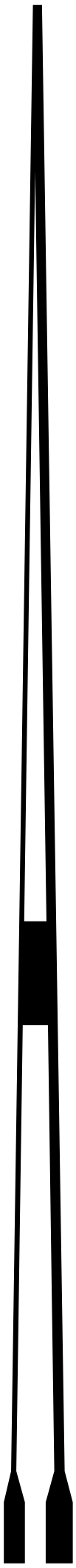
UTOMATED RELIABILITY - BASED DESIGN



PROGRAM FOR ONE-WAY REINFORCED CONCRETE SLABS TO BS8110 (1985)

BY

ABUBAKAR IDRIS

B.ENG. (CIVIL)

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Zaria.

August 1999.

DECLARATION

I hereby declare that this thesis is my own research work and has been carried out by myself and it has not been previously presented for a higher degree.

All sources of information which are not my own original work are specifically acknowledged by references.

Abubakar Idris

August, 1999

CERTIFICATION

This project thesis entitled: **"AUTOMATED RELIABILITY-BASED DESIGN PROGRAM FOR REINFORCED CONCRETE SLABS TO BRITISH STANDARD CODE, BS8110 (1985), REQUIREMENTS)"** by Abubakar Idris, meets the regulations governing the award of the degree of Master of Science (Structural Engineering) of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

.....	4.10.99 .....
Dr. J.O. Afólayan Chairman, Supervisory Committee	Date
.....	4/10/99 .....
Engr. S. S. Letcha Member, Supervisory Committee	Date
.....	4.10.99 .....
Dr. K. J. Osinubi Head of Department	Date
.....	21/08/00 .....
Dean of Postgraduate School	Date

DEDICATION

To my Late Parents, may souls rest in Peace. Amen.

ACKNOWLEDGMENT

I am greatly indebted to my main supervisor, Dr. J.O. Afolayan for introducing me into this area of study and for his encouragement and advice. His constructive critisims and approchability during the course of this work cannot be over emphasized.

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I am also grateful to Prof. O. Adebisi who showed particular interest in my work. Engr. Abejide who is always ready to advice will not be left out in this angle as well as Engr. A. T. Olowosulu and to members of staff of Civil Engineering Department.

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This thesis would not be complete if my appreciation is not given to my friends with and whom I shared some of the most cherished moments especially Engr. Ali, Amana, Saidu, and, T.J., to mention but a few. My about-to-wed wife is also not left out.

Finally, Praise be to God Who gives strength, wisdom and guidance.

ABSTRACT

The BS8110 (1985) is the revised version of CP110 (1972) and is used for the design of reinforced concrete elements. In order to maintain safe designs, partial safety factors are applied to material properties as well as the loadings on the structure. Studies have shown that these factors result in high investment and inconsistensy in the safety level for varying design parameters and practical uncertainties.

The work in this thesis provides FORTRAN-77 modules for the design of one way slabs at uniform reliability level using the first-order reliability method. Based on the results obtained from the program, plots of reinforcement ratio versus load ratio and reinforcement ratio versus effective depth for each set of design input parameters are provided. It is shown for uniformity in reliability level, there must be a linear relationship between the reinforcement ratio and effective depth. However, a non linear relationship exists between the reinforcement ratio and the ratio of dead to live load. Equations showing these effects are also developed and can be used directly for safe designs. Within the requirements of BS8110 for one-way slabs and practical uncertainties, the results of this study show that calculation and processing time is greatly reduced. Also, safety of the designed element is known a priori and problems of over -and /or -under designs are eliminated with a trade-off between safety and economy.

4

Summarily, the design of one way slabs at uniform reliability level can be carried out using one of the following: (1) the computer program; (2) the plots presented or (3) the use of the equations determined.

TABLE OF CONTENTS

CHAPTER		PAGE
	Title Page	i
	Declaration	ii
	Certification	iii
	Dedication	iv
	Acknowledgement	v
	Abstract	vi
	Table of Contents	vii
	List of Tables	xi
	List of Figures	xii
	Notations	xiv
	CHAPTER ONE	**1**
	INTRODUCTION	1
1.1	General	1
1.2	Problem statement and Justification of Study	2
1.3	Objectives and Scope	4
	CHAPTER TWO	6
	LITERATURE REVIEW	6
2.1	Slabs	6
2.2	Work done on Reliability Application - in Structural Engineering	7
2.3	Work done on Computer Application - in Structural Engineering	9

	CHAPTER THREE	11
	ANALYTICAL TECHNIQUES	11
3.1	Analysis and Design of Slabs	11
3.1.1	Moments and Shear Forces	11
3.1.2	Limit State of Deflection	11
3.1.3	Limit State of Cracking	12
3.1.4	Minimum area of Reinforcement	13
3.1.5	Concrete cover for Durability	13
3.1.6	Fire Resistance	14
3.2	Safety Index Concept	14
3.3	Second-Moment Formulation	16
3.4	Generalization as a Function of- Basic Variables	23
3.4.1	First Order Interpretation	23
3.5	Transformation of Basic Variables	24
3.5.1	Non-Normal Variates	25
3.5.2	Correlated Variates	26
3.6	FORM5	29
3.7	Design Criterion	31
	CHAPTER FOUR	32
	PROGRAM DEVELOPMENT	32
4.1	Program Flowchart	32
4.2	Program Listing with explanation of - each Segment	35
4.3	Program Listing for Parameters of - FORM5 as used in the Program	37

4.4	The Program Statement	41
4.5	Input Data Definitions	56
	CHAPTER FIVE	57
	DEMONSTRATION AND DISCUSSION OF RESULTS	57
5.1	Demonstration and Discussion	57
5.2	Typical Design Example of a Slab	58
5.3	Results Obtained from Design Program	60
	CHAPTER SIX	90
	CONCLUSION AND RECOMMENDATIONS	90
6.1	Conclusion	90
6.2	Recommendations	91
	REFERENCES	92

LIST OF TABLES

TABLE		PAGE
5.1	Table of Results for Simply Supported Slab	60
5.2	Table of Results for Simply Supported Slab	60
5.3	Table of Results for Simply Supported Slab	61
5.4	Table of Results for Cantileverd Supported Slab	61
5.5	Table of Results for Cantileverd Supported Slab	62
5.6	Table of Results for Cantileverd Supported Slab	62
5.7	Table of Results for Propped Cantilevered Slab	63
5.8	Table of Results for Propped Cantilevered Slab	63
5.9	Table of Results for Propped Cantilevered Slab	64
5.10	Table of Results for Fixed-Fixed Slab	64
5.11	Table of Results for Fixed-Fixed Slab	65
5.12	Table of Results for Fixed-Fixed Slab	65

LIST OF FIGURES

FIGURE		PAGE
3.1	Basic Strength Capacity-Load Effect Problem	14
3.2	Limit State Boundary	17
3.3	Flow Chart for the Computation of β	30
4.1	Program Flowchart	34
5.1	Reinforcement Ratio versus Load ratio for Simply Supported Slab	66
5.2	Reinforcement Ratio versus Effective depth for Simply Supported Slab	67
5.3	Reinforcement Ratio versus Load ratio for Simply Supported Slab	68
5.4	Reinforcement Ratio versus Effective depth for Simply Supported Slab	69
5.5	Reinforcement Ratio versus Load ratio for Simply Supported Slab	70
5.6	Reinforcement Ratio versus Effective depth for Simply Supported Slab	71
5.7	Reinforcement Ratio versus Load ratio for Cantilevered Supported Slab	72
5.8	Reinforcement Ratio versus Effective depth for Cantilevered Supported Slab	73
5.9	Reinforcement Ratio versus Load ratio for Cantilevered Supported Slab	74
5.10	Reinforcement Ratio versus Effective depth for Cantilevered Supported Slab	75
5.11	Reinforcement Ratio versus Load ratio for Cantilevered Supported Slab	76
5.12	Reinforcement Ratio versus Effective depth for Cantilevered Supported Slab	77
5.13	Reinforcement Ratio versus Load ratio for Propped Cantilevered Supported Slab	78
5.14	Reinforcement Ratio versus Effective depth	

for Propped Cantilevered Supported Slab 79

5.15 Reinforcement Ratio versus Load ratio
for Propped Cantilevered Supported Slab 80

5.16 Reinforcement Ratio versus Effective depth
for Propped Cantilevered Supported Slab 81

5.17 Reinforcement Ratio versus Load ratio
for Propped Cantilevered Supported Slab 82

5.18 Reinforcement Ratio versus Effective depth
for Propped Cantilevered Supported Slab 83

5.19 Reinforcement Ratio versus Load ratio
for Fixed-Fixed Supported Slab 84

5.20 Reinforcement Ratio versus Effective depth
for Fixed-Fixed Supported Slab 85

5.21 Reinforcement Ratio versus Load ratio
for Fixed-Fixed Supported Slab 86

5.22 Reinforcement Ratio versus Effective depth
for Fixed-Fixed Supported Slab 87

5.23 Reinforcement Ratio versus Load ratio
for Fixed-Fixed Supported Slab 88

5.24 Reinforcement Ratio versus Effective depth
for Fixed-Fixed Supported Slab 89

E[ ]	Expected value (mean)
$F_R(s)$	Cummulative distribution fuction for R
$f_S(s)$	Probability density function for S
g( )	Safety Margin
P_f	Probability of failure
R	Strength capacity
S	Load effect
V	Coefficient of variation
Var[ ]	Variance
σ_R	Standard deviation of strength capacity
σ_s	Standard deviation of load effect
ρ	Reinforcement ratio
ρ_o	correlation coefficient
Φ	Standard deviation function
μ	Mean of g( )
α	Load ratio
β	Safety index
β	Target Safety index
ε	Level of acceptance of the design point

CHAPTER ONE

1.0 INTRODUCTION

1.1 GENERAL

The work presents a modest response to the need for simplified methods of analysing and designing reinforced concrete slabs at a constant reliability level.

Reinforced concrete is a composite element made up of steel and concrete. It is also made up of cement and aggregates mixed and cured with water so that a good degree of uniformity can be obtained in its behaviour.

Reinforced concrete is very good in compression but poor in tension. Under loads, compressive and tensile stresses are set up . Reinforcing Steel which is a material with very high tensile capacity is used to bear the tensile weakness of the concrete. The tensile stress of concrete is usually neglected because it is very small (BS8110, 1985).

According to O'connor (1992), reliability is defined as the ability of an item to perform a required function under stated conditions for a stated period of time. Failure on the other hand is defined as the termination of the ability of an item to perform a required function. Thus, reliability and failure are related (Weissman et al, 1994).

Uncertainties exist in design (BS8110, 1985), Uncertainties include human errors, negligence, poor workmanship, neglected loadings, prediction of the failure events and modelling. Under-estimation of these uncertainties sometimes leads to adverse results such as collapse. For

example, the collapse of a 12-Storey building in Egypt killing many people (CNN, 1996). Floor slabs to multi-storey building in Leeds (Mays and Hutchinson, 1998) have failed due to cracking. This failure was attributed to deficiency in Shear capacity and top flexural reinforcement. Therefore, it is because of uncertainties that question of safety and performance arise (Rojiani and Woeste, 1982).

Structural reliablity theory is therefore concerned with the rational treatment of uncertainties in structural engineering and with the methods for assessing the safety and serviceability of civil engineering and other structures (Thoft-christensen and Baker, 1982).

In order to evaluate the effect of uncertainties, advanced principles of reliability are employed. The reliability based rating used in this thesis does not only examine the uncertainties and consequences of failure but also brings out the level of safety associated with the BS8110 design criteria for reinforced concrete slabs.

By bringing computer tools to the desk of virtually every engineer, analysis and design packages open way to much more widespread use of computer techniques, and they challenge the inventiveness of engineers to find new ways of using these techniques in tackling design problems (Falk, 1985).

Design programs therefore take care of the manual procedures and to consider the level of safety associated with any code of practice(Benasson et al,1980).

1.2 PROBLEM STATEMENT AND JUSTIFICATION OF STUDY

The CP110(1972) and BS8110(1985) which are prescriptive and deterministic, do not consider explicitly the variability inherent in the variables usually considered during design. However, safety factors are assigned to the variables in the design equations without adequate consideration for their uncertain characteristics. There is the lack of variance of the safety measures introduced by the codes of practice.

Reliability analysis embodies the combination of all available probablistic and statistical techniques as well as the modern science of decision making in such a manner that the reliability estimate does not only reflect the quality and quantity of the available information but provides a consistency with present state of stress analysis, failure analysis, and environmental and load analysis (Afolayan, 1992).

In spite of the incorporation of these so called safety factors, the current deterministic design methods cannot provide consistent safety levels. Afolayan(1990) showed that safety levels implicit in two international codes: AISI(1968) and ECCS(1987) for cold-formed structural steel members are not consistent. The inconsistency of the levels of safety and unexpected failure of some structures due to uncertainties or any other reasons justify a code review not only to check and maintain safety level but also to properly accomodate randomness and uncertainties.

Benasson et al (1980) were of the opinion that since

computers are coming into more general use in design offices, computer programs based on CP110 (1972) code requirements and any forthcoming code requirement should be used for design purposes, so as to save time in design. They stressed that computer programs should not only be used for complicated designs but that simple design procedures can also benefit from such an operation.

In this line, the need for a reliability-based design program for reinforced concrete slabs cannot be over-emphasized. Such a program simplifies the design process and equally systematically predicts the level of safety expected.

1.3 OBJECTIVES AND SCOPE

In view of the fact that uncertainties exist in the design of structures, and considering the cost of lives and materials as a result of poor design using the codes of practice, it is possible to improve on the existing methods to provide more accurate and economical designs. With this in mind, the main objective of this work is to come with a Fortran program for the design of various slab-types at a uniform reliability level using the first-order reliability technique.

The elegance of the program lies in the concise input numerical values for the basic design parameters. The expected randomness of the design variables are built into the design process. As opposed to the use of the means and variances only in pseudo-probability design concept, the practical

distributions of the individual design variables are employed for the derivation of their safe values.

Probability distributions of the decision variables are established from the previous works in literature while the mean values are selected in accordance with the BS8110 (1985) requirements.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 SLABS

Reinforced concrete slabs are used in floors and walls of buildings and as the deck of bridges. The floor system of a structure can take many forms such as in situ solid slabs, ribbed slabs or precast units. Slabs may span in one direction or in two directions and they may be supported on monolithic concrete beams, steel beams, walls or directly by columns.

Continuous slabs should in principle be designed to withstand the most unfavourable arrangement of loads. Because there are greater opportunities for redistribution of loads in slabs, analysis may however often be simplified by the use of single load case (BS8110, 1985). Values based on the single load case as well as those for two-way spanning slabs are tabulated in BS8110, 1985).

In general, moments and shear forces will be found as for beams. They may also be found from an elastic analysis such as those of Pigeaud and Westergaad, or alternatively by Johansen's yield line method or Hillerborg's strip method provided the ratio between support and span moments are similar to those obtained by the elastic theory.

Although the analysis will be carried out at ultimate limit state it is the serviceability limit state of deflection that will be the important criterion. The serviceability limit state of cracking is controlled by bar spacing rules. The

KASSIM IBRAHIM

span/effective-depth ratio and the cover are important factors in the selection of slab thicknesses.

2.2 WORK DONE ON RELIABILITY APPLICATION IN STRUCTURAL ENGINEERING

Engineering design decisions are surrounded by uncertainties which result from the random nature of loading and stuctural resistance as well as the load and resistance prediction models. The effect of such uncertainties is included in design through the use of safety factors which are based on engineering judgment and previous experience with similar structure.

Due to the fact that safety involves a consideration of random variables and realization of the limitations in design by the deterministic method (Rojiani and Woeste, 1982), it is now generally accepted that the rational approach to the analysis of structural safety is through the use of probablistic models.

The probablistic method of design in structural engineering applications is a concept which is rapidly gaining wide and international acceptance. For example, the use of the Load and Resistance Factor Design (LRFD) for structural steel design in America, Ravindra and Galambos (1978) opined that some adjustments in the load and resistance factor design in steel are necessary using reliability approaches. It was concluded among other criteria that the safety index in steel design can be varied to account for the importance of the

structure. Also, optimal levels of reliability for different types of structures could be obtained from an expected total cost of optimization process. The strength of structural members under various load combinations and variations using probablistic approaches are therefore necessary.

Bell and Elms (1972) gave a theoretical model that describes the behaviour of a reinforced concrete slab throughout the whole of its loading range from initial loading to ultimate collapse. The model was based on the use of anisotropic finite elements and a successive approximation. It synthesizes the section properties of a slab from fundamental properties of concrete and reinforcing steel. They suggested that a more complex investigation into the effect of physical parameters such as reinforcement distributions, moments, deflections, shear strength and cracking to mention but a few be carried out so as to ascertain the intrinsic ultimate strength of a reinforced concrete slab.

Muspratt (1972) demonstrated the sensitivity of an optimized slab to load distribution using direct Monte-Carlo simulation. He concluded that optimization does offer some savings of reinforcement and concrete in reinforced concrete slabs when adequate design controls and construction patterns or procedures are followed.

Weissman et al (1994) described a reliability model that predicts the probability that a continously reinforced concrete pavement section with a certain amount of distress manifestation will last at least a certain number of

equivalent-single-axle-load applications. It was opined that the model is accurately predicting local conditions not replicated in the particular area. They stated further that the model prediction will minimize material, maintenance and management costs including failure of the pavement.

Abejide (1996) carried out a reliability analysis of simply supported, singly reinforced solid slabs. He concluded the application of reliability to design criteria of such slab will ensure minimum investment and failure of such reinforced concrete members.

2.3 WORK DONE ON COMPUTER APPLICATION IN STRUCTURAL ENGINEERING

The main purpose of application of computer-aided designs in structural optimization is to shorten calculations and processing time (Jozwiak, 1986).

Falk (1985) asserted that in addition to rapid calculation, the packages should provide user with planned and organized solution procedures, and compel them to follow those procedures. He developed EASI for slab design to AISC requirements, ST19M for concrete slab design to ACI code; among other packages (Falk, 1985).

Many more packages have been developed for the design of reinforced concrete elements that are mainly computer implimentations of manual design procedures. Abejide (1994) developed a computer program KOM22 for the design of continuous pre-stressed concrete beams.

Saouma et al (1985) developed an interactive graphics program for the analysis and design of reinforced concrete frame buildings in accordance with the ACI 318-83. The pre-processor, processor (analysis and design), and post-processor were individually presented. Hardware, graphical devices and portability of the program were also discussed. The first version of the program was shown to be satisfactory for initial preliminary design.

For the purpose of this work, a FORTRAN-77 based modules were used to develop the reliability-based design program for various types of slabs. The modules are based on the BS8110 (1985) design procedures and the concept of the first-order reliability analysis. The module for the reliablity processing is based on the FORM5 algorithm (FORM, 1988).

CHAPTER THREE

3.0 ANALYTICAL TECHNIQUES

3.1 ANALYSIS AND DESIGN OF SLABS (BS8110, 1985)

3.1.1 Moment and Shear Forces (BS8110: Clause 3.5)

In the current British design practice, the moments and shear forces on slabs are usually determined by elastic analysis. Moments and shear forces for one-way continuous slabs may be determined as for beams (tabulated in BS8110, 1985).

For more complicated cases, an elastic analysis may be carried out using the finite difference or finite element methods as described by Instituion of Structural Engineers Manual (Kong and Evans, (1989)). BS8110 (1985) permits the use of the powerful yield line method and Hillerborg's strip method. However, these two methods provide no information on deflection and cracking. BS8110: clause 3.5.2.1. requires that, where these methods are used, the ratios between support and span moments should be similar to those obtained by elastic analysis.

3.1.2 Limit State of Deflection (BS8110:Clause 3. 5. 7)

In design, deflection is usually controlled by limiting the ratio of the span to the effective depth. The following steps are usually followed to check the deflection requirements.

1. Select the basic span/depth ratio from table 3.10 of

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the code, namely, 7 for cantilever slabs, 20 for simply supproted slabs and 26 for continuous slabs.

2. The basic span/depth ratio is now multiplied by a modification factor obtained from Table 3.11 of the code, to allow for the effect of the tension reinforcement.

3. The product obtained from (2) above can be multiplied where necessary by a modification factor, obtained from Table 3.12 to allow for the effect of the compression reinforcement.

With respect to BS8110 (1985) deflection requirements are met when the actual span/effective depth ratio does not exceed that obtained from step 3 above.

3.1.3 Limit State of Cracking (BS8110:Clause 3.5.8)

In design, crack widths are usually controlled by limiting the spacing of the reinforcement. The requirement is that the clear spacing between bars should not exceed 3d or 750mm whichever is lesser, where d is the effective depth of the slab. This requirement applies to both main and distribution bars. However, the Institution of Structural Engineers Manual (Kong and Evans, 1989) gives separate but more restrictive, recommendation for main bars and distribution bars: main bar spacings should not exceed 3d or 300mm; distribution bar spacings should not exceed 3d or 400mm.

Having checked that the above requirement is met, no further check on bar spacing is necessary, provided that at

least one of the three conditions below is met:

1. for fy = 250N/mm^2 : h < 250mm (where h is the overall depth of the slab);
2. for fy = 460N/mm^2 : h < 200mm;
3. the steel ratio ρ (As/bd) < 0.3%; (As = min recommended area).

If none of the above three conditions is met, then the main bar spacing should be limited as given in clause 3.12.11.2.3 of the BS8110.

3.1.4 Minimum area of reinforcement (BS8110: Clause 3.12.5.3 and 3.12.11.2.9)

The area of reinforcement in each direction should not be less than:

a) 0.13% of bh for fy = 460N/mm^2

b) 0.24% of bh for fy = 250N/mm^2

Where the control of shrinkage and temperature cracking is important, the minimum areas in (a) and (b) above should be increased to 0.25% and 0.3% respectively.

3.1.5 Concrete cover for durability (BS8110: Clause 3.3)

Table 3.4 in BS8110 (1985) gives the nominal covers to meet the durablilty requirements for slabs and other structural members. Nominal cover is as defined in BS8110: Clause 3.3.1.1, namely; the nominal cover is the design depth of concrete to all reinforcement, including links.

3.1.6 Fire Resistance (BS8110: Clause 3.3.6)

Simplified rules for curtailment of bars for simply supported slabs, cantilever slabs and continuous slabs may be used provided that:

i) the loading is predominantly uniformly distributed;

ii) for continuous slabs, the spans do not differ by more than 15% of the longest span.

3.2 SAFETY INDEX CONCEPT

In order to explicitly accommodate uncertainties in reliability, the strength capacity (R) and the loading effect (S) of a structural system or component are considered as random variables. The main objective of reliability analysis of any system or component is to ensure that R is never exceeded by S (R > S). Conversely, the probability of the complimentary event (R < S) is the measure of failure.

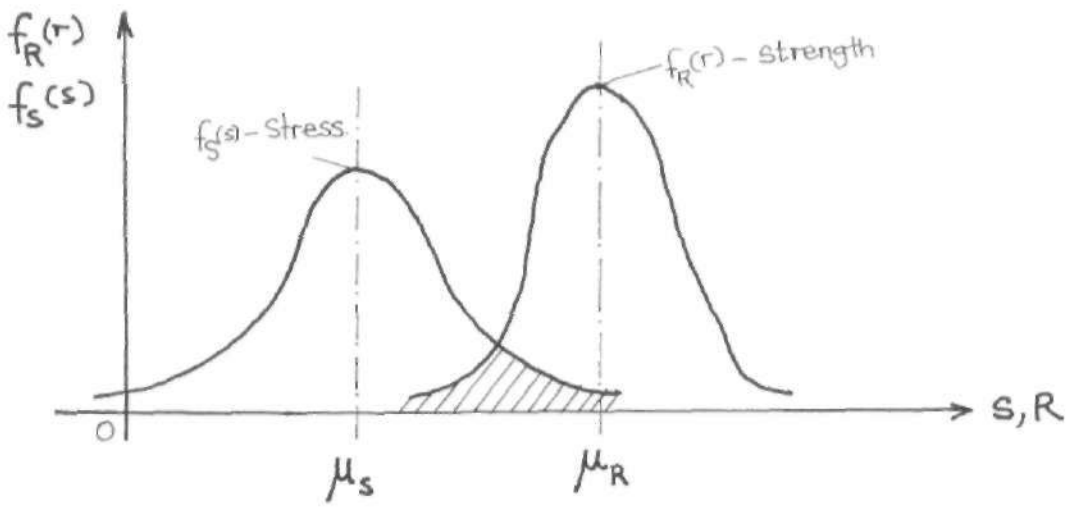


Fig. 3.1: Basic strength capacity-load effect problem.

For a simple structural member with known distribution functions for R and S in some specified mode of failure, the probability of failure, p_f is given as

$$p_f = \int_0^\infty F_R(s)\, f_S(s)\, ds \qquad (3.1)$$

where $F_R(s)$ is the cummulative distribution function for R and $f_S(s)$ is the probability density function of S. Equation (3.1) is considered valid when R and S are statistically independent and is best understood by plotting the density functions of R and S as shown in Fig. 3.1.

It should be emphasized that R and S must be expressed in the same dimension (e.g., loads or bending moments). R and S can also be related in terms of safety margin, M, as

$$M = R - S \qquad (3.2)$$

considering an ideal situation, if R and S are statistically independent and normally distributed, p_f may be expressed as

$$p_f = P(M \leq 0) \qquad (3.3)$$

Thus

$$\mu_M = E[M] = \mu_R - \mu_S \qquad (3.4)$$

and

$$\sigma_M^2 = Var[M] = \sigma_R^2 + \sigma_S^2 \qquad (3.5)$$

Since R and S are normally distributed, M, a linear function of R and S would also be normally distributed, and $(M-\mu_M)/\sigma_M$ is a unit standard normal, giving:

$$p_f = \Phi\left(\frac{0-\mu_M}{\sigma_M}\right) = \Phi\left(\frac{\mu_S-\mu_R}{\sqrt{\sigma_S^2+\sigma_R^2}}\right) \qquad (3.6)$$

where Φ is the standard normal distribution function and μ_S,

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μ_R, σ_S and σ_R are respectively the means and standard deviations of the random variables S and R. In other words,

$$p_f = \Phi(-\mu_M/\sigma_M) = \Phi(-\beta) \qquad (3.7)$$

where β is the reliability index or safety index and may be defined as the number of standard deviations by which μ_M exceeds zero.

According to Thoft-Christensen and Baker (1982), for a more general case where R and S are jointly normally distributed with correlation coefficient, ρ, in equation (3.7) still holds but σ_M is given as

$$\sigma_M = (\sigma_R^2+\sigma_S^2+2\rho_o\sigma_R\sigma_S)^{1/2} \qquad (3.8)$$

If R and S are both lognormally distributed, p_f may be expressed as

$$p_f = \Phi\left[\frac{\ln\left(\frac{\mu_S}{\mu_R}\sqrt{\frac{V_R^2+1}{V_S^2+1}}\right)}{\sqrt{\ln[(V_R^2+1)(V_S^2+1)]}}\right] \qquad (3.9)$$

where V is coefficient of variation.

Convenient analytical expressions for p_f do exist for some other combinations of distributions of R and S. However, this simple representation of the problem is rarely of much practical use in structural reliability analysis. In general equation (3.1) needs to be evaluated by numerical methods.

3.3 SECOND-MOMENT FORMULATION

The computation of p_f requires the knowledge of the distribution of $F_R(r)$ and $f_S(s)$. In practice, data are

insufficient or not available. Even when the distributions are prescribed, the evaluation of the probabilities, generally requiring numerical integration of equation (3.1), may be impracticable. Most often, information are limited to the mean and variance of the respective random variables. As a result, the implementation of reliability concept is restricted to the first and the second moments.

The boundary separating the safe and failure state is the limit state function defined by

$$M = R - S = 0 \tag{3.10}$$

Introducing reduced variate $R' = (R - \mu_R)/\sigma_R$ then

$$R = \sigma_R R' + \mu_R \tag{3.11}$$

Similarly,

$$S = \sigma_S S' + \mu_S \tag{3.12}$$

Substituting equation (3.11) and (3.12) into the limit state equation (3.10) gives

$$\sigma_R R' - \sigma_S S' + \mu_R - \mu_S = 0 \tag{3.13}$$

Equation (3.13) is a linear equation of the reduced variables and the graphical representation is shown in Fig. 3.2.

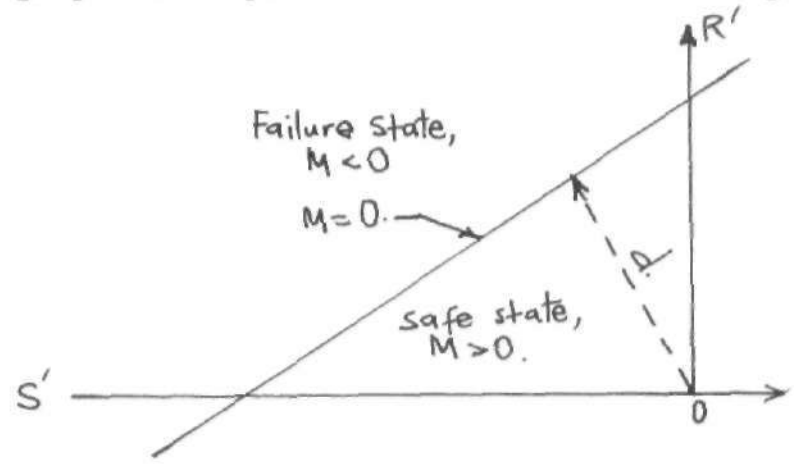


Fig. 3.2: Limit state boundary.

The straight line graph is the boundary between the safe region and the failure state. The line represents a limit state function. Representing equation (3.13) in the cartesian coordinate system gives

$$y = \frac{\sigma_R}{\sigma_S} X + \frac{(\mu_R - \mu_S)}{\sigma_S} \tag{3.14}$$

The perpendicular to line M = 0 and passing through the origin is

$$y = \frac{-\sigma_S}{\sigma_R} X \tag{3.15}$$

Solving for x and y from equations (3.14) and (3.15)

$$X = \frac{\sigma_R (\mu_R - \mu_S)}{(\sigma_R^2 + \sigma_S^2)} \tag{3.16}$$

and

$$y = -\sigma_S \frac{(\mu_R - \mu_S)}{(\sigma_R^2 + \sigma_S^2)} \tag{3.17}$$

therefore the line distance d, from the origin to M = 0 in the general form is

$$d = \sqrt{(x-0)^2 + (y-0)^2} = \sqrt{x^2 + y^2} \tag{3.18}$$

Substituting equations (3.16) and (3.17) in equation (3.18)

$$d = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} = \frac{\mu_m}{\sigma_m} = \beta \tag{3.19}$$

3.4 GENERALIZATION AS A FUNCTION OF BASIC VARIABLES

The variables R and S are functions made up of different basic variables. In order to investigate the effect of the variables on the performance of a structural system, a limit state equation in the terms of the basic design variable is required. This limit state equation is referred to as the performance or state function and expressed as

$$g(x_i) = g(x_1, x_2, \ldots, x_n) = R - S \qquad (3.20)$$

where X_i for $i = 1,2,\ldots, n$ represent the basic design variables. The limit state of the system can then be expressed as

$$g(x_i) = 0 \qquad (3.21)$$

Graphically, the line $g(x_i) = 0$ represents the failure surface while $g(x_i) > 0$ represents the safe region and $g(x_i) < 0$ corresponds to the failure region. Introducing the set of uncorrelated reduced variates.

$$x'_i = \frac{(X_i - \mu_{x_i})}{\sigma_{x_i}}, i = 1,2,\ldots.n, \qquad (3.22)$$

The limit state equation is given as

$$g(\sigma_{x1}X'_1 + \mu_{x1}, \sigma_{x2}X'_2 + \mu_{x2}, \ldots, \sigma_{xn}X'_n + \mu_{xn}) = 0 \qquad (3.23)$$

Similarly, just as in the case of equation (3.18), the distance from a point $X'_i = (X'_1, X'_2, \ldots, X'_n)$ on the failure surface $g(x'_i) = 0$ to the origin of X_i space is given as

$$D = \sqrt{X'^2_1 + X'^2_2 + \ldots + X'^2_n} \qquad (3.24)$$

In matrix form:

$$D = (X'_1 X'_2 \ldots X'_n) \begin{vmatrix} X'_1 \\ X'_2 \\ \cdot \\ \cdot \\ \cdot \\ X'_n \end{vmatrix} = (X'^{t}_{i} X)^{1/2} \qquad (3.25)$$

The point on the failure surface ($X'_1{}^*$, $X'_2{}^*$, ..., $X'_n{}^*$), having the minimum distance to the origin may be determined by minimizing the function D and subjecting equation (3.24) to the constraint $g(X_i) = 0$. For this purpose, the method of langrange's multiplier may be used. Let

$$L = D + \lambda g(X_i) \qquad (3.26)$$

Substituting equation (3.25) in (3.26) gives

$$L = (X'^{t}_{i} X'_i)^{1/2} + \lambda g(X_i) \qquad (3.27)$$

where λ is the value of the multiplier.

In scalar notation

$$L = \sqrt{X'^2_1 + X'^2_2 + \ldots + X'^2_n} + \lambda g(x_1, x_2, \ldots, x_n) \qquad (3.28)$$

in which $X_i = \sigma_{xi} X'_i + \mu_{xi}$

Minimizing L, we obtain n+1 equations with n+1 unknown as

$$\frac{\partial L}{\partial x_i} = \frac{X'_i}{\sqrt{X'^2_1 + X'^2_2 + \ldots + X'^2_n}} + + \frac{\lambda \partial g}{\partial X'_i} = 0 \qquad (3.29)$$

and

$$\frac{\partial L}{\partial \lambda} = g(X_1, X_2, \ldots, X_n) = 0 \qquad (3.30)$$

The solution to equations (3.29) and (3.30) would yield the

most probable failure point (X'_1^*, X'_2^*, ..., X'_n^*). Introducing the gradient vector,

$$G=\frac{\partial g}{\partial X'_1},\frac{\partial g}{\partial X'_2},\ldots,\frac{\partial g}{\partial X'_n} \tag{3.31}$$

in which

$$\frac{\partial g}{\partial X'_i}=\frac{\partial g}{\partial X_i}\cdot\frac{\partial X_i}{\partial X'_i}=\sigma_{X_i}\frac{\partial g}{\partial X_i} \tag{3.32}$$

therefore, in vector form we have

$$\frac{\boldsymbol{X}'}{(\boldsymbol{X}'^{t}\boldsymbol{X}')^{1/2}}+\lambda G=0 \tag{3.33}$$

from which

$$\boldsymbol{X}'=\lambda DG \tag{3.34}$$

From equation (3.25)

$$D=[(\lambda DG^{t})(\lambda DG)]^{1/2}=\lambda D(G^{t}G)^{1/2} \tag{3.35}$$

and

$$\lambda=(G^{t}G)^{-1/2} \tag{3.36}$$

Substituting equation (3.36) into equation (3.34) gives

$$\boldsymbol{X}'=\frac{-GD}{(G^{t}G)^{1/2}} \tag{3.37}$$

Multiplying both sides of equation (3.37) by G^t we have

$$G^{t}\boldsymbol{X}'=\frac{-G^{t}GD}{(G^{t}G)^{1/2}}=-(G^{t}G)^{1/2}D$$

implying

$$D = \frac{-G^{t} X'}{(G^{t} G)^{1/2}} \tag{3.38}$$

From equation (3.19), the minimum distance from the origin to the line representing the failure surface equals β and therefore equation (3.38) becomes

$$\beta = \frac{-G^{*t} X'^{*}}{(G^{*t} G^{*})^{1/2}} \tag{3.39}$$

where G* is the gradient vector at the most probable failure point ($X'_1{}^{*}$, $X'_2{}^{*}$, ..., $X'_n{}^{*}$). In scalar form equation (3.39) is

$$\beta = \frac{\sum_i X'^{*}_i \left(\frac{\partial g}{\partial X'_i}\right)_{*}}{\sqrt{\sum_i \left(\frac{\partial g}{\partial X'_i}\right)^{2}_{*}}} \tag{3.40}$$

where the derivatives are performed at ($X'_1{}^{*}$, $X'_2{}^{*}$, ..., $X'_n{}^{*}$) and the most probable point on the failure surface for minimum β from equation (3.37),

$$X'^{*} = \frac{-G^{*} \beta}{(G^{*t} G^{*})^{1/2}} \tag{3.41}$$

and in the scalar form is

$$X'^{*}_i = \alpha^{*}_i \beta \quad ; i = 1, 2, \ldots, n \tag{3.42}$$

where

$$\alpha_i = \frac{\left(\frac{\partial g}{\partial X'_i}\right)_{*}}{\sqrt{\sum_i \left(\frac{\partial g}{\partial X'_i}\right)^{2}_{*}}}$$

are direction cosines along the axes X'_i

3.4.1 First Order Interpretation:

Considering the generalized state function, g(x) expanded in a Taylor series at a point X^* which is on the failure surface $g(x^*) = 0$ gives

$$g(X_1, X_2, \ldots, X_n) = g(X_1^*, X_2^*, \ldots, X_n^*) + \sum_{i=1}^{n} (X_1 - X_i^*)\left(\frac{\partial g}{\partial x_i}\right)_* + \frac{\sum_{i=1}^{N}\sum_{j=1}^{n} (X_i - X_i^*)(X_j - X_j^*)}{\left(\frac{\partial^2 g}{\partial x_i \partial x_j}\right)_*} + \ldots \quad (3.43)$$

where the derivatives are evaluated at $(X_1^*, X_2^*, \ldots, X_n^*)$. On the failure surface, $g(X_1^*, X_2^*, \ldots, X_n^*) = 0$, hence

$$g(X_1, X_2, \ldots, X_n) = \sum_{i=1}^{n} (X_i - X_i^*)\left(\frac{\partial g}{\partial X_i}\right)_* + \frac{\sum_{i=1}^{n}\sum (X_i - X_i^*)(X_j - X_j^*)}{\left(\frac{\partial^2 g}{\partial X_i \partial X_j}\right)_*} + \ldots \quad (3.44)$$

Introducing the concept of reduced variate we have

$$X_i - X_i^* = (\sigma_{X_i} X_i + \mu_{X_i}) - (\sigma_{X_i} X_i^{1*} + \mu_{X_i}) = \sigma_{X_i}(X'_i - X'^*_i)$$

and

Truncating equation (3.44) at the first order linear term and substituting the reduced variate into it gives

$$\frac{\partial g}{\partial x_i} = \frac{\partial g}{\partial X'_i}\left(\frac{\partial X_i}{\partial X_i}\right) = \frac{1}{\sigma_{x_i}}\left(\frac{\partial g}{\partial X'_i}\right)$$

$$g(X_i) = \sum_{i=1}^{n} (X'_i - X'^*_i)\left(\frac{\partial g}{\partial X'_i}\right)_* \qquad (3.45)$$

Applying the properties of expectation and variance, the mean value of equation (3.45) is

$$\mu_g \approx -\sum_{i=1}^{n} X'^*_i\left(\frac{\partial g}{\partial X'_i}\right)_* \qquad (3.46)$$

and the corresponding variance is

$$\sigma_g^2 \approx \sum_{i=1}^{n} \sigma^2_{x'_i}\left(\frac{\partial g}{\partial X'_i}\right)_*^2 = \sum_{i=1}^{n}\left(\frac{\partial g}{\partial X'_i}\right)_*^2 \qquad (3.47)$$

Thus, the ratio

$$\frac{\mu_g}{\sigma_g} = \frac{\sum_{i=1}^{n} X'^*_i\left(\frac{\partial g}{\partial X'_i}\right)_*}{\sqrt{\sum_{i=1}^{n}\left(\frac{\partial g}{\partial X'_i}\right)_*^2}} \qquad (3.48)$$

Equation (3.48) conforms to equation (3.40), therefore

$$\beta = \mu_g/\sigma_g \qquad (3.49)$$

3.5 TRANSFORMATION OF BASIC VARIABLES

The prescribed probability distributions of the basic variables may be non-normal and correlation between their pairs may exist. It is necessary therefore to highlight practicable methods of their transformation into forms

suitable for the estimation of the safety index and the evaluation of equation (3.7).

3.5.1 Non-Normal Variates

In a situation where the original PDF, $f_{x_i}(X_i)$ and CDF, $F_{x_i}(X_i)$ are not normal, it is necessary to transform the distribution into an equivalent normal distribution at design point (X_i^*).

Equating the CDF with the standard cumulative density function gives:

$$\Phi\left[\frac{X_i^* - \mu_{X_i}^N}{\sigma_{X_i}^N}\right] = F_{x_i}(x_i) \qquad (3.50)$$

where $\mu_{x_i}^N$, $\sigma_{x_i}^N$ are respectively the mean and standard deviation of equivalent normal distribution of the basic variable X_i.

Thus, the equivalent mean is

$$\mu_{x_i}^N = X_i^* - \sigma_{x_i}^N \Phi^{-1}[F_{x_i}(X_{x_i}^*)] \qquad (3.51)$$

where as equating the probability densities gives

$$\frac{1}{\sigma^N{}_{x_i}}\varphi\left(\frac{X_i^* - \mu_{x_i}^N}{\sigma_{x_i}^N}\right) = f_{x_i}(X_i^*) \qquad (3.52)$$

Substituting equation (3.51) into equation (3.52)

$$\frac{1}{\sigma_{x_i}^N}\varphi\left(\frac{X_i^* - X_i^* + \sigma_{x_i}^N \Phi^{-1}(F_{x_i}(X_i)}{\sigma_{x_i}^N}\right) = f_{x_i}(X_i^*)$$

This implies that

$$\sigma_{X_i}^N = \frac{\varphi\left(\Phi^{-1}\left[F_{X_i}(X_i^*)\right]\right)}{f_{X_i}(X_i^*)} \tag{3.53}$$

In the above equations μ^N_{xi}, $\delta_{xi}{}^N$ and the design points X_i^* are unknowns and as a result iterative method is employed until convergence is achieved.

3.5.2 Correlated Variates

For correlated variates, transformation into a set of uncorrelated variate must be carried out. Considering a system which is a function of two variates X_1 and X_2 with expected means $E[X_1] = \mu_{x1}$, and $E[X_2] = \mu_{x2}$. The covariance of X_1 and X_2 is denoted by $cov[X_1, X_2]$ and is defined as

$$\begin{aligned} Cov[X_1, X_2] &= E[(X_1 - \mu_{X_1})(X_2 - \mu_{X_2})] \\ &= E[x_1, x_2] - E[x_1]E[x_2] \end{aligned} \tag{3.54}$$

The correlation coefficient is given by

$$\rho_{X_1X_2} = \frac{Cov[x_1, x_2]}{\sigma_{X_1}\sigma_{X_2}} \tag{3.55}$$

where σ_{x1} and σ_{x2} are standard deviation of X_1 and X_2 respectively. A pair of variates is said to be uncorrelated if

$$Cov[X_1, X_2] = 0 \tag{3.56}$$

and correlation coefficient satisfies the inequality

$$-1 \le \rho_{X_1X_2} \le 1 \tag{3.57}$$

The required transformation is necessarily dependent on the covariance or covariance matrix of the original variate and may be obtained as follows. Suppose the covariance matrix

of the original variate X_1, X_2, ..., X_n is

$$|c| = \begin{vmatrix} \sigma^2_{x_1} & cov[x_1, x_2] & \cdots & cov[x_1, x_n] \\ cov[x_2, x_1] & \sigma^2_{x_2} & \cdots & cov[x_2, x_n] \\ \cdot & \cdot & \cdots & \cdot \\ cov[x_n, x_1] & cov[x_n, x_2] & \cdots & \sigma^2_{x_n} \end{vmatrix} \qquad (3.58)$$

The corresponding covariance between pairs of the reduced variate $\mathbf{X}_i'$ and $\mathbf{X}_j'$ is

$$Cov[x'_i, x'_j] = E[(X'_i - \mu_{x_i})(X'_j - \mu_{x_j})]$$

$$= E\left[\left(\frac{X_i - \mu_{x_i}}{\sigma_{x_i}}\right)\left(\frac{X_j - \mu_{x_j}}{\sigma_{x_j}}\right)\right]$$

$$= \frac{E[(X_i - \mu_{x_i})(X_j - \mu_{x_j})]}{\sigma_{x_i}\sigma_{x_j}} = \rho_{x_i x_j} \qquad (3.59)$$

Equation (3.59) shows that the covariance between a pair of reduced variates $\mathbf{X}'_i$ and $\mathbf{X}'_j$ is equal to the correlation coefficient between the corresponding pair of the original variate X_i and X_j. Therefore, the covariance matrix of the reduced variate ($\mathbf{X}'_1$, $\mathbf{X}'_2$, ..., $\mathbf{X}'_n$) is the corresponding correlation matrix of the original variate (X_1, X_2, ..., X_n). Thus, the covariance matrix of $\mathbf{X}'_1$, $\mathbf{X}'_2$, ..., $\mathbf{X}'_n$ is

KASHIM IBRAHIM

$$|C'| = \begin{vmatrix} 1 & \rho_{12} & \rho_{13} & \cdot & \cdot & \cdot & \rho_{1n} \\ \rho_{21} & 1 & \rho_{23} & \cdot & \cdot & \cdot & \rho_{2n} \\ \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\ \rho_{n1} & \rho_{n2} & \rho_{n3} & \cdot & \cdot & \cdot & 1 \end{vmatrix} \tag{3.60}$$

The required set of (uncorrelated) transformed variates can be obtained from $\mathbf{X}'$ through the following orthogonal transformation.

$$Y = T^t \mathbf{X}' \tag{3.61}$$

in which

$$\mathbf{X}' = \{\mathbf{X}'_1, \mathbf{X}'_2, \ldots, \mathbf{X}'_n\} \tag{3.62a}$$

and

$$Y = \{Y_1, Y_2, \ldots, Y_n\} \tag{3.62b}$$

are the required set of uncorrelated transformed variate. T^t is the transpose of an orthogonal transformation matrix which is composed of the eigenvectors corresponding to the eigenvalues of the correlation matrix [C]. Specifically T is such that

$$T^t [C] T = [\lambda] \tag{3.63}$$

in which [λ] is a diagonal matrix of eigenvalues of [C]. It may be emphasized that the matrix [C'] is real and symmetric as $\rho_{ij} = \rho_{ji}$. With the orthogonal transformation, the safety index becomes.

$$\beta = \frac{G^{*t} X^*}{(G^{*t} [C'] G^*)^{1/2}} \tag{3.64}$$

In brief, when the basic variables X_i are correlated, the first step is to obtain a set of uncorrelated variables Y_i. The next

step is to normalize the uncorrelated variables and thereby a set of normalised and uncorrelated variables $Z_i = (Z_1, Z_2, \ldots, Z_n)$ is obtained. Finally the safety index can be defined in the Z coordinate system as previously explained. Fig. 3.3 shows a flow chart of the numerical algorithm for the computation of β.

3.6 FORM5

FORM5 is a computer package which uses the algorithm depicted in Fig. 3.3 and is used in this study to compute the implied safety levels for the different limit states, outlining the design criteria for reinforced concrete slabs using BS8110 (1985).

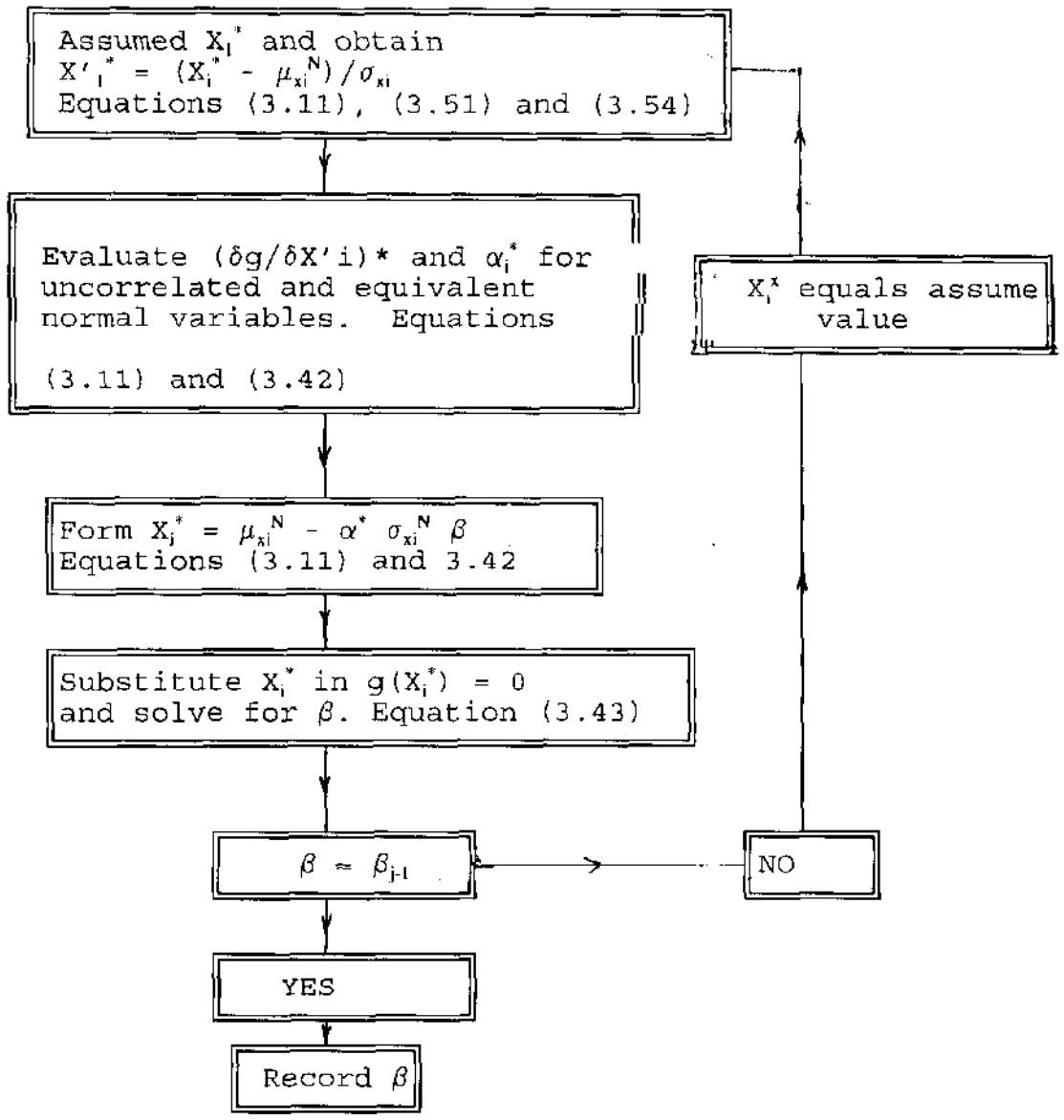


Fig. 3.3 Flow chart for the computation of β

FORM5 requires the user to specify the input parameters of the limit state function and writing a subroutine on the stochastic models of the distributions of the various design parameters.

3.7 DESIGN CRITERION

As proposed in this study, a design is considered satisfactory if the following condition is satisfied:

$$(\beta-\beta_c)^2 \leq \epsilon \qquad (3.65)$$

where β= calculated safety index obtained from the reliability program on the basis of the input variables, β_T=Target safety index and ϵ is the level of acceptance of the design points.

If the above condition does not hold, the design of the individual slab is repeated until it is satified. When the condition is satisfied for varying values of the design variables,the parameters so obtained are considered to provide uniform reliability level.

CHAPTER FOUR

PROGRAM DEVELOPMENT

4.1 PROGRAM FLOWCHART

A FORTRAN program has been developed to aid the design of reinforced concrete slabs to BS8110 (1985) at a constant reliability level.

The flowchart of the program is as shown in figure 4.1. The program starts by reading the values of live load, diameter of steel to be used in the design, the nominal cover, and characteristic strength of concrete and steel. These values are read from a data file.

The program user has to inform the computer the values of the proposed overall depth of the slab, the load ratio as well as the span of the slab (the shorter span). Also to be fed into the computer are the support conditions of the slab. In other word, the program asks the user to state whether the slab is simply supported, cantilevered, propped cantilevered or fixed-fixed ended slab.

The program then asks the user to give the smaller and larger dimensions of the panel to be designed. Hence the panel can be designed as a one-way or a two-way panel.

If the design satisfies the ultimate and serviceability limit states, reliability analysis of the designed section is carried out by the program. A design is finally acceptable if the calculated safety index is approximately equal to the

target safety index. The design procedure is repeated by the program until equation (3.65) is satisfied.

The program terminates after a list of a suitable slab section or after an advice for trial of another section has been given.

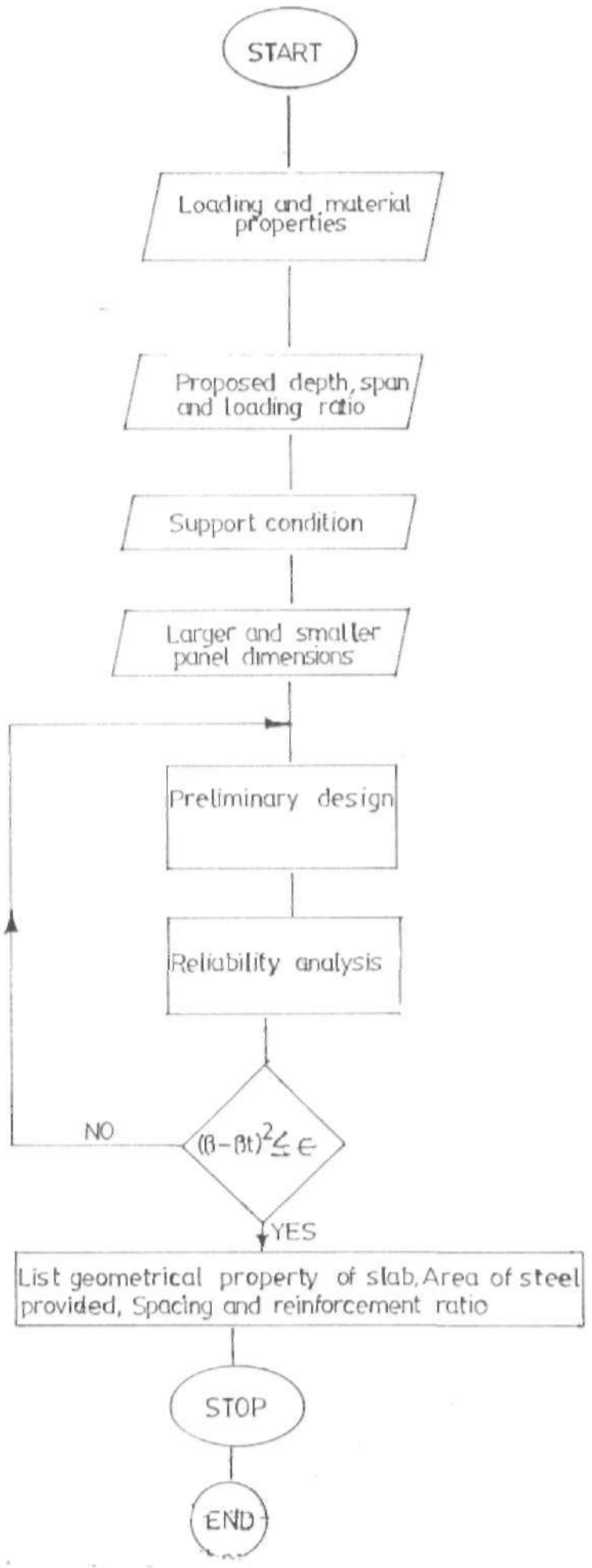


Figure 4.1 Program Flow Chart

4.2 PROGRAM LISTING WITH EXPLANATION OF EACH SEGMENT.

The type of parameters used in the program are:

INTEGER (I) or REAL (R) or CHARACTER (C) parameters (P) according to FORTRAN -77 Convention

ASP	-	I-P	Area of steel provided
ASR	-	I-P	Area of steel required
BARSP	-	I-P	Spacing of bars
NUM	-	I-P	Number according to support conditions
DATANUM	-	I-P	Number of data set in the data set file
RESPONSE	-	C-P	Response whether Yes or No
ALPHA	-	R-P	Load ratio
AK	-	R-P	a margin for the design
FORM5	-		a sub program
FX	-	R-P	Limit state
IER	-	I-P	Error flag
EX	-	I-P	Mean Values
QK	-	R-P	Live load
DIA	-	I-P	Diameter of bar
FCU	-	I-P	Characteristic strength of concrete
FY	-	I-P	Characteristic strength of steel
H	-	I-P	Depth of section
SS	-	C-P	Simply supported
CL	-	C-P	Cantilevered
PC	-	C-P	Propped cantilevered
FF	-	C-P	Fixed-fixed
WF	-	R-P	Weight of finishes
GK	-	R-P	Dead load

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B	-	I-P	Width of slab
DL	-	R-P	Design load
AM	-	R-P	Applied Moment
ED	-	I-P	Effective Depth
RAT1	-	R-P	Ratio
Z	-	R-P	Lever Arm
DIAA	-	I-P	Diameter of bar for iterative purposes
ASPP	-	I-P	Area of steel provided for iterative purposes
BARSPP	-	I-P	Spacing bars for iterative purposes
ASH	-	R-P	Actual shear stress
GAM	-	R-P	A Coefficient
CSH	-	R-P	limiting shear stress
RAT2	-	R-P	Ratio
FS	-	R-P	Service stress of steel
TRMF	-	R-P	Tension Reinforcement Modification factor
ALLD	-	R-P	Allowable deflection
ACLD	-	R-P	Actual deflection
SPB	-	I-P	Actual spacing between Bars
ALSP	-	(R-P)	Allowable spacing between Bars
TARG	-	R-P	Target Safety index
BETA	-	R-P	Safety index
Y/N	-	C-P	Yes or No respectively
K	-	I_P	Counter
IP	-	"	"
IB	-	"	Spacing between bars
IBB	-	"	Spacing between bars for iterative

purposes

IP - " Area of reinforcement provided

IPP - " Area of reinforcement provided for iterative purposes

ID - " Diameter of bars

4.3 PROGRAM LISTING FOR PARAMETERS OF FORM5 AS USED IN THE PROGRAM

There are few subroutines in FORM5 which can be called directly. These are as follows:

1. YINIT (N, IV, VP, IRHO, COV, NC)

This is an initialisation - subroutine. It gives default values to the arrays VP, IV, Cov.

2. YKOPF (NAUS, N, IV, EX, SX, VP, IRHO) YKOPF produces an output for the stochastic model. It prints variable number, the type of distribution, the mean value, the standard deviation, the coefficient of variation, the repetition number, the occurence probability and the value of IRHO.

3. YFAUS (NAUS, N, F, "String") This subroutine prints the vector F of dimension N in the output medium NAUS with the title "String". String is a CHARACTER variable with 10 (mandatory) characters.

4. YMAUS (NAUS, N1, N2, F, "String") This subroutine prints the matrix F of dimension (N1, N2) to the output medium NAUS. N1 is the column dimension of F. N2 ≤ N1 is the number of rows and columns to be printed.

"String" is a CHARACTER variable with 10 (mandatory!) characters.

The parameter list of FORM5 (N, IV, EX, SX, VP, GG, IRHO, COV, NC, EIVEC, NE, V1, NAUS, BETA, X, U, ZES, IER).

N:<INTEGER> number of uncertain variables and dimension of the vectors EX, SX, X and U. The maximum number N in this version of FORMS is N=60.

IV:<INTEGER> matrix of dimension (2,N) whose first element in the i-th column assigns the probability distribution type J to the variable X (i) and whose second element defines the type of input for the distribution parameters.

EX:<REAL*8> Vector of mean values of uncertain variables.

SX:<REAL*8> Vector of standard deviations of uncertain variables.

VP:<REAL*8> Matrix with 10 rows and N columns. The first two rows will be set by FORM5 except for user defined distributions. The third and fourth rows have to be specified by the user in some cases. Rows 5 to 8 must always be specified by the user. Note: IV(2,N) and VP(10,N) may be preset using YINIT.

GG name of the user defined subroutine containing the mechanical model (state function). This subroutine has to be declared as EXTERNAL in the main program calling FORM5.

IRHO:<INTEGER> control parameter for correlation

IRHO=0: no correlations between X-variables

IRHO=1: correlations permitted

COV:<REAL*8> matrix with NC rows and NC columns

For IRHO=1 the user has to give the correlation coefficients in this matrix. Only the lower triangle is used. For IRHO=0 COV will not be used.

NC :<INTEGER> dimension of the matrix COV.

For IRHO=0 NC must be greater than or equal to one.

for IRHO=1, NC must be greater than or equal to N

EIVEC:<REAL*8> Matrix with NE rows and NE columns, an auxiliary matrix to store the Eigenvectors which are on EIVEC on output of FORM5 for IRHO=1(case of correlations).

NE :<INTEGER> dimension of EIVEC

For IRHO=0 NE must be ≥ 1

For IRHO=1 it must hold NE ≥ N.

V1:<REAL*8> control parameter for iteration. usually it suffices to set V1=1.0. In case of non-convergence for V1 = 1 try V1 < 1.0.

NAUS:<INTEGER> Print control - and unit number parameter:

For NAUS ≥ 99 OR naus <0 all prints suppressed

For 0 ≤ NAUS <98 PRINT TO THE UNIT #NAUS

BETA:<REAL*8> on input: For BETA=0 the starting solution on X is set by FORM5 to X(i) = EX(i).

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For BETA > 0 a starting solution is expected on X(i), i=N. on Output: safety index (= //u-*//)

X :<REAL*8> vector of dimension N

on Input: Possibly initial solution vector

on Output: Coordinates of the checking point in X-space

U :<REAL*8> vector of dimension N

on Output: Coordinates of the checking point in X-space

ZES :<REAL*8> vector of dimension 3. Contains the values:

ZES(1)= value of the uncertain variable Z

ZES(2)= mean of Z

ZES(3)= standard deviation of Z

Z is the linearisation of g(X) at the checking point

IER :<INTEGER> error indicator

IER= 0 no error

IER> 0 six digit number on output. The first three digits are used to locate the error and its cause. The last three digits are used to pass a value IER not equal to zero set by the user within the state subroutine in case of an error

4.4 THE PROGRAM

The design program is as follows:

```
                    PROGRAM THESIS
C   PROGRAM IS FOR THE DESIGN OF DIFFERENT TYPES OF SLABS
C    IN WHICH SUPPORT CONDITIONS IS THE MAJOR DIFFERENCE.
     IMPLICIT DOUBLE PRECISION (A-H,O-Z)
     INTEGER DIA,ASP,BARSP
     integer diaa,aspp,barspp
     INTEGER SPAN,NUM,DATANUM
     CHARACTER Response
     REAL Alpha,ak
     COMMON FORM5,ICRT
     COMMON XX(5),FX,IER
     common/gidris/span,ALPHA,num,beta,rho
     common ex(5)
     OPEN(Unit=1,File='Dataset',Status='old')
     OPEN(Unit=2,File='My.Res',Status='new')
     read(1,*) datanum
     do 1001 II= 1,DATANUM
     READ(1,*,END=1001) wf,qk,dia,cover,fcu,fy
     WRITE(2,25)
25   FORMAT(X,'Reliability-Based Slab Design to
    + BS8110 By Idris Abubakar,(M.Sc.Thesis,1998)')
12      WRITE(*,*) 'GIVE THE VALUEs OF H, span, alpha '
     READ (*,*) H,span,alpha
     write(2,*) h,span,alpha
     write(2,*) dia
```

```
15      WRITE (*,11)
11      format(5X, 'ENTER:'/,
     +    3X,'1..... FOR SS (SIMPLY SUPPORTED)'/,
     +    3X,'2..... FOR CL (CANTILEVERED SUPPORT)'/,
     +    3X,'3..... FOR PC (PROPPED CANTILEVERED SUPPORT)'/,
     +    3X,'4..... FOR FF (FIXED-FIXED SUPPORT)'/)
     WRITE(*,*)  'NUM'
     READ(*,*) NUM
     write(2,*) 'num=',num
     if (num.le.4) goto 10
     if (num.gt.4.) write(*,51)
51      format(5x, 'number is not within range',//)
          goto 15
C       CALCULATION OF LOADINGS
10      GK=24.0*H*1.D-3+WF
     B=1000.
     DL=1.4*GK+1.6*QK
C       ULTIMATE BENDING MOMENT
     IF (NUM .EQ.1) THEN
          SAM=DL*(span**2)/8.0
      ELSEIF (NUM .EQ.2) THEN
          SAM=DL*(span**2)/2.0
          ELSEIF (NUM .EQ.3) THEN
           SAM=DL*(span**2)/8.0
          ELSEif(num.eq.4)then
                SAM=DL*(span**2)/12.0
     ENDIF
```

```
      am= sam
      ED=H-(DIA)/2.0-COVER
      RAT1=AM/(B*(ED**2)*FCU)
          Z=ED*(0.5+SQRT(0.25-RAT1/0.9))
      ASR=AM/(0.87*FY*Z)
      IASR = Ifix(ASR)
      CALL BARSPC (DIA,IASR,ASP,BARSP)
      WRITE(2,*) DIA,BARSP,ASP
      call barspcc (dia,iasr,diaa,aspp,barspp)
      write(2,*) diaa,barspp,aspp
      IF(ASPP.LE.ASP.and.aspp.gt.asr) THEN
           ASP=ASPP
           BARSP=BARSPP
           dia=diaa
      ELSEIF (ASP.LE.ASPP.and.asp.gt.asr) THEN
           ASP=ASP
           BARSP=BARSP
           dia=dia
      ENDIF
C        CHECKS FOR SHEAR
      IF (NUM .EQ.1) THEN
         ASH= 0.5*DL*span/(ED*B)
           ELSE IF (NUM .EQ.2) THEN
             ASH = DL*span/(ED*B)
            ELSE IF (NUM .EQ.3) THEN
              ASH = 5.0*DL*span/(8.0*ED*B)
                ELSE IF(NUM .EQ.4) THEN
```

```
                ASH = 2.0*DL*span/(3.0*ED*B)
      END IF
      GAM = 1.25
      CSH =0.79*(100.0*ASP/(B*ED))**(1/3)*(400/ED)**(1/4)/GAM
      IF(ASH .LT.CSH) THEN
      WRITE (2,41)ASR,ASP
41       format(2x,'Area of Steel Required (mm2)= 'F7.2 /,
     +  2x,'Area of Reinforcement Provided (mm2) = 'F7.2 /)
         WRITE(2,*) 'Shear is OK!'
        ELSE
         WRITE(2,*) 'SHEAR IS NOT OK!'
         h=h+5
         dia=12.
         goto 10
      END IF
C        DEFLECTION CHECKS
      RAT2 = AM/(B*(ED**2))
      FS =(5.0/8.0)*FY*(ASR/ASP)*1.0
      TRMF =0.55+(477-FS)/(120.0*(0.9+RAT2))
      IF (TRMF .GT.2.0) THEN
        TRMF=2.0
      END IF
      IF (NUM.EQ.1) THEN
        ALLD = TRMF*20.0
          ELSE IF (NUM .EQ.2) then
           alld=trmf*7.0
          ELSE IF (NUM.EQ.3) THEN
```

```
                ALLD = TRMF*20.0
              else if (NUM.EQ.4) then
               alld=trmf*26.0
          END IF
          ACLD = span/ED
          IF (ACLD .LT.ALLD) THEN
             WRITE(2,16) ACLD,ALLD
    16          format(3x,'Actual Deflection (mm) = 'F5.2 /,
          +       3x,'Allowable Deflection (mm) = 'F5.2 )
             WRITE (2,*) 'L/D IS OK!'
                 ELSE
    c          dia=12
               h=h+5
              goto 10
          END IF
    C        CRACK CHECKS
    19       SPB = BARSP-DIA
          ALSP = 3.0*ED
          write(2,13) spb,alsp
    13     format(3x,'Actual spacing between bars (mm)= ',F7.2 /,
          +   3x,'Allowable Spacing between bars (mm) = 'F7.2 /)
          IF(ALSP. GT. 3.D2) ALSP= 3.D2
          IF (ALSP.gt.SPB) THEN
             WRITE(2,*) 'SPACING IS OK!'
               goto 39
             ELSEIF (Dia.eq.6) THEN
               WRITE (2,*) 'SOLUTION IS NOT FEASIBLE'
```

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```
            dia=10.
            h=h+5
            goto 10
            ELSEIF (Dia.EQ.8.) THEN
            dia=10.
            h=h+5.
            ELSEIF (Dia.EQ.10) THEN
            h=h+5.
            goto 10
            ELSEIF (Dia.EQ.12) THEN
            h=h+5.
            goto 10
          END IF
 39       RHO=ASP/(ED*1.D3)
c         RHO= rho*10.
           ex(1)=fcu
           ex(2)=fy
           ex(3)=ed
           ex(4)=qk
           ex(5)=rho
          write (2,*) ex(1),ex(2),ex(3),ex(4),ex(5)
          n5=5
          CALL KIDRIS(n5,ex,targ)
          beta=targ
          WRITE(*,*) BETA
          write(2,*) 'beta=',beta
          betat=3.39d0
```

```
      ak=(beta-betat)**2
         write(2,*) 'ak=',ak
      IF(ak.LE.0.10) then
      WRITE(*,*) 'Design is o.k.'
      WRITE(2,*) 'Design is o.k.'
            write(*,*) 'results see my.res'
198          write(2,222) h,dia,asp,barsp,fcu
222      format(5x,'depth of section (mm)=',F10.2 /,
     +   5x,'diameter of Reinforcing bar (mm)=',F10.2 /,
     +   5x,'Area of Reinforcement Provided (mm2)=',F10.2 /,
     +     5x,'Spacing Between Bars (mm)=',F10.2 /,
     +      5x,'FCU (N/mm2)=',F10.2/)
            GOTO 313
       else
                write(*,*) 'design not satisfactory'
                write(2,*) 'design not satisfactory'
c             qk= 2.5
c             cover=20.
c             fcu=25.
c             fy=250.
c             dia=12.
              H=H+5.
      if (H. GT. 2.D2) then
           H= 2.D2
           dia = dia-2
      endif
          if(beta.lt.0.0) then
```

```
c          fcu=25.
           h=h+5.
          endif
           GOTO 10
       endif
313       write (*,*) 'Choose an appropriate slab (Y/N) ? '
       READ(*,*) response
      if (response.eq.'Y') then
c           dia=12.
c           cover=20.
c           fcu=25.
c           fy=250.
c           dia=12.
c           qk=2.5
            goto 12
      elseif (response.EQ.'y') then
            goto 12
      endif
      if (response.eq.'N')  goto 1001
      if (response.EQ.'n')  goto 1001
1001     continue
       write(*,*) 'results see my.res'
444      STOP
      END
c                SUBROUTINE  BARSPACE
      SUBROUTINE BARSPC (ID,IC,IP,IB)
      integer id,ic,ip,ib
```

```
      DIMENSION  IBAR (9,9)
         DATA ((IBAR(I,J),J=1,9),I=1,9)/
     +      566, 377, 283, 226, 189, 162, 142, 113, 94,
     +     1010, 671, 503, 402, 335, 287, 252, 201, 168,
     +     1570, 1050, 785, 628, 523, 449, 393, 314, 262,
    +     2260, 1510, 1130, 905, 754, 646, 566, 452, 377,
     +    4020, 2680, 2010, 1610, 1340, 1150, 1010, 804, 670,
     +   6280, 4190, 3140, 2510, 2090, 1800, 1570, 1260, 1050,
     +   9820, 6550, 4910, 3930, 3270, 2810, 2450, 1960, 1640,
     + 16100, 10700, 8040, 6430, 5360, 4600, 4020, 3220,2680,
    +25100, 16800, 12600, 10100, 8380, 7180, 6280,5030, 4190/
      IF (ID.LE.12)  I = ID/2-2
      IF (ID.EQ.16)  I = 5
      IF (ID.EQ.20)  I = 6
      IF (ID.EQ.25)  I = 7
      IF (ID.EQ.32)  I = 8
      IF (ID.EQ.40)  I = 9
      IP = IBAR (I,1)
      WRITE(*,*)'IP=',IP
      DO 30 J=9,1,-1
      K = IBAR (I,J)
      IF (K.LE.IC) GOTO 30
      IF(K.GT.IC) GOTO40
30       continue
40          IP=K
         IN=J
      IF (IN .LE.7) IB =IN*25+25
```

```
      WRITE(*,*)'IB=',IB,IP
      IF (IN .GE.8) IB =IN*50-150
      WRITE(*,*)'IB=',IB,IP
      RETURN
      END
C                SUBROUTINE  BARSPACE
      SUBROUTINE BARSPCC (ID,IC,idd,IPP,IBB)
      integer id,ic,ip,ib
      integer idd,ipp,ibb
      DIMENSION  IBAR (9,9)
         DATA ((IBAR(I,J),J=1,9),I=1,9)/
     +      566, 377, 283, 226, 189, 162, 142, 113, 94,
     +      1010, 671, 503, 402, 335, 287, 252, 201, 168,
     +      1570, 1050, 785, 628, 523, 449, 393, 314, 262,
     +     2260, 1510, 1130, 905, 754, 646, 566, 452, 377,
     +  4020, 2680, 2010, 1610, 1340, 1150, 1010, 804, 670,
     + 6280, 4190, 3140, 2510, 2090, 1800, 1570, 1260, 1050,
     + 9820, 6550, 4910, 3930, 3270, 2810, 2450, 1960, 1640,
     +16100, 10700, 8040, 6430, 5360, 4600, 4020, 3220, 2680,
     +25100, 16800, 12600, 10100, 8380, 7180, 6280, 5030, 4190/
        IF (ID.LE.12) then
             ID=ID-2
             I = ID/2-2
           ELSEIF (ID.EQ.16) then
             ID=ID-4
             I = 4
           ELSEIF (ID.EQ.20) then
```

```
      ID=ID-4
      I = 5
     ELSEIF (ID.EQ.25) then
      ID=ID-5
      I = 6
     ELSEIF (ID.EQ.32) then
      ID=ID-7
      I = 7
     ELSEIF (ID.EQ.40) then
      ID=ID-8
      I = 8
     elseif (id.eq.48) then
      id=id-8
      i=9
     ENDIF
     IP = IBAR (I,1)
     WRITE(*,*)'IP=',IP
     DO 30 J=9,1,-1
     K = IBAR (I,J)
     IF (K.LE.IC) GOTO 30
     IF(K.GT.IC) GOTO40
30      continue
40         IP=K
        IN=J
     ipp=ip
     IF (IN.LE.7) IB=IN*25+25
     IF (IN.GE.8) IB=IN*50-150
```

```
      WRITE(*,*)'IB=',IB,IPP
      idd=id
      ibb=ib
      RETURN
      END
      SUBROUTINE KIDRIS(n5,ex,targ)
C  THE SUBROUTINE TESTS THE RELIABILITY LEVEL FOR THE DESIGN
C     OF SLABS BASED ON BS8110 RECOMMENDATIONS.
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C     LIMIT STATE FUNCTION IS DECLARED  EXTERNAL
C        X1=cube strength (N/mm2),FCU- (lognormal)
C        X2=Yield strength(N/mm2),fy- (lognormal)
c        X3=Effective depth,ED - (normal)
C        X4=Live load -(gumbel)
C        X5=Reinforcement Ratio -(normal)
      EXTERNAL QIDRIS
      dimension ex(n5)
      DIMENSION XX(5),SX(5),VP(10,5),COV(5,5),ZES(3),
     2            UU(5),EIVEC(5,5),IV(2,5)
      CHARACTER*10 PRT
C      WIDTH B OF THE SLAB IS SET FOR THE PROGRAM AND THE
C      LIMIT_ STATE SUBROUTINE IN THE FOLLOWING COMMON BLOCK
      common/gidris/span,ALPHA,num,beta
      data n/5/,nc/5/,ne/5/,irho/0/
C
C   MEAN AND STANDARD DEVIATION OF THE VARIABLES AS WELL AS
C       THE PARAMETERS OF FORM5 ARE GIVEN IN THIS DATA BLOCK
```

```
      SX(1) = EX(1)*1.5d-1
      SX(2) = EX(2)*1.5d-1
      SX(3) = EX(3)*6.0d-2
      SX(4) = EX(4)*3.0d-1
      SX(5) = EX(5)*6.0d-2
C        THE RATIO OF DEAD TO LIVE LOADS (DETERMINISTIC)
C        THE RESULTS ARE WRITTEN ON FILE NO.7
      NAUS=7
C        PRINT TO SCREEN:
      ICRT=0
      OPEN (7,FILE='master.res',STATUS='OLD',ERR=10)
      GOTO 20
10       OPEN (7,FILE='master.res',STATUS='NEW')
C
C     PRESETTING THE VARIABLES VP,COV,AND IV DONE USING YINIT
20       CALL YINIT (N,IV,VP,IRHO,COV,NC)
      IV(1,1)=3
      IV(1,2)=3
      IV(1,4)=7
      DO 100 i=1,N
100      XX(I)=EX(I)
      V1=0.5D0
      BETA=1.D0
C
c INITIAL_SOLUT._ESTIMATE(IF NOT MEAN VALUES, SEE LOOP "100")
      WRITE(NAUS,5000)
5000 FORMAT(////,5X,70('*'),/,30X,'FORM5',/,5X,70('*'),/,5X,
```

```
     1     20X,'EXAMPLE USING "QIDRIS", 5 VARIABLES:')
C       THE STOCHASTICAL MODEL IS PRINTED USING "YKOPF"
      CALL YKOPF (NAUS,N,IV,EX,SX,VP,IRHO)
C       PRINT ALSO TO SCREEN
      WRITE(ICRT,*) 'START OF FORM5 '
      WRITE (ICRT,*) 'STOCHASTIC MODEL:'
      CALL YKOPF (ICRT,N,IV,EX,SX,VP,IRHO)
C       THE MATRIX COV IS PRINTED USING "YMAUS"
      PRT=' COV  '
      CALL YMAUS (NAUS,NC,N,COV,PRT)
C       THE MAIN SUBROUTINE IN FORM IS CALLED
      CALL FORM5(N,IV,EX,SX,VP,QIDRIS,IRHO,COV,NC,
     +          EIVEC,NE,V1,NAUS,BETA,XX,UU,ZES,IER)
C       THE COORDINATES OF THE BETA-POINT ARE PRINTED WITH
C         THE TITLE VECTOR UU
      PRT=' UU '
      CALL YFAUS(NAUS,N,UU,PRT)
C    COMPUTE SAFETY INDEX AND CORRESPONDING PROBABILITY OF
C       FAILURE
      TARG=BETA
C    COORDINATES OF THE BETA_POINT ARE PRINTED WITH THE TITLE
C       VECTOR ZES
C       THE VECTOR ZES IS PRINTED WITH THE TITLE VECTOR ZES
      PRT= ' ZES '
      CALL YFAUS (NAUS,3,ZES,PRT)
      WRITE (ICRT,*) 'END OF FORM5 :IER =',IER
      WRITE(ICRT,*) 'RESULTS SEE FILE master.res'
```

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```
      write(2,*) beta
      return
      END
           SUBROUTINE QIDRIS(N,XX,FX,IER)
C
C       LIMIT STATE FUNCTION
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
        DIMENSION XX(N)
      common/gidris/span,ALPHA,num,beta
      REAL FN
      FN=8.D0
      IF(NUM.EQ.2) THEN
          FN=2.D0
      ELSEIF(NUM.EQ.4) THEN
          FN=1.2D1
      endif
      if(alpha.gt.0) then
      GFX=0.87*XX(2)*XX(5)*1.D3*(XX(3)**2)
      QFX= GFX*(1-0.97*XX(2)*XX(5)/XX(1))
      FX= QFX-(0.875*ALPHA+1.D0)*1.6D0*XX(4)*(span**2)/FN
      ier=0
        else
         FX=1.D+20
         IER=1
      endif
      RETURN
      END
```

4.5 INPUT DATA DEFINITIONS

Data needed for the design of slabs need to be read into the program. The type and order of such data are given below. (Note that their definitions have already been given)

Case 1:

Data read from the data file are as follows:

Qk, Dia, cover, fcu, fy

Case 2:

Data which the program asks the user to feed in are as follows:

H, Span, Alpha, larger and smaller panel dimensions.

CHAPTER FIVE

DEMONSTRATION AND DISCUSSION OF RESULTS

5.1 DEMONSTRATION AND DISCUSSION

The program developed was used to generate results for various spans, characteristic strengths of steel and concrete and for various load ratios as well as various support conditions. The results are as tabulated in Tables 5.1 to 5.12. Three tables were presented for each of the four support conditions. The load ratios were varied for each span, characteristic strengths of steel and concrete. An initial value of the overall depth was assumed to be 60mm to 75mm depending upon the user's choice. The program now carries out the design to satisfy both the British Standard BS8110 (1985) and the First Order Reliability Method (FORM, 1988) requiremnts. To satisfy these requirements, an iterative design procedure is adopted by the program, that is, the design parameters are changed until the said conditions are satisfied. The program now provides the user with the final overall depth of the slab, the effective depth and the reinforcement ratio. These are as shown in Tables 5.1 to 5.12.

Plots of reinforcement ratios versus load ratios, and reinforcement ratios versus effective depths for varying spans, characteristic strengths of steel and concrete are made. The plots are as shown in Figures 5.1 to 5.24.

It is shown for uniformity in reliability level, there must be a linear relationship between the reinforcement ratio and effective depth. However, a non linear relationship exists between the reinforcement ratio and the ratio of dead to live load. Equations showing these effects are also developed and can be used directly for safe designs. These observations are on the basis of meeting a targetted safety index of 3.

5.2. **Typical Design Example of a slab.**

It is required to design a simply supported one-way slab of span = 3000mm, f_{cu} = 25 N/mm^2 and fy = 460 N/mm^2, assuming a load ratio, α = 4.

The objective here is to design the said slab according to the BS8110 requirements. In addition, the First Order Reliability (FORM) approach has to be adopted for a target safety level. This implies that the safety level of this particular design should be greater than or equal to a targetted safety index. In essence, equation (3.65) has to be satisfied for this design.

From Fig. 5.1 the reinforcement ratio, ρ corresponding to this load ratio = 0.2947

From Fig. 5.2, an effective depth d = 105.66mm corresponds to ρ = 0.2947

Therefore from

$$\rho_1 = \frac{100A_s}{bd} = 0.2947$$

it implies that,

$$A_s = \frac{0.2947bd}{100} = \frac{0.2947x10^3x105.66}{100}$$

Thus, A_s = 311.4 mm^2/m.

Assuming 20mm Concrete cover to reinforcement, and a bar size of 10mm an Overall depth, h is given as

$$h = d + cover + \frac{\Phi}{2} = 105.66 + 20 + 5 = 140.66 mm$$

The above example shows that with the use of the charts, there is simplicity in the design procedure followed since there are only three steps involved.

This gives the ease with which the slab is designed to meet the ultimate and serviceability limit states requirements set out in the BS8110 (1985) as well as the safety index format of the FORM. Calculation time is therefore greatly reduced and the safety of the designed element is known a priori. This is an aspect that the current design procedure of BS8110 (1985) cannot provide.

5.3 RESULTS OBTAINED FROM DESIGN PROGRAM

TABLE 5.1

Simply Supported Slab (Span= 3000mm, Fcu=25 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	140	117	0.24
2	135	111	0.26
3	130	105	0.30
4	130	105	0.30
5	140	117	0.24

TABLE 5.2

Simply Supported Slab (Span= 2000mm, Fcu=20 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	120	96	0.18
2	110	86	0.20
3	120	96	0.21
4	110	86	0.23
5	120	96	0.18

TABLE 5.3

Simply Supported Slab (Span= 3000mm, Fcu=30 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	135	106	0.27
2	130	105	0.30
3	130	105	0.30
4	135	109	0.34
5	130	105	0.30

TABLE 5.4

Cantilevered Supported Slab (Span= 1500mm, Fcu=20 N/mm^2 , Fy=250N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	145	121	0.41
2	140	116	0.43
3	140	116	0.43
4	140	115	0.45
5	150	126	0.39

TABLE 5.5

Cantilevered Supported Slab (Span= 2000mm, Fcu=25 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	140	116	0.43
2	140	116	0.43
3	135	110	0.47
4	135	110	0.48
5	140	116	0.43

TABLE 5.6

Cantilevered Supported Slab (Span= 1750mm, Fcu=25 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	160	135	0.46
2	155	130	0.48
3	160	135	0.46
4	155	130	0.48
5	155	130	0.46

TABLE 5.7

Propped Cantilevered Supported Slab (Span= 2000mm,

Fcu=20 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO p=100*As/B*D
1	110	86	0.20
2	100	76	0.22
3	100	76	0.32
4	110	86	0.23
5	120	96	0.18

TABLE 5.8

Propped Cantilevered Supported Slab (Span= 3000mm,

Fcu=20 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO p=100*As/B*D
1	150	126	0.20
2	140	115	0.23
3	140	115	0.23
4	140	116	0.25
5	140	115	0.23

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TABLE 5.9

Propped Cantillevered Supported Slab (Span= 3500mm,

Fcu=25 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	165	141	0.20
2	160	136	0.21
3	155	131	0.26
4	155	131	0.26
5	160	136	0.21

TABLE 5.10

Fixed-Fixed Supported Slab (Span= 4500mm,

Fcu=25 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	165	140	0.28
2	155	129	0.29
3	155	129	0.29
4	155	130	0.30
5	165	140	0.28

TABLE 5.11

Fixed-Fixed Supported Slab(Span= 5000mm, Fcu=25 N/mm2, Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	185	161	0.25
2	180	155	0.29
3	180	155	0.29
4	175	150	0.30
5	180	155	0.29

TABLE 5.12

Fixed-Fixed Supported Slab (Span= 6000mm, Fcu=30 N/mm^2 , Fy=460N/mm^2)

LOAD RATIO	DESIGN OVERALL DEPTH H(mm)	EFFECTIVE DEPTH d(mm)	REINFORCEMENT RATIO ρ=100*As/B*D
1	200	174	0.32
2	200	174	0.32
3	210	185	0.34
4	210	185	0.34
5	200	174	0.32

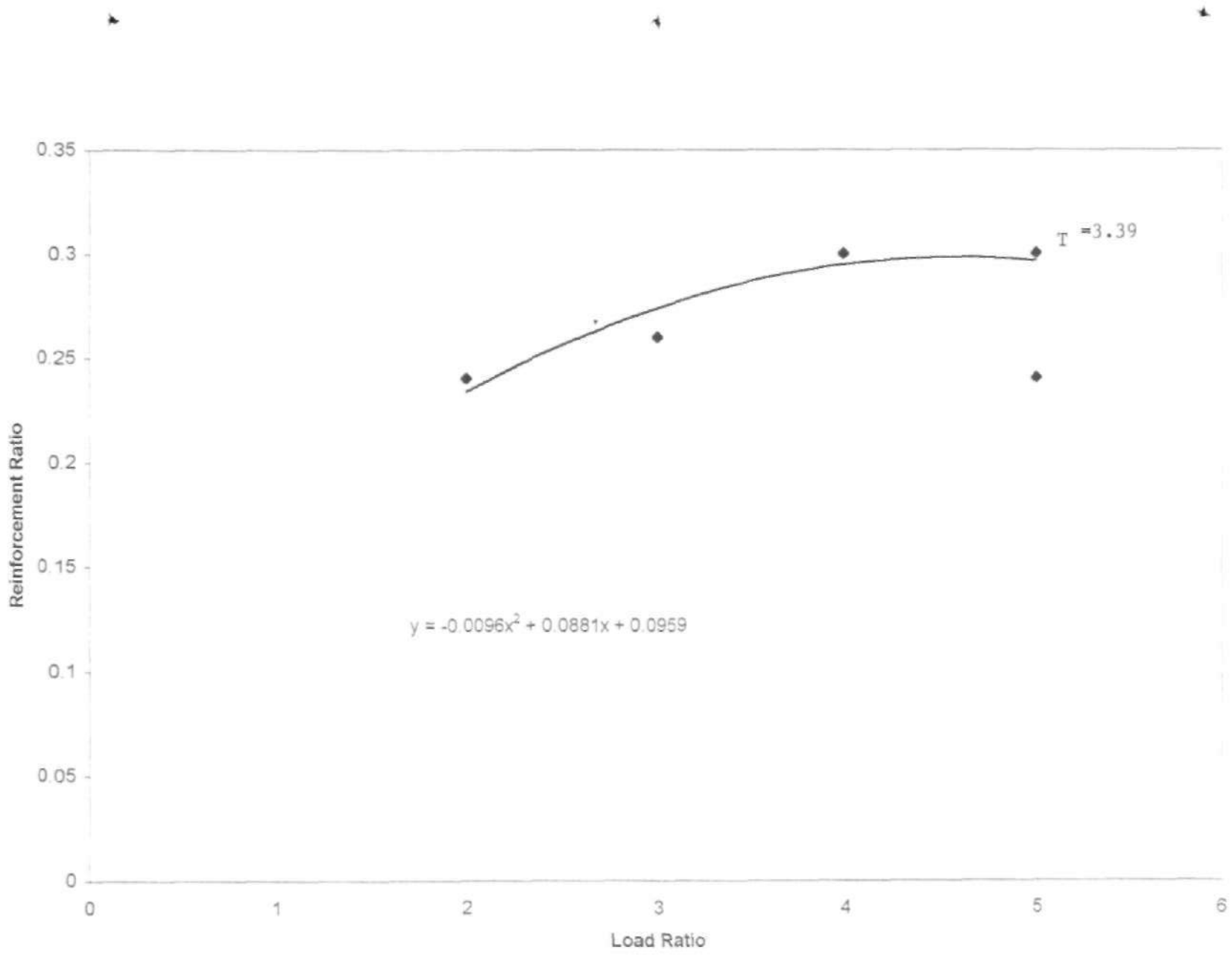


Fig. 5.1 Reinforcement Ratio versus Load Ratio for Simply Supported Slab Span= 3000mm, Fcu= 25 N/mm², Fy = 460 N/mm²

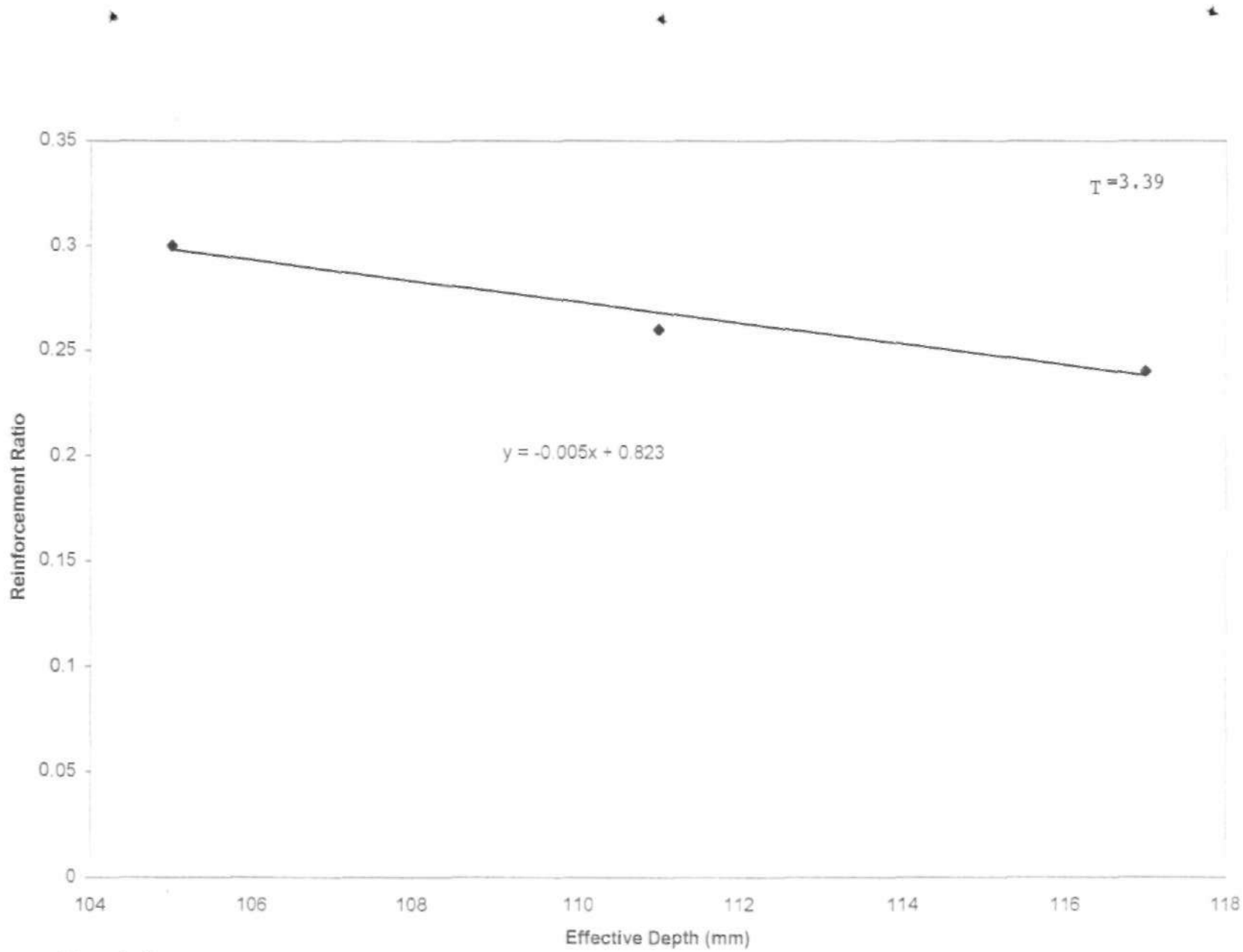


Fig. 5.2 Reinforcement Ratio versus Effective Depth for Simply Supported Slab Span=3000mm, Fcu = 25 N/mm^2 , Fy = 460 N/mm^2.

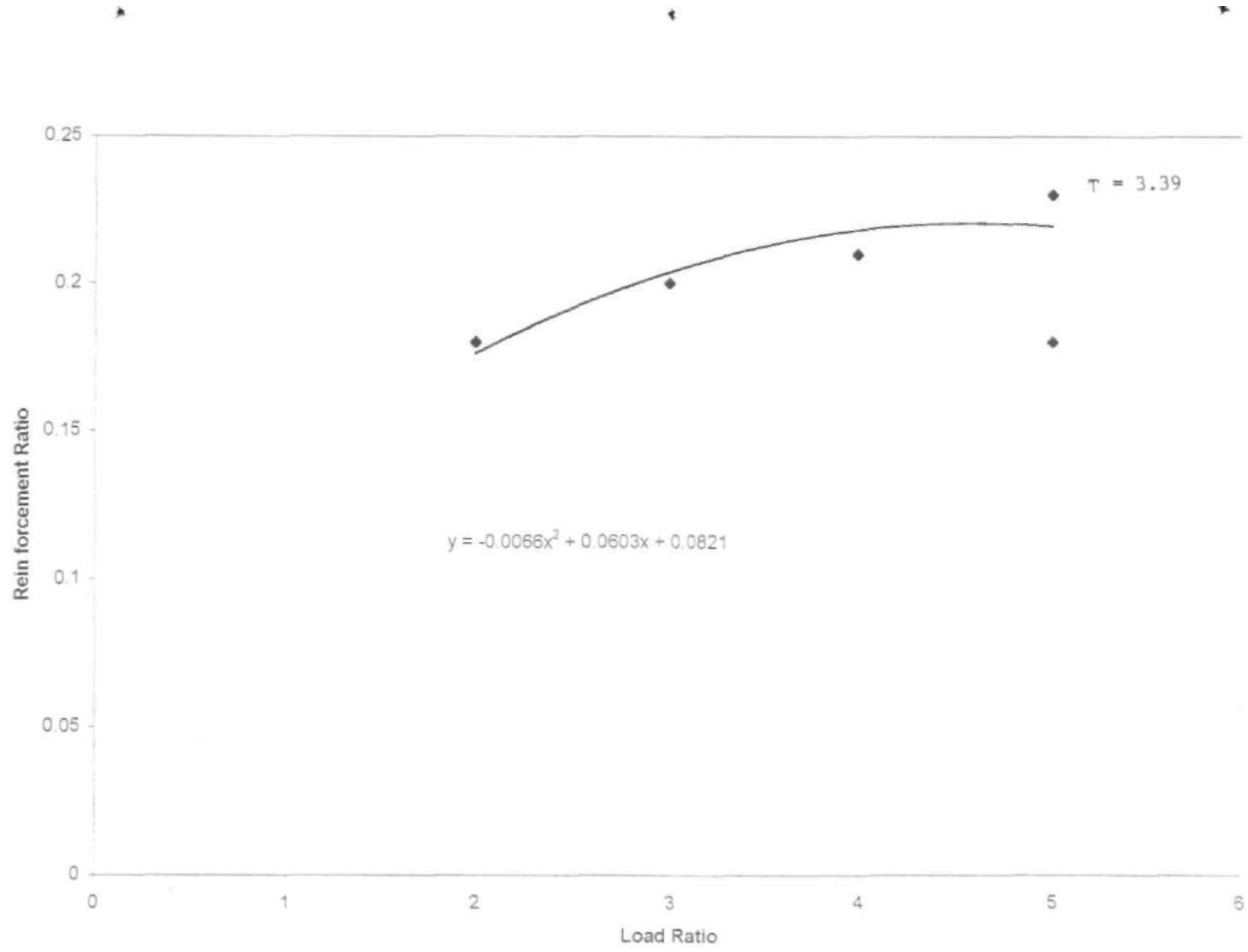


Fig. 5.3 Reinforcement Ratio versus Load Ratio for Simply Supported Slab Span= 2000mm. Fcu= 20 N/mm², Fy = 460 N/mm²

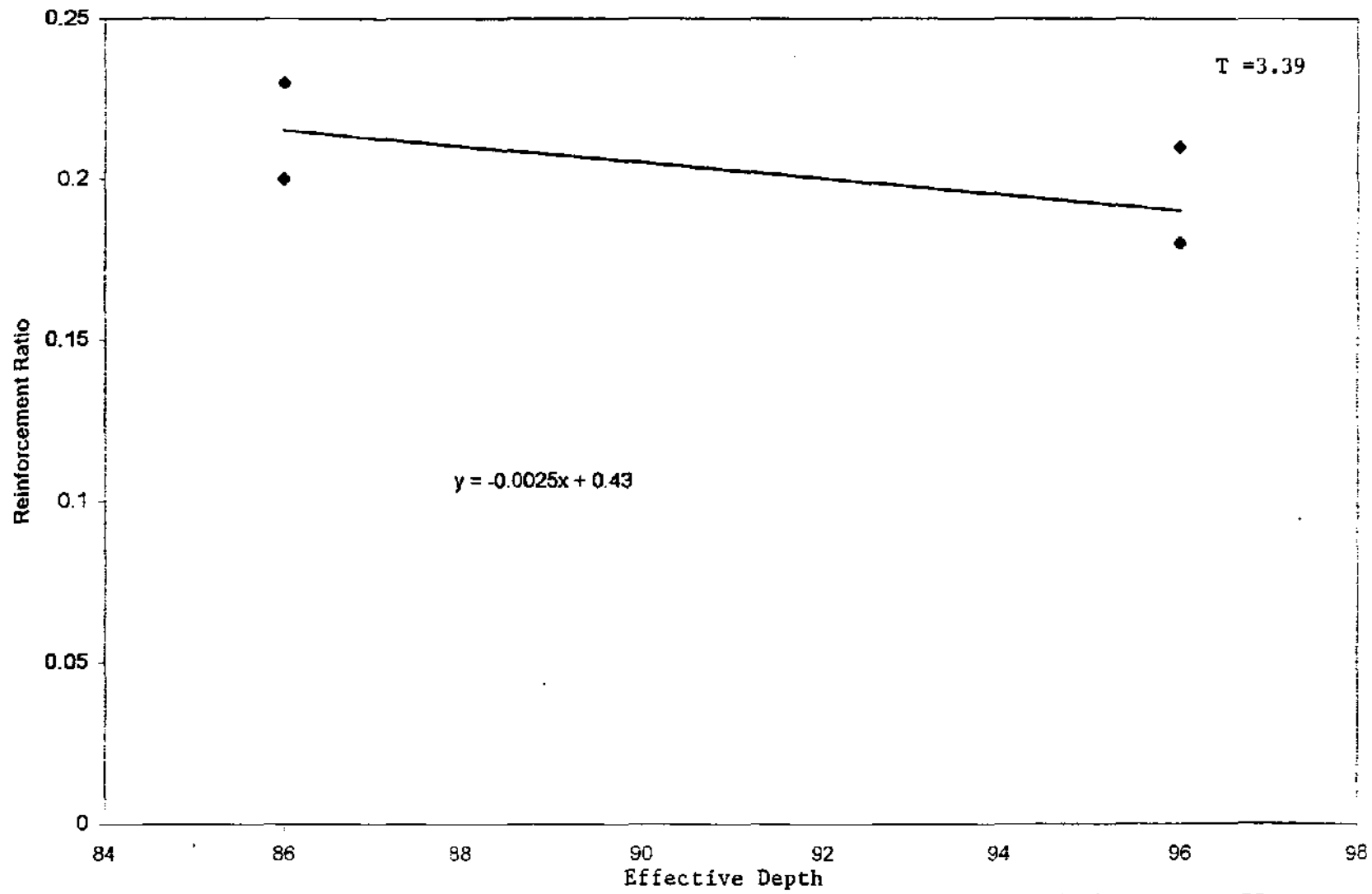


Fig. 5.5 Reinforcement Ratio versus Effective Depth for Simply Supported Slab span = 2000mm, Fcu = 20/mm2, Fy = 460 = N/mm2

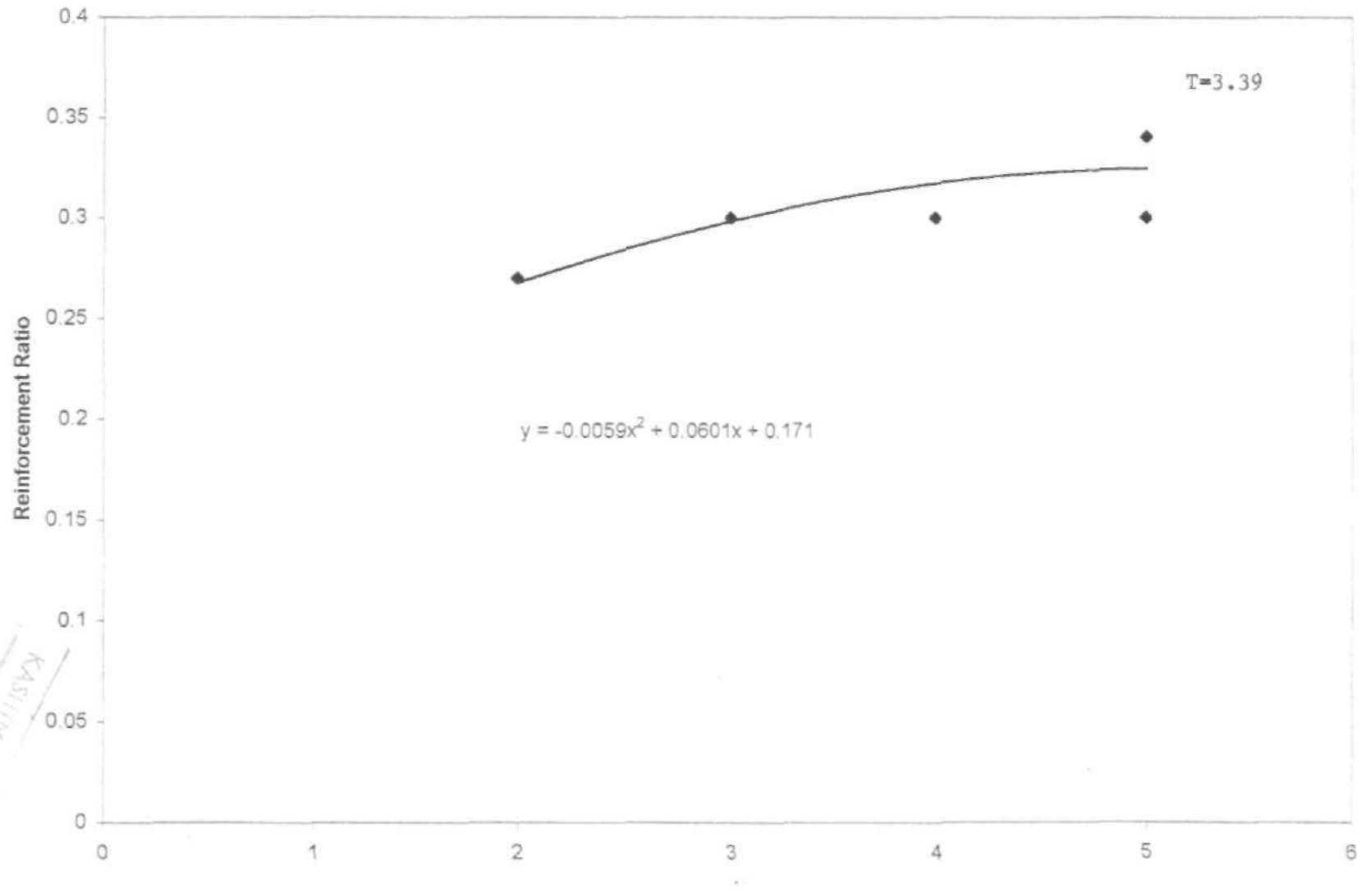


Fig. 5.5 Reinforcement Ratio versus Load Ratio for Simply Supported Slab Span= 3000mm. Fcu= 30 N/mm², Fy = 460 N/mm²

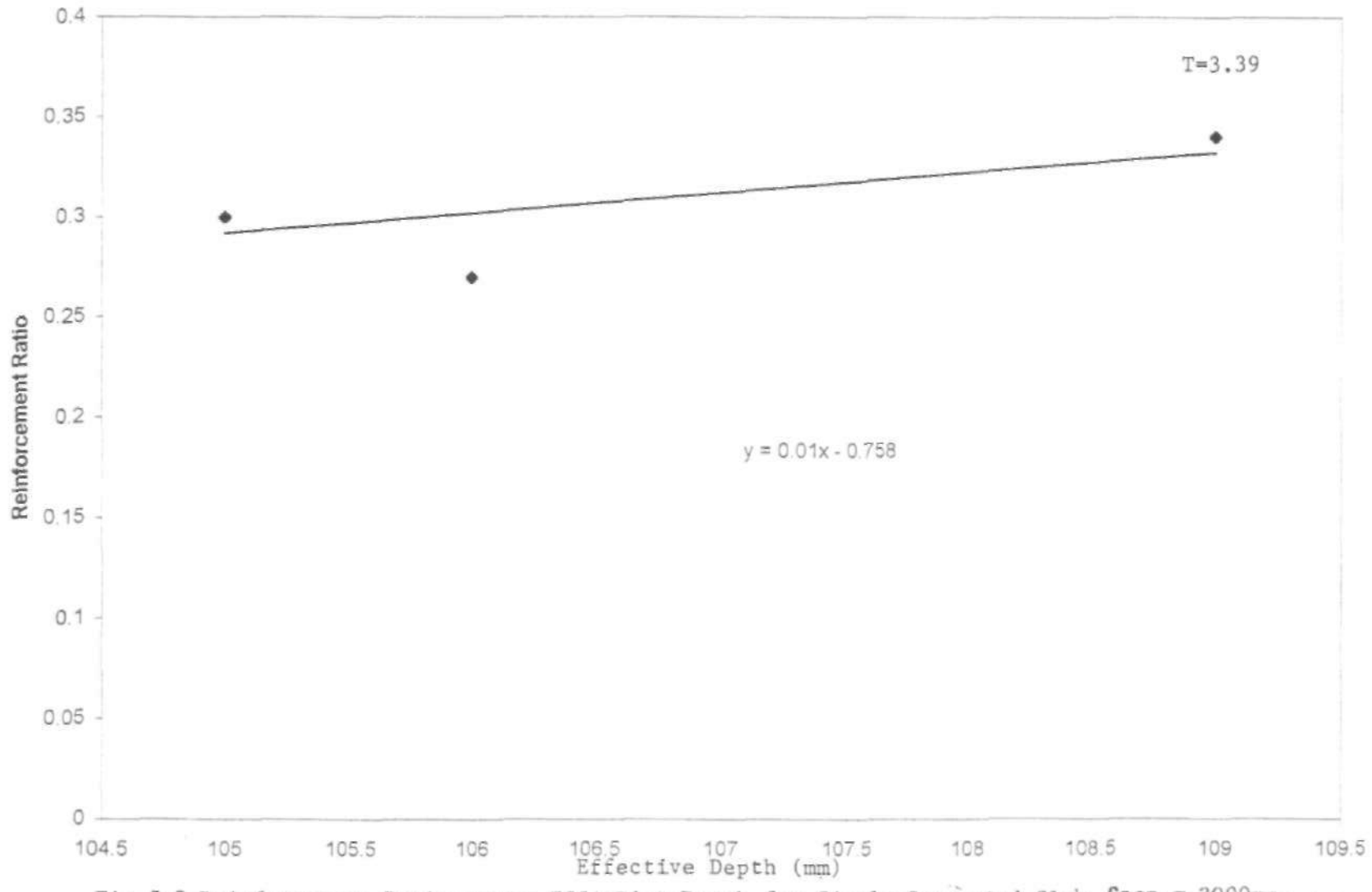


Fig.5.7 Reinforcement Ratio versus Effective Depth for Simply Supported Slab Span = 3000mm, Fcu = 30 N/mm^2, Fy = 460 N/mm^2

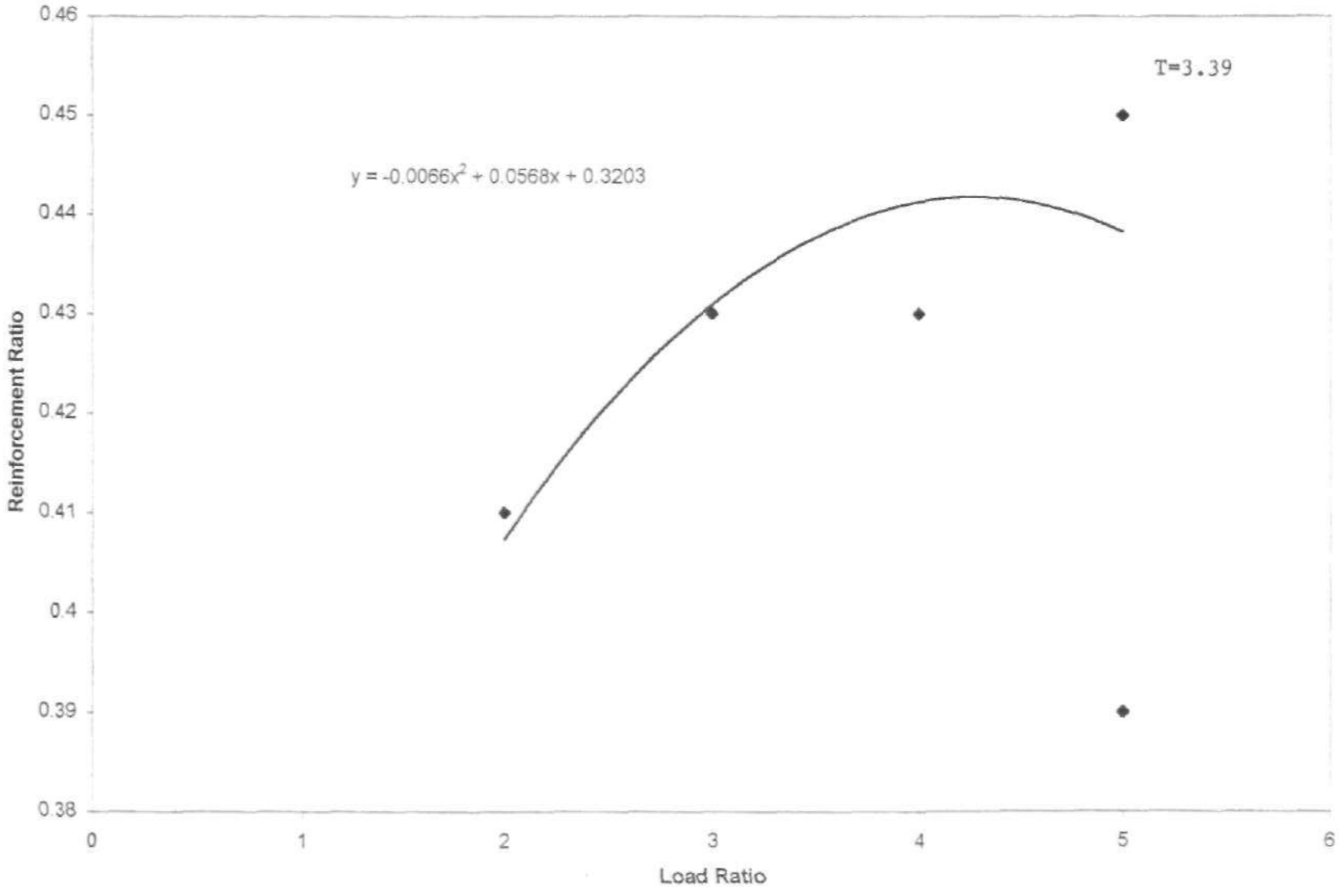


Fig. 5.7 Reinforcement Ratio versus Load Ratio for Cantilevered Slab, Span= 1500mm, Fcu= 20 N/mm², Fy = 250 N/mm²

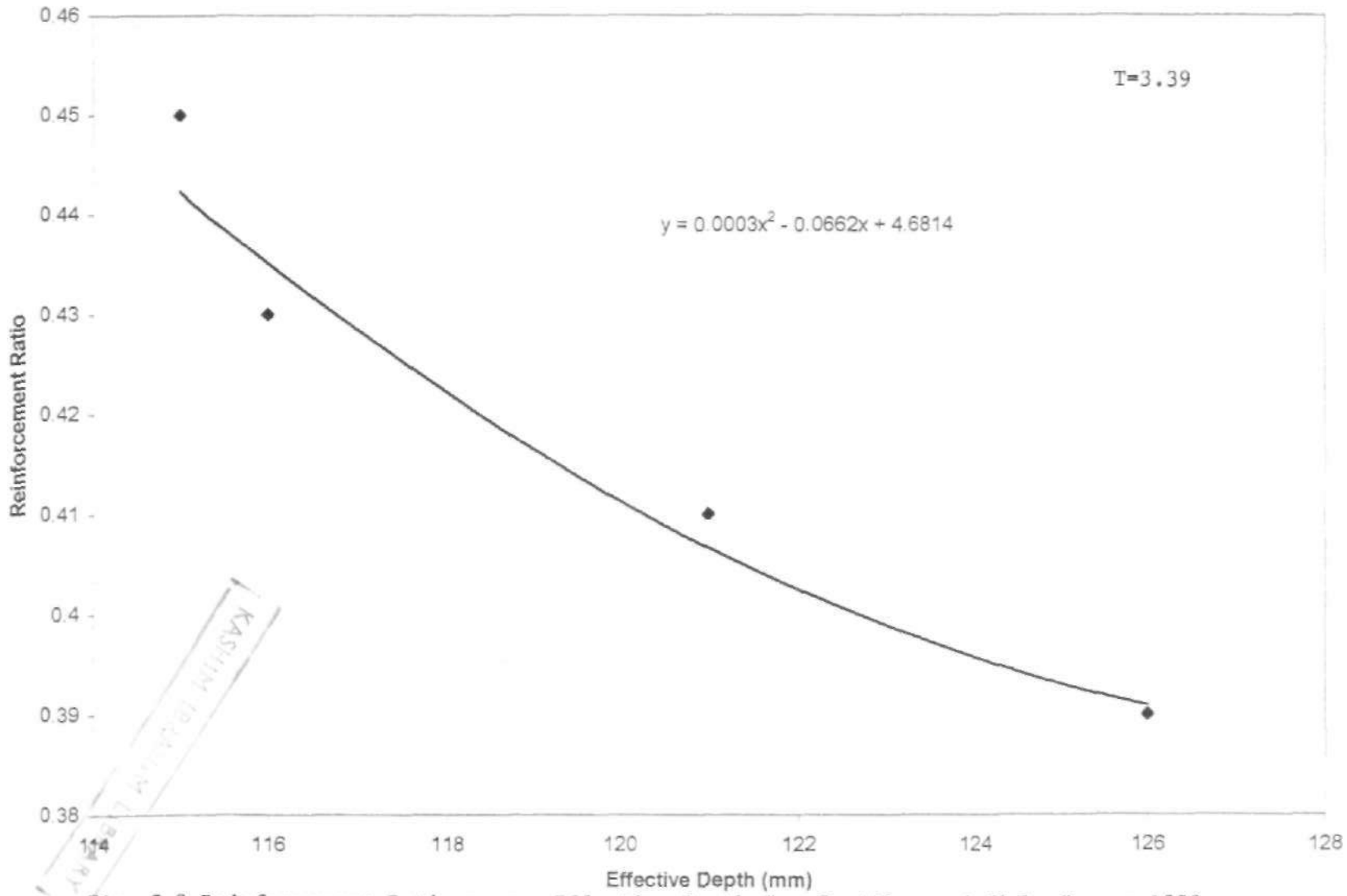


Fig. 5.8 Reinforcement Ratio versus Effective Depth for Cantilevered Slab, Span = 1500mm, Fcu = 20 N/mm^2, Fy = 250 N/mm^2

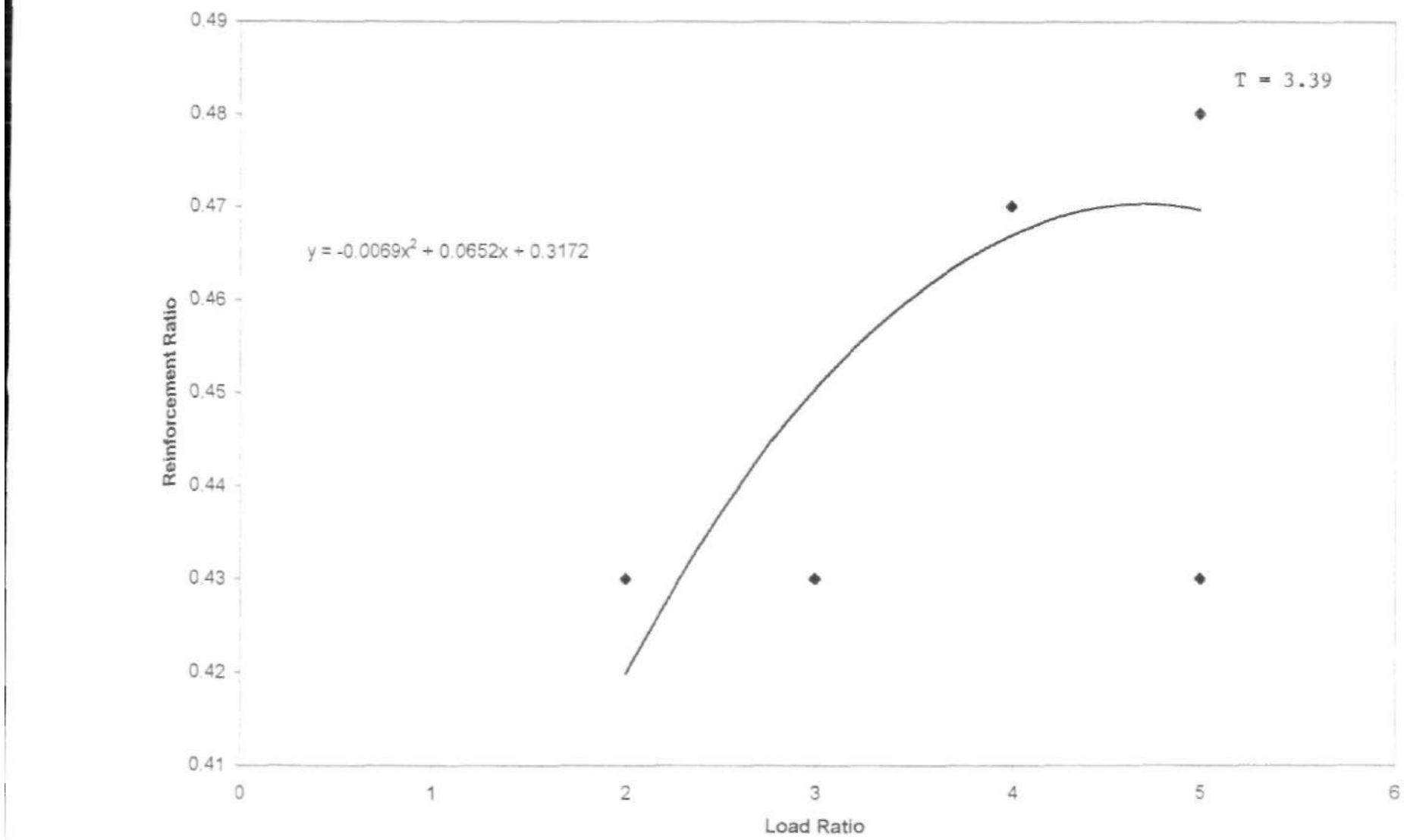


Fig. 5.9 Reinforcement Ratio versus Load Ratio for Cantilevered Slab, Span= 2000mm, Fcu= 25 N/mm², Fy = 460 N/mm²

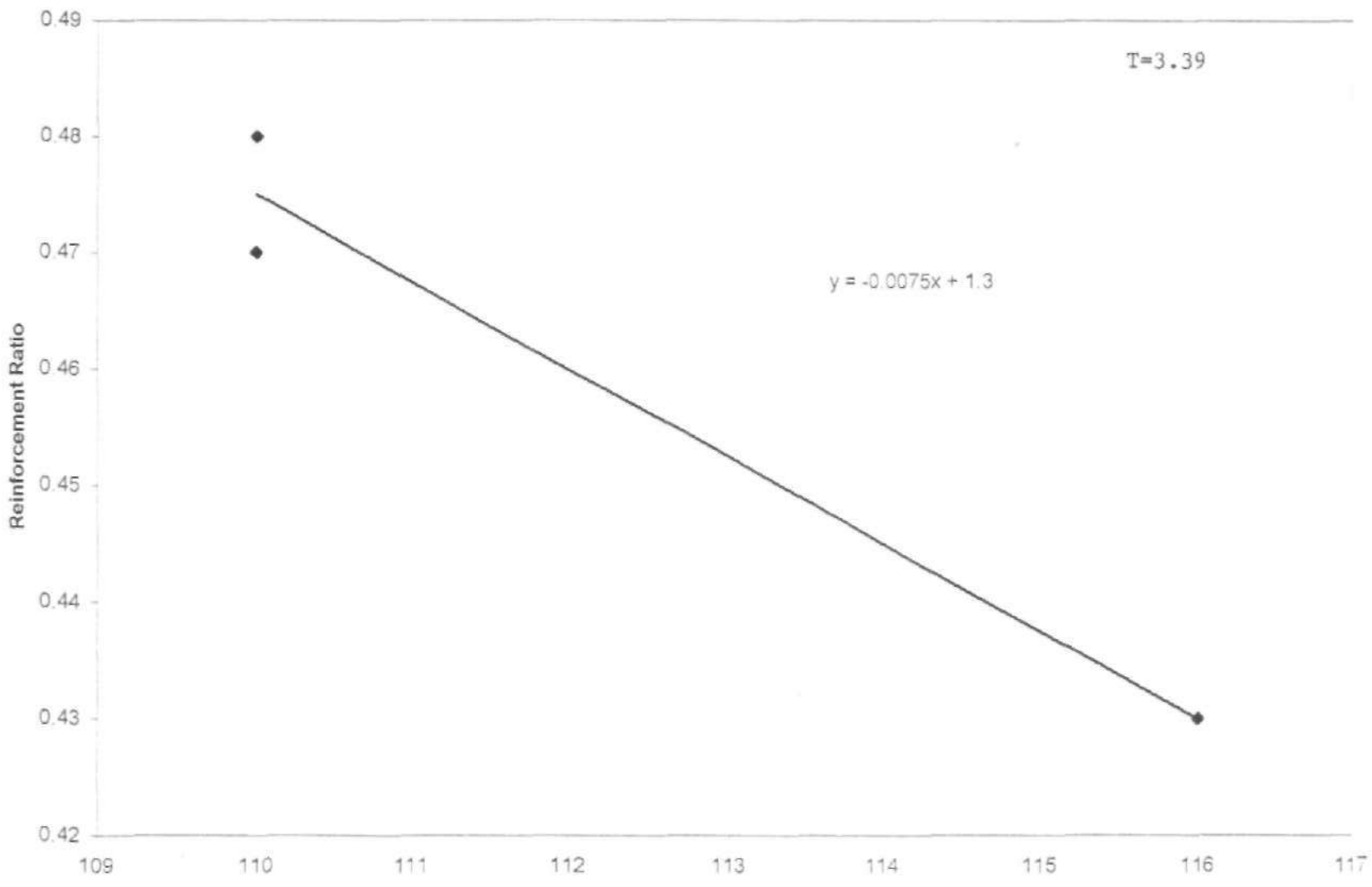


Fig. 5.11 Reinforcement Ratio versus Load Ratio for cantilevered Slab, Span = 2000 mm, Fcu = 25 N/mm^2, Fy = 460 N/mm^2

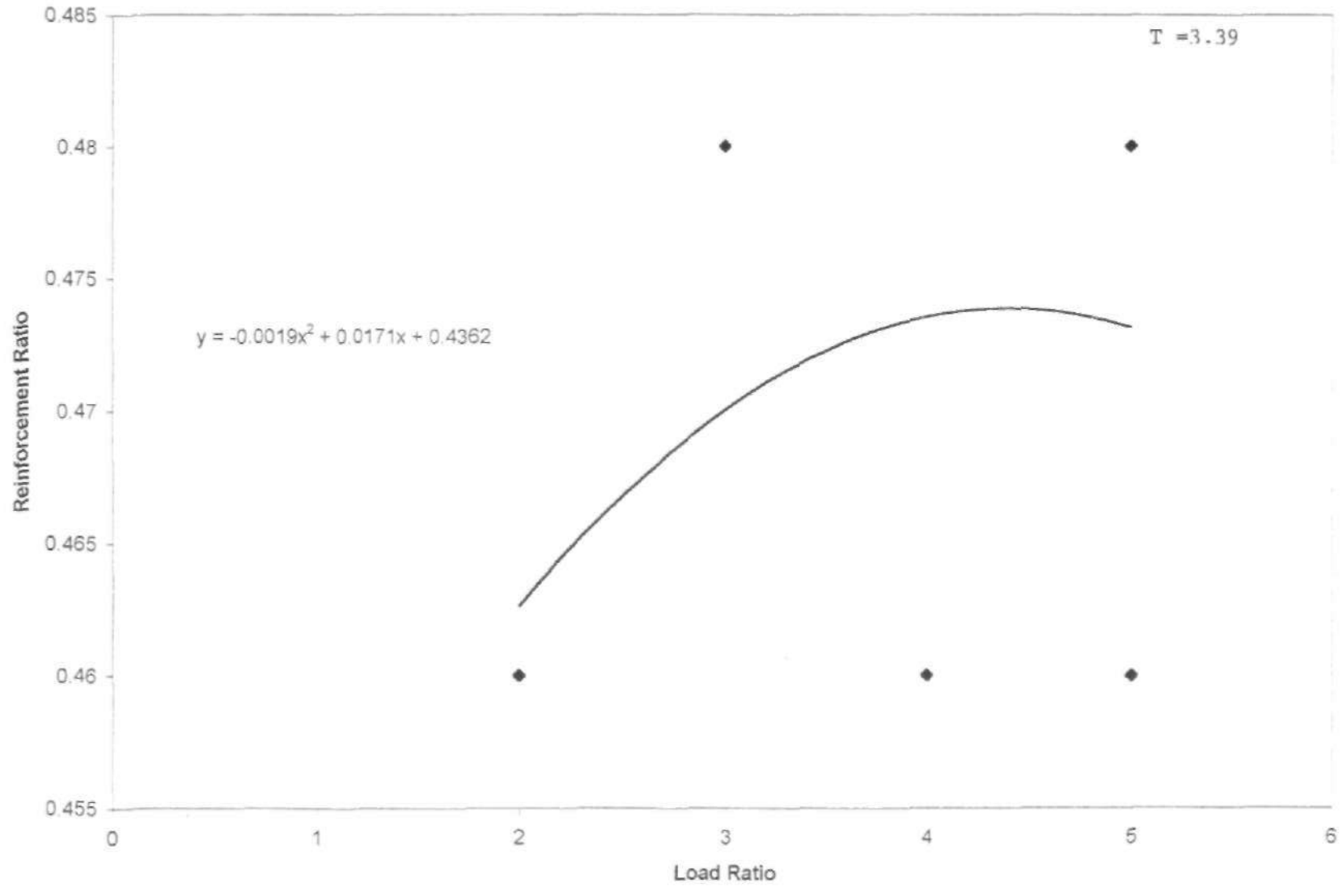


Fig. 5.11 Reinforcement Ratio versus Load Ratio for Cantilevered Slab, Span= 1750mm, Fcu= 25 N/mm², Fy = 460 N/mm²

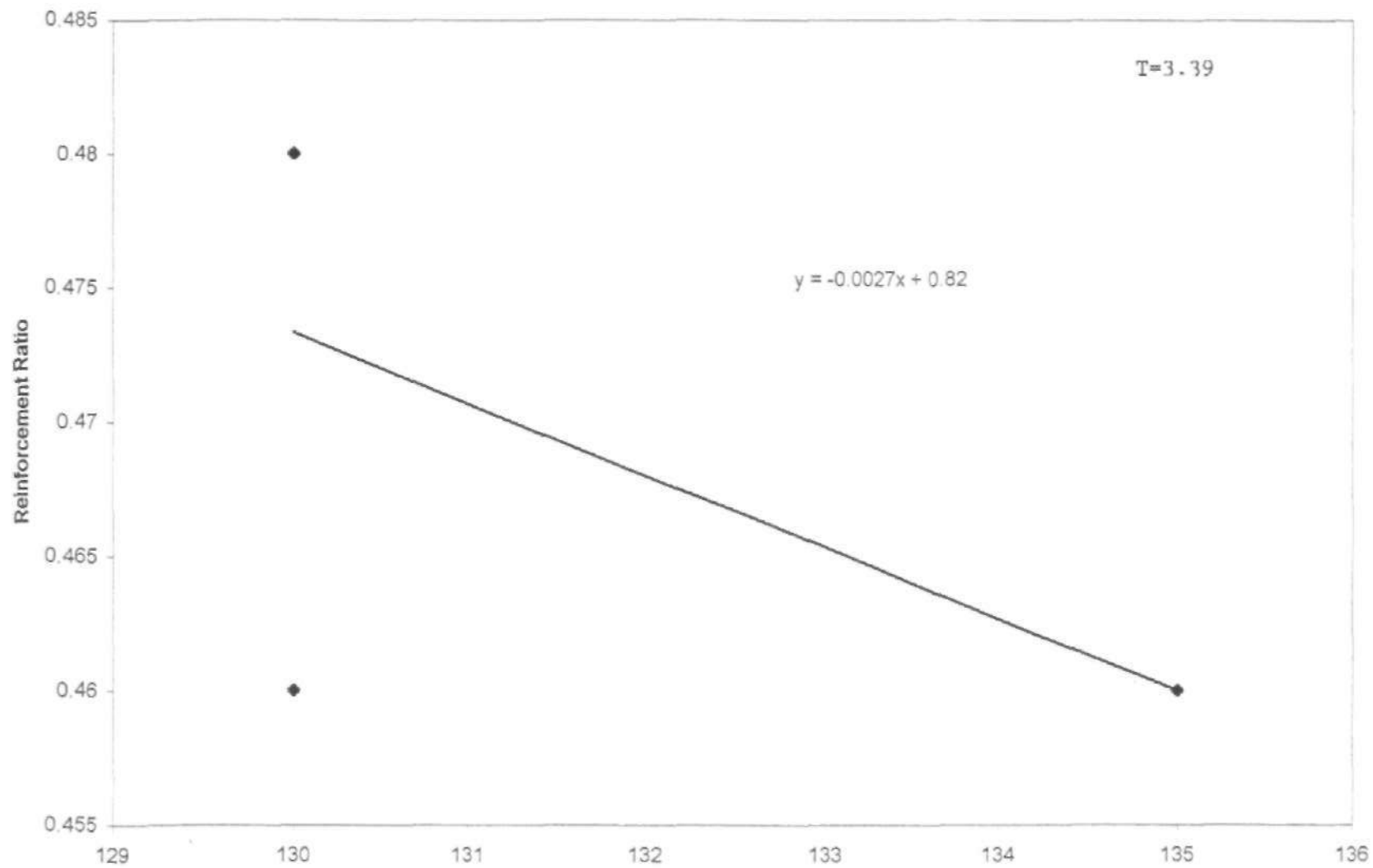


Fig. 5.12 Reinforcement Ratio versus Effective Depth for Cantilevered Span, Span = 1750mm, Fcu = 25 N/mm^2, Fy = 460 N/mm^2

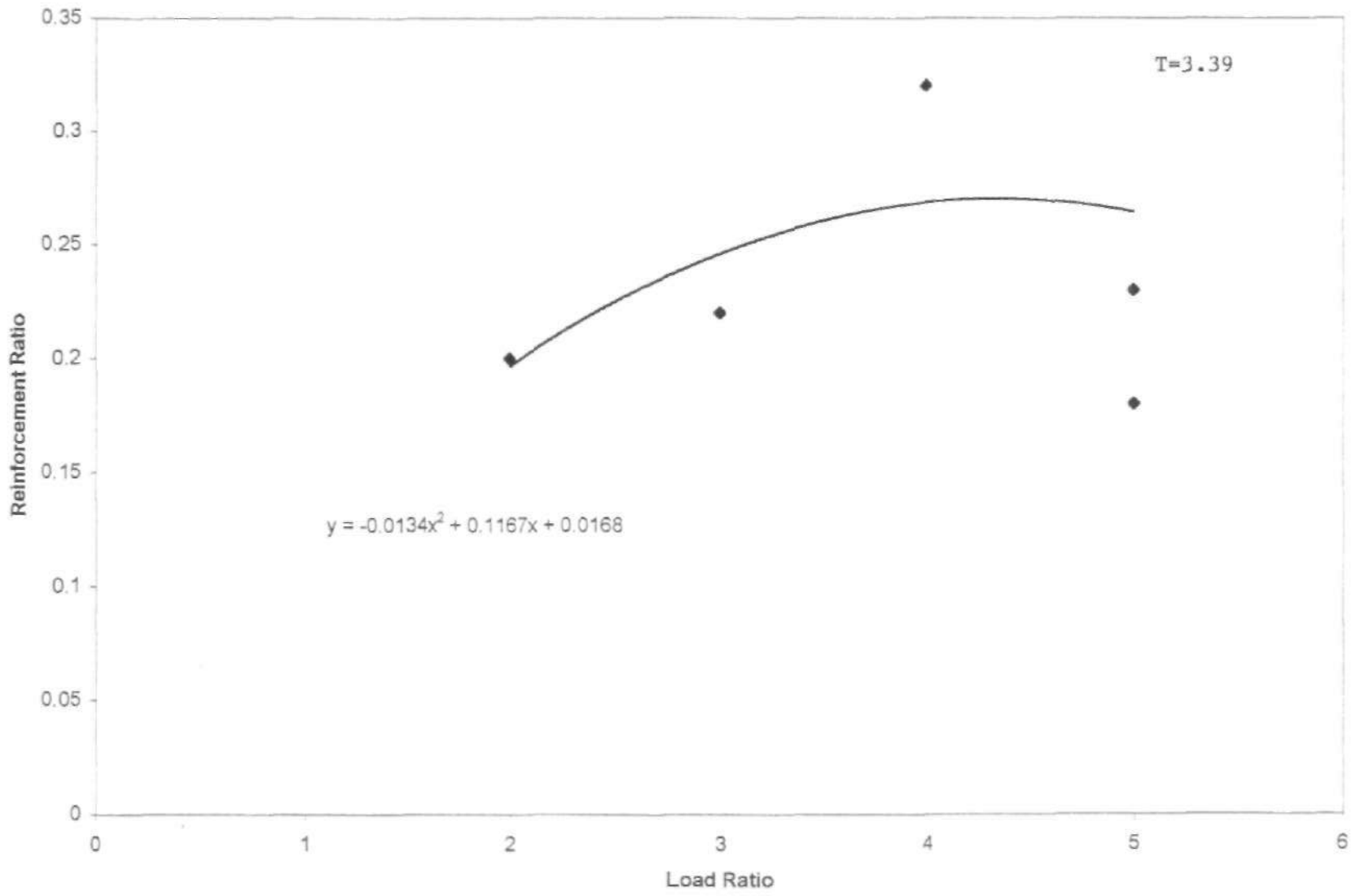


Fig. 5.13 Reinforcement Ratio versus Load Ratio for Propped Cantilevered Slab, Span= 2000mm, Fcu= 20 N/mm², Fy = 460 N/mm²

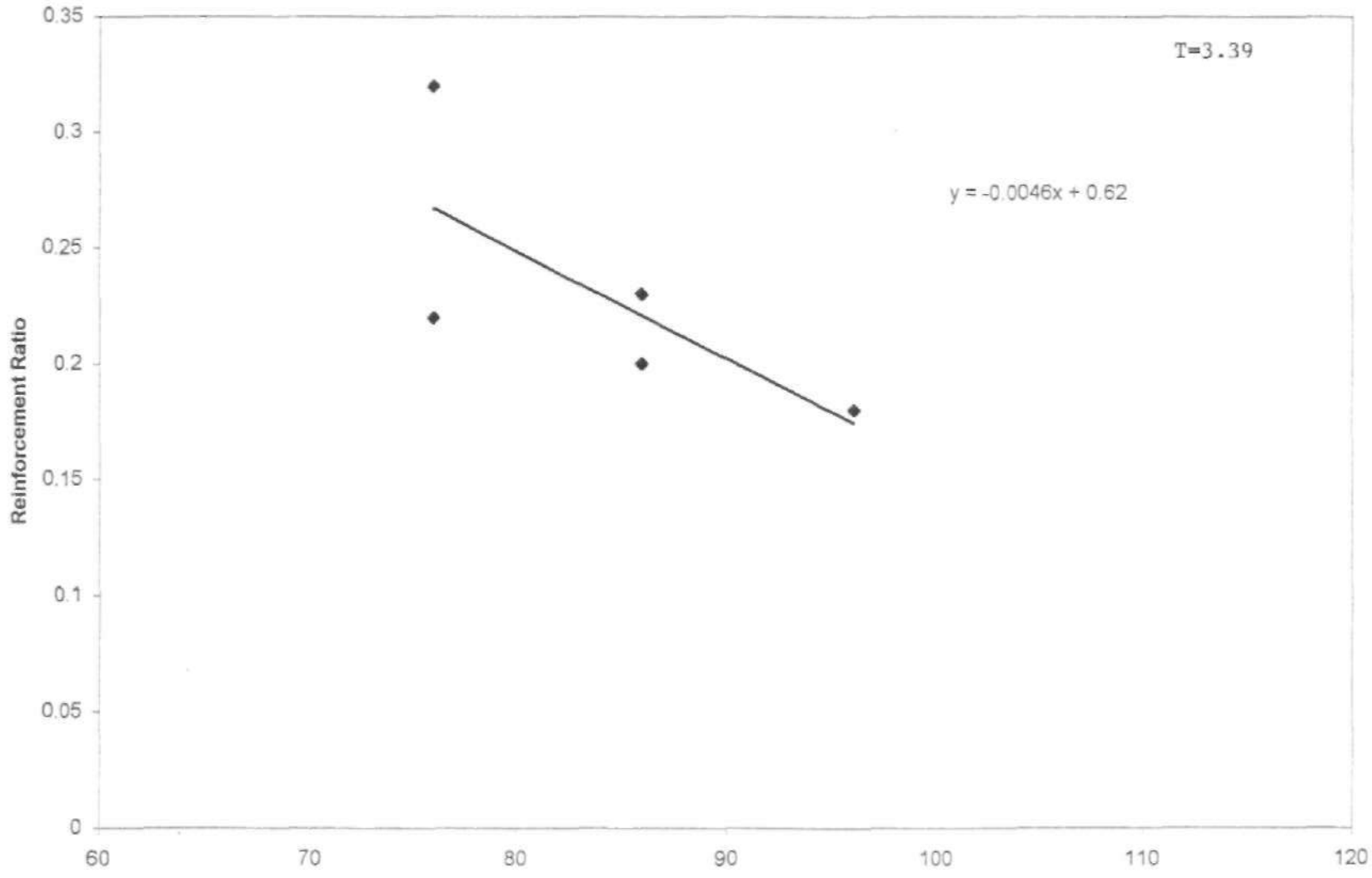


Fig. 5.14 Reinforcement Ratio versus Effective Depth for propped Cantilevered Slab, Span = 2000 mm, Fcu = 20/mm^2 Fy, = 460 N/mm^2

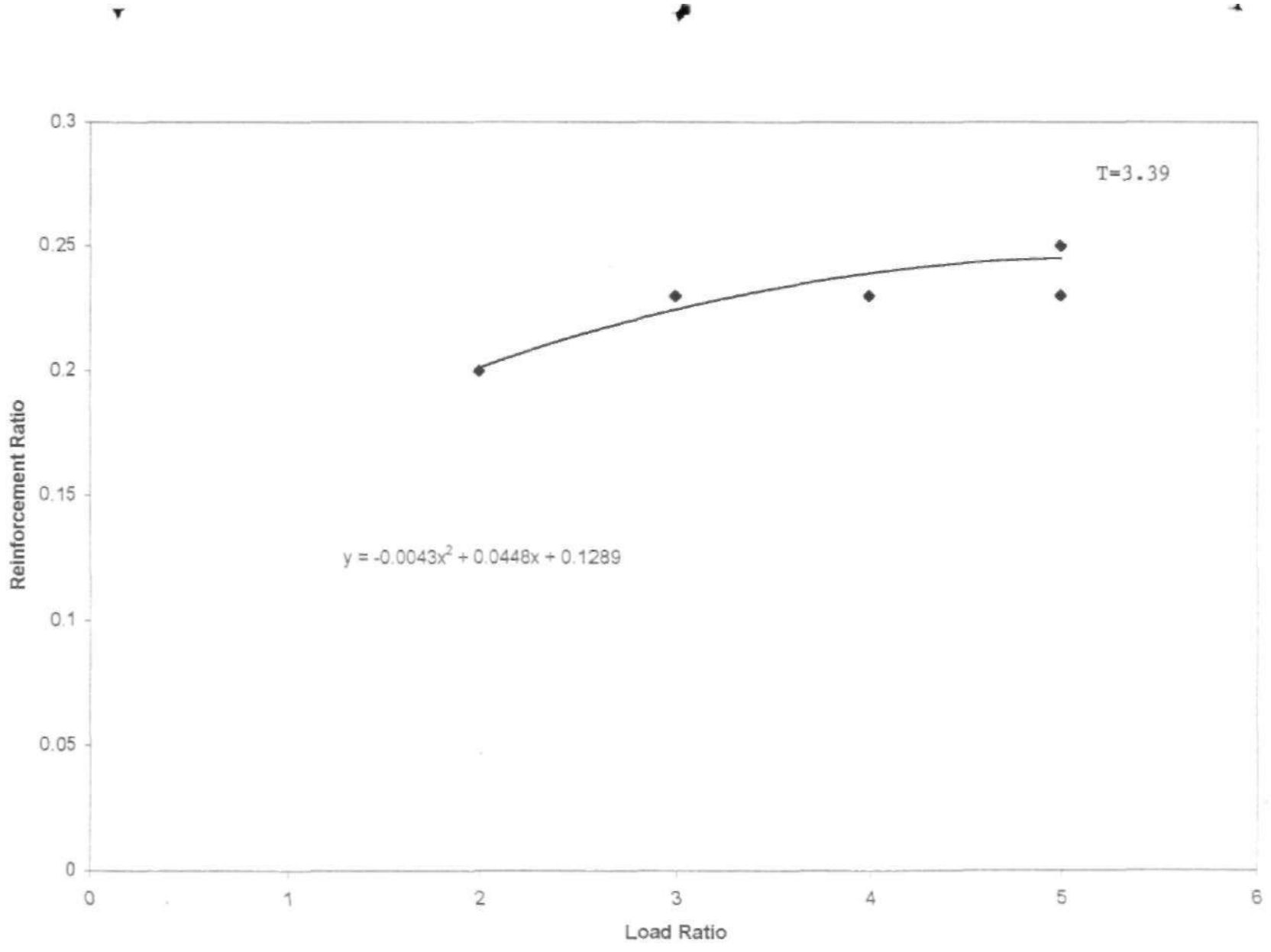


Fig. 5.15 Reinforcement Ratio versus Load Ratio for Propped Cantilevered Slab, Span= 3000mm, Fcu= 20 N/mm², Fy = 460 N/mm²

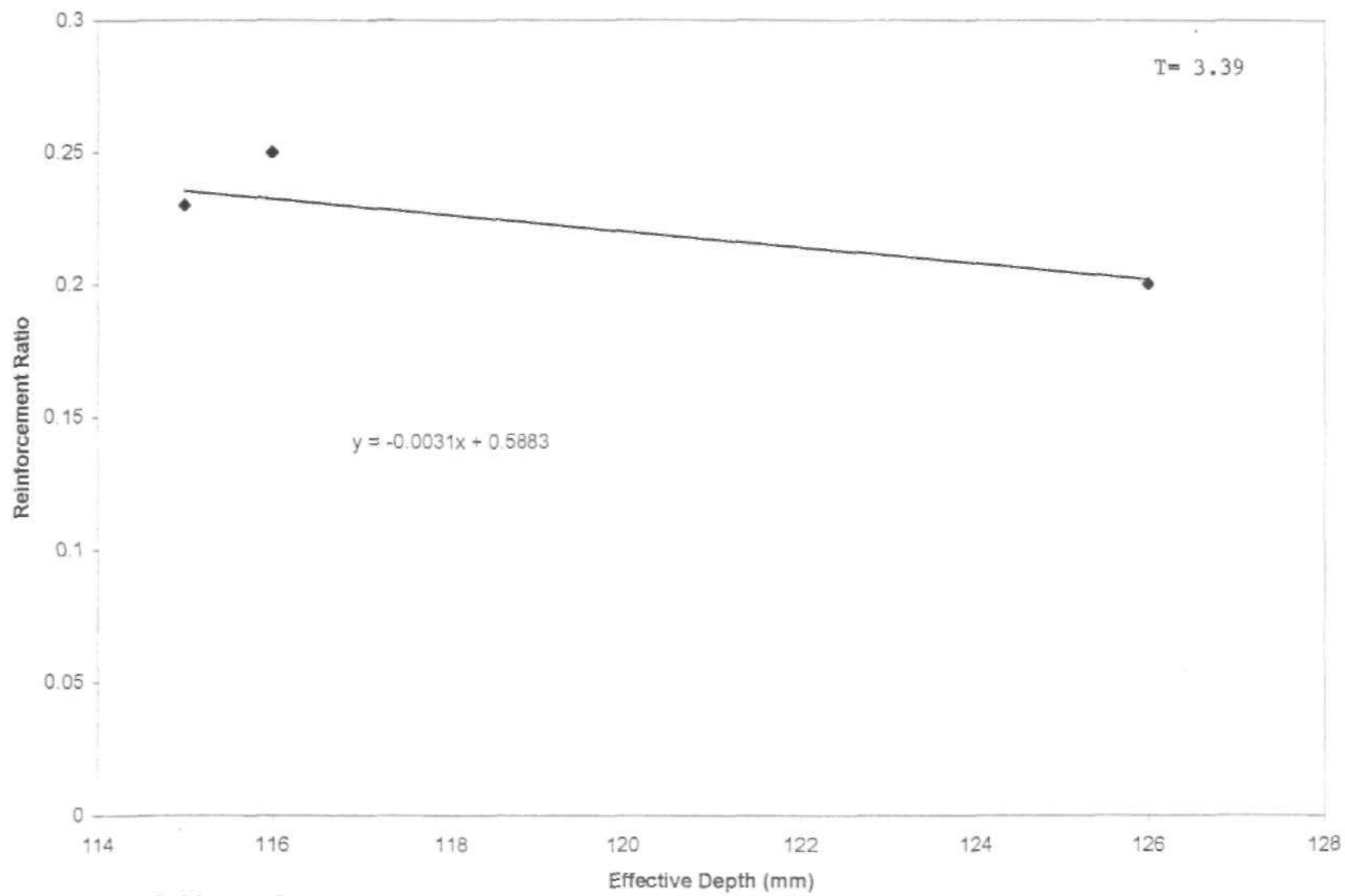


Fig. 5.16 Reinforcement Ration versus Effective Depth for propped Cantilevered Slab, Span = 3000mm, Fcu = 20 N/mm^2 Fy = 460 N/mm^2

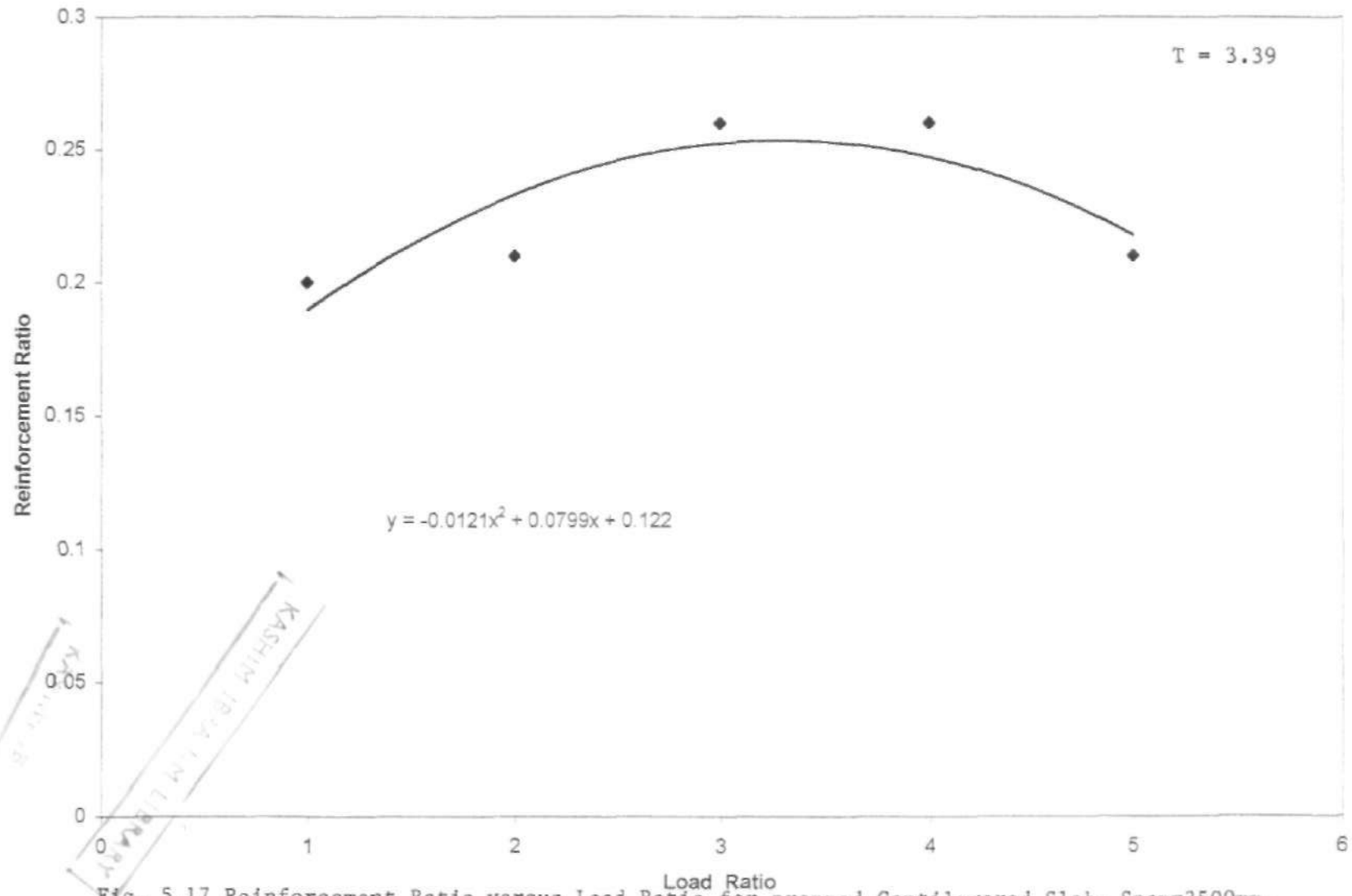


Fig. 5.17 Reinforcement Ratio versus Load Ratio for propped Cantilevered Slab, Span=3500mm, Fcu = 25 N/mm^2, Fy = 460 N/mm^2

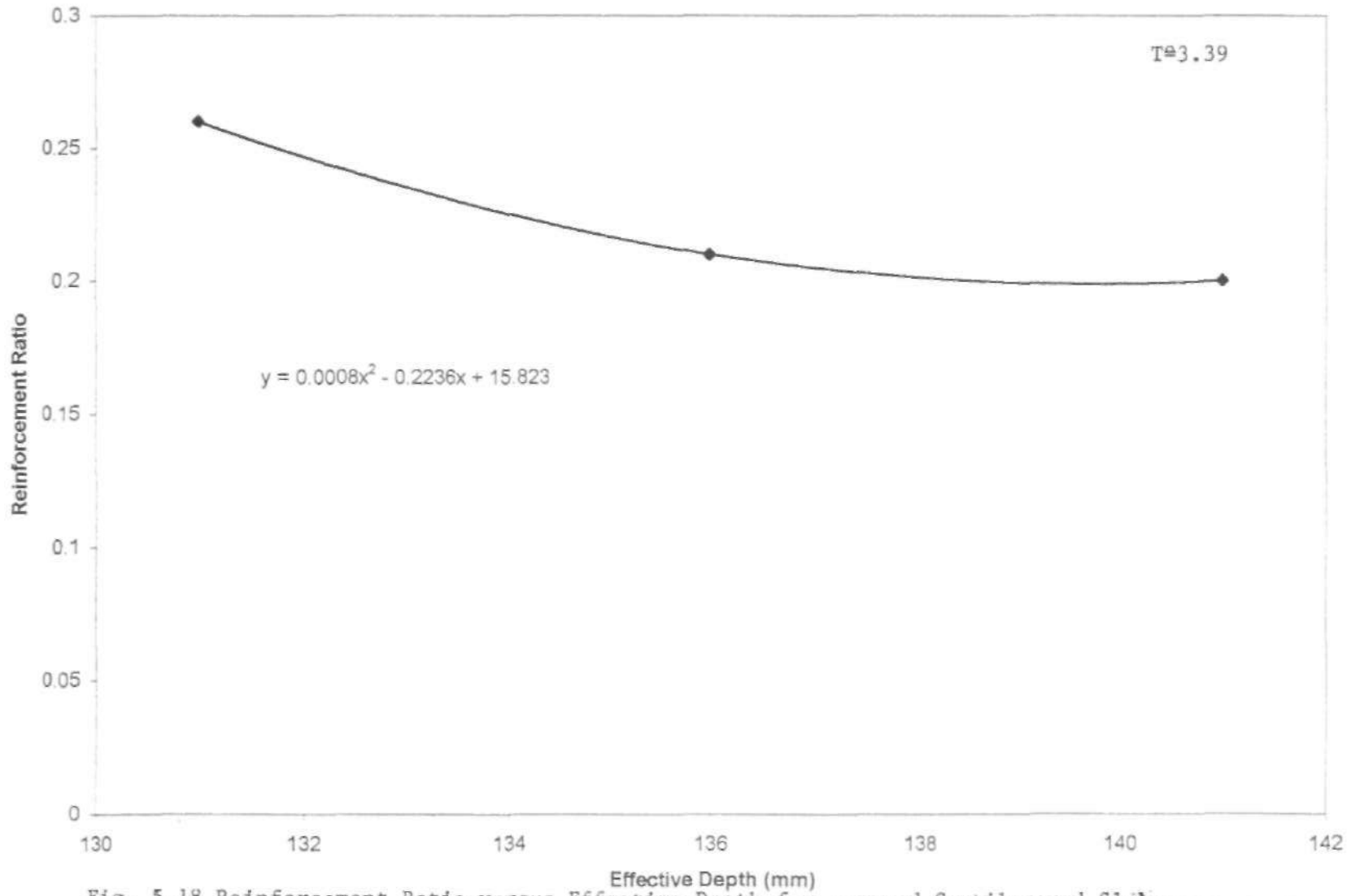


Fig. 5.18 Reinforcement Ratio versus Effective Depth for propped Cantilevered Slab - Span = 3500mm, Fcu = 25 N/mm2, Fy = 460 N/mm2

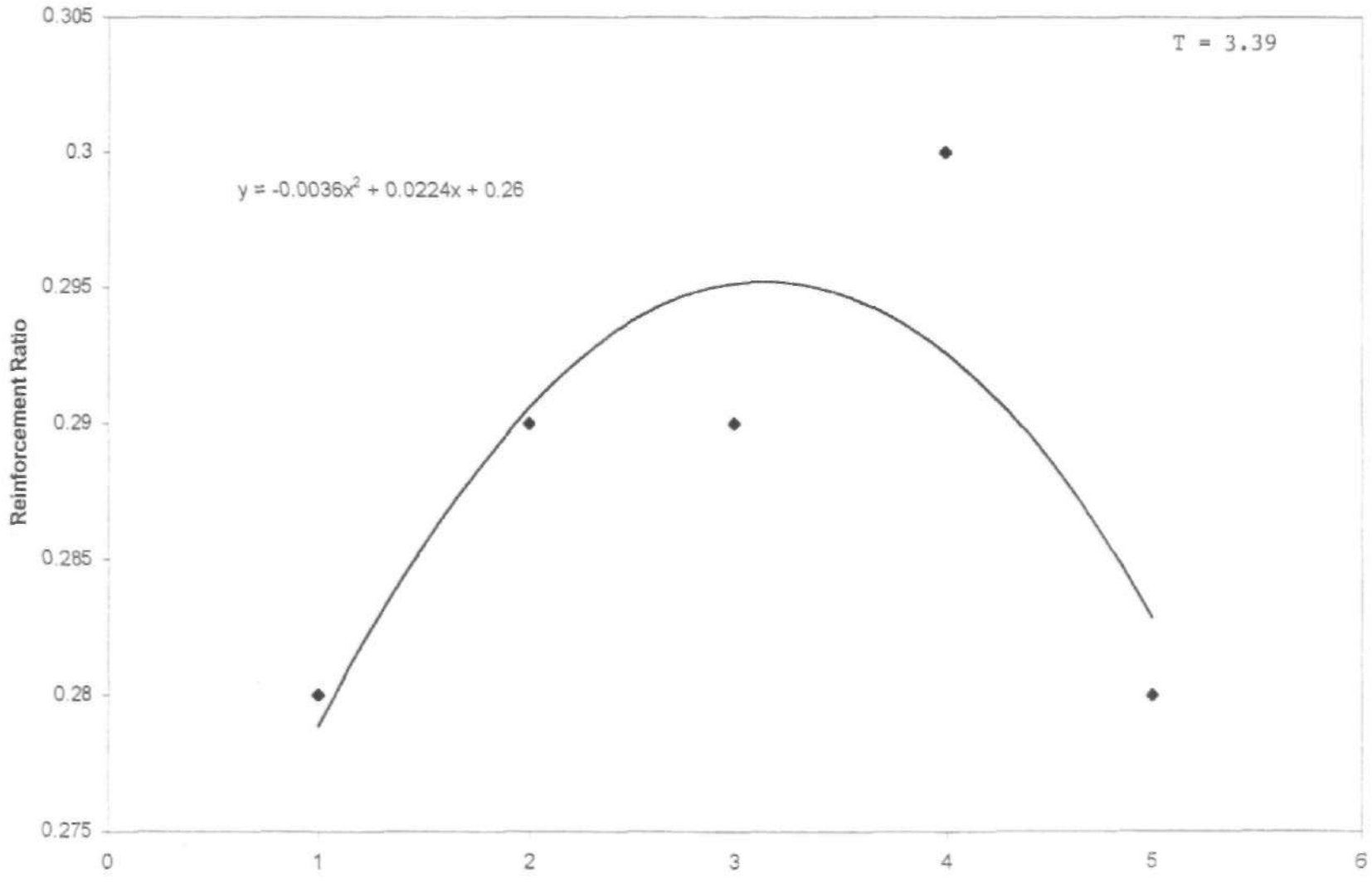


Fig. 5.19 Reinforcement Ratio versus Load Ratio for Fixed-Fixed Slab, Span = 4500mm, Fcu = 25 N/mm2 , Fy = 460 N/mm2

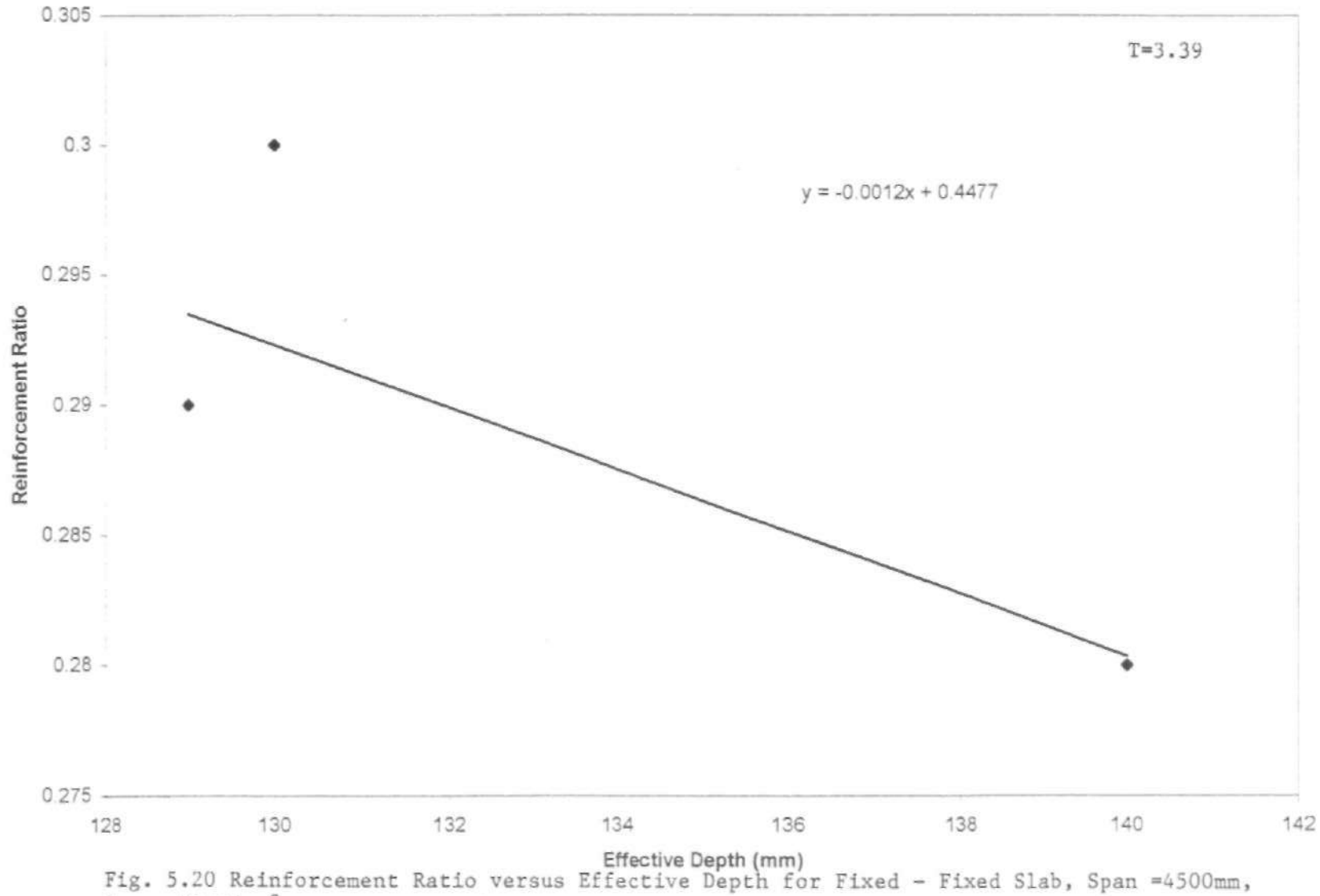


Fig. 5.20 Reinforcement Ratio versus Effective Depth for Fixed - Fixed Slab, Span =4500mm, Fcu = 25 N/mm^2, Fy = 460 N/mm^2

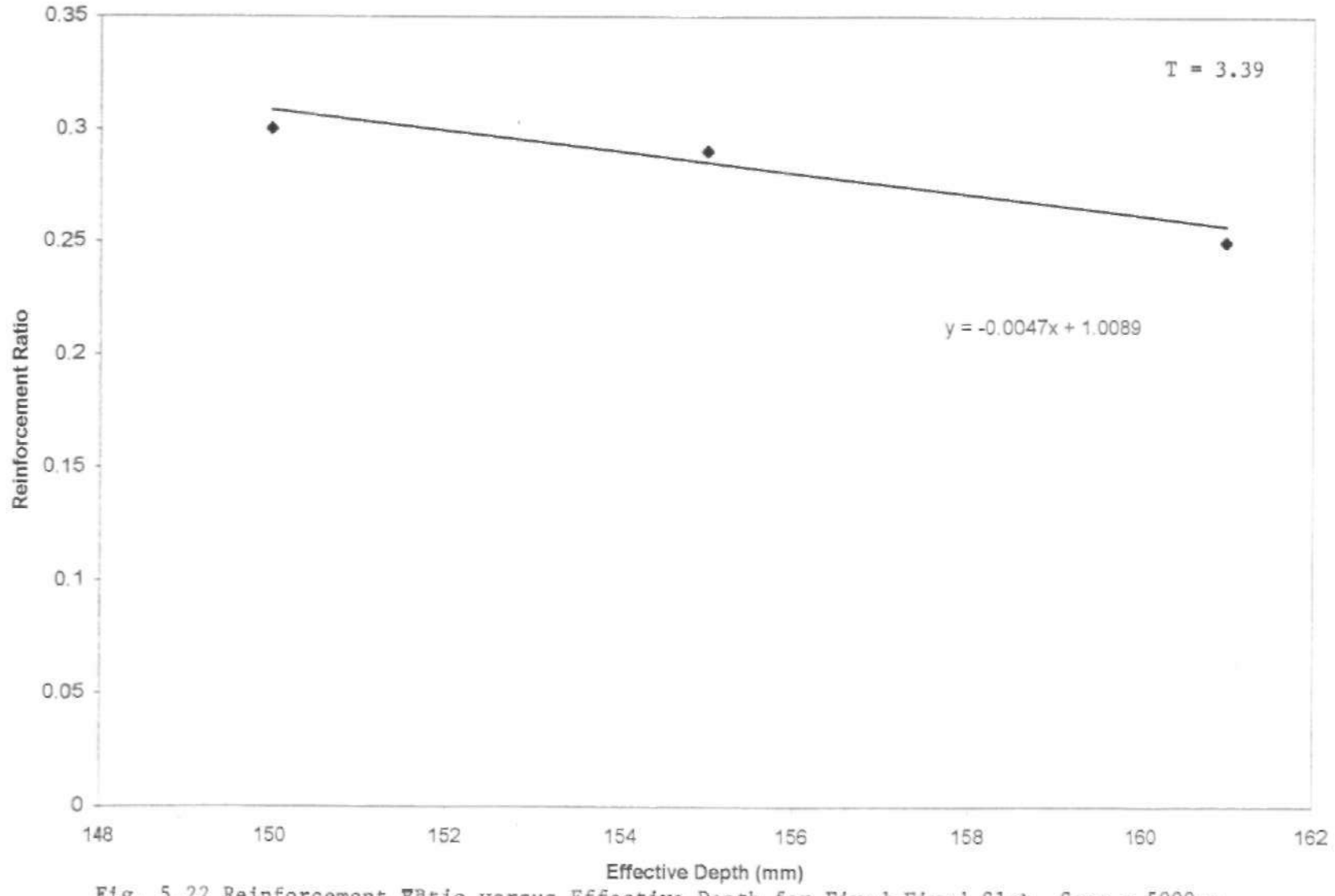


Fig. 5.22 Reinforcement Ratio versus Effective Depth for Fixed-Fixed Slab, Span = 5000mm, F_{cu} = 25 N/mm^2, Fy = 460 N/mm^2

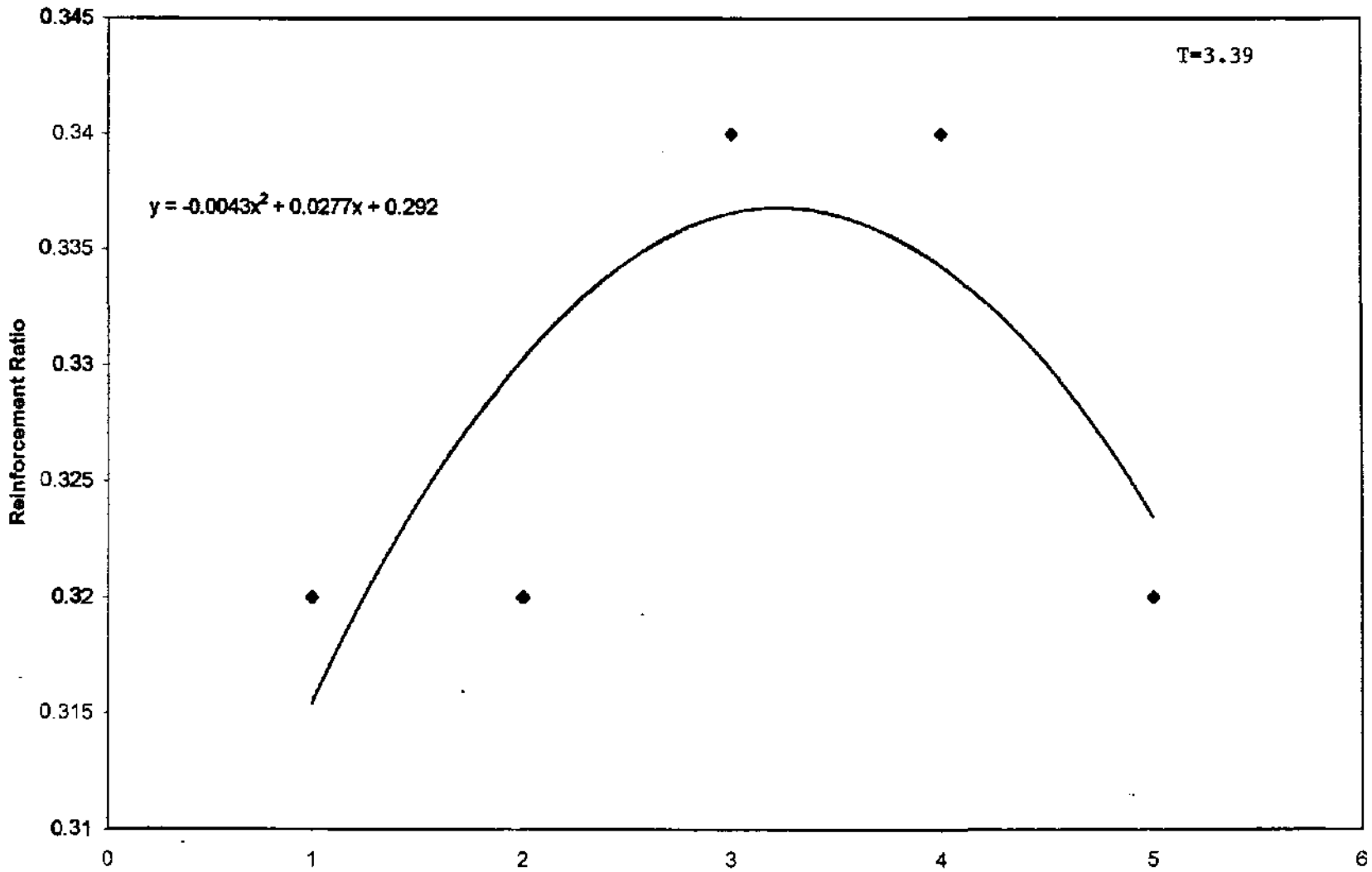


Fig. 5.23 Reinforcement Ratio versus Load Ratio for Fixed-Fixed Slab. Span = 5000mm, Fcu = 25 N/mm2, Fy = 460 N/mm2

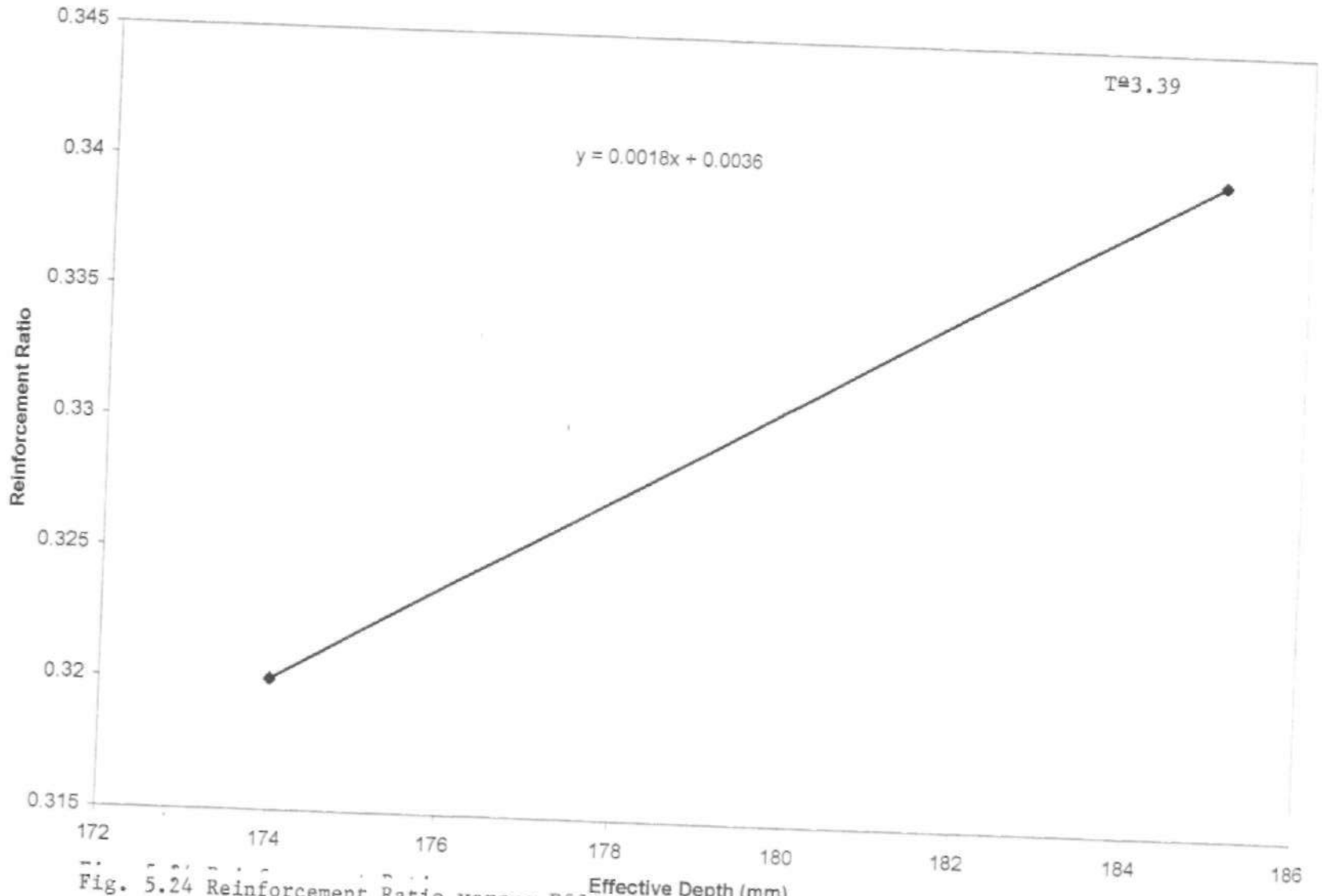


Fig. 5.24 Reinforcement Ratio versus Effective Depth for Fixed-Fixed Slab, Span =5000mm, Fcu = 25 N/mm2, Fy = 460 N/mm2

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

6.1 CONCLUSION

The design of reinforced concrete slabs at uniform reliability level have been presented with the aid of a computer program in FORTRAN-77 language. Suggestions and recommendations made by various institutions for the design of reinforced concrete slabs have been the watch word. First Order Reliability Method (FORM) of analysis of safety was strictly followed in the program.

On the basis of the results obtained from the program, design charts have been plotted. The design charts can practically be used for designing one-way slabs with varying spans, material properties and support conditions. The results obtained from such plots give designs at a constant reliability level.

The use of the charts developed in this thesis saves designer's calculation and processing time. In addition, because of the uncertainties that exist in the design variables, the designer is assured that his results have undergone reliability analysis and have satisfied the safety index format.

6.2 RECOMMENDATION

The program and the charts are recommended for design purposes for uniform reliability levels. It is believed that the adoption of the program and the use of the charts will

eliminate problems of over-and/or under-designs. Consequently, a trade-off between safety and economy can be properly formulated.

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