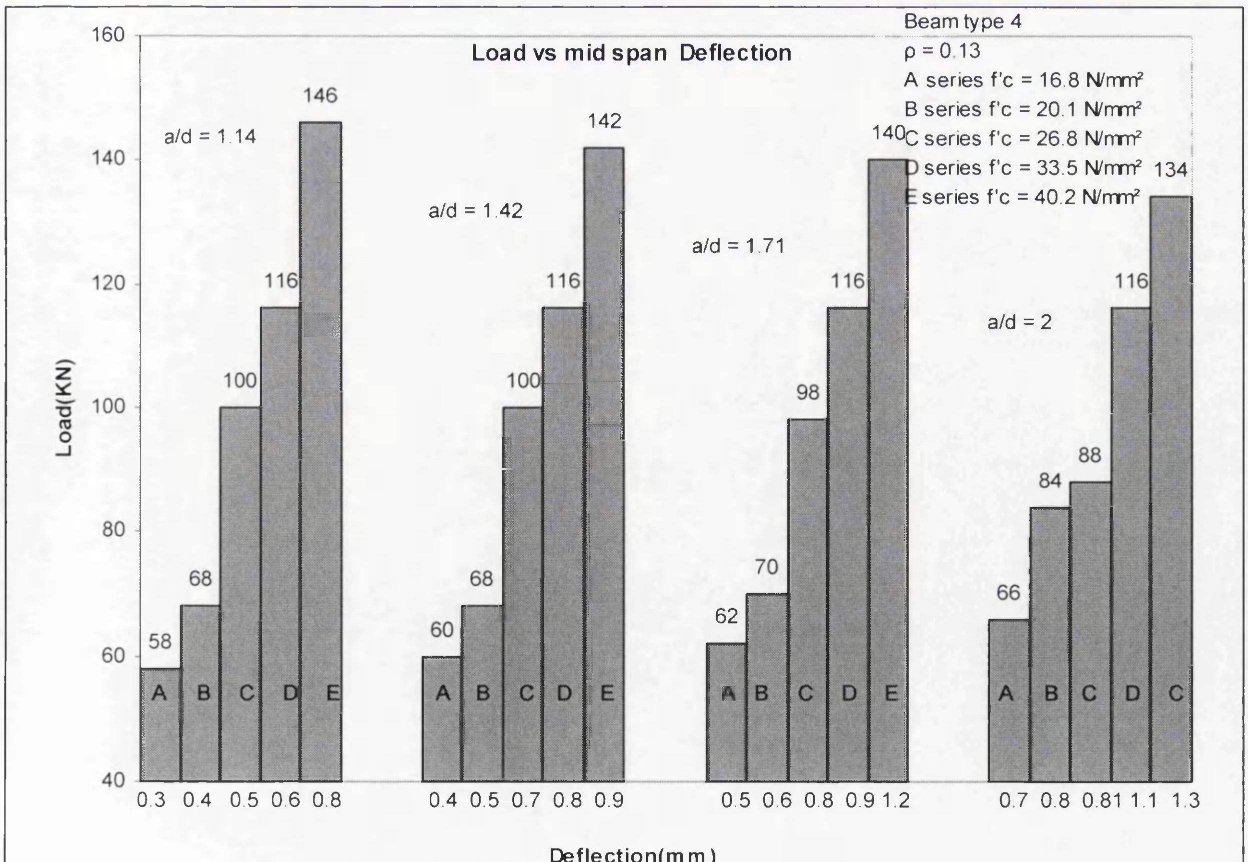

Fig. 42 Load-mid span deflection

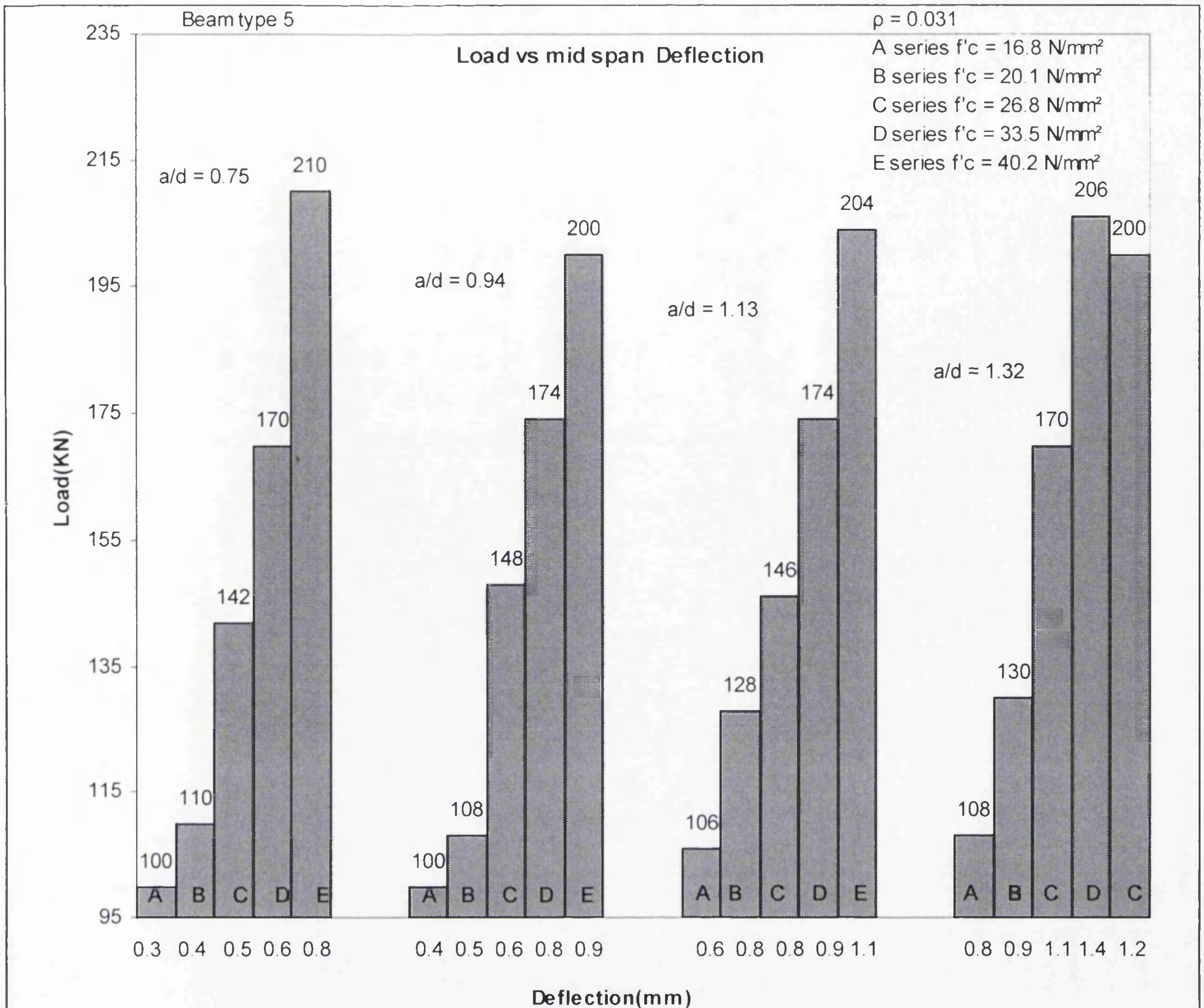

Fig. 43 Load-mid span deflection

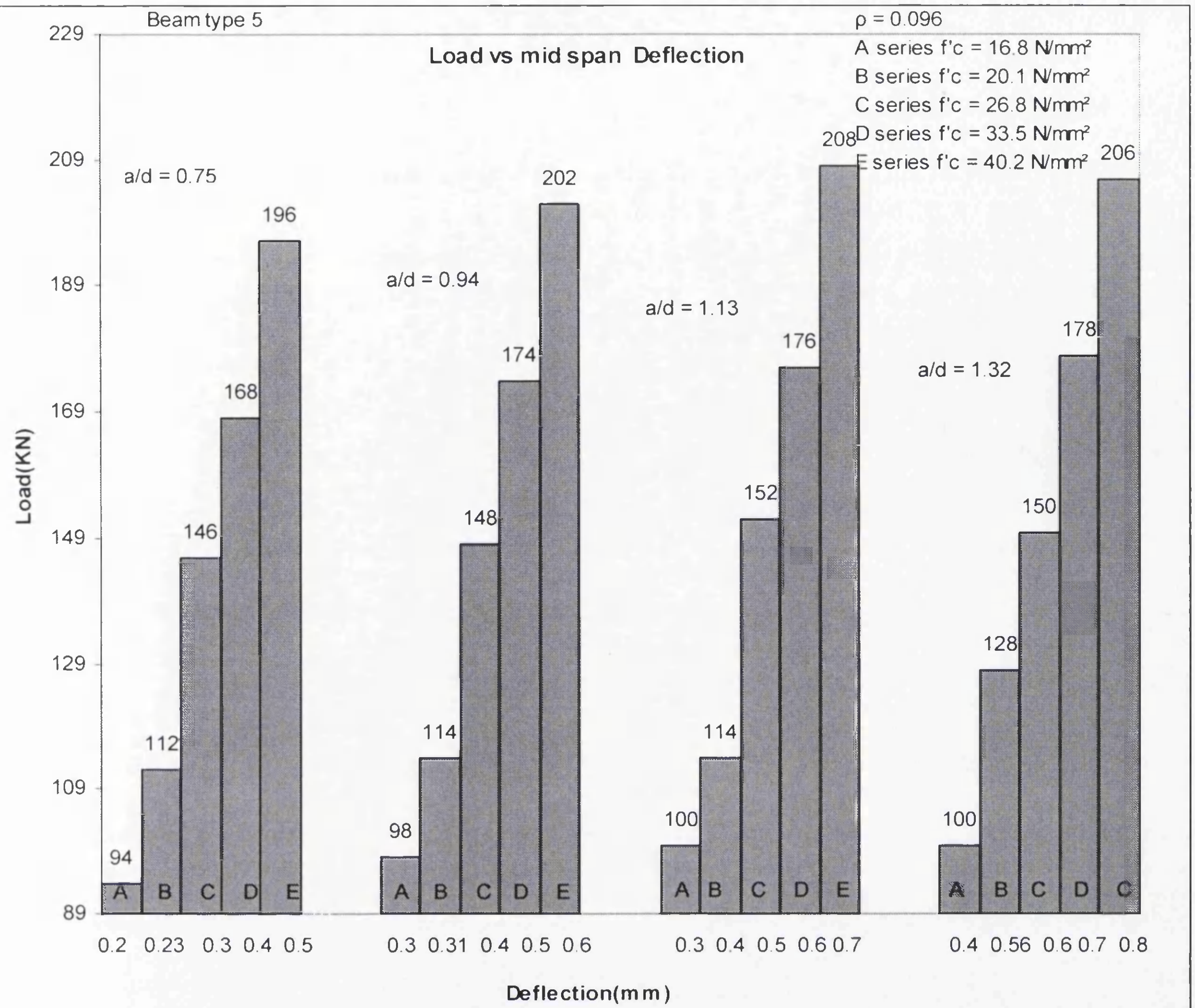

Fig. 44 Load-mid span deflection

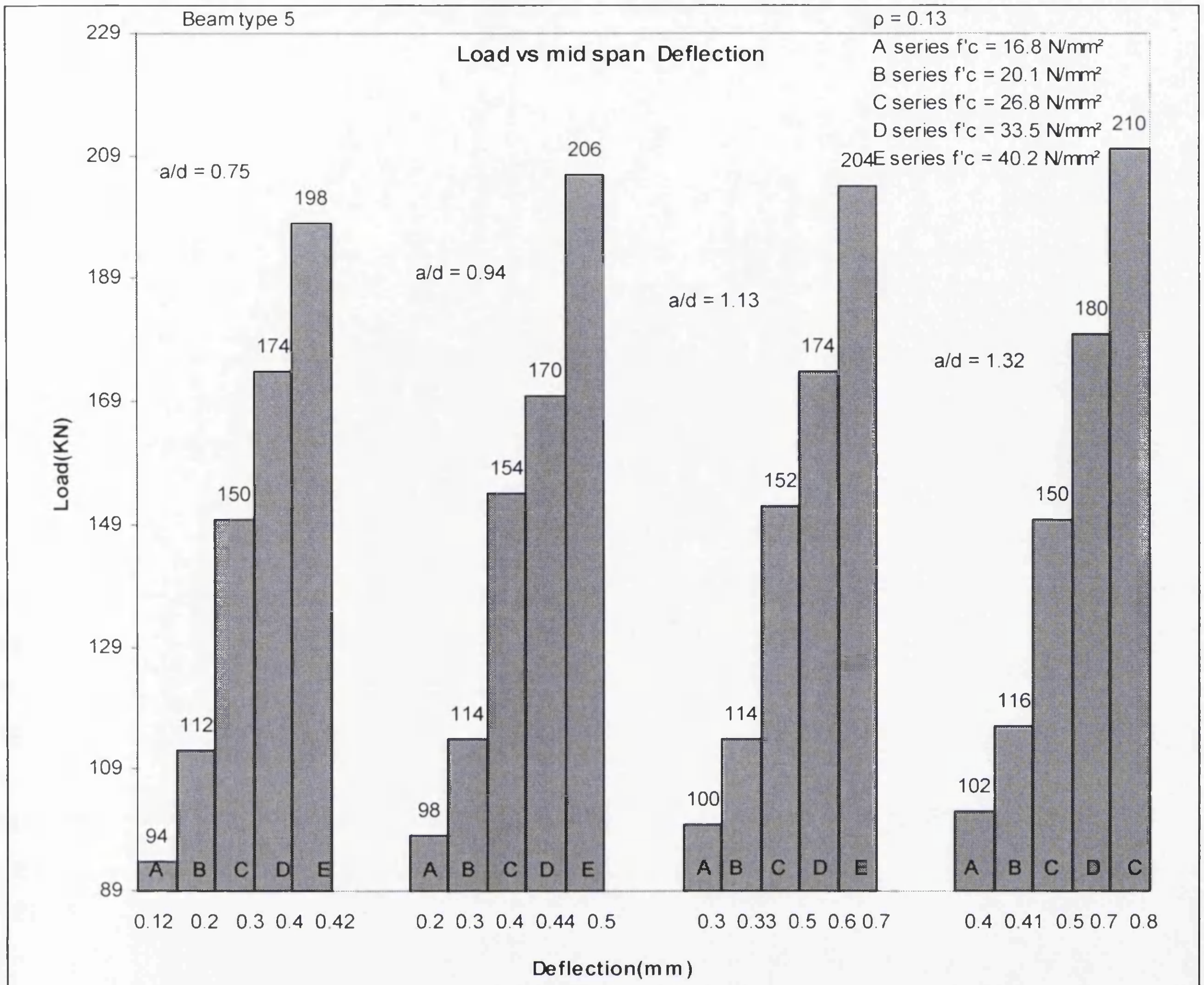

Fig. 45 Load-mid span deflection

APPENDIX - B

Batch File of Beam type 1

```
/TITLE, DEFLECTION OF BEAM
/PREP7
ET, 1, SOLID65
R, 1
R, 1, 3, 0.1,0, 0, 3, 0.1
RMORE, 90,0,3,0.1,0,90
RCON
STAT
!--------------------------------------------------------
MP, EX, 1, 30144
MP, NUXY, 1, 0.15
TB, CONCR, 1
TBDATA, 1, 0.05, 0.2, 3, 16.75
TB, MKIN, 1
TBTEMP, STRAIN
TBDATA, 1, 0.000139, 0.0005, 0.00075, 0.0012, 0.0035
TBTEMP,
TBDATA, 1, 4.19, 10.16, 14.77, 16.65, 16.75
!-----------------------------------------------------------
MP, EX, 3, 30144
MP, NUXY, 3, 0.15
TB, MKIN, 3
TBTEMP,, STRAIN
TBDATA, 1, 0.000139, 0.0005, 0.00075, 0.0012, 0.0035
TBTEMP,
TBDATA, 1, 4.19, 10.16, 14.77, 16.65, 16.75
!------------------------------------------------------------
ET, 2, LINK8            ! LONGITUDINAL STEEL
R, 2, 50.25
MP, EX, 2,200000
MP, NUXY, 2, 0.3
TB, bkin, 2
TBDATA, 1,460
ET, 4, LINK8            ! LINK STEEL
R, 4, 28.27
MP, EX, 4,200000
MP, NUXY, 4, 0.3
TB, bkin, 4
TBDATA, 1,250
```

```
/VIEW, 1,1,1,1
/ANG, 1
/REP, FAST
/AUTO, 1
/REP
LPLOT
! LIST OF Nodes
N, 1,0,0,0
N, 2, 0,0,15
N, 3, 0,0,30
N, 4, 0,0,45
N, 5, 0,0,60
N, 6, 0,0,75
!=======
N, 7, 0,15,0
N, 8, 0, 15, 15
N, 9, 0, 15, 30
N, 10, 0, 15, 45
N, 11, 0, 15, 60
N, 12, 0, 15, 75
!=======
N, 13, 0,25,0
N, 14, 0, 25, 15
N, 15, 0, 25, 30
N, 16, 0, 25, 45
N, 17, 0, 25, 60
N, 18, 0, 25, 75
!========
N, 19, 0,35,0
N, 20, 0, 35, 15
N, 21, 0, 35, 30
N, 22, 0, 35, 45
N, 23, 0, 35, 60
N, 24, 0, 35, 75
!========
N, 25, 0,45,0
N, 26, 0, 45, 15
N, 27, 0, 45, 30
N, 28, 0, 45, 45
N, 29, 0, 45, 60
N, 30, 0, 45, 75
!=========
```

```
N, 31, 0,55,0
N, 32, 0, 55, 15
N, 33, 0, 55, 30
N, 34, 0, 55, 45
N, 35, 0, 55, 60
N, 36, 0, 55, 75
!=========
N, 37, 0,65,0
N, 38, 0, 65, 15
N, 39, 0, 65, 30
N, 40, 0, 65, 45
N, 41, 0, 65, 60
N, 42, 0, 65, 75
!=========
N, 43, 0,75,0
N, 44, 0, 75, 15
N, 45, 0, 75, 30
N, 46, 0, 75, 45
N, 47, 0, 75, 60
N, 48, 0, 75, 75
!========
N, 49, 0,85,0
N, 50, 0, 85, 15
N, 51, 0, 85, 30
N, 52, 0, 85, 45
N, 53, 0, 85, 60
N, 54, 0, 85, 75
!========
N, 55, 0, 100, 0
N, 56,0,100,15
N, 57,0,100,30
N, 58,0,100,45
N, 59,0,100,60
N, 60,0,100,75
NGEN, 41, 60,1,60,1,25
! LIST OF ELEMENTS AND NODES THEY CONNECT
TYPE, 1
MAT, 1
REAL, 1
E, 1, 2, 8,7,61,62,68,67
E, 2, 3, 9,8,62,63,69,68
E, 3,4,10,9,63,64,70,69
```

E, 4, 5,11,10,64,65,71,70
E, 5, 6,12,11,65,66,72,71
!===============
E, 7, 8,14,13,67,68,74,73
E, 8, 9,15,14,68,69,75,74
E, 9, 10,16,15,69,70,76,75
E, 10, 11,17,16,70,71,77,76
E, 11, 12,18,17,71,72,78,77
!===============
E, 13, 14,20,19,73,74,80,79
E, 14, 15,21,20,74,75,81,80
E, 15, 16,22,21,75,76,82,81
E, 16, 17,23,22,76,77,83,82
E, 17, 18,24,23,77,78,84,83
!===============
E, 19, 20,26,25,79,80,86,85
E, 20, 21,27,26,80,81,87,86
E, 21, 22,28,27,81,82,88,87
E, 22, 23,29,28,82,83,89,88
E, 23, 24,30,29,83,84,90,89
!===============
E, 25, 26,32,31,85,86,92,91
E, 26, 27,33,32,86,87,93,92
E, 27, 28,34,33,87,88,94,93
E, 28, 29,35,34,88,89,95,94
E, 29, 30,36,35,89,90,96,95
!===============
E, 31, 32,38,37,91,92,98,97
E, 32, 33,39,38,92,93,99,98
E, 33,34,40,39,93,94,100,99
E, 34, 35, 41,40,94,95,101,100
E, 35, 36, 42,41,95,96,102,101
!================
E, 37, 38, 44,43,97,98,104,103
E, 38, 39, 45,44,98,99,105,104
E, 39, 40,46,45,99,100,106,105
E, 40,41,47,46,100,101,107,106
E, 41,42,48,47,101,102,108,107
!=================
E, 43,44,50,49,103,104,110,109
E, 44,45,51,50,104,105,111,110
E, 45,46,52,51,105,106,112,111

```
E, 46,47,53,52,106,107,113,112
E, 47,48,54,53,107,108,114,113
!==================
E, 49,50,56,55,109,110,116,115
E, 50,51,57,56,110,111,117,116
E, 51,52,58,57,111,112,118,117
E, 52,53,59,58,112,113,119,118
E, 53,54,60,59,113,114,120,119
EGEN, 40, 60, 1, 45,,,,,,, 25

TYPE, 2
MAT, 2
REAL, 2                    ! LONGITUDINAL STEEL
E, 8, 68
E, 68,128
E, 128,188
E, 188,248
E, 248,308
E, 308,368
E, 368,428
E, 428,488
E, 488,548
E, 548,608
E, 608,668
E, 668,728
E, 728,788
E, 788,848
E, 848,908
E, 908,968
E, 968, 1028
E, 1028, 1088
E, 1088, 1148
E, 1148, 1208
E, 1208, 1268
E, 1268, 1328
E, 1328, 1388
E, 1388, 1448
E, 1448, 1508
E, 1508, 1568
E, 1568, 1628
E, 1628, 1688
E, 1688, 1748
```

E, 1748, 1808
E, 1808, 1868
E, 1868, 1928
E, 1928, 1988
E, 1988, 2048
E, 2048, 2108
E, 2108, 2168
E, 2168, 2228
E, 2228, 2288
E, 2288, 2348
E, 2348, 2408
!====
E, 9, 69
E, 69,129
E, 129,189
E, 189,249
E, 249,309
E, 309,369
E, 369,429
E, 429,489
E, 489,549
E, 549,609
E, 609,669
E, 669,729
E, 729,789
E, 789,849
E, 849,909
E, 909,969
E, 969, 1029
E, 1029, 1089
E, 1089, 1149
E, 1149, 1209
E, 1209, 1269
E, 1269, 1329
E, 1329, 1389
E, 1389, 1449
E, 1449, 1509
E, 1509, 1569
E, 1569, 1629
E, 1629, 1689
E, 1689, 1749
E, 1749, 1809

E, 1809, 1869

E, 1869, 1929

E, 1929, 1989

E, 1989, 2049

E, 2049, 2109

E, 2109, 2169

E, 2169, 2229

E, 2229, 2289

E, 2289, 2349

E, 2349, 2409

!====

E, 10, 70

E, 70,130

E, 130,190

E, 190,250

E, 250,310

E, 310,370

E, 370,430

E, 430,490

E, 490,550

E, 550,610

E, 610,670

E, 670,730

E, 730,790

E, 790,850

E, 850,910

E, 910,970

E, 970, 1030

E, 1030, 1090

E, 1090, 1150

E, 1150, 1210

E, 1210, 1270

E, 1270, 1330

E, 1330, 1390

E, 1390, 1450

E, 1450, 1510

E, 1510, 1570

E, 1570, 1630

E, 1630, 1690

E, 1690, 1750

E, 1750, 1810

E, 1810, 1870

E, 1870, 1930
E, 1930, 1990
E, 1990, 2050
E, 2050, 2110
E, 2110, 2170
E, 2170, 2230
E, 2230, 2290
E, 2290, 2350
E, 2350, 2410
!====
E, 11, 71
E, 71,131
E, 131,191
E, 191,251
E, 251,311
E, 311,371
E, 371,431
E, 431,491
E, 491,551
E, 551,611
E, 611,671
E, 671,731
E, 731,791
E, 791,851
E, 851,911
E, 911,971
E, 971, 1031
E, 1031, 1091
E, 1091, 1151
E, 1151, 1211
E, 1211, 1271
E, 1271, 1331
E, 1331, 1391
E, 1391, 1451
E, 1451, 1511
E, 1511, 1571
E, 1571, 1631
E, 1631, 1691
E, 1691, 1751
E, 1751, 1811
E, 1811, 1871
E, 1871, 1931

E, 1931, 1991

E, 1991, 2051

E, 2051, 2111

E, 2111, 2171

E, 2171, 2231

E, 2231, 2291

E, 2291, 2351

E, 2351, 2411

!=====

E, 53,113

E, 113,173

E, 173,233

E, 233,293

E, 293,353

E, 353,413

E, 413,473

E, 473,533

E, 533,593

E, 593,653

E, 653,713

E, 713,773

E, 773,833

E, 833,893

E, 893,953

E, 953, 1013

E, 1013, 1073

E, 1073, 1133

E, 1133, 1193

E, 1193, 1253

E, 1253, 1313

E, 1313, 1373

E, 1373, 1433

E, 1433, 1493

E, 1493, 1553

E, 1553, 1613

E, 1613, 1673

E, 1673, 1733

E, 1733, 1793

E, 1793, 1853

E, 1853, 1913

E, 1913, 1973

E, 1973, 2033

E, 2033, 2093
E, 2093, 2153
E, 2153, 2213
E, 2213, 2273
E, 2273, 2333
E, 2333, 2393
E, 2393, 2453
!====
E, 50,110
E, 110,170
E, 170,230
E, 230,290
E, 290,350
E, 350,410
E, 410,470
E, 470,530
E, 530,590
E, 590,650
E, 650,710
E, 710,770
E, 770,830
E, 830,890
E, 890,950
E, 950, 1010
E, 1010, 1070
E, 1070, 1130
E, 1130, 1190
E, 1190, 1250
E, 1250, 1310
E, 1310, 1370
E, 1370, 1430
E, 1430, 1490
E, 1490, 1550
E, 1550, 1610
E, 1610, 1670
E, 1670, 1730
E, 1730, 1790
E, 1790, 1850
E, 1850, 1910
E, 1910, 1970
E, 1970, 2030
E, 2030, 2090

```
E, 2090, 2150
E, 2150, 2210
E, 2210, 2270
E, 2270, 2330
E, 2330, 2390
E, 2390, 2450

                ! TRANVERSE STEEL
TYPE, 2
MAT, 2
REAL, 4

E, 128,129
E, 129,130
E, 130,131
E, 131,137
E, 137,143
E, 143,149
E, 149,155
E, 155,161
E, 161,167
E, 167,173
E, 173,172
E, 172,171
E, 171,170
E, 170,164
E, 164,158
E, 158,152
E, 152,146
E, 146,140
E, 140,134
E, 134,128
!======
E, 368,369
E, 369,370
E, 370,371
E, 371,377
E, 377,383
E, 383,389
E, 389,395
E, 395,401
E, 401,407
```

E, 407,413
E, 413,412
E, 412,411
E, 411,410
E, 410,404
E, 404,398
E, 398,392
E, 392,386
E, 386,380
E, 380,374
E, 374,368
!======
E, 608,609
E, 609,610
E, 610,611
E, 611,617
E, 617,623
E, 623,629
E, 629,635
E, 635,641
E, 641,647
E, 647,653
E, 653,652
E, 652,651
E, 651,650
E, 650,644
E, 644,638
E, 638,632
E, 632,626
E, 626,620
E, 620,614
E, 614,608
!======
E, 848,849
E, 849,850
E, 850,851
E, 851,857
E, 857,863
E, 863,869
E, 869,875
E, 875,881
E, 881,887

E, 887,893

E, 893,892

E, 892,891

E, 891,890

E, 890,884

E, 884,878

E, 878,872

E, 872,866

E, 866,860

E, 860,854

E, 854,848

!======

E, 1088, 1089

E, 1089, 1090

E, 1090, 1091

E, 1091, 1097

E, 1097, 1103

E, 1103, 1109

E, 1109, 1115

E, 1115, 1121

E, 1121, 1127

E, 1127, 1133

E, 1133, 1132

E, 1132, 1131

E, 1131, 1130

E, 1130, 1124

E, 1124, 1118

E, 1118, 1112

E, 1112, 1106

E, 1106, 1100

E, 1100, 1094

E, 1094, 1088

!========

E, 1328, 1329

E, 1329, 1330

E, 1330, 1331

E, 1331, 1337

E, 1337, 1343

E, 1343, 1349

E, 1349, 1355

E, 1355, 1361

E, 1361, 1367

E, 1367, 1373
E, 1373, 1372
E, 1372, 1371
E, 1371, 1370
E, 1370, 1364
E, 1364, 1358
E, 1358, 1352
E, 1352, 1346
E, 1346, 1340
E, 1340, 1334
E, 1334, 1328
!========
E, 1568, 1569
E, 1569, 1570
E, 1570, 1571
E, 1571, 1577
E, 1577, 1583
E, 1583, 1589
E, 1589, 1595
E, 1595, 1601
E, 1601, 1607
E, 1607, 1613
E, 1613, 1612
E, 1612, 1611
E, 1611, 1610
E, 1610, 1604
E, 1604, 1598
E, 1598, 1592
E, 1592, 1586
E, 1586, 1580
E, 1580, 1574
E, 1574, 1568
!========
E, 1808, 1809
E, 1809, 1810
E, 1810, 1811
E, 1811, 1817
E, 1817, 1823
E, 1823, 1829
E, 1829, 1835
E, 1835, 1841
E, 1841, 1847

E, 1847, 1853

E, 1853, 1852

E, 1852, 1851

E, 1851, 1850

E, 1850, 1844

E, 1844, 1838

E, 1838, 1832

E, 1832, 1826

E, 1826, 1820

E, 1820, 1814

E, 1814, 1808

!========

E, 2048, 2049

E, 2049, 2050

E, 2050, 2051

E, 2051, 2057

E, 2057, 2063

E, 2063, 2069

E, 2069, 2075

E, 2075, 2081

E, 2081, 2087

E, 2087, 2093

E, 2093, 2092

E, 2092, 2091

E, 2091, 2090

E, 2090, 2084

E, 2084, 2078

E, 2078, 2072

E, 2072, 2066

E, 2066, 2060

E, 2060, 2054

E, 2054, 2048

!========

E, 2288, 2289

E, 2289, 2290

E, 2290, 2291

E, 2291, 2297

E, 2297, 2303

E, 2303, 2309

E, 2309, 2315

E, 2315, 2321

E, 2321, 2327

```
E, 2327, 2333
E, 2333, 2332
E, 2332, 2331
E, 2331, 2330
E, 2330, 2324
E, 2324, 2318
E, 2318, 2312
E, 2312, 2306
E, 2306, 2300
E, 2300, 2294
E, 2294, 2288
/Solu
! APPLIED FORCE
x=13000
F, 655, FY,-x/6
F, 656, FY,-x/6
F, 657, FY,-x/6
F, 658, FY,-x/6
F, 659, FY,-x/6
F, 660, FY,-x/6

F, 1855, FY,-x/6
F, 1856, FY,-x/6
F, 1857, FY,-x/6
F, 1858, FY,-x/6
F, 1859, FY,-x/6
F, 1860, FY,-x/6

FLST, 2, 6, 1,
FITEM, 2,121
FITEM, 2,122
FITEM, 2,123
FITEM, 2,124
FITEM, 2,125
FITEM, 2,126
D,P51X, ,0, , , , UX,UY,UZ, , ,

FLST, 2, 6, 1,
FITEM, 2, 2281
FITEM, 2, 2282
FITEM, 2, 2283
FITEM, 2, 2284
```

```
FITEM, 2, 2285
FITEM, 2, 2286
D,P51X, ,0, , , , ,UY,UZ, , ,

SSTIF, ON
AUTOTS, Off
NSUBST, 9
TIME, 24
OUTRES, ALL, ALL
NEQIT, 220
NLGEOM, ON
Solve

/POST1
/DSCALE, 1, 10
PLDISP, 1                    ! PLOT DISPLACED SHAPE
FINISH
/POST26
/AXLAB, Y, FORCE
/AXLAB, X, DISPLACEMENT
NSOL, 2, 1206, U, Y
RFORCE, 3, 126, F, Y
PROD, 2, 2,,,,,,-1
PROD, 3, 3,,,,,,+1
XVAR, 2
PLVAR, 3                     ! PLOT DISPLACEMENT VS FORCE
PRVAR, 2, 3
```