ISO-SAFETY DESIGN OF FLAT SLABS IN ACCORDANCE WITH EUROCODE 2

BY

Ibrahim ALIYU
***[M.Sc./ENG/2931/2011-2012*]**

**A THESIS SUBMITTED TO THE SCHOOL OF POSTGRADUATE STUDIES,
AHMADU BELLO UNIVERSITY
ZARIA-NIGERIA,**

**IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD
MASTER OF SCIENCE DEGREE IN CIVIL ENGINEERING (M.Sc. (CIVIL)).**

**DEPARTMENT OF CIVIL ENGINEERING
FACULTY OF ENGINEERING,
AHMADU BELLO UNIVERSITY
ZARIA-NIGERIA**

FEBRUARY, 2015

DECLARATION

I hereby declare that this thesis titled '' **Iso-safety Design of Flat Slabs in accordance with Eurocode 2**'' was done by me in the Department of Civil Engineering, under the supervision of Dr Idris Abubakar and Dr Amana Ocholi. It has not been previously presented for the award of any degree. All sources of information which are not originally mine are specially acknowledged by reference.

_____ _____

Ibrahim ALIYU Date

Name of Student

CERTIFICATION

This thesis titled “Iso-safety Design of Flat Slabs in accordance with Eurocode 2” by Ibrahim ALIYU, meet the requirements of the School of Postgraduate Studies, Ahmadu Bello University, Zaria, for the award of degree of Master of Science (M.Sc) in Civil Engineering.

Dr. I. Abubakar	**Date**
Chairman Supervisory Committee	
Dr. A. Ocholi	**Date**
Member Supervisory Committee	
Dr. Y.D. Amartey	**Date**
Head of Department	
Prof. Zoaka A. Hassan	**Date**
Dean, School of Postgraduate Studies.	

DEDICATION

I dedicate this work to the memory of my late father; Alhaji Aliyu Ibrahim, may His Soul rest in peace.

ACKNOWLEDGEMENTS

All thanks and praise are to Almighty Allah (SWT), the Beneficent, the Merciful. Special thanks and gratitude goes to my supervisor Dr. Idris Abubakar for his mentorship, guidance and encouragement during the course of this thesis and my entire stay in the University, I also extend my profound gratitude to my second supervisor, Dr Amana Ocholi for his valuable contributions and to Dr Y.D Amartey for his endless encouragement and support. I must at this point graciously acknowledge Dr. Jibrin Muhammed Kaura for mentoring, encouraging me and never letting me down, May Allah (SWT) be there for you as you have always been for me, Ameen; to Engr Adamu Lawan, for encouragement and being someone I can rely upon despite been far away, thank you so much.

I must mention and thank with all my heart my wife Khadijah Tanimu for being relentless in seeing I work on this thesis and providing me the most dependable partnership; to my child Fadimatu for the loss of my attention necessary for this task. My heart-felt gratitude goes to my mother Safiya Abdullahi thou no words can portray my gratitude. My Sincere appreciation and thanks for sundry support and assistance goes to the entire staff of Civil Engineering department, my colleagues, brothers and friends especially, Nuradeen Abdullahi, Engr Iliyasu Ibrahim, Engr Aliyu Usman, Engr A,A Murana, Engr Nasiru Mijinyawa, Engr Sadiq Muhammad, Engr Bashir Tanimu, Yawale Muhammad, Yakubu S. Ladan, Engr Ashiru Muhammad, Engr Abdulmumin Shuaibu, Engr Ahmad Magaji, Mustapha Abdullahi, Engr A.A Aliyu, Engr Abdulrasheed and Engr Y.K Galadima to mention some but few.

ABSTRACT

This research work focuses on the development of Iso-safety design charts for flexural design of flat slabs at predefined reliability levels in accordance with Eurocode 2 (2004) design criteria. Constitutive models for reinforcing steel and concrete were selected in accordance with the Eurocode 2 design requirements and subsequently the flexural limit state function was derived. Charts were developed for the flexural design of rectangular reinforced concrete sections with respect to the position of neutral axis ($\frac{x}{d}$) for each concrete grade (f_{ck}) and steel grade (f_{yk}). Uncertainties in loading and geometrical properties were obtained and a program was developed taken into consideration EC2 design requirements, a safety index β value of 1.81 was achieved for various points on each of the generated curve using First Order Reliability Method (FORM). Reliability-based design charts called Iso-safety charts were produced to target safety indices; β_T of 3.3, 3.8 and 4.3 as the minimum recommended for the three failure consequence classes by Eurocode 0 (2002). This recommendation shows that Eurocode 2 design of flat slabs considering flexural failure with safety index value of 1.81 provides designs that are below the recommended target safety indices. A flat slab was there after designed using the charts and was shown that for the same loading and geometrical considerations, the area of flexural reinforcement required increased by 40%, 55% and 75% over Eurocode 2 design for corresponding target safety indices of 3.3, 3.8 and 4.3 respectively. Sensitivity analysis of these provided reinforcements was carried out on other flat slab failure modes and was observed that at low reinforcement ratios punching shear safety is dependent majorly on the effective depths rather than the flexural reinforcement.

TABLE OF CONTENTS

Content	Page
Cover Page	
Title page	i
Declaration	ii
Certification	iii
Dedication	iv
Acknowledgement	v
Abstract	vi
Table of contents	vii
List of Figures	x
List of Tables	xii
List of Appendices	xiv
Notation	xv
CHAPTER ONE: INTRODUCTION	
1.1 General	1
1.2 Problem Statement And Justification Of Study	3
1.2.1 Statement of Research Problem	3
1.2.2 Justification of the Study	4
1.3 Aim and Objectives	5
1.3.1 Aim	5
1.3.2 Objectives	6
1.4 Scope of the Research	6
CHAPTER TWO: LITERATURE REVIEW	
2.1 Structural Reliability and Iso-safety	7
2.2 Limit State Design	10
2.2.1 Ultimate Limit State	10
2.2.2 Serviceability Limit States	10

2.3 Flat Slabs 11
2.3.1 Component of Flat Slabs 12
2.3.2 Benefits of Reinforced Concrete Flat Slab 12
2.4 Eurocode2 (2004) Design Provisions for Flat slabs 12
2.4.1 Flexure in Reinforced Concrete Flat Slab 13
2.4.2 The Procedure for Calculating Flexural Reinforcement 14
2.4.3 Punching Shear in Reinforced Concrete Flat Slabs 16
2.4.4 The Procedure for punching shear check and Reinforcement Determination 18
2.4.5 Deflection in Flat Slabs 21
2.4.6 The Procedure for Deflection Check of Flat slabs 21
2.5 Methods of Reliability Analysis 23
2.6 Reliability based design 24
2.6.1 Target Reliability 25
2.6.2 Consequence of Failure or Malfunctioning of structures and their classes 25
2.6.3 Reliability Classes 26

CHAPTER THREE: METHODOLOGY

3.1 Constitutive Models 27
3.1.1 Concrete Constitutive Model 27
3.1.2 Steel Constitutive Model 28
3.2 The Rectangular Stress Block and Design Equation 29
3.3 Design Equation for Charts 31
3.4 First Order Reliability Method (FORM) 34
3.5 Computation of Reliability Index 35
3.6 Limit State Functions 37
3.6.1 Limit State Function for Iso-safety Design Charts 37
3.6.2 Limit State Function for Punching Shear 37
3.6.3 Limit State Function for Deflection 39
3.7 Analysis Procedure 40
3.7.1 Iso-safety Charts 40
3.7.2 Punching Shear 41

3.7.3 Deflection 42
3.8 Program Flow-chart for Iso-safety Charts 43

CHAPTER FOUR: RESULTS AND DISCUSSION
4.1 Analysis of Results 45
4.1.1 Plot of Eurocode2 Design charts 45
4.1.2 Estimated Safety index for the Generated Eurocode2 Design Charts 47
4.2 Iso-safety Design Charts 47
4.3 Punching Shear and Deflection 52
4.3.1 Effect of Flexural Reinforcement on First Critical section Punching Shear safety 53
4.3.2 Effect of Slab Effective Depth on Punching Shear Safety at First Critical Section 54
4.3.3 Effect of Varying Concrete Grade on Punching Shear safety 55
4.3.4 Effect of Load Ratio (Variable to Permanent) on Punching Shear Safety 56
4.3.5 Effect of Column Head Size on Safety Index 57
4.3.6 Effect of Flexural Reinforcement on Critical Section from Panel Drop Punching Shear Safety 58
4.3.7 Effect of Slab Effective Depth on Punching Shear safety of Critical Section From Panel Drop 59
4.3.8 Effect of Panel Drop Size on the safety of Critical Section from Panel Drop 60
4.3.9 Effective depth Effect on Column Face Safety 61
4.3.10 Effect of Flexural reinforcement on Deflection 61
4.3.11 Effect of Varying Concrete Grade on deflection 62
4.3.12 Safety Index (on Deflection) Variation with Slab Effective Depth 63
4.3.13 Effect of Slab length on Deflection safety 64
4.4 Illustrative Example on the Use of the Charts 65

CHAPTER FIVE: CONCLUSION AND RECOMMENDATIONS
5.1 Conclusion 68
5.2 Recommendations 69

REFERENCES 71

LIST OF FIGURES

Figure 2.1: Types of Flat Slabs 11

Figure 2.2: Recommended values of β 17

Figure 2.3: Punching Shear layout 20

Figure 2.4: Basic Span-Effective depth ratios 22

Figure 3.1: Parabolic-rectangular stress-strain diagram for concrete in compression 28

Figure 3.2: Short-term design stress-strain curve for reinforcement 28

Figure 3.3: Singly reinforced section with rectangular stress block 29

Figure 3.4: Program flow-chart for Iso-safety charts 44

Figure 4.1: Design Chart for singly reinforced rectangular sections f_{yk}=410N/mm^2 45

Figure 4.2: Design Chart for singly reinforced rectangular sections f_{yk} =460N/mm^2 46

Figure 4.3: Design Chart for singly reinforced rectangular sections f_{yk} =500N/mm^2 46

Figure 4.4: Iso-safety Design chart for singly reinforced concrete section, β=3.3 and f_{yk} = 410N/mm^2 48

Figure 4.5: Iso-safety Design chart for singly reinforced concrete section, β_T=3.3 and f_{yk} = 460N/mm^2 48

Figure 4.6: Iso-safety Design chart for singly reinforced concrete section, β_T =3.3 and f_{yk} = 500N/mm^2 49

Figure 4.7: Iso-safety Design chart for singly reinforced concrete section, β_T =3.8 and f_{yk} = 410N/mm^2 49

Figure 4.8: Iso-safety Design chart for singly reinforced concrete section, β_T =3.8 and f_{yk} = 460N/mm^2 50

Figure 4.9: Iso-safety Design chart for singly reinforced concrete section, β_T =3.8 and f_{yk} = 500N/mm^2 50

Figure 4.10: Iso-safety Design chart for singly reinforced concrete section, β_T =4.3 and f_{yk} = 410N/mm^2 51

Figure 4.11: Iso-safety Design chart for singly reinforced concrete section, β_T =4.3 and f_{yk} = 460N/mm^2 51

Figure 4.12: Iso-safety Design chart for singly reinforced concrete section, β_T =4.3 and f_{yk} = 500N/mm^2 52

Figure 4.13: Safety Index versus Reinforcement ratio (with and without drop panel at the First critical section from column face) 54
Figure 4.14: Safety Index versus effective depth for different target safety index of Flexural reinforcement at the first critical section from column face 55
Figure 4.15: Safety index versus Concrete Grade at the three punching shear sections of a flat slab 56
Figure 4.16: Safety index versus Load ratio at the three punching shear sections of a Flat slab 57
Figure 4.17: Safety index versus column head diameter (at the first critical section and at the column face of a flat slab) 58
Figure 4.18: Safety index versus reinforcement ratio at the critical section from Drop panel of flat slab 59
Figure 4.19: Safety index versus Effective Depth for Different Target Safety Index of Flexural reinforcement at the critical section from drop panel 60
Figure 4.20: Safety index versus Drop dimension at the critical section from the Drop Panel of a Flat Slab 60
Figure 4.21: Safety index versus Effective Depth at the Column face of flat slab 61
Figure 4.22: Safety index versus Reinforcement ratio for the deflection of flat slab 62
Figure 4.23: Safety index versus Concrete Grade for the deflection of flat slab 63
Figure 4.24: Safety index versus Effective depth for the deflection of flat slab 63
Figure 4.25: Safety index versus Slab length for the deflection of flat slab 64

LIST OF TABLES

Table 2.1: Bending Moment Coefficients for flat slabs 14

Table 2.2: Values of K' 15

Table 2.3: z/d for singly reinforced rectangular sections 15

Table 2.4: Division of moments between strips 16

Table 2.5: Minimum percent reinforcement requirement 16

Table 2.6: Values for $v_{Rd,max}$ 19

Table 2.7: Values for $f_{ywd,ef}$ 19

Table 2.8: Factors to be used with table 2.9 for $f_{ck} \neq 30$ 19

Table 2.9: $v_{Rd,c}$ resistance of members without shear reinforcements 20

Table 2.10: Target Reliability β for class RC2 structural Members 25

Table 2.11: Definition of Consequence classes 25

Table 2.12: Recommended minimum values for Reliability index β (Ultimate limit state) 26

Table 3.1: Statistical models of basic design variables 41

Table 4.1: Summary of Design 67

Table A1: Results from which developed EC2 chars are plotted for $f_{yk} = 410\text{N/mm}^2$ 76

Table A2: Results from which developed EC2 chars are plotted for $f_{yk} = 460\text{N/mm}^2$ 77

Table A3: Results from which developed EC2 chars are plotted for $f_{yk} = 500\text{N/mm}^2$ 77

Table B1: Results from which iso-safety charts are plotted for for $f_{yk} = 410\text{N/mm}^2$ and $\beta_T = 3.3$ 78

Table B2: Results from which iso-safety charts are plotted for for $f_{yk} = 460\text{N/mm}^2$ and $\beta_T = 3.3$ 78

Table B3: Results from which iso-safety charts are plotted for for $f_{yk} = 500\text{N/mm}^2$ and $\beta_T = 3.3$ 79

Table B4: Results from which iso-safety charts are plotted for for $f_{yk} = 410\text{N/mm}^2$ and $\beta_T = 3.8$ 79

Table B5: Results from which iso-safety charts are plotted for for $f_{yk} = 460\text{N/mm}^2$ and $\beta_T = 3.8$ 80

Table B6: Results from which iso-safety charts are plotted for for $f_{yk} = 500\text{N/mm}^2$ and $\beta_T = 3.8$ 80

Table B7: Results from which iso-safety charts are plotted for for $f_{yk} = 410\text{N/mm}^2$

and $\beta_T = 4.3$ 81
Table B8: Results from which iso-safety charts are plotted for for $f_{yk} = 460N/mm^2$ and $\beta_T = 4.3$ 81
Table B9: Results from which iso-safety charts are plotted for for $f_{yk} = 500N/mm^2$ and $\beta_T = 4.3$ 82
Table C1: Load ratio versus safety index at the Column face, First critical section from Column face and critical section from panel drop 83
Table C2: Concrete Grade versus safety index at the first critical section, critical section From drop panel and the column face 83
Table C3: Reinforcement ratio versus safety index at the first critical section from the Column face, with and without drop panel 84
Table C4: Safety index versus effective depth at the first critical section (when slab is Designed using iso-safety charts targeting different safety index values) 84
Table C5: Safety index versus Column head diameter at the first critical section and Column face 85
Table C6: Safety index versus effective depth at the column face 85
Table C7: Safety index versus reinforcement ratio at the critical section from drop panel 86
Table C8: Safety index versus effective depth at critical section from drop panel (slab Designed using iso-safety charts targeting different safety index values) 86
Table C9: Safety index versus drop dimension at the critical section from drop panel 87
Table C10: Safety index versus reinforcement ratio for the deflection of flat slab 87
Table C11: Safety index versus concrete grade for the deflection of flat slab 88
Table C12: Safety index versus effective depth for the deflection of flat slab 88
Table C13: Safety index versus Slab length for the deflection of flat slab 88

LIST OF APPENDICES

Appendix A: Results from which EC2 charts are plotted 73

Appendix B: Results from which Iso-safety charts are plotted 74

Appendix C: Results of Reliability Analysis 79

Appendix D: Programs Listings 85

SYMBOL	NOTATIONS
v_{Ed}	Applied ultimate Shear stress
A_{sw}	Area of punching shear reinforcement
A_{St}	Area of tension reinforcement
b_f	Breath of flange
b_w	Breath of web
M	Bending Moment
b	Breath of section
d_1	Column head diameter
ρ'	Compression Reinforcement ratio
F_{cc}	Compressive force
$v_{Rd,c}$	Concrete punching shear capacity
ε_{cc}	Concrete Strain
f_{cu}	Concrete Strength(Grade)
CC	Consequence Class
A_c	Cross-sectional area of concrete
S	Depth of Compression zone
X_i	Design Basic Variable, I
F	Design load
X^*	Design Point Variable
f_{yd}	Design Steel strength
V_{ED}	Design value of punching shear force
d	Effective depth
$f_{ywd,ef}$	Effective design strength of shear reinforcement
E_s	Elastic modulus of steel
α_i	Factor of Influence for a variable
a and b	Length and breadth of drop respectively (d_4)
$U_{out,ef}$	Length of outer perimeter where shear reinforcement is not required
Z_1	Lever arm
G(x) or Z	Limit State Function
θ_S	Load model uncertainty

S	Loading effect
ε_{cu2}	Maximum Concrete strain
$v_{Rd,max}$	Maximum Permissible shear stress
A_{smax}	Maximum reinforcement required
$v_{Ed,max}$	Maximum Shear stress
f_{ctm}	Mean concrete tensile strength
μ	Mean Value
A_{smin}	Minimum reinforcement required
x	Neutral axis depth
γ_c	Partial factor of safety for concrete
γ_s	Partial factor of safety for steel
U_i	Perimeter at the section under consideration
G_k	Permanent Load
Z	Performance function
P_f	Probability of Failure
$A_{s,prov}$	Provided flexural reinforcement
S_r	Radial Spacing of reinforcement
δ	Redistribution ratio
ρ^o	Reference reinforcement ratio ($\sqrt{f_{ck}}\ 10^{-3}$
RC	Reliability Class
β	Reliability index
$A_{s,reqd}$	Required flexural reinforcement
R	Resistance
θ_R	Resistance model uncertainty
SLS	Serviceability Limit State
L	Slab span
L_{eff}	Slab Effective span
σ	Standard Deviation
β_1	Standard factors for column positions
X	Standardized variable

ε_s	Steel strain
f_{yk}	Steel Strength
β_T	Target reliability index
F_{st}	Tensile force
ρ	Tensile Reinforcement ratio
Q_k	Variable Load
σ^2	Variance
ε_y	Yield strain

CHAPTER ONE

INTRODUCTION

1.1 GENERAL

Flat-slab system of construction is one in which the beams used in the conventional methods of constructions are done away with. The slab directly rests on the columns and load from the slab is directly transferred to the columns and then to the foundation (Anitha *et al.*, 2007).

The Common practice of design and construction is to support the slabs by beams and support the beams by columns. This may be called beam-slab construction. The beams reduce the available net clear ceiling height. Hence in warehouses, offices and public halls sometimes beams are avoided and slabs are directly supported by columns. This type of construction is also aesthetically appealing. The slabs which are directly supported by columns are called flat slabs.

For many years, it has been assumed in the design of structural systems that all loads and strengths are deterministic. The strength of an element is determined in such a way that it exceeded the load with a certain margin. The ratio between the strength and the load was denoted as the safety factor which is considered as a measure of the reliability of the structure. In codes of practice for structural systems, values for loads, strengths as well as safety factors are prescribed (Sorensen, 2004).

The safety factors are traditionally determined on the basis of experience and engineering judgment. However, in recent codes such as Eurocode 2 partial safety factors are used. Characteristic values of the uncertain loads and resistance are specified and partial safety

factors are usually based on experience or calibrated to existing codes or to the measures of the reliability obtained by probabilistic techniques.

Eurocode 2 establishes principles and requirements for safety, serviceability and durability of structures. It uses a statistical approach to determine realistic values for actions that occur in combination with each other. Partial factors for actions are given in this Eurocode, whilst partial factors for materials are prescribed in other relevant Eurocode. (Anitha *et al.,* 2007).

The beam design chart for singly reinforced beams in part 3 of BS8110 (1997) is equally in use for the design of slabs as singly reinforced beam has similar design procedure to a slab. The charts have been prepared in accordance with the assumption laid down in the code, with the intention that they may be used as standard charts and so avoid duplication of efforts by individual design offices (Lukman, 2012*).* BS8110 Part 3 provides design charts for singly reinforced beams, doubly reinforced beams and rectangular columns. This design charts cannot be used to obtain the complete detailed design of any member but they may be used as an aid when analyzing the cross section of a member at the ultimate limit state (Lukman, 2012*).* The charts have been based on the assumptions laid down in BS8110 Part 1, use been made of the parabolic-rectangular stress block throughout (BS8110, 1997).

The design of flat slabs in accordance with Eurocode 2 (EC 2) is essentially similar with BS 8110. However, the layout and content of Eurocode 2 will initially appear alien to designers familiar with BS 8110 (Moss *et al.,* 2006). EC 2 does not contain the derived formulae or specific guidance on determining moments and shear forces; this has arisen because it has been European practice to give principles in the codes and for the detailed application to be presented in other sources.

Eurocodes do not provide design charts for design of slabs, beams and columns, but Mosley *et al.,* (2007) generates doubly reinforced concrete beam design chart and an interaction diagram for columns were however developed and generated (Mosley *et al.,* 2007; Reynolds *et al.,* 2008).

This research however focuses on the generation of Iso-safety design charts considering EC 2 design requirements for flat slabs at predefined Target safety index values using First Order Reliability Method (FORM)

1.2 PROBLEM STATEMENT AND JUSTIFICATION OF STUDY

1.2.1 Statement of Problem

According to Ditlevsen and Madsen (2005) engineering judgment is the art of being able to decide whether results obtained from a structural analysis or design model is sufficiently realistic that the engineer dare base his practical decisions on these results. Also, Abubakar and Pius (2007) observed that the aim of a structural design is to produce design and drawings for a safe and economical structure that fulfills its intended purpose.

The risk of reaching any of the limit states (ultimate and serviceability) should be avoided in design (Juned, 2003) as it is extremely difficult to define risk in a single set of words because of the high level of confusion surrounding the aspects of the subject. In general, risk could be established in qualitative and quantitative aspect. The latter is usually termed Engineering Risk Analysis. It is important to the observer that qualitative aspect of risk conveys a level understanding about failure or success of some defined event. In such way, risk comes relative to hazard and safeguards, where hazard is defined as a source of damage or injury. This brings the need to design reliable structures with the level of reliability known.

Reliable design is required because according to Ibrahim (2009), the design of civil engineering structures is full of uncertainties starting right from the material whose actual value (characteristics value) varies from point to point within the material to the load whose actual extent and variation cannot be accurately predicted. The design is undertaken on the basis of code recommendation.

Lukman (2012) observed design is a decision making process. Consequently, design problems, in contrast to analytical problems rarely have unique solutions. Hence, designers endeavor to optimize design to achieve important objectives that would satisfy operational and economic requirements within acceptable safety margins

1.2.2 Justification of the Study

According to Lukman (2012) Iso-safety design charts provide sections with uniform reliability. This is because they are formulated such that they have uniform safety levels which is not the case with the original BS 8110 Part 3 (1997) beam design charts. The charts will enable designers to be in a position to stipulate safety indices in line with specific demands rather than be restricted to an 'arbitrary' level of performance which is not specified. Where a greater loss is envisaged in the unlikely event of failure, engineers would be able to specify higher values of target safety indices in the design. The Iso-safety or reliability-based design charts thus fulfill one of the cardinal aims of engineering design which is the assurance of system performance including safety.

The study also noted that unlike the BS 8110-3 (1997) beam design charts, the proposed charts afforded explicit information on the safety of the beam being designed. The choice of

the target reliability index is made to correspond to any value recommended for the ultimate limit state in majority of structural reliability literature.

Abubakar (1999) proposed iso-safety design charts for the design of one way slabs in accordance with BS8110 (1985). It was recommended that there was the need to design the slabs using the iso-safety charts to ensure a compromise between safety and economy in design.

Abejide (2014) carried out a reliability analysis considering bending, shear and deflection criteria of reinforced concrete slabs and observed that the safety margins proposed for singly reinforced concrete slabs using CP110 (1972); BS8110 (1985) and Eurocode 2 (2004) codes for design are not achieved at all. The reliability levels were also found to be non uniform, thus the current design formulations are not as safe and reliable as predicted, therefore, the design formulations in these codes need a review so that they can at least meet the target for approved structural safety.

It is therefore based on the above that there was the need to develop Iso-safety charts for the design of flat slabs in accordance with EC 2. This will allow designers of the flat slabs to design with certainty of safety and economy of the slab being designed.

1.3 AIM AND OBJECTIVES

1.3.1 Aim

The aim of this research work is to propose Iso-safety design charts for flat slabs at predefined reliability levels in accordance with Eurocode 2 design criteria.

1.3.2 Objectives

The research objectives are to:

i. Identify the modes of failure of reinforced concrete flat slab
ii. Obtain suitable equations based on the requirements of EC-2 (2004) that will serve as the limit state function in the reliability analysis.
iii. Determine the implied safety level of the slab design equation to EC-2 (2004) using First Order Reliability Method (FORM)
iv. Generate a reinforced concrete flat slab flexural design interaction curve to EC-2.
v. Develop an algorithm for the generation of flat slab design interaction curve at various target safety levels
vi. Conduct sensitivity analysis on the basic design variables

1.4 SCOPE OF THE RESEARCH

The research work covers the generation of Iso-safety design charts considering concrete strengths, f_{cu} = 25, 30, 35 and 50 N/mm^2 and reinforcing steel strengths, f_y = 410, 460 and 500 N/mm^2 respectively according to EC-2 (2004) for the flexural design of reinforced concrete flat slabs and reliability analysis.

CHAPTER TWO

LITERATURE REVIEW

2.1 STRUCTURAL RELIABILITY AND ISO-SAFETY

Reliability is often understood as the probability that a structure will not fail to perform its intended function over a given period of time. Structural design has been moving towards a more rational and probability-based approach referred to as limit state design; such design takes into account more information than the deterministic methods in designing structural components (Melchers, 1999)

The study of structural reliability is concerned with the calculation and prediction of the probability of limit state violation for engineered structures at any stage during their life. In particular, the study of structural safety is concerned with the violation of the ultimate or serviceability limit states for the structure (Madsen *et al.*,1986). Reliability is therefore the branch of structural engineering which is concerned with the analysis and probabilistic assessment of design random variables in order to predict whether specified limit state would be violated and in doing this, uncertainties inherent in structural design have to be taken into consideration (Doty,1985).

Muhammed and Magaji (2010) came up with reliability-based design charts for singly reinforced concrete sections (Beams and Slabs) based on the ultimate moment of resistance with respect to the concrete sections.

An Iso-safety function on the other hand represents a selection of appropriate design parameters based on a prescribed reliability level of the structural element against the occurrence of a specified limit state (Lukman, 2012).

Lukman (2012) focused on the generation of the charts from an analytical perspective: by theoretical formulation of the bending moment capacity of any rectangular reinforced concrete section.

Uche (2000) developed a model for the Iso-safety design of reinforced concrete columns to BS8110 (1985) design criteria and opined that a development of this approach requires a standard safety level or rather target reliability index (β) be identified as a quantitative measure of acceptable performance with regard to a specific limit state which is the Iso-safety function.

Afolayan (2005) observed that Natural phenomena shows that loadings and other parameters which are concerned in structural design vary in value such as the strength of any given material and the sizes of the identical units. It is therefore necessary to systematically quantify uncertainties and apply them in design process.

Ema *et al.,* (2004) carried out an assessment of the seismic behaviour of RC flat slab building structures using two pseudo-dynamic tests on a full scale flat-slab model of a three storey RC structure which is a representative of existing flat-slab structures in European seismic areas and underlined that these structures exhibit significant higher flexibility compared to traditional frame structures becoming more sensitive to second order effects. They also stated that in order to limit deformation demands under earthquake excitations, combination with other stiffer structural systems as shear-walls is advisable.

Abubakar, (2006), after conducting a reliability analysis of structural design parameters of strip footings to BS8110 (1997) found that the minimum reinforcement ratio of 0.2% recommended by the code of practice for this type of footing is only safe at higher effective depths; at lower effective depths, reinforcement ratios between 0.3%and 0.4% are safer, Also, the safety indices considering bottom reinforcements are higher than those for top reinforcements, this is because of the moment coefficient given by the code that needs to be investigated. This indicates that code recommendations should not be relied upon in terms of uniform safety as they do not provide a uniform structural safety level and probabilistic approach and reliability-based design recommended.

Atim (2006) carried out a reliability analysis of the BS 8110 (1985) column design charts and also revealed that the reliability indices are not uniform for points on the existing BS 8110 (1985) charts and that the reliability indices are rather low for ultimate limit state consideration.

Melchers (1999) is of the opinion that real structures only rarely fail in a serious manner, but when they do, it is often due to causes not directly related to the predicted nominal loading or strength probability distribution considered. Other factors such as human errors, negligence, poor workmanship or neglected loadings are most often involved and also Structural failure might be considered to be the occurrence of one or more types of undesirable structural responses including the violation of predefined limit states. Thus collapse of all or part of a structure, major cracking and excessive deflection are some possible forms of failure.

2.2 LIMIT STATE DESIGN

The purpose of design is to achieve acceptable probabilities that a structure will not become unfit for its intended use, that is, it will not reach a limit state. Thus any way in which a structure may cease to be fit for use will constitute a limit state and the design aim is to avoid any of such conditions being reached during the expected life the structure (Mosley *et al.*, 2007).

2.2.1 Ultimate Limit State

Ultimate limit state requires that the structure must be able to withstand, with an adequate factor of safety against collapse, the loads for which it is designed to ensure safety of the building occupants and/or the safety of the structure itself.(Mosley *et al.*, 2007).

2.2.2 Serviceability Limit States

The most important serviceability limit states are:

(1) Deflection: - The appearance or efficiency of any part of the structure must not be adversely affected by deflection nor should the comfort of the building users be adversely affected

(2) Cracking: - Local damage due to cracking and spalling must not affect the appearance, efficiency or durability of the structure.

(3) Durability: - This must be considered in terms of the proposed life of the structure and its conditions of exposure. (Mosley *et al.*, 2007).

Other limit states that may be reached includes: -

(4) Excessive vibration- which may cause discomfort or alarm as well as damage

(5) Fatigue- This should considered is cyclic loading is likely.

(6) Fire resistance- Must be considered in terms of resistance to collapse, flame penetration and heat transfer

(7) Special circumstances- any special requirement of the structure which are not covered by any of the more common limit states must be taken into account (e.g. earthquake resistance) (Mosley *et al.*, 2007)

2.3 FLAT SLABS

A flat slab is a reinforced concrete slab supported directly by concrete columns without the use of intermediary beams. The slab may be of constant thickness or in the area of the column it may be thickened as a drop panel. The column may also be of constant section or it may be flared to form a column head or capital. The drop panels are effective in reducing the shearing stresses where the column is liable to punch through the slab, and they also provide an increased moment of resistance where the negative moments are greatest

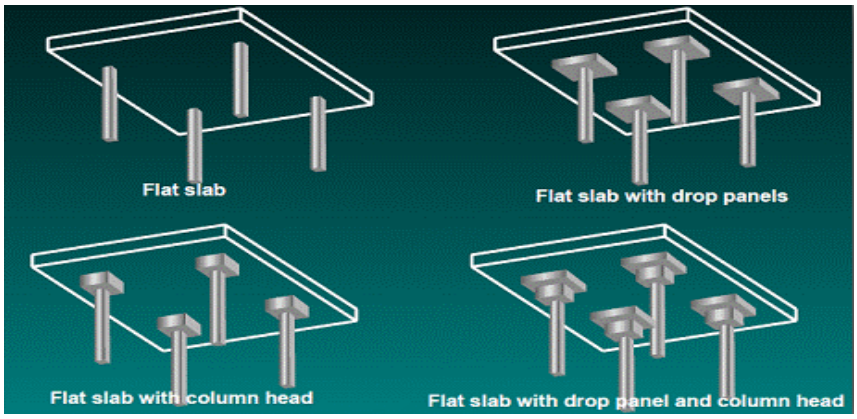


Figure 2.1: Types of Flat Slab (Anitha *et al.*, 2007).

2.3.1 Component of Flat Slabs

a. **Drops**: To resist the punching shear which is predominant at the contact of slab and column support, the drop dimension should not be less than one -third of panel length in that direction (Anitha *et al.*, 2007).

b. **Column heads**: Certain amount of negative moment is transferred from the slab to the column at the support. To resist this negative moment the area at the support needs to be increased, this is facilitated by providing column capital/heads (Anitha *et al.*, 2007)

2.3.2 Benefits of Reinforced Concrete Flat Slab

Flat slabs have many advantages over beam slab construction among which include:

i. Flexibility in Room Layout: Flat slab design allows Architect to introduce partition walls anywhere required and allows for the choice of omitting false ceiling and finish soffit of slab with coating. It also gives room for the owner to change the size of the room layout.

ii. Savings in building height Time and cost: Lower storey height will reduce building weight due to lower partitions and this also reduces foundation load. The formwork used (Braced panel of wood, metal or plastic) can be used repeatedly as the construction project progresses thereby reducing the cost of formwork as it accounts for a major portion of the overall cost of a concrete building frame.

2.4 EUROCODE 2 (2004) DESIGN PROVISIONS FOR FLAT SLABS

The analysis and design of concrete flat slabs to Eurocode is a process which is essentially the same as when using BS 8110. However, the layout and content of Eurocode 2 may appear

unusual to designers familiar with BS 8110. Eurocode 2 does not contain the derived formulae or specific guidance on determining moments and shear forces. This has arisen because it has been European practice to give principles in the codes and for the detailed application to be presented in other sources such as textbooks (Moss *et al.*, 2006)

2.4.1 Flexure in Reinforced Concrete Flat Slabs

The design procedure for flexure include a derived formulae based on the simplified rectangular stress block from Eurocode 2. Where appropriate Table 2.1 may be used to determine bending moments for flat slabs. Whichever method of analysis is used, Clause. 9.4.1 of EC2 requires the designer to concentrate the reinforcement over the columns. Annex I of the Eurocode gives recommendations for the equivalent frame method on how to apportion the total bending moment across a bay width into column and middle strips to comply with Clause 9.4.1. Designers using grillage, finite element or yield line methods may also choose to follow the advice in Annex I to meet this requirement. Eurocode 2 offers various methods for determining the stress-strain relationship of concrete.

It also gives recommendations for the design of concrete up to class C90/105. However, for concrete strength greater than class C50/60, the stress block is modified. It is important to note that concrete strength is based on the cylinder strength and not the cube strength (i.e. for class C28/35 the cylinder strength is 28 MPa, whereas the cube strength is 35 MPa)

Table 2.1: Bending moment coefficients for flat Slabs

	End support/Slab Connection				First Interior Support	Interior Spans	Interior Supports
	Pinned		Continuous				
	End Support	End span	End Support	End Span			
Moment	0	0.086FL	-0.04FL	0.075FL	-0.086FL	0.063FL	-0.063FL

Notes

1. Applicable to slabs where the area of each bay exceeds 30m^2 $Q_k \leq 1.25G_k$ and $q_k \leq 5kN/m^2$
2. F is the Total design ultimate load, L is the effective span
3. Minimum Span > 0.85 longest span, minimum of three spans
4. Based on 20% redistribution at supports and no decrease in span moments

Source: Moss *et al.,* (2006)

2.4.2 The procedure for Calculating Flexural Reinforcement (Moss et al, 2006):

1. Carry out analysis of slab to determine design moments M, where appropriate use coefficients from Table 2.1.

2. Check and ensure f_{ck} is ≤ C50/60

3. Determine K from $K = \frac{M}{f_{ck} b d^2}$ (2.1)

4. Determine K' from K'= 0.60δ – 0.18 δ^2 – 0.21 where δ ≤ 1 or from Table 2.2, to ensure ductile fracture K' should be limited to 0.168

5. Confirm K≤K' for slabs (i.e No compression reinforcement required)

6. Obtain lever arm Z from table or from $Z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53k}\right] \leq 0.95d$, (2.2)

This limitation is not a requirement of Eurocode 2, it is considered a good practice

7. Calculate tension Reinforcement required from $A_s = \frac{M}{0.87 Z f_{yk}}$ (2.3)

8. Check Minimum reinforcement Requirement $A_{smin} = \frac{0.26 f_{ctm} b_t d}{f_{yk}}$ (2.4)

9. Check maximum Reinforcement Requirement $A_{smax} = 0.04 A_c$ (2.5)

For tension and compression reinforcements outside lap location

Where in equation (2.1) to (2.5) f_{ck} is the Concrete Grade, M is the bending moment, f_{yk} is characteristic steel stress f_{ctm} is the mean tensile strength of concrete, A_c is Concrete cross-sectional area, A_s is the area of reinforcement, while b and d are the breath and effective depth respectively.

Table 2.2: Values of K'

% redistribution	б (redistribution ratio)	K'
0	1.0	0.208
10	0.90	0.182
15	0.85	0.168
20	0.80	0.153
25	0.75	0.137
30	0.70	0.120

Source: Moss *et al.,* (2006)

Table 2.3: z/d for singly reinforced rectangular sections

K	Z/d	K	Z/d
≤0.05	0.950	0.13	0.868
0.06	0.944	0.14	0.856
0.07	0.934	0.15	0.843
0.08	0.924	0.16	0.830
0.09	0.913	0.17	0.816
0.10	0.902	0.18	0.802
0.11	0.891	0.19	0.787
0.12	0.880	0.20	0.721

Source: Moss *et al.,* (2006)

Table 2.4: Division of moments between strips

	Column strip	**Middle strip**
Negative moment at edge of column	100% but not more than $0.17 b_e d^2 f_{ck}$	0
Negative moment at internal column	60 – 80%	40 – 20%
Positive moment in span	50-70%	50 – 30%

Source: Mosley *et al.,* (2007)

Table 2.5: Minimum percent reinforcement Requirement

f_{ck}	f_{ctm}	**Minimum % ($0.26 f_{ctm}/f_{yk}$)**
25	2.6	0.13%
28	2.8	0.14%
30	2.9	0.15%
32	3.0	0.16%
35	3.2	0.17%
40	3.5	0.18%
45	3.8	0.20%
50	4.1	0.21%
Key Where f_{yk} = 500 Mpa		

Source: Moss *et al.,* (2006)

2.4.3 Punching Shear in Reinforced Concrete Flat Slabs

The design value of the punching shear force V_{ED} will usually be the support reaction at the ultimate limit state. In principle the design for punching shear in Eurocode 2 and BS 8110 is similar. The main differences are as follows:

1. Standard factors for edge and corner columns that allow for moment transfer (β) are greater in Eurocode 2 as shown in figure 2.2. However, β can be

calculated directly from Expressions (6.38) to (6.46) of the code to give more efficient designs.

2. In Eurocode 2 the maximum value of shear at the column face is not limited to 5 MPa, and depends on the concrete strength used.
3. With Eurocode 2 the permissible shear resistance when using shear links is higher, although such designs may not be economic or desirable.
4. The control perimeters for rectangular columns in Eurocode 2 have rounded corners.
5. Where shear reinforcement is required the procedure in Eurocode 2 is simpler; the point at which no shear reinforcement is required can be calculated directly and then used to determine the extent of the area over which shear reinforcement is required.
6. It is assumed that the reinforcement will be in a radial arrangement. However, the reinforcement can be laid on a grid provided the spacing rules are followed.

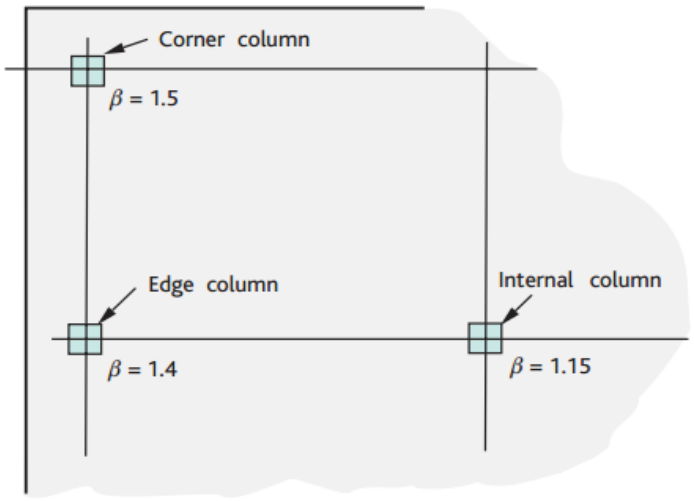


Figure 2.2: recommended standard values for β (Moss *et al.,* 2006**)**

2.4.4 The Procedure for Punching shear check and reinforcement determination is as follows: (Moss *et al.*, 2006**)**

1. Determine the value of β_1 factor by referring to figure 2 or expressions 6.38 to 6.46 of the Eurocode.
2. Determine the value of design shear stress at face of the column $V_{Ed,max}$ from

   $$v_{Ed,max} = \beta_1 {V_{Ed}}/{(U_i\, d_{eff})} \quad (2.6)$$

   where U_i is the perimeter of the column.
3. Determine the value of $V_{Rd,max}$ from table 6
4. Check to ensure $V_{Ed,max} < V_{Rd,max}$, if not redesign the slab
5. Determine the value of design shear stress v_{Ed} from

   $$v_{Ed} = \beta_1 V_{Ed}/(U_1 d_{eff}) \quad (2.7)$$

   where U_1 is the length of control perimeter
6. Determine the concrete punching shear capacity without shear reinforcement from table or from $v_{Rd,c} = 0.12k(100\rho_l f_{ck})^{\frac{1}{3}} \geq 0.035k^{\frac{3}{2}}\sqrt{f_{ck}}$ (2.8)

   where $k = 1 + \sqrt{\frac{200}{d_{eff}}} \leq 2\ and\ \rho_l = (\rho_{ly}\rho_{lz})^{1/2} \leq 0.02$ and $\rho_{ly}.\rho_{lz}$ are the reinforcement ratios in two orthogonal direction for fully bonded tension steel taken over a width equal to the column with plus 3d each side.
7. Check if $v_{Ed} > v_{Rd,c}$ and continue otherwise no punching shear reinforcement required
8. Determine area of punching shear reinforcement from: $A_{sw} = (v_{Ed} - 0.75v_{Rd,c})s_r.U_1/(1.5f_{ywd,ef})$ (2.9)

where s_r is the radial spacing of shear reinforcement, $f_{ywd,ef} = 250 + 0.25d_{eff} \leq f_{ywd}$ (see table 7)

9. Determine the length of the outer perimeter where shear reinforcement not required from $U_{out,ef} = \beta_1 . {V_{Ed}}/{(v_{Rd,c}\, d\,)}$ (2.10)

10. Determine layout of punching shear reinforcement

Where V_{ED} is the Design value of punching shear force, $v_{Ed,max}$ is Maximum Shear stress, U_i is the Perimeter at the section under consideration, $v_{Rd,max}$ is the Maximum Permissible shear stress and v_{Ed} is the Applied ultimate Shear stress

Table 2.6: Values for $v_{Rd,max}$

f_{ck}	$V_{Rd,max}$
20	3.31
25	4.05
28	4.48
30	4.75
32	5.02
35	5.42
40	6.05
45	6.64
50	7.20

Source: Moss *et al.*, (2006)

Table 2.7: Values for $f_{ywd,ef}$

d_{eff}	$f_{ywd,ef}$
150	288
175	294
200	300
225	306
250	313
275	319
300	325
325	331
350	338

Source: Moss *et al.*, (2006)

Table 2.8: Factors to be used with table 2.9 for $f_{ck} \neq 30$ (Moss et al, 2006)

f_{ck}	25	28	32	35	40	45	50
Factor	0.94	0.98	1.02	1.05	1.10	1.14	1.19

Source: Moss *et al.*, (2006)

Table 2.9: $v_{Rd,c}$ resistance of members without shear reinforcement

ρ_1	Effective depth, d (mm)										
	≤200	225	250	275	300	350	400	450	500	600	750
0.25%	0.54	0.52	0.50	0.48	0.47	0.45	0.43	0.41	0.40	0.38	0.36
0.50%	0.59	0.57	0.56	0.55	0.54	0.52	0.51	0.49	0.48	0.47	0.45
0.75%	0.68	0.66	0.64	0.63	0.62	0.59	0.58	0.56	0.55	0.53	0.51
1.00%	0.75	0.72	0.71	0.69	0.68	0.65	0.64	0.62	0.61	0.59	0.57
1.25%	0.80	0.78	0.76	0.74	0.73	0.71	0.69	0.67	0.66	0.63	0.61
1.50%	0.85	0.83	0.81	0.79	0.78	0.75	0.73	0.71	0.70	0.67	0.65
1.75%	0.90	0.87	0.85	0.83	0.82	0.79	0.77	0.75	0.73	0.71	0.68
≥2.00%	0.94	0.91	0.89	0.87	0.85	0.82	0.80	0.78	0.77	0.74	0.71
K	2.000	1.943	1.894	1.853	1.816	1.756	1.707	1.667	1.632	1.577	1.516

Source: Mosley *et al.*, (2007)

Table 2.9 was prepared for $f_{ck} = 30$ N/mm^2 where ρ_l exceeds 0.4%, table 2.8 may be used.

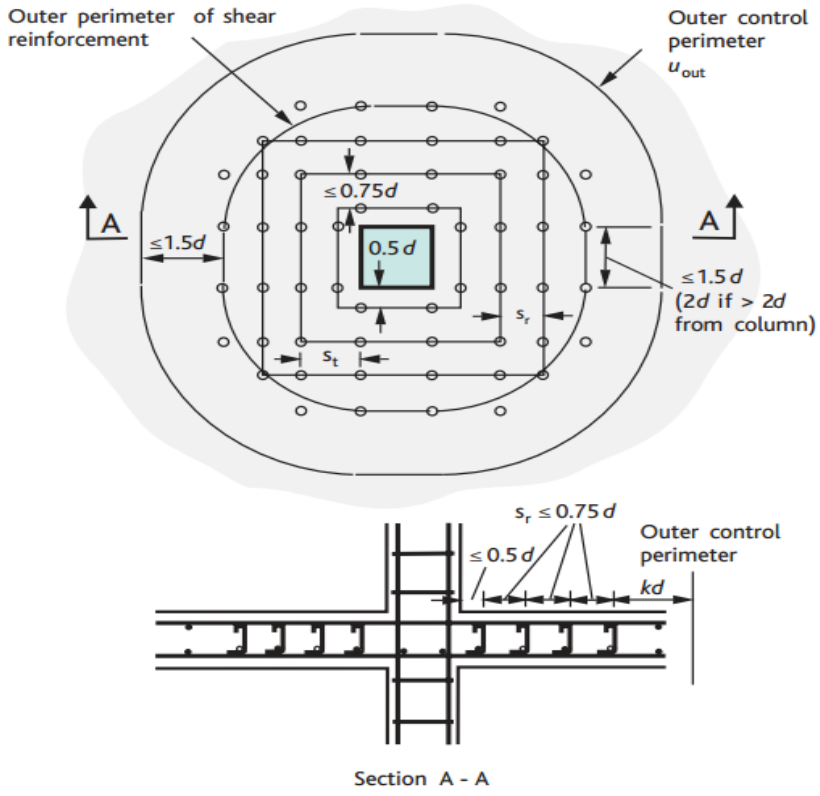


Figure 2.3: Punching shear layout (Moss *et al.*, 2006**)**

2.4.5 Deflection in Flat Slabs

Eurocode 2 has two alternative methods of designing for deflection; either by limiting span-to-depth ratio or by assessing the theoretical deflection using the Expressions given in the Eurocode.

The span-to-depth ratios should ensure that deflection is limited to span/250 and this is the procedure presented in the following step by step sequence, The Background paper to the United Kingdom National Annex7 notes that the span-to-depth ratios are appropriate where the structure remains propped during construction or until the concrete attains sufficient strength to support the construction loads. It can generally be assumed that early striking of formwork will not significantly affect the deflection after installing the cladding and/or partitions.

There are numerous factors that affect deflection. These factors are also often time-related and interdependent, which makes the prediction of deflection difficult. The main factors are, **Concrete tensile strength, Creep and Elastic modulus**. Other factors include: Degree of restraint, Magnitude of loading, Time of loading, Duration of loading, Cracking of the concrete, Shrinkage, Ambient conditions, Secondary load-paths and Stiffening by other elements (Moss et al 2006)

2.4.6 The Procedure for Deflection check of flat slabs. (Moss et al, 2006)

a. Determine basic L/d from Figure

b. Determine factor 1 (F1), for ribbed or waffle slabs $1 = 1 - 0.1((\frac{b_f}{b_w}) - 1) \geq 0.8$, otherwise F1 = 1. (b_f is the flange breath and b_w is the rib breath)

c. Determine factor 2 (F2), where slab span exceeds 8.5m and supports brittle partition F2 = $8.5/l_{eff}$. Otherwise F2 = 1.0

d. Determine factor 3 (F3) , F3 = $\frac{A_{s,prov}}{A_{s,reqd}} \leq 1.5$ or $310/\partial_s \leq 1.5$, where ∂_s is the stress in the reinforcement calculated under the characteristic combination of SLS loads

e. Check if $basic\ l/d \times F1 \times F2 \times F3 \geq Actual\ l/d$, if yes check is complete otherwise $A_{s,prov}$ is increased, F3 recomputed and step 5 rechecked.

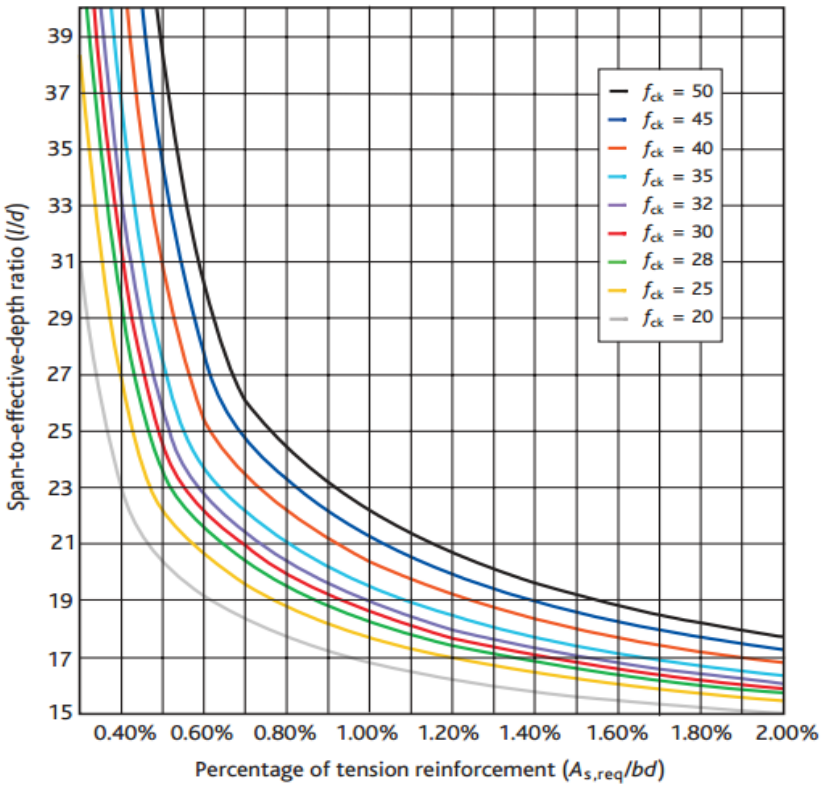


Figure 2.4: Basic span to effective depth ratio (Moss *et al.*, 2006**)**

Figure (2.2) assumes simply supported flat slab (K=1.2) and compression reinforcement ρ' has been taken as Zero, moreover the curves are based on the following expressions:

$$\frac{l}{d} = K[11 + \frac{1.5\sqrt{f_{ck}}}{\rho}\ \rho_0 + 3.2\sqrt{f_{ck}}\ (\frac{\rho_0}{\rho} - 1)^{1.5}] \quad (2.11)$$

where $\rho < \rho_0$ and

$$\frac{l}{d} = K\left[11 + \frac{1.5\sqrt{f_{ck}}}{\rho - \rho'}\ \rho_0 + \frac{\sqrt{f_{ck}}}{12}\ \sqrt{\frac{\rho}{\rho_0}}\right] \quad (2.12)$$

where $\rho > \rho_0$

2.5 METHODS OF RELIABILITY ANALYSIS

The following approaches can be used in order to determine the safety of a structure (Vrijling, 2000).

(i) Deterministic approach (level 0 approach): - The design is based on average situations and the appropriate safety factor is included in order to obtain a safe structure.

(ii) Semi probabilistic approach (level 1 approach): - A characteristic value is used in the design. Like the load which is not exceeded in 95% of the cases, or the strength which is available for 95% of the construction materials.

(iii) Probabilistic approach (level II and III): - In this approach a full statistical distribution of all variables are taken into account.

a. Level II: - Comprises a number of approximate methods in which the distribution functions are transformed into standard normal or standard Gaussian distributions. In order to approximate the probability of failure, the mathematical formulation of the problem has to be licensed.

b. Level III: - In this approach the probability distribution, functions of the stochastic variables are fully taken into account, if the problem in non linear, this is taken into account as well. (Vrijling, 2000)

2.6 RELIABILTY-BASED DESIGN

In reliability-based design the limit state function or failure function is given by equation (2.13)

$$G = R - S \tag{2.13}$$

where

G = 0 divides the design space into two regions, the safety region (G > 0) and the failure region (G < 0). Because of the uncertainties in loads and strength, G is a random variable itself. As a result, it cannot be certain in advance whether G falls into the safe region or failure region. In reliability-based design, a structural element is designed such that the probability that G is positive is sufficiently high. In mathematical term, this is expressed as:

$$Reliability = Probability\,[G > 0] \geq Target\ Reliabilty \tag{2.14}$$

In engineering practice, the safety index, β, instead of structural reliability is often used to represent the reliability level. When G has a normal distribution, β has a one-to-one correspondence with the structural reliability, given by

$$\beta = -\Phi^{-1}(1 - Reliability) = {}^{\mu_g}/_{\sigma_g} \tag{2.15}$$

Where μ_g and σ_g are the mean and standard deviation of the G-function and Φ is the cumulative distribution function for the standard normal distribution. In the case where G has

other distributions, Equation (2.15) does not stand, but in general a larger β corresponds to a higher reliability level

2.6.1 Target Reliability Values

Target values for the reliability index β for various design situations, and for reference periods of 1 year and 50 years, are indicated in the Table 2.10. The values of β in the table correspond to levels of safety for reliability class RC2 (Annex B of Eurocode 0) structural members.

Table 2.10: Target Reliability β for class RC2 Structural Members

Limit State	Target Reliability index	
	1 year	50 years
Ultimate	4.7	3.8
Fatigue		1.5 to 3.8
Serviceability	2.9	1.5

Source: Eurocode 0 (2002)

2.6.2 Consequence of Failure or Malfunctioning of Structures and their Classes:

The consequence of failure class of a structural member, it's description as well as Examples are given in Table 2.11

Table 2.11: Definition of consequence classes

Consequences Class	Description	Examples of buildings and Civil Engineering works
CC3	High consequence for loss of human life, or economic, social or environmental	Grandstands, public buildings where consequences of failure are high (e.g a concert hall)
CC2	Medium consequence for loss of human life, economic, social or environmental consequences consideration	Residential and office buildings, public buildings where consequences of failure are medium (e.g an office building)
CC1	Low consequence for loss of human life, and economic, social or environmental consequences small or negligible	Agricultural buildings where people do not normally enter (e.g storage buildings), green houses

Source: Eurocode 0. (2002)

2.6.3 Reliability Classes:

The reliability classes (RC) may be defined by the β reliability index concept. Three reliability classes RC1, RC2 and RC3 may be associated with the three consequences classes CC1, CC2 and CC3. Table 2.12 gives recommended minimum values for the reliability index associated with reliability classes.

Table 2.12: Recommended minimum values for Reliability index β (Ultimate Limit state)

Reliability Class	Minimum Values for β	
	1 year reference period	**50 years reference period**
RC3	5.2	4.3
RC2	4.7	3.8
RC1	4.2	3.3

Source: Eurocode 0. (2002)

CHAPTER THREE

METHODOLOGY

3.1CONSTITUTIVE MODELS

One of the first steps of reliability analysis is the selection of an appropriate constitutive model. According to Lukman (2012) constitutive models refer to models generated by obeying relevant constitutive laws, the laws to be followed in this case are those set out by the EC 2. The selection is very essential as it defines the characteristics (stress, strain, strength, etc.) of the section or system to be analyzed. It should be recalled that in the analysis of a rectangular cross-section to determine the ultimate moment of resistance, the assumptions given in section 3 of EN 1992-1-1 (2004) are to be followed.

3.1.1 Concrete Constitutive Models

The behavior of structural concrete as is represented by figure 3.1 shows a rectangular stress – strain relationship up to a strain ε_{c2}, from which point the strain increases while the stress remains constant. Strain ε_{c2} is specified as a function of the characteristic cylinder strength of the concrete (f_{ck}). The ultimate design stress is given by ;

$$\frac{\propto f_{ck}}{\gamma_c} = \frac{0.85 f_{ck}}{1.5} = 0.567 f_{ck} \qquad (3.1)$$

Where the factor of 0.85 allows for the difference between the bending strength and the cylinder crushing strength of the concrete and $\gamma_c = 1.5$ is the usual partial safety factor for the strength of concrete. The ultimate strain of $\varepsilon_{cu2} = 0.0035$ is typical for classes of concrete ≤ C50/60

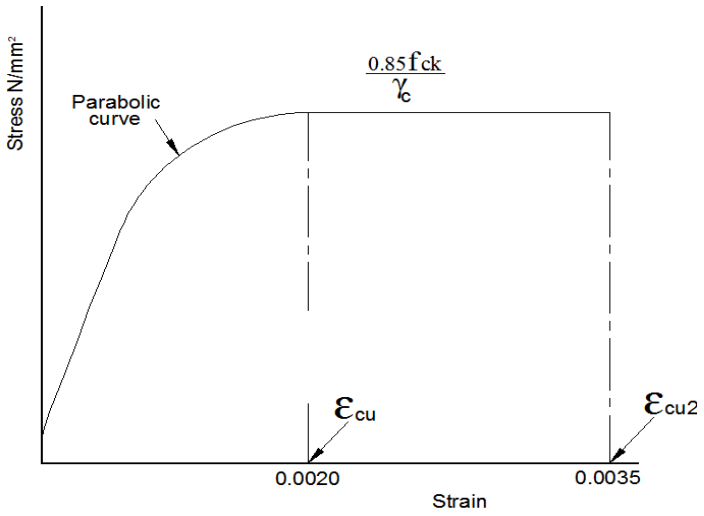


Fig 3.1: Parabolic-rectangular Stress-strain Diagram for concrete in Compression (Mosley et,al 2007)

3.1.2 Steel Constitutive Model

The representative short-term design stress-strain curve for reinforcement is given in figure 3.2. The behavior of steel is identical in tension and compression, being linear in the elastic range up to the design yield stress of f_{yk}/γ_s where f_{yk} is the characteristic yield stress and γ_s is the partial factor of safety.

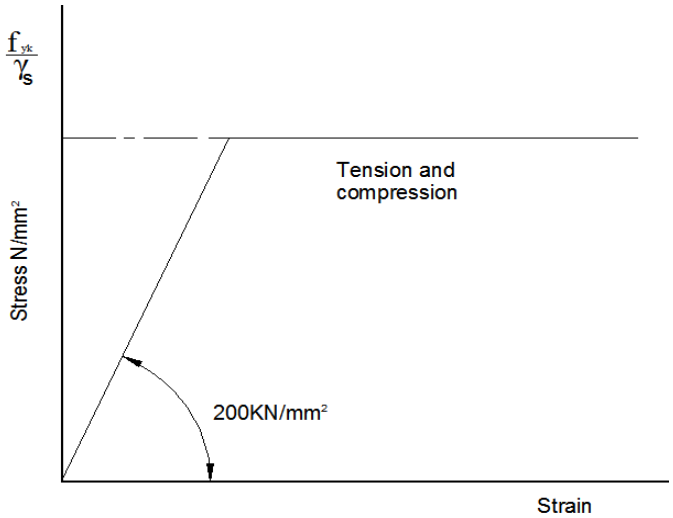


Figure 3.2: Short-term Design Stress-strain Curve for Reinforcement (Mosley et, al. 2007)

Within the elastic range, the relationship between the stress and strain is

Stress = elastic modulus × strain

$$= E_s \times \varepsilon_s \tag{3.2}$$

With the elastic modulus of steel E_s= 200 KN/m^2 so that the design yield strain is

$$\varepsilon_y = (\frac{f_{yk}}{\gamma_s})/\mathrm{E_s} \tag{3.3}$$

At the ultimate limit for f_{yk}= 500N/mm^2

$\varepsilon_y = 500/(1.15 \times 200 \times 10^3)$

= 0.00217

3.2 RECTANGULAR STRESS BLOCK AND SECTION DESIGN EQUATIONS

The relationship between stress and strain discussed above and the EN 1992-1-1 (2004) requirements results in the consideration of a concrete stress block having a rectangular shape referred to as a "rectangular stress block" (Mosley *et al.*, 2007) illustrated in Figure 3.3.

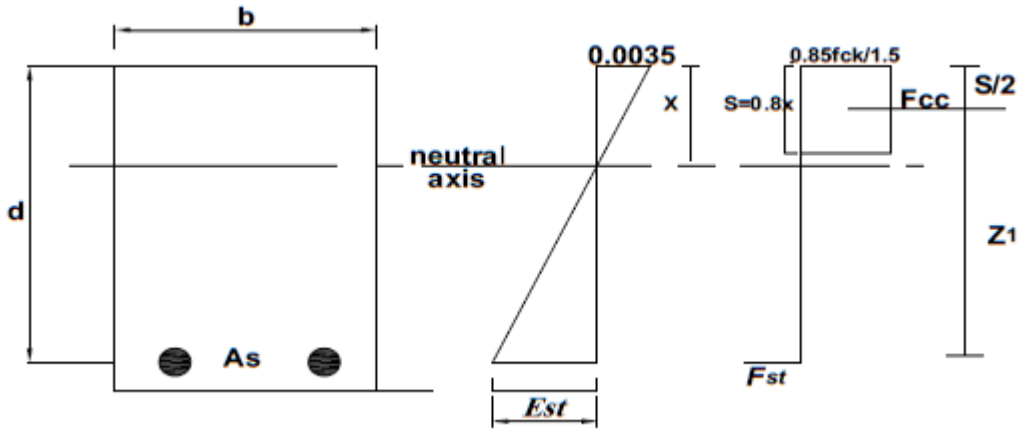


Figure 3.3: Singly Reinforced Section with Rectangular Stress Block (Mosley *et al.*, 2007)

The theory of bending for reinforced concrete assumes that the concrete will crack in the regions of tensile strains and that, after cracking, all the tension is carried by the

reinforcement. It also assumes that plain sections of structural members remains plain after straining, so that across the section there must be a linear distribution of strains (Mosley *et al.*, 2007).

As there is compatibility of strains between the reinforcement and the adjacent concrete, the steel strains, ε_{st}, can be determined from the strain diagram. The relationship between the depth of neutral axis,(*x*,) and the maximum concrete strain, ε_{cu2}, and steel strain, ε_{st}, is given by:

$$\varepsilon_{st} = \varepsilon_{cu2}\left(\frac{d-x}{x}\right) \quad (3.4)$$

Where *d* is the effective depth of the beam.

Having determined the strains, the stresses in the reinforcement can be evaluated from the stress-strain curve of figure 3.2 (Mosley *et al.*, 2007)

For analysis of sections with known steel strain, the depth of the neutral axis can be determined by rearranging the above equation as:

$$x = \frac{d}{1+\left(\varepsilon_{st}/\varepsilon_{cu2}\right)} \quad (3.5)$$

For steel with f_y = 500N/mm^2, at the ultimate limit state, ε_{st} and ε_{cc} are 0.00217 and 0.0035 for concrete class ≤ C50/60 respectively. Inserting these values into equation yields

$$x = \frac{d}{1+\left(0.00217/0.0035\right)} = 0.617d \quad (3.6)$$

Hence to ensure yielding of tension steel at ultimate limit state.

$x \leq 0.617d$

At the ultimate limit state, it is important that member sections in flexure should be ductile and that failure should occur with the gradual yielding of the tension steel and not by sudden

catastrophic compression failure of concrete. Therefore, the code EN 1992-1-1 (2004) limits the depth of neutral axis, $x \leq 0.45d$ (Mosley *et al.,* 2007).

3.3 DESIGN EQUATIONS FOR CHARTS

It can be seen from the Figure 3.3 that the stress block does not extend to the neutral axis of the section but has a depth *S=0.8x.* This will result in the centroid of the stress block being *S/2 = 0.40x* from the top edge of the section, which very nearly the same as rectangular-parabolic stress block (Mosley *et al.*, 2007).

The equations for the design charts are obtained by taking moments about the neutral axis. Thus,

$$M = F_{cc}\left(x - \frac{s}{2}\right) + F_{st}(d - x) \tag{3.7}$$

Where $F_{cc} = \frac{0.85}{1.5} f_{ck} bS$

And $F_{st} = \frac{f_{yk}}{1.15} A_{st}$

Or $F_{st} = 0.87 f_{yk} A_{st}$ (3.8)

Substituting into equation (3.7) we have

$$M = \frac{0.85}{1.5} f_{ck} b(S)(x - \frac{S}{2}) + 0.87 f_{yk} A_{st}(d - x) \tag{3.9}$$

With *S=0.8x*

Equation (3.9) becomes

$$M = \frac{0.85}{1.5} f_{ck} b(0.8x)(x - \frac{0.8x}{2}) + 0.87 f_{yk} A_{st}(d - x)$$

$$M = 0.4536 f_{ck} bx(x - 0.4x) + 0.87 f_{yk} A_{st}(d - x) \tag{3.10}$$

For equilibrium,

$F_{cc} = F_{st}$

This implies:

$\frac{0.85}{1.5} f_{ck} b(0.8x) = 0.87 f_{yk} A_{st}$

$0.4536 f_{ck} bx = 0.87 f_{yk} A_{st}$ (3.11)

Dividing above equation through by *bd* and making $A_{st}/_{bd}$ the subject of the formulae yields:

$\frac{A_{st}}{bd} = 0.4536 \frac{f_{ck}}{0.87 f_{yk}} \frac{x}{d}$ (3.12)

From which

$\frac{x}{d} = \frac{0.87 f_{yk} A_{st}}{0.4536 f_{ck}}$

Substituting equation (3.11) into equation (3.10) gives:

$M = 0.87 f_{yk} A_{st}(x - 0.4x) + 0.87 f_{yk} A_{st}(d - x)$ (3.13)

Diving equation (3.13) through by bd^2 gives:

$\frac{M}{bd^2} = 0.87 f_{yk} \frac{A_{st}}{bd^2}(0.6x) + 0.87 f_{yk} \frac{A_{st}}{bd^2}(d - x)$

Rearranging yields:

$\frac{M}{bd^2} = 0.87 f_{yk} \frac{A_{st}}{bd}\left(0.6\frac{x}{d}\right) + 0.87 f_{yk} \frac{A_{st}}{bd}(1 - \frac{x}{d})$ (3.14)

Letting $\frac{A_{st}}{bd} = \rho$

Equation (3.14) becomes

$\frac{M}{bd^2} = 0.87 f_{yk} \rho \left(0.6\frac{x}{d}\right) + 0.87 f_{yk} \rho (1 - \frac{x}{d})$ (3.15)

Factorizing equation (3.15)

$\frac{M}{bd^2} = 0.87 f_{yk} \rho \left(0.6\frac{x}{d} + 1 - \frac{x}{d}\right)$

$\frac{M}{bd^2} = 0.87 f_{yk} \rho \left(1 - \frac{0.4x}{d}\right)$ (3.16)

From figure 3.3 lever arm Z_1 is the distance between the centroid of the steel where the tensile force *Fst* acts and the center of the compressive force *Fcd* of the concrete in compression.

$$Z_1 = d - 0.4x \tag{3.17}$$

Diving equation (3.17) by *d* we have:

$$\frac{Z_1}{d} = 1 - 0.4\frac{x}{d} \leq 0.95 \tag{3.17a}$$

Substituting equation (3.17a) into equation (3.16) gives:

$$\frac{M}{bd^2} = 0.87 f_{yk} \rho \frac{Z_1}{d} \tag{3.18}$$

From figure (3.3) for equilibrium

$$M = F_{cc} Z_1 + F_{st} Z_1 \tag{3.19}$$

$$M = 0.567 f_{ck} bs Z_1 = 0.87 f_{yk} A_{st} Z_1$$

Dividing (3.19) by bd^2 and substituting for $S=0.8x$ yields

$$\frac{M}{bd^2} = 0.4536 f_{ck} \frac{x}{d}\frac{Z_1}{d} = 0.87 f_{yk} \frac{A_{st}}{bd}\frac{Z_1}{d}$$

From the above equation

$$\frac{M}{bd^2} = 0.4536 f_{ck} \frac{x}{d}\frac{Z_1}{d} \tag{3.20}$$

And

$$\frac{M}{bd^2} = 0.87 f_{yk} \frac{A_{st}}{bd}\frac{Z_1}{d} \tag{3.21}$$

It should be noted at this point that equation (3.20) gives the same result as equation (3.18). In all cases for the generation of design charts to EN 1992-1-1 (2004) design requirements, any of the mentioned equations can be used with ρ (reinforcement ratio) as given.

From stress block in Figure (3.3), removing the factors of safety and equating the forces

$0.85 f_{ck} b(0.8x) = f_{yk} A_{st}$

Dividing Equation (3.22) by *bd* and rearranging will give:

$$\frac{A_{st}}{bd} = 0.68 \frac{f_{ck}}{f_{yk}} \frac{x}{d} \quad (3.22)$$

the same way Equation (3.22) is derived, for equilibrium.

$M = f_{yk} A_{st} Z_1$

Diving Equation (3.38) by bd^2 and rearranging yields:

$$\frac{M}{bd^2} = \rho f_{yk} \frac{Z_1}{d} \quad (3.23)$$

Where ρ is given by Equation (3.22).

3.4 FIRST ORDER RELIABILITY METHOD

In FORM, it is assumed that R (resistance) and S (load) are independent and both normally distributed (or have been transformed to normally distributed variables). ''Z = R – S'' is thus supposed to be a linear function of normally distributed variables, it thus follows from theory that Z is normally distributed. This implies that the mean μ , and the variance σ^2 of Z can be written as ;

$$Z = R - S \quad (3.24)$$

$\mu(Z) = \mu(R) - \mu(S)$

$\sigma^2(Z) = \sigma^2(R) + \sigma^2(S)$

The probability of failure follows from

$$P\{Z < 0\} = \int_0^{\infty} fz(\xi) d\xi = \Phi(-\beta) \quad (3.25)$$

Where $\beta = \mu(Z)/\sigma(Z)$

$fz(\xi)$=probability density function of Z

$\Phi(-\beta)$=standard normal distribution for the variable β

β =Hasofer-Lind reliability index

For β>2, $\Phi(-\beta) \simeq (1/\beta\sqrt{2\pi})^{-\beta 2/2}$ (3.26)

Z in equation (3.9) is a function of more than one variable, only if Z is a linear function and all variables are normally distributed (and independent) Equation (3.24) is an equality and not an approximation.

3.5 COMPUTATON OF RELIABILITY INDEX

As both "load" S and "strength" R may depend on more than one variable, in order to perform a level II calculation, the variables $X_1, X_2, \ldots\ldots X_n$ have to be independent and it must be possible to linearize the reliability function Z. suppose the reliability function fulfills this requirements and the variables X_i are all normally distributed and independent then

$$Z_{lin} = Z(X_i^*, X_i^*, \ldots\ldots X_n^*) + \sum_{i=1}^{n} \left(\frac{\partial z}{\partial xi}\right)_{x_i = x_i^*} (X_i - X_i^*) = 0 \quad (3.27)$$

Z_{lin}=linearized reliability function

$\left(\frac{\partial z}{\partial xi}\right)_{x_i = x_i^*}$ = partial derivative of Z with respect to x_i evaluated in $x_i = x_i^*$

The mean value and standard deviation of Z_{lin} are

$$\mu_{Z_{lin}} = Z(X_i^*, X_i^*, \ldots\ldots X_n^*) + \sum_{i=1}^{n} \left(\frac{\partial z}{\partial xi}\right)_{x_i = x_i^*} (\mu_{x_i} - X_i^*) \quad (3.28)$$

$$\sigma_{Z_{lin}} = \sqrt{\sum_{i=1}^{n} \left(\frac{\partial z}{\partial xi}\right)^2_{xi=xi^*} \sigma_{xi}^2} \quad (3.29)$$

respectively.

If the mean values $X_i^* = \mu_{xi} \ldots\ldots X_n^* = \mu_{xn}$ are substituted, a so called mean value approximation is obtained, if the failure boundary is non linear a better approximation can be achieved by linearising the reliability function in the design point, which is only defined if the variables are normally distributed (or are transformed). The design point is defined as a point on the failure boundary in which the (joint) normal probability density function is maximum.

The design point $X_i^* = \mu_{x_i} - \alpha_i \beta \sigma_{x_i}$ (3.30)

Where; α_i= the factor of influence for the variable $= \frac{\sigma_{x_i}}{\sigma_{Z_{lin}}} \frac{\partial z}{\partial xi}$ (3.31)

Unless Z is a linear, the design point cannot be determined directly.

The research work contains Three (3) parts both of which require the use of FORM. In the first part, EC2 design charts were developed and safety index computed for the chart. The second part entails the generation of Iso-safety design charts to various target safety index value (β). While in the last part, sensitivity analysis of some of the basic design variables (of the various failure modes of the flat slab designed using the developed charts) on the reliability index (β) was conducted. That is the variables are taken in turn and varied and their effect on the reliability index (β) is noted.

3.6 LIMIT STATE FUNCTIONS

3.6.1 Limit State Function for Iso-safety Design Charts

The limit state function otherwise called the performance function equation *G(X)*, defined as the difference between the strength (R) and load (S) is given by:

$$G(X) = R - S \tag{3.32}$$

In terms of the variables therefore,

$$\mathrm{G}(X) = A - B \tag{3.33}$$

where:

$$A = \rho f_{yk} \frac{z}{d} \tag{3.34}$$

$$B = \frac{M}{bd^2} = 0.4536 \alpha f_{ck} \frac{x}{d} \frac{z}{d} \tag{3.35}$$

If *G(X)* is negative, the structure will fail. A positive *G(X)* implies a safe structure while a zero value of the limit state function indicates a point exactly on the failure surface. The value of B as given in Equation 3.34 (load capacity) is multiplied by the coefficient 'α' as defined in section below.

Hence, the limit state function for this analysis is given by:

$$\mathrm{G}(X) = \rho f_{yk} \frac{z}{d} - 0.4536 \alpha f_{ck} \frac{x}{d} \frac{z}{d} \tag{3.36}$$

Where α = moment stress factor

3.6.2 Limit State Function for Punching Shear

a) At the column Head

$$G(x) = V_{Rmax} - V_{ED} \tag{3.37}$$

$$V_{Rmax} = 0.5 \theta_R U d (0.6 \left(1 - \frac{f_{ck}}{250}\right)) \frac{f_{ck}}{1.5}$$

$$V_{ED} = 1.15\theta_S(F - \frac{\pi d^2}{4}\frac{F}{L^2})$$

$$F = design\ load = Q_K(1.35 + 1.5\alpha)$$

$$U = \pi d$$

$\theta_R =$ Resistance model uncertainty

$\theta_S =$ Load model uncertainty

b) First Critical Section (2d from column face)

$$G(x) = V_{Rdc}Ud_1 - V_{Eff}$$

$$V_{Rdc} = 0.12\theta_R(1 + \sqrt{\frac{200}{d}})(100\rho f_{ck})^{1/3}$$

$$V_{Rdcmin} = 0.035\theta_R(1 + \sqrt{\frac{200}{d}})^{3/2}\ f_{ck}^{0.5}$$

$$V_{Eff} = 1.15\theta_S(F - \frac{\pi(d_1 + 4d_2)^2}{4}\frac{F}{L^2})$$

$$U = \pi(d_1 + 4d_2)$$

Where $d_1 =$ Column head diameter

d_2= Effective depth of the slab

c) At the drop panel

$$G(x) = V_{Rdc}Ud_3 - V_{Eff} \qquad (3.38)$$

$$V_{Rdc} = 0.12\theta_R(1 + \sqrt{\frac{200}{d}})(100\rho f_{ck})^{1/3}$$

$$V_{Rdcmin} = 0.035\theta_R(1 + \sqrt{\frac{200}{d}})^{3/2}\ f_{ck}^{0.5}$$

$$V_{Eff} = 1.15\theta_S(F - A_P\frac{F}{L^2})$$

$A_P = (d_4 + 3d_3^2) - (4 - \pi)2d_3^2$

$U = 2a + 2b + 2\pi(2d_3)$

Where a and b are the length and breadth of drop respectively

$d_3 =$ Effective depth of slab

$d_4 =$ Dimension of the drop (a or b)

3.6.3 Limit State Function for Deflection

$$G_x = \theta_R\left(\frac{L}{d}\right)_{Limiting} - \theta_S\left(\frac{L}{d}\right)_{Actual} \quad (3.39)$$

$\left(\frac{L}{d}\right)_{Limiting} = K\left(11 + \frac{1.5\sqrt{f_{ck}}}{\rho}\rho_0 + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{1.5}\right.$ Where $\rho \leq \rho_0$

Or

$\left(\frac{L}{d}\right)_{Limiting} = K\left(11 + \frac{1.5\sqrt{f_{ck}}}{\rho - \rho^1}\rho_0 + \sqrt{\frac{f_{ck}}{12}}\sqrt{\frac{\rho^1}{\rho}}\right)$ Where $\rho > \rho_0$

$\rho^1 =$ Compression reinforcement (Zero in this case)

$\rho^0 =$ Reference reinforcement ratio$= 10^{-3}\sqrt{f_{ck}}$

K = 1.2 for flat slabs.

Eurocode 2 recommends limiting deflection to be multiplied by the ratios 8.5/L for flat slab length > 8.5m, $500/f_{yk}$ for steel grades different from 500N/mm^2 and $As_{required}/As_{provided}$ when the provided reinforcement is different from the required reinforcement.

3.7 ANALYSIS PROCEDURE

3.7.1 Iso-safety charts

For each concrete grade (f_{ck}) and steel grade (f_{yk}), corresponding values of A_{st}/bd (equation 3.12) are calculated with respect to the position of neutral axis ($\frac{x}{d}$), this provides solutions for the x-axis of the design charts which singly reinforced rectangular concrete sections according to Eurocode 2 (EN 1992-1-1 (2004)) where based. For the y-axis ($\frac{M}{bd^2}$), (equation 3.16) is applied also with respect to the neutral axis depth, concrete grade (f_{ck}) and steel grade (f_{yk}), to calculate the moment stress.

For the generation of Iso-safety Charts, subroutines were developed in FORTRAN programming language, these subroutines were synchronized with First Order Reliability Method (FORM 5); which is a program for reliability analysis (Gollwitzer et, al (1988), equations (3.20) and (3.23) were used with equation (3.20) multiplied by a factor "α" (Moment Stress factor). Three basic variables are selected for this analysis. These are concrete grade (f_{ck}), steel grade (f_{yk}) and neutral axis depth (x/d), the statistical properties of the variables are as shown in table 13.

For each point on the generated EC2 charts a reliability analysis is carried out and the results obtained are tabulated in the appendix (tables). This is aimed at estimating the implied reliability index of the generated charts.

The second part of the study is the generation of the Iso-safety charts; the reliability analysis of the generated EC2 charts is done at fixed reinforcement ratio to determine variable values that corresponds to a target reliability level. For each concrete grade (f_{ck}), steel grade (f_{yk}) and neural axis depth (x/d), the reinforcement ratio ($\boldsymbol{\rho}$) is kept constant while the moment stress,

$M/_{bd^2}$, is allowed to vary systematically until a target value of safety index, β_T, is attained. The values of $M/_{bd^2}$ thus obtained are plotted against the computed values of $\boldsymbol{\rho}$ to generate the new reliability-based (Iso-safety) design charts for the target value of β_T.

Table 3.1: Statistical models of the basic design variables

S/No	Design Variable	Unit	Distribution model	Mean	Coefficient of variation
1	Concrete Compressive Strength (f_{ck})	N/mm^2	Lognormal	25	0.17
2	Steel Strength (f_{yk})	N/mm^2	Lognormal	500	0.15
3	Dead load (G_k)	Newton	Normal	279700	0.10
4	Effective depth of slab (d)	mm	Normal	205 and 305	0.025
5	Column head diameter d_2	mm	Normal	1200	0.06
	Slab length (L)	mm	Normal	6500	0.06
6	Reinforcement ratio (ρ)	_	Normal	0.16 and 0.20	0.16
7	Panel drop dimension (d_1)	mm	Normal	2500	0.06
8	Neutral axis position $(x/_d)$	_	Lognormal	0.07672	0.01
9	Resistance model uncertainty ($\emptyset_R$)	_	Lognormal	1.1	0.07
10	Load model uncertainty $(\emptyset_S)$	_	Lognormal	1.0	0.2

Source: Bartlett *et al* (2003), Afolayan (2005), Lukman (2012) and Jibrin *et al*. (2014)

3.7.2 Punching shear

Three sections of the flat slab were considered for punching shear analysis; these are the column face, first critical section from the column face (i.e a distance of 2d from the column face) and a critical section at the panel drop, the resistance of both the two critical sections is a function of the flexural reinforcement and other design parameters. The flat slab was designed for flexure using the developed EC 2 design chart and the Iso-safety charts Targeting Safety

Indices (β_T) of 3.3, 3.8 and 4.3 based on EC2 recommendation and corresponding to the three(3) failure consequence classes mentioned. Using the three flexural reinforcement calculated by targeting the mentioned safety indices (3.3, 3.8 and 4.3), FORM was used to check the punching shear safety of the flat slab and sensitivity analysis was conducted for each target safety.

3.7.3 Deflection

A reliability analysis is carried out for deflection to ascertain the safety of the flat slab using the three flexural reinforcement ratios provided by the three safety target (3.3, 3.8 and 4.3). Sensitivity analysis was also carried out to know how the safety of the structure will be affected by varying some of the design parameters. It was however in this case ensured that the Limiting deflection is multiplied by 8.5/L for flat slab length > 8.5m as recommended by EC2.

The procedure for this research is summarized below:

1. Selecting a constitutive model for flexure in flat slab as implied in EC-2 (2004)
2. Defining the limit state equations for the failure modes considered
3. Using appropriate basic equations, design curves to EC-2 (2004) were plotted, and these agree with the limit state equation.
4. Stochastic characteristic of the basic variables were estimated.
5. The Computation of safety index **β** value for the various points on the design curve developed in step 3 above using First Order Reliability Method (FORM).
6. Subroutines in a FORTRAN module were developed and synchronized with that of First-Order Reliability Method (FORM5) and Design Charts to different target

reliability indices were generated according to EC-2 (2004). Target reliability level choice is made to correspond to values recommended for the ultimate limit state in Eurocode 0 (2002). The selected target values for the reliability index, β, are 3.3, 3.8 and 4.3 as the minimum recommended by the code for consequence class CC1, CC2 and CC3 respectively as shown in table 1.

7. Other failure modes of flat slab (Punching shear and Deflection) were then considered and safety checks (reliability analysis) was carried out to ascertain the safety of the flat slab when the flexural reinforcement provided is corresponding to the three(3) Iso-safety targets (3.3, 3.8 and 4.3). Sensitivity analysis was also carried out.

3.8 PROGRAM FLOWCHART FOR ISO-SAFETY CHARTS

The flowchart of the FORTRAN based program synchronized with FORM 5 used for the production of Iso-safety charts is shown in Figure 3.4. The program starts by calling on the user to supply a target value of safety index, β_T, After which it reads the other parameters (f_{ck} f_{yk} *and x/d*) which are also supplied by the user and it computes moment stress and reinforcement ratio. FORM 5 is subsequently called as a subroutine to calculate the reliability index of the provided data, the program then request for other parameters if the safety index value calculated is not the same as the target value and continues until the target is met. That is Once the safety index β is computed, then the program checks the calculated value of β against the target safety index, β_T, specified at the onset. If the difference $(\beta_T - \beta)^2 \leq \varepsilon$ known as acceptance level (Abubakar 1999; Akindahunsi 2009), the program writes down β, $\frac{M}{bd^2}$ and $\frac{A_{st}}{bd}$ and then stops, otherwise (i.e if $(\beta_T - \beta)^2 > \varepsilon$) FORM is called again until the values that yield the target safety index is obtained. The flow chart for FORM for computation of β_T is given in Figure 3.4

Program 'PROGRAM DESIGN CHART' is used to estimate the safety index of the generated EC2 design charts and program 'PROGRAM ISO-SAFETY CHARTS' is used for the generation of the Iso-safety charts.

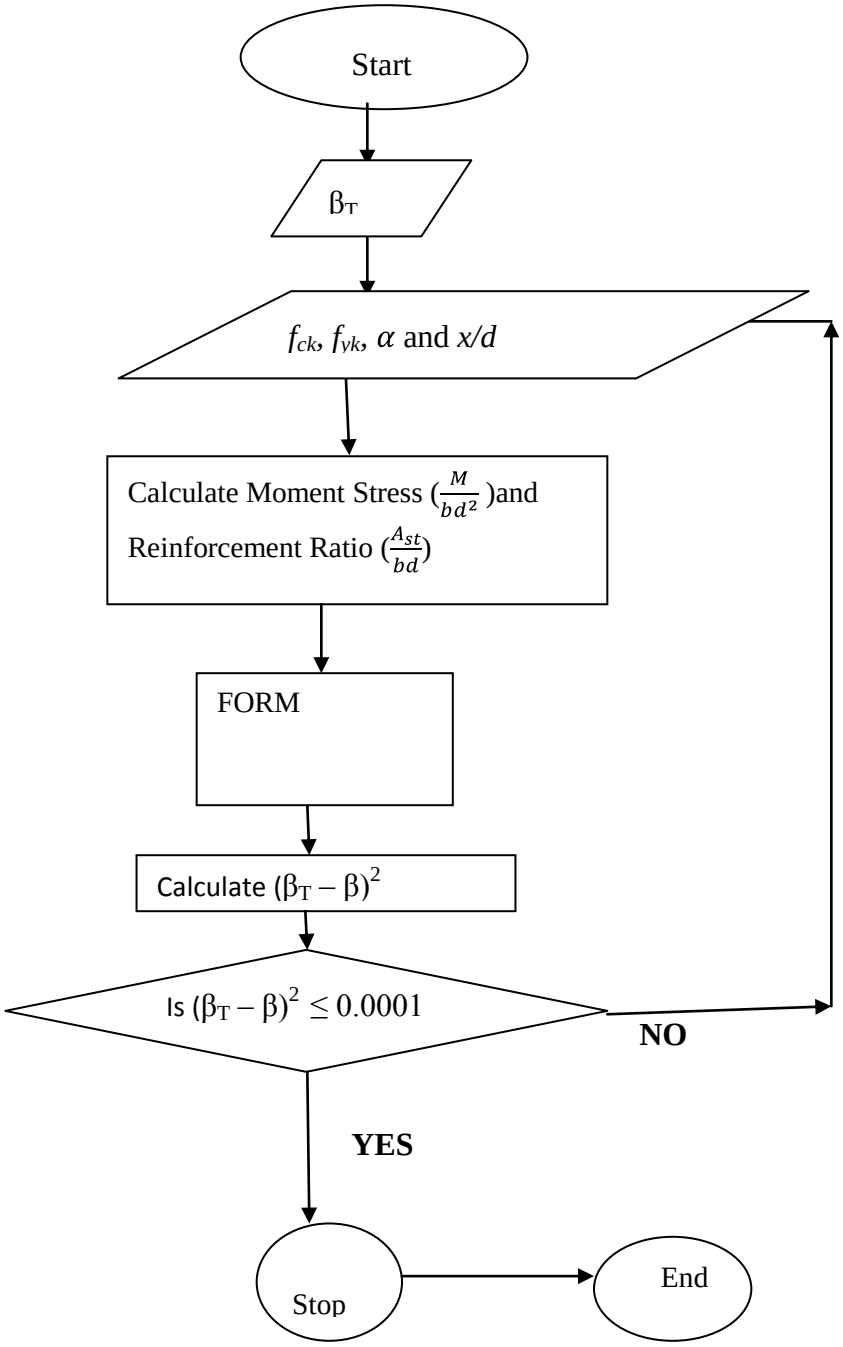


Figure 3.4: Program Flowchart for Iso-safety charts

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 ANALYSIS OF RESULTS

4.1.1 Plot of Eurocode 2 Design Charts

The Charts are plotted from the equations derived in section 3. Equations (3.18) and (3.12) define the abscissa and ordinate respectively. The results from which the charts are plotted are given in Appendix A. The charts are shown in Figures (4.1) to (4.3).

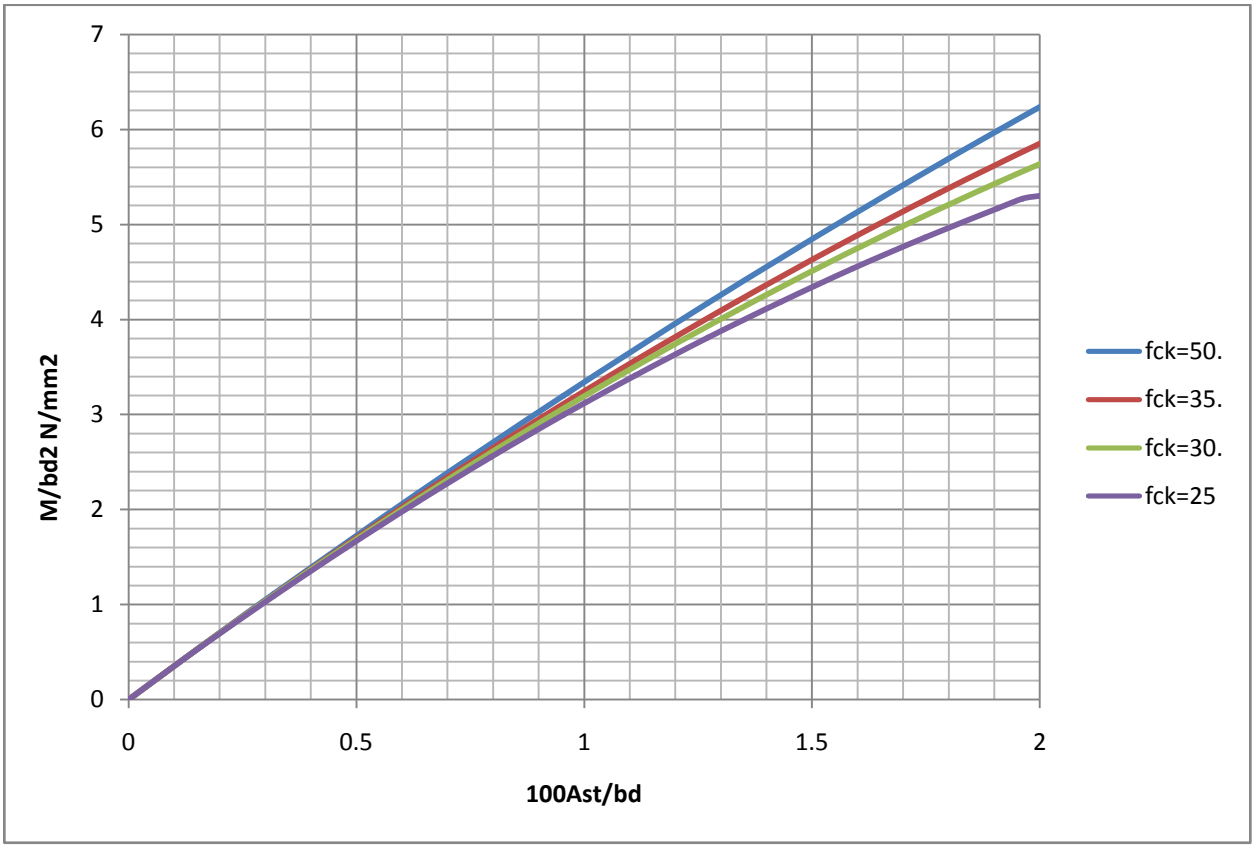


Figure 4.1: Design Chart for Singly Reinforced Rectangular Sections. f_{yk} = 410 N/mm^2

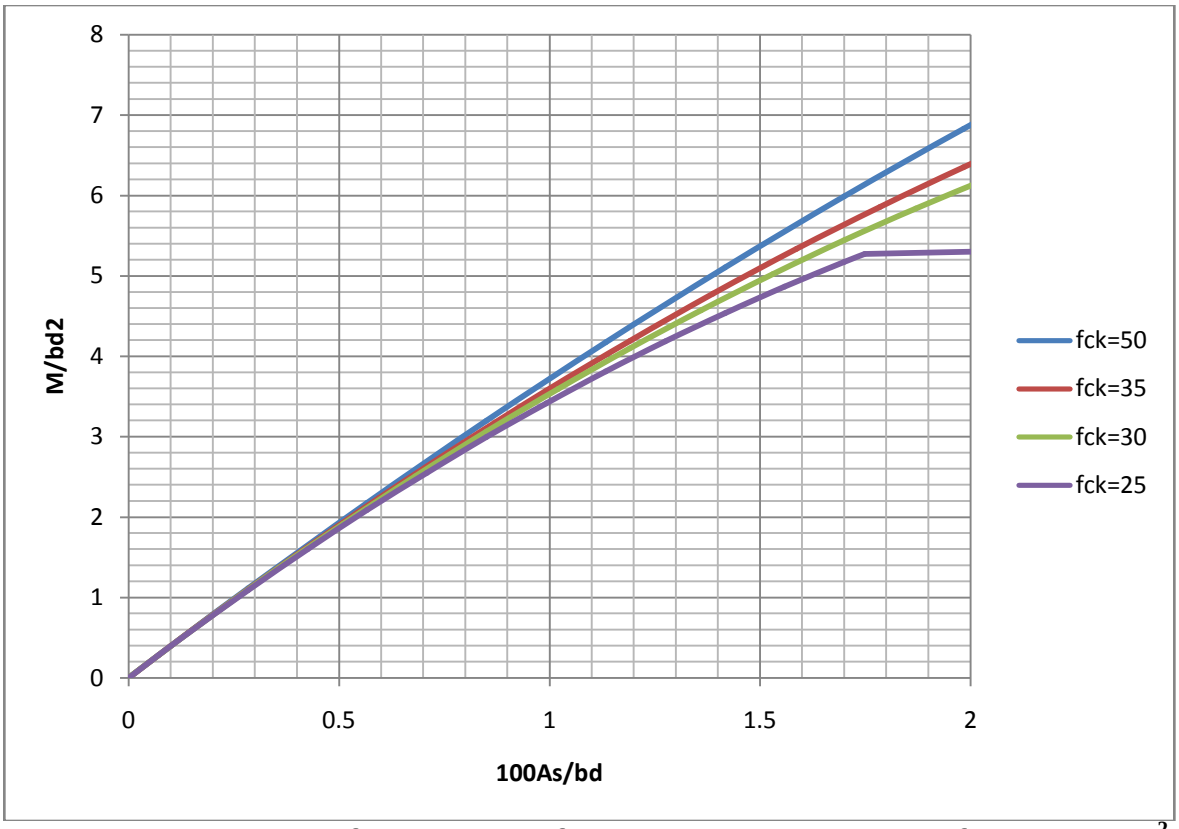


Figure 4.2: Design Chart for Singly Reinforced Rectangular Sections. f_{yk} = 460 N/mm^2

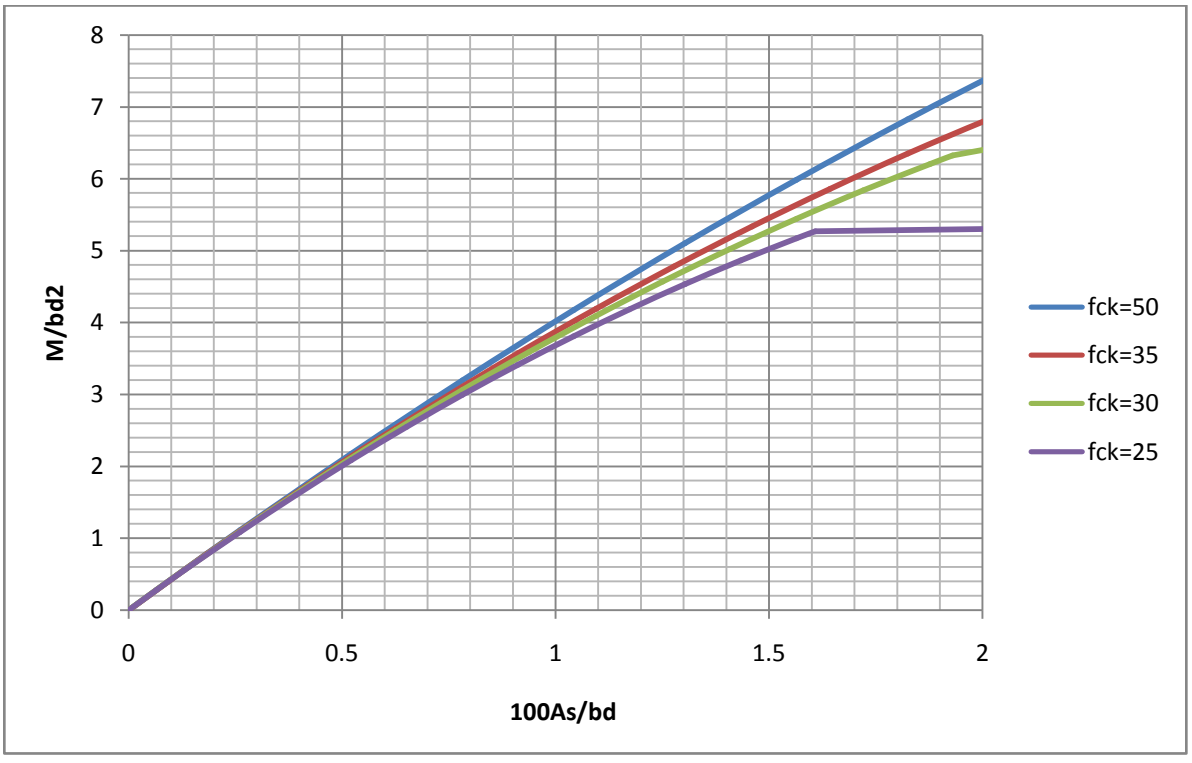


Figure 4.3: Design Chart for Singly Reinforced Rectangular Sections f_{yk} = 500 N/mm^2

4.1.2 Estimated Safety Index for the Generated Eurocode2 Design Charts

All the points on each curve are selected and the safety index, β, is computed for each point. The charts have 4 curves for steel grades 410, 460 and 500 N/mm^2 respectively. Each curve represents a particular grade of concrete.

Safety indices have been calculated and a typical result is given as β_0 in appendix B. These safety indices were calculated via FORM5 (Gollwitzer, *et al.,* 1988). The program "PROGRAM DESIGN CHARTS" was written for this purpose and is given in appendix C. A reliability index (β_0) of 1.81 is computed and is found to be uniform between grades of steel and concrete which shows the consistency of the design charts.

4.2 ISO-SAFETY DESIGN CHARTS

In developing the Iso-safety charts, the coefficient 'α' was multiplied to the load effect (Equation (3.20)) so as to meet the desired safety target. The use of the charts given below will provide explicit information on the safety index of the sections being designed. The iso-safety charts are given in the Figures 4.4 to 4.12 for the singly reinforced concrete sections.

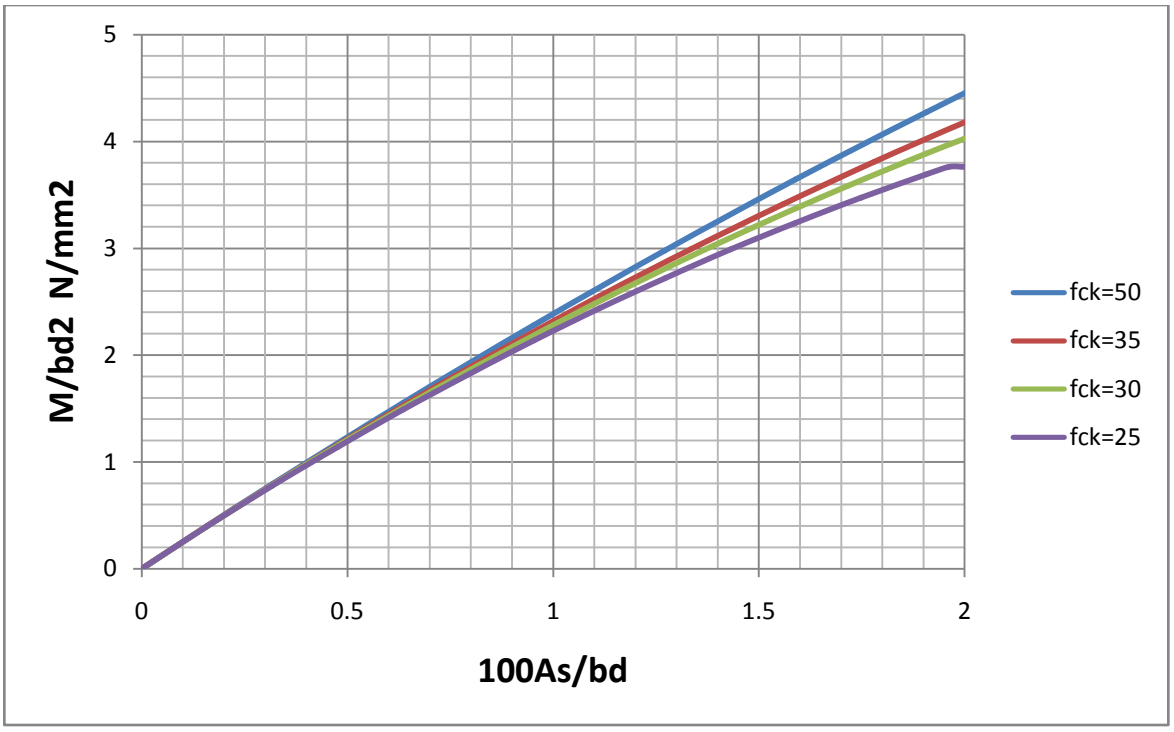


Figure 4.4: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 3.3 and f_{yk} = 410 N/mm^2

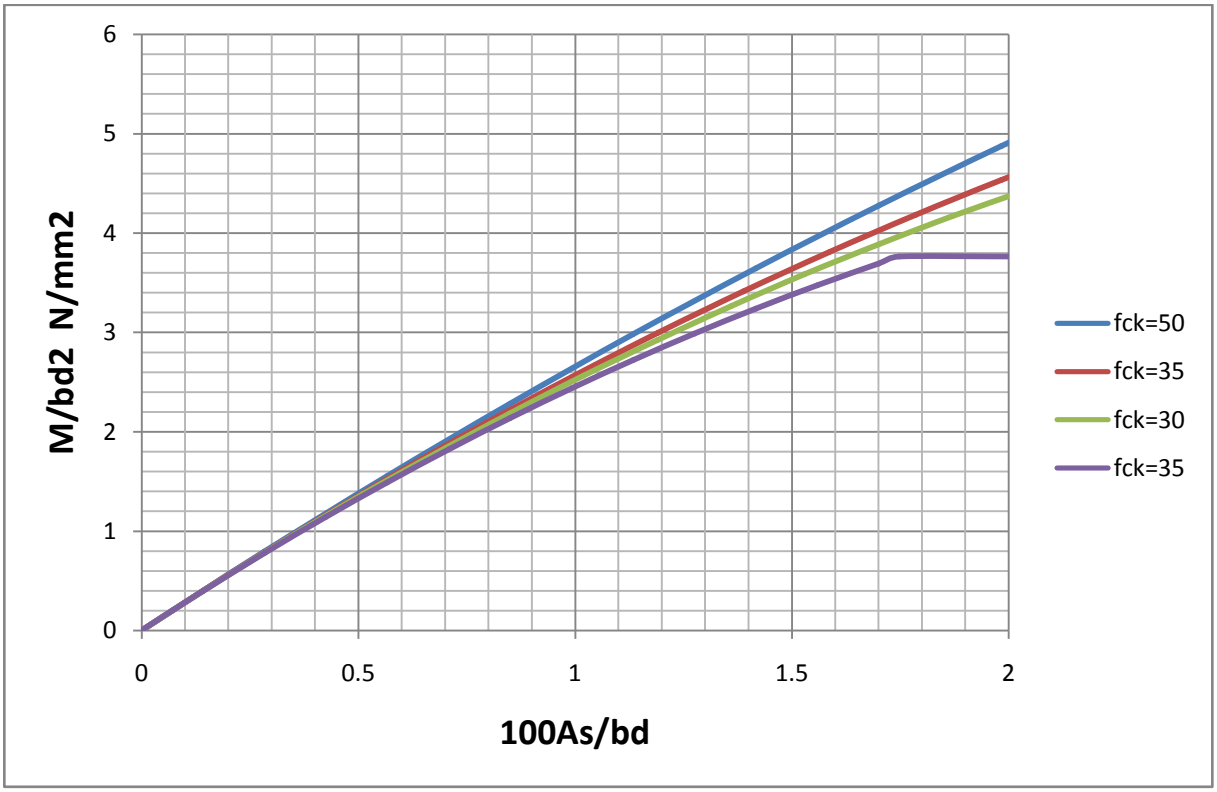


Figure 4.5: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 3.3 and f_{yk} = 460 N/mm^2

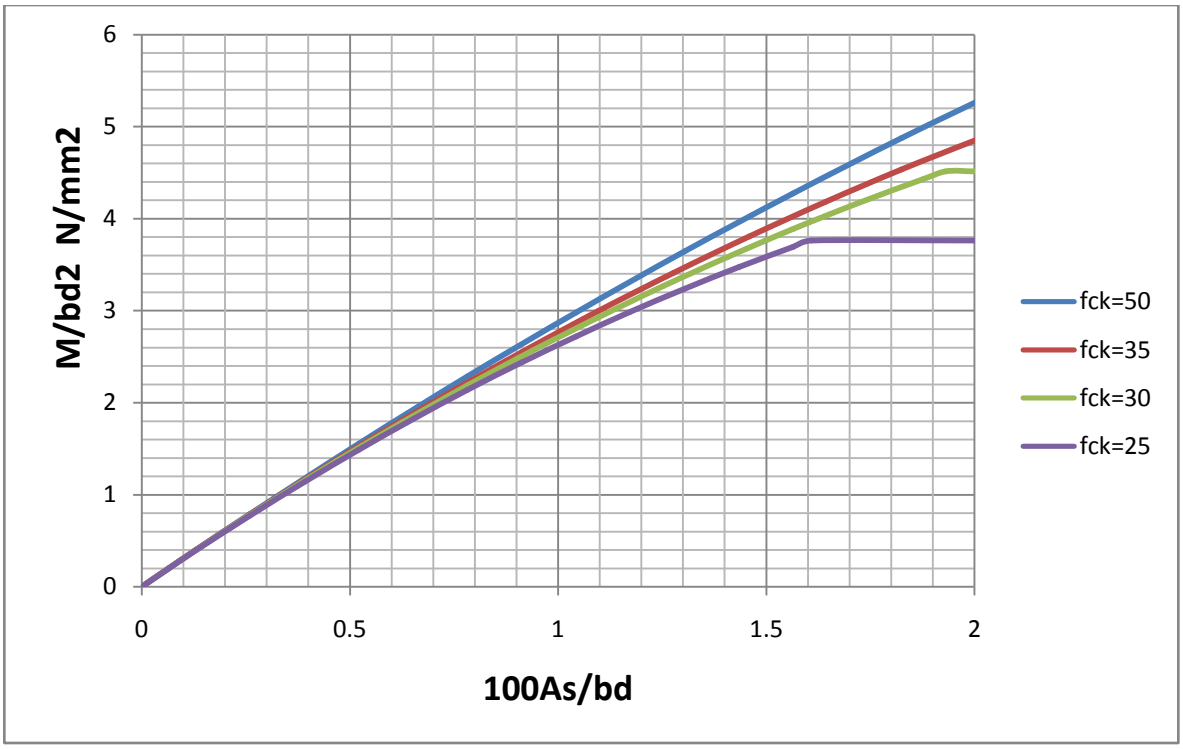


Figure 4.6: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 3.3 and f_{yk} = 500 N/mm

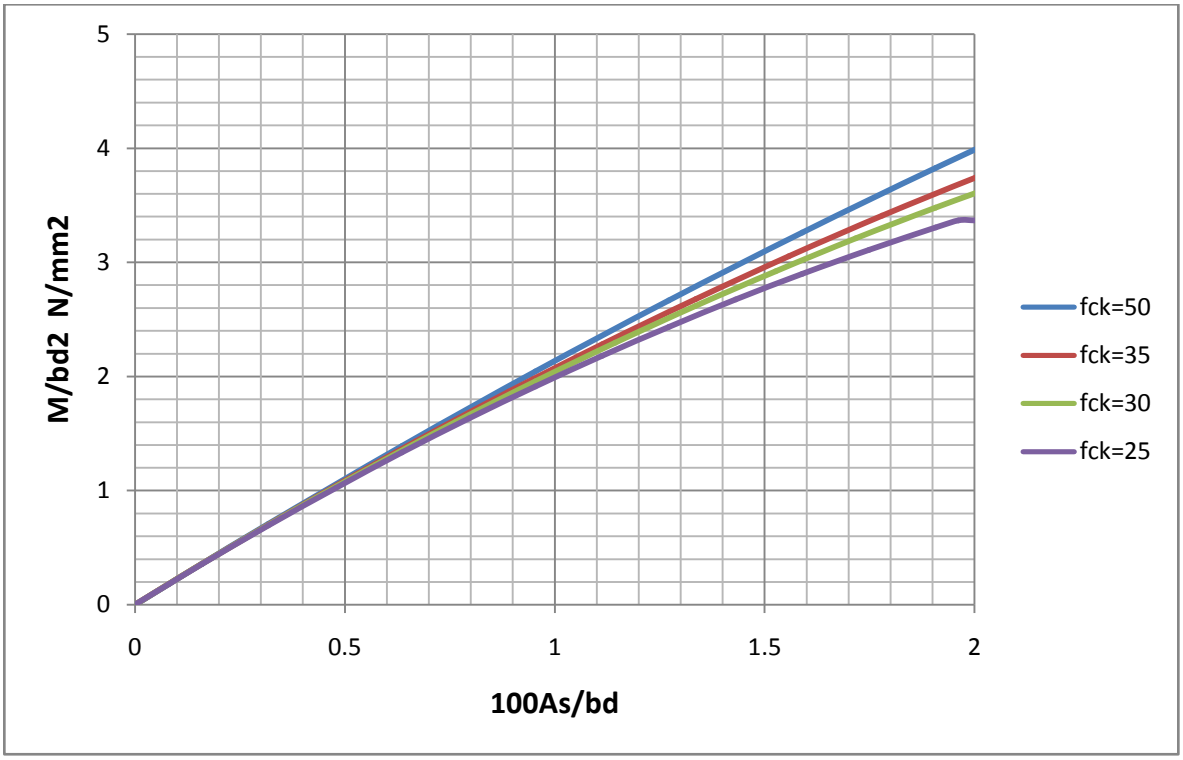


Figure 4.7: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 3.8 and f_{yk} = 410 N/mm^2

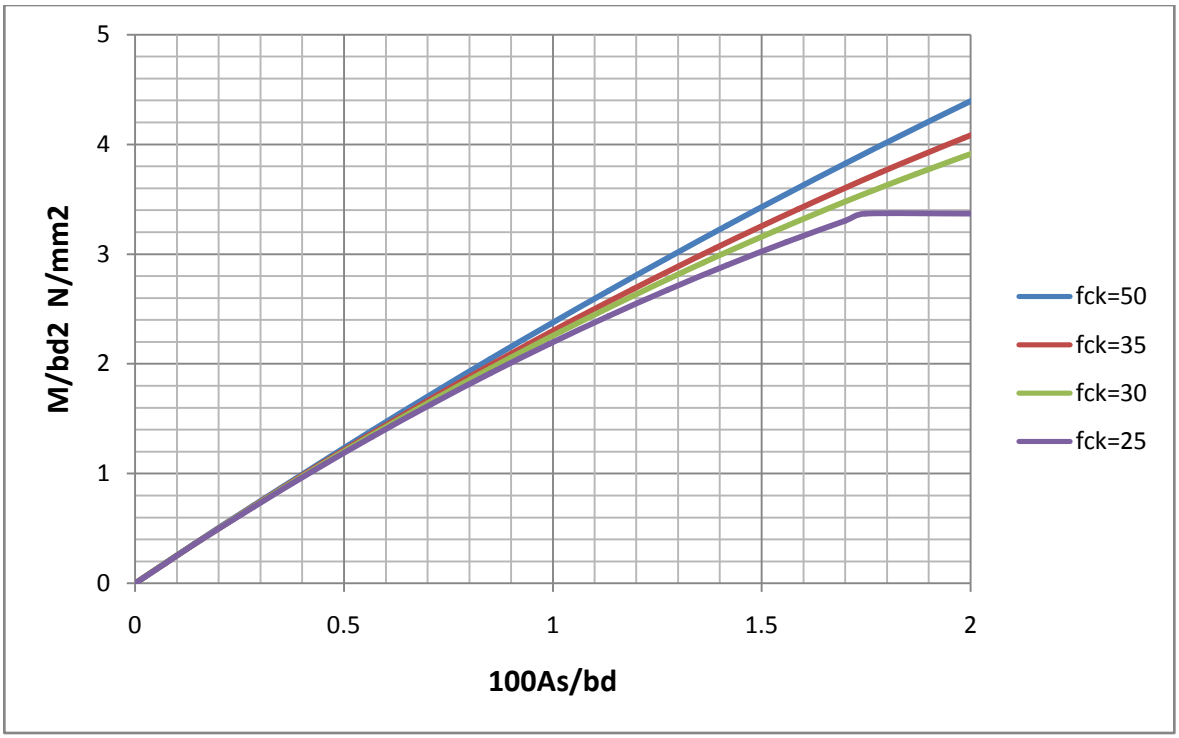


Figure 4.8: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 3.8 and f_{yk} = 460 N/mm^2

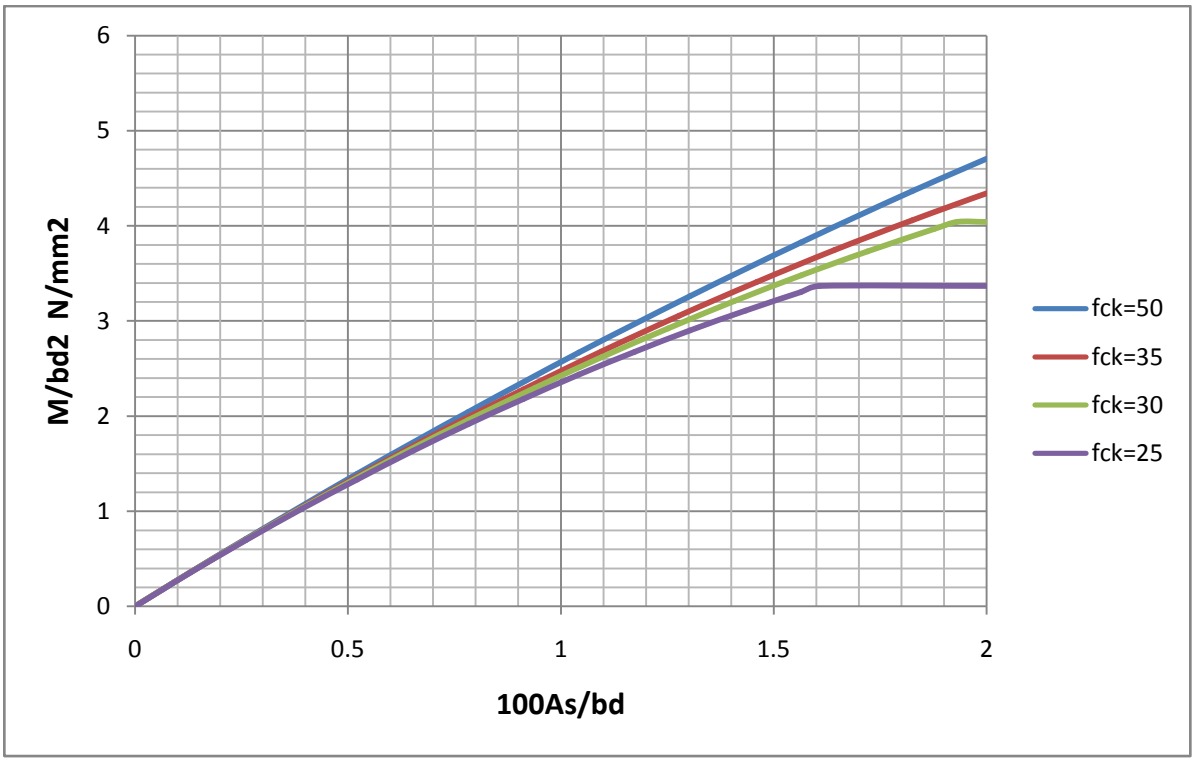


Figure 4.9: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 3.8 and f_{yk} = 500 N/mm^2

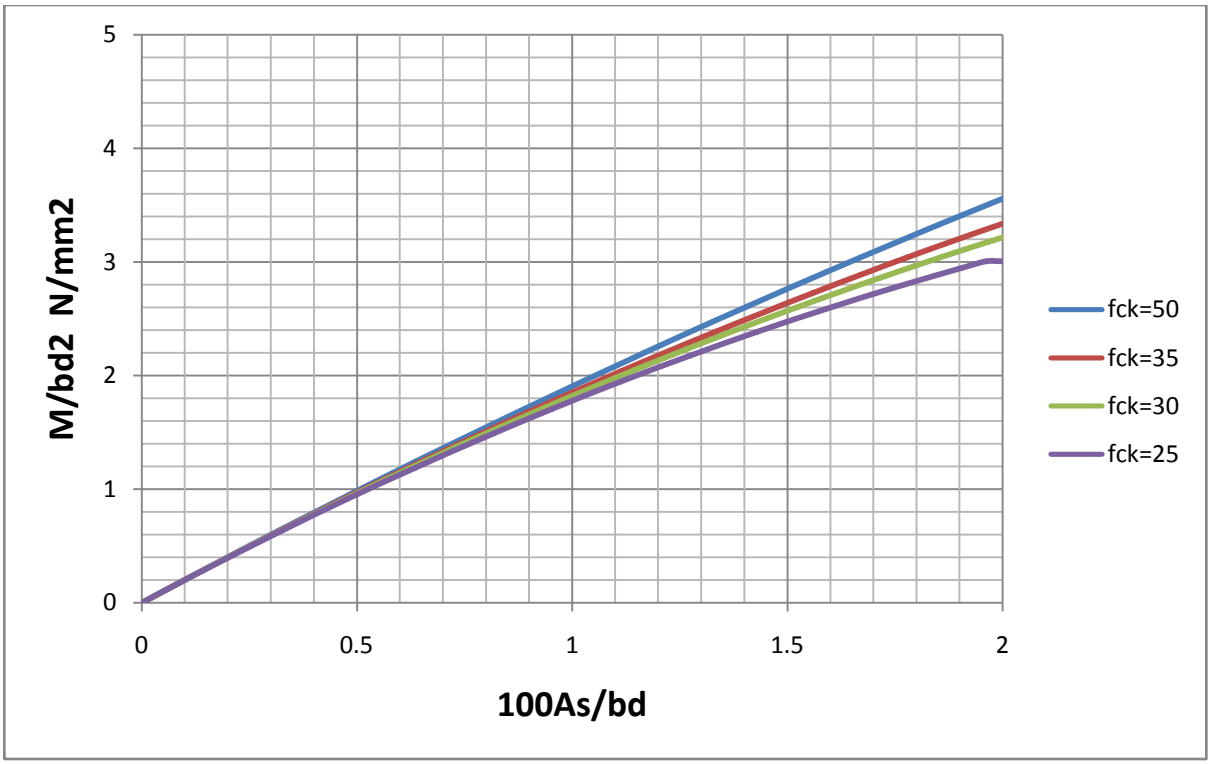


Figure 4.10: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 4.3 and f_{yk} = 410 N/mm^2

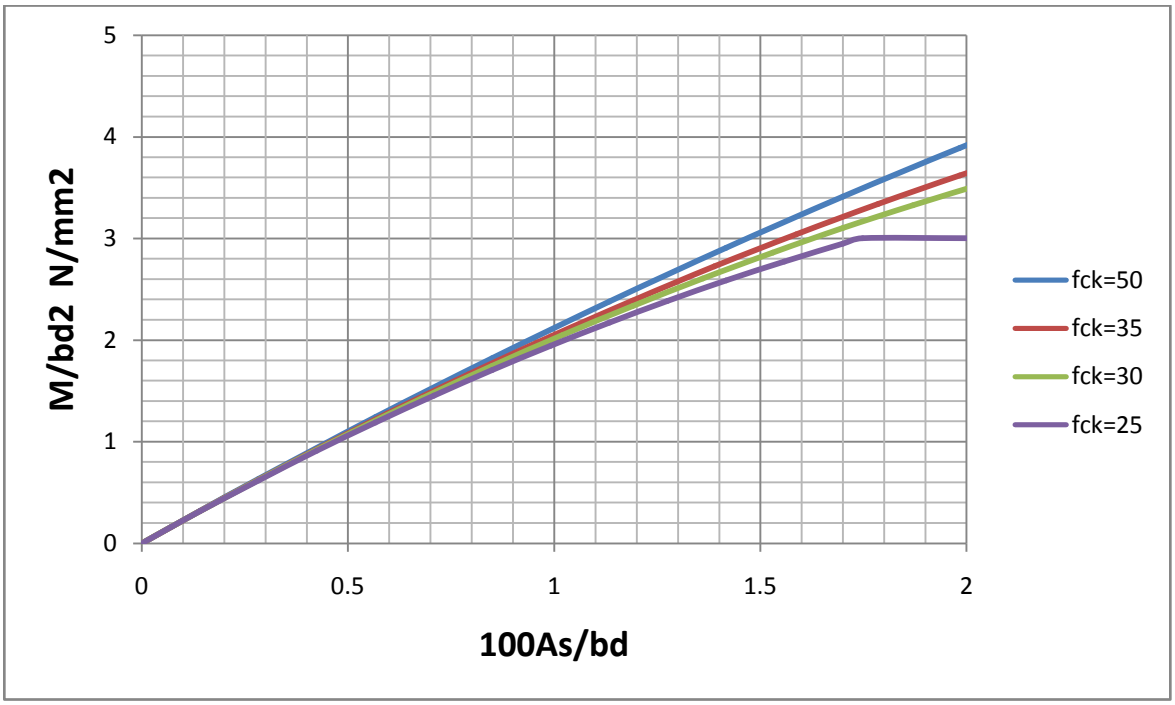


Figure 4.11: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 4.3 and f_{yk} = 460 N/mm^2

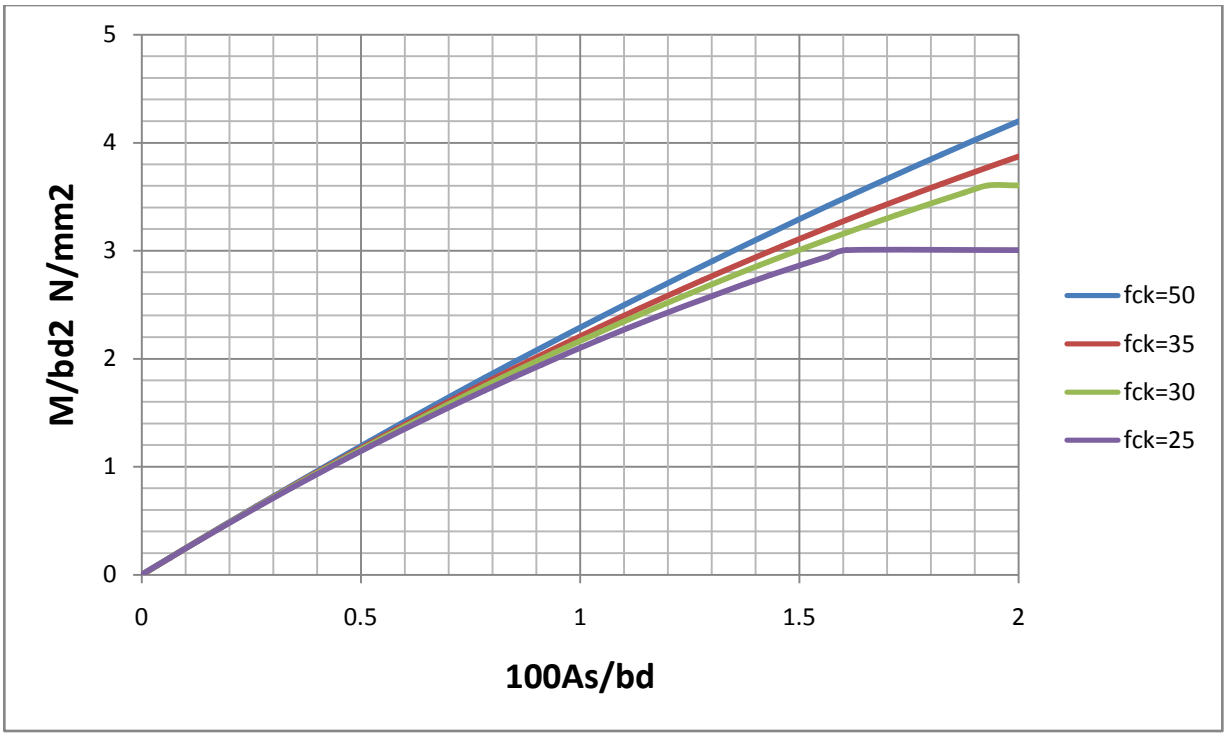


Figure 4.12: Iso-safety Design Chart for Singly Reinforced Concrete Sections, β = 4.3 and f_{yk} = 500 N/mm^2

4.3 PUNCHING SHEAR AND DEFLECTION

Porco et al. (2013) examined punching shear using various codes (Eurocode and model code (2010)) and concluded that in all codes punching shear capacity calculations are to be based on the critical perimeter which is located between 0.5 and 2d from the face of the column and also in both codes examined the punching shear capacity depends on the flexural reinforcement ratio, though its influence is quite different in each code.

The primary aim of this analysis was therefore to portray the influence of flexural reinforcement ratio provided using EC 2 design requirements and flexural reinforcement ratio provided based on the developed iso-safety charts having β_T of 3.3, 3.8 and 4.3

4.3.1 Effect of Flexural Reinforcement on First Critical Section Punching Shear Safety

Figure 4.13 shows the effect of percent flexural reinforcement on the first critical section punching shear safety index of the flat slab, the analysis is carried out with or without a slab drop panel at the column position. It can be seen from the plot that at lower reinforcement ratio values of 0.1 to 0.3% the safety index is fairly constant and starts to increase with increase reinforcement ratio at around 0.4%, this is due to the fact that EC 2 proposes the use of minimum punching shear capacity values at lower reinforcement ratio values, the minimum is not a function of provided flexural reinforcement, only when the minimum is exceeded by the equation which is a function of flexural reinforcement that the minimum is discarded.

The plot also shows that without drop the safety index at the first critical section is very low compared to that with drop, a β value - 0.159 was achieved without drop compared to 1.73 with drop at 0.1 to 0.3% reinforcement ratios, at a maximum ρ value of 2% without drop β = 2.24 while with drop it is 4.4, this shows that without drop at the first critical section the punching shear safety of the flat slab is low as the target β of 3.3, 3,8 and 4.3 recommended EC0 (2002) for the three failure consequence classes were not achieved, with a slab drop panel at 0.9, 1.5 and 1.9% reinforcement ratios the β targets 3.3, 3.8 and 4.3 were respectively achieved.

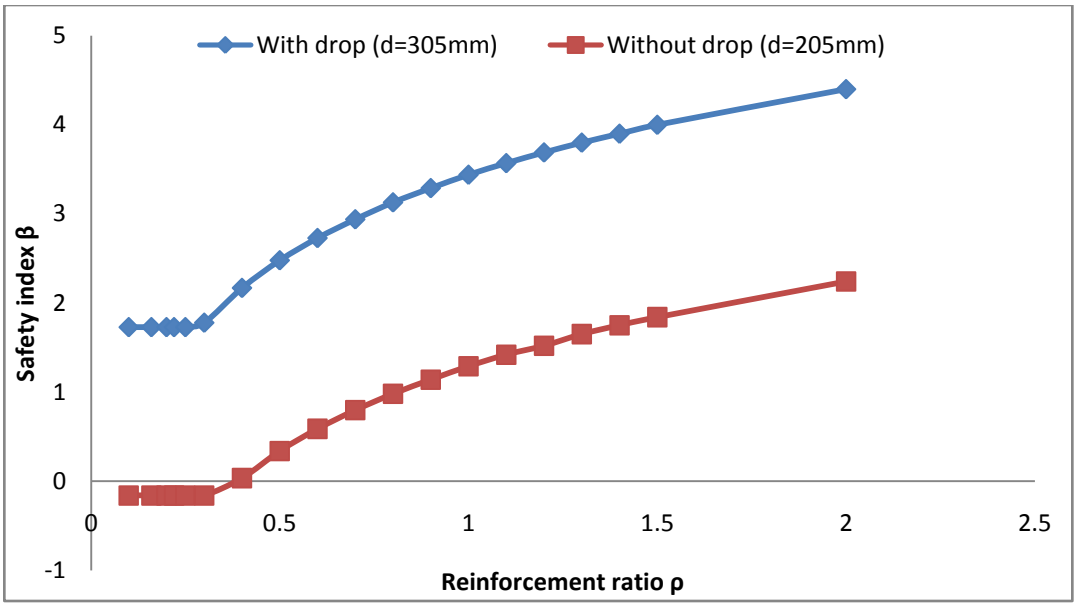


Figure 4.13: Safety Index versus Reinforcement ratio (at the First Critical Section from the Column face)

4.3.2 Effect of Slab Effective Depth on Punching Shear Safety at First Critical Section

The variation of punching shear safety index β with flat slab effective depth at the first critical section is presented in Figure 4.14. The figure shows the variation for four (4) different flexural reinforcement amounts, that is, when the flexural reinforcement provided is in accordance with EC2 and when β targets of 3.3 3.8 and 4.3 were set for the amount of reinforcement provided. The results show that there is no significant difference in the β values with the different flexural reinforcement safety index targets for the slab considered this may be due to the fact though punching shear capacity is a function of flexural reinforcement but it's influence low in Eurocode as explained by Porco *et al.,* (2013) especially at low reinforcement ratio values; a noticeable difference in the β can be seen at an effective depth of 400mm for a β target 4.3. The safety index values increase with increase in effective depth of the slab, at a depth of 200mm the safety index value was -0.266 at 250 mm and 400mm β=0.742 and 3.22 respectively for EC2, β_t of 3.3 and 3.8 flexural requirement but β = 3.43 for

β_t of 4.3 flexural requirement. Generally β of 3.3 and 3.8 is achievable between 400 and 450mm and 480mm for β=4.3 in the first three flexural reinforcement requirement except when flexural β_T= 4.3 where a β of 3.3, 3.8 and 4.3 were achieved at 400, 410 and 450mm respectively.

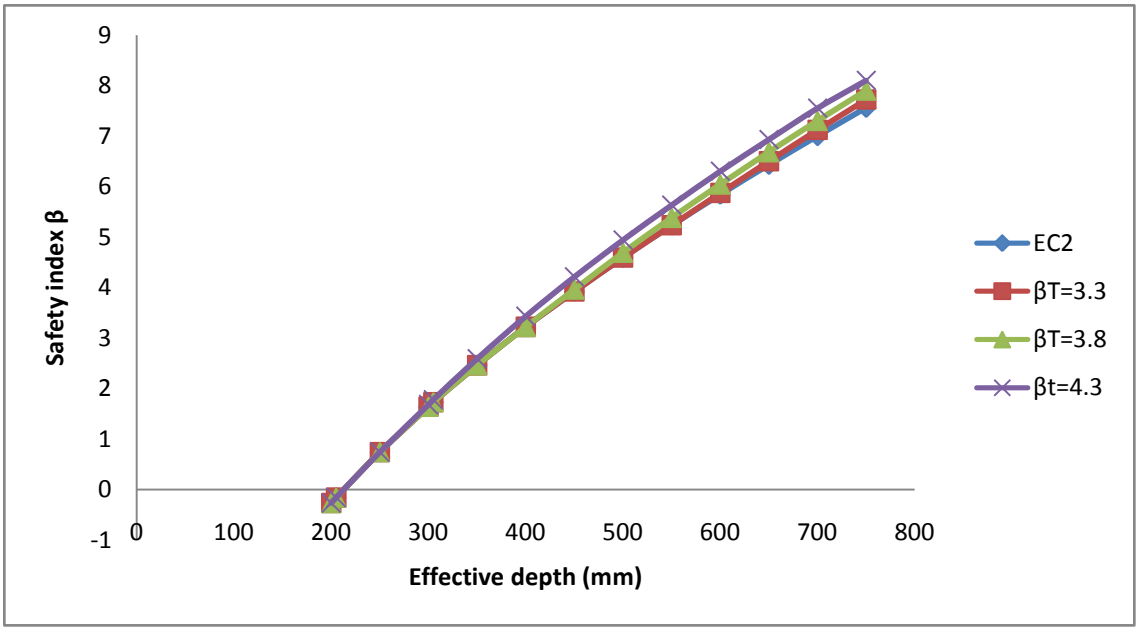


Figure 4.14: Safety Index versus Effective Depth for Different Target flexural reinforcement at the First Critical Section

4.3.3 Effect of Varying Concrete grade on Punching Shear Safety

An important parameter in the design of reinforced concrete elements is the concrete grade (Characteristic strength of concrete). Figure 4.15 shows the variation of safety index with concrete grade at different punching shear sections of a flat slab. It can be observed that the safety index increases with increase in concrete strength and vice-versa; this is so because there is an increase in the crushing strength of the concrete, in other words the strength of the section is increased with corresponding increase in section punching shear capacity, there by producing a safer and more reliable structure. The concrete grade vary from 25 to 50N/mm^2

and β values correspondingly change from 6.99 to 9.25 for punching at the column face, 3.18 to 4.58 and 1.73 to 3.12 at the critical section from drop panel and first critical section respectively. This shows that the column face is safer, followed by Critical section from drop panel then the first critical section.

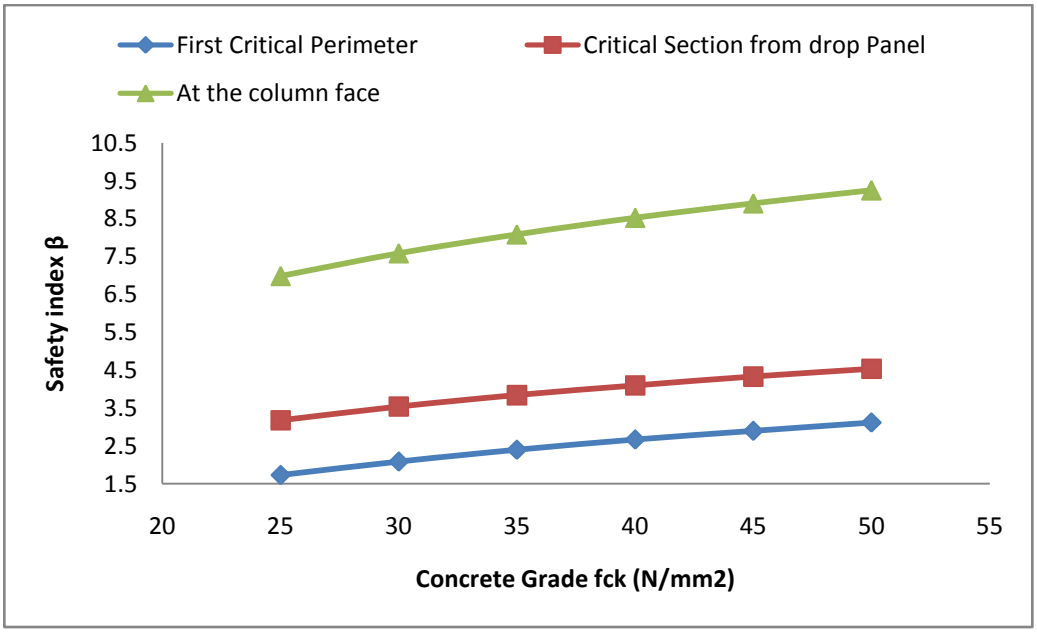


Figure 4.15: Safety Index versus Concrete Grade of flat slab (Three Punching Shear Sections)

4.3.4 Effect of Load ratio (Variable to Permanent) on Punching Shear Safety.

The result of varying load ratio from 0.1 to 1.4 for punching of flat slab at the face of the column, First critical section and critical section from the panel drop are presented in Figure 4.16. As observed by Jibrin *et al.,* (2014) with increase in load ratio a decrease in safety index was also observed. This can be attributed to the fact that as load increases on structural elements at constant design situation, the probability of failure also increases, at the design load ratio of 0.755 a β value of 1.7, 3.2 and 7.0 was observed for first critical section, critical section from drop panel and at the column face respectively, this further prove that the first critical section is the least safe and should be governing section for punching shear design. To

achieve the recommended 3.8 target safety index, the load ratio should not be more than 0.2 for the first critical section and 0.5 for the critical section from drop panel while the column face can withstand greater loads.

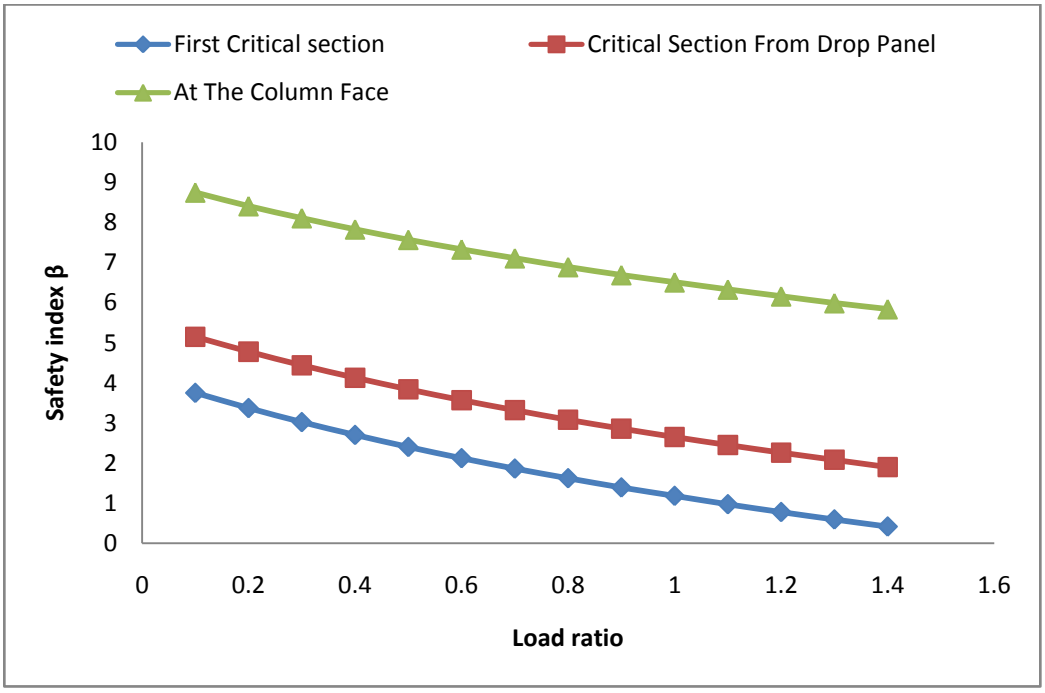


Figure 4.16: Safety Index versus Load Ratio (at the Three Punching Shear Sections)

4.3.5 Effect of Column Head Size on Safety Index

In Figure 4.17 an increase in safety index was observed with increase in the column head diameter, this is so because increasing the column head diameter reduces the total surface area of the slab itself there by reducing the total loading area controlled by the flat slab. The figure shows the variation at the first critical section and at the column face, the critical section from drop panel is a function of drop dimension not column head therefore not included in this figure, should the target β of 3.8 is to be achieved by controlling the column head diameter, a minimum diameter of not less than 2300mm must be ensured for the first critical section and 500mm considering the column face.

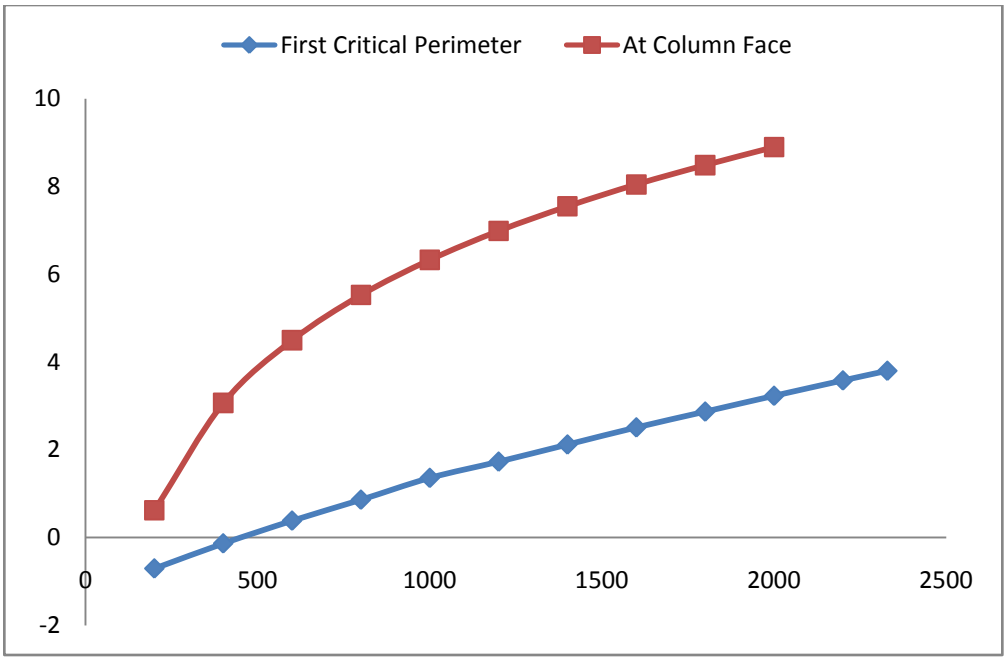


Figure 4.17: Safety Index versus Column Head Diameter (at first critical section and column face)

4.3.6 Effect of Flexural Reinforcement on Critical Section from Panel drop Punching shear safety

The effect of increasing flexural reinforcement ratio on the safety index for punching of flat slab at the critical section from panel drop is presented on Figure 4.18. The result follows a similar trend with that of first critical section but in this case the section is safer as at 0.1% reinforcement ratio the safety index is 3.18 compared to -0,159 on the same point at the first critical section. This is so because as the section moves away from the column, the total loading area contributing to punching at the section reduces and also as explained on Figure 4.13, the punching shear capacity of the section at lower reinforcement ratio values is constant.

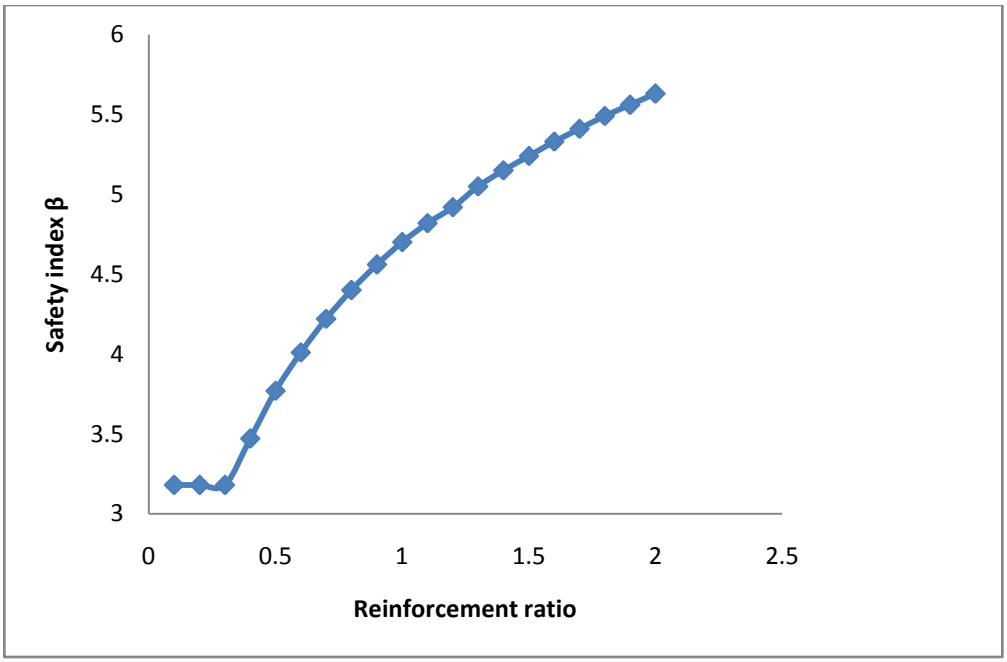


Figure 4.18: Safety Index versus Reinforcement ratio (at Critical Section from Drop Panel)

4.3.7 Effect of Slab Effective Depth on Punching Shear Safety of Critical Section from Panel Drop

As pointed out by Porco *et al.*, (2013) that the influence of flexural reinforcement ratio is quite different in each code, the influence in EC2 is insignificant at lower reinforcement ratio values. Figure 4.19 shows the variation of safety index with effective depth at the critical section from drop panel for EC2 and Target β 3.3, 3.8 and 4.3 flexural reinforcement provisions, as explained in the case of first critical section, the safety index is not significantly affected by changes in the amount of flexural reinforcement for the slab considered. At this section the safety index is higher at the same effective depth compared to first critical section of the slab, to achieve a punching shear safety index of 3.8, the effective depth should averagely be 250mm for EC2, β_T of 3.3, 3.8 and 4.3 flexural reinforcement provision.

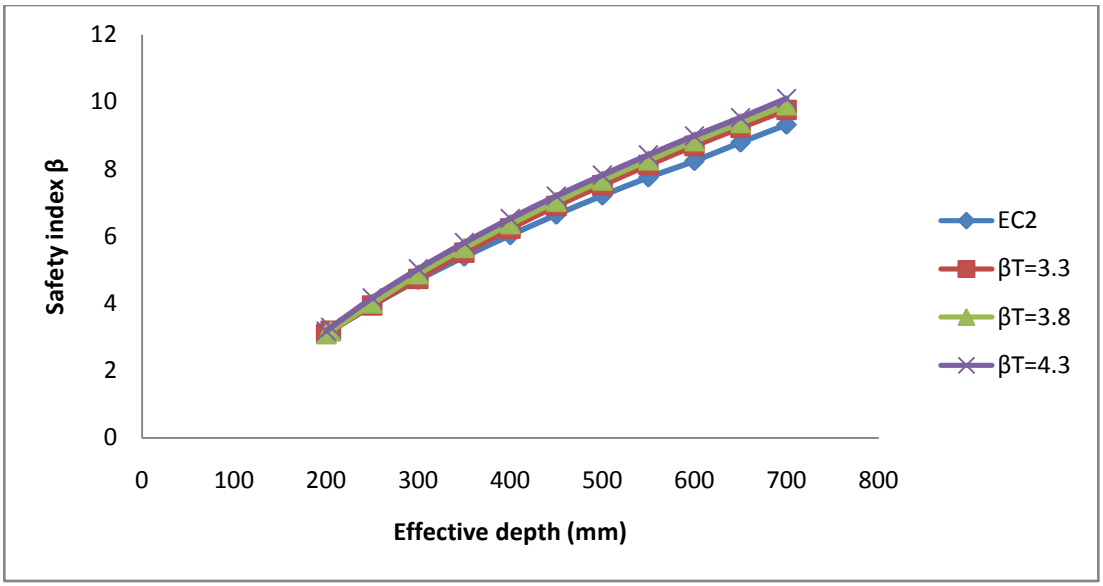


Figure 4.19: Safety Index versus Effective Depth for Different Target flexural reinforcement at Critical Section from Drop Panel

4.3.8 Effect of Panel Drop size on the Safety of Critical Section from Panel drop

Figure 4.20 shows an increase in safety index values with increase in panel drop dimension at the critical section from drop panel starting from design column head dimension of 1200mm to 3000mm. This is because the critical section perimeter is increased thereby reducing the loading area contributing to punching of the sections, at 2800mm for a normal use flat slab a safety index of 3.8 will be achieved at the section.

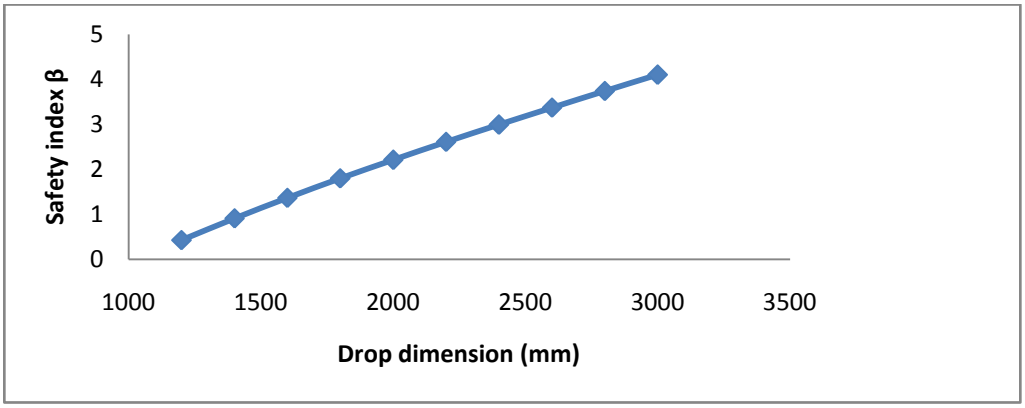


Figure 4.20: Safety Index versus Drop Dimension at the Critical Section from the Drop Panel of a Flat Slab

4.3.9 Effective depth Effect on Column Face Safety

From figure 4.21, safety index values increase with effective depth increase. This is due to the fact that as the effective depth increases the punching cross sectional area increases with corresponding punching shear capacity increase. The column face is therefore very safe in terms of punching because even at a low effective depth of 200mm (i.e without drop) the safety index was 5.5 which is high compared with the 3.18 at the critical section from drop panel. At 305mm effective depth which is the design effective depth at the column face, the safety index observed was 6.93. This shows that punching wise the column face is the least of a designer's problem.

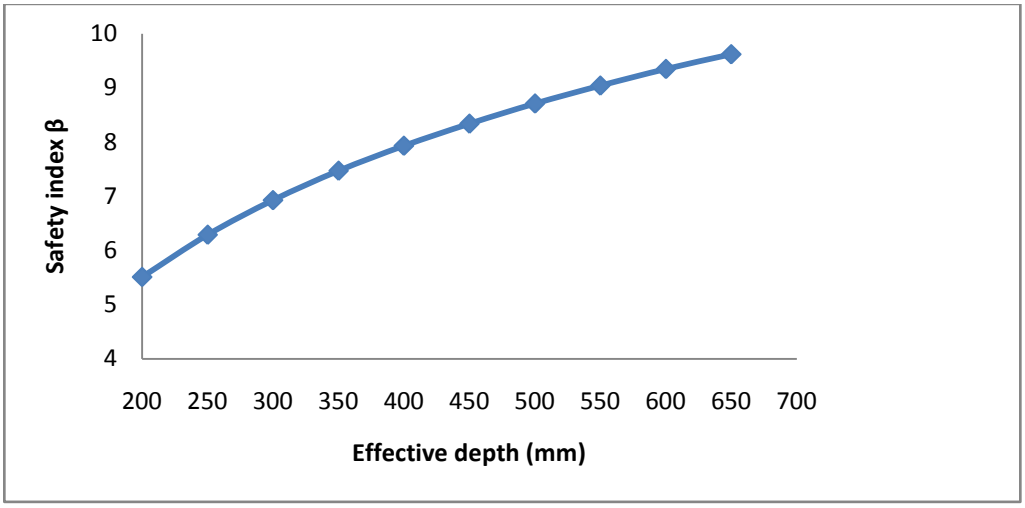


Figure 4.21: Safety Index versus Effective Depth at the Column Face of a Flat Slab

4.3.10 Effect of Flexural Reinforcement on Deflection

EC0 (2002) recommends a target β of 1.5 for serviceability limit states, therefore 1.5 will be the reference safety index value for deflection.

The relationship between safety index and percent flexural reinforcement on deflection is as shown in Figure 4.22. In this plot safety index increases with increase in percent flexural

reinforcement, this may be due to the fact that as the amount of flexural reinforcement is increased at constant effective depth of slab, the section becomes more rigid resulting in a safer section. The percent reinforcement was varied from 0.1 to 2% with safety index ranging from 0.694 to 11.413 respectively. For the section considered at the design ρ of 0.2% the safety index was 3.042, when the flexural ρ = 0.28%, 0.31% and 0.35% corresponding to flexural β_t = 3.3, 3.8 and 4.3 respectively, the deflection safety index observed was 4.01, 4.45 and 5.00 respectively. The target β of 1.5 recommended by EC0 was observed at 0.14% flexural reinforcement.

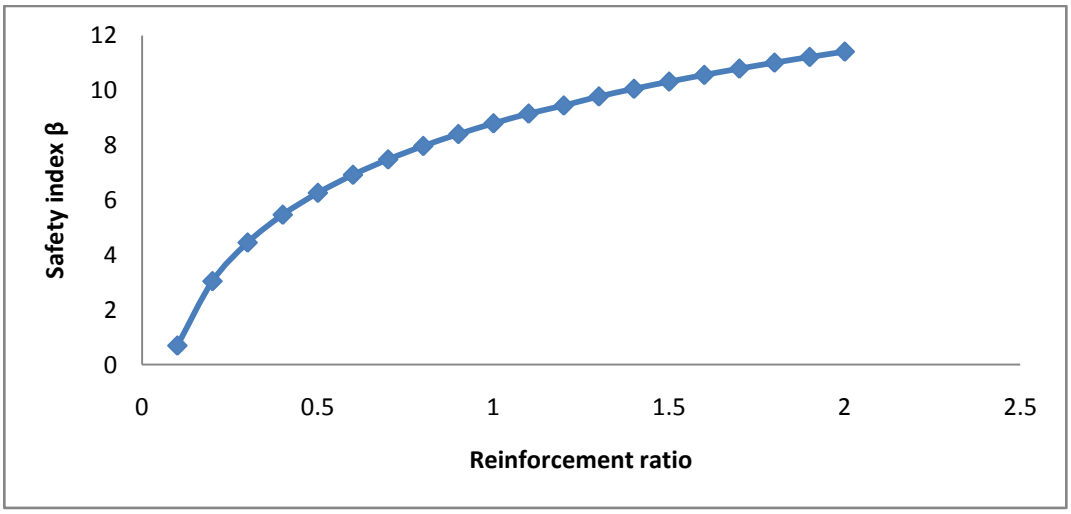


Figure 4.22: Safety Index versus Reinforcement Ratio, for the Deflection of Flat Slab

4.3.11 Effect of varying Concrete Grade on Deflection

Figure 4.23 shows a relationship between the characteristic strength of concrete and safety index, at 25N/mm^2 β = 3.0 which gradually increases to 5.62 at 50N/mm^2. The safety index increase may be attributed to the fact that increasing the strength is same as increasing concrete stiffness there by producing a more stable concrete with a low probability of deflection failure.

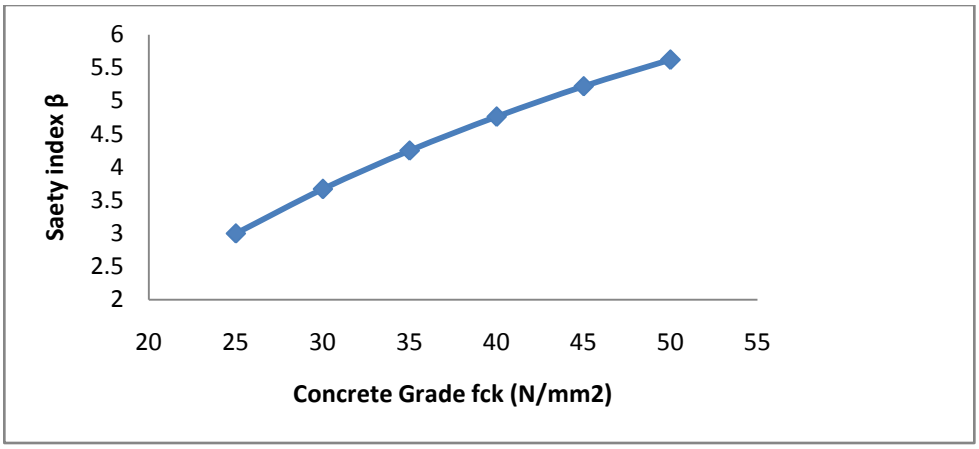


Figure 4.23: Safety Index versus Concrete Grade, for the Deflection of Flat Slab

4.3.12 Deflection safety variation with Slab Effective Depth

The effective depth of flat slab is an important parameter in the deflection of flat slabs. Figure 4.24 shows the variation of safety index with effective depth of slab, at 200mm, the minimum recommended by the code, a safety index of 3.0 was observed which increases with effective depth increase. Deflection is a serviceability limit state and a β of 1.5 is ok as recommended by the code, this shows that the depth is satisfactory

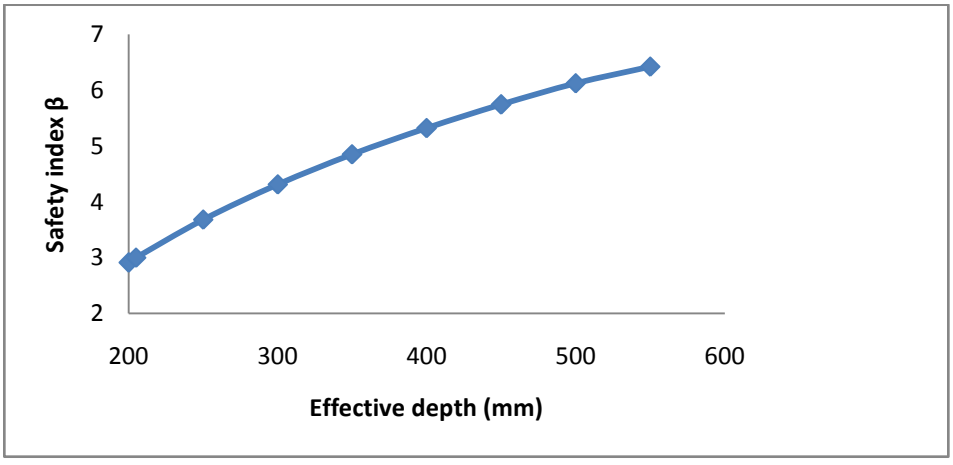


Figure 4.24: Safety Index versus Effective Depth, for the Deflection of a Flat Slab

4.3.13 Effect of Slab Length on Deflection Safety

Figure 4.25 shows what happens to safety index as the length of the flat slab increases or decreases. The safety index was found to decrease with increase in the length of the slab, this may be because according to Mosley *et al.*, 2007, The length of the flat slab greatly influence its deflection as deflection is more controlled by placing a limit on the span and depth ratios, in this case the span is increased at constant depth (i.e limiting deflection increased). A safety index of 3.0 was found to correspond to the design length of 6.5m. Not going below the safety index target recommended by EC0 of 1.5 a length of 9.2m is ok, going above 9.2m doesn't mean the slab fails but the safety index will be lesser than the target. From the plot a deflection failure of the flat slab under consideration will be assumed imminent when the length reaches 12m that is when $\beta = -0.187$ and other design parameters are kept constant.

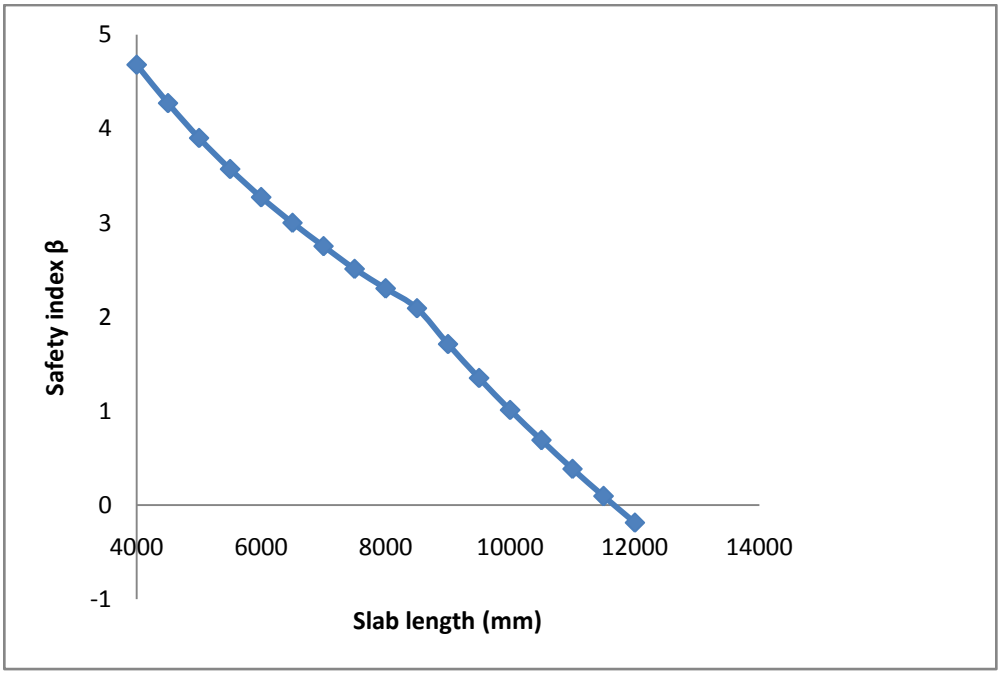


Figure 4.25: Safety Index versus Slab Length for the Deflection of Flat Slab

4.4 ILLUSTRATIVE EXAMPLE ON THE USE OF THE CHARTS

Design the cross section of a flat slab for the ultimate moment given:

f_{ck} = 25 N/mm^2

f_{yk} = 500 N/mm^2

Overall depth of slab = 250 mm

Drop panel dimension = 2.5 m square

Depth of drop panel = 100mm

Column head diameter = 1.2 m

Variable load = 5 kN/m^2

(Source: Mosley *et al.,* (2007))

Solution

Weight of slab + drop = $0.25\times25\times6.5^2 + 0.1\times25\times2.5^2 = 279.7$ kN

Ultimate load on floor = $1.35\times279.7 + 1.5\times5\times6.5^2 = 695$kN/panel

L = clear span between column head + (slab thickness at either end)/2

L = $(6.5 – 1.2) + (350\times2)/(2\times1000) = 5.65$ m

Assuming Mean depth of 2 layers of reinforcement = 20

Concrete cover = 25

Effective depth d = 250 – 45 = 205 mm at the span

d = 350-45 = 305 mm at the supports

Bending reinforcement:

From Table 2.1 for interior spans: +ve moment = 0.063FL = 0.063×695×5.65 = 247kNm

The width of the middle strip is (6.5 – 2.5) = 4m > half panel dimension

There proportion of moment taken by the middle strip can be taken as 0.45 (Table 2.4) (Mosley *et al.*, 2007)

$$0.45 \times \frac{4}{6.5/2} = 0.55$$

The middle strip +VE moment = 0.55 × 247 = 136kNm

Design:

$$\frac{M}{bd^2} = \frac{136 \times 10^6}{4000 \times 205^2} = 0.81$$

(a) from EC 2 design charts (Fig 4.2): $\frac{100A_{st}}{bd} = 0.2$

$$A_{st} = 0.2 \times 4000 \times 205 \times 10^{-2} = 1640mm^2$$

(b) When β = 3.3 (Fig 4.5): $\frac{100A_{st}}{bd} = 0.28$

$$A_{st} = 0.28 \times 4000 \times 205 \times 10^{-2} = 2296mm^2$$

(c) When β = 3.8 (Fig 4.8): $\frac{100A_{st}}{bd} = 0.31$

$$A_{st} = 0.31 \times 4000 \times 205 \times 10^{-2} = 2542mm^2$$

(d) When β = 4.3 (Fig 4.11): $\frac{100A_{st}}{bd} = 0.31$

$$A_{st} = 0.35 \times 4000 \times 205 \times 10^{-2} = 2870mm^2$$

$A_{stmax} = 0.04A_c = 0.04 \times 250 \times 4000 = 40000mm^2$

$A_{stmin} = 0.0013bd = 0.0013bd = 0.0013 \times 4000 \times 205 = 1066mm^2$

Since $A_{stmax} > A_{st} > A_{stmin}$ The Design is ok

Table 4.1: Summary of Design

S/No	Chart type	$100A_{st}/bd$	Area of Steel A_{streq}
1	Eurocode 2 Design Charts β = 1.81	0.20	$1640mm^2$
2	Iso-safety Design Chart β_t = 3.3	0.28	$2296mm^2$
3	Iso-safety Design Chart β_t = 3.8	0.31	$2542mm^2$
4	Iso-safety Design Chart β_t = 4.3	0.35	$2870mm^2$

From the results presented on table (4.1) , it was observed that EC 2 gave a cheaper area of steel requirement but the safety index of 1.81 indicate failure of the section at Ultimate Limit state as it is lower than the minimum recommended (3.3) by EC0(2002). Also using the iso-safety design, as the β_T was increased the area of steel required increases by 40%, 55% and 75% when β_T =3.3, 3.8 and 4.3 respectively for the example considered.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

Upon analysis of the results, the following conclusions are made;

a. Constitutive model for flexure of flat slabs in accordance with Eurocode 2 (2004) was derived and corresponding design charts were produced.

b. Reliability-based design charts (Iso-safety charts) were produced to target safety indices; β_T of 3.3, 3.8 and 4.3 as the minimum recommended for the three failure consequence classes by Eurocode 0 (2002)

c. The safety index of designed flat slab considering flexural failure criterion using Eurocode 2 was 1.81 which falls below the recommended value given by Eurocode 0 (2002)

d. It was shown using the developed charts that for the same loading and geometrical considerations, the area of flexural reinforcement required increased by 40%, 55% and 75% over Eurocode 2 design for corresponding target safety indices of 3.3, 3.8 and 4.3 respectively.

e. From the above, it was therefore shown that Eurocode 2 design of flat slabs considering flexural failure criterion at higher loading condition provides designs that are below the recommended target safety indices.

f. Three sections; the Column face, Critical section from column face and Critical section from drop panel were analyzed for punching shear failure, the results indicated that

flexural reinforcement has little significance on the punching shear safety of the flat slab considered.

g. In line with (6) above, the variation in effective depth also influence the safety of the slab with higher sections having higher safety indices

h. The first critical section (2d from the face of the column) is the most critical punching section of the three punching shear sections analysed. It was also noted that providing drop panels at flat slabs column positions greatly reduces the probability of punching shear failure.

i. Generally, Safety index increases with increase in characteristic strength of concrete, increase in column head diameter and drop dimension but reduces with increase in load ratio.

j. Deflection was found to be affected by the percentage of steel reinforcement provided,

k. The span and depth of the slab were found to be important deflection design parameters; increase in depth causes an increase in the safety index while a span increase causes a decrease in the safety index with corresponding higher failure probability.

5.2 RECOMMENDATIONS

1. The charts (EC2 and Iso-safety charts) proposed in this study are recommended for the ultimate limit state design of singly reinforced concrete rectangular sections.
2. The results obtained from sensitivity analysis are recommended to serve as guide to structural engineers under similar design situation.

3. It is recommended that further studies involving system-reliability analysis be extended to this research so as to include punching shear design and serviceability checks into the design charts.

REFERENCES

Abubakar, I. (1999). ‘’Automated Reliability-based Program for One-way Slabs using BS8110 (1985)’’ A Thesis Submitted to the Postgraduate School Ahmadu Bello University Zaria, for the Award of Master of Science Degree, Department of Civil Engineering, Ahmadu Bello University Zaria.

Abubakar, I. (2006) ‘’ Reliabilty analysis of Structural Design Parameters of strip footings’’ *Journal of Applied Science and Research* 2(7): 397-401,INSInet Publication

Abubakar, I. and Mohammed, U. A., (2007), “Reliability Investigation of Steel Cased Columns*”, Australian Journal of Basic and Applied Sciences*, 1(4): pp 561-570.

Abubakar I and Pius Edache (2007) ‘’ Reliability Analysis of Simply Supported Steel Beams’’ *Australian Journal of Basic and Applied Sciences*, 1(1): 20-29,

Abejide O.S., (2014) ‘’Reliability Analysis of Bending, Shear and Deflection Criteria of Reinforced Concrete Slabs’’ *Nigerian Journal of Technology*. (33) 3: 394 - 400

Afolayan, J. O., (2005), “Probability-Based Design of Glued Thin-Webbed Timber Beams”, *Asian Journal of Civil Engineering (Building and Housing)* 6, (1-2): 75-84

Akindahunsi A, A. Afolayan J O., (2009) *“*Developed Reliability-Based Interaction Curves For Design of Reinforced Concrete Columns*” Jornal of Theoretical and Applied Mechanics* 47, (4) : 943-955.

Anitha M, Rahman B Q and Vijay JJ (2007) ‘*’Analysis and design of flat slabs Using various codes*’’ International Institute of Information Technology Hyderabad, (Deemed University)

Atim, I. T., (2006), "Reliability-Based Design Charts for Short Symmetrically Reinforced Concrete Columns", A Thesis Submitted to the Postgraduate School Ahmadu Bello University Zaria, for the Award of Master of Science Degree, Department of Civil Engineering, Ahmadu Bello University Zaria

Bartlett F.M, Hong H.P and Zhou W (2003) '' Load factor calibration for the proposed 2005 edition of the National Building Code of Canada Statics of Loads and Load effects'' *Canadian Journal of civil Engineering*, 30:

BS8110-1, (1997), "The Structural Use of Concrete", British Standard Institution, London.

BS8110-3, (1997), "The Structural Use of Concrete", British Standard Institution, London.

Ditlevson O and Madsen H O (2005) ''Structural Reliabilty Methods'' Coastal, Maritime and Structural Engineering, Department of Mechanical Engineering, Technical University of Denmark.

Doty, L.A., (1985) "Reliability for the Technologies." Industrial Press New York.

Ema C, Paulo, C, and Giorgios, A. (2004) '' Assessment of The Seismic Behaviour of Rc Flat Slab Building Structures'' Paper No. 2630 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada

EN 1992-1-1 (2004) (Eurocode 2) ''Design of concrete structures-part 1-1, General rules and rules for buildings'' Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC

Eurocode 0 (2002) "Eurocode-Basis of structural design". British standards. CEN Brussels ,

Gollwitzer S, Abdo T and Rackwiz, K (1988) "*First Order Reliability Method (FORM)*." Users Manual,RCP-GM BH, Munich, West Germany.

Ibrahim A (2009), ‘’Probabilistic Assessment of Axially Loaded Short Column Subjected to Early Age Loading’’ Unpublished B.Eng Research Project, Department of Civil Engineering, Ahmadu Bello University, Zaria

Juned L S (2003) ‘’ *Risk and Hydraulic Reliabilty analysis of Water Distribution Systems*’’ MSc thesis presented to the College of Graduate Studies, King Fahd University of Petroleum and Minerals, Dhahran31261, Saudi Arabia

Lukman H S., (2012), “Iso-safety Design Charts for Singly reinforced concrete beams”, A Thesis Submitted to the Postgraduate School Ahmadu Bello University Zaria, for the Award of Master of Science Degree, Department of Civil Engineering, Ahmadu Bello University Zaria.

Madsen, H. O., Krenk, S. and Lind, N. C., (1986), “*Methods of Structural Safety*”, Prentice-Hall, Englewood Cliffs, N.J.

Melchers, R. E. (1999). Structural Reliability Analysis and Prediction. Second Edition, John Wiley and Sons Limited, Chichester, England

Muhammed J K, Adamu L, Ibrahim A, and Mohammed S M, (2014) “Safety of Early Age Loaded Reinforced Concrete Members*’’ Asian Journal of Engineering and Technology* 02(1): 2321-2462

Muhammed J. K. and Magaji S, (2010), “Reliability-Based Design Charts For Singly Reinforced RC Members Based on BS 8110 (1997) Design Code”, *African Journal of Pure and Applied Science*, ARD/2010/016.

Mosley, W.H, Bungey, J.H and Hulse, R. (1999). ‘*’Reinforced concrete design’’* (5^{th} edition). Palgrave Macmillan press Limited

Mosley, W. H., Bungey, J. H. and Hulse R., (2007), "*Reinforced Concrete Design*", 6th Edition. Palgrave Macmillan, New York

Moss R and Brooker O and Webster R (2006). "*How to design concrete structures using Eurocode 2*''. The concrete center, Riverside House, 4 meadows Bussiness park station Approach, Blackwater camberley Surrey.

Mostafa, A.H. (1992). "Optimal Design of Reinforced Concrete Frames" M Sc thesis presented to the College of Graduate Studies, King Fahd University of Petroleum and Minerals, Dhahran, Saudi Arabia.

Porco. F, Uva, G, Sangirard. M and Casolo. S. (2013) '*' About* the Reliability of Punching Verifications in Reinforced Concrete Flat Slabs*" The open construction and Building Technology Jornal*, 7: 74-87

Reynolds C E, James C T and Anthony J T (2008). '' Reinforced Concrete Designer's Handbook'' 11th Ed. Taylor and Frances Publishing Company, Abingdon, UK

Sorensen J. D. (2004), ''Note 1+2: Structural Reliability'' Institute of Building Technology and Structural Engineering Aalborg University, Sohngaardsholmsvej 57, DK-9000 Aalborg, Denmark.

Uche, O. A., (2000), "Iso-safety Design of Reinforced Concrete Columns to BS8110 (1985*)*", A Thesis Submitted to the Postgraduate School Ahmadu Bello University Zaria, for the Award of Master of Science Degree, Department of Civil Engineering, Ahmadu Bello University Zaria.

Vrijling, J. K., (2000), "Lecture Notes on Probabilistic Design", *Technische Universitiet, Delft*

Yang J. and DeWolf J. (2002), "Reliability Assessment of Highway Truss Sign Supports". *Journal of Structural Engineering*, ASCE 128(11): 1429-1438

APPENDICES

APPENDIX A: RESULTS FROM WHICH EC2 CHARTS WERE PLOTTED

Table A1: Results from which the developed EC2 charts are plotted for fyk = 410 N/mm2

Concrete Grades fyk = 410 N/mm2								
	fck = 25 N/mm2		**fck = 30 N/mm2**		**fck = 35 N/mm2**		**fck = 50 N/mm2**	
x/d	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**
0.001	0.003179	0.0113	0.003815	0.0136	0.004451	0.0159	0.006358	0.0227
0.025	0.079479	0.2807	0.095374	0.3368	0.11127	0.3929	0.158957	0.5613
0.05	0.158957	0.5557	0.190749	0.6668	0.22254	0.7779	0.317914	1.1113
0.1	0.317914	1.0886	0.381497	1.3064	0.44508	1.5241	0.635828	2.1773
0.15	0.476871	1.5989	0.572246	1.9187	0.66762	2.2385	0.953743	3.1979
0.2	0.635828	2.0866	0.762994	2.5039	0.89016	2.9212	1.271657	4.1731
0.25	0.794786	2.5515	0.953743	3.0618	1.1127	3.5721	1.589571	5.103
0.3	0.953743	2.9938	1.144491	3.5925	1.33524	4.1913	1.907485	5.9875
0.35	1.1127	3.4133	1.33524	4.096	1.55778	4.7787	2.225399	6.8267
0.4	1.271657	3.8102	1.525988	4.5723	1.78032	5.3343	2.543314	7.6205
0.45	1.430614	4.1845	1.716737	5.0214	2.00286	5.8582	2.861228	8.3689
0.5	1.589571	4.536	1.907485	5.4432	2.225399	6.3504	3.179142	9.072
0.55	1.748528	4.8649	2.098234	5.8378	2.447939	6.8108	3.497056	9.7297
0.6	1.907485	5.171	2.288982	6.2052	2.670479	7.2395	3.814971	10.342
0.617	1.961531	5.27	2.353837	6.324	2.746143	7.378	3.923061	10.54

Table A2: Results from which the developed EC2 charts are plotted for fyk = 460 N/mm2

fyk = 460 N/mm2								
	fck = 25 N/mm2		**30 N/mm2**		**35 N/mm2**		**50 N/mm2**	
x/d	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**
0.001	0.002834	0.01134	0.0034	0.0136	0.00397	0.01587	0.0056672	0.0227
0.025	0.07084	0.28067	0.085007	0.3368	0.09918	0.39293	0.1416792	0.5613
0.05	0.141679	0.55566	0.170015	0.66679	0.19835	0.77792	0.2833583	1.1113
0.1	0.283358	1.08864	0.34003	1.30637	0.3967	1.5241	0.5667166	2.1773
0.15	0.425037	1.59894	0.510045	1.91873	0.59505	2.23852	0.850075	3.1979
0.2	0.566717	2.08656	0.68006	2.50387	0.7934	2.92118	1.1334333	4.1731
0.25	0.708396	2.5515	0.850075	3.0618	0.99175	3.5721	1.4167916	5.103
0.3	0.850075	2.99376	1.02009	3.59251	1.1901	4.19126	1.7001499	5.9875
0.35	0.991754	3.41334	1.190105	4.09601	1.38846	4.77868	1.9835082	6.8267
0.4	1.133433	3.81024	1.36012	4.57229	1.58681	5.33434	2.2668666	7.6205
0.45	1.275112	4.18446	1.530135	5.02135	1.78516	5.85824	2.5502249	8.3689
0.5	1.416792	4.536	1.70015	5.4432	1.98351	6.3504	2.8335832	9.072
0.55	1.558471	4.86486	1.870165	5.83783	2.18186	6.8108	3.1169415	9.7297
0.6	1.70015	5.17104	2.04018	6.20525	2.38021	7.23946	3.4002999	10.342
0.617	1.748321	5.26997	2.097985	6.32397	2.44765	7.37796	3.4966417	10.54

Table A3: Results from which the developed EC2 charts are plotted for fyk = 500 N/mm2

	fyk = 500 N/mm2							
	fck = 25 N/mm2		**30 N/mm2**		**35 N/mm2**		**50 N/mm2**	
x/d	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**
0.001	0.002607	0.01134	0.003128	0.0136	0.00365	0.01587	0.005214	0.0227
0.025	0.065172	0.28067	0.078207	0.3368	0.091241	0.39293	0.130345	0.5613
0.05	0.130345	0.55566	0.156414	0.6668	0.182483	0.77792	0.26069	1.1113
0.1	0.26069	1.08864	0.312828	1.3064	0.364966	1.5241	0.521379	2.1773
0.15	0.391034	1.59894	0.469241	1.9187	0.547448	2.23852	0.782069	3.1979
0.2	0.521379	2.08656	0.625655	2.5039	0.729931	2.92118	1.042759	4.1731
0.25	0.651724	2.5515	0.782069	3.0618	0.912414	3.5721	1.303448	5.103
0.3	0.782069	2.99376	0.938483	3.5925	1.094897	4.19126	1.564138	5.9875
0.35	0.912414	3.41334	1.094897	4.096	1.277379	4.77868	1.824828	6.8267
0.4	1.042759	3.81024	1.25131	4.5723	1.459862	5.33434	2.085517	7.6205
0.45	1.173103	4.18446	1.407724	5.0214	1.642345	5.85824	2.346207	8.3689
0.5	1.303448	4.536	1.564138	5.4432	1.824828	6.3504	2.606897	9.072
0.55	1.433793	4.86486	1.720552	5.8378	2.00731	6.8108	2.867586	9.7297
0.6	1.564138	5.17104	1.876966	6.2052	2.189793	7.23946	3.128276	10.342
0.617	1.608455	5.26997	1.930146	6.324	2.251837	7.37796	3.21691	10.54

APPENDIX B: RESULTD FROM WHICH ISO-SAFETY CHARTS WERE PLOTTED

Table B1: Results from which iso-safety charts were plotted for fyk=410N/mm^2 and β_T=3.3

β=3.3	fyk=410N/mm2						
fck=50		35N/mm2		30N/mm2		25N/mm2	
M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd
0.016187	0.006358	0.011331	0.004451	0.009712	0.003815	0.008094	0.003179
0.40079	0.158957	0.280553	0.11127	0.240474	0.095374	0.200395	0.079479
0.793482	0.317914	0.555438	0.22254	0.476089	0.190749	0.396741	0.158957
1.554578	0.635828	1.088204	0.44508	0.932747	0.381497	0.777289	0.317914
2.283286	0.953743	1.5983	0.66762	1.369972	0.572246	1.141643	0.476871
2.979608	1.271657	2.085725	0.89016	1.787765	0.762994	1.489804	0.635828
3.643542	1.589571	2.550479	1.1127	2.186125	0.953743	1.821771	0.794786
4.275089	1.907485	2.992562	1.33524	2.565053	1.144491	2.137545	0.953743
4.874249	2.225399	3.411975	1.55778	2.92455	1.33524	2.437125	1.1127
5.441022	2.543314	3.808716	1.78032	3.264613	1.525988	2.720511	1.271657
5.975409	2.861228	4.182786	2.002859	3.585245	1.716737	2.987704	1.430614
6.477408	3.179142	4.534185	2.225399	3.886445	1.907485	3.238704	1.589571
6.94702	3.497056	4.862914	2.447939	4.168212	2.098234	3.47351	1.748528
7.384245	3.81497	5.168971	2.670479	4.430547	2.288982	3.692122	1.907485
7.525524	3.923061	5.267866	2.746143	4.515314	2.353837	3.762762	1.961531

Table B2: Results from which iso-safety charts were plotted for fyk=460N/mm^2 and β_T=3.3

β=3.3	fyk=460N/mm2						
fck=50N/mm2		35N/mm2		30N/mm2		25N/mm2	
M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd
0.016187	0.005667	0.011331	0.003967	0.009712	0.0034	0.008094	0.002834
0.40079	0.141679	0.280553	0.099175	0.240474	0.085007	0.200395	0.07084
0.793482	0.283358	0.555438	0.198351	0.476089	0.170015	0.396741	0.141679
1.554578	0.566717	1.088204	0.396702	0.932747	0.34003	0.777289	0.283358
2.283286	0.850075	1.5983	0.595052	1.369972	0.510045	1.141643	0.425037
2.979608	1.133433	2.085725	0.793403	1.787765	0.68006	1.489804	0.566717
3.643542	1.416792	2.550479	0.991754	2.186125	0.850075	1.821771	0.708396
4.275089	1.70015	2.992562	1.190105	2.565053	1.02009	2.137545	0.850075
4.874249	1.983508	3.411975	1.388456	2.92455	1.190105	2.437125	0.991754
5.441022	2.266867	3.808716	1.586807	3.264613	1.36012	2.720511	1.133433
5.975409	2.550225	4.182786	1.785157	3.585245	1.530135	2.987704	1.275112
6.477408	2.833583	4.534185	1.983508	3.886445	1.70015	3.238704	1.416792
6.94702	3.116941	4.862914	2.181859	4.168212	1.870165	3.47351	1.558471
7.384245	3.4003	5.168971	2.38021	4.430547	2.04018	3.692122	1.70015
7.525524	3.496642	5.267866	2.447649	4.515314	2.097985	3.762762	1.748321

Table B3: Results from which iso-safety charts were plotted for fyk=500N/mm^2 and β_T=3.3

β=3.3	**fyk=500N/mm2**						
fck=50N/mm2		**35N/mm2**		**30N/mm2**		**25N/mm2**	
M/bd2	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**	**100As/bd**	**M/bd2**	**100As/bd**
0.016187	0.005214	0.011331	0.00365	0.009712	0.003128	0.008094	0.002607
0.40079	0.130345	0.280553	0.091241	0.240474	0.078207	0.200395	0.065172
0.793482	0.26069	0.555438	0.182483	0.476089	0.156414	0.396741	0.130345
1.554578	0.521379	1.088204	0.364966	0.932747	0.312828	0.777289	0.26069
2.283286	0.782069	1.5983	0.547448	1.369972	0.469241	1.141643	0.391034
2.979608	1.042759	2.085725	0.729931	1.787765	0.625655	1.489804	0.521379
3.643542	1.303448	2.550479	0.912414	2.186125	0.782069	1.821771	0.651724
4.275089	1.564138	2.992562	1.094897	2.565053	0.938483	2.137545	0.782069
4.874249	1.824828	3.411975	1.277379	2.92455	1.094897	2.437125	0.912414
5.441022	2.085517	3.808716	1.459862	3.264613	1.25131	2.720511	1.042759
5.975409	2.346207	4.182786	1.642345	3.585245	1.407724	2.987704	1.173103
6.477408	2.606896	4.534185	1.824828	3.886445	1.564138	3.238704	1.303448
6.94702	2.867586	4.862914	2.00731	4.168212	1.720552	3.47351	1.433793
7.384245	3.128276	5.168971	2.189793	4.430547	1.876965	3.692122	1.564138
7.525524	3.21691	5.267866	2.251837	4.515314	1.930146	3.762762	1.608455

Table B4: Results from which iso-safety charts were plotted for fyk=410N/mm^2 and β_T=3.8

β=3.8	**fyk=410**						
fck=50N/mm2		**35N/mm2**		**30N/mm2**		**25N/mm2**	
M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd
0.014487	0.006358	0.010141	0.004451	0.008692	0.003815	0.00724336	0.003179
0.35869	0.158957	0.251083	0.11127	0.215214	0.095374	0.17934493	0.079479
0.710133	0.317914	0.497093	0.22254	0.42608	0.190749	0.35506673	0.158957
1.391282	0.635828	0.973897	0.44508	0.834769	0.381497	0.69564094	0.317914
2.043445	0.953743	1.430412	0.66762	1.226067	0.572246	1.02172263	0.476871
2.666624	1.271657	1.866637	0.89016	1.599974	0.762994	1.3333118	0.635828
3.260817	1.589571	2.282572	1.1127	1.95649	0.953743	1.63040846	0.794786
3.826025	1.907485	2.678218	1.33524	2.295615	1.144491	1.91301259	0.953743
4.362248	2.225399	3.053574	1.55778	2.617349	1.33524	2.1811242	1.1127
4.869487	2.543314	3.408641	1.78032	2.921692	1.525988	2.43474329	1.271657
5.34774	2.861228	3.743418	2.002859	3.208644	1.716737	2.67386986	1.430614
5.797008	3.179142	4.057905	2.225399	3.478205	1.907485	2.89850392	1.589571
6.217291	3.497056	4.352104	2.447939	3.730375	2.098234	3.10864545	1.748528
6.608589	3.81497	4.626012	2.670479	3.965153	2.288982	3.30429446	1.907485
6.735027	3.923061	4.714519	2.746143	4.041016	2.353837	3.36751373	1.961531

Table B5: Results from which iso-safety charts were plotted for fyk=460N/mm^2 and β_T=3.8

β=3.8	fyk=460						
fck=50N/mm2		35N/mm2		30N/mm2		25N/mm2	
M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd
0.014487	0.005667	0.010141	0.003967	0.008692	0.0034	0.007243	0.002834
0.35869	0.141679	0.251083	0.099175	0.215214	0.085007	0.179345	0.07084
0.710133	0.283358	0.497093	0.198351	0.42608	0.170015	0.355067	0.141679
1.391282	0.566717	0.973897	0.396702	0.834769	0.34003	0.695641	0.283358
2.043445	0.850075	1.430412	0.595052	1.226067	0.510045	1.021723	0.425037
2.666624	1.133433	1.866637	0.793403	1.599974	0.68006	1.333312	0.566717
3.260817	1.416792	2.282572	0.991754	1.95649	0.850075	1.630408	0.708396
3.826025	1.70015	2.678218	1.190105	2.295615	1.02009	1.913013	0.850075
4.362248	1.983508	3.053574	1.388456	2.617349	1.190105	2.181124	0.991754
4.869487	2.266867	3.408641	1.586807	2.921692	1.36012	2.434743	1.133433
5.34774	2.550225	3.743418	1.785157	3.208644	1.530135	2.67387	1.275112
5.797008	2.833583	4.057905	1.983508	3.478205	1.70015	2.898504	1.416792
6.217291	3.116941	4.352104	2.181859	3.730375	1.870165	3.108645	1.558471
6.608589	3.4003	4.626012	2.38021	3.965153	2.04018	3.304294	1.70015
6.735027	3.496642	4.714519	2.447649	4.041016	2.097985	3.367514	1.748321

Table B6: Results from which iso-safety charts were plotted for fyk=500N/mm^2 and β_T=3.8

β=3.8	fyk=500						
fck=50N/mm2		35N/mm2		30N/mm2		25N/mm2	
M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd
0.014487	0.005214	0.010141	0.00365	0.008692	0.003128	0.007243	0.002607
0.35869	0.130345	0.251083	0.091241	0.215214	0.078207	0.179345	0.065172
0.710133	0.26069	0.497093	0.182483	0.42608	0.156414	0.355067	0.130345
1.391282	0.521379	0.973897	0.364966	0.834769	0.312828	0.695641	0.26069
2.043445	0.782069	1.430412	0.547448	1.226067	0.469241	1.021723	0.391034
2.666624	1.042759	1.866637	0.729931	1.599974	0.625655	1.333312	0.521379
3.260817	1.303448	2.282572	0.912414	1.95649	0.782069	1.630408	0.651724
3.826025	1.564138	2.678218	1.094897	2.295615	0.938483	1.913013	0.782069
4.362248	1.824828	3.053574	1.277379	2.617349	1.094897	2.181124	0.912414
4.869487	2.085517	3.408641	1.459862	2.921692	1.25131	2.434743	1.042759
5.34774	2.346207	3.743418	1.642345	3.208644	1.407724	2.67387	1.173103
5.797008	2.606896	4.057905	1.824828	3.478205	1.564138	2.898504	1.303448
6.217291	2.867586	4.352104	2.00731	3.730375	1.720552	3.108645	1.433793
6.608589	3.128276	4.626012	2.189793	3.965153	1.876965	3.304294	1.564138
6.735027	3.21691	4.714519	2.251837	4.041016	1.930146	3.367514	1.608455

Table B7: Results from which iso-safety charts were plotted for fyk=410N/mm^2 and β_T=4.3

β=4.3	fyk=410						
fck=50N/mm2		35N/mm2		30N/mm2		25N/mm2	
M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd
0.012922	0.006358	0.009046	0.004451	0.007753	0.003815	0.006461	0.003179
0.319958	0.158957	0.223971	0.11127	0.191975	0.095374	0.159979	0.079479
0.633452	0.317914	0.443417	0.22254	0.380071	0.190749	0.316726	0.158957
1.24105	0.635828	0.868735	0.44508	0.74463	0.381497	0.620525	0.317914
1.822792	0.953743	1.275954	0.66762	1.093675	0.572246	0.911396	0.476871
2.378678	1.271657	1.665075	0.89016	1.427207	0.762994	1.189339	0.635828
2.90871	1.589571	2.036097	1.1127	1.745226	0.953743	1.454355	0.794786
3.412886	1.907485	2.38902	1.33524	2.047732	1.144491	1.706443	0.953743
3.891207	2.225399	2.723845	1.55778	2.334724	1.33524	1.945604	1.1127
4.343673	2.543314	3.040571	1.78032	2.606204	1.525988	2.171837	1.271657
4.770284	2.861228	3.339199	2.002859	2.862171	1.716737	2.385142	1.430614
5.17104	3.179142	3.619728	2.225399	3.102624	1.907485	2.58552	1.589571
5.54594	3.497056	3.882158	2.447939	3.327564	2.098234	2.77297	1.748528
5.894985	3.81497	4.12649	2.670479	3.536991	2.288982	2.947493	1.907485
6.007771	3.923061	4.20544	2.746143	3.604663	2.353837	3.003885	1.961531

Table B8: Results from which iso-safety charts were plotted for fyk=460N/mm^2 and β_T=4.3

β=4.3	fyk=460N/mm2						
fck=50N/mm2		35N/mm2		30N/mm2		25N/mm2	
M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd
0.012922	0.005667	0.009046	0.003967	0.007753	0.0034	0.006461	0.002834
0.319958	0.141679	0.223971	0.099175	0.191975	0.085007	0.159979	0.07084
0.633452	0.283358	0.443417	0.198351	0.380071	0.170015	0.316726	0.141679
1.24105	0.566717	0.868735	0.396702	0.74463	0.34003	0.620525	0.283358
1.822792	0.850075	1.275954	0.595052	1.093675	0.510045	0.911396	0.425037
2.378678	1.133433	1.665075	0.793403	1.427207	0.68006	1.189339	0.566717
2.90871	1.416792	2.036097	0.991754	1.745226	0.850075	1.454355	0.708396
3.412886	1.70015	2.38902	1.190105	2.047732	1.02009	1.706443	0.850075
3.891207	1.983508	2.723845	1.388456	2.334724	1.190105	1.945604	0.991754
4.343673	2.266867	3.040571	1.586807	2.606204	1.36012	2.171837	1.133433
4.770284	2.550225	3.339199	1.785157	2.862171	1.530135	2.385142	1.275112
5.17104	2.833583	3.619728	1.983508	3.102624	1.70015	2.58552	1.416792
5.54594	3.116941	3.882158	2.181859	3.327564	1.870165	2.77297	1.558471
5.894985	3.4003	4.12649	2.38021	3.536991	2.04018	2.947493	1.70015
6.007771	3.496642	4.20544	2.447649	3.604663	2.097985	3.003885	1.748321

Table B9: Results from which iso-safety charts were plotted for fyk=500N/mm^2 and β_T=4.3

β=4.3	fyk=500						
fck=50N/mm2		35N/mm2		30N/mm2		25N/mm2	
M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd	M/bd2	100As/bd
0.012922	0.005214	0.009046	0.00365	0.007753	0.003128	0.006461	0.002607
0.319958	0.130345	0.223971	0.091241	0.191975	0.078207	0.159979	0.065172
0.633452	0.26069	0.443417	0.182483	0.380071	0.156414	0.316726	0.130345
1.24105	0.521379	0.868735	0.364966	0.74463	0.312828	0.620525	0.26069
1.822792	0.782069	1.275954	0.547448	1.093675	0.469241	0.911396	0.391034
2.378678	1.042759	1.665075	0.729931	1.427207	0.625655	1.189339	0.521379
2.90871	1.303448	2.036097	0.912414	1.745226	0.782069	1.454355	0.651724
3.412886	1.564138	2.38902	1.094897	2.047732	0.938483	1.706443	0.782069
3.891207	1.824828	2.723845	1.277379	2.334724	1.094897	1.945604	0.912414
4.343673	2.085517	3.040571	1.459862	2.606204	1.25131	2.171837	1.042759
4.770284	2.346207	3.339199	1.642345	2.862171	1.407724	2.385142	1.173103
5.17104	2.606896	3.619728	1.824828	3.102624	1.564138	2.58552	1.303448
5.54594	2.867586	3.882158	2.00731	3.327564	1.720552	2.77297	1.433793
5.894985	3.128276	4.12649	2.189793	3.536991	1.876965	2.947493	1.564138
6.007771	3.21691	4.20544	2.251837	3.604663	1.930146	3.003885	1.608455

APPENDIX C: RESULTS OF RELIABILITY ANALYSIS

Table C1: Load ratio versus safety index at the Column face, first critical section from the column face and critical section from drop panel

load ratio	B_1	B_2	B_3
0.1	3.75	8.75	5.15
0.2	3.37	8.41	4.78
0.3	3.02	8.11	4.44
0.4	2.7	7.83	4.13
0.5	2.4	7.57	3.84
0.6	2.12	7.33	3.57
0.7	1.86	7.11	3.32
0.8	1.62	6.89	3.08
0.9	1.39	6.69	2.86
1	1.18	6.51	2.65
1.1	0.971	6.33	2.45
1.2	0.777	6.16	2.26
1.3	0.592	5.99	2.08
1.4	0.415	5.84	1.9

Table C2: Concrete Grade versus safety index at the first critical section from the column face, critical section from drop panel and at the column face.

Fck(N/mm2)	B_1	B_2	B_3
25	1.73	3.18	6.99
30	2.09	3.54	7.59
35	2.4	3.84	8.09
40	2.67	4.1	8.53
45	2.9	4.33	8.91
50	3.12	4.54	9.25

Table C3: Reinforcement ratio versus safety index at the first critical section from the column face, with and without a panel drop.

Reinforcement ratio	**B_1**	**B_2**
0.1	1.73	-0.159
0.16	1.73	-0.159
0.2	1.73	-0.159
0.22	1.73	-0.159
0.25	1.73	-0.159
0.3	1.78	-0.159
0.4	2.17	0.0361
0.5	2.48	0.34
0.6	2.73	0.588
0.7	2.94	0.798
0.8	3.13	0.981
0.9	3.29	1.14
1	3.44	1.29
1.1	3.57	1.42
1.2	3.69	1.52
1.3	3.8	1.65
1.4	3.9	1.75
1.5	4	1.84
2	4.4	2.24

Table C4: Safety Index versus Effective Depth for Different Target flexural reinforcement at the First Critical Section

Effective depth(mm)	β=EC2	βT=3.3	βT=3.8	βt=4.3
200	-0.266	-0.266	-0.266	-0.266
205	-0.16	-0.16	-0.16	-0.16
250	0.74	0.742	0.742	0.742
300	1.64	1.64	1.64	1.68
305	1.73	1.73	1.73	1.78
350	2.46	2.46	2.46	2.59
400	3.22	3.22	3.22	3.43
450	3.92	3.92	3.96	4.21
500	4.59	4.59	4.69	4.94
550	5.23	5.23	5.38	5.63
600	5.84	5.87	6.04	6.3
650	6.43	6.5	6.68	6.93
700	7	7.12	7.3	7.55
750	7.56	7.72	7.9	8.1

Table C5: Safety index versus Column head diameter at the first critical section and column face.

column head diameter(mm)	B_1	B_2
200	-0.703	0.624
400	-0.132	3.07
600	0.386	4.5
800	0.863	5.53
1000	1.36	6.33
1200	1.73	6.99
1400	2.12	7.55
1600	2.51	8.05
1800	2.87	8.49
2000	3.23	8.9

Table C6: safety index versus effective depth at the Column face

effective depth(mm)	beta
200	5.51
250	6.29
300	6.93
350	7.47
400	7.93
450	8.34
500	8.71
550	9.04
600	9.35
650	9.62
700	9.88

Table C7: Safety index versus reinforcement ratio at the critical section from drop panel

Reinforcement ratio	Safety index
0.1	3.18
0.2	3.18
0.3	3.18
0.4	3.47
0.5	3.77
0.6	4.01
0.7	4.22
0.8	4.4
0.9	4.56
1	4.7
1.1	4.82
1.2	4.92
1.3	5.05
1.4	5.15
1.5	5.24
1.6	5.33
1.7	5.41
1.8	5.49
1.9	5.56
2	5.63

Table C8: safety index versus effective depth at critical section from drop panel (when slab is designed using iso-safety charts targeting different safety index values)

Effective depth (mm)	βT=3.3	βT=3.8	βT=4.3
200	3.09	3.09	3.18
205	3.18	3.18	3.29
250	3.94	4	4.17
300	4.73	4.87	5.03
350	5.51	5.65	5.81
400	6.23	6.36	6.53
450	6.9	7.03	7.2
500	7.52	7.66	7.82
550	8.12	8.26	8.42
600	8.69	8.83	8.99
650	9.23	9.37	9.53
700	9.76	9.9	10.1

Table C9: Safety Index versus drop dimension at the critical section from the drop panel

Drop dimention(mm)	beta
1200	0.43
1400	0.915
1600	1.37
1800	1.8
2000	2.21
2200	2.61
2400	2.99
2600	3.37
2800	3.74
3000	4.1

Table C10: Safety index versus reinforcement ratio for the deflection of flat slab

Reinforcement ratio	beta
0.1	0.694
0.2	3.042
0.3	4.45
0.4	5.465
0.5	6.262
0.6	6.921
0.7	7.482
0.8	7.971
0.9	8.406
1	8.797
1.1	9.152
1.2	9.447
1.3	9.778
1.4	10.057
1.5	10.318
1.6	10.562
1.7	10.793
1.8	11.01
1.9	11.217
2	11.413

Table C11: Safety index versus concrete grade for the deflection of flat slab

Fck(N/mm2)	beta
25	3
30	3.67
35	4.25
40	4.76
45	5.22
50	5.62

Table C12: Safety index versus effective depth for the deflection of flat slab

effective depth(mm)	beta
200	2.91
205	3
250	3.68
300	4.31
350	4.85
400	5.32
450	5.74
500	6.12
550	6.42

Table C13: Safety index versus slab length for the deflection of flat slab

slab length(mm)	beta
4000	4.68
4500	4.27
5000	3.9
5500	3.57
6000	3.27
6500	3
7000	2.75
7500	2.51
8000	2.3
8500	2.09
9000	1.71
9500	1.35
10000	1.01
10500	0.688
11000	0.383
11500	0.0929
12000	-0.187

APPENDIX D: PROGRAM LISTING

APPENDIX D1 PROGRAM DESIGN CHAT

```
C     RELIABILITY ANALYSIS OF A REINFORCED CONCRETE
C     X(1)=FCK
C     X(2)=FY
C     X(3)=X/D
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      EXTERNAL bababa3
      DIMENSION  X(3),EX(3),SX(3),VP(10,3),COV(3,3),ZES(3),
     +           UU(3),EIVEC(3,3),IV(2,3)
      CHARACTER*10 PRT
      COMMON/bababa3 /rho
      WRITE(*,*)'INPUT FCK FY X/D  '
      READ(*,*)EX(1),EX(2),EX(3)
      SX(1)=0.17*EX(1)
      SX(2)=0.15*EX(2)
      SX(3)=0.01*EX(3)
     rho= 0.68*EX(1)*EX(3)/EX(2)
      DATA N/3/,NC/3/,NE/3/,IRHO/0/
      NAUS=7
      ICRT=0
      OPEN(7,FILE='try3.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(7,FILE='try3.RES',STATUS='NEW')
20    CALL YINIT (N,IV,VP,IRHO,COV,NC)
      IV(1,1)=3
      IV(1,2)=3
      IV(1,3)=3
      DO 100 I=1,N
100      X(I) = EX(I)
      CONTINUE
         V1=1.0D0
      BETA=1.D0
      WRITE (NAUS,5000)
5000  FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
     +   20X,'SAFETY LEVEL)
      CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
      WRITE (ICRT,*) ' START OF FORM5'
      WRITE (ICRT,*) ' STOCHASTIC MODEL :'
      CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
      PRT='  COV  '
      CALL YMAUS (NAUS,NC,N,COV,PRT)
      CALL FORM5 (N, IV, EX, SX, VP, bababa3, IRHO, COV, NC,
     +            EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
      PRT='  UU  '
      CALL YFAUS (NAUS,N,UU,PRT)
      PRT='  ZES  '
      CALL YFAUS (NAUS,3,ZES,PRT)
        WRITE(NAUS,504)
       write(NAUS,505)(uu(i)/beta,i=1,n)
```

```
504     FORMAT(/,3X,'ALPHA VECTOR:')
505     format(3x,/3X,6(2X,E8.2)/)
      WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
      WRITE (ICRT,*) ' RESULTS SEE FILE try.RES'
      STOP
      END

      SUBROUTINE bababa3 (N, X, FX, IER)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION X(N)
      COMMON /bababa3/rho
C          G(X) = R-S
        R = X(2)*rho*(1-0.4*X(3))
         S = 0.4536*X(1)*X(3)*(1-0.4*X(3))
C     CHECK FOR ERRORS, CALCULATE FX
      IF (R.NE.0..AND.S.NE.0)THEN
           FX = R-S
        IER = 0
      ELSE
        FX = 1.D+20
        IER = 1
      ENDIF
      RETURN
      END
```

APPENDIX D2 PROGRAM ISSOSAFETY CHARTS

```
C       RELIABILITY BASED Issosafety charts
        IMPLICIT DOUBLE PRECISION (A-H,O-Z)
       DIMENSION  EX(3)
        COMMON /bababa3/RHO,ALP
      OPEN(11,FILE='tryu.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(11,FILE='tryu.RES',STATUS='NEW')
20     OPEN(12,FILE='try4.RES',STATUS='OLD',ERR=13)
      GOTO 21
13    OPEN(12,FILE='try4.RES',STATUS='NEW')
21     WRITE(*,*)'X/D  '
       READ(*,*)XXD
      WRITE(11,*)'MOMENT STRESS'
      WRITE(12,*)'REINFOCEMENT RATIO'
      ALP=1
      XFCK=25
      XFY=500
      BETAT=3.8
15    RHO1=0.4536*XFCK*XXD/(0.87*XFY)
      XM1=0.4536*XFCK*XXD*(1-0.4*XXD)
      GOTO 2222
25    ALP=0.640-0.001
2222  XM=ALP*XM1
      EX(1) = XFCK
```

```
      EX(2) = XFY
      EX(3) = XXD
      CALL BABABA2 (EX,RHO,BETA,XEETA)
      BETA=XEETA
      EPSILON=(BETAT-BETA)**2
      IF (EPSILON.LE.0.0001) GOTO 24
      GOTO 25
24    WRITE(*,*)BETA,XM,RHO1
      WRITE(11,33)XM
      RHO2=100*RHO1
      WRITE(12,33)RHO2
33    FORMAT(2X,''F10.6)
      WRITE(*,*)'input new X/D  '
        READ(*,*)XXD
      IF(XXD.EQ.0.0)GOTO 26
c     GOTO 26
c25   WRITE(*,*)'INPUT ALPHA '
c     READ(*,*)ALP
      RHO1=0.4536*XFCK*XXD/(0.87*XFY)
      XM1=0.4536*XFCK*XXD*(1-0.4*XXD)
      GOTO 25
26    STOP
      END


      SUBROUTINE BABABA2 (EX,RHO,BETA,XEETA)
C     RELIABILITY ANALYSIS OF A REINFORCED CONCRETE
C     X(1)=FCK
C     X(2)=FY
C     X(3)=X/D
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      EXTERNAL bababa3
      DIMENSION  X(3),EX(3),SX(3),VP(10,3),COV(3,3),ZES(3),
     +           UU(3),EIVEC(3,3),IV(2,3)
      CHARACTER*10 PRT
      SX(1)=0.17*EX(1)
      SX(2)=0.15*EX(2)
      SX(3)=0.01*EX(3)
      RHO= 0.68*EX(1)*EX(3)/EX(2)
      DATA N/3/,NC/3/,NE/3/,IRHO/0/
      NAUS=7
      ICRT=0
      OPEN(7,FILE='try3.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(7,FILE='try3.RES',STATUS='NEW')
20    CALL YINIT (N,IV,VP,IRHO,COV,NC)
      IV(1,1)=3
      IV(1,2)=3
      IV(1,3)=3
      DO 100 I=1,N
100      X(I) = EX(I)
      CONTINUE
```

```
         V1=0.5D0
       BETA=1.D0
       WRITE (NAUS,5000)
5000   FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
      +   20X,'SAFETY LEVEL ')
       CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
       WRITE (ICRT,*) ' START OF FORM5'
       WRITE (ICRT,*) ' STOCHASTIC MODEL :'
       CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
       PRT='  COV  '
       CALL YMAUS (NAUS,NC,N,COV,PRT)
       CALL FORM5 (N, IV, EX, SX, VP, bababa3, IRHO, COV, NC,
      +            EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
       XEETA=BETA
       PRT='  UU  '
       CALL YFAUS (NAUS,N,UU,PRT)
       PRT='  ZES  '
       CALL YFAUS (NAUS,3,ZES,PRT)
         WRITE(NAUS,504)
        write(NAUS,505)(uu(i)/beta,i=1,n)
504      FORMAT(/,3X,'ALPHA VECTOR:')
505      format(3x,/3X,6(2X,E8.2)/)
       WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
       WRITE (ICRT,*) ' RESULTS SEE FILE ERIC2.RES'
       RETURN
       END

       SUBROUTINE bababa3 (N, X, FX, IER)
       IMPLICIT DOUBLE PRECISION (A-H,O-Z)
       DIMENSION X(N)
       COMMON /bababa3/RHO,ALP
C           G(X) = R-S
         R = X(2)*RHO*(1-0.4*X(3))
          S = 0.4536*ALP*X(1)*X(3)*(1-0.4*X(3))
C      CHECK FOR ERRORS, CALCULATE FX
       IF (R.NE.0..AND.S.NE.0)THEN
            FX = R-S
         IER = 0
       ELSE
         FX = 1.D+20
         IER = 1
       ENDIF
       RETURN
       END
```

APPENDIX D3 PROGRAM FOR PUNCHING SHEAR AT COLUMN FACE

```
    PROGRAM  ASK
   C     RELIABILITY ANALYSIS OF A REINFORCED CONCRETE
   C  X(1)=CONCRETE GRADE
   C  X(2)=DEAD LOAD
```

```
C  X(3)=DIAMETER OF COLUMN HEAD
C     X(4)= LOAD RATIO
C     X(5)=EFFEECTIVE DEPTH AT COLUMN HEAD
c  X(6)=RESISTANCE MODEL UNCERTAINTY
C  X(7)=LOAD MODEL UNCERTAINTY
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      EXTERNAL SHEAR
      DIMENSION  X(6),EX(6),SX(6),VP(10,6),COV(6,6),ZES(3),
     +           UU(6),EIVEC(6,6),IV(2,6)
      CHARACTER*10 PRT
      COMMON/SHEAR /XALPHA,XAA
   DATA N/6/,NC/6/,NE/6/,IRHO/0/
   WRITE(*,*)'ENTER COLUMN DIAMETER.......'
      READ(*,*)XDD
   XAA=0.7554
      EX(1)=25
   EX(2)=279700
   EX(3)=XDD
   EX(4)=305
   EX(5)=1.1
   EX(6)=1.0
   SX(1)=EX(1)*0.17
   SX(2)=EX(2)*0.10
   SX(3)=EX(3)*0.06
   SX(4)=EX(4)*0.025
   SX(5)=EX(5)*0.07
   SX(6)=EX(6)*0.2
      NAUS=7
      ICRT=0
      OPEN(7,FILE='ask.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(7,FILE='ask.RES',STATUS='NEW')
20    CALL YINIT (N,IV,VP,IRHO,COV,NC)
      IV(1,1)=3
      IV(1,5)=3
   IV(1,6)=3
      DO 100 I=1,N
100      X(I) = EX(I)
      CONTINUE
         V1=1.D0
      BETA=1.D0
      WRITE (NAUS,5000)
5000  FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
     +   20X,'SAFETY LEVEL SHEAR CHECK ON MERCY2')
      CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
      WRITE (ICRT,*) ' START OF FORM5'
      WRITE (ICRT,*) ' STOCHASTIC MODEL :'
      CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
      PRT='  COV  '
      CALL YMAUS (NAUS,NC,N,COV,PRT)
      CALL FORM5 (N, IV, EX, SX, VP, SHEAR, IRHO, COV, NC,
     +            EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
```

```
      PRT='  UU  '
      CALL YFAUS (NAUS,N,UU,PRT)
      PRT='  ZES  '
      CALL YFAUS (NAUS,3,ZES,PRT)
        WRITE(NAUS,504)
       write(NAUS,505)(uu(i)/beta,i=1,n)
504     FORMAT(/,3X,'ALPHA VECTOR:')
505     format(3x,/3X,6(2X,E8.2)/)
      WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
      WRITE (ICRT,*) ' RESULTS SEE FILE ERIC2.RES'
      STOP
      END

      SUBROUTINE SHEAR (N, X, FX, IER)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION X(N)
      COMMON /SHEAR/XALPHA,XAA
C       G(X) = R-S
   XD=X(2)*(1.35+1.5*XAA)
        R = 0.5*3.142*X(5)*X(3)*X(4)*(0.6*(1-X(1)/250))*(X(1)/1.5)
         S = 1.15*X(6)*(XD-3.142*X(3)*X(3)*XD/(4*6500*6500))
C  CHECK FOR ERRORS, CALCULATE FX
      IF (R.NE.0..AND.S.NE.0)THEN
           FX = R-S
        IER = 0
      ELSE
        FX = 1.D+20
        IER = 1
      ENDIF
      RETURN
      END
```

APPENDIX D4 PROGRAM FOR PUNCHING SHEAR AT FIRST CRITICAL SECTION

```
   PROGRAM  ASKA
C     RELIABILITY ANALYSIS OF A REINFORCED CONCRETE
C     X(1)=CONCRETE GRADE
C     X(2)=DEAD LOAD
C     X(3)=EFFECTIVE DEPTH
C     X(4)=REINFORCEMENT RATIO
C     X(5)=DIAMETER OF COLUMN HEAD
C     X(6)=RESISTANCE MODEL UNCERTAINTY
C     X(7)=LOAD MODEL UNCERTAINTY
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      EXTERNAL SHEAR
      DIMENSION  X(7),EX(7),SX(7),VP(10,7),COV(7,7),ZES(3),
     +           UU(7),EIVEC(7,7),IV(2,7)
      CHARACTER*10 PRT
      COMMON/SHEAR /XALPHA,XK1
     DATA N/7/,NC/7/,NE/7/,IRHO/0/
```

```
      WRITE(*,*)'ENTER EFFECTIVE DEPTH'
      READ(*,*)XD
      XALPHA=0.7554
C     WRITE(*,*)'INPUT REINFORCEMENT RATIO....  '
C     READ(*,*)XRR
       EX(1)=25
      EX(2)=279700
      EX(3)=XD
      EX(4)=0.16
      EX(5)=1200
      EX(6)=1.1
      EX(7)=1.0
      SX(1)=EX(1)*0.17
      SX(2)=EX(2)*0.1
      SX(3)=EX(3)*0.025
      SX(4)=EX(4)*0.05
      SX(5)=EX(5)*0.06
      SX(6)=EX(6)*0.07
      SX(7)=EX(7)*0.20
      a=200
      XC=(a/305)
      XK1=1+(XC**(0.5))
C     XV1=0.12*XK1*(EX(4)*EX(1))**(0.3333333333)
C     XV2=0.035*(XK1**(1.5))*(EX(1)**(0.5))
       NAUS=7
       ICRT=0
       OPEN(7,FILE='aska.RES',STATUS='OLD',ERR=10)
       GOTO 20
10     OPEN(7,FILE='aska.RES',STATUS='NEW')
20     CALL YINIT (N,IV,VP,IRHO,COV,NC)
       IV(1,1)=3
       IV(1,6)=3
      IV(1,7)=3
       DO 100 I=1,N
100       X(I) = EX(I)
       CONTINUE
          V1=1.D0
       BETA=1.D0
       WRITE (NAUS,5000)
5000   FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
     +   20X,'SAFETY LEVEL SHEAR CHECK ON MERCY2')
       CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
       WRITE (ICRT,*) ' START OF FORM5'
       WRITE (ICRT,*) ' STOCHASTIC MODEL :'
       CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
       PRT='  COV  '
       CALL YMAUS (NAUS,NC,N,COV,PRT)
       CALL FORM5 (N, IV, EX, SX, VP, SHEAR, IRHO, COV, NC,
     +            EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
       PRT='  UU  '
       CALL YFAUS (NAUS,N,UU,PRT)
       PRT='  ZES  '
```

```
      CALL YFAUS (NAUS,3,ZES,PRT)
        WRITE(NAUS,504)
       write(NAUS,505)(uu(i)/beta,i=1,n)
504     FORMAT(/,3X,'ALPHA VECTOR:')
505     format(3x,/3X,6(2X,E8.2)/)
      WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
      WRITE (ICRT,*) ' RESULTS SEE FILE ERIC2.RES'
C     write(*,*)xc,xk1,xv1,xv2
      STOP
      END

      SUBROUTINE SHEAR (N, X, FX, IER)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION X(N)
      COMMON /SHEAR/XALPHA,XK1
C          G(X) = R-S
     a=200
     XC=(a/X(3))
     XK1=1+(XC**(0.5))
     XV1=0.12*XK1*(X(4)*X(1))**(0.33333333333333333333)
C    XV2=0.035*(XK1**(1.5))*(X(1)**(0.5))
C    IF(XV1.GE.XV2) GOTO 333
C    GOTO 334
333  XV=XV1
C    GOTO 335
C334 XV=XV2
335  XD=X(2)*(1.35+1.5*XALPHA)
      R = X(6)*XV*3.142*(X(5)+4*X(3))*X(3)

    S = 1.15*X(7)*(XD - 3.142*((X(5)+4*X(3))**2)*XD/(4*6500*6500))
C    CHECK FOR ERRORS, CALCULATE FX
      IF (R.NE.0..AND.S.NE.0)THEN
           FX = R-S
        IER = 0
      ELSE
        FX = 1.D+20
        IER = 1
      ENDIF
      RETURN
      END
```

APPENDIX D5 PROGRAM FOR PUNCHING SHEAR AT CRITICAL SECTION FROM DROP PANEL

```
       PROGRAM  ASKA2
C     RELIABILITY ANALYSIS OF A REINFORCED CONCRETE
C     X(1)=CONCRETE GRADE
C     X(2)=DEAD LOAD
C     X(3)=EFFECTIVE DEPTH
C     X(4)=REINFORCEMENT RATIO
C     X(5)=DImenstion of drop
```

```
C     X(6)=RESISTANCE MODEL UNCERTAINTY
C     X(7)=LOAD MODEL UNCERTAINTY
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      EXTERNAL SHEAR
      DIMENSION  X(7),EX(7),SX(7),VP(10,7),COV(7,7),ZES(3),
     +           UU(7),EIVEC(7,7),IV(2,7)
      CHARACTER*10 PRT
      COMMON/SHEAR /XALPHA,XAL
      DATA N/7/,NC/7/,NE/7/,IRHO/0/
      WRITE(*,*)'ENTER LOAD RATIO.......'
      READ(*,*)XALPH
      XAL=XALPH
      EX(1)=25
      EX(2)=279700
      EX(3)=205
      EX(4)=0.20
      EX(5)=2500
      EX(6)=1.1
      EX(7)=1.0
      SX(1)=EX(1)*0.17
      SX(2)=EX(2)*0.1
      SX(3)=EX(3)*0.025
      SX(4)=EX(4)*0.05
      SX(5)=EX(5)*0.06
      SX(6)=EX(6)*0.07
      SX(7)=EX(7)*0.2
      NAUS=7
      ICRT=0
      OPEN(7,FILE='aska2.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(7,FILE='aska2.RES',STATUS='NEW')
20    CALL YINIT (N,IV,VP,IRHO,COV,NC)
      IV(1,1)=3
      IV(1,6)=3
      IV(1,7)=3
      DO 100 I=1,N
100      X(I) = EX(I)
      CONTINUE
         V1=1.D0
      BETA=1.D0
      WRITE (NAUS,5000)
5000  FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
     +   20X,'SAFETY LEVEL SHEAR CHECK ON MERCY2')
      CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
      WRITE (ICRT,*) ' START OF FORM5'
      WRITE (ICRT,*) ' STOCHASTIC MODEL :'
      CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
      PRT='  COV  '
      CALL YMAUS (NAUS,NC,N,COV,PRT)
      CALL FORM5 (N, IV, EX, SX, VP, SHEAR, IRHO, COV, NC,
     +            EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
      PRT='  UU  '
```

```
      CALL YFAUS (NAUS,N,UU,PRT)
      PRT='  ZES  '
      CALL YFAUS (NAUS,3,ZES,PRT)
        WRITE(NAUS,504)
       write(NAUS,505)(uu(i)/beta,i=1,n)
504     FORMAT(/,3X,'ALPHA VECTOR:')
505     format(3x,/3X,6(2X,E8.2)/)
      WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
      WRITE (ICRT,*) ' RESULTS SEE FILE ERIC2.RES'
     STOP
      END

      SUBROUTINE SHEAR (N, X, FX, IER)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION X(N)
      COMMON /SHEAR/XALPHA,XAL
C          G(X) = R-S
     a=200
     XC=(a/X(3))
     XK1=1+(XC**(0.5))
     XV1=0.12*XK1*(X(4)*X(1))**(0.333333333333333333333)
     XV2=0.035*(XK1**(1.5))*(X(1)**(0.5))
     IF(XV1.GE.XV2) GOTO 333
     GOTO 334
333  XV=XV1
     GOTO 335
334  XV=XV2
335  XD=X(2)*(1.35+1.5*XAL)
     XA=((X(5)+3*X(3))**(2))-(4-3.142)*(2*X(3))**(2)
      R = XV*X(6)*(4*X(5)+2*3.142*2*X(3))*X(3)
         S = 1.15*X(7)*(XD-XA*XD/(6500*6500))
C     CHECK FOR ERRORS, CALCULATE FX
      IF (R.NE.0..AND.S.NE.0)THEN
           FX = R-S
        IER = 0
      ELSE
        FX = 1.D+20
        IER = 1
      ENDIF
      RETURN
      END
```

APPENDIX D6 PROGRAM FOR DEFLECTION WHEN L ≤ 8.5m

```
 PROGRAM  ASKAN
C     RELIABILITY ANALYSIS OF A REINFORCED CONCRETE
C     X(1)=CONCRETE GRADE
C     X(2)=REINFORCENMENT RATIO
C     X(3)=LENGTH
C      X(4)=EFFECTIVE DEPTH
C     X(5)= RESISTANCE MODEL UNCERTAINTY
```

```
C     X(6)= LOAD MODEL UNCERTAINTY
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      EXTERNAL DEFLECTION
      DIMENSION  X(6),EX(6),SX(6),VP(10,6),COV(6,6),ZES(3),
     +           UU(6),EIVEC(6,6),IV(2,6)
      CHARACTER*10 PRT
      COMMON/DEFLECTION/XRR
      DATA N/6/,NC/6/,NE/6/,IRHO/0/
      WRITE(*,*)'INPUT SLAB LENGTH IN MM      '
      READ(*,*)XR
      EX(1)=25
      EX(2)=0.2/100
      EX(3)=XR
      EX(4)=205
      EX(5)=1.1
      EX(6)=1.0
      SX(1)=EX(1)*0.17
      SX(2)=EX(2)*0.05
      SX(3)=EX(3)*0.06
      SX(4)=EX(4)*0.025
      SX(5)=EX(5)*0.07
      SX(6)=EX(6)*0.2
      XRR=SQRT(EX(1))
      WRITE(*,*)XRR,EX(2)
      NAUS=7
      ICRT=0
      OPEN(7,FILE='askan.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(7,FILE='askan.RES',STATUS='NEW')
20    CALL YINIT (N,IV,VP,IRHO,COV,NC)
      IV(1,1)=3
      IV(1,5)=3
      IV(1,6)=3
      DO 100 I=1,N
100      X(I) = EX(I)
      CONTINUE
         V1=1.D0
      BETA=1.D0
      WRITE (NAUS,5000)
5000  FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
     +   20X,'SAFETY LEVEL SHEAR CHECK ON MERCY2')
      CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
      WRITE (ICRT,*) ' START OF FORM5'
      WRITE (ICRT,*) ' STOCHASTIC MODEL :'
      CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
      PRT='  COV  '
      CALL YMAUS (NAUS,NC,N,COV,PRT)
      CALL FORM5 (N, IV, EX, SX, VP, DEFLECTION, IRHO, COV, NC,
     +            EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
      PRT='  UU  '
      CALL YFAUS (NAUS,N,UU,PRT)
      PRT='  ZES  '
```

```
      CALL YFAUS (NAUS,3,ZES,PRT)
        WRITE(NAUS,504)
       write(NAUS,505)(uu(i)/beta,i=1,n)
504     FORMAT(/,3X,'ALPHA VECTOR:')
505     format(3x,/3X,6(2X,E8.2)/)
      WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
      WRITE (ICRT,*) ' RESULTS SEE FILE ERIC2.RES'
     STOP
      END

      SUBROUTINE DEFLECTION (N, X, FX, IER)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION X(N)
      COMMON /DEFLECTION/XRR
C          G(X) = R-S
     XRR=(X(1)**0.5)/1000
     IF(XRR.GE.X(2)) GOTO 333
     GOTO 334
333  XR1=3.2*(X(1)**0.5)*(XRR/X(2)-1)**1.5
     R = 1.2*X(5)*(11+1.5*(X(1)**0.5)*(XRR/X(2))+XR1)
     GOTO 337
334  R=1.2*X(5)*(11+1.5*(X(1)**0.5)*(XRR/X(2)))
337    S = X(6)*X(3)/X(4)
C    WRITE(*,*)S,R,XRR,X(2)
C    CHECK FOR ERRORS, CALCULATE FX
      IF (R.NE.0..AND.S.NE.0)THEN
           FX = R-S
        IER = 0
      ELSE
        FX = 1.D+20
        IER = 1
      ENDIF
      RETURN
      END
```

APPENDIX D7 PROGRAM FOR DEFLECTION WHEN L > 8.5m

```
       PROGRAM  ASKAN1
C     RELIABILITY ANALYSIS OF A REINFORCED CONCRETE
C     DECK SLAB SUPPORTED ON STEEL GIRDERSC X(1)=CONCRETE GRADE
C     X(2)=REINFORCENMENT RATIO
C     X(3)=LENGTH
C     X(4)=EFFECTIVE DEPTH
C     X(5)= RESISTANCE MODEL UNCERTAINTY
C     X(6)= LOAD MODEL UNCERTAINTY
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      EXTERNAL DEFLECTION
      DIMENSION  X(6),EX(6),SX(6),VP(10,6),COV(6,6),ZES(3),
     +           UU(6),EIVEC(6,6),IV(2,6)
      CHARACTER*10 PRT
      COMMON/DEFLECTION/XRR,XR
```

```
      DATA N/6/,NC/6/,NE/6/,IRHO/0/
      WRITE(*,*)'INPUT SLAB LENGTH IN MM      '
      READ(*,*)XR
       EX(1)=25
      EX(2)=0.2/100
      EX(3)=XR
      EX(4)=205
      EX(5)=1.1
      EX(6)=1.0
      SX(1)=EX(1)*0.17
      SX(2)=EX(2)*0.05
      SX(3)=EX(3)*0.06
      SX(4)=EX(4)*0.025
      SX(5)=EX(5)*0.07
      SX(6)=EX(6)*0.2
      XRR=SQRT(EX(1))
      WRITE(*,*)XRR,EX(2)
       NAUS=7
       ICRT=0
       OPEN(7,FILE='askan1.RES',STATUS='OLD',ERR=10)
       GOTO 20
10     OPEN(7,FILE='askan1.RES',STATUS='NEW')
20     CALL YINIT (N,IV,VP,IRHO,COV,NC)
       IV(1,1)=3
      IV(1,5)=3
      IV(1,6)=3
       DO 100 I=1,N
100       X(I) = EX(I)
       CONTINUE
          V1=1.D0
       BETA=1.D0
       WRITE (NAUS,5000)
5000   FORMAT (////,5X,70('*'),/,30X,'F O R M 5',/,5X,70('*'),/,
     +   20X,'SAFETY LEVEL SHEAR CHECK ON MERCY2')
       CALL YKOPF (NAUS, N, IV, EX, SX, VP, IRHO)
       WRITE (ICRT,*) ' START OF FORM5'
       WRITE (ICRT,*) ' STOCHASTIC MODEL :'
       CALL YKOPF (ICRT, N, IV, EX, SX, VP, IRHO)
       PRT='  COV  '
       CALL YMAUS (NAUS,NC,N,COV,PRT)
       CALL FORM5 (N, IV, EX, SX, VP, DEFLECTION, IRHO, COV, NC,
     +            EIVEC, NE, V1, NAUS, BETA, X, UU, ZES, IER)
       PRT='  UU  '
       CALL YFAUS (NAUS,N,UU,PRT)
       PRT='  ZES  '
       CALL YFAUS (NAUS,3,ZES,PRT)
         WRITE(NAUS,504)
        write(NAUS,505)(uu(i)/beta,i=1,n)
504     FORMAT(/,3X,'ALPHA VECTOR:')
505     format(3x,/3X,6(2X,E8.2)/)
       WRITE (ICRT,*) ' END OF FORM5 : IER =',IER
       WRITE (ICRT,*) ' RESULTS SEE FILE ERIC2.RES'
```

```
      STOP
      END

      SUBROUTINE DEFLECTION (N, X, FX, IER)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION X(N)
      COMMON /DEFLECTION/XRR,XR
C          G(X) = R-S
      XRR=(X(1)**0.5)/1000
      IF(XRR.GE.X(2)) GOTO 333
      GOTO 334
333   XR1=3.2*(X(1)**0.5)*(XRR/X(2)-1)**1.5
      R = 1.2*X(5)*(8500/XR)*(11+1.5*(X(1)**0.5)*(XRR/X(2))+XR1)
      GOTO 337
334   R=1.2*X(5)*(8500/XR)*(11+1.5*(X(1)**0.5)*(XRR/X(2)))
337     S = X(6)*X(3)/X(4)
C     WRITE(*,*)S,R,XRR,X(2)
C     CHECK FOR ERRORS, CALCULATE FX
      IF (R.NE.0..AND.S.NE.0)THEN
           FX = R-S
        IER = 0
      ELSE
        FX = 1.D+20
        IER = 1
      ENDIF
      RETURN
      END
```