ANALYTICAL STUDIES ON HIGH-PERFORMANCE CONCRETE T-BEAM BRIDGES

Thesis

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By

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(Virendra Kumar)

CERTIFICATE

This is to certify that the thesis entitled **ANALYTICAL STUDIES ON HIGH-PERFORMANCE CONCRETE T-BEAM BRIDGES** submitted in partial fulfilment of the requirements for the degree of **Doctor of Philosophy** with major in **Structural Engineering** of the college of Post-Graduate Studies, G. B. Pant University of Agriculture and Technology, Pantnagar, is a record of **bona fide** research carried out by Mr. Virendra Kumar Id. No. 17775, under my supervision, and no part of the thesis has been submitted for any other degree or diploma.

The assistance and help received during the course of this investigation have been acknowledged.

VBhargava

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We, the undersigned, members of the Advisory Committee of Mr. Virendra Kumar, Id No. 17775, a candidate for the degree of Doctor of Philosophy with major in Structural Engineering agree that the thesis entitled ANALYTICAL STUDIES ON HIGH-PERFORMANCE CONCRETE T-BEAM BRIDGES may be submitted in partial fulfilment of the requirements for the degree.

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ABBREVIATION AND NOTATIONS

ω	Effective reinforcement ratio or reinforcement index
$(As)_{min}$	Minimum area of tensile reinforcement
δc	Specific creep of concrete
τ_c	Permissible shear stress
τ_{co}	Basic permissible shear stress in deck slab
τ_{max}	Maximum permissible shear stress
Ast	Magnitude of tension (longitudinal) reinforcement
BMR1	Flexural resistance of longitudinal girder corresponding to tension failure
BMR2	Flexural resistance of longitudinal girder corresponding to compression failure
BMR2DFCK	Flexural resistance of longitudinal girder corresponding to compression failure using DFCK as girder concrete strength
BMR2EFCK	Flexural resistance of longitudinal girder corresponding to compression failure using EFCK as girder concrete strength
BMR2FCK	Flexural resistance of longitudinal girder corresponding to compression failure using FCK as girder concrete strength
Bult	Ultimate Bending Moment
b_w or b	Width of the web of the girder
Cc	Creep coefficient of concrete
d	Effective depth of the girder section
DEBM1 or debm1	Design bending moment in girder section 1 due to IRC loads

DEBM2 or debm2	Design bending moment in girder section 2 due to IRC loads
DEBM3 or debm3	Design bending moment in girder section 3 due to IRC loads
DESF1 or desf1	Design shear force in girder section 1 due to IRC loads
DESF2 or desf2	Design shear force in girder section 2 due to IRC loads
DESF3 or desf3	Design shear force in girder section 3 due to IRC loads
D_{fck}	characteristic strength of concrete in deck slab
DS or (ds)	Deck slab thickness
Ec	Secant modulus of elasticity of concrete
E_{fck}	Weighted average compressive strength in composite girder section
EPONTL1 or epontl1	Point load associated with EUDLGS1 and acting at centre of the girder
EPONTL2 or epontl2	Point load associated with EUDLGS2 and acting at centre of the girder
EPONTL3 or epontl3	Point load associated with EUDLGS3 and acting at centre of the girder
EUDLGB1 or eudlgb1	Equivalent UDL for DEBM1
EUDLGB2 or eudlgb2	Equivalent UDL for DEBM2
EUDLGB3 or eudlgb3	Equivalent UDL for DEBM3
EUDLGS1 or eudlgs1	Equivalent UDL for DESF1
EUDLGS2 or eudlgs2	Equivalent UDL for DESF2
EUDLGS3 or eudlgs3	Equivalent UDL for DESF3
$f`_c$ or f_c	28 day cylinder strength of concrete (150mm×300mm)
$f`_{cj}$	Concrete compressive strength at the time of transfer of prestress (as a fraction of the 28 day cylinder strength)
$f`_{ij}$	Concrete compressive strength at the time of transfer of prestress (as a fraction of the 28 day cube strength)
$f`_r$	Modulus of rupture strength of concrete
$f`_{sp}$	Tensile splitting strength of concrete

f'_t	Tensile strength of concrete
f_{ck}	28 day cube strength of concrete $(150\times150\times150)$mm
f_{ck}	Characteristic strength of concrete in girders
FHWA	Federal Highway Administration, USA
f_{pt}	Stress due to prestress at the tensile fiber at a distance y from centroid of the section
f_t	Maximum permissible tensile stress in concrete
f_y	Yield strength of steel
HPC	High-performance concrete
HSC	High-strength concrete
ICON or (icon)	Identity of concrete grade in main girders (1 to 8)
ICON1 or (icon1)	Identity of concrete grade in deck slab (1 to 5)
IG or (ig)	Girder identity (1 to 8)
IPRSEQ or (iprseq)	Prestressing sequence identity
IPS or (ips)	Prestressing cables identity (1 to 7)
K1	$1.14 - 0.7d \geq 0.5$ (d in m)
K2	$0.5 + 0.25\times100\times(Ast/bd) \geq 1.0$
M	Bending moment corresponding to maximum shear force due to ultimate load
M_t	Cracking moment of the girder section
$M_{ult}(D_{fck})$	Ultimate moment of resistance corresponding to D_{fck}
$M_{ult}(E_{fck})$	Ultimate moment of resistance corresponding to E_{fck}
$M_{ult}(f_{ck})$	Ultimate moment of resistance corresponding to f_{ck}
NCB or (ncb)	Number of cross beams
NG or (ng)	Number of longitudinal girders
NSC	Normal-strength concrete
Product (product)	$K1\times K2$
S or SPAN or span	Span (centre to centre of bearing) of the bridge
shearmax	Maximum shear stress (V/ bd)
Shearmax	Shear stress due to load (V/ bd)

shearmb	Mobilised shear strength (V_c / bd)
SHRP	Strategic Highway Research Programme
snetb	Net stress at bottom fibre of the girder
snett	Net stress at top fibre of the girder
snettm	Net stress at topmost fibre of the girder
SRATIO	Ratio of cantilever length of the deck slab and kerb width
TCOST or tcost	Cost of bridge superstructure per m of bridge length normalised w.r.t.. cost of one ton of M30 concrete
V	Ultimate shear force
V_c	Ultimate shear resistance offered by concrete
v_c or Shearmb	Cracking shear strength (V_c / bd)
V_{ci} or V_{cw}	Shear resistance of concrete section cracked in flexure
V_{cw} or V_{co}	Shear resistance of uncracked concrete section
V_s	Shear resistance offered by the shear reinforcement
V_u or V_n	Ultimate shear resistance of the section ($V_c + V_s$)
W_c	Unit weight of concrete
X_u	Depth of neutral axis or parabolic stress block

1. INTRODUCTION

1.1 Introduction

Traditionally, the process of design in structural engineering and particularly in bridge engineering has relied on the designer's experience, intuition and ingenuity. Although this process has worked well as evidenced by the existence of many excellent structures, it is not a rigorous approach to design. Moreover, it can be laborious and time consuming, and often leads to sub-optimal design.

The use of high-strength concrete in bridges is unlikely to advance quickly without a clear economic incentive.

Most existing concrete codes were based on experiments and engineering practice on members and structures fabricated from low- and medium-strength concretes. From the existing literature it is seen that the structural properties of high-strength concrete are very much different from those of normal-strength concrete. Since high-strength concrete is a brittle material, both strength and ductility are important. It is also important that designers should be able to estimate the difference in the safety margins between structures using conventional strength concretes designed to current codes of practice and their high-strength concrete alternatives. The related code provisions are, therefore, required to be critically examined with reference to use of high-strength concrete. Extrapolation of various existing code provisions to higher strength materials is unjustified and may be dangerous. With increasing use of high-strength concrete, it is necessary to investigate the applicability of the current IRC design codes to this material.

A survey of the published literature on the behaviour of prestressed and reinforced high-strength concrete beams and bridge girders was aimed at verifying the ACI code and AASHTO recommendations. It appears that there is a lack of published research work on optimization oriented design of bridges as per various relevant IRC codes and the verification of the IRC design recommendations for high-strength concrete bridges. Therefore, an assessment of design recommendations for high-strength concrete bridges with concrete strength in excess of 60 MPa is needed to supplement the provisions of the related IRC codes of practice.

1.2 Aims of the Investigation

The bridges with cast-in-situ, reinforced concrete decks on precast, prestressed concrete I-girders are commonly known as slab-on-girders bridges. The typical cross section of the such bridge superstructure is shown in Fig. 1. The simply supported slab-on girders bridge is chosen for the present investigation because it represents the most common type of prestressed concrete bridge constructed during the last few decades. The objectives of the present investigation are:

I) To develop a computer programme for analysis and optimized design of the super-structure of the simply supported prestressed concrete slab-on -girder bridges. The design and analysis will be strictly as per various related IRC codes recommendations.

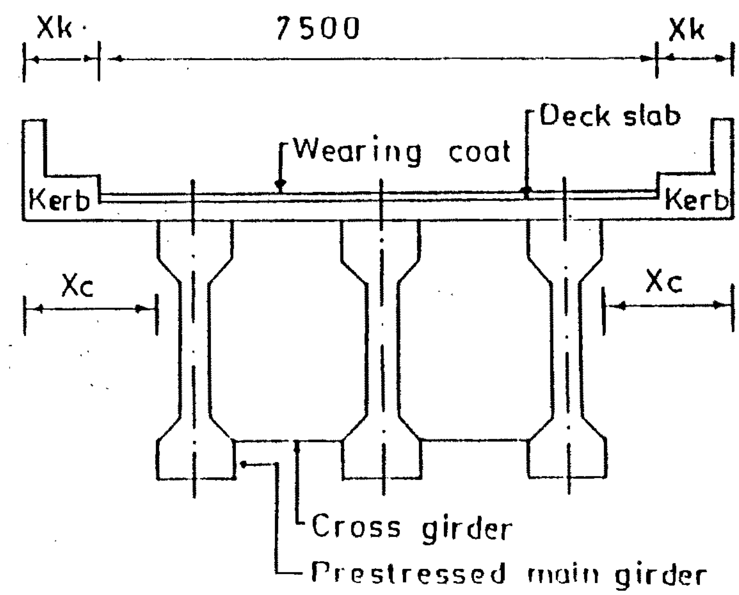


FIG. 1 CROSS-SECTION OF BRIDGE DECK

II) To study the effect of various parameters of bridge super-structure on the cost and efficiency of the super-structure.

III) To provide a equivalent uniformly distributed load for computing the bending moment and shear force in the girders due to various IRC loadings, for all the possible alternative designs of the super-structures for span range of 18m to the maximum possible for all the girder sections considered in the study.

IV) To develop the design tables and graphs for cost-effective preliminary design of the post-tensioned slab-on-girder bridges.

V) To asses the suitability of the present IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997) code provisions for their applicability to high-strength concrete used in bridges.

VI) To propose the revisions in present IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997) code provisions, if found necessary, for extending their applicability to high-strength concrete.

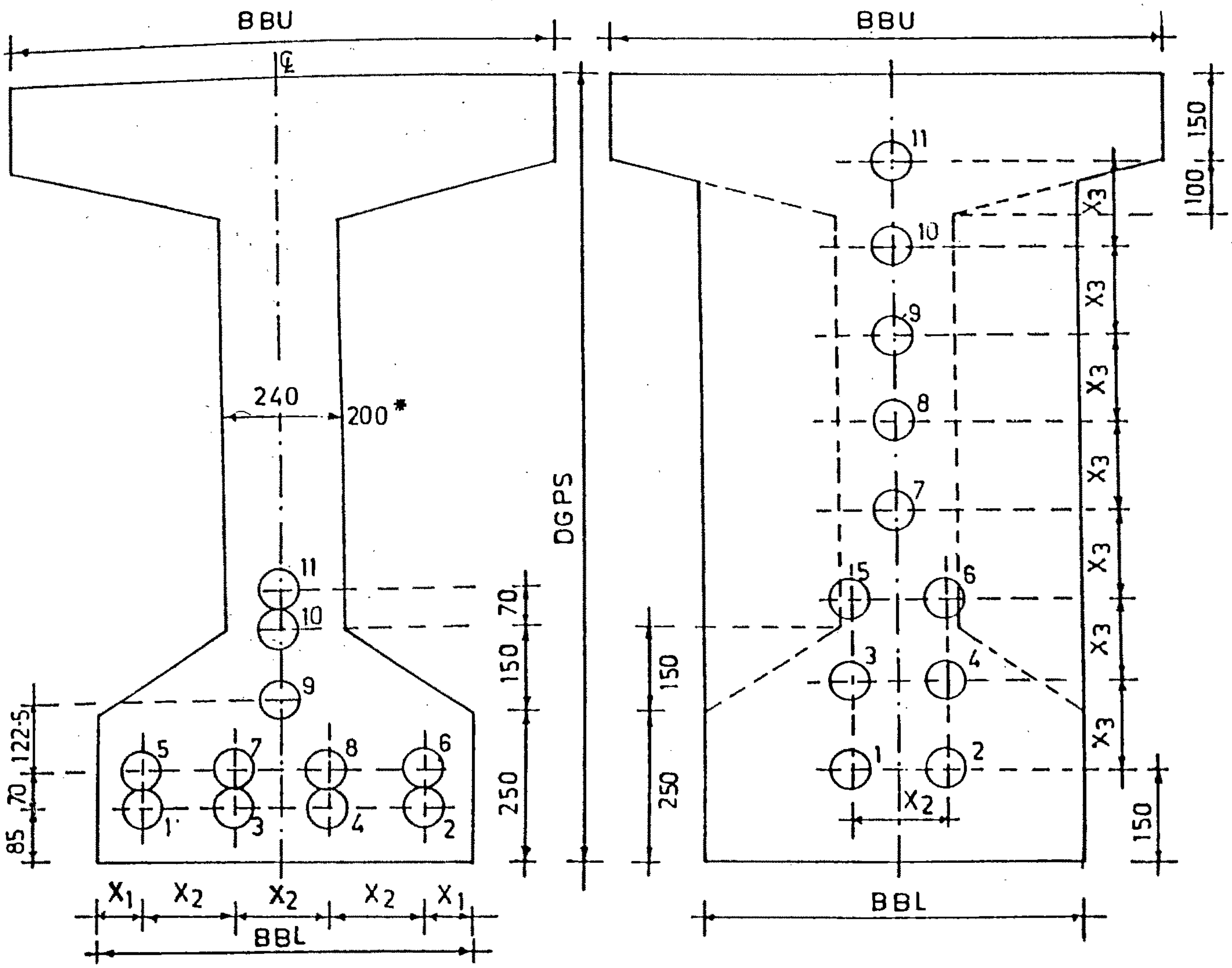
VII) To study the economic aspects of use of high-strength concrete in highway slab-on-girder bridges.

1.3 Scope of the Investigation

The simply supported, composite, prestressed concrete slab-on-girder bridge is chosen for the present investigation. A detailed computer programme is developed in FORTRAN for analysis and optimised design of bridges of the present investigation. The analysis and design is done according to the various relevant IRC codes; IRC: 5 - 1985 (1985), IRC: 6 - 1966 (1990), IRC: 18 - 1985 (1997), IRC: 21 - 1987 (1997) and IRC:22 - 1986 (1991). Eight sections which are close to standard sections (two AASHO sections; type V and type VI, one Oregon section, two Washington sections; type 100S and type 120S, three Colorido sections; type G66, type G72, and type G80) have been used [Frances and Jacques (1971)]. Slight modifications in dimensions were necessary to incorporate the specifications of IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997). The details of geometry of the chosen girder sections is given in Fig. 2. Concrete strength used in deck slab have 28 day cube compressive strength of 30, 35, 40, 45 and 50 MPa. As per IRC: 18 - 1985 (1997) , the minimum grade of concrete to be used in deck slab in composite construction should have a strength at least equal to 30 MPa. The concrete grades higher than 50 MPa do not seem to be advantageous in deck slab because IRC: 21 - 1985 (1997) restricts the minimum thickness of deck slab as 200mm.

In pre-stressed girders the concrete grades used have characteristic strength values of 35, 40, 45, 50, 60, 70, 80 and 90 MPa. This wide range of concrete used in girder covers the normal-strength concrete (with strength less than 60 MPa) as well as high-strength concrete. Normal-strength concrete is considered in study to asses the advantage of high-strength concrete over the normal-strength concrete. The loading considered on bridge are IRC class AA and class A of IRC: 6- 1966 (1990), as the bridges with high-strength concrete are expected to be mainly provided on major highways.

The standard Freyssinet pre-stressing cables (1984) covering a wide range of pre-stressing force are used in the investigation. The used standard pre-stressing Freyssinet cable systems are 12ϕ5, 12ϕ7, 12ϕ8, 24ϕ7, 24ϕ8, 6T13, and



SECTION IDENTITY	BBU	BBL	DGPS	X_1	X_2	X_3
1	600	615 (600*)	1500	105 (90*)	140	175
2	1070	710	1600	130	150	185
3	1070	710	1830	130	150	220
4	1220	660	1830	105	150	220
5	710	620 (610*)	1870	105 (95*)	140	225
6	710	660	1680	105	150	200
7	710	660	1830	105	150	220
8	710	660	2030	105	150	250

NOTE: ALL DIMENSONS ARE IN MM

FIG. 2 VARIOUS SECTIONS USED IN INVESTIGATION

*: MINIMUM VALUES AS PER IRC 18-1985, 1990 EDITION

12T13. Eleven number of cables are used in all the sections. The position of cables in various girder sections are shown in Fig. 2.

The pre-stressing cables are to be stressed in three stages. In the first stage of pre-stressing, few cables are stressed at 7 days age of the girder. Maximum number of cables are stressed in second stage when the girder is of 28 days age, after the casting of the cross beams and positioning the shuttering and reinforcement for deck slab. Remaining cables are stressed in third stage on 60th day of casting of girder, the casting of deck slab and fixing the railings.

Twenty alternatives of stressing different number and combination of cables in three stages of pre-stressing are considered so that a wide variation in span and concrete grade may be covered as far as safety of girders at the time of transfer of pre-stress is concerned.

As per AASHO (1977) specification, the maximum spacing of cross beams is 12 metre centres. As Courbon's method is not used for load distribution among girders, the minimum number of cross beams used is 3. The maximum number of cross beams considered is 12 so that the spacing of cross beams do not exceed 12 m even for maximum possible span of the bridge when the highest grade of concrete and the deepest girder section (girder section 8) is used. The number of longitudinal girders varied from 2 to 5.

The overhanging part of the deck slab is varied between one to three times the width of the kerb. The bending moments in girder is calculated using distribution factor which is critical [calculated using Morrice-Little (Rowe) and Hendry- Jarger (1958) methods]. As per IRC: 21 - 1987 (1997), the shear force due to a load within 5.5 m from the support also depends upon the reaction at girder, assuming the deck slab simply supported or continuous over the girders as unyielding supports. Thus, critical shear distribution coefficient was maximum of the coefficients calculated as per Morrice-Little (Rowe), Hendry - Jaeger (1958) and the above discussed IRC: 21 - 1987 (1997) clause.

1.4 Thesis Organisation

The thesis is mainly divided into six chapters. The first chapter is in the nature of an introduction to high-strength concrete and its application to highway bridges. The main objectives and parameters of the thesis are described. The presentation of the entire thesis has been kept as brief as possible, consistent with readability and comprehension. As far possible repetition is avoided though sometimes it is necessary to repeat some matter in the different chapters for the sake of continuity and clearity.

The second chapter deals with the review of literature. The topics on the properties of high-strength concrete and its application to highway bridges are highly relevant here. Hence, these have been covered in more detail. However, topics such as application of high-strength concrete in compression elements and other structures are covered only briefly. The number of references on high-strength concrete highway bridges are very limited and so have been covered in detail.

Chapter three deals with the analysis and formulation. Most of the assumptions made in the analysis and design are based on the provisions of Indian and other relevant standards on highway bridges. The complete analytical study is divided into; parametric study, need of amendments in IRC: 18 - 1985

(1997) and IRC: 21 - 1987 (1997), proposal of amendments in IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997) for effective use of high-strength concrete in highway bridges and economic studies on the bridges in the present investigation. A number of tables and graphs are prepared to study the effect of various superstructure parameters on the optimization of the superstructure. These graphs and tables may help the designer to get a cost effective preliminary design.

The fourth chapter deals with the proposed amendments in IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997). The proposed amendments are critically discussed and are based on the well documented analytical and experimental investigations on the properties and application of high-strength concrete in highway bridges and other similar structural elements.

Chapter fifth deals with the discussion of results of the present study and their comparison to the results of similar studies available in the literature. The brief discussion on the economics and optimized design of slab-on-girder highway bridges is also included.

The sixth chapter deals with the main conclusions of this study. The areas requiring further analytical and experimental investigations are identified. The author would like to pursue some of these areas outside the scope of this thesis without time constraints being imposed. The references are given after the appendices.

2. REVIEW OF LITERATURE

2.1 Introduction

The present investigation is mainly on the use of high-strength concrete in highway bridges with the aim that the designers be able to estimate the difference in the safety margins and cost economy between bridges using conventional strength concrete designed to current code of practices and their high-strength concrete alternatives. Thus literature on high-strength concrete is reviewed to complete the overview.

High-strength concrete was defined in different ways in different regions. A brief literature survey is presented on the varying definitions of high-strength concrete as given by various investigators.

Since necessary modifications in various IRC codes, for advantageous use of high-strength concrete in highway bridges, are to be proposed, all the related mechanical properties of high-strength concrete and their design implications are reviewed in detail.

For relative cost estimation purposes, the required properties of constituents materials and methods of mix proportioning are briefly reviewed.

The application of high-strength concrete in various structures is reviewed briefly. In highway bridges it may be used effectively to reduce the cost of the structure. The application of high-strength concrete in highway bridges is reviewed in detail.

2.2 High-strength Concrete - Definition

With the variability in physical properties and availability of concrete making materials in different regions, the definition of high-strength concrete varies with location. In general, it is defined as the concrete with a uniaxial compressive strength greater than what is ordinarily obtained in a region. As the development is continued, the definition of high-strength concrete has changed.

Carrasquillo and Nilson et al. (1981) defined the high-strength concrete as concrete having compressive strength in the range of 41 to 83 MPa.

As per **ACI 363R (1984),** in 1950, 1960 and in early 1970, a concrete with a compressive strength of about 34 MPa, 41 to 52 MPa and about 62 MPa respectively was considered as high-strength concrete. The definition of high-strength concrete is different in different countries. For instance where as in Australia, high-strength concrete is defined as concrete having a 28 day compressive strength of minimum 50 MPa, in Europe, it is defined as a concrete with a 28 day compressive strength of minimum 60 MPa.

Adelman and Cousins (1990) recommended that high-strength concrete should have 28 day compressive strength in the neighbourhood of 69 MPa.

Tang and Patnaikuni (1992) classified the high strength concrete on the basis of compressive strength of concrete cylinder at 56 day as given below:

Concrete	Normal-High-Strength Concrete	Very-High-Strength Concrete	Ultra-High-Strength Concrete	Super-High-Strength Concrete
Strength (MPa)	50 to 100	100 to 150	150 to 200	Over 200

Goodspeed and Vanikar (1996) reported that (Federal Highway Administration, USA) FHWA proposed to define high-performance concrete using long-term performance criteria. The proposed definition consists of four durability related parameters (1) freezing and thawing, (2) abrasion, (3) chloride permeability and (4) scaling and four strength parameters (1) compressive strength, (2) modulus of elasticity, (3) creep and (4) shrinkage. Associated with each definition parameter are performance criteria, testing procedures to measure performance, and recommendations to related performance to adverse field conditions. Performance characteristics of HPC and test methods for determining the grades of HPC are given in Tables 1 & 2. To specify a high-performance concrete mixture using the FHWA definition, a user states, based on field conditions, the level of performance desired for each performance characteristic. Updates will have to be made to keep the definition current with improvements in technology and field experience. They also reported the studies by SHRP which defined the high-performance concrete as:

(1) a maximum water-cement ratio of 0.35,

(2) a minimum durability factor of 80%, determined as per ASTM C 666 procedure , and

(3) a minimum strength criteria of either

(a) 21 MPa within 4 hours after placement (very early strength, VES),

(b) 34 MPa within 24 hours (high early strength HES), or

(c) 69 MPa within 28 days (very high strength VHS).

TABLE 1 PERFORMANCE CHARACTERISTIC FOR HIGH PERFORMANCE STRUCTURAL CONCRETE

PERFORMANCE	STANDARD	FHWA HPC PERFORMANCE GRADE			
CHARACTERISTIC	TEST METHOD	1	2	3	4
FREEZ/THAW DURABILITY (x= relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666	$60\% \leq x < 80\%$	$80\% \leq x$		
SCALING RESISTANCE (x= visual rating of the surface after 50 cycles)	ASTM C 672	$x = 4,5$	$x = 2,3$	$x = 0,1$	
ABRASION RESISTANCE (x = avg. Depth of wear in mm)	ASTM C 944	$2.0 > x \geq 1.0$	$1.0 > x \geq 0.50$	$0.50 > x$	
CHLORIDE PERMEABILITY (x = coulombs)	AASHTO T 277 ASTM C 1202	$3000 \geq x > 2000$	$2000 \geq x > 800$	$800 \geq x$	
STRENGTH (x = compressive strength)	AASHTO T 22 ASTM C 39	$41 \leq x < 55$ MPa	$55 \leq x < 69$ MPa	$69 \leq x < 97$ MPa	$x \geq 97$ MPa
ELASTICITY (x = modulus of elasticity)	ASTM C 469	$28 \leq x < 40$ GPa	$40 \leq x < 50$ GPa	$x \geq 50$ GPa	
SHRINKAGE (x = microstrain)	ASTM C 157	$800 > x \geq 600$	$600 > x \geq 400$	$400 > x$	
CREEP (x= microstrain / pressure unit)	ASTM C 512	$(75 \geq x > 60)$/ MPa	$(60 \geq x > 45)$/ MPa	$(45 \geq x > 30)$/ MPa	$30 \geq x$ MPa

TABLE 2 DETAILS OF TEST METHODS FOR DETERMINING HPC PERFORMANCE GRADES

PERFORMANCE CHARACTERISTIC S	STANDARD TEST METHODS	NOTES
FREEZ/THAW DURABILITY	AASHTO T 161 ASTM C 666	1. Test specimens 76.2×76.2×279.4 mm cast or cut from 152.4×304.8 mm cylinder. 2. Acoustically measure dynamic modulus until 300 cycles.
SCALING RESISTANCE	ASTM C 672	1. Test specimen to have a surface area of 46451 mm^2. 2. Perform visual inspection after 50 cycles.
ABRASION		1. Concrete shall be tested at 3 different locations. 2. At each location, 98 Newtons, for three, 2 minutes, abrasion period shall be applied for a total of 6 minutes of abrasion time per location. 3. The depth of abrasion shall be determined per ASTM C 799 Procedure B.
CHLORIDE PERMEABILITYT	AASHTO T 277 ASTM C 1202	1. Test per standard test method.
STRENGTH	AASHTO T 22 ASTM C 39	1. Moulds shall be rigid metal or one time used rigid plastic. 2. Cylinders shall be 100 mm dia. × 200 mm long or 150 mm dia. × 300 mm long. 3. Ends shall be capped with high strength capping compound, ground parallel, or placed onto neoprene pads per AASHTO specifications for concretes. 4. Use of neoprene pads on early age testing of concrete exceeding 70 MPa at 56 days should use neoprene pads on the 56 day test. 5. The 56 day strength is recommended.
ELASTICITY	ASTM C 469	1. Test per standard test method.
SHRINKAGE	ASTM C 157	1. Use 76.2 × 76.2 × 28.5 mm specimens. 2. Shrinkage measurements are to start 28 days after moist curing and be taken for a drying period of 180 days.
CREEP	ASTM C 512	1. Use 152 ×305 mm specimens. 2. Cure specimens at 73° F and 50% RH after 7 days until loading at 28 days. 3. Creep measurements are to be taken for a creep loading period of 180 days.

Iravani (1996) defined the high-performance concrete as concrete that meets special performance and uniformity requirements that can not always be achieved routinely by using conventional materials and normal mixing, placing and curing practices. These requirements may include the following enhancements:

(1) ease of placements and consolidation without affecting strength,

(2) long-term mechanical properties,

(3) early high strength,

(4) toughness,

(5) volume stability, and

(6) longer life in severe environments.

According to **ACI-ASCE Committee 441 (1997)**, high-strength concrete should have compressive strength in excess of 70 MPa.

Diniz and Frangopol (1997) considered 50 MPa as the lower limit of high-strength concrete.

According to **Foster and Attard (1997)**, a concrete with a compressive strength of 41-56 MPa, approximately 75 MPa and 90 MPa is termed as low-strength, medium-strength and high-strength concrete respectively.

Pessiki and Pieroni (1997) adopted the following definition: low-strength concrete, medium-strength concrete and high-strength concrete should have compressive strength of the order of 41.4 MPa, 41.4 to 55.2 MPa and in excess of 55.2 MPa respectively.

According to **Shah (1997)**, compressive strength is not sufficient for the purpose of distinguishing various attributes of concrete. However, it is common to describe concrete using compressive strength as sufficient criterion. This is because compressive strength is the most commonly used measure of concrete quality. Thus, compressive strength is used as a criterion for high-performance concrete.

French et al. (1998) considered a concrete with a compressive strength greater than 41 MPa as high-strength concrete.

On the basis of the 56 day compressive strength **Iravani (1998)** classified the high strength concrete as:

High-Strength Concrete	65 to 95 MPa
Ultra-High-Strength Concrete	105 to 120 MPa

2.3 Mechanical Properties of High-strength Concrete

Empirical equations used to predict properties of concrete have been based on tests of concrete with traditional materials having compressive strength less than approximately 41 MPa. **Carrasquillo and Nilson et al. (1981)** and **French et al. (1998)** considered the extrapolation of these empirical equations to materials of higher strength and different microstructures as unjustified and dangerous. Increasing use of high-strength concrete has made it necessary to review it's various properties in detail.

Smadi et al. (1987) emphasised on the need of comprehensive knowledge of the material's fundamental properties that determine it's deformational characteristics under load. Such properties considered were the creep properties under loading and the shrinkage properties under drying conditions.

Iravani (1996) considered high-strength concrete as a relatively new material. Some results of research on conventional concrete might not be entirely applicable to high-strength concrete. An extended comprehensive knowledge of high-strength concrete was considered extremely necessary. Mechanical properties of high-strength concrete were considered among those fundamental properties.

Chin et al. (1997) considered the knowledge of complete stress-strain relationship of concrete vital to achieve a rational design of structures and structural components involving concrete.

2.3.1 Stress-strain behaviour in uniaxial compression

Axial-stress versus strain curves for concrete of strength up to 120 MPa as reported by **Wee et al. (1996)** are shown in Fig. 3. Results of various researchers like **Wang et al. (1978a), Carrasquillo and Nilson et al. (1981), Shah et al. (1981) and Wee et al. (1996)** have shown that the shape of the ascending part of the stress-strain curves was more linear and steeper for high-strength concrete , and the strain at maximum stress is slightly higher for high-strength concrete. The slope of descending part became steeper for high-strength concrete.

Wang et al. (1978a), Carreria and Chu (1985), Wee et el. (1996), and **Ibrahim and MacGregor (1996)** etc. proposed the analytical models of complete stress-strain curve of high-strength concrete.

Carrasquillo and Slate et al. (1981) reported less internal micro-cracking in high-strength concrete than lower-strength concrete for a given imposed axial strain.

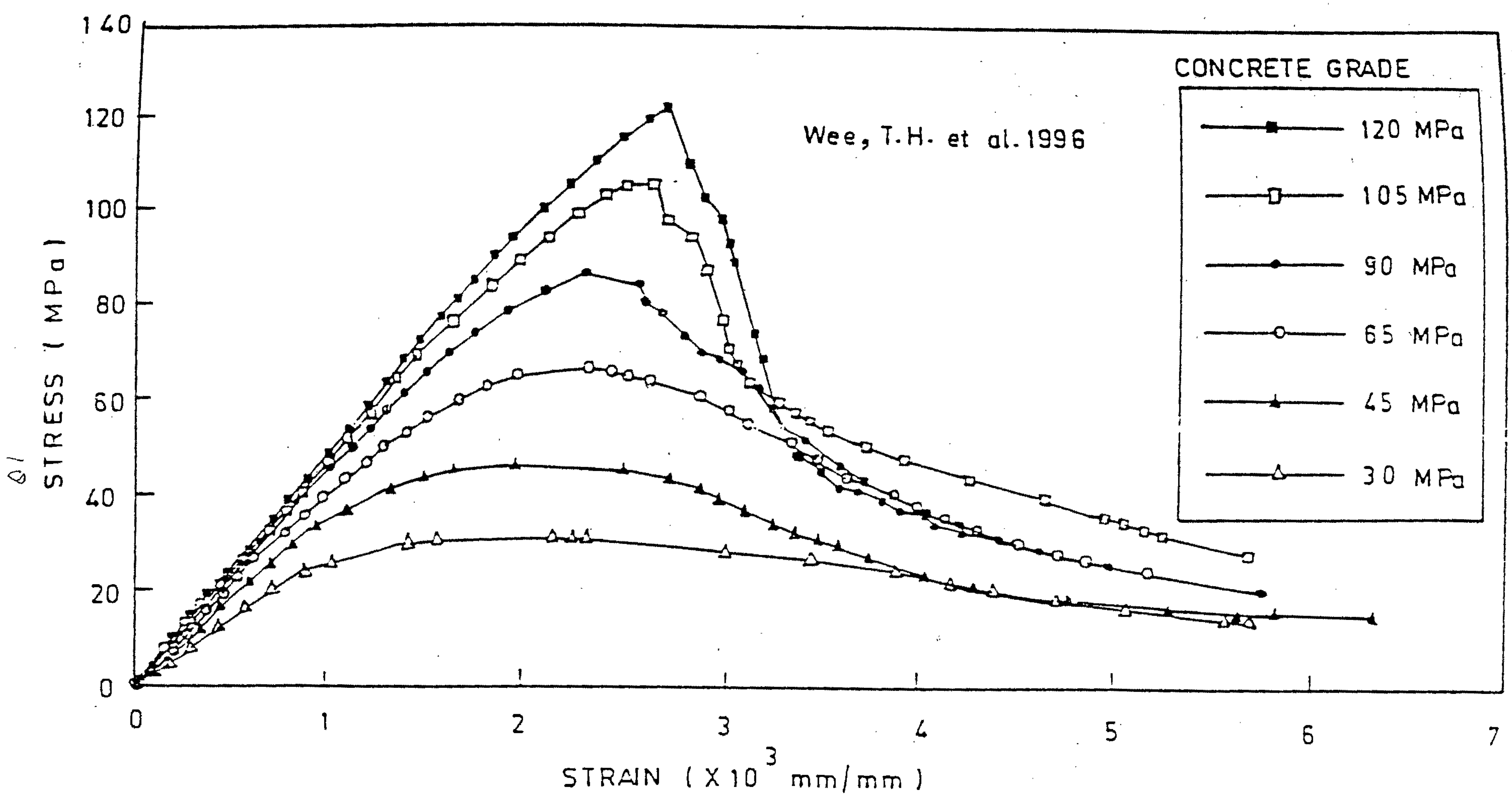


Fig. 3 TYPICAL STRESS-STRAIN CURVES FOR DIFFERENT GRADES OF CONCRETE

The findings of **Ahmad and Shah (1982a)** has shown relatively less lateral strains in high-strength concrete in comparison to normal-strength concrete.

Ibrahim and MacGregor (1996) characterised the ascending branch of stress-strain curves of unconfined high-strength concrete in compression as almost linear.

Wee et el. (1996) emphasised the need of a suitable standard test method for generating the complete stress-strain curve of high-strength concrete.

Neville (1997) reported that linear part of the stress-strain curve in high-strength concrete extended to a stress that was as high as 85% of the ultimate strength or even higher.

Sarkar et al. (1997) tested 100 × 300 mm cylinders at 28 day. They observed that the slope of ascending branch of stress-strain curve was fairly linear up to 75% of the ultimate strength.

Sidek Sener (1997) reported that large cylinders tend to fail in a more brittle, while smaller cylinder tend to fail in a less brittle manner or more plastic manner.

Iravani and MacGregor (1998) observed that ascending branch of short-time stress-strain curves had steeper slope and were more linear over a greater range as the compressive strength increased. The deviation from straight line occurred at about 65 to 70, 75 to 80, above 85 and above 85 percentage of the ultimate for concrete strengths of 65 MPa, 95 MPa, 105 MPa and 120 MPa at 56 day of age, respectively.

2.3.2 Modulus of elasticity

The broad relation between the modulus of elasticity of concrete and it's compressive strength is well known. But there is no agreement on the precise form of the relation. According to **Neville (1997)** " there can be no unique relation because the modulus of elasticity of concrete is affected both by the modulus of elasticity of aggregate and the volumetric content of aggregate in concrete."

Pauw (1960) recommended the following equation for the modulus of elasticity of concrete.

$$Ec = 0.0428(Wc)^{1.5}\sqrt{fc'}\,MPa$$

Based on the experimental work of Pauw (1960), **AASHTO (1977)** and **ACI 318 (1989)** recommended the following equation for modulus of elasticity of normal weight (Wc = 2320 Kg / m^3 i.e. 145 pcf).

$$Ec = 4730\sqrt{fc'}\,MPa$$

IS: 456 - 1978 (1981) proposed the following equation for estimating the modulus of elasticity (at 33% of ultimate strength) of normal weight concrete.

$$Ec = 6370\sqrt{fc'}\,MPa$$

Research work of **Carrasquillo and Nilson et al. (1981)**, and **Aiticin and Mehta (1990)** have shown that the type of coarse aggregate used to make high-strength concrete was most significant parameter affecting the modulus of elasticity of concrete.

Carrasquillo and Nilson et al. (1981) reported about the overestimation of modulus of elasticity of concrete above 41 MPa by ACI 318-1977 (equation same as in ACI 318-1989). The following equation was proposed by them for high-strength concrete.

$$E_c = 3320\sqrt{f_c'} + 6900.........MPa$$

ACI 363R (1984) recommended the above equation by **Carrasquillo and Nilson et al. (1981)** for high-strength concrete in the strength range of 21 to 83 MPa.

Jobse and Moustafa (1984) reported that the modulus of elasticity of 70 MPa concrete was better predicted by the expression given below (for normal weight concrete) than by the equation proposed by ACI 318 -1989 and AASHTO 1977.

$$E_c = 3766\sqrt{f_c'}\,MPa$$

Shah and Ahmad (1985) proposed the following equation for modulus of elasticity. This equation is reported to be comparable to the ACI 318 -1989 equation for low- and normal-strength concrete but is more accurate for high-strength concrete.

$E_c = 3.3945 \times 10^{-5} \times W^{2.5} \times \sqrt[0.325]{f_c'}$ where W in Kg./m^3 and f_c' in MPa

Kaufman and Ramirez (1989) reported that the modulus of elasticity of concrete produced using limestone and natural gravels were best represented by ACI 318 -1989 and ACI 363R 1984 equations respectively.

Adelman and Cousins (1990) reported that the modulus of elasticity of high-strength concrete produced using limestone was predicted accurately by ACI 318-1989 equation.

Hwee and Rangan (1990) obtained the chord modulus of elasticity at 40% of the compressive strength of commercially available high-strength concrete. The experimental values were reported to be in good agreement with those calculated using **ACI 363R (1984)** expression.

Baalbaki et al. (1992) and **Cetin and Carrasquillo (1998)** reported that high-performance concrete lack a single equation to estimate the elastic modulus with reasonable accuracy. This could be attributed to different coarse aggregates, as well as to the maturity of the cement paste. As a result, high-strength concrete appeared to lack an empirical equation that represented elastic modulus in terms of compressive strength. Any required elastic modulus value was suggested to include in specifications along with the compressive strength. Due to lack of good estimate, the measured value was recommended to be used in place of any predicted value.

Setunge et al. (1993) reported that failure envelop of very high-strength concrete was very little depend upon the compressive strength and mineralogical properties of coarse aggregates.

Khan et al. (1995) reported overestimation of modulus of elasticity of very early age concrete and concrete strength above 50 MPa if ACI 318 -1989 expression was used. On the other hand ACI 363R 1984 expression was found to provide a more conservative estimate of the average stiffness for high concrete strengths.

Krishnaraju (1995) summarised the values of modulus of elasticity of concrete as recommended by the British code for structural concrete, BS-8110 (1985) and DIN 4227.

Cube Strength at appropriate age (MPa)	20	25	30	40	50	60
Modulus of elasticity (MPa × 10^3) BS-8110	24	25	26	28	30	32

Cylinder strength (MPa)	22.5	30	45	60
Modulus of elasticity (MPa × 10^3) DIN 4227	24	30	35	40

Attard and Setunge (1996) reported that ACI-318 (1989) equation though approximate, was accurate only within ± 20% for normal strength concrete. On the basis of their experimental results they found that ACI 318 -1989 equation gave reasonable estimate for mixes with stronger crushed aggregates (the Hornfels and Rhyodacite) while ACI 363R 1984 gave reasonable estimate for mixes containing the weaker aggregates (the Vesicular Basalt).

Iravani (1996) summarised the literature review on modulus of elasticity of concrete. It was reported that supplementary cementitious materials (such as silica fume and fly ash) and specimen size affected the modulus of elasticity but their effect was negligible as compared to the effect of others parameters. He recommended the following equation for the concrete strength range from 55 to 125 MPa. It was measured at 56 day and corresponded to 40% of ultimate strength.

$$E_c = 4700 C_a \sqrt{f_c'}\ MPa$$

Aggregate	Limestone	Dolomite	Quartzite	granite	Trap Rock
Ca	0.92	0.92	0.97	0.87	0.97

Wee et al. (1996) recommended the equation given below for initial tangent modulus for concrete strength range of 50 to 120 MPa.

$$E_c = 10200 \times {}^{1/3}\sqrt{f_c'}\ MPa$$

According to **Neville (1997)**, none of the expressions for the modulus of elasticity of concrete considered the bond between the aggregate and surrounding hardened cement paste. The bond depends on the interface zone, which is known to have a different microstructure from the bulk of the hardened cement paste and which is the locus of early microcracking, known as bond microcracking.

French et al. (1998) reported that, for high-strength concrete, the ACI 318 - 1995 equation (same as ACI 318 -1989 equation) overestimated the measured modulus of elasticity whereas the ACI 363R 1984 relation slightly overestimated the modulus of elasticity. The early age modulus of elasticity is important to the precast prestressed industry for investigating the effects such as elastic shortening. The 1 day modulus of elasticity was reported to be approximately 97% of the 28 day modulus of elasticity.

2.3.3 Modulus of rupture

The values reported by various investigators for modulus of rupture of both light-weight and normal weight concrete fall in the range of $0.623\sqrt{fc'}$ and $0.996\sqrt{fc'}$ MPa.

AASHTO (1977) design recommendations specify the following values as allowable stress limit after prestress loss for member with bonded tendons.

$$fr' = 0.4983\sqrt{fc'}\,MPa$$

Carrasquillo and Nilson et al. (1981) recommended the following equation, based on the moist cured specimens tested at the age of 7, 28, and 95 days for modulus of rupture strength fr'. This equation was adopted by the **ACI 363 (1984)** for estimating the modulus of rupture in the strength range from 21 to 83 MPa.

$$fr' = 0.94\sqrt{fc'}\,MPa$$

Shah and Ahmad (1985) proposed the following equation for finding the modulus of rupture for concrete strength up to 84 MPa.

$$fr' = 0.4379 \times {}^{2/3}\sqrt{fc'}\,MPa$$

ACI 318 (1989) proposed the following equation for modulus of rupture strength.

$$fr' = 0.6228\sqrt{fc'}\,MPa$$

Kaufman and Ramirez (1989) reported that AASHTO - 1977 gave a very much conservative value whereas ACI 363R - 1984 was an upper bound to their test data. The use of ACI 318 - 1989 equation was recommended for design of prestressed bridge girders.

Adelman and Cousins (1990) reported that their test data were well represented by the equation proposed by ACI 363R (1984) whereas the ACI 318 (1989) equation gave significantly less values than those found experimentally.

Iravani (1996) reported that the results of his study and various other researches were in the range of $\pm 10\%$ of the equation recommended by ACI 363R - 1984. The range of applicability of ACI 363R - 1984 equation for modulus of rupture was proposed to be extended to high-performance concrete with or without supplementary cementitious materials with compressive strength up to 120 MPa at the age of 28 days. He proposed the following equation on the basis of the specimens tested at the age of 56 days and cured continuously at 100% relative humidity (concrete strength range from 50 to 100 MPa).

$$fr' = 0.97\sqrt{fc'}\,MPa$$

2.3.4 Tensile splitting strength

Mirza et al. (1979) found the mean split cylinder strength from a large number of tests of concrete from various localities.

$$f'_{sp} = 0.5315\sqrt{fc'}\,MPa$$

Carrasquillo and Nilson et al. (1981) suggested the following equation for the prediction of tensile splitting strength of normal weight concrete for the strength range from 21 to 83 MPa.

$$f'_{sp} = 0.54\sqrt{fc'}\,MPa$$

ACI 363R (1984) summarised the available investigations on tensile splitting strength of concrete. It was reported that at low strengths, the indirect tensile strength might be as high as 10% of the compressive strength but at higher strength it might reduce to 5%. The tensile splitting strength was reported about 70% of the flexural strength. The Committee recommended the following equation for normal weight concrete for concrete strength ranging from 21 to 83 MPa.

$$f'_{sp} = 0.59\sqrt{fc'}\,MPa$$

Raphel (1984) proposed the following expression for splitting cylinder tensile strength of concrete.

$$f'_{sp} = 0.342 \times {}^{2/3}\sqrt[3]{fc'}\,MPa$$

Shah and Ahmad (1985) proposed the following equation, based on the available experimental data on split cylinder tests on concrete of low-, medium- and high-strengths, to predict the average split tensile strength for concrete of strength up to 84 MPa.

$$f'_{sp} = 0.4622 \times {}^{0.55}\sqrt[5]{fc'}\,MPa$$

The lower bound of test results was given by

$$f'_{sp} = 0.4983\sqrt{fc'}\,MPa$$

Kaufman and Ramirez (1989) reported that the ACI 363R 1984 equation, for predicting the tensile strength of concrete, better represented the behaviour of structural members under field conditions.

Iravani (1996) proposed the following equation after testing specimens air cured continuously at 100% relative humidity and tested at the age of 56 days. The estimated proportion of coarse aggregate fractured during the test was generally more than 95%.

$$f'_{sp} = 0.57\sqrt{fc'}\,MPa$$

Based on the results of his own and other investigations he recommended the extension of the range of applicability of ACI 363R 1984 equation for tensile splitting strength of concrete with or without supplementary cementitious materials with compressive strength up to 120 MPa at the age of 28 days.

On the bases of their investigation on concrete with 100×100 mm cube compressive strength up to 120 MPa, **Sarkar et al. (1997)** proposed the relationship given below.

$$f'_{sp} = 0.564 \times \sqrt[0.55]{fc'}\,MPa$$

They observed that relationships developed by Carrasquillo, R.L.; Nilson, A.H. et al. 1981, ACI 363R 1984 and Shah, S.P. & Ahmad, S.H. 1985 underestimated the splitting cylinder tensile strength of high-strength concrete mixes with fc' up to 120 MPa. It was recommended that the expressions suggested by them and Raphel (1984) may be used for prediction of the splitting cylinder tensile strength of high-strength concrete.

2.3.5 Shrinkage

Little information is available on the shrinkage behaviour of high-strength concrete. Review of literature on shrinkage of concrete is available in **ACI 363R (1984)**. Relatively high initial shrinkage was reported in high-strength concrete. But after drying for 180 days very little difference was observed between the shrinkage of high-strength and lower-strength concretes made with Dolomite or Limestone. Shrinkage was reported to be unaffected by changes in water-cement ratio but was approximately proportional to the percentage of water by volume in the concrete. Field studies had shown that shrinkage of high strength concrete is similar to that of lower-strength concrete. It was observed that the shrinkage of high-strength concrete was about the same as that of normal-strength concrete.

Smadi et al. (1987) reviewed the investigations on shrinkage of concrete. Increase in aggregate content was reported to reduce the shrinkage of concrete. It was observed that increase in water-cement ratio would intensify the shrinkage of cement paste and would accelerate the volume contraction process by providing more space for free water diffusion. He reported greater shrinkage for low-strength concrete as compared to high-strength concrete. However the high-strength concrete showed a little greater shrinkage than medium-strength concrete. The average magnitude of shrinkage strain for low-, medium- and high strength concretes at 60 days of drying were observed as 365, 200 and 266 μ in./ in., respectively.

Hwee and Rangan (1990) conducted experimental investigation on a commercially available high-strength concrete. The average shrinkage strain after about three months of concrete curing was about 450 μ in./ in. By comparing the values of shrinkage strain of high-strength concrete with the values of normal-strength concrete mentioned in Australian Specification AS 3600 - 1988, the estimated final shrinkage strain of high-strength concrete was not significantly different. **Ngab et al. (1981)** have also reported the similar conclusion.

Bloom and Bentur (1995) Studied free and restrained shrinkage of concrete with and without silica fume. The presence of silica fume increased free plastic shrinkage of the concrete and led to earlier cracking than a similar low water-cement ratio concrete, with no silica fume. The sealed low water-binder ratio concrete developed considerable residual stress, but none of them cracked. This stress was higher in silica fume concrete.

2.3.6 Creep

The creep coefficient (Cc) and specific creep (δ_c) can be related through the modulus of elasticity (Ec).

$$Cc = Ec \times \delta_c$$

Ngab et al. (1981) found little difference between the creep of high-strength concrete under drying and sealed conditions. The maximum specific creep was less for high-strength concrete than lower-strength concrete. Similar to the normal-strength concrete, the creep in high-strength concrete is reported to decrease as the age at loading increased and a linear relationship with the applied stress was observed.

Available information on creep of concrete was compiled by **ACI 363R (1984)**. It was reported that the total strain observed in sealed high-strength concrete under a sustained loading of 30% of the ultimate strength was the same as that of lower-strength concrete when expressed as a ratio of short-term strain. Under drying conditions, this ratio was 25% lower than the lower-strength concrete. Increase in water-cement ratio caused the increase in specific creep. High-strength concrete was reported to show less creep than normal-strength concrete when loaded to a given percentage of compressive strength. On the basis of the investigations of Smadi et al. (1982), ACI 363R (1984) indicated that high-strength concrete has a specif creep only about 20% that of lower strength concrete and a creep coefficient about 30% as high.

Smadi et al. (1987) drawn the following conclusions from their investigations:

I) The creep strain, creep coefficient and specific creep, at least up to 60 days, for stresses up to same percentage of ultimate strength, including the overloads, were smaller for high-strength concrete than for medium- and low-strength concretes.

II) The total strain (elastic + shrinkage + creep) for a given stress-strength ratio was greater the higher the strength at early ages after loading; at later ages it may or may not be smaller, depending upon the stress intensity.

III) High-strength concrete had a higher creep-stress proportionality limit than that for low- and medium-strength concretes. The linearity point was found to be about 65% of the ultimate for the former and about 45% for the others.

IV) The stress in high-strength concrete may be increased up to $0.65 fc'$ without causing significant crack formation, provided the total deflection is not excessive.

V) At any given stress-strength ratio and any time after loading, the specific creep is greater the lower the strength of concrete at any time of loading.

Hwee and Rangan (1990) observed that the creep coefficient for higher strength concrete was equal to 1.08, after 50 days of loading, at stress-strength ratio of 0.40. This is significantly smaller than that for normal-strength concrete. Similar findings were also reported by **Ngab et al. (1981)**.

2.3.7 Strength gain with age

Carrasquillo and Nilson et al. (1981) observed higher rate of strength gain by high-strength concrete at early ages as compared to lower strength concrete but at later ages the difference was not significant. They found a typical ratio of 7 day to 95 day strength of 0.60 for low-strength concrete, 0.65 for medium-strength concrete and 0.73 for high-strength concrete.

ACI 363R (1984) reported that high-strength concrete continued to gain strengths above and beyond design requirements with the passage of time, more than lower-strength concretes. It reported the ratio of 7 day to 28 day strengths ratio of 0.80 to 0.90 for high-strength concrete and 0.70 to 0.75 for lower strength concretes.

Hwee and Rangan (1990) reported that strength of commercial high-strength concrete at 28 days was 93% of the nominal strength.

Iravani (1996) reported ratios of 7 day to 28 day and 7day to 91 day compressive strength of 0.85 to 0.88 and 0.75 to 0.79 respectively for high-strength concrete.

2.3.8 Size effect

The size effect in control specimens is very important issue. Because these are used to determine the structural properties, which in turn, governs the design of a structure and can affect the final design of a structure. In general strength decrease with the increase in specimen size.

Carrasquillo and Nilson (1981) found the average ratio of compressive strengths of 152×305 mm to 102×203 mm cylinder for 41 to 83 MPa concrete close to 0.90 regardless of test age.

Nasser and Al-Manseer (1987) reported that ratio of average strength of 75×150 mm cylinder rodded 10, 15 or 25 times in two layers to that 150×300 mm cylinders rodded 25 times in three layers was found to be 103%.

Lessard et al. (1993) investigated the size effect in the concrete strength range from 72 to 126 MPa. The 100×200 mm specimens had a strength which was 105% that of the 150×300 mm specimens. He suggested that any reported results of high-strength concrete should indicate the type and size of the specimen considered.

Iravani (1996) reported the ratio of compressive strength of 150×300 to 100×200 mm cylinders as 0.94. The size of spherical seat of bearing block and stiffness of test machine and the quality of consolidation were observed to affect this ratio.

Chin et al. (1997) reported that when compared with 100×200 mm cylinder strengths, both 150×300 mm cylinders and $100 \times 100 \times 200$ mm prisms yielded about 2% lower strength and 100 mm cubes gave 3% higher strengths.

Sidek Sener (1997) reported the results of failure of cylinders under the uniaxial compression. The failure was ductile if the load remained constant at increasing deformation after the ultimate state reached; whereas in brittle failure, the load decreased after maximum. Due to different amount of energy consumption, the failure of smaller specimens generally were ductile mode (plastic), while for the larger specimens it appeared to be a brittle mode (elastic).

French et al. (1998) indicated that, on average, 100×200 mm cylinders tested 6% higher than companion 150×300 mm cylinders.

2.3.9 Sustained load strength

The design of concrete structures is based on the short-time compressive strength of concrete. Normal-strength concrete loaded at a late age with sustained stress in excess of approximately 70 to 75% of it's short-term strength at the time of loading may fail under sustained stress after a period of several minutes to several months.

Based on the 4 hours sustained load study **Shah and Chandra (1970)** assumed that the sustained load strength of normal-strength concrete was within 70 to 80% of the short-term strength.

Carrasquillo and Slate et al. (1981) reported that high-strength concrete could be loaded to a higher stress-strength ratio without initiating a self propagating mechanism leading to disruptive failure.

According to **Ngab et al. (1981)**, 39 MPa concrete specimens, loaded at the age of 30 days, failed under a sustained stress of 65% of the short-term strength, while 62 MPa high-strength concrete, loaded at the same age, did not fail under 85% of the short-term strength during the 60-day sustained load period.

Iravani and MacGregor (1998) presented in detail the literature review on sustained load studies on concrete. The results of 15 years of sustained load study on concrete with 28 day cylinder strength up to 50 MPa were also reported. It was concluded that the sustained load strength was around 80% of the ultimate, regardless of the compressive strength and eccentricity. They studied compressive strength of 100×200 mm cylinders under high sustained stresses. The sustained load specimens were subjected to sustained stresses for 3 months, if they did not fail earlier. The concrete had 56 day strength of 65 to 120 MPa. Based on the results of their study and previous studies, the following conclusions were drawn:

I) The ratio of sustained load strength to short-term strength of high-strength concrete increased as the compressive strength increased. The sustained load strength was between 70 to 75% of the short-term strength for 65 MPa concrete, 75 to 80% for 95 MPa concrete (without silica fume) and 85 to 90% for 105 MPa concrete (with silica fume) and 85 to 90% for 120 MPa concrete (with silica fume).

II) The sustained load-strength ratio of high-strength concrete under eccentric loads acting at or near Kern point was approximately 5% higher than that under concentric loading.

III) The sustained load-strength ratio of silica fume ultra-high-strength concrete was 10% higher than that of high-strength concrete without silica fume and normal-strength concrete.

IV) If the specimen had the same test parameters as those tested under sustained stresses, the deviation the stress-strain curve from a straight line could be taken as an approximate estimate of the sustained load strength of high-strength concrete.

V) The effect of the composite non-homogeneous structure of high-strength and the creep of the paste were dominant effects under high sustained loads.

2.4 Structural Design Considerations

High-strength concrete has some characteristics and engineering properties that may be different from those of lower-strength concretes. Internal changes resulting from short-term and sustained loads and environmental factors are known to be different. Directly related to these internal differences are distinctions in mechanical properties that must be recognised by design engineers in predicting the performance and safety of structures. These distinctions are increasingly important as strength increases.

Leslie et al. (1976) suggested that difference in various properties of high-strength concrete might affect the characteristic behaviour of high-strength concrete beams. In some cases improvements were seen, and in other cases less satisfactory behaviour was reported. According to **ACI 363R (1984)**, where essentially no information is yet available, such as bond, anchorage and development length, it is conservative to base design on properties of a lower-strength concrete. In areas such as diagonal tension and torsion, design using a lower material strength in the calculation is not necessarily safe because of the differences in failure characteristics of the higher strength materials. These and other questions are reviewed in the following section. Design criterion related to beams and slabs are reviewed in detail as slab-on-girders bridge is the structure of the present study.

2.4.1 Compressive stress distribution

The compressive stress distribution in beams is directly related to the shape of the stress-strain curve in uniaxial compression. Consequently, for high-strength concrete, which displays differences in shape, it is reasonable to expect differences in flexural compressive stress distribution, particularly at loads approaching ultimate. However, the difference in calculated strength values of beams depend on steel ratio and other factors. The various shapes of the stress-strain curves proposed by different researchers are shown in Fig. 4. Most concrete codes do not explicitly cover concrete with strength above 50 to 60 MPa.

Leslie et al. (1976) reported that for concrete with fc' above 55 MPa, a triangular stress block with the extreme fibre stress at f'_c and zero stress at neutral axis was conservative model, as against the ACI 318 - 1971 rectangular block, for predicting the behaviour of beams.

The Indian code **IS: 456 - 1978 (1981)** proposed a Stress-strain curve with fixed value of various stress block parameters i.e. independent of concrete strength.

Wang et al. (1978b) reported that rectangular stress distribution gave sufficiently accurate prediction of ultimate load of reinforced concrete beams with high-strength concrete. The value of maximum compressive strain at ultimate load was always higher than 0.003.

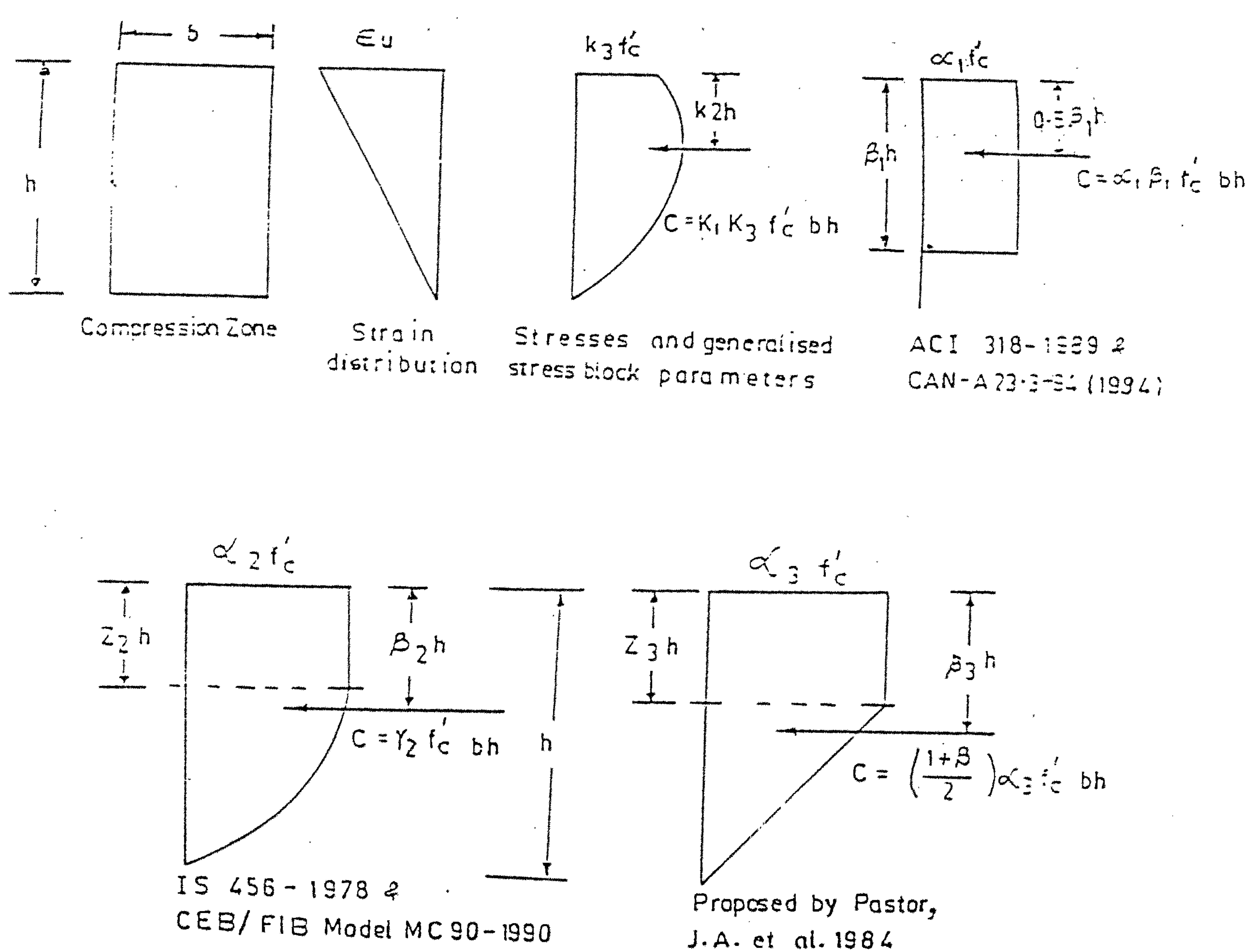


Fig. 4 VARIOUS IDEALISED STRESS-STRAIN CURVES OF CONCRETE

According to **ACI 363R (1984)** there is significant difference in separate values of stress block parameters; k_1, k_2 and k_3 depending on concrete strength. But, for under reinforced beam, the combined effect of variation in stress block parameters are compensative. It was also recommended that for under reinforced beams, the ACI 318 (1983) methods could be used without change, at least for concrete strength (fc') up to 83 MPa (fck = 80 MPa) Test values are best predicted using actual stress-strain curves, but either the rectangular or triangular distribution gave acceptable lower bound to experimental values.

Present **ACI 318 (1989)** rectangular stress block was originally derived by **Mattock (1961)**. Later on it's applicability was increased for high-strength concrete by lowering the limit on β_1 as 0.65 for concrete strength in excess of 55 MPa. The parameter α_1 was assumed to have a constant value of 0.85. The parameter β_1 was equal to 0.85 for concrete strength fc' up to 30 MPa and was reduced continuously at the rate of 0.08 for each 10 MPa of strength in excess of 30 MPa. The parameter β_1 was not taken less than 0.65. The limiting compressive strain is taken as 0.003 and was independent of fc'.

Ibrahim and MacGregor (1996) studied the stress-strain curves and the stress block parameters of high-strength and ultra-high-strength concrete column specimens with shape of the compression zone (rectangular and triangular) as one of the variables. In general, the stress-strain curves of the rectangular specimens and the cylindrical tended to show agreement; the stress-strain curves of the triangular specimens did not. They concluded that it was due to the assumption that stress was a function of the strain only, ignoring the effect of the shape of the cross-section.

According to **ACI Committee 441R-96 (1997)** when smaller value of parameter β_1 was selected, the product $\alpha_1 \times \beta_1$ resulted in conservative estimate of the total compression force in concrete. But it would lead to an over estimate of the lever arm and, hence, to an over estimation of the moment resisted by the compression in concrete. Thus, for under reinforced beams the present ACI 318 - 1989 rectangular stress block parameters would result in conservative estimate of moment of resistance. This is most serious for columns failing in compression and with small eccentricity of axial load.

Ibrahim and MacGregor (1997) discussed in detail the compression stress block from four current codes [Canadian code A23.2 (1994), Norwegian code NS 3473 (1989), Finnish code Rak MK4 (1989) and CEB/FIP Model MC90 (1990)] that do allow design for high-strength concrete sections are reviewed. They proposed modifications in stress block parameters, based on the investigation on high-strength concrete (up to 100 MPa) and ultra-high-strength concrete (above 100 MPa) sections. They reported that the current ACI 318 - 1989 rectangular stress block was not conservative for high-strength concrete columns failing in compression.

Mansur et al. (1997) studied the flexural behaviour of high-strength concrete beams of concrete compressive strength (fc') 60 to 110 MPa. They reported that the stress-strain curves for concrete in compression obtained from the flexural tests were remarkably similar to those generated from uniaxial loaded specimens. He reported that ACI's simplified design method using an equivalent rectangular stress block could be used in the strength design of the high-strength concrete flexural members.

Xie et al. (1997) analytically analysed the rectangular tied columns with concrete strengths of 37.5 to 75 MPa. They found that rectangular stress block in ACI 318 - 1989 overestimated the flexural strength of high-strength concrete columns developing compression failure.

2.4.2 Limiting compressive strain

The lower value of ultimate strain of high-strength concrete reported by some researchers was, probably, due to the energy release from testing machine. The values of maximum strain reported by **Ibrahim and MacGregor (1997)** (0.0033 to 0.0046 for high-strength concrete and 0.0039 to 0.0043 for ultra-high-strength concrete) were considerably higher then the limiting strain values of 0.003 (in ACI 318-1989) and 0.0035 (in IS 456-1978).

2.4.3 Minimum tension reinforcement

An upper limit on the tensile steel ratio for beams (always less than balanced ratio) is set to ensure a gradual failure due to yielding of steel. A lower limit of tensile steel ratio is set to guard against sudden failure of very lightly reinforced beams upon concrete cracking, when the tension formerly carried by the concrete is transferred to the steel reinforcement. Minimum tension reinforcement requirement in post-tensioned (bonded) girders is reviewed here in detail as it is one of the structural elements of the structure of the present study.

As per **IRC: 18 - 1985 (1997)**, minimum longitudinal reinforcement in beams shall not be less than 0.25% and 0.15% of the gross area of the section for mild steel and HYSD bars respectively, when fck is not more than 45 MPa. For Concrete strength greater than 45 MPa, the provision shall be increased to 0.3% and 0.18% respectively.

According to **Clarke (1987)** the minimum areas of tension reinforcement was based on the ground of steel being used (0.15% for S 460 & 0.25% for S 250, based on the effective depth of the section). The principle behind the adoption of these values was that tensile capacity of the steel should exceed the tensile capacity of the concrete when it cracked. Thus the minimum area provided was recommended to increase with increasing tensile capacity of concrete i.e. with the square root of the cube compressive strength of the concrete. On the basis that 40 MPa was the maximum grade of reinforced concrete, it was suggested that quoted minimum area of tension reinforcement should be multiplied by $\sqrt{\frac{fck}{40}}$ when high strength concrete is used (where concrete strength was in MPa).

According to **ACI 318 (1989)** the total amount of prestressed and non-prestressed reinforcement shall be adequate to develop an ultimate load in flexure at least 1.2 times the cracking load calculated on the bases of modulus of rupture of $0.6228\sqrt{fc'}$ MPa. Similar recommendations were made by **Ghosh (1987)**.

Freyermuth and Alami (1997) reviewed the CEB/FIP Model MC90 (1990) and ACI 318 (1995). The CEB/FIP Model MC90 (1990) was reported to provide uniform provisions for the minimum area of longitudinal reinforcement for reinforced and prestressed concrete members. It also suggested that minimum reinforcement requirements for beams should also be applicable to slabs. Freyermuth and Alami (1997) suggested that 0.0015 $b_t d$ (b_t is the average width of the concrete zone in tension) proposed by CEB/FIP Model MC90 (1990) be replaced by $0.002 b_w d$ (b_w is the web width). The parameter b_w was selected in preference to CEB/FIP parameter b_t to simplify the calculation process. Concrete strength was included, by dividing 0.002 by the square root of 4000 psi and $\sqrt{fc'}$ was added to the equation giving

$(As)\min = 0.000381 bwd\sqrt{fc'}$ where fc' is in MPa.

2.4.4 Shear design

AASHTO (1977) specifications recommends that the shear design may be done according to ACI 318 specifications but the minimum shear reinforcement recommended by it, which is double of that recommended by ACI 318, must be provided.

IS: 456 - 1978 (1981), ACI 318 (1989), and various other standards assumed that flexure and shear could be handled separately for the worst combination of flexure and shear at a given section. The interaction between the two was addressed indirectly by detailing rules for flexural reinforcements cut-off points.

According to **Carrasquillo and Nilson et al. (1981)** and **Carrasquillo and Slate et al. (1981)** high strength concrete loaded in axial compression fractured suddenly and, in so doing, resulted in a failure surface that was smooth. This was in contrast to the rugged failure surface, characteristic of the lower-strength concrete.

Clarke (1983) analysed the available experimental results on shear capacity of high-strength concrete beams with and without shear reinforcement. He reported the steady gain in strength with increasing concrete strength.

ACI committee 363R (1984) suggested that the shear strength deficiency might be produced in high-strength concrete due to reduced aggregate interlock on smooth surface resulted due to diagonal tension cracks. This shear strength deficiency was not considered by most of the present codes.

Mphonde and Frantz (1984) investigated the shear behaviour of high-strength concrete beams without shear reinforcement. The variables were concrete strength (21 to 103 MPa) shear-span to depth ratio (3.6, 2.5 or 1.5). Test results indicated that for the slender beams the ACI 318 (1977) (same as ACI 318-1989) were conservative, but their accuracy varied greatly with concrete strength. The effect of concrete strength on shear capacity became more significant as the beam became shorter (shear-span to depth ratio decreases) and the ACI equations underestimated the actual shear strength by 71% at high concrete strengths. A new regression equation was presented to more accurately predict ultimate shear capacity of slender beams over the entire range of concrete strength tested.

IRC: 18 - 1985 (1997) suggests that at any section the ultimate shear resistance of the concrete alone (Vc), shall be considered for the sections both uncracked (web shear cracks) and cracked in flexure, and lesser value be taken. The splitting tensile and modulus of rupture strength of concrete are taken as $0.24\sqrt{fck}$ and $0.37\sqrt{fck}$ respectively.

IRC: 21 -1987 (1997) put a limit on maximum permissible shear stress as $0.07\,fck$ or 2.5 MPa. Web reinforcements are recommended to be designed for full shear force without considering the shear resisted by concrete.

Kaufman and Ramirez (1988) studied the ultimate behaviour of the high-strength concrete prestressed I-beams with truss as fundamental behaviour model. Three modes of failure were observed; flexural, web crushing and shear tension. ACI 318 (1989) and AASHTO (1977) provisions for shear was found to be cumbersome and obscure when applied to continuous structure with moving loads. This complexity resulted from the highly empirical approach followed in such procedures. Most of the codes ACI 318 (1989), AASHTO (1977) and IRC: 18 - 1985 (1997) etc. evaluate the shear capacity of beams using a sectional approach. This deficiency was reported to be overcome by the authors by using truss model based on the overall beam behaviour to the applied load, representing the distribution of internal forces at failure. The higher concrete compressive strength was reported to increase the load carrying capacity of the diagonal truss members, resulting in increased shear strength but adequate detailing of the member was also stressed.

Kotsovos (1988) observed that shear resistance appeared to be associated with the region of the path along which the compression force was transmitted to support and not, as widely considered, the region of the beam below neutral axis i.e. the aggregate interlock.

Bazant and Kazemi (1991) reported that for ultimate load there was a strong size effect, while for first diagonal crack initiation load the size effect was small or negligible. Imposition of certain margin of safety against the crack initiation load was reported not to provide a uniform margin of safety against the ultimate load. Consequently, it was recommended that a requirement based on the ultimate load had to be introduced into design codes which meant that the size effect had to be considered.

According to **ACI Committee 421 (1992)** the nominal shear strength $(Vn = Vc + Vs)$, of slabs resisted by concrete and steel could be taken as high as $0.664\sqrt{fc'}$ instead of $0.498\sqrt{fc'}$ and Vc equal to $0.166\sqrt{fc'}$, where concrete compressive strength was in MPa.

Koenig and Grimm (1993) did the finite element analysis of shear behaviour of longitudinally reinforced high-strength concrete members. They reported that the tensile behaviour of concrete was the most important factor concerning the influence of the shear behaviour. The influence of the aggregate interlock and dowel action of the reinforcement was very small.

Chung and Ahmad (1994) observed that concrete tensile strength ft' affected the pre-diagonal tension cracking and post-diagonal tension cracking stiffness of shear critical reinforced concrete beams. The concrete tensile strength also had a significant effect on prediction of ultimate load carrying capacity of shear critical reinforced concrete beams. For beams a value of $4\sqrt{fc'}$ was reported to be an adequate estimate of ft'

Vecchio et al. (1994) reported that in general, the compression softening formulation and analysis procedure (for shear) for normal-strength concrete was equally applicable to high-strength concrete.

Ozcebe et al. (1995) disussed shear design provisions of ACI 318M (1995). It proposed the equations similar to those proposed by IRC: 18 - 1985 (1997). In comparison to IRC: 18 - 1985 (1997) equations, the equations proposed by ACI 318 (1989) are less conservative. Unlike IRC: 18 - 1985 (1997), ACI 318 (1989) makes a difference in composite and non-composite sections.

Collins et al. (1996) observed that shear carried by tensile stresses in the concrete (Vc) was a function of longitudinal strains (ε_x) in web of the member. As ε_x increased, Vc decreased. Increase in magnitude of moment or application of axial tension was reported to decrease Vc. Application of prestressing or the increase in the area of longitudinal reinforcement resulted in decrease in ε_x and hence increase in Vc.

Due to the different mechanical properties of high-strength concrete, **Kim and Park (1996)** stressed the need of the prediction model to reliably estimate the shear strength of beams made with high-strength concrete.

Yoon et al. (1996) reviewed shear design recommendations of various codes. The Canadian standard CSA A23.3 (1994) took into account the size of a member in calculation of concrete contribution in resisting shear (V_c). For section with effective depth greater than 300 mm, the V_c is recommended to be calculated by the expression given below.

$$V_c = \left(\frac{220}{1000+d}\right) b_w d \sqrt{f_c'} \qquad \text{(N, mm units)}$$

According to **ASCE-ACI Committee 445 (1998)**, "design procedure proposed for regulatory standards should be safe, correct in concept, and simple to understand, and should not necessarily add to either design or construction costs. The primary shortcomings of ACI 318 (1995) (and in most of the standards) were the many empirical equations and rules for special cases, and particularly the lack of a clear model that could be extrapolated to cases not directly covered. The committee discussed various new approaches for the shear design of the structural concrete which would provide a unified, intelligible, and safe design framework for proportioning structural concrete under combined load effects.

Marti (1999) reviewed the recent development of strut-and-tie model, compression field and limit analysis approaches. While it was emphasized that strut-and-tie models and stress field considerations should be part of tool box of every designer, it was pointed out that deformation and failure mechanism considerations were particularly useful for the evaluation of the existing structure.

Rebeiz (1999) critically reviewed the shear equations, suggested by ACI 318-1995 (same as ACI 318-1989), for beams without shear reinforcement on the basis of the experimental data on beams with concrete compressive strength up to 104 MPa. He reported that ACI shear equations were mostly valid for normal-strength concrete and did not apply to high-strength concrete.

Shahawy and Cai (1999) proposed a new tied-arch model for the shear design of pre-tensioned prestressed concrete members. The proposed model is strongly recommended in the design and capacity rating of deep beams but is also useful for regular members.

2.4.5 Minimum shear reinforcement

In general, all the design codes require provision of minimum shear reinforcement for beams. Most of the expressions given in codes for minimum shear reinforcement are empirical in nature and are not based on well-established, accepted criterion. Therefore, it becomes must to revise the requirements whenever additional new information become available.

It is generally agreed that reinforced beams should have adequate shear reinforcement to prevent sudden and brittle failure after formation of the diagonal crack, and also to keep the crack width at an acceptable level. However, there is no established quantitative criterion for reserve strength required beyond cracking strength and limits for the crack width.

AASHTO (1977) requires that the minimum reinforcement must be capable of resisting a shear stress of 0.689 MPa.

While, **IS: 456 - 1978 (1981), IRC: 18 - 1985 (1997)** and **ACI 318 (1989)** require minimum shear reinforcement when shear force V is greater than 0.5 times the shear resistance offered by the concrete Vc, some other codes, such as the Turkish code TS 500 (1983), require minimum shear reinforcement regardless of the level of design shear. All the above codes specify the minimum shear reinforcement as a function of the yield strength of the shear reinforcement only. Concrete strength is not included in the equations given for minimum shear reinforcement. The use of high-strength concrete raised some doubts on the validity of the equations given for minimum shear reinforcement, since these equations were based on tests of beams made of normal-strength concrete.

IRC: 18 - 1985 (1997) also requires the minimum shear reinforcement corresponding to a shear strength of 0.4 MPa when web steel is stressed to 87% of it's yield strength. Thus all the shortcomings of the ACI 318 (1983) will also be encountered if design is made according to IRC: 18-1985 (1997) and high-strength concrete is used.

Recent tests by **Elzanaty et al. (1986a), Shuaib et al. (1986)**, and **Roller & Russel (1990)**, on beams with high-strength concrete indicated that reserve strength beyond diagonal cracking strength decreased as concrete strength increased.

According to **Clarke (1987)**, for interface shear at least, long term capacity is roughly half that of the short term capacity, and hence he recommended to replace the figure of 0.4 MPa (0.345 MPa in ACI 318-1989) by half the values of the ultimate shear stress for the concrete.

ACI 318 (1989) included some revision to correct the minimum shear reinforcement requirement for concrete strength higher than 69 MPa. The revision states, "Values of $\sqrt{fc'}$ greater than 8.3 MPa shall be permitted in calculating Vc, Vci and Vcw for reinforced or prestressed concrete beams and concrete joists construction having minimum web reinforcement equal to $\frac{fc'}{34.5}$ (MPa) times, but not more than three times the amount permitted by ACI 318 (1983)."

Investigations by **Johnsons & Ramirez (1989)** also showed that beams having the minimum shear reinforcement required by ACI 318 (1983) had limited reserve strength when higher strength concrete (fc' $\rangle$ 69 MPa) was used.

Roller and Russel (1990) investigated the beams using high-strength concrete (69, 117 & 124 MPa) and designed according to ACI 318 (1983). They reported the failure of some of the specimens in shear at a strength that was not only less than the calculated V, but also less than the calculated Vc. This indicated that the code provisions might overestimated the Vc term for high-strength concrete beams containing the minimum required amount of web reinforcement.

Krauthammer (1992) questioned the use of 0.345 MPa by the ACI 318 (1989) as the shear stress which must be resisted by the minimum shear reinforcement provided. He found no reason for the above recommendation. Inspired by the fact that major part of the shear resistance offered by the beam without web reinforcement (33 to 50%) was offered by the interface resistance **Park and Paulay 1975**), he developed design recommendations for minimum shear reinforcement that could ensure full shear transfer across the cracks by aggregate interlocks. He proposed that the shear to be resisted by minimum shear reinforcement should be 0.448 MPa instead of 0.345 MPa.

Yoon et al. (1996) studied the minimum shear reinforcement requirements for low, medium and high-strength concrete beams. They critically reviewed ACI 318 (1983) and Canadian standard CSA A23.3 (1994). According to ACI 318 (1983), where shear reinforcement was required for non prestressed members subjected to shear and flexure only, the minimum nominal shear stress provided by shear reinforcement (V_s/b_wd) should be 0.345 MPa. Thus members of high-strength concrete with minimum shear reinforcement according to ACI 318 (1983) resulted in shear response without adequate reserve of strength after cracking. On the other hand, if designed according to CSA A23.3 (1994) adequate reserve strength after cracking was observed. They also concluded that provision of an appropriate amount of minimum shear reinforcement also helped to control bond splitting cracks that could otherwise lead to brittle shear-bond failure.

Ozcebe et al. (1999) discussed in detail the minimum shear reinforcement provisions in Turkish code TS 500 (1983) and Canadian code CSA A23.3 (1994). The equation given in TS 500 (1983) for minimum shear reinforcement was derived by equating the diagonal cracking strength of the beam to the shear strength of the same beam with shear reinforcement. The diagonal cracking strength was magnified by a factor of 1.5. Since the minimum shear reinforcement included the tensile strength of concrete, no additional revision was made for high-strength concrete. The revised expression formulating the minimum shear requirements in CSA A23.3-(1994) is reported to be same as the one given in TS 500-1983, except a smaller constant was used. They investigated the minimum shear reinforcement requirement in beams of high-strength concrete (60 to 80 MPa) subjected to flexure and shear. They proposed an expression for minimum shear reinforcement on the same basis as the one adopted by TS 500 (1983), except that a smaller magnification factor (1.3) was used. They reported that their proposed equation, TS 500 (1983) code provisions and ACI 318 (1989) provisions were safe as far as crack width and reserve shear capacity were concerned.

2.4.6 Ductility

High-strength concrete is a far more brittle material than the normal-strength concrete. As a result, **Leslie et al. (1976)** expressed doubts about the ductility inherent in members made of high-strength concrete.

Wang et al. (1978b) reported that for an identical reinforcement, the ductility ratio of reinforced concrete beams increased with the increasing compressive strength of concrete.

ACI 363R (1984) reported that (for beams with no compression and no confinement steel) for heavily reinforced beams deflection ductility decreased as concrete strength increased. It was also reported that increase in compression steel and confinement steel enhanced the ductility but no definite trend was reported.

Kaufman and Ramirez (1988) reported that in prestressed concrete I-beams adequate ductility could be achieved in high-strength beams by proper detailing. Deflection ductility was reported to reduce with the increase in reinforcement index.

Shin et al. (1989) observed that for any combination of main reinforcement and lateral reinforcement, a member with high-strength concrete suffered less deflection than a similar member with low-strength concrete at the same load level up to the stage of yielding. The ductility of high-strength concrete was reported to be generally higher than those of beams of normal-strength concrete. For the same strength of concrete the ductility index was observed to decrease as the steel ratio increased.

Pendyala et al. (1996) reported that high-strength concrete flexural member exhibited a higher plastic ductility when compared to normal-strength concrete, but might exhibit comparable ductility if softening was taken in to account. They reported the review of the investigation by Attard and Mendis (1993). It was reported that structure could continue to carry additional loads even after some of the plastic hinges in members had gone into softening. The following reasons were summarised for brittle behaviour of high-strength concrete:

I) Abrupt drop of descending branch of the stress-strain curve (softening part) after the peak has reached.

II) Smooth fractures across micro cracks and the lack of aggregate interlock.

Alca et al. (1997) reported that the beams having concrete strength around 90 MPa were as deformable as the beams having compressive strength around 50 MPa. All the beams deformed more than predicted by ACI 318 code.

Mansur et al. (1997) indicated that in over-reinforced concrete beams both ultimate strength and ductility could be enhanced by increasing the concrete strength.

Sarkar et al. (1997) reported that the ductility index for a specified reinforcement ratio in flexure increased with the increase in compressive strength of concrete.

Marti (1999) reported that increase in concrete strength decreased the mechanical reinforcement ratio (reinforcement ratio $\times$ fy/fc)and resulted in increased flexural ductility index.

2.4.7 Permissible stresses in post tensioning

The detail of the permissible stresses in post tensioning as adopted by various investigators and recommended by different design codes are summarised in table 3.

TABLE 3 PERMISSIBLE STRESSES IN POST TENSIONING

	Permissible stress during transfer (MPa)		permissible stress during service (MPa)	
	compression	tension	compression	tension
AASHTO 1977	$0.55\,f'_{cj}$	not allowed	$0.40\,fc'$	$0.498\sqrt{fc'}$
Jobse & Moustafa 1984	$0.55\,f'_{cj}$	not allowed	$0.40\,fc'$	$0.623\sqrt{fc'}$ -0.731
IRC 18 - 1985 (1997)	$0.45\,fckj \leq 20$	$0.045\,fckj \leq 2$	$0.33\,fck$	not allowed
ACI 318 - 1989	$0.60\,f'_{cj}$	$0.5\sqrt{f'_{cj}}$ at simple sup $0.25\sqrt{f'_{cj}}$ at other place	$0.45\,fc'$	$0.5\sqrt{fc'}$ uncracked sec. $\sqrt{fc'}$ cracked section
Hassanian, & Loov, (1999)	$0.60\,f'_{cj}$	$0.5\sqrt{f'_{cj}}$	$0.45\,fc'$	$0.5\sqrt{fc'}$

The Indian standards **IS: 1343 - 1980 (1981)** defines three types of prestressed concrete structures while discussing the cracking as limit state of serviceability. It allows no tensile stress for type 1 structures. In type 2 structures where no visible cracks are allowed, the permissible tensile stress during service loads is not allowed to exceed 3.0 MPa. However, where part of the service loads is temporary in nature, this value may exceed by 1.5 MPa, provided under the permanent component of the service load the stresses remains compressive. For type 3 members in which cracking is permitted, the permissible tensile stresses is related to the maximum crack width allowed. For limiting crack width of 0.1mm and 0.2mm the permissible tensile stresses varies from 4.1 to 4.8 MPa and 5.0 to 5.8 MPa respectively depending upon the concrete grades

2.5 Mix Proportioning of High-strength Concrete

One of the aims of the present investigation is to study the economic consideration of using high-strength concrete in highway bridges. Hence literature on mix proportioning of high-strength concrete has been reviewed with a purpose of cost comparison of various grades of concrete.

Ting and Patnaikuni (1992) studied the concrete of compressive strength 100 to 150 MPa. He suggested a binder content of 560 Kg. per cubic metre of concrete with an optimum water-binder ratio of 0.23. Maximum size of coarse aggregate used was 7 to 14 mm. A condensed silica fume dosage of 10% was recommended.

ACI 211 (1993) recommended the maximum size of aggregate between 12 to 18 mm. Volume of coarse aggregate was 30% of the total volume of aggregates for 12 mm maximum size of coarse aggregate. Water-binder ratio recommended was from 0.26 to 0.41 without high-range water reducers and 0.26 to 0.48 with high-range water reducers. The fly-ash (of class c) replaced the cement content by 20 to 35% of the total weight of cementitious materials. Dosage of superplasticizers was 260 to 520 ml per 50 Kg. of cement. Water content recommended was 165 Kg. per cubic metre of concrete for a slump of 25 to 75 mm. Maximum amount of cementitious materials suggested was 590 Kg. per cubic metre of concrete.

The **MOST (1995)** specifications (Ministry of Surface Transport, India) suggested a minimum cement content of 400 Kg. per cubic metre of concrete with a maximum limit of 540 Kg. per cubic metre of concrete. Maximum water-cement ratio recommended is 0.45 and 0.40 for reinforced cement concrete and prestressed concrete respectively.

According to **Gutierrez and Canovas (1996)** "there is not a method available for mix proportioning of high-strength concrete as simple as those developed for normal concrete. This situation obliges one to consult the existing bibliography looking for mix proportioning used with success in other works as a starting point, completed with additional laboratory work to adjust the final mix". They also proposed a method for producing a concrete of 28 day compressive strength up to 110 MPa. Water-binder ratio was varied from 0.27 to 0.305. Cement content was varied from 400 to 500 Kg. per m^3. The range of use of silica fume content was varied from zero to 15% of the weight of cement. Fine aggregate used was river sand. Various coarse aggregates with maximum size of 10 to 14 mm were used. About 3% superplasticizer (sulphonated naphthalene-formaldehyde condensate in liquid form with 64% not combined water) was added. They also suggested that silica fume should not be added with water-cement ratio above 0.40 as this mineral admixture was only economically advantageous for high-strength concrete. Different relations between concrete strength and water-cement ratio were suggested corresponding to different percentage of silica fume added.

Cetin and Carrasquillo (1998) reported that the coarse aggregate beyond 40% of the concrete volume reduced the compressive strength. Optimum aggregate content was somewhere 36 and 40% when 167 Kg. of water, 597 Kg. of cement per cubic metre of concrete were used. The water-cement ratio used was 0.28.

2.6 Use of High-strength Concrete in Structures

High-strength concrete has been made and used by prestressed concrete producers for many years. However, the producers used the high early strength of concrete so that the formwork might be removed, the concrete members prestressed, and the forms reused every day without taking advantage of the high strength of concrete in the structural design.

Because the concrete is usually designed to resist compressive forces, it is clear that the application of high-strength concrete will be most advantageous in columns and other compression members, arches, rigid frames for tunnels etc. **ACI Committee 363R (1984)** summarises the use of high-strength concrete in areas like buildings, bridges, offshore structures, precast and prestressed structure, concrete piles and highway etc.

There have been many applications of high-strength concrete in bridge girders. However published information on actual structures is limited. **ACI Committee 363R (1984)** listed some existing bridges in which the use of high-strength concrete had been reported. Much of the research work has been directed towards the cast-in-place applications. Very little information is available regarding the production and behaviour of high-strength concrete for precast prestressed applications. Very little work has been done in optimizing the precast, prestressed concrete slab-on -girders highway bridges.

Jacques (1971) reported the study on long span prestressed concrete bridge girders. Nine standard girder sections were considered in the study. The strength of cast-in-place deck was kept constant. The thickness of deck slab was assumed constant for a given range of girder spacing. Concrete strengths in girder were (fc') 42 MPa and 49 MPa. Cost study is reported on the basis of total bridge cost for various girder spacing and spans. Prevalent AASHTO specifications were used for design. Diaphragms at 12 m spacing with depth equal to one-third of girder depth were used. Bridge girder design charts for various chosen girder sections were presented. Girder spacing was varied from 0.9 to 3.7m. No comment was made regarding the maximum permissible deflection. It was reported that if the section was not properly balanced (i.e.bottom flange had relatively more concrete in comparison to top flange) than a 9% increase in depth resulted only 2 to 3% increase in span capability. By merely reducing the concrete area in bottom flange in comparison to top flange area, the span capability was reported to increase by 3m.

For composite bridge deck superstructure **Rabbat and Russel (1982)** reported a 15% increase in span capability with an increase in girder concrete compressive strength from 35 to 48 MPa.

Jobse and Moustafa (1984) discussed the advantages of using high-strength concrete in bridges. They reported that increased tensile strength increased the permissible stress range at service load design in prestressed concrete.

They investigated the potential use of high-strength concrete in highway bridges. Four different solid girder sections were selected for the purpose. Three concrete grades with compressive strength of 42, 56 and 70 MPa were considered. Allowable stresses considered were conformed to AASHTO specifications. It was reported that high-strength concrete could be beneficial if the design was governed by serviceability limit states rather than by ultimate capacity. Increase in span capabilities was reported with the increase in concrete compressive strength. Reduction in number of girders from 9 to 4 was reported if 72 MPa concrete was used in place of 42 MPa concrete. The allowable stress in tension at transfer was proposed to be revised in such a way so that a constant margin of safety against flexural cracking in prestressed girder could be obtained.

One of the most common type of superstructure construction used in bridges is the composite, prestressed girders and reinforced concrete deck bridge. For this type of bridge construction **Carpenter (1980)** and **Shah & Ahmad (1985)** have shown that an increase in concrete strength and stiffness could result in increased span lengths and/or longer transverse girder spacing for bridge with high-strength concrete girders.

Adelman and Cousin (1990) investigated T-Beam bridge deck system for cost effectiveness if high-strength concrete is used in prestressed pretensioned bridge girder design. Seven AASHTO girders were selected and five transverse girder spacings (3.4, 2.7, 2.2, 1.9 and 1.7m) were chosen. Designs were optimized by increasing the girder spacing until the service load stresses in the exterior girder were nearly equal to those in the interior girder. The simple supported girders were designed in accordance with AASHTO (1977). No comment was made regarding the number and spacing of cross beams and ultimate load design of girders. An increase in concrete design compressive strength from 41 to 69 MPa, resulted in on average 10% increase in span capability of prestressed girders used in routine bridge design.

Lounis and Cohn (1993) defined the optimal bridge superstructure as "one of minimum total cost, using standard girder sections and traffic loading". They identified three level of optimization and indicated that overall economic impact increased with higher levels. Component Optimization involved optimization of dimensions of the girder and deck slab, the prestressed and non prestressed reinforcement, and tendon profile. Naaman (1972) and Geren & Tadros (1994) reported such type of optimization. Configuration optimization is concerned with finding the optimal combination of longitudinal and transverse components within a given bridge. Less work has been reported on this type of optimization. It was reported that Torres (1966) undertook such optimization. System optimization involved optimization of the overall bridge structural system, including materials, structural type and configuration as well as component sizes. This is most complex optimal design problem and very few attempts to solve it are known. They reported such optimization.

According to **Russel (1994)**, if high-strength concrete is used longer spans could be achieved by using shallower sections.

According to **Roller et al. (1995)** lower creep and shrinkage of high-performance concrete reduced the prestressing losses.

Goodspeed and Vanikar (1996) reported the studies of FHWA on the application of high-strength concrete in highway bridges. It was reported that a significant improvement in concrete durability resulted from an increase in strength which was not being regularly specified for highway structures. That high-performance concrete was not being specified more frequently might be because engineers did not have confidence that high-strength concrete was more durable, that it could be reliably achieved in the field, that the higher strength could not always be used, or combinations thereof. Table 4 relates the recommended performance of high-strength concrete to various exposure conditions of highway structures.

Ma et al. (1997) have shown that variation in overall cost resulting from variation of shear reinforcement required by different shear design methods was negligible.

French et al. (1998) conducted a parametric study to determine the effect of increased concrete strength on maximum achievable girder span lengths and spacing for a series of prestressed I-girder sections. Four grade of concrete (48, 69, 83 and 103 MPa) and three girder spacings (1.22, 2.13 and 3.05m) were considered for a 15.8m wide bridge with a constant deck slab thickness as 230mm. It was reported that the use of higher strength concrete might cause slight reduction in prestressing force because of the increase in allowable stresses. The compressive strength at release near the hold-down points tend to control the wider spaced girder whereas 28 day nominal strength tend to control

TABLE 4 RECOMMENDATIONS FOR THE APPLICATION OF HPC GRADES

EXPOSURE CONDITIONS	Recommended **HPC** Grade for Given Exposure Condition			
	N/A*	Grade 1	Grade 2	Grade 3
Freez/Thaw Durability Exposure (x= F/T cycles per year)	x< 3	3 ≤ x < 50	50 ≤ x	
Scaling Resistance Applied Salt (x= tons/lane-mile-year)	x< 5.0	5.0 ≤ x		
Abrasion Resistance (x= average daily traffic, studded tires allowed)	No studs/chains	x ≤ 50,000	50,000 ≤ x < 100,000	100,000 ≤ x
Chloride Permeability Applied Salt (x= tons/lane-mile-year)	x< 1	1.0≤ x < 3.0	3.0 ≤ x < 6.0	6.0 ≤ x

* N/A stands for "not applicable" and indicates a situation in which specification of an **HPC** performance grade is unnecessary.

the narrower spaced girders. An increase of 16% in the maximum span length was reported when high-strength concrete (69 MPa) was used in place of normal strength concrete (48 MPa). A further gain of 25% in maximum span capability of the girder (with 83 MPa concrete) was reported when higher strength concrete was used in deck slab also (deck slab thickness was kept constant). Doubts were raised regarding the extension of ACI 318 (1995) & AASHTO (1992) provisions for high-strength concrete prestressed bridge girders as these were based on empirical relationships developed from isolated tests of specimens with compressive strength not exceeding 55 MPa. No comment was made regarding the ultimate shear strength and maximum allowable deflection as the design criteria.

Cao and Shing (1999) observed that the design formula suggested by AASHTO (1977) specifications overestimated the negative bending moment in a slab-girders deck. In general, a higher girder flexibility was reported to result in a larger differential deflection of the girders, and consequently, a smaller negative bending moment in the deck. To take advantage of girder deflection, an analytical solution was suggested. Based on the analysis of a four span bridge, the maximum negative bending moment in the deck obtained with the proposed method was as low as 42% of that obtained with the AASHTO (1977) specifications. Hence, the proposed method was reported to lead to a significant reduction of the top reinforcing steel in bridge decks but the girder deflection increased the positive bending moment in the decks. These findings become much more important if high-strength concrete is used in girders as it will increase the span capability and hence the flexibility of the girders.

Hassanian and Loov (1999) studied the optimal design of continuous precast, prestressed slab-on-girders bridge. It included aspects of all three levels. Bridge design conformed to Ontario Highway Bridge Design Code (OHBDC) provisions. Five standard I-girders were studied. Design variables included were: prestressing force, eccentricities at girder mid span and at ends (the tendons were draped at third points of the span), non prestressed reinforcement in the slab and the girders, girder concrete compressive strength at 28 day age (40 to 120 MPa) and the deck slab thickness (minimum deck slab thickness was 225mm and the deck slab strength was fixed at 35 MPa. Form work cost was not included in the cost function. The various costs were normalized with respect to the cost of 40 MPa concrete. For formulation of problem, only flexural constraints at transfer, during construction, at service and at ultimate were considered, in addition to practical constraints which included the limits for design variables. Shear and deflection conditions were not considered. Design was optimized by adjusting the girder spacing until the service load stresses in the exterior girder were equal to those in the interior. The range of span adopted in the study was from 20 to 52m. The incremental increase of span was chosen as 2m. A prestress loss of 20% was assumed in the design. A loss of 25% resulted in the cost increase of about 1.5% whereas a loss of 15% caused the reduction in the cost of superstructure by less than 1%. A marginal difference in the cost were reported when the ratio of the concrete compressive strength at transfer to 28 day strength was varied over a range of possible values (i.e. 0.60 to 0.80). The strength of deck concrete was assumed to have no effect on the girder design. It was reported that the limits on compression at transfer and tension at service were active. The tension at transfer constraint was active only for short spans. The limit on compressive stress at service was not active for any of the girders. A shallower girder with higher strength concrete was reported to be more economical than a deeper girder with lower strength concrete. The study proved that for a given span the use of fewer high-strength girders was more economical than a larger number of normal-strength concrete girders.

Picard and Massicotte (1999) examined the serviceability limit states in various codes like Canadian code CSA S6 (1988), AASHTO (1992) and French prestressed design code B.E.P.L. (1991). Except B.E.P.L. (1991), in all these standards(and in IRC 18-1985) there were no provisions to limit shear stress under service loads even though flexural stresses were limited.They advocated of the limit on shear stress under service loads in prestressed girder.

2.7 Conclusions

On the bases of the literature reviewed the following conclusions can be drawn:

I) High-strength concrete may be defined as concrete with 28 day strength of 60 to 100 MPa.

II) Most of the existing concrete codes are based on experimental and engineering practice on members and structures fabricated of normal-strength concrete. The mechanical properties of high-strength concrete are very much different from those of normal-strength concrete. Thus, existing IRC codes provisions are needed to be critically examined in reference to the use of high-strength concrete.

III) Most of the published literature on both prestressed and reinforced high-strength concrete bridge girders were aimed to verify the ACI and AASHTO recommendations. Necessary revisions are to be made in IRC 18 (1985) and IRC 21 (1987) for adopting high-strength concrete in bridge design.

IV) It appears there is a lack of published research work on the optimization oriented design of post-tensioned composite slab-on-girders bridges as per various IRC codes.

V) It seems there is lack of detailed design aids for preliminary dimensioning of bridges. Thus, design aides in either tabular form or in graphical form are needed for preliminary design of highway bridges.

VI) Simplified equivalent loadings to various IRC standard loads are not available. To make the analysis and design of bridge girders simple, equivalent UDL of various IRC loads are required.

3. ANALYSIS AND FORMULATION

3.1 Introduction

The traditional process of design in structural engineering, and particularly in bridge engineering, relied on the designer's experience, intuition and ingenuity. Many excellent structures were built on this basis though it is not a rigorous approach of design. Moreover, it is laborious and time consuming process and often leads to sub-optimal designs.

The problem is, therefore, proposed to be formulated as an optimal design problem as opposed to the conventional trial and error approach. Computer oriented methods of structural analysis combined with design optimization techniques have been used to arrive at an optimization system to perform extensive economic studies on this type of bridge.

From the literature review on optimal design of bridges, it is identified that there are three levels of optimization and that the overall economic impact increases with higher levels. Level 1 involves the optimization of dimensions of the longitudinal girder and deck slab, reinforcement and tendon profile. Level 2 is concerned with finding the optimal combination of the longitudinal and transverse components for a given bridge. Level 3 involves the optimization of the overall bridge structural system, including materials, structural type and configuration, as well as the component sizes. The optimization in the present study includes various aspects of all these three levels.

3.2 Method of Analysis

Certain standard cross-sectional shapes have evolved over the years for short- and medium-span highway bridge girders. Members having these standard shapes can be mass produced by precasting plants, often using long line methods and permanent reusable metal forms. Great cost savings are possible, compared with construction that requires special formwork either in a precasting plant or at the construction site. Consequently, standard sections are preferred, even though properties may not be optimum for a particular set of design constraints.

The members are analysed and designed on the basis of flexural design according to allowable stresses. The cross-section, prestressing forces and cable profiles are checked to ensure that the specified limiting stresses are not exceeded as the longitudinal girder section passes from the unloaded stage to the service load stage. Both concrete and steel are considered elastic in this range and the full cross-section of the longitudinal girder is assumed effective. Finally the section is checked for the service deflection and the ultimate strength in flexure and shear.

This approach is reasonable considering that one of the most important objective of prestressing is to improve the service load performance. Furthermore, it is usually the performance criteria at the service load level that determines the amount of prestress force to be used, although the requirements of strength may determine the total tensile steel area.

3.2.1 Assumptions

The analysis and design of the slab-on-girder bridges of the present investigation is based on the following structural design assumptions:

1. All the designs conform to IRC: 5 - 1985 (1985), IRC: 6 - 1966 (1990), IRC: 18 -1985 (1997), IRC: 21 - 1987 (1997), IRC: 22 - 1966 (1966) and IRC: 22 - 1986 (1991) provisions except where otherwise noted i.e. where proposed modifications are used.
2. A 75mm thick M35 grade concrete wearing surface has been proposed over the concrete deck.
3. Steel railings (weighing 35 Kg. per metre length) are provided.
4. Anchorage shear connectors are provided.
5. Post-tensioning is done in three stages; after 7, 28 and 60 days of casting of the longitudinal girders.
6. Standard Freyssinet prestressing cables are used in the study.
7. The longitudinal girders are checked at five sections, i.e. x = 0, L/8, L/4, 3L/8 and L/2 under service and ultimate load conditions.
8. Clear width of roadway adopted is 7.5m.
9. The height of the kerb/footpath above the level of wearing coat is taken as 225mm. The width of the kerb and footpath is adopted as 600mm and 1500mm respectively, from the inner face of the handrail.
10. The minimum deck slab thickness is taken as 200mm.
11. S415 grade steel is used as non-prestressed reinforcement. The minimum diameter of bars used as non-prestressed reinforcement is 10mm.

3.2.2 Deck slab

IRC: 21 - 1987 (1997) requires a minimum clear cover to reinforcement as 40mm. On the basis of maximum deck slab thickness of 300mm and the maximum bar size of 25mm, a 50mm thick effective cover is provided. The design bending moments in one-way and cantilever slabs are calculated using the unit width method. For two-way slabs the Pigeaud method is used. Pigeaud coefficients are taken from Reynolds and Steedman (1992). The poissons ratio of concrete is taken as 0.15. Design shear force in the deck slab is calculated using the unit width method and assuming the interior panel rigidly supported over unyielding longitudinal and cross girders.

IRC: 21 - 1987 (1997) requires that minimum reinforcement in the transverse direction in the deck slabs should be 0.12% for S415 grade steel. IRC: 22 - 1986 (1991) requires that to take care of differential shrinkage stresses, the minimum tensile reinforcement in the longitudinal direction in cast-in-situ slabs shall not be less than 0.2% for all grades of steel.

3.2.3 Longitudinal girders

3.2.3.1 Properties of longitudinal girders

A) Geometrical properties

IRC: 18 -1985 (1997) provisions require that the minimum thickness of the web shall not be less than 180mm plus diameter of the duct hole. For the standard Freyssinet prestress cables the sheath diameter ranges from 30 to 66mm. Thus web thickness shall not be less than 210 to 240mm depending upon the type of cable. A minimum web thickness of 240mm is provided in all the longitudinal girders used in the investigation.

The cross-sectional details along with the cables location in various longitudinal girder sections are shown in Fig. 2. The maximum width of the bottom flange of longitudinal girder is 710mm. IRC: 18 - 1985 (1997) requires that the width of end block should be equal to the width of the bottom flange of the longitudinal girder. Thus towards the end block the web thickness should be increased from 240mm to maximum 710mm with a splay in plan not more than 1 in 6. The web thickening in all the longitudinal girders is obtained over a length of 4.2m i.e. the splay in plan will not be more than 1 in 6.

IRC: 18 - 1985 (1997) requires that the length of the end block in no case shall be less than 600mm nor less than its width. The maximum width of the end block is 710mm in longitudinal girders 2 and 3. The length of the end block beyond the centre line of supports towards the centre of the longitudinal girder is taken as 800 mm which will satisfy the code provisions for all the longitudinal girder sections in this study. An overhang of 400 mm beyond the centre line of the supports is also provided. Thus the total length of the end block in all the longitudinal girder sections is 1.2m. The longitudinal sectional plan of longitudinal girder is shown in Fig. 5.

B) Cable profile

The location of different cables in the various girders at their ends and at mid span are shown in Fig.2. In elevation the dip is provided by the parabolic profile of the cables and the splay in plan is provided by the reverse curve. The

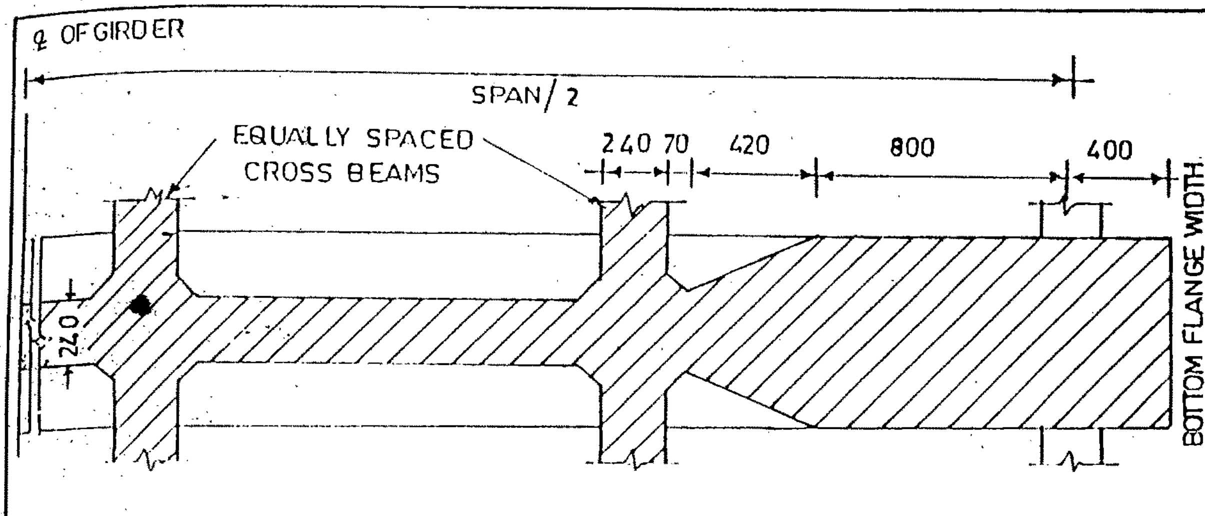
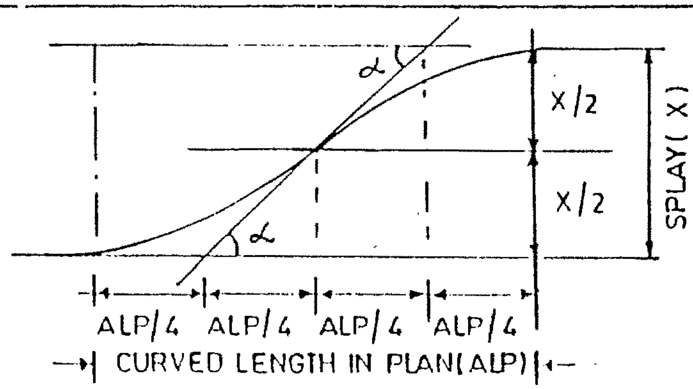
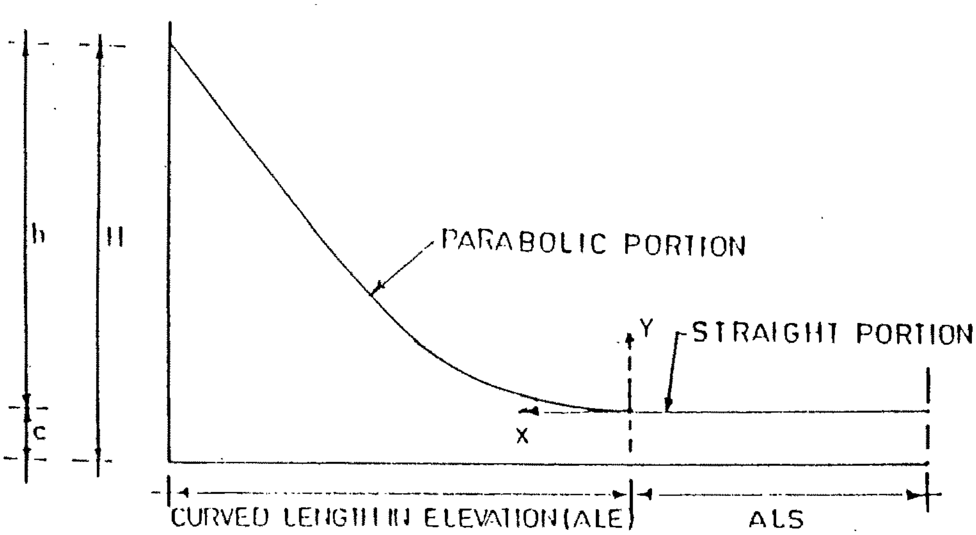


FIG 5 SECTIONAL PLAN OF MAIN PSC GIRDER



(A) BENDING OF CABLE IN PLAN



(B) BENDING OF CABLE IN ELEVATION

FIG. 6 BENDING OF CABLE

various parameters defining the cable profile are shown in Fig.6. These are determined in such a way that the cables are always within the longitudinal girder and not more than two cables are splayed at one section (preferably symmetrically placed cables). The values of cable profile parameters in all the girders are shown in Tables 5 - 12.

3.2.3.2 Load distribution in longitudinal girders

The distribution coefficients are calculated by Morrice-Little Method [using distribution coefficients tabulated by Sarkar et al. in Krishna Raju (1988)] and Hendry-Jaeger Method [Hendry & Jaeger (1958)]. Courbons's method is not used in the study as in some cases its limitations may be violated. Critical values of the coefficients obtained by using the above two methods are used for the load distribution. The transverse load positions corresponding to both; the maximum and minimum eccentricities in the transverse direction are considered. For Class A loading both the cases; single train and two trains of loads per two lane-width are considered.

3.2.3.3 Longitudinal girder stresses and prestress loss

The various forces considered for the analysis at different stages are shown in Table 13. The deck shuttering load is taken as 5 KN/m [UPSBC (1989)] and the longitudinal girder section is assumed to behave as non-composite till the deck shuttering is removed. The stresses due to differential shrinkage are evaluated as per IRC: 22 - 1966 (1966).

TABLE 5 CABLE PROFILE PARAMETERS FOR LONGITUDINAL GIRDER SECTION 1

Cable No.	h (mm)	C (mm)	H (mm)	Splay (mm)	ALP curved length in plan	ALE curved length in elevation	ALC total curved length	ALS straight length	AL
1 & 2	65	85	150	140	Start at 0.075L or 2m from end whichever is less, Curved length =2.5m	0.1L or 3m whichever is less	(0.075L + 2.5m) or 4.5m whichever is less	0.5L - ALC	0.5L
3 & 4	215	85	300	0	Nil	0.2L or 9m whichever is less	0.2L or 9m whichever is less	0.5L - ALC	0.5L
5 & 6	295	155	450	140	Start at 0.25L or 9m from end whichever is less, Curved length =2.5m	0.3L or 11m whichever is less	(0.25L + 2.5m) or 11.5m whichever is less	0.5L - ALC	0.5L
7	495	155	600	70	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.35L	0.35L	0.15L	0.5L
8	595	155	750	70	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.40L	0.40L	0.10L	0.5L
9	622.5	277.5	900	0	Nil	0.40L	0.40L	0.10L	0.5L
10	650	400	1050	0	Nil	0.45L	0.45L	0.05L	
11	730	470	1200	0	Nil	0.50L	0.50L	Nil	0.5L

TABLE 6 CABLE PROFILE PARAMETERS FOR LONGITUDINAL GIRDER SECTION 2

Cable No.	H (mm)	C (mm)	H (mm)	Splay (mm)	ALP curved length in plan	ALE curved length in elevation	ALC total curved length	ALS straight length	AL
1 & 2	65	85	150	150	Start at 0.075L or 2m from end whichever is less, Curved length =2.5m	0.1L or 3m whichever is less	(0.075L + 2.5m) or 4.5m whichever is less	0.5L - ALC	0.5L
3 & 4	250	85	335	0	Nil	0.2L or 8m whichever is less	0.2L or 8m whichever is less	0.5L - ALC	0.5L
5 & 6	365	155	520	150	Start at 0.25L or 8m from end whichever is less, Curved length =2.5m	0.3L or 10m whichever is less	(0.25L + 2.5m) or 10.5m whichever is less	0.5L - ALC	0.5L
7	550	155	705	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.35L	0.35L	0.15L	0.5L
8	735	155	890	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.40L	0.40L	0.10L	0.5L
9	797.5	277.5	1075	0	Nil	0.40L	0.40L	0.10L	0.5L
10	860	400	1260	0	Nil	0.45L	0.45L	0.05L	
11	975	470	1445	0	Nil	0.50L	0.50L	Nil	0.5L

TABLE 7 CABLE PROFILE PARAMETERS FOR LONGITUDINAL GIRDER SECTION 3

Cable No.	h (mm)	C (mm)	H (mm)	Splay (mm)	ALP curved length in plan	ALE curved length in elevation	ALC total curved length	ALS straight length	AL
1 & 2	65	85	150	150	Start at 0.075L or 2m from end whichever is less, Curved length =2.5m	0.1L or 3m whichever is less	(0.075L + 2.5m) or 4.5m whichever is less	0.5L - ALC	0.5L
3 & 4	285	85	370	0	Nil	0.2L or 7.5m whichever is less	0.2L or 7.5m whichever is less	0.5L - ALC	0.5L
5 & 6	435	155	590	150	Start at 0.25L or 7.5m from end whichever is less, Curved length =2.5m	0.3L or 9.5m whichever is less	(0.25L + 2.5m) or 10.0m whichever is less	0.5L - ALC	0.5L
7	655	155	810	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.35L	0.35L	0.15L	0.5L
8	875	155	1030	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.40L	0.40L	0.10L	0.5L
9	972.5	277.5	1250	0	Nil	0.40L	0.40L	0.10L	0.5L
10	1070	400	1470	0	Nil	0.45L	0.45L	0.05L	
11	1220	470	1690	0	Nil	0.50L	0.50L	Nil	0.5L

TABLE 8 CABLE PROFILE PARAMETERS FOR LONGITUDINAL GIRDER SECTION 4

Cable No.	h (mm)	C (mm)	H (mm)	Splay (mm)	ALP curved length in plan	ALE curved length in elevation	ALC total curved length	ALS straight length	AL
1 & 2	65	85	150	150	Start at 0.075L or 2m from end whichever is less, Curved length =2.5m	0.1L or 3m whichever is less	(0.075L + 2.5m) or 4.5m whichever is less	0.5L - ALC	0.5L
3 & 4	285	85	370	0	Nil	0.2L or 7.5m whichever is less	0.2L or 7.5m whichever is less	0.5L - ALC	0.5L
5 & 6	435	155	590	150	Start at 0.25L or 7.5m from end whichever is less, Curved length =2.5m	0.3L or 9.5m whichever is less	(0.25L + 2.5m) or 10.0m whichever is less	0.5L - ALC	0.5L
7	655	155	810	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.35L	0.35L	0.15L	0.5L
8	875	155	1030	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.40L	0.40L	0.10L	0.5L
9	972.5	277.5	1250	0	Nil	0.40L	0.40L	0.10L	0.5L
10	1070	400	1470	0	Nil	0.45L	0.45L	0.05L	
11	1220	470	1690	0	Nil	0.50L	0.50L	Nil	0.5L

TABLE 9 CABLE PROFILE PARAMETERS FOR LONGITUDINAL GIRDER SECTION 5

Cable No.	h (mm)	C (mm)	H (mm)	Splay (mm)	ALP curved length in plan	ALE curved length in elevation	ALC total curved length	ALS straight length	AL
1 & 2	65	85	150	140	Start at 0.075L or 2m from end whichever is less, Curved length =2.5m	0.1L or 3m whichever is less	(0.075L + 2.5m) or 4.5m whichever is less	0.5L - ALC	0.5L
3 & 4	290	85	375	0	Nil	0.2L or 7.5m whichever is less	0.2L or 7.5m whichever is less	0.5L - ALC	0.5L
5 & 6	445	155	600	140	Start at 0.25L or 7.5m from end whichever is less, Curved length =2.5m	0.3L or 9.5m whichever is less	(0.25L + 2.5m) or 10.0m whichever is less	0.5L - ALC	0.5L
7	670	155	825	70	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.35L	0.35L	0.15L	0.5L
8	895	155	1050	70	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.40L	0.40L	0.10L	0.5L
9	997.5	277.5	1275	0	Nil	0.40L	0.40L	0.10L	0.5L
10	1100	400	1500	0	Nil	0.45L	0.45L	0.05L	
11	1255	470	1725	0	Nil	0.50L	0.50L	Nil	0.5L

TABLE 10 CABLE PROFILE PARAMETERS FOR LONGITUDINAL GIRDER SECTION 6

Cable No.	h (mm)	C (mm)	H (mm)	Splay (mm)	ALP curved length in plan	ALE curved length in elevation	ALC total curved length	ALS straight length	AL
1 & 2	65	85	150	150	Start at 0.075L or 2m from end whichever is less, Curved length =2.5m	0.1L or 3m whichever is less	(0.075L + 2.5m) or 4.5m whichever is less	0.5L - ALC	0.5L
3 & 4	265	85	350	0	Nil	0.2L or 7.5m whichever is less	0.2L or 7.5m whichever is less	0.5L - ALC	0.5L
5 & 6	395	155	550	150	Start at 0.25L or 7.5m from end whichever is less, Curved length =2.5m	0.3L or 9.5m whichever is less	(0.25L + 2.5m) or 10.0m whichever is less	0.5L - ALC	0.5L
7	595	155	750	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.35L	0.35L	0.15L	0.5L
8	795	155	950	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.40L	0.40L	0.10L	0.5L
9	872.5	277.5	1150	0	Nil	0.40L	0.40L	0.10L	0.5L
10	950	400	1350	0	Nil	0.45L	0.45L	0.05L	
11	1080	470	1550	0	Nil	0.50L	0.50L	Nil	0.5L

TABLE 11 CABLE PROFILE PARAMETERS FOR LONGITUDINAL GIRDER SECTION 7

Cable No.	h (mm)	C (mm)	H (mm)	Splay (mm)	ALP curved length in plan	ALE curved length in elevation	ALC total curved length	ALS straight length	AL
1 & 2	65	85	150	150	Start at 0.075L or 2m from end whichever is less, Curved length =2.5m	0.1L or 3m whichever is less	(0.075L + 2.5m) or 4.5m whichever is less	0.5L - ALC	0.5L
3 & 4	285	85	370	0	Nil	0.2L or 7.5m whichever is less	0.2L or 7.5m whichever is less	0.5L - ALC	0.5L
5 & 6	435	155	590	150	Start at 0.25L or 7.5m from end whichever is less, Curved length =2.5m	0.3L or 9.5m whichever is less	(0.25L + 2.5m) or 10.0m whichever is less	0.5L - ALC	0.5L
7	655	155	810	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.35L	0.35L	0.15L	0.5L
8	875	155	1030	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.40L	0.40L	0.10L	0.5L
9	972.5	277.5	1250	0	Nil	0.40L	0.40L	0.10L	0.5L
10	1070	400	1470	0	Nil	0.45L	0.45L	0.05L	
11	1220	470	1690	0	Nil	0.50L	0.50L	Nil	0.5L

TABLE 12 CABLE PROFILE PARAMETERS FOR LONGITUDINAL GIRDER SECTION 8

Cable No.	H (mm)	C (mm)	H (mm)	Splay (mm)	ALP curved length in plan	ALE curved length in elevation	ALC total curved length	ALS straight length	AL
1 & 2	65	85	150	150	Start at 0.075L or 2m from end whichever is less, Curved length =2.5m	0.1L or 3m whichever is less	(0.075L + 2.5m) or 4.5m whichever is less	0.5L - ALC	0.5L
3 & 4	315	85	400	0	Nil	0.2L or 6.75m whichever is less	0.2L or 6.75m whichever is less	0.5L - ALC	0.5L
5 & 6	535	155	650	150	Start at 0.25L or 6.75m from end whichever is less, Curved length =2.5m	0.3L or 8.75m whichever is less	(0.25L + 2.5m) or 9.25m whichever is less	0.5L - ALC	0.5L
7	785	155	900	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.35L	0.35L	0.15L	0.5L
8	1035	155	1150	75	Curved length in plan of 1.5m is to be provided within ALE in flange area	0.40L	0.40L	0.10L	0.5L
9	1122.5	277.5	1400	0	Nil	0.40L	0.40L	0.10L	0.5L
10	1250	400	1650	0	Nil	0.45L	0.45L	0.05L	
11	1430	470	1900	0	Nil	0.50L	0.50L	Nil	0.5L

TABLE 13 VARIOUS STAGES OF LOADING OF PRESTRESSED GIRDERS

Sr. No.	Loading Stages	Section
1	Casting of girders	Non-composite
2	First stage prestressing of cables on 7^{th} day	Non-composite
3	Net (immediately after first stage prestress i.e. 1 + 2)	Non-composite
4	Casting of cross beams	Non-composite
5	Fixing of deck shuttering and deck reinforcement	Non-composite
6	Losses between 7^{th} to 28^{th} day	Non-composite
7	Net (3+4+5+6)	Non-composite
8	Second stage of prestressing after 28 days	Non-composite
9	Net (7+8)	Non-composite
10	Casting of deck slab	Non-composite
11	Net (9+10)	Non-composite
12	Removal of deck shuttering	Composite (T-beam)
13	Fixing of railing	Composite (T-beam)
14	Losses due to differential shrinkage	Composite (T-beam)
15	Prestress loss between 28 to 60 days	Composite (T-beam)
16	Net (11+12+13+14+15)	Composite (T-beam)
17	Third stage of prestressing on 60^{th} day	Composite (T-beam)
18	Net (16+17)	Composite (T-beam)
19	Casting of wearing coat	Composite (T-beam)
20	Net (18+19)	Composite (T-beam)
21	Prestress loss (60^{th} to $\propto$ days)	Composite (T-beam)
22	Net (20+21)	Composite (T-beam)
23	Live Load	Composite (T-beam)
24	Net (22+23)	Composite (T-beam)

The loss due to elastic shortening is computed exactly based on the sequence of prestressing and the slip of anchorage at the ends is taken as 5 mm [UPSBC (1989)]. Loss due to slip of wires is calculated as proposed by Huang (1969) with the difference that the stress variation in the cables along the length is taken as linear. The friction loss is computed assuming wire cables and Bright metal sheathing. For most part of the period between 28th and 60th day, the longitudinal girder section acts as a composite section. Hence for working out the loss of stress due to creep and shrinkage of concrete in this period, the section is taken as a composite. Dead load of shuttering for slab will be acting at the time of second stage of prestressing. However, this will be removed much before the completion of 60 days, thus shuttering load is not taken into account for calculating the loss due to creep of concrete.

3.2.3.4 Deflection in longitudinal girders

The short term or instantaneous deflection of prestressed members is governed by the bending moment distribution along the span and the flexural rigidity of the member. The flexural rigidity is calculated assuming a homogeneous longitudinal girder section. The bending moment distribution along the span is very complicated due to different distribution of various loadings on the girders. Hence, the bending moment distribution is assumed linear with the maximum at the centre and zero over the simple supports. Mohr's moment area theorem is used for calculating the deflection. The maximum permissible deflection is taken as span/400 [UPSBC (1989)].

3.2.3.5 Ultimate load analysis

In the post-cracking stage, the behaviour of a prestressed concrete member is more akin to that of a reinforced concrete member and hence the theories used for estimating the flexural strength of reinforced concrete section are also used for prestressed concrete sections.

The ultimate moment of resistance corresponding to under-reinforced and over-reinforced sections are calculated according to IRC: 18 - 1985 (1997), unless otherwise mentioned.

Prestressed concrete members subjected to transverse loads may fail in shear before their full flexural strength is attained, if they are not adequately designed for shear. There are two modes of shear cracking in structural concrete beams. Web-shear cracks generally start from an interior point, when the local principal tensile stress exceeds the tensile strength of concrete. Flexural shear cracks develop when the combined flexural and shear tensile stresses produce a principal tensile stress exceeding the tensile strength of concrete. The ultimate shear resistance corresponding to both web-shear and flexural-shear failures are calculated as per IRC: 18 - 1985 (1997), unless otherwise stated.

3.2.3.6 Non-prestressed reinforcement

Minimum non-prestressed reinforcement is provided according to the IRC: 18 - 1985 (1997). Untensioned reinforcement in prestressed concrete members is required for structural or constructional purposes. Various types of reinforcements are not added together.

In the longitudinal direction, 10 mm bars are placed along the periphery at spacing not exceeding 200 mm centres and at each corner one bar is placed. The total longitudinal reinforcement provided is not less than the minimum required by IRC: 18 - 1985 (1997).

In the vertical direction, minimum reinforcement in the bulbs of the beam is provided as per IRC: 18 - 1985 (1997). This reinforcement is uniformly spaced along the length of the beam.

To resist the longitudinal shear at the interface of the in-situ slab and longitudinal girder fabricated at different times, shear connectors of S415 grade steel are provided as per IRC: 22 - 1986 (1991). The top of the flange of the longitudinal girder is made rough for effective bonding. This reinforcement also prevent separation of the two units in the direction perpendicular to the contact surface.

Transverse reinforcement is provided according to IRC: 18 - 1985 (1997) unless otherwise mentioned.

3.2.4 Cross girders

The bending moment in the cross beams due to superimposed and live loads are calculated using the Morrice-Little method. In an equivalent orthotropic plate, maximum transverse moments occur at the centre of the bridge when the loads have the least transverse eccentricity and one of the wheel loads should move on the centre line of the bridge. The longitudinal position of the loads should correspond to the longitudinal moment at the centre of the span. The bending moment in cross beam is calculated assuming it rests on partially restrained longitudinal girder supports. The coefficient used for computing the maximum bending moment is calculated as sum of the 2/3rd and 1/3rd of the coefficients corresponding to fixed and simple supports respectively. The shear force is computed assuming the cross beam is supported on unyielding restrained longitudinal girder supports.

3.2.5 Construction sequence and prestressing schedule

The cables are stressed in three stages but 20 different sequences of prestressing are considered to adopt the best possible use of a particular type of cable under the given combination of various design parameters. The detail of sequence of prestressing of cables is given in Table 14. The following sequence of construction [UPSBC (1988)] has been adopted:

1. A few cables are stressed in sequence after 7 days of casting of the girders.
2. The cross beams are cast and the shuttering of the deck slab and its reinforcement fixed before the 28th day after casting of the girders.
3. Some more cables are stressed in sequence after the 28th day of casting of girders, after fixing of the deck shuttering and reinforcement.
4. The deck slab is cast immediately after the 2nd stage of prestressing.
5. The hand rails are fixed before the 60th day after casting of the girders.
6. The remaining cables are stressed in sequence on 60th day of casting of the girders.
7. The wearing coat is cast after the 60th day of casting of the girders.

The detail of sequence of prestressing of cables is given in Table 14.

TABLE 14 DETAIL OF SEQUENCE OF PRESTRESSING OF CABLES

Prestressing Sequence Identity (IPRSEQ)	First Stage Cables	Second Stage Cables	Third Stage Cables
1	1, 2 & 9	3, 4, 5, 6, 7 & 8	10 & 11
2	1, 2 & 10	3, 4, 5, 6, 7 & 8	9 & 11
3	1, 2 & 11	3, 4, 5, 6, 7 & 8	9 & 10
4	1, 2, 9 & 10	3, 4, 5, 6, 7 & 8	11
5	1, 2, 9 & 11	3, 4, 5, 6, 7 & 8	10
6	1, 2, 10 & 11	3, 4, 5, 6, 7 & 8	9
7	3, 4 & 9	1, 2, 5, 6, 7 & 8	10 & 11
8	3, 4 & 10	1, 2, 5, 6, 7 & 8	9 & 11
9	3, 4 & 11	1, 2, 5, 6, 7 & 8	9 & 10
10	3, 4, 9 & 10	1, 2, 5, 6, 7 & 8	11
11	3, 4, 9 & 11	1, 2, 5, 6, 7 & 8	10
12	3, 4, 10 & 11	1, 2, 5, 6, 7 & 8	9
13	5, 6 & 9	1, 2, 3, 4, 7 & 8	10 & 11
14	5, 6 & 10	1, 2, 3, 4, 7 & 8	9 & 11
15	5, 6 & 11	1, 2, 3, 4, 7 & 8	9 & 10
16	5, 6, 9 & 10	1, 2, 3, 4, 7 & 8	11
17	5, 6, 9 & 11	1, 2, 3, 4, 7 & 8	10
18	5, 6, 10 & 11	1, 2, 3, 4, 7 & 8	9
19	5 & 6	1, 2, 3, 4, 7, 8 & 9	10 & 11
20	7 & 8	1, 2, 3, 4, 5, 6 & 9	10 & 11

3.2.6 Equivalent UDL for BM & SF in longitudinal girders

The calculation of bending moment and shear force in girders due to various IRC loadings become easy if these complicated actual loadings are replaced by a simpler loading. For the purpose of bending moment, the actual loading on simply supported bridges can be replaced by an equivalent uniformly distributed load. The shear force at the centre of the span may or may not be zero depending upon the longitudinal load position. Thus for simulating the shear force, the actual loading is to be replaced by equivalent uniformly distributed load and an associated point load acting at the centre of the span.

The equivalent loading is calculated in such a way that the bending moment and shear force due to it will be an envelop to the actual bending moment and shear force in the longitudinal girder respectively. The equivalent loading is worked out for different combinations of number of cross beams (3 to 12) and longitudinal girders (2 to 5). The equivalent loading for different girders in a bridge deck system are given after taking into account the load distribution among the girders. It also takes into account the impact effect of the actual load. For load distribution purposes, one of the sections used in the study was considered.

3.2.7 Parametric study

The cost of carrying a given load can be substantially reduced by using high-strength concrete. But for effective use of high-strength concrete in highway bridges, it is necessary to first reduce the cost as much as possible by making suitable adjustments in various parameters of a bridge superstructure. Hence a detailed study of variation of forces in the deck slab and main longitudinal girders of a composite prestressed concrete bridge due to the variation in various bridge superstructure parameters is needed.

Basic theories are available for finding the moments in the two principal directions of the deck slab under point loads. Similarly well established load distribution methods for concrete bridge deck systems are available. But these theories have discussed the different aspects in isolation. For a cost effective design of a bridge, all the components of the bridge and the standard loadings likely to come on it must be considered simultaneously.

In the present investigation, all the possible longitudinal and transverse placement of loads which may be critical for the bending moment and shear force in the deck slab and longitudinal girders are considered along with the impact effect of the live loads. The self weight of the longitudinal girder has, however, not been considered. The design constraints were not any restriction for parametric study. The number of cross beams is varied from 3 to 12 while the span of the bridge is varied from 20m to 80m. The maximum and minimum number of longitudinal girders are 5 and 2 respectively. The transverse spacing of the girders is varied by changing the cantilever length of the deck slab. The minimum cantilever length of the deck slab is taken equal to the width of the kerb. The cantilever length was changed in terms of the ratio of cantilever length of deck slab to kerb width, defined as SRATIO. The value of SRATIO is varied from 1.0 to 3.0 in the increment of 0.25. The various parametric studies performed are as follows:

3.2.7.1 Variation of forces in longitudinal girders

The various parameters which may affect the maximum bending moment and shear force in longitudinal girders are NCB, NG, SRATIO and SPAN.

1. Maximum bending moment v/s number of cross beams for given SRATIO, number of girders, span and loading.
2. Maximum bending moment v/s number of girders for given SRATIO, number of cross beams, span and loading.
3. Maximum bending moment v/s span for given SRATIO, number of girders, number of cross beams and loading.
4. Maximum bending moment v/s SRATIO for given span, number of girders, number of cross beams and loading.
5. Maximum shear force v/s number of cross beams for given SRATIO, number of girders, span and loading.
6. Maximum shear force v/s number of girders for given SRATIO, number of cross beams, span and loading.
7. Maximum shear force v/s span for given SRATIO, number of girders, number of cross beams and loading.
8. Maximum shear force v/s SRATIO for given span, number of girders, number of cross beams and loading.

3.2.7.2 Variation of forces in deck slab

The variation of maximum bending moment and shear force in the deck is discussed in terms of its thickness required to resist these forces. The minimum deck slab thickness is 200 mm.

1. Deck slab thickness v/s number of girders for a given number of cross beams, deck slab concrete grade, span, SRATIO and loading.
2. Deck slab thickness v/s number of cross beams for given number of girders, deck slab concrete grade, span, SRATIO and loading.
3. Deck slab thickness v/s span for given number of cross beams, deck slab concrete grade, number of girders, SRATIO and loading.
4. Deck slab thickness v/s deck slab concrete grade for given number of cross beams, span, number of girders, SRATIO and loading.
5. Deck slab thickness v/s SRATIO for given number of cross beams, deck slab concrete grade, number of girders, span and loading.

3.3 Formulation of Optimal Design Problem

Most structures are designed iteratively. A preliminary trial design is first estimated and then analysed. If it is found to be adequate, it is called a feasible design. If the trial design is unsatisfactory, the designer changes it and repeats the analysis until a feasible one is obtained. There are usually an infinite number of feasible designs, and designers strive to find the best one within the time available.

Optimization techniques can aid the designer in finding the optimal design. The designer must identify a set of design variables that describe the structure, an objective function that measures the merits of the alternate designs, and design constraints that must be satisfied. The next step involves transcribing the formulation of the specific problem into a well defined standard mathematical model.

The present optimization problem is very close to an optimal control problem. It has two types of variables, namely, the control (design) and state variables. The control variables govern the evolution of the system from one stage to the next stage and the state variables describe the behaviour of the system at any stage. The objective function (the minimum superstructure cost) is a function of the control variables. The problem is very complicated because some of the state variables are also functions of all the control variables. In case of the practical constraints, the state variable may be a function of a particular control variable. All the control variables are integers. State variables may be integers or may have real values. The present problem is highly non-linear because the cost function, as well as almost all the constraints, are non-linear functions. A simple analytical solution to the present problem is not feasible unless assumptions are made to simplify the cost function. These assumptions may result in loss of generality of the solution and its scope of applicability. The suitability of a particular design of a concrete structure greatly depends upon the fact whether the required raw materials are locally available or are to be transported from far away places. Thus out of many alternate possible designs, one design may be economically feasible in one region and another in other region. Thus an attempt has been made to find an optimized design for all the grades of concrete used in the study.

The present study does not aim at any particular region of the country. The cost of various raw materials used depends upon their local availability. If they are not locally available, the transportation cost plays an important role while optimizing the cost function. Thus it becomes necessary to tabulate all the structurally possible designs along with their costs so that a particular design with minimum cost and feasible with locally available materials may be adopted. The transportation cost may be added to the tabulated cost, if the concrete materials are to be transported and the decision may be taken accordingly.

To satisfy all these requirements, a procedure was adopted which aims at starting the solution with a set of variables resulting in minimum cost of the structure and satisfying all the design constraints. The increment in the various variables are taken in such a sequence that the incremental increase in cost is probably minimum. The details of the programme are given in flow chart.

3.3.1 Design variables

The design of a structure can be completely described by a set of design variables and pre-assigned parameters. Only the design variables are modified during the optimization. For a standard I-girder, the cross-sectional dimensions are pre-assigned parameters. The profile of the cables is dependent on the span of the longitudinal girder and is a pre-assigned parameter. The best advantage of the cable profile during prestress transfer stage is taken by adopting 20 alternate sequences of prestressing the eleven cables. Typical cross sectional dimensions of the main bridge girders investigated are shown in Fig.2. The details of the cable profile are shown in Fig. 6 and Tables 5 to 12. The dimensions of cross beam and other elements of the bridge cross section are also pre-assigned variables. The design variables include the following:

1. Prestressing force
2. Number of girders

3. Number of cross beams
4. Transverse girder spacing
5. Deck slab concrete compressive strength at 28 days age
6. Longitudinal girder concrete compressive strength at 28 days age
7. Type of longitudinal girder

3.3.2 Design constraints

There are two kinds of constraints; namely, functional and practical constraints. Compliance with serviceability and ultimate limit state provisions are functional restraints. The practical restraints include the limits for the design variables.

3.3.2.1 Functional constraints

In the present formulation, the following functional constraints are considered.

1. The gross thickness of deck slab should not be less than 200 mm.
2. Flexural constraints at transfer

At nine stages of construction and transfer of prestress, all the longitudinal girder sections are checked at five cross sections (x=0, L/8, L/4, 3L/8 and L/2) for permissible compressive and tensile stresses at transfer.

3. Flexural constraints at service loads

The girders are checked for permissible stresses at service loads. The permissible compressive and tensile stress at transfer and service loads are as per IRC: 18 - 1985 (1997,) unless otherwise mentioned.

4. Allowable deflection

The elastic deflection at various stages of construction and transfer of prestress and under service loads are not allowed to exceed L/400 [UPSBC (1989)].

5. Flexural constraints at ultimate loads

The girders are checked for moment of resistance at five sections along its length under an ultimate load equal to $1.25 \times$ permanent load + $2 \times$ super imposed load + $2.5 \times$ live load including impact.

6. Shear constraints at ultimate loads

The girders are checked for the maximum permissible shear stress under the ultimate load as discussed above.

The moment of resistance and maximum permissible shear stress are calculated as per IRC: 18 - 1985 (1997) unless otherwise mentioned.

3.3.2.2 Practical constraints

1. Longitudinal girder Identity, $1 \leq IG \leq 8$
2. Prestressing cable Identity, $1 \leq IPS \leq 7$

The details of various prestressing cables used in the present investigation are given in Table 15.

3. Number of cross beams, $3 \leq NCB \leq 12$
4. Number of girders, $2 \leq NG \leq 5$
5. Concrete grade in deck slab, $1 \leq DFCK \leq 5$

The concrete grades corresponding to different deck slab concrete identity (ICON1) are given in Table 16.

6. Concrete grade in girders, $1 \leq FCK \leq 8$

The concrete grades corresponding to different longitudinal girder concrete identity (ICON) are shown in Table 17.

7. Transverse girder spacing, $1 \leq SRATIO \leq 3$

3.3.3 Objective function (Cost function)

The cost function is based on the minimum cost of the superstructure. This assumes that the cost of piers, abutments and approaches is relatively unaffected by changes in the number of girders. The superstructure includes the deck, cross girders and main girders. The cost function includes the cost of materials, shuttering and bearings. Because the present study does not aim at any specific region, the cost of labour and transportation are not included in the cost function.

One of the aims of the present investigation is to tabulate all the alternate designs with their costs. Hence all the fixed costs like wearing coat, handrails, kerbs etc. are also included in the cost function. The cost of formwork is taken in terms of per cubic metre of concrete used in the superstructure.

The effect of the variation of the longitudinal girder depth is not included in the study. The increased depth increases overall cost because additional earthwork is needed to increase the embankment height or excavation depth. When the clearance under the bridge is not a factor, the increased longitudinal girder depth may actually reduce the cost marginally because of the resulting lower pier height.

Current costs of various materials are taken either from the suppliers (quotations were called for high range water reducer and the cost of prestressing

TABLE 15 CHARACTERISTICS OF PRESTRESSING CABLES

Prestressing Cable Identity (IPS)	1	2	3	4	5	6	7
Description	12ϕ5	12ϕ7	12ϕ8	6T13	24ϕ7	24ϕ8	12T13
Tendon Area(mm^2)	235	462	603	557	922	1206	1115
Ultimate Tensile Stress (MPa)	1600	1500	1400	1800	1500	1400	1800
Duct Diameter (mm)	33	39	48	48	57	60	66
Weight of Cable (Kg/m)	1.8	3.6	4.70	4.50	7.20	9.40	9.0

TABLE 16 DECK SLAB CONCRETE GRADES

Deck slab concrete identity (ICON1)	1	2	3	4	5
Concrete Strength (MPa)	30	35	40	45	50

TABLE 17 LONGITUDINAL GIRDER CONCRETE GRADES

Longitudinal Girder concrete identity (ICON)	1	2	3	4	5	6	7	8
Concrete Strength (MPa)	35	40	45	50	60	70	80	90

cables and related materials were made available by M/S Freyssinet Prestressed Company Limited, Bombay) or from the estimate of a bridge obtained from U.P. State Bridge Corporation Limited (UPSBC), Lucknow. The cost of various materials are shown in Table 18.

Costs of various grades of concrete are based on the requirements of MOST (1995), IS: 10262 - 1982 (1982) and ACI - 211 (1993). Minimum quantity of cement and maximum permissible water-cement ratio are as per MOST (1995). Mix design of concrete with 28 days compressive strength less than 50 MPa is done according to IS: 10262 - 1982 (1982) and over 50 MPa according to ACI - 211 (1993). The estimated cost of various grades of concrete are summarised in Table 19.

The costs of various items were normalised with respect to the cost per ton of 30 MPa concrete. The cost of the bridge superstructure is represented as the cost per metre of the bridge span (TCOST).

$$\begin{aligned}TCOST = {} & (VDSLAB + VKERB + VWCOAT) \times DCCI \times UWC + (VLGD + VCGD) \\ & \times GCCI \times UWC + ASTUNTS \times CCI13 + ASTHRAIL \times CCI14 + (CCI12 + \\ & CCI15) \times (2 \times NG \times NT)/AL + TSHEAT \times CCI16 + WTCABLE \times CCI17 + \\ & (VDSLAB + VKERB + VWCOAT + VLGD + VCGD) \times CCI18 + \\ & (2 \times NG \times CCI19)/AL\end{aligned}$$

Where,

TCOST	Total cost of one metre length of bridge superstructure / cost of one ton of M30 concrete
VDSLAB	Volume of concrete in deck slab (m^3 per m span of the bridge)
VKERB	Volume of concrete in kerbs (m^3 per m span of the bridge)

VWCOAT	Volume of concrete in wearing coat (m^3 per m span of the bridge)
VLGD	Volume of concrete in main prestressed girders (m^3 per m span of the bridge)
VCGD	Volume of concrete in cross beams (m^3 per m span of the bridge)
DCCI	Cost comparison index of concrete in superstructure excluding the girders (cost of one ton of concrete / cost of one ton of M30 concrete)
GCCI	Cost comparison index of concrete in the girders
UWC	Unit weight of concrete (ton per cubic metre)
ASTUNTS	Weight of un-tensioned steel in superstructure (ton per metre of bridge span)
CCI13	Cost of one ton of steel / cost of one ton of M30 concrete
ASTHRAIL	Weight of steel in handrails (ton per metre of bridge span)
CCI14	Cost of one ton of structural steel / cost of one ton of M30 concrete
CCI12	Cost of one pair of anchorage / cost of one ton of M30 concrete
CCI15	Cost of one end tube / cost of one ton of M30 concrete
TSHEAT	Length of BM tubes (metre per metre of bridge span)
CCI16	Cost of one metre of BM tube / cost of one ton of M30 concrete
WTCABLE	Weight of prestressing cables (ton per metre of bridge span)
CCI17	Cost of one ton of prestressing steel / cost of one ton of M30 concrete
CCI18	Cost of shuttering per m^3 of concrete in superstructure per m of bridge span / cost of one ton of M30 concrete

CCI19	Cost of one bearing / cost of one ton of M30 concrete
NG	Number of main prestressed girders
NT	Number of prestressing tendons in one main prestressed longitudinal girder
AL	Span of the bridge (m), including the overhangs of the girder

3.3.4 Economic studies

In order to asses the potential economic advantages of using high-strength concrete in bridges, the optimization system was used to generate all the possible minimum cost superstructure designs using various combinations of the design parameters under study.

The composite simply supported post-tensioned slab-on-girder bridges are used for this study. The bridge has two traffic lanes with a clear width of roadway of 7.5m. All the design parameters discussed earlier are varied due to their potentially significant impact on the structural efficiency and hence the cost of the structure.

I) Longitudinal girder cross section

It was not possible to examine every I-girder cross section because of the large number of available sections. It is believed that the girder types used for this study constitute an adequate range over which conclusions could be generalised. Eight sections close to standard sections (Two AASHTO sections; type V and type VI, one Oregon section, two Washington sections; type 100S and

TABLE 18 UNIT COSTS OF VARIOUS CONSTITUENT MATERIALS OF CONCRETE

Sr. No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Item	H.T. wire (T)	Strand (T)	Tor- Steel bars (T)	Structural Steel (T)	Cement (T)	Anchorages (per set) wire (strands)	High Range Water Reducer (L)	Sheathing (m)	Tube Units (each)	Bearing (each)	Shuttering (m^3 concrete)	Coarse Aggregate (T) 20 (12) mm size	Fine Aggregate (T)	Fly-as (per 5 Kg.)
Cost (Rs.)	28000	33000	16000	19000	25000	700 (1100)	100	40	900	8500	500	300 (350)	200	2

TABLE 19 UNIT COSTS OF DIFFERENT GRADES OF CONCRETE

Sr. No.	1	2	3	4	5	6	7	8	9
Concrete Grade (MPa)	30	35	40	45	50	60	70	80	90
Cost/m^3 (Rs.)	1470	1505	1555	1580	1840	1910	1980	2055	2125

type 120S, three Colorado sections; type G66, type G72 and type G80) were used. Slight modifications in the dimensions were necessary to incorporate the relevant IRC codal specifications, as the aim of the study is to investigate the possibility of extending the IRC codes specifications for using high-strength concrete in highway bridges.

II) Transverse girder spacing

The impact of the transverse spacing of the longitudinal girder could not be directly evaluated. In the study, the deck slab thickness is also allowed to vary. Thus when the transverse girder spacing is changed , both the thickness and the effective width of the flange of the girders are changed. The parametric study have shown that maximum shear force in the girders was more susceptible to the variation in transverse girder spacing than the maximum bending moment. The transverse girder spacing is defined in terms of SRATIO. The value of SRATIO is varied from 1 to 3 in the increment of 0.25. The minimum cantilever length is governed by the provision of a kerb and the maximum was governed by the discussion held with U.P. State Bridge Corporation Limited, Lucknow. The maximum cantilever length of the deck slab was taken as 2.25m.

III) Deck slab concrete grade

Five grades of concrete (30, 35, 40, 45 and 50 MPa) are considered for use in the deck slab. The moment of resistance of the composite longitudinal girder section depends upon the thickness and concrete grade of the flange of the longitudinal girder. The use of concrete of strength higher than 50 MPa in deck slabs may not be of much advantage, because IRC: 21 - 1987 (1997) restricts the deck slab thickness to a minimum of 200 mm.

IV) Grade of concrete in longitudinal girder

Currently the maximum grade of concrete in bridges in India is 50 to 60 MPa. The concrete grades used in girders in the present study are 35, 40 ,45, 50, 60, 70, 80 and 90 MPa. The wide range of concrete used in the girders cover the normal- and high- strength concretes. Normal-strength concrete is considered in the study to asses the advantage of high-strength concrete over normal-strength concrete. This study looked beyond the current concrete production capabilities in India.

V) Prestressing force

Jobse and Moustafa (1984) reported that the maximum available prestressing force might restrict the span length capabilities of deeper sections when high-strength concrete is used. To minimise this possibility, the standard Freyssinet prestressing cables (1984) covering a wider range of prestressing force were used. The normally used Standard prestressing Freyssinet cable systems are 12ϕ5, 12ϕ7, 12ϕ8, 24ϕ7, 24ϕ8, 6T13 and 12T13. Eleven cables are used in all the sections. The position of cables in the various longitudinal girder sections are shown in Fig. 2.

VI) Number of longitudinal girders

The minimum number of girders that may be used for a given span depends on the total width of the bridge. For slab-on-girders bridge, this limit may range from 2 to 4 [Hassanain and Loov (1999)]. For two lane bridges of medium spans, the minimum two girders must be used. The parametric study of the present investigation has shown that use of more than five girders in two lane bridges was not advantageous.

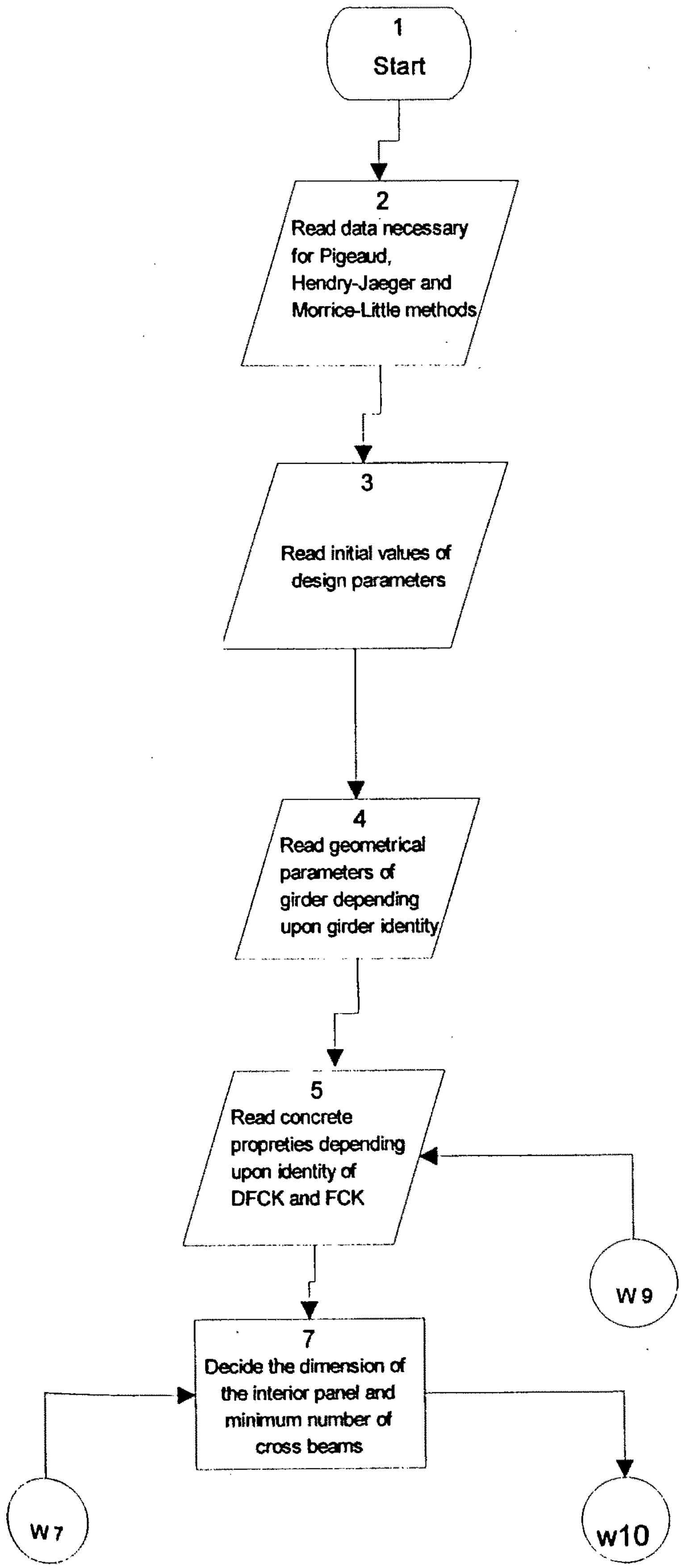
VII) Number of cross beams

AASHTO (1977) recommends the maximum spacing of cross beams as 12m. Thus the minimum number of cross beams used in the investigation is 3 or such that the spacing of cross beams is not more than 12m. An attempt was made to get the best possible distribution of loads among girders by increasing the number of cross beams i.e. by increasing the transverse rigidity of the bridge deck system. The maximum number of cross beams considered in the investigation is 12.

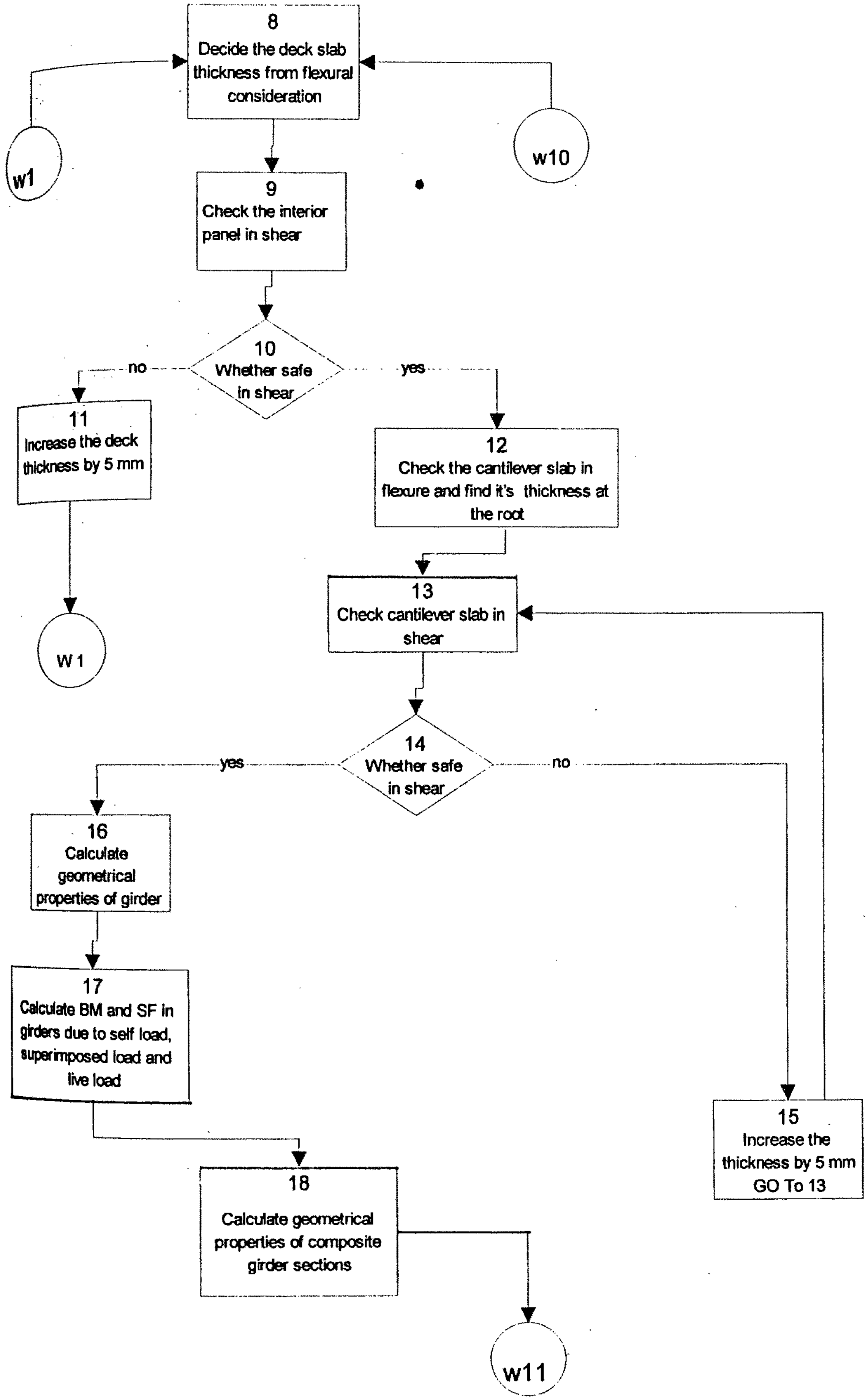
VIII) Span of longitudinal girder

The span range considered in the present study is from 18m to a structurally possible maximum of about 50m. The incremental increase of span considered is 1m. This small increment allowed an accurate trend to be obtained and made it easier to identify the maximum spans for each longitudinal girder cross-section.

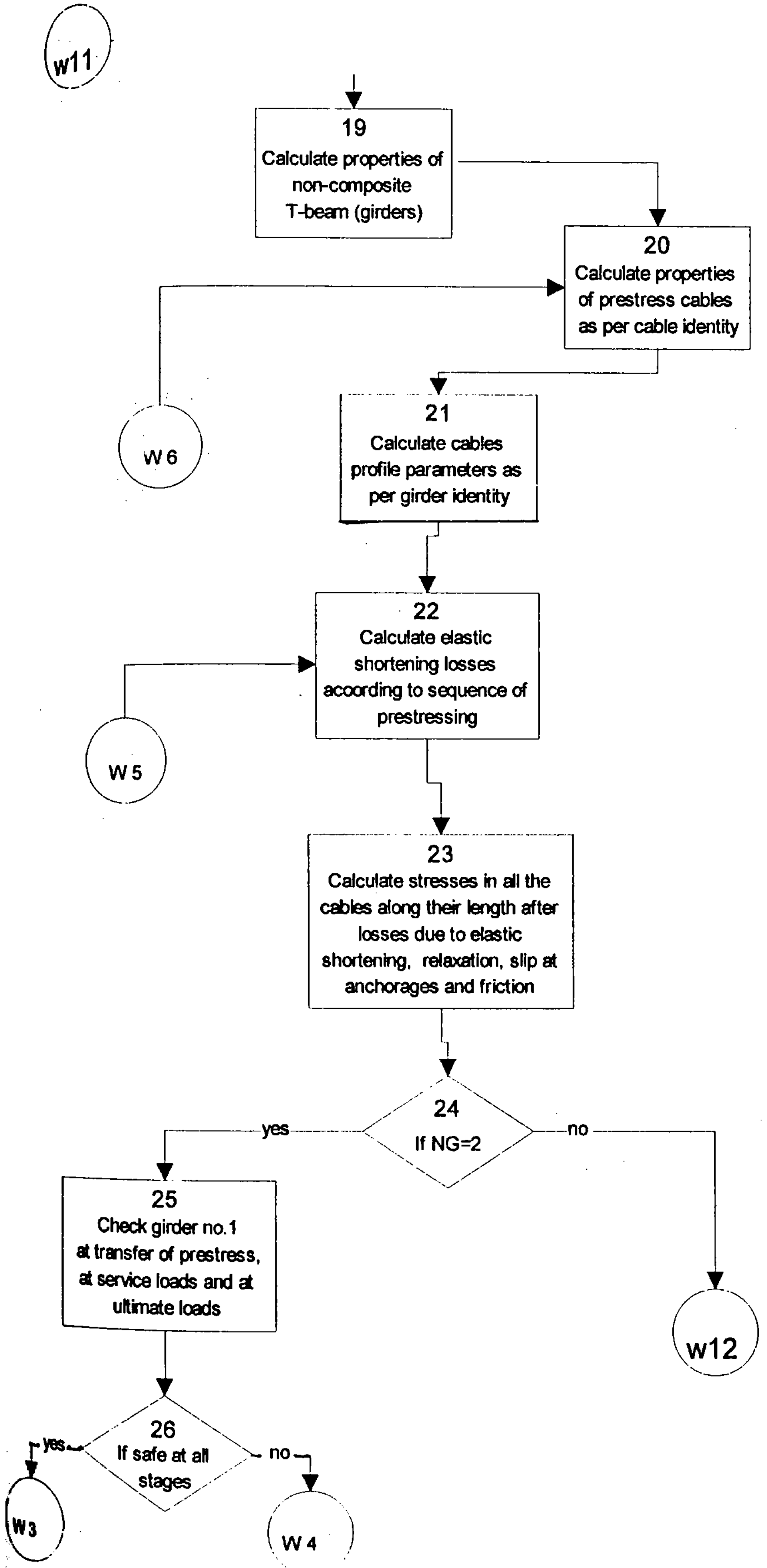
3.4 FLOW-CHART



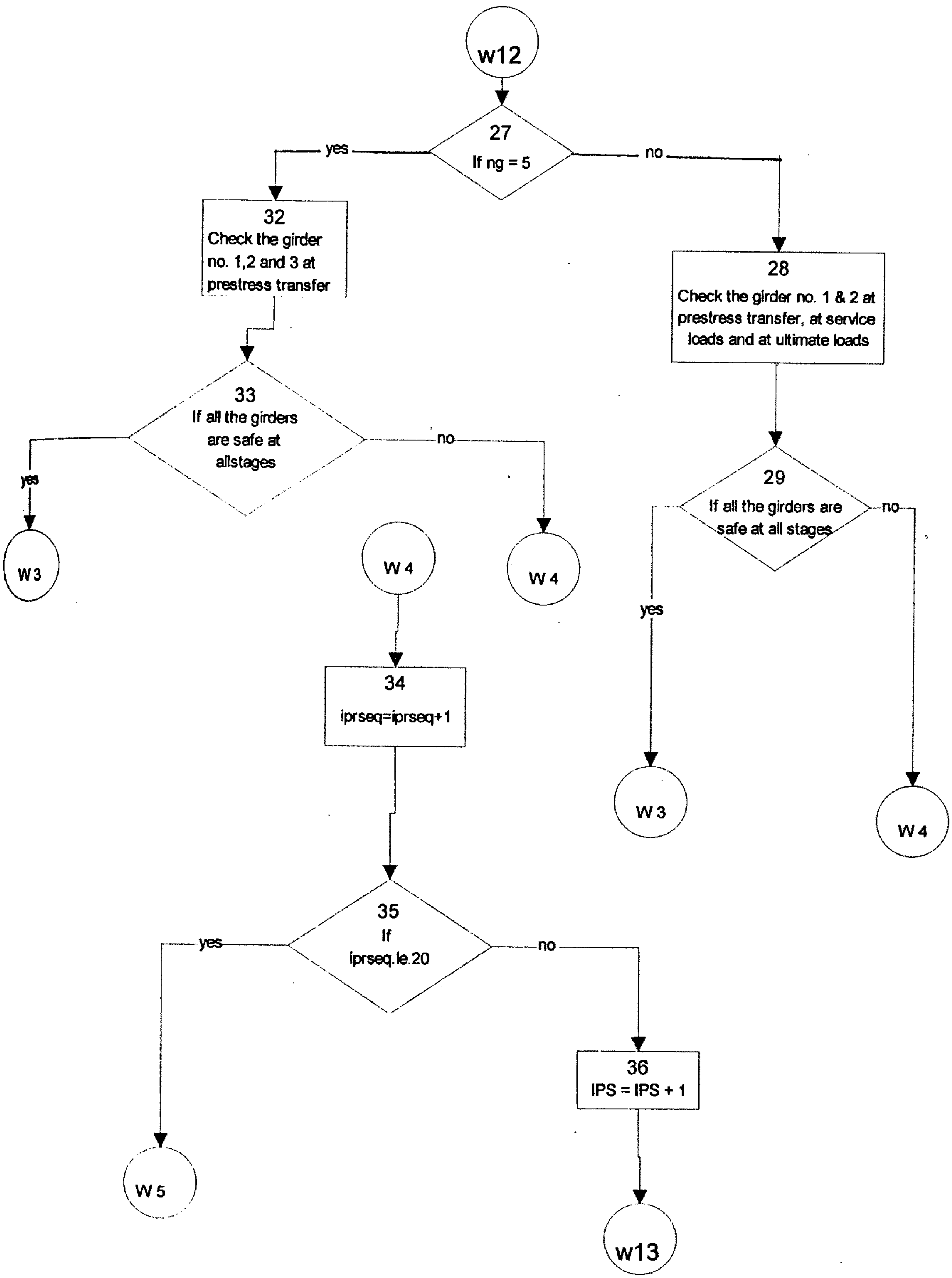
contd.


8
Decide the deck slab thickness from flexural consideration
w1
w10
9
Check the interior panel in shear
10
Whether safe in shear
no
yes
11
Increase the deck thickness by 5 mm
W 1
12
Check the cantilever slab in flexure and find it's thickness at the root
13
Check cantilever slab in shear
14
Whether safe in shear
yes
no
16
Calculate geometrical properties of girder
17
Calculate BM and SF in girders due to self load, superimposed load and live load
15
Increase the thickness by 5 mm GO To 13
18
Calculate geometrical properties of composite girder sections
w11

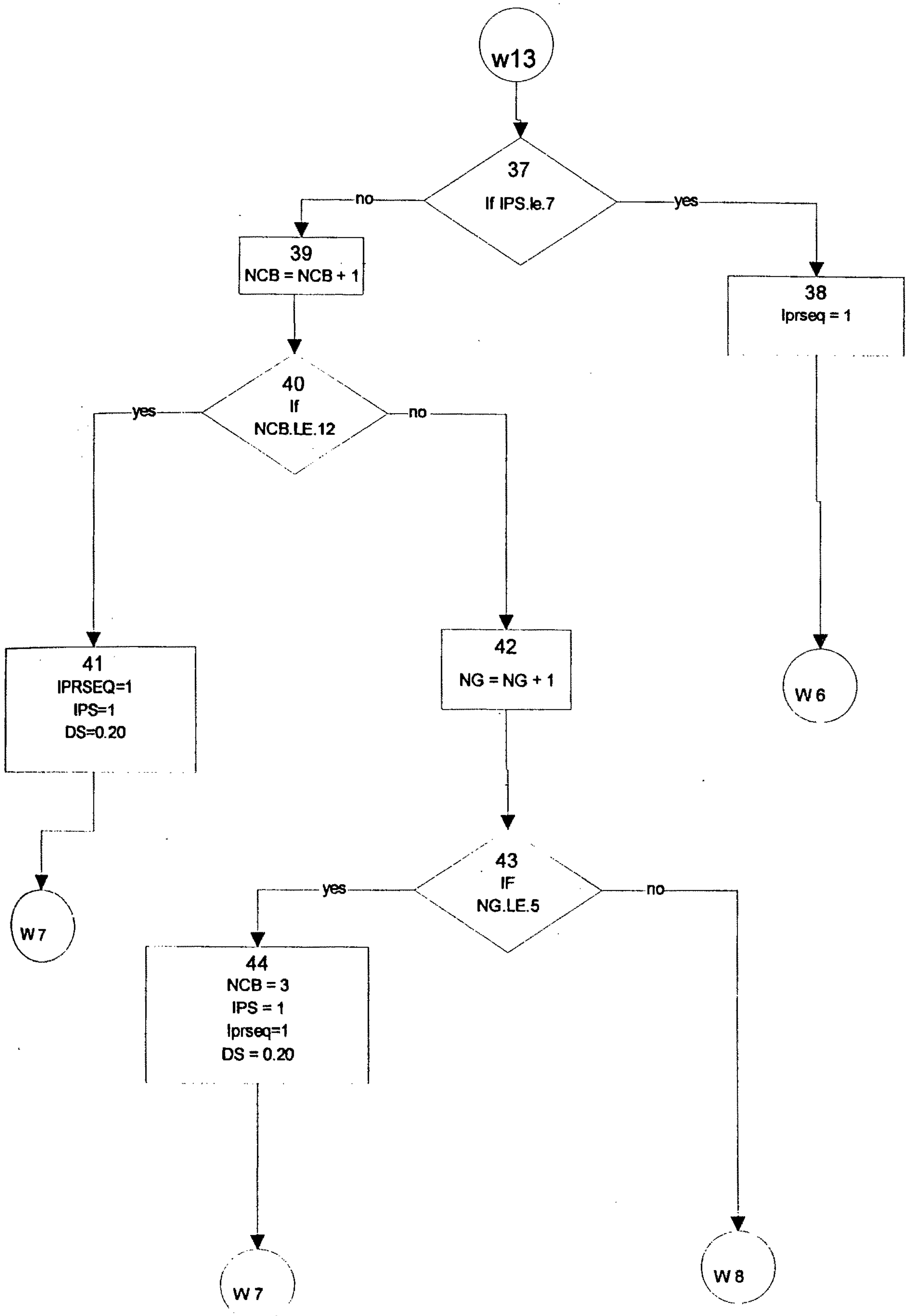
contd.


w11
19
Calculate properties of non-composite T-beam (girders)
20
Calculate properties of prestress cables as per cable identity
W 6
21
Calculate cables profile parameters as per girder identity
22
Calculate elastic shortening losses according to sequence of prestressing
W 5
23
Calculate stresses in all the cables along their length after losses due to elastic shortening, relaxation, slip at anchorages and friction
24
If NG=2
yes
no
25
Check girder no.1 at transfer of prestress, at service loads and at ultimate loads
w12
26
If safe at all stages
yes
no
W3
W 4

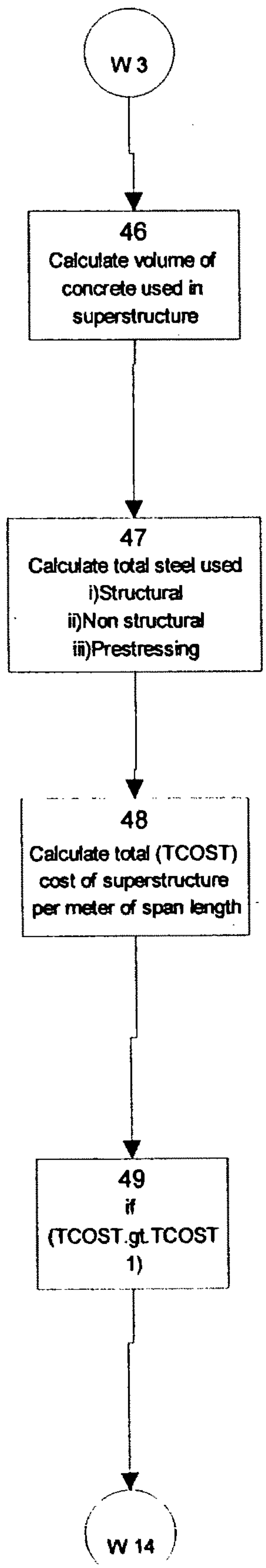
contd.


w12
27
If ng = 5
yes
no
32
Check the girder no. 1,2 and 3 at prestress transfer
28
Check the girder no. 1 & 2 at prestress transfer, at service loads and at ultimate loads
33
If all the girders are safe at allstages
yes
no
W 3
W 4
29
If all the girders are safe at all stages
yes
no
W 3
W 4
W 4
34
iprseq=iprseq+1
35
If iprseq.le.20
yes
no
W 5
36
IPS = IPS + 1
w13

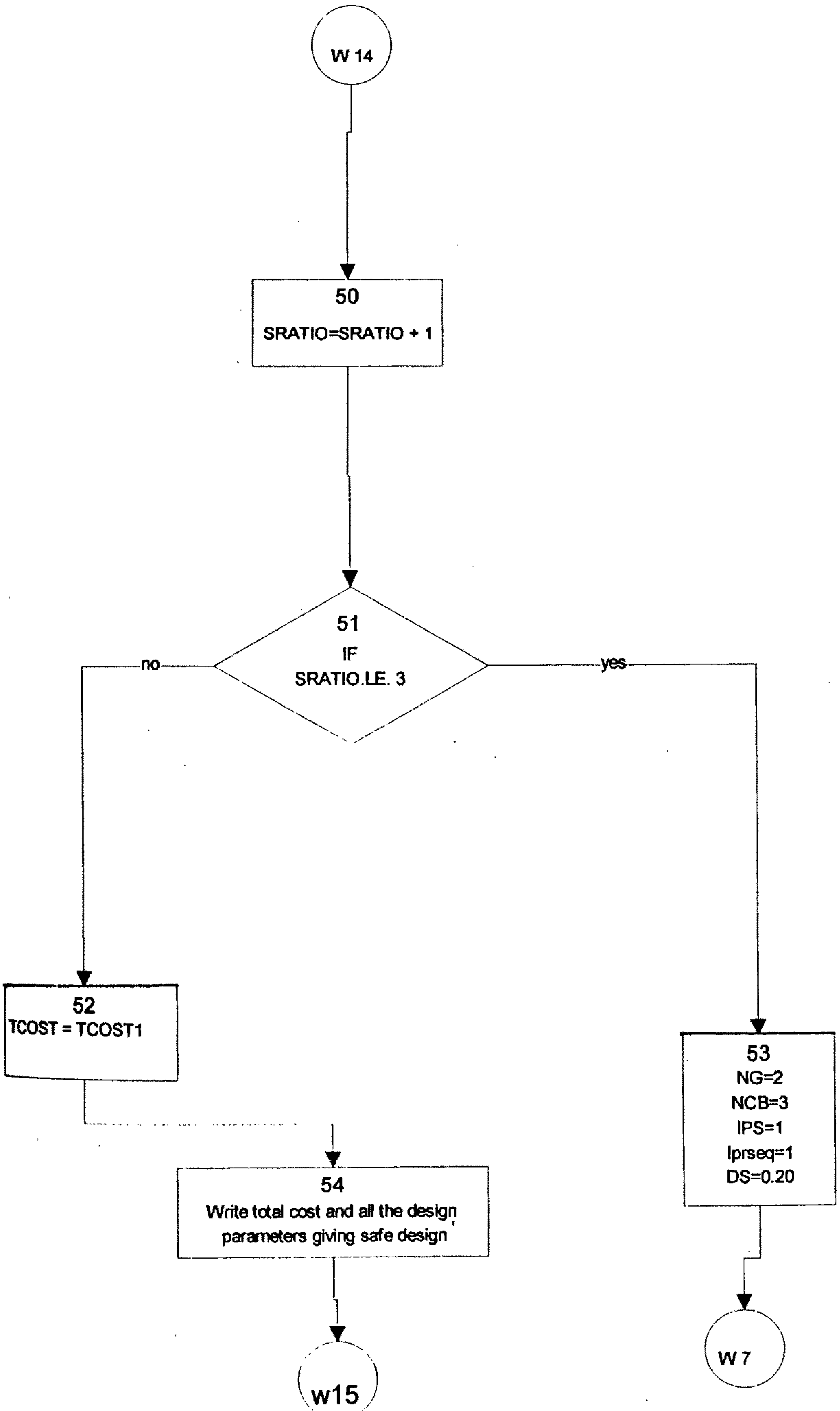
contd.


w13
37
If IPS.le.7
no
yes
39
NCB = NCB + 1
38
Iprseq = 1
40
If
NCB.LE.12
yes
no
41
IPRSEQ=1
IPS=1
DS=0.20
42
NG = NG + 1
W 6
W 7
43
IF
NG.LE.5
yes
no
44
NCB = 3
IPS = 1
Iprseq=1
DS = 0.20
W 7
W 8

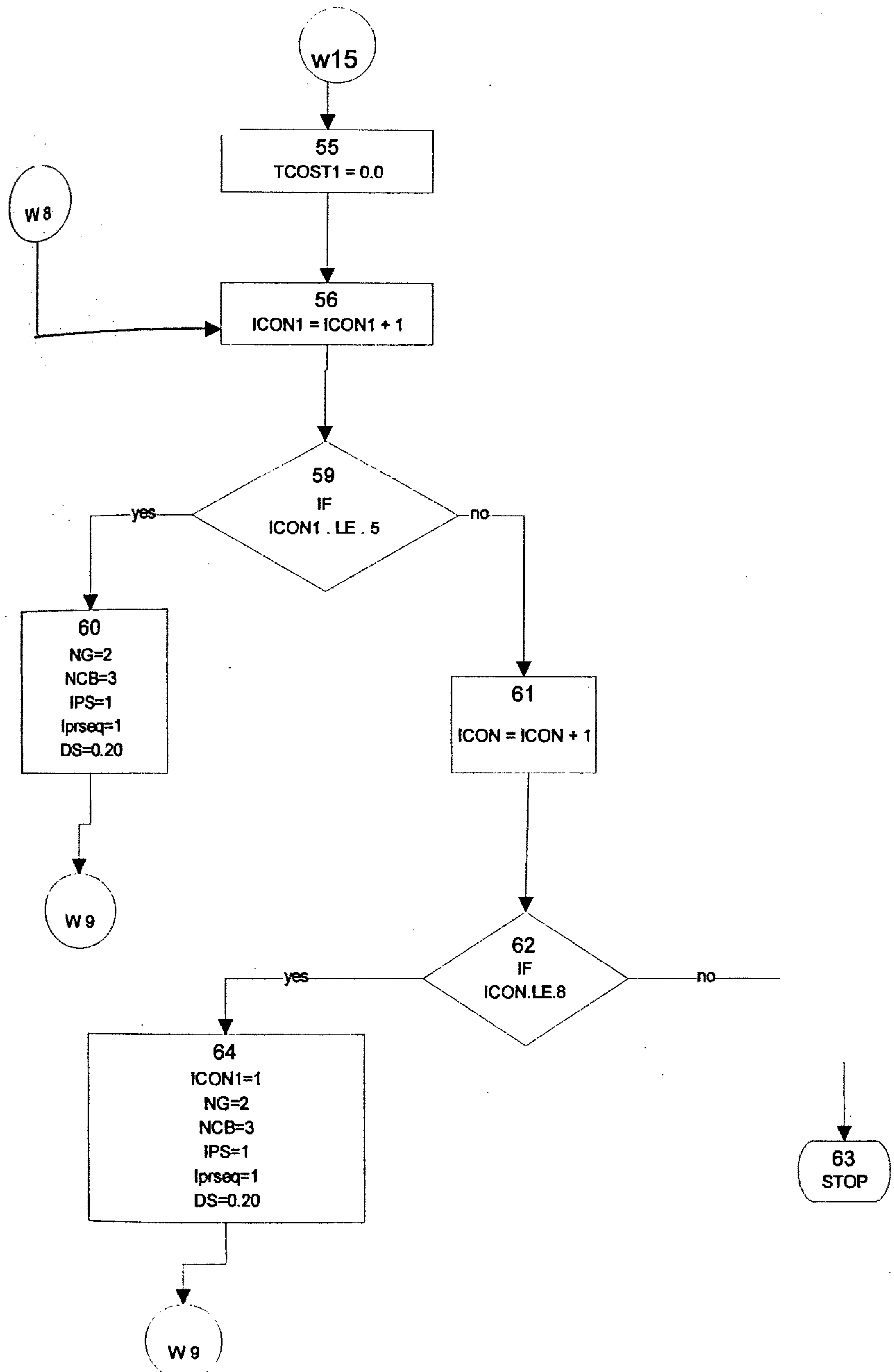
contd


W 3
46
Calculate volume of concrete used in superstructure
47
Calculate total steel used
i)Structural
ii)Non structural
iii)Prestressing
48
Calculate total (TCOST) cost of superstructure per meter of span length
49
if
(TCOST.gt.TCOST 1)
W 14

contd.


W 14
50
SRATIO=SRATIO + 1
51
IF
SRATIO.LE. 3
no
yes
52
TCOST = TCOST1
53
NG=2
NCB=3
IPS=1
Iprseq=1
DS=0.20
54
Write total cost and all the design parameters giving safe design
w15
W 7

contd.


w15
55
TCOST1 = 0.0
W 8
56
ICON1 = ICON1 + 1
59
IF
ICON1 . LE . 5
yes
no
60
NG=2
NCB=3
IPS=1
Iprseq=1
DS=0.20
61
ICON = ICON + 1
W 9
62
IF
ICON.LE.8
yes
no
64
ICON1=1
NG=2
NCB=3
IPS=1
Iprseq=1
DS=0.20
63
STOP
W 9

3.5 Conclusions

In this chapter an introduction has been provided with respect to the scope of the proposed analytical investigations. The analytical work is subdivided into three main parts;

1. Computer programme for analysis and design of the slab-on-girder bridges,
2. Parametric study of the variation of the forces in deck slab & longitudinal girders and
3. Estimation and optimization of the cost of the superstructure of the prestressed slab-on girder bridges.

Analysis and design of the bridge superstructure has been carried out considering the design constraints; flexural constraints at transfer of prestress and at the service loads, the deflection constraint at the service loads and flexural & shear constraints at the ultimate loads. The provisions of MOST (1995) and the various related IRC codes have been followed for analysis and design of the bridges. For load distribution in girders the Morrice-Little and the Hendry-Jaeger methods are used. The value of distribution coefficients critical of these two methods has been adopted in design. For the purpose of the computation of the live load bending moments and shear forces, equivalent uniformly distributed loads for IRC class AA and class A loadings are presented.

The use of high-strength concrete should reduce the cost of the bridges. But the higher cost of production of high-strength concrete may offset this advantage to some extent. The effect of variation of the values of various parameters of superstructure on the forces in the deck slab and longitudinal girders is studied with the aim of obtaining the cost effective superstructure by adjusting the values of these parameters.

Cost effectiveness of a concrete structure greatly depends on its place of construction. The cost of transportation of the constituent materials of concrete will affect the cost effectiveness of the structure to a great extent. This is not of much advantage to consider the cost effectiveness of the bridge with respect to a particular region. Hence, in the present study, the computer programme was made in such a way that all the possible cost effective alternate designs, using various combination of the concrete grades in the deck slab and girders, may be obtained. The results so obtained may be used for cost effective preliminary dimensioning of the bridges using the concrete produced out of the local materials.

4. PROPOSED REVISIONS IN IRC 18 & IRC 21 CODES

4.1 Introduction

From the literature reviewed on mechanical properties and structural design considerations of high-strength concrete, it is clear that there are differences in the basic behaviour of the two materials; high-strength concrete and normal-strength concrete. Various researchers stressed that it was very dangerous to extend directly the present code provisions, derived basically on the basis of the results of the experimental studies on normal-strength concrete specimens and structures, to high strength concrete.

Two types of revisions are proposed in the IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997) codes:

I) IRC codes do not allow the increase in the value of various strength parameters with the increase in the concrete compressive strength beyond a certain value of the concrete compressive strength. Such restrictions are probably put to indirectly discourage the use of high-strength concrete as its production, placing, compaction and curing etc. requires very strict quality control. AASHTO-1977 permits the use of higher strength concrete ($f_c' > 41$ MPa) provided it is ensured that the control over the materials and fabrication procedure would provide the required strength.

II) When failure mechanism, under a particular type of load, of high-strength concrete is basically different from that of the normal-strength concrete. In such circumstances the present code provisions can not be extended without modification to high-strength concrete.

4.2 Proposed Revisions in IRC: 21-1987 (1997)

The slab-on-girders bridge is the structure of the present study hence, only those provisions of the code which are related to the reinforced concrete deck slab of the bridge are critically reviewed. Most of the bridge codes require that the thickness of the deck slab should not be less than a minimum specified value. IRC: 21 - 1987 (1997) recommends that deck slab thickness should not be less than 200 mm. From the literature reviewed it can be concluded that it is not economically advantageous to use high-strength concrete in bridge deck because its thickness can not be reduced continuously by using high-strength concrete. It is more economical to use high-strength concrete in the prestressed girders in comparison to the deck slab. Thus in the present investigation, the maximum grade of concrete used in the deck slab is 50 MPa which is within the upper limit of normal-strength concrete.

IRC: 21 - 1987 (1997) discourage the use of concrete of compressive strength higher than 35 MPa whereas the maximum strength of concrete used in the deck slab, in the present investigation, is 50 MPa. Thus, revisions proposed in IRC: 21 -1987 (1997) are basically of category I and are recommended for concrete strength up to 60 MPa.

4.2.1 Clause number 303.1

4.2.1.1 Permissible flexural compressive stress (σ_{cbc})

The code recommends the permissible stress in flexural compression as 33% of the 28 day cube strength with a maximum value of 11.5 MPa. This permissible stress is for serviceability limit state.

From Fig. 3, it can be observed that linearity of the ascending branch of stress-strain curves increases with the increased compressive strength of the concrete. The deviation of the curves from the straight line is reported to be the indication of the development of cracks in the concrete. For a concrete of strength about 60 MPa, the deviation takes place at about 60% of the ultimate strength. In case of concrete of strength in the range of 25 to 40 MPa the ascending branch deviates at about 40% of the ultimate strength. Although the high strength concretes is reported to be brittle but high-strength reinforced concrete possess flexural ductility at least comparable to (may be even higher) that of the normal-strength concrete. Hence, there seems no reason to restrict the value of σ_{cbc} to 11.5 MPa.

AASHTO (1977), clause no. 1.5.26, does not put any such limit and it recommends σ_{cbc} as 40% of 28 day cylinder strength i.e. approximately 32% of the cube strength.

Thus, it is recommended that the value of σ_{cbc} may be taken as 33% of 28 day cube strength up to 60 MPa.

For still higher strength concretes, this permissible value of σ_{cbc} may be much more conservative estimate.

4.2.1.2 Modulus of elasticity

The values of modulus of elasticity of various grades of concrete tabulated in IRC: 21 - 1987 (1997) may be represented by the expression

$$E_c = 5700\sqrt{f_{ck}} \quad \text{MPa}$$

Modulus of elasticity of normal-strength concrete is little depend upon the type of the aggregates used in producing it. Whereas physical and mineralogical properties of the aggregates are the important factors affecting the modulus of elasticity of high-strength concrete. In high-strength concrete, the strength of cement matrix is in general comparable to that of the aggregates and in case of ultra-high-strength concrete the cement matrix may be even stronger than the aggregates used in concrete. **Neville (1997)** reported that modulus of elasticity of high-strength concrete was also depend upon the bond between the aggregate and surrounding hardened cement paste. This is why a number of expressions for modulus of elasticity of high-strength concrete have been proposed by different investigators in different regions.

Fig. 7 shows all the important expressions for modulus of elasticity along with the experimental results available from various references. It can be observed that the expression suggested by IRC: 21 - 1987 (1997) overestimated the modulus of elasticity of concrete of compressive strength higher than 40 MPa.

All the experimental data shown in figure are best represented by the expression suggested by Shah and Ahmad (1985).

$$E_c = 3.3945 \times 10^{-5} \times W^{2.5} \times {}^{0.325}\sqrt{f_c'}$$

For normal weight concrete W = 2400 Kg/m^3

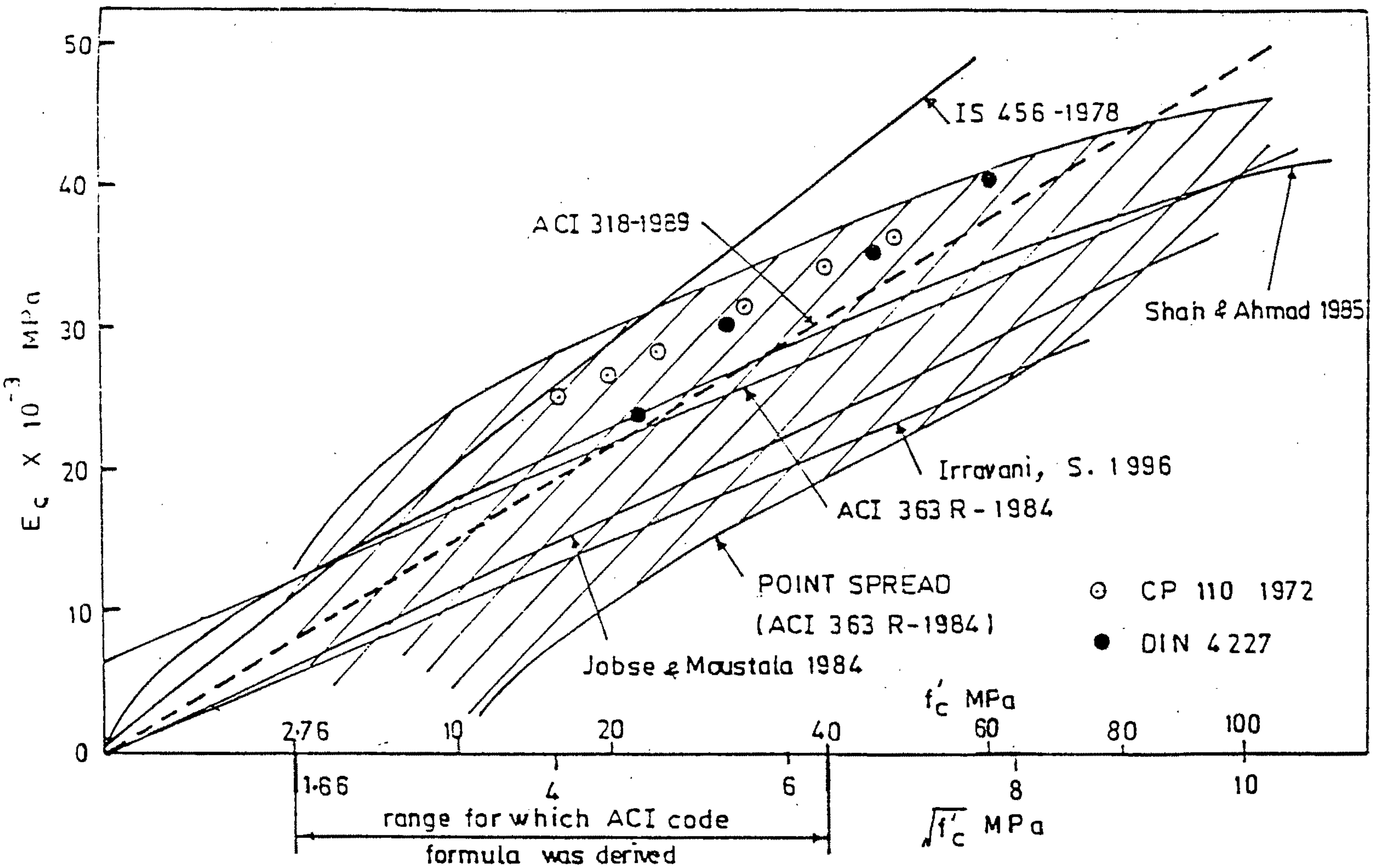


Fig. 7 SECANT MODULUS OF ELASTICITY VERSUS CONCRETE STRENGTH (PLAIN, NORMAL WEIGHT CONCRETE).

Taking cylinder strength (fc') = 0.8 × concrete cube Strength (fck)

$$E_c = 8908.57 \times \sqrt[0.325]{fck} \quad \text{MPa}$$

$$Ec \approx 8910 \times \sqrt[0.325]{fck} \quad \text{MPa}$$

Thus, following expression is recommended for calculating the modulus of elasticity of normal weight concrete in the strength range from 20 to 100 MPa.

$$Ec = 8910 \times \sqrt[0.325]{fck} \quad \text{MPa}$$

4.2.2 Clause number 304.7.2 maximum permissible shear stress

The expression for maximum permissible shear stress (τ_{max}) in slabs as proposed in IRC: 21 - 1987 (1997) is

$$\tau_{max} = 0.7 \times fck \leq 2.5 \text{ MPa}$$

The above clause is seemed to base on the concept that shear capacity of reinforced concrete members do not increase indefinitely with the increase in the concrete compressive strength because aggregate interlock account for a lower proportion of the shear strength of the section as the concrete compressive strength increases.

The reduction in shear resistance due to aggregate interlock with the increase in concrete compressive strength is true. But overall shear resistance of the concrete is reported to increase with the increase in compressive strength. This is because the influence of tensile strength of concrete on its shear strength becomes more and more prominent in comparison to aggregate interlock as the concrete compressive strength increases. The reduction in water-binder ratio required to produce high-strength concrete results in a stiffer cement matrix and hence the increased tensile strength of the concrete.

Fig.8 shows the continuous increase in the shear strength of concrete with the increase in its compressive strength. The shear strength of the concrete is not the linear function of its compressive strength as assumed in the code. It is more appropriate to assume the variation of shear strength of concrete as approximately proportional to square root of its compressive strength. Thus restricting the maximum permissible shear stress of concrete in slabs to 2.5 MPa does not seem to be justified. For one way slab action article no. 1.5.26 of AASHTO (1977) recommends a maximum permissible shear stress as given below. It allows continuous increase of concrete shear stress with the increase in its compressive strength.

$$\tau_{\max} = 0.3675\sqrt{f_{ck}}$$

The above expression is a little more conservative in comparison to the one proposed by IRC: 21 - 1987 (1997), but the values predicted by the two expressions are comparable.

Thus, the following expression is recommended for analytically finding the maximum permissible shear stress of concrete in the strength range from 20 to 60 MPa.

$$\boxed{\tau_{\max} = 0.3675\sqrt{f_{ck}}}$$

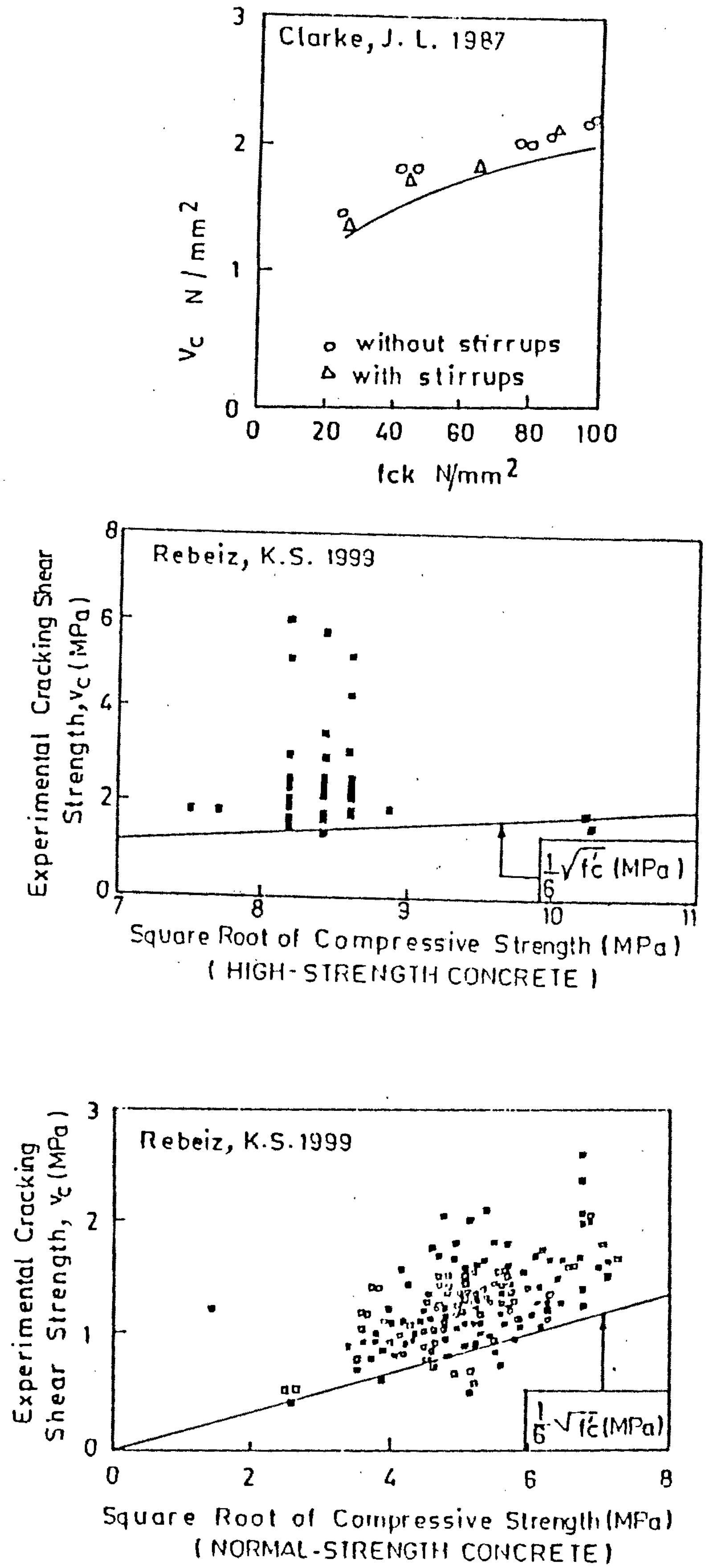


Fig. 8 SHEAR CAPACITY OF VARIOUS STRENGTH CONCRETE BEAMS

4.2.3 Clause number 304.7.3 permissible shear stress

The code recommends that slabs and similar members may be designed without shear reinforcement if the shear stress (τ) is less than the permissible shear stress (τ_c) given by the expression:

$$\tau_c = K_1 \times K_2 \times \tau_{co}$$

Where K_1 and K_2 are parameters which take into account the effect of the depth and the longitudinal reinforcement of the slab and τ_{co} is the basic permissible shear stress.

Within the practical range of reinforcement and the thickness of the deck slab, the product of K_1 and K_2 varies between 0.948 to 1.035.Therefore, it may be taken as 1 (refer Appendix A). τ_{co} may be approximated by the expression given below to represent its tabular values given in IRC: 21 - 1987 (1997).

$$\tau_{co} \approx 0.016 f_{ck} \leq 0.50 MPa$$

Similar to $\tau_{\max}$, the τ_{co} should also not be represented as linear function of concrete strength and its value should increase indefinitely (at least up to the upper limit of high-strength concrete i.e. about 100 MPa) with the increase in the concrete compressive strength.

AASHTO (1977), article no. 1.5.26, assumed the permissible shear stress in slabs as proportional to the square root of the concrete compressive strength and is allowed to increase with the increase in concrete compressive strength.

$$\tau_{co} = 0.0706\sqrt{f_{ck}} \quad \text{MPa}$$

The values of τ_{co} for a given strength of concrete calculated using the above expression are comparable to the respective values tabulated in IRC: 21 - 1987 (1997).

Thus, the following expression is recommended for calculating the values of τ_{co} in the strength range from 20 to 60 MPa.

$$\tau_{co} = 0.0706\sqrt{f_{ck}} \quad \text{MPa}$$

4.2.4 Clause number 305.19 minimum reinforcement in slabs

The IRC: 21 - 1987 (1997) code recommends a minimum area of reinforcement in slabs in either direction as 0.12 and 0.15% of its gross area for S-415 and S-240 grades of steel respectively. The above provision is for temperature and shrinkage stresses. To take care of stresses due to differential shrinkage in cast-in-situ slab of composite construction, the minimum reinforcement in slab in longitudinal direction of bridge should be 0.2%.

Freyermuth & Alami (1997) reviewed CEB/FIP model code (1990). They reported a uniform provision for beams and slabs with minimum reinforcement as 0.15 and 0.25% for S-400 and S-220 grades of steel respectively. AASHTO (1977) and Freyermuth & Alami (1997) suggested that the minimum area of reinforcement for S 400 grade steel should be 0.2%. ACI 318 (1989) also recommends that the minimum area of steel in slabs should be about 0.2%.

According to ACI363R (1984) specific heat, diffusivity, thermal conductivity and coefficient of thermal expansion of higher strength concrete (up to about 60 MPa) have been found to fall generally within the usual range for lower strength concrete. The shrinkage of high-strength concrete is reported to be similar to that for lower-strength concrete.

Thus, to cover a wider range of concrete strength, it is recommended that minimum area of reinforcement in slabs should be taken as 0.2% and 0.3% of the gross area for S-415 and S-240 grades of reinforcing steel respectively for concrete strength up to 60 MPa.

4.3 Proposed Revisions in IRC: 18 - 1985 (1997)

In comparison to IRC: 21 - 1987 (1997), the IRC: 18 - 1985 (1997) covers a higher range of concrete strengths (up to about 60 MPa). At present in India the maximum grade of concrete in practice in prestressed concrete bridges is in the neighbourhood of 70 MPa. The production of concrete of compressive strength of about 100 MPa has now become commercially viable. If not exhaustive, sufficient information on most of the mechanical properties and short term behaviour of high-strength concrete is available in research literature. This information may be used to update the design considerations of the codes to include high-strength concrete applications in various type of structures. One of the aims of present investigation is to study the cost effectiveness of using high-strength concrete in place of normal-strength concrete in highway bridges over a wide range of span from 20m to maximum structurally possible as per design requirements of various relevant IRC codes. Thus, concrete used in the prestressed girders in the present investigations covers strength range from 35 to 90 MPa.

Like IRC: 21 - 1987 (1997), IRC: 18 -1985 (1997) also discourages the use of higher strength concrete by stagnating the values of various permissible stresses for concrete compressive strength beyond 55 MPa. Only those provisions of this code are critically reviewed which are related to the post-tensioned girders of slab-on-girder bridges. The proposed revisions in IRC: 18 - 1985 (1997) are of both the types discussed earlier and cover a concrete compressive strength from 30 to 100 MPa.

4.3.1 Clause number 7.1 permissible temporary stresses in concrete

This clause requires that the temporary compressive stress in the extreme fibre of concrete at full transfer shall not exceed $0.45\,f_{cj}$ (concrete compressive strength at the time of transfer of prestress), subject to a maximum value of 20 MPa. The temporary tensile stress in the extreme fibre is not allowed to exceed 10% of the permissible temporary compressive stresses i.e. $0.045\,f_{cj}$

The code indirectly discourages the use of concrete of compressive strength higher than about 60 MPa by restricting the value of temporary compressive stress at full transfer at 20 MPa.

High-strength concretes are reported to exhibit a higher strength gain at early ages as compared to lower-strength concretes. The ratio of 7 day to 28 day strength of high-strength concrete is in the range of 0.80 to 0.90 whereas for normal-strength concrete it is about 0.60 to 0.70. Moreover, high-strength concretes are reported to possess sufficiently high reserve strength (in terms of MPa) to attain even after 28 days of age.

The straight portion of the ascending branch of the stress-strain curves of high-strength concretes is reported to be between 70 to 85% of the ultimate strength. Some researchers even reported that ascending part of the stress-strain curve of high-strength concrete was almost linear up to the ultimate strength. Deviation of ascending branch from straight line shows the beginning of the development of cracks. Thus, high-strength concretes can temporarily support much higher stresses in comparison to normal-strength concrete without initiation of the development of cracks. Hence, permissible temporary stresses in compression in high-strength concretes should be even more than 0.45 f_{cj} .

Thus, we observed that it is not only unjustified to limit the value of permissible temporary compressive stress to 20 MPa but also the expression suggested by the code to calculate its value seems to be very much conservative for high-strength concretes.

Assuming a factor of safety of about 1.25 on the linear part (about 75% of ultimate strength) of high-strength concrete, the permissible temporary compressive stress may be taken as 0.60 f_{cj} .

From literature reviewed it is clear that the indirect tensile strength of concrete do not increase linearly with the increase in its compressive strength. From Figs.10 &, 11, it can be observed that tensile strength may be better represented as proportional to square root of the compressive strength. It was also reported that for normal-strength concrete, the indirect tensile strength was approximately 10% of the compressive strength but at higher strength a lesser value was observed.

ACI 318 (1989) recommended and Hassanian & Loov (1999) used the following value of the permissible temporary tensile stress at transfer for designing the bridge girder.

Permissible tensile stress at transfer of prestress

$= 0.5\sqrt{f'_{cj}}$ (cylinder strength)

$= 0.447\sqrt{f_{cj}}$ (cube strength)

From Fig. ~~10~~ 11, it is clear that lower bound of the experimental data of normal- and high-strength concretes may be given by the expression suggested by ACI 318 (1989). Taking a factor of safety of 1.25 on the values represented by lower bound expression, the permissible temporary tensile stress at prestress transfer may be given as

$= (7.5 \sqrt{fc'}) \div 1.25$ psi

$= 0.5 \sqrt{f'_{cj}}$ MPa (cylinder strength)

$= 0.447 \sqrt{f_{cj}}$ MPa (cube strength)

Thus, permissible temporary stresses at full transfer may be given by the following expressions.

I) Compressive stress

$= 0.45 f_{cj}$ for $f_{cj} < 60\,MPa$

$= 0.60 f_{cj}$ for $f_{cj} \geq 60\,MPa$ up to 100 MPa

II) Tensile stress

$= 0.447\sqrt{f_{cj}}$ MPa (where f_{cj} is the cube strength at jth day)

4.3.2 Clause number 7.2 permissible stresses in concrete under service loads

The code recommends that compressive stresses in concrete during service shall not exceed $0.33 f_{ck}$ and no tensile stress shall be permitted.

High-strength concretes are reported to gain considerable strength above and beyond 28 day design requirements with the passage of time. While the percentage gain of strength of high-strength concrete from 7 days to 90 days may be more or less equal to the concretes in lower strength ranges, but the order of magnitude of the strength gain in MPa is actually higher.

The ratio of sustained load strength to short term strength of high-strength concrete was reported to be higher than that for normal-strength concrete and this ratio was observed to increase with the increase in compressive strength of concrete.

Thus for high-strength concrete permissible compressive stress during service may be increased without reducing the reserve strength in MPa because it posses higher reserve strength after 28 day age and also it can be subjected to higher ratio of sustained load to short term strength.

Taking the factor of safety of 1.5 on the lower limit of linearity of ascending branch of high-strength concrete (about 70% of ultimate strength), the permissible compressive stress for high-strength concrete may be given as

$$= 0.70 f_{ck} \div 1.5$$

$$= 0.45 f_{ck}$$

It was also recommended by many investigators that higher permissible stresses may be adopted for high-strength concrete. The tensile strength of high-strength concrete was reported to be higher in comparison to normal-strength concrete. Thus to take full advantage of high-strength concrete, tensile strength must be permitted in prestressed girders of the bridge.

Unlike the IRC: 18 (1985), many other codes and researchers like AASHTO (1977), ACI 318 (1989), Hassanian and Loov (1999) allowed the tensile stresses equal to $0.5\sqrt{fc'}$ ($0.447\sqrt{fcj}$) MPa in prestressed girders during service loads.

The rate of gain of tensile strength of high-strength concrete is higher at early ages as compared to its compressive strength. In comparison to normal-strength concrete high-strength concrete posses higher reserve strength (in MPa) at 28 day age. Jobse and Moustafa (1984) preferred to maintain the reserve strength constant in terms of MPa and not as a percentage of compressive strength of concrete.

For service load design also, let us take the factor of safety of 1.5 on lower bound of experimental data of flexural tensile strength.

Tensile stress allowed $= (0.5571\sqrt{fck} \div 1.5)\,MPa$

$$= 0.3714\sqrt{fck}\,MPa$$

Reserve strength $= 0.1857\sqrt{fck}\,MPa$

For fck = 60 MPa, reserve strength = 1.44 MPa

The permissible tensile and compressive stresses during transfer of prestress as well as under service loads may independently govern the design of the prestressed girders. From Appendix B, we may observe that the compressive stresses developed may govern the design while tensile stresses developed are within the permissible tensile stress and vice versa.

For concrete strength of 60 MPa and higher (up to 100 MPa) this reserve strength is proposed to be maintained.

Thus, following expressions are suggested for permissible stresses in concrete during service.

I) Compressive stress

$= 0.33 fck$ for $fck \langle 60\, MPa$

$= 0.45 fck$ for $100 \geq f_{ck} \geq 60\, MPa$

II) Tensile stress

$= 0.3714 \sqrt{fck}\, MPa$ for $fck \langle 60\, MPa$

$= 0.5571 \sqrt{fck} - 1.44\, MPa$ for $100 \geq f_{ck} \geq 60\, MPa$

4.3.3 Clause number 10.2 modulus of elasticity of concrete

The IRC: 18 - 1985 (1997) recommends the following expression for modulus of elasticity of concrete (Ec) if experimental value is not available.

$$Ec = 5700 \sqrt{fck}\, MPa$$

From the Fig. 7, it is clear that the above expression gives a very high estimate of E_c for concrete of compressive strength higher than 40 MPa.

The following expression is proposed for modulus of elasticity of concrete from 20 to 100 MPa.

$$E_c = 8910 \times \sqrt[0.325]{f_{ck}}\ MPa$$

4.3.4 Clause number 11.2 creep of concrete

The code tabulate the values of specific creep as a percentage of f_{ck} corresponding to different maturity of concrete at the time of stressing.

At any given stress-strength ratio and time after loading, the specific creep is reported to be greater the lower the strength of concrete at the time of loading. The specific creep and creep coefficient of high-strength concrete are reported to be much smaller than that of normal-strength concrete. From Fig. 9 it is evident that the difference in creep coefficient of normal- and high-strength concretes increases with time after loading.

Unfortunately no detailed information is yet available for specific creep strain at different maturity at the time of prestressing of high-strength concrete. The application of the value of specific creep strains of normal-strength concrete to high-strength concrete is not justified. The calculated creep loss in high-strength concrete by using the value of specific creep as per IRC: 18 - 1985 (1997) will be very high because it is normally subjected to comparatively much higher stresses.

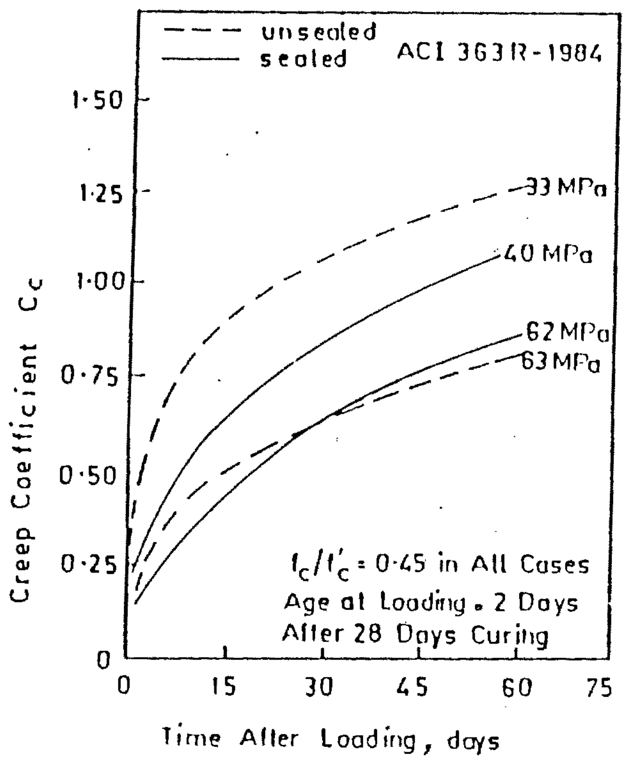


Fig. 9 RELATIONSHIP BETWEEN CREEP COEFFICIENT AND TIME OF SEALED AND UNSEALED CONCRETE SPECIMENS

The available information on creep of high-strength concrete has been compiled in ACI 363R (1984). It is reported that high-strength concrete has a specific creep only about 20% of that of the normal-strength concrete.

Thus, it is proposed that for concrete with compressive strength from 60 MPa to 100 MPa, the specific creep should be taken equal to 20% of the value reported in IRC: 18 - 1985 (1997).

Maturity of concrete at the time of stressing as a percentage of f_{ck}	Creep strain per 10 MPa
40	1.88×10^{-4}
50	1.66×10^{-4}
60	1.44×10^{-4}
70	1.22×10^{-4}
75	1.12×10^{-4}
80	1.02×10^{-4}
90	0.88×10^{-4}
100	0.80×10^{-4}
110	0.72×10^{-4}

4.3.5 Clause number 11.3 shrinkage of concrete

The code tabulates the values of the residual shrinkage strain at different ages of concrete ranging from 3 to 90 day at the time of prestressing.

The concrete of compressive strength of 60 MPa and higher may be economically produced by using mineral admixture (like silica fume & fly-ash etc.)

and high-range water reducers to get the desired workability with least possible amount of water.

The presence of silica fume is reported to increase the plastic shrinkage but the addition of high-range water reducers caused the shrinkage to reduce.

A small difference in shrinkage at early ages between the high-strength concrete and the normal-strength concrete was observed but the final shrinkage was reported to be little influenced by the strength of the concrete. Many laboratory and field studies have shown that shrinkage of high-strength concrete was similar to that of normal-strength concrete.

Since not much detailed information is available on shrinkage of high-strength concrete and the final shrinkage of the two concretes (high- and low-strength) is reported to be similar, it is proposed that the provisions of present code may be extended to high-strength concrete also.

The provisions of the code for shrinkage of normal-strength concrete may be used for high-strength concrete also.

4.3.6 Clause number 13 ultimate flexural strength

The IRC: 18 - 1985 (1997) proposes two expressions for the moment of resistance of the girder section under ultimate loads corresponding to the failures by yielding of steel and by the crushing of concrete. It avoids the compression failure by ensuring that the moment of resistance corresponding to the yielding of steel is less than that due to the crushing of concrete.

Most of the concrete codes do not explicitly cover concrete strength above 50-60 MPa. Expression for flexural strength of concrete sections adopted by ACI 318 (1989), IRC: 18 - 1985 (1997) and AASHTO (1977) are based on the investigations on concrete of strength up to about 60 MPa. All these expressions are reported to overestimate the strength of the sections failing in flexural compression. The stress-strain curve of concrete was idealised in different shapes by researchers to best represent their experimental data on of high-strength concrete members failing in flexural compression. These different idealised stress-strain curves are shown in Fig.4.

It is almost unanimously reported by many investigators that the existing codes expressions gave conservative estimate of the flexural strength of members failing by yielding of steel. This is because the independent variation in various stress block parameters was significant but their combined effect was compensative for members failing in flexural tension.

For under reinforced section, the actual shape of the compression block is of little importance so long as the internal lever arm to the compression resultant is close to the true value. Over reinforced beams are not allowed by both IRC:18 - 1985 (1997) and AASHTO (1977). It is reported by many investigators that ACI 318 (1989)/AASHTO (1977) expressions were conservative lower bound to the experimental data. Thus, ACI 318 (1989)/AASHTO (1977) expressions still seems to be valid for prestressed concrete bridge girders.

The expression corresponding to avoidance of compression failure recommended by IRC: 18 - 1985 (1997) is more conservative in comparison to

AASHTO (1977) expression. IRC: 18 - 1985 (1997) recommends the following expression for T-beam failing by the crushing of concrete.

$$M_{ult} = 0.176 b d^2 f_{ck} + 0.533(B_f - b)\left(d_b - t/2\right) \times t \times f_{ck}$$

and the corresponding expression recommended by AASHTO (1977) is

$$M_{ult} = 0.20 b d^2 f_{ck} + 0.68(B_f - b)\left(d_b - t/2\right) \times t \times f_{ck}$$

Thus for non composite section IRC: 18 - 1985 (1997) expression may be used for high-strength concrete (up to 100 MPa) flexural members failing in tension.

The above IRC: 18 - 1985 (1997) expression corresponds to maximum permitted value of 'effective reinforcement ratio' [IS: 1343 - 1980 (1981)] or 'the reinforcement index' [ACI 318 (1989)] defined as

$\omega = \dfrac{A_p f_p}{b d f'_{ck}}$ where

$f'_{ck} = f_c'$ cylinder strength in ACI code

$\quad = f_{ck}$ cube strength in IRC and IS codes

A_p = Area of prestressing steel

f_p = Stress in prestressing steel

The maximum allowable value of reinforcement index assure that the prestressing steel will be slightly into its yield range at failure. AASHTO (1977)and ACI - 318 (1989) standards allow $\omega \leq 0.30$. Taking $f_c' = 0.8\ f_{ck}$, the value of ω corresponding to cube strength will be 0.375. IS: 1343 - 1980 (1981) recommends the maximum value of ω as 0.40 which is very close to the value permitted by AASHTO (1977) and ACI - 318 (1989) codes.

According to IS: 1343 - 1980 (1981), the value of x_u/d for post tensioning with effective bond and ω = 0.40, is 0.653 where

d = effective depth and

x_u = depth of parabolic stress block i.e. depth of neutral axis.

For the value of x_u/d equal to 0.653, the neutral axis will always lie in the web of T-beam girders used in bridges. Thus, compression zone will consists of cast-in situ slab of, in general, comparatively lower strength concrete and part of the prestressed web of high-strength concrete bridge girders.

Df_{ck} = Grade of concrete in cast-in-situ slab

f_{ck} = grade of concrete in prestressed girders

IRC: 18 - 1985 (1997) recommends the following expression for ultimate moment of resistance of a T-beam corresponding to failure by crushing of concrete.

$$M_{ult} = 0.176 b d^2 f_{ck} + 0.533(B_f - b)\left(d_b - t/2\right) \times t \times f_{ck}$$

In the above expression, if fck is taken as the concrete grade in deck slab i.e. $Dfck$ then the calculated value of M_{ult} will be an underestimation and if fck is taken as concrete grade in prestressed girder then the calculated value of M_{ult} will be an overestimation of the ultimate flexural strength corresponding to compression failure.

Bate and Bennett (1975) recommended that if the compression zone contained a part of the precast element also, the average compressive strength ($Efck$) computed by considering the areas of in-situ and precast concrete should be used in computing the compressive force.

Thus, weighted average compressive strength may be calculated by using the following expression.

$$Efck = \left\{Dfck \times t + fck\left(x_u - t\right)\right\} \div x_u$$

From Appendix C, it may be observed that the value of moment of resistance of the girder section is about 48% higher if Ef_{ck} is used in comparison to the use of $Dfck$ in the expression. Similarly, the moment of resistance of the girder may be as high as 300% if fck is used in comparison to the use of $Dfck$ in the expression. Thus, the use of the Ef_{ck} in the expression gives better estimation of the ultimate flexural strength corresponding to the compression failure.

Ultimate flexural strength of composite T-beams of bridge should be calculated by using the following expressions.

I) Failure by yield of steel

$$M_{ult} = 0.90 d_b \times A_s \times f_p$$

II) Failure by crushing of concrete

$$M_{ult} = 0.176 b\, d^2 Ef_{ck} + 0.533(B_f - b)\left(d_b - t/2\right) \times t \times Df_{ck}$$ where

$$Ef_{ck} = \left\{Df_{ck} \times t + f_{ck}\left(x_u - t\right)\right\} \div x_u$$

4.3.7 Clause number 14.1 ultimate shear design

The IRC: 18 - 1985 (1997) recommends the calculations for shear only for the ultimate load. At any section the ultimate shear resistance of the concrete alone, V_c is calculated for sections both uncracked and cracked in flexure, and if necessary, shear reinforcement should be provided.

For uncracked section (clause no. 14.1.2)

$$V_{co} = 0.67 bd \sqrt{f_t^2 + 0.8 f_{cp} f_t}$$

where f_t, maximum principal tensile stress (f_{sp}, splitting cylinder tensile strength)

$$= 0.24 \sqrt{f_{ck}}$$

For cracked section (clause no. 14.1.3)

$$V_{cr} = 0.037 b d_b \sqrt{f_{ck}} + \frac{M_t}{M} V \geq 0.1 b d \sqrt{f_{ck}}$$

where M_t, cracking moment

$$= \left(f_r + 0.8 f_{pt}\right) \frac{I}{y}$$

f_r (modulus of rupture strength) = $0.37\sqrt{f_{ck}}$

Shear reinforcement (clause no. 14.1.4) be provided if shear force $V \geq V_c/2$. Minimum shear reinforcement be provided if $V \leq V_c$. Minimum shear reinforcement must be capable of developing a shear stress of 0.4 MPa in concrete.

$$\frac{(A_{sv})\min}{S_v} \times \frac{0.87 f_{yv}}{b} = 0.4$$

When shear force V exceeds the V_c, shear reinforcement be provided such that

$$\frac{A_{sv}}{S_v} = \frac{V - V_c}{0.87 f_{yv} d_t}$$

The maximum shear strength V_s, provided by shear reinforcement was restricted indirectly by restricting the maximum shear force V (clause no. 14.1.5). The maximum permitted value of the maximum allowable shear stress $\tau_{\max}$ $\left(V/bd\right)$ is 5.5 MPa. The values of $\tau_{\max}$ are tabulated for different grades of concrete.

Shear phenomenon in concrete is a very complex one. Thus, most of the codes provide many empirical yet simple equations, based on the experiments on the specimens and structure, for predicting the shear strength of concrete and for detailing the shear reinforcement. These empirical equations do not posses required generality and can not be directly extended to new materials and to the new type of structures. Thus different investigators provided different approaches for safe and economical shear design of structural concrete. The ASCE - ACI committee 445 (1998) recommended that proposed regulatory standards of design procedure should be not only safe but correct in concept and simple to understand.

Procedure adopted by IRC: 18 - 1985 (1997) for shear design of prestressed concrete flexural members is similar to the one recommended by ACI 318 (1989) and AASHTO (1977). It is also very simple in application and easy to understand. Thus, for design purposes the existing procedure of IRC:18 - 1985 (1997) is recommended for application to high-strength concrete also.

The effect of aggregate interlock on shear capacity of high-strength concrete is not very significant as against for normal strength concrete where its contribution to shear capacity is considerable. The one of the major factors affecting the shear capacity of high-strength concrete is its tensile strength. The shear carried by the tensile stresses in concrete was reported to be a function of the longitudinal tensile strain. The application of prestressing force causes the reduction in longitudinal tensile strains and hence results in increased shear capacity of flexural members.

The ACI 318 (1989) expressions for shear design, also permitted by AASHTO (1977) for design of bridge girders, are less conservative in comparison to the similar expressions adopted by IRC:18 - 1985 (1997).

The test results of various investigators on high-strength concrete flexural members indicated that for slender beams, the ACI's beams shear strength equations were conservative for concrete strength range from 20 to about 100 MPa. But for deep beams, in which the effect of concrete strength on shear capacity became significant, the ACI equations highly underestimated the actual shear strength.

The present beam shear strength expressions recommended by IRC:18 - 1985 (1997) are more conservative in comparison to the expressions proposed by ACI 318 (1989). Thus the expressions suggested by IRC:18 - 1985 (1997) need modification to take care of the higher tensile strength of high-strength concrete.

Fig. 10 summarises all the available experimental data and various expressions for calculating the splitting cylinder tensile strength of concrete proposed by different investigators. It is observed that experimental data on high-strength concrete are best represented by the expressions proposed by ACI 363R (1984).

$$f_{sp} = 0.59\sqrt{f_c'}$$

$$= 0.528\sqrt{f_{ck}}$$

Taking factor of safety of 1.5 for ultimate load analysis, the permissible principal tensile stress f_t may be given by

$$f_t = \frac{f_{sp}}{1.5} = 0.352\sqrt{f_{ck}}$$

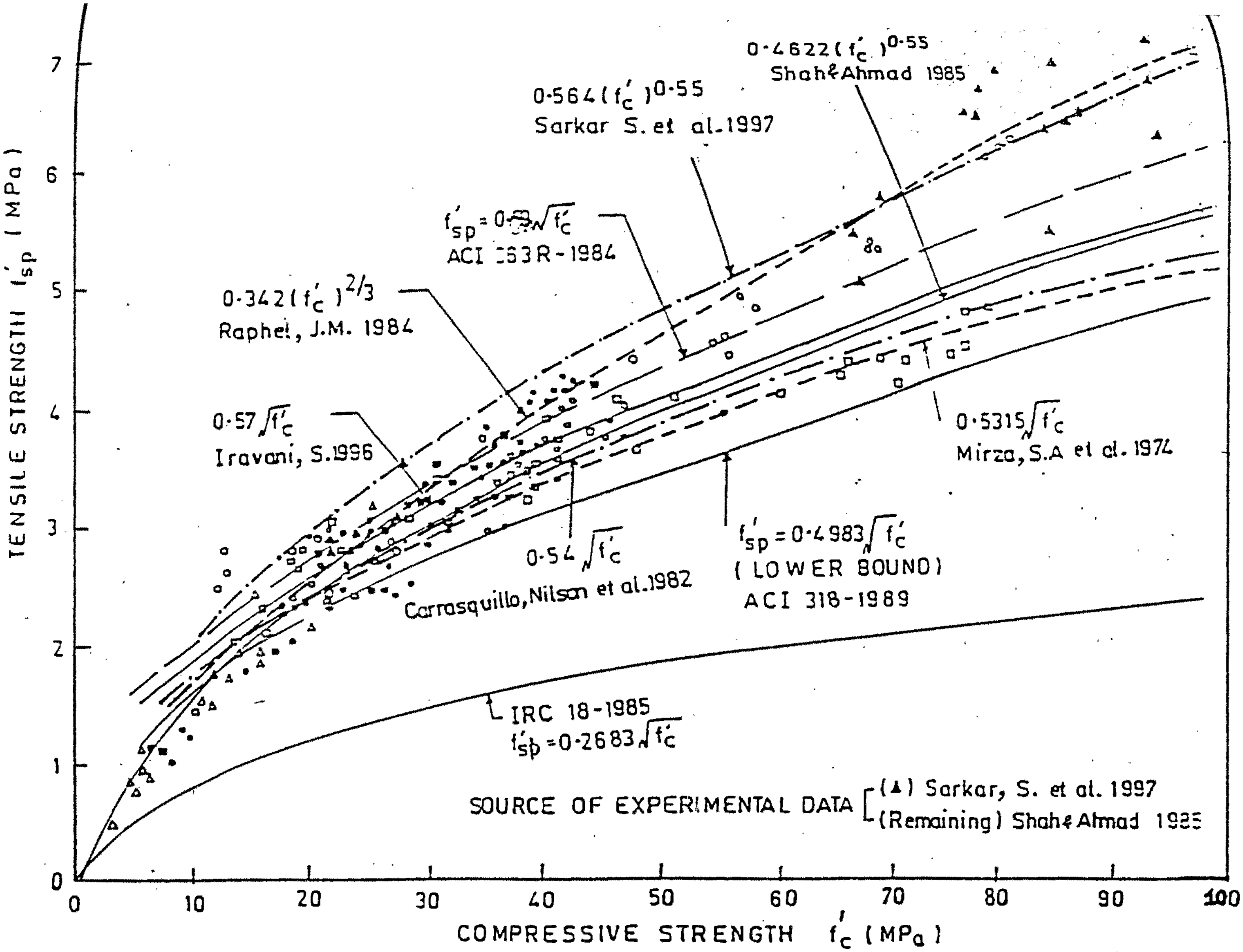


Fig. 10 SPLIT CYLINDER TENSILE STRENGTH OF PLAIN, NORMAL WEIGHT CONCRETE

This value is higher than the one $\left(0.24\sqrt{fck}\right)$ proposed by IRC:18 -1985 (1997) for normal strength concrete but less than the lower bound expression $\left(0.446\sqrt{fck}\right)$.

Thus, it is proposed that for the concrete of compressive strength of 60 MPa and higher, the permissible principal tensile stress be calculated by the expression given below.

$$f_t = 0.352\sqrt{fck}$$

The second part of the expression for V_{cr} i.e. $\frac{M_t}{M}V$ (shear required to cause the flexural crack at the point in question) dominates the shear capacity of the section cracked in flexure. The contribution of the first part i.e. $0.037bd_b$ (additional increment of shear required to change the flexural crack to a flexure-shear crack) is very marginal. Thus second part of the expression needs critical review in reference to high-strength concrete.

The available experimental data and various expressions for beam flexural tensile strength (f_r') are summarised in Fig. 11. It is visible that the beam flexural tensile strength of high-strength concrete is best represented by the expression suggested by Shah and Ahmad (1985).

$$f_r' = 0.4379 \times {}^{2/3}\sqrt[3]{f_c'}$$

$$= 0.38 \times {}^{2/3}\sqrt[3]{fck}$$

Thus, the permissible modulus of rupture strength of high-strength concrete may be obtained by

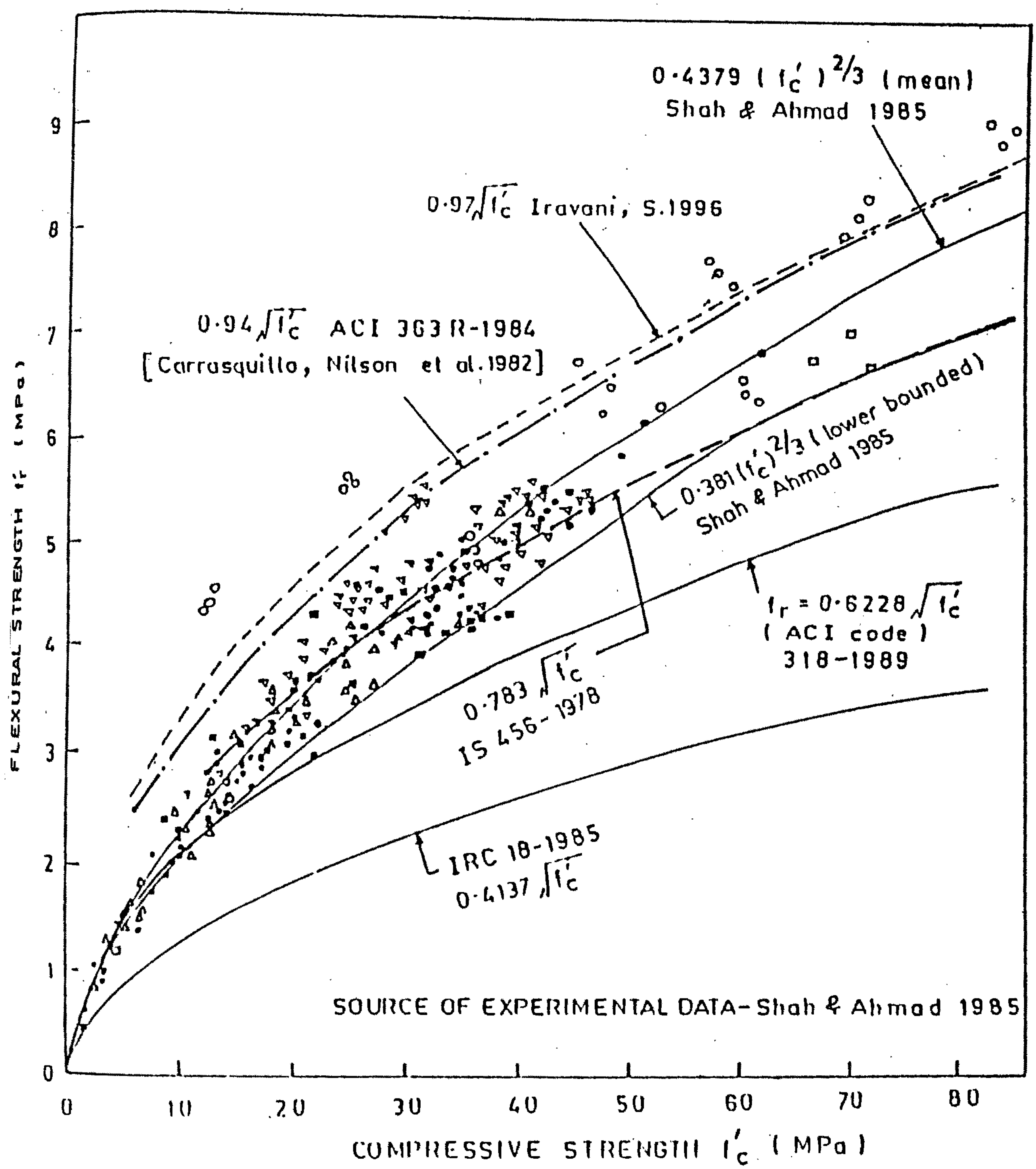


Fig. 11 BEAM FLEXURAL TENSILE STRENGTH OF PLAIN, NORMAL WEIGHT CONCRETE.

applying a factor of safety of 1.5.

$$f_r = \frac{0.38 \times {}^{2/3}\sqrt[3]{f_{ck}}}{1.5} = 0.25 \times {}^{2/3}\sqrt[3]{f_{ck}}$$

This value of permissible modulus of rupture strength of high-strength concrete is higher than the one $\left(0.37\sqrt{f_{ck}}\right)$ suggested by IRC:18 - 1985 (1997) for normal strength concrete.

<u>Thus, it is proposed that the permissible modulus of rupture strength of concrete of strength 60 MPa and higher may be found by using the expression given below.</u>

$$f_r = 0.25 \times {}^{2/3}\sqrt[3]{f_{ck}}$$

ACI 318 (1989) recommends the following expression for cracking shear strength v_c, ($v_c = V_c/bd$) of beams without shear reinforcement.

$v_c = 0.166\sqrt{f_c'}$ i.e. ($v_c = 0.1486\sqrt{f_{ck}}$)

Fig.8 shows the experimental results of normal- and high-strength concrete beams along with the plot of the above equation recommended by ACI 318 (1989). It is clear that the above equation gives a conservative estimate of cracking shear strength of concrete with compressive strength ranging from about 20 to 100 MPa.

Taking a factor of safety of 1.5, the permissible cracking shear stress of concrete may be given by the expression given below.

$$v_{cr} = \frac{0.1486\sqrt{f_{ck}}}{1.5} \approx 0.1\sqrt{f_{ck}}$$

$V_{cr} = 0.1bd\sqrt{f_{ck}}$ (equation recommended by IRC 18 - 1985)

The equation, for shear strength of concrete beams without shear reinforcement, proposed by ACI 318 (1989) gives very conservative estimate for high-strength concrete beams. This fact is clear from Fig. 8. The various investigators proposed different expressions for shear strength of high-strength concrete which took into account the effect of reinforcement ratio and shear span-depth ratio. These were reported to better predict the shear strength of high-strength concrete beams without shear reinforcement. But all these equations are not no simple in form as that of ACI 318 (1989) equation which may directly be applied to bridge girders under complicated loading.

ACI 318 (1989) equation for shear strength of concrete beams without web reinforcement is a conservative lower bound for high-strength concrete beams up to about 100 MPa. Thus, it is proposed that for high-strength concrete also (60 to 100 MPa) the minimum value of V_{cr} should be calculated by the existing expression $(0.10\sqrt{f_{ck}}$ MPa$)$.

Both ACI 318 (1989) and IRC:18 - 1985 (1997) require that minimum shear reinforcement must be capable of developing a shear stress of 0.345 and 0.40 MPa respectively. From the literature reviewed it is clear that the minimum shear reinforcement corresponding to a shear stress of 0.40 MPa is highly unsatisfactory for high-strength concrete flexural members.

The accuracy of the expressions for $V(= V_s + V_c)$, V_{co} and V_{cr} was reported to be greatly influenced by the provision of minimum shear reinforcement. The failure of beams at shear force which was even less than the calculated cracking shear strength were reported if inadequate minimum shear reinforcement was provided.

AASHTO (1977) allowed the use of expressions for V, V_{co} and V_{cr} recommended by ACI 318 (1989) [expressions similar to those recommended by IRC:18 - 1985 (1997)] provided the minimum shear reinforcement was capable of mobilising a shear stress of 0.689 MPa which is double the value recommended by ACI 318 (1989).

ACI committee 318 (1988) proposed the revision in ACI 318 (1983) for extending the applicability of expressions for V, V_{co} and V_{cr} to concrete of strength higher than 69 MPa. The concrete of strength higher than 69 MPa could be used in calculating the V, V_{co} and V_{cr} of beams provided the minimum shear reinforcement is equal to $f_c'/34.5$ (MPa) times, but not more than three times the amount permitted by ACI 318 (1983)i.e. corresponds to mobilisation of shear stress of 0.345 MPa.

It is possible that even the above discussed provision of minimum shear reinforcement may not be adequate if the values of V, V_{co} and V_{cr} are calculated by using the values of f_t and f_r (redefined for high-strength concrete) calculated as per the expressions proposed in this investigation.

The minimum shear reinforcement should aim to provide adequate reserve capacity beyond diagonal cracking i.e. it should prevent brittle failure upon first diagonal cracking. Ozcebe et al. (1999) reported that Turkish Code TS 500 (1983) used this concept. The expression for minimum shear reinforcement was derived by equating the diagonal cracking strength of the beam to the shear strength of the same beam with shear reinforcement. The diagonal cracking strength was magnified by a factor of 1.5 i.e. the shear reinforcement was capable of mobilising 50% of the shear resistance offered by concrete in girder section.

The results of maximum shear stress and the shear resistance offered by the girder section (without shear reinforcement) for various combinations of parameters of bridge superstructure are tabulated in Appendix D. From these results it may be observed that:

I) The failure due to excessive shear is possible even if the girder section is safe in ultimate flexure.

II) The minimum shear reinforcement provided according to existing provisions of IRC:18 - 1985 (1997) is not capable of mobilising the shear equal to 50% of the cracking shear resistance of the section. Thus, minimum shear reinforcement will not offer sufficient reserve strength against shear failure. This situation is possible when

$(V - V_c) \leq 0.40$ MPa (i.e. minimum shear reinforcement is provided), but

$V_c/2 > 0.40$ MPa.

The expression for minimum shear reinforcement may be developed by using the expression proposed in the present investigation for V_{co} and V_{cr} . Let,

V_c = lesser of the two V_{co} and V_{cr} as proposed by IRC:18 - 1985 (1997).

For safety, the shear strength of beam with shear reinforcement V , should be greater than the diagonal cracking i.e.

$V \geq$ diagonal cracking strength

$\Rightarrow (V_s + V_c) \geq$ diagonal cracking strength

For safety and sufficient reserve strength, the diagonal cracking strength should be magnified by a factor of 1.5. The value of magnification factor is chosen as 1.5 on the basis of the investigation by Ozcebe et al. (1999). They reported that the beams that had minimum shear reinforcement corresponding to magnification factor equal to 1.5 satisfied both the crack width and reserve strength criterion. In these beams the crack width was approximately equal to 0.1 to 0.2mm (sufficiently less than the maximum limit of crack width of 0.3mm at the stage when a diagonal crack fully develop), and reserve strength (Vu / Vc) ratios were greater than 1.5.

Diagonal cracking strength = $1.5 \times V_c$. Thus,

$$V_s + V_c = 1.5V_c$$

$$\Rightarrow V_s = 0.5V_c$$

$$\Rightarrow \frac{(A_{sv})_{\min} \times 0.87 f_{yv} \times d_t}{S_v} = 0.5V_c$$

$$\Rightarrow (A_{sv})_{\min} = \frac{0.575V_c \times S_v}{f_{yv} \times d_t}$$

For extra safety the condition imposed by AASHTO (1977) should also be applied. Thus, the minimum shear reinforcement should also be capable of mobilising a shear stress of 0.689 MPa. Hence,

$$(A_{sv})_{\min} = \frac{0.575V_c \times S_v}{f_{yv} \times d_t} \text{ but} \geq \frac{0.689 \times b_w \times S_v}{0.87 f_{yv}}$$

The tabulated values of maximum shear stress $\tau_{\max}$ (clause no.14.1.5), may be represented by a expression given below.

$$\tau_{\max} = 0.75\sqrt{f_{ck}} \leq 5.5\,MPa$$

It has already been shown that the shear strength of concrete beams increased with increase of the concrete strength. Thus, limiting the value of τ_{max} to 5.5 MPa is not justified if concrete of strength higher than 60 MPa is used.

Bate and Bennett (1975) reported that CP 110 (1972) suggested an expression for maximum permissible shear stress ($\tau_{max} = 0.75\sqrt{f_{ck}}$) without applying any restriction on its maximum value.

ACI 318 (1989) restricts the maximum permissible shear stress indirectly. It recommends that shear taken by steel should not exceed the value given by the expression shown below.

$$v_s = \frac{V_s}{bd} = 8\sqrt{f_c'}psi = 0.5942\sqrt{f_{ck}}\,Mpa$$

As has already been shown that $v_c = 0.1486\sqrt{f_{ck}}$

Thus,

$$v(\tau_{max}) = v_c + v_s$$

$$v(\tau_{max}) = 0.7422\sqrt{f_{ck}}$$

The above value of τ_{max} is comparable to the expression representing the tabulated values of τ_{max} recommended by IRC:18 - 1985 (1997).

Thus, it is recommended that the maximum allowable shear stress should be found by using the expression $\tau_{max} = 0.75\sqrt{f_{ck}}$ for both normal- and high-strength concretes (up to 100 MPa) without restricting the maximum value of τ_{max} in MPa.

Thus following is proposed regarding the ultimate shear design (clause no. 14.1) of prestressed concrete bridge girders of concrete strength ranging from about 30 to 100 MPa.

I)Clause No. 14.1.2

Maximum principal tensile stress f_t,used in expression for calculating shear strength of section uncracked in flexure should be taken as:

$$f_t = 0.24\sqrt{f_{ck}} \ldots\ldots\ldots\ldots f_{ck} \langle 60\,MPa$$

$$f_t = 0.352\sqrt{f_{ck}} \ldots\ldots\ldots\ldots f_{ck} \geq 60\,MPa \quad \text{(up to 100 MPa)}$$

II) clause No. 14.1.3

Modulus of rupture strength for calculating the cracking moment used in the expression for shear strength of section cracked in flexure should be taken as:

$$f_r = 0.37\sqrt{f_{ck}} \ldots\ldots\ldots f_{ck} \langle 60\,MPa$$

$$f_r = 0.25 \times {}^{2/3}\sqrt[3]{f_{ck}} \ldots\ldots\ldots f_{ck} \geq 60\,Mpa \quad \text{(up to 100 MPa)}$$

Minimum value of cracking shear strength be taken as(up to 100 MPa concrete):

$$V_{cr} \geq 0.1bd\sqrt{f_{ck}}$$

III) Clause No. 14.1.4

Minimum shear reinforcement be taken as:

$$\left(A_{sv}\right)\min = \frac{0.575V_c \times S_v}{f_{yv} \times d_t} \geq \frac{0.689b_w \times S_v}{0.87f_{yv}}$$

IV) **Clause No. 14.1.5**

The maximum permissible shear stress in longitudinal girders be calculated using the expression given below (up to 100 MPa concrete).

$$\tau_{max} = 0.75\sqrt{f_{ck}}$$

4.3.8 Clause number 15.3 minimum longitudinal reinforcement

According to IRC:18 - 1985 (1997) , the minimum longitudinal reinforcement in beams shall not be less than 0.25% and 0.15% of the gross sectional area for S240 and S415 grade steel respectively, where the specified grade of concrete is less than 45 MPa. In case the grade of concrete exceeds 45 MPa, the provision shall be increased to 0.3% and 0.18% respectively.

It is also suggested by many investigators that a unified approach be adopted for beams and slabs (reinforced or prestressed) and minimum reinforcement be taken as 0.3% and 0.2% for S240 and S415 grade of steel respectively.

The code takes into account the strength of steel for recommending the minimum reinforcement. The principle behind the adoption of minimum tensile reinforcement is that the tensile capacity of steel should exceed the tensile capacity of the concrete when it cracks. The tensile capacity of the concrete is approximately proportional to the square root of its strength. Thus when concrete strength increases, the tensile capacity of the members corresponding to development of cracks also increases, so the minimum reinforcement requirement must increase.

Freyermuth and Alami (1997) recommended that 0.15% be replaced by 0.2%. According to Clarke (1987), the 0.15% and 0.30% was adopted on the basis that 40 MPa was the maximum grade of concrete for reinforced concrete. It was suggested that quoted minimum area of tension reinforcement should be multiplied by $\sqrt{(f_{ck}/40)}$ when high-strength concrete is used.

Thus, minimum area of tension reinforcement be provided according to the expressions given below (as % of gross sectional area).

I) For S415 Grade Steel

$$(A_{st})_{\min} = 0.2\% \ldots\ldots\ldots\ldots f_{ck} \leq 40\,MPa$$

$$(A_{st})_{\min} = 0.2 \times \sqrt{\frac{f_{ck}}{40}}\% \ldots\ldots\ldots\ldots f_{ck} \rangle 40\,MPa \text{ (up to 100 MPa)}$$

II) For S240 Grade Steel

$$(A_{st})_{\min} = 0.3\% \ldots\ldots\ldots\ldots f_{ck} \leq 40\,MPa$$

$$(A_{st})_{\min} = 0.3 \times \sqrt{\frac{f_{ck}}{40}}\% \ldots\ldots\ldots\ldots f_{ck} \rangle 40\,MPa \text{ (up to 100 MPa)}$$

4.4 Conclusions

The various reasons which warrant the necessity of revisions of the existing IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) codes provisions if high-strength concrete is to be used in bridges have been discussed in the introduction part of this chapter. The present study has shown that the existing IRC codes provisions can not be extended to high-strength concrete. The codes not only discourage the use of high-strength concrete but also render the bridge structurally uneconomical.

The various properties of concrete affecting the response and safety of structure under loads are critically discussed and compared with the available experimental data on normal- and high-strength concretes.

The IRC codes provisions are critically compared with the ACI and AASHTO codes and the results of various investigations on the high strength concrete highway bridges. Some other codes reported to have incorporated certain amendments for effective use of high-strength concrete, are also reviewed.

The necessary amendments related to the basic properties of high-strength concrete and the structural design considerations for highway bridges in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) are proposed. These amendments are expected to overcome the deficiency in these codes related to the application of high-strength concrete in highway bridges. Results obtained after using recommended amendments are comparable to the information available in literature.

5. RESULTS AND DISCUSSION

5.1 Introduction

The results of the theoretical investigation are discussed and compared with those available in the literature. The limitations of the theoretical analysis are examined so that the areas for further investigation and scope of application could be identified. The economic aspects of application of the high-strength concrete in composite prestressed concrete simply supported slab-on-girder bridges are also discussed.

The cost of carrying the given load can be substantially reduced by using high-strength concrete. But for effective use of high-strength concrete in highway bridges, the possible reduction of cost by adjustment in the various parameters of the bridge superstructure is discussed.

The limitations of the existing IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) in dealing with the application of high-strength concrete are identified and critically discussed. The modifications in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) are proposed in chapter 4, considering the difference in the basic properties of high-strength and normal-strength concretes. The results of these proposed amendments are discussed.

The parameters governing the design of composite prestressed concrete bridges are discrete variables. A method of optimization of the slab-on-girder bridges is developed and presented in chapter 3. The resulted optimized designs are tabulated and compared with the design of an existing bridge (made available by U. P. State Bridge corporation Limited, Lucknow).

The results of the economic studies are presented in the form of tables and graphs which may be used for economical preliminary design of composite prestressed concrete simply supported slab-on-girder bridges over a span range of 20m to 50m.

5.2 Parametric Study

For parametric study, the section of the precast girder used is Washington 100S standard section and the value of SRATIO is kept as 1.

5.2.1 Forces in longitudinal girders

The forces considered in the longitudinal girders are due to only IRC class AA and class A loadings excluding forces due to self and superimposed load.

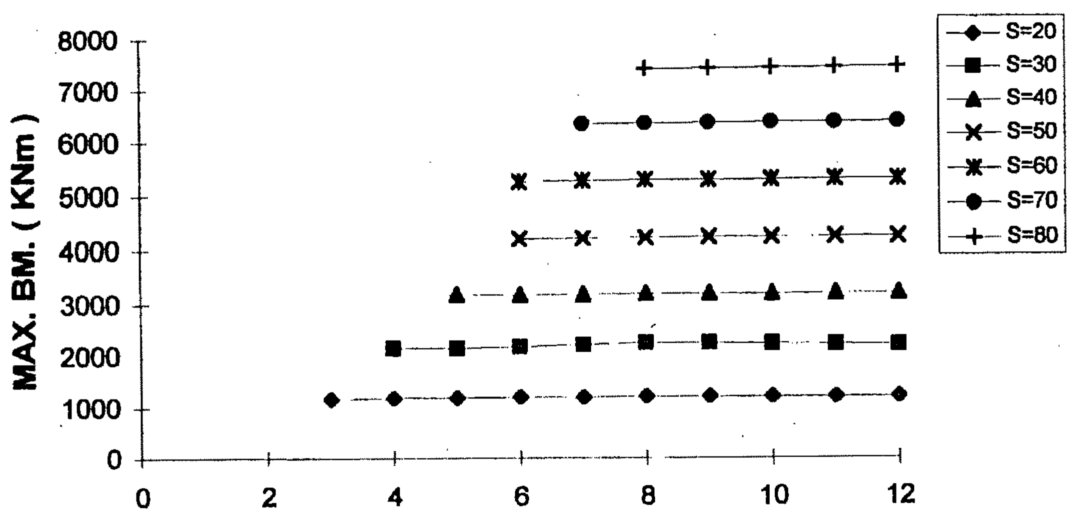
5.2.1.1 Variation of maximum bending moment

I) Maximum bending moment v/s number of cross beams

Fig. 12 shows the variation of maximum BM in longitudinal girder with the increase in number of cross beams from 3 to 12. Over a range of number of longitudinal girders from 2 to 5 and span range from 20 to 80m, the maximum BM in the longitudinal girders remains practically unchanged with the increase in number of cross beams.

The main purpose of the provision of the transverse beams in a bridge deck system is to increase its transverse rigidity. This helps in better load distribution among the longitudinal girders. Appendix E shows the variation of the flexural parameter of Morrice - Little. It may be observed that over the span range

MAX. BM. V/S NCB FOR NG=4 CLASS A LOADING, SRATIO = 1



NO. OF CROSS BEAMS

FIG.12

of 20 to 80m and number of cross beams 3 to 12, the flexural parameter of Morrice - Little varies from 0.40 to 0.10. From Figs. 13-16 it can be observed that over this range of variation of flexural parameter (0.1-0.4), there is a maximum change of about 5% (for NG = 5) in the value of distribution coefficient (Morrice - Little). Thus, in the practical range of variation of number of cross beams, a very small variation in the load distribution and hence in the value of maximum bending moment in the longitudinal girder is expected.

II) Maximum bending moment v/s number of longitudinal girders

From Fig. 17 it may be observed that the maximum bending moment in the longitudinal girder reduces with the increase in number of longitudinal girders. The rate of decrease of bending moment with the number of longitudinal girders is higher in the lower range number of longitudinal girders than in the higher range of number of longitudinal girders. For span = 20m, NG = 2, NCB = 12 and class AA loads, the maximum bending moment reduces by 31.26% when number of longitudinal girders increases from 2 to 3 whereas this reduction is only 8.07% when number of longitudinal girders increases from 4 to 5. It may also be observed that the rate of decrease of bending moment with the increase in number of longitudinal girders reduces as the span of the bridge increases.

The reduction in the value of bending moment in longitudinal girders with the increase in number of longitudinal girders depends upon the change in the load distribution efficiency of the bridge deck system. From Fig. 18 it may be observed that the reduction in the value of distribution factor is 33.47% and 24.85% with the change in number of girders from 2 to 3 and 3 to 4 respectively.

DF V/S NCB NG=2 SPAN=25M CLASS AA LOADING (TRACKED) MMAE-EG=MORRICE MAX.ECC. EXTERIOR GIRDER, HMAE-EG=HENDRY MAX ECC. EXTERIOR GIRDER, DES-DF DESIGN DISTRIBUTION FACTOR, SRATIO = 1

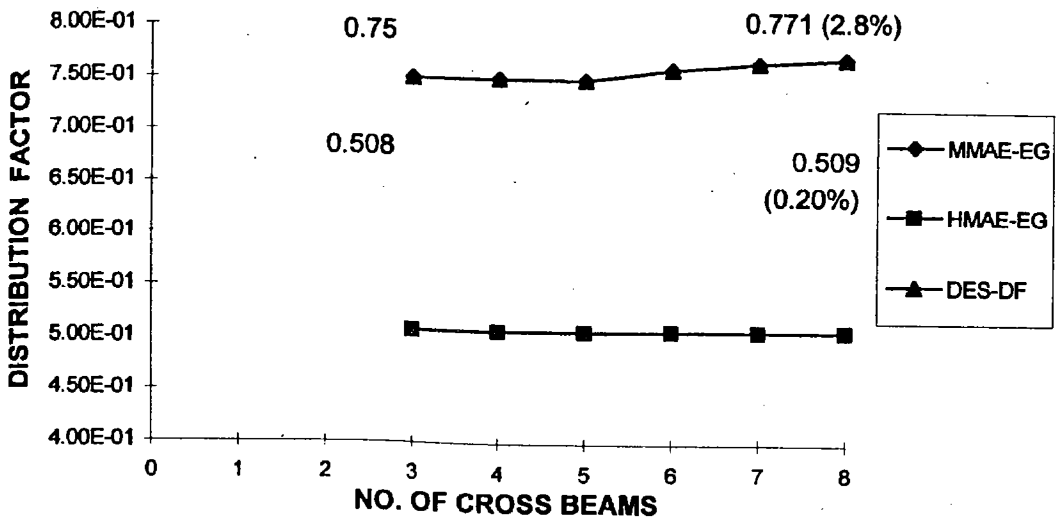


Fig. 13

DF V/S NCB NG=3 SPAN=25M CLASS AA LOADING (TRACKED), MMIE-IG= MORRICE MIN. ECC. INTERIOR GIRDER, HMIE-IG= HENDRY MIN.ECC. INTERIOR GIRDER, SRATIO = 1

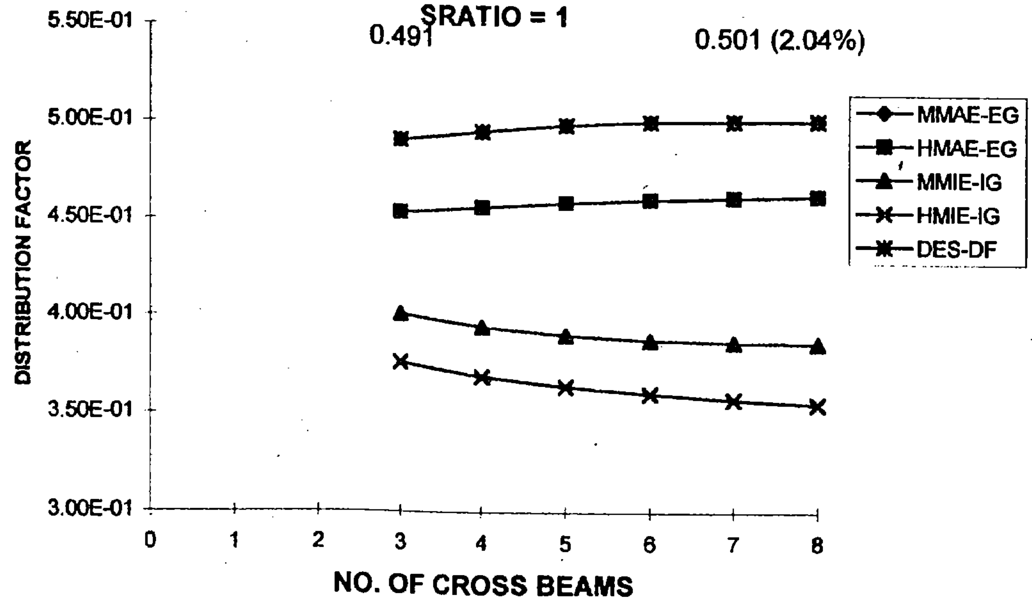


Fig. 14

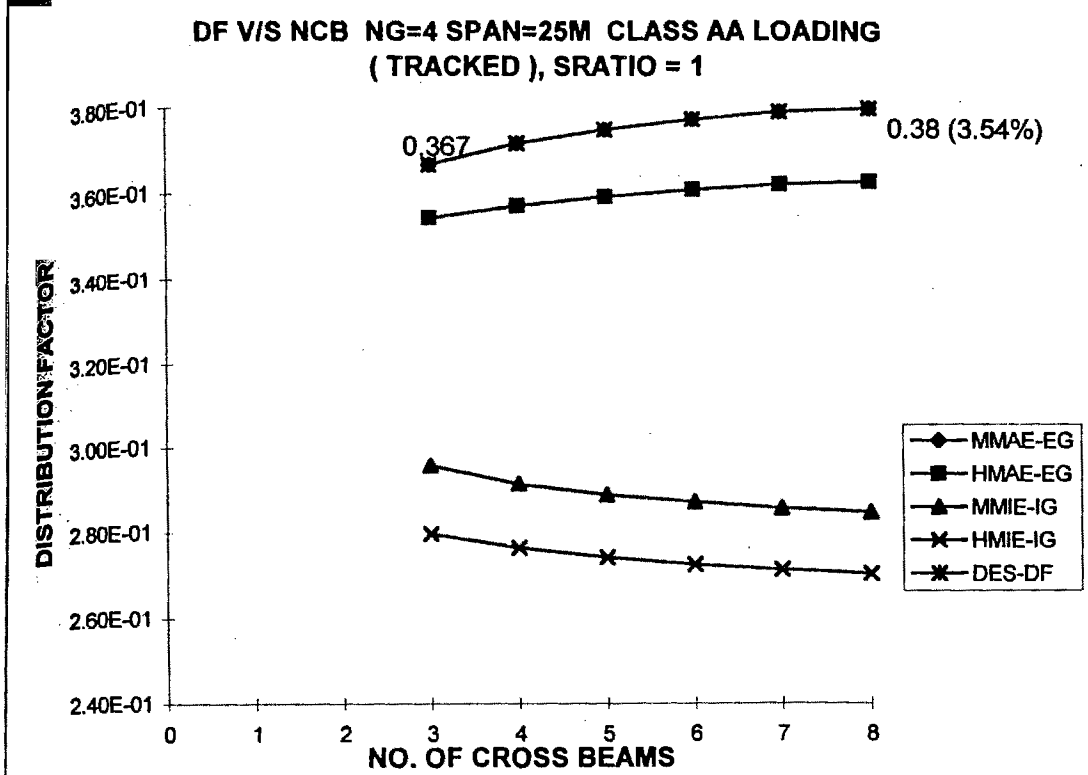

DF V/S NCB NG=4 SPAN=25M CLASS AA LOADING (TRACKED), SRATIO = 1
DISTRIBUTION FACTOR
3.80E-01
3.60E-01
3.40E-01
3.20E-01
3.00E-01
2.80E-01
2.60E-01
2.40E-01
0.367
0.38 (3.54%)
0
1
2
3
4
5
6
7
8
NO. OF CROSS BEAMS
MMAE-EG
HMAE-EG
MMIE-IG
HMIE-IG
DES-DF

Fig. 15

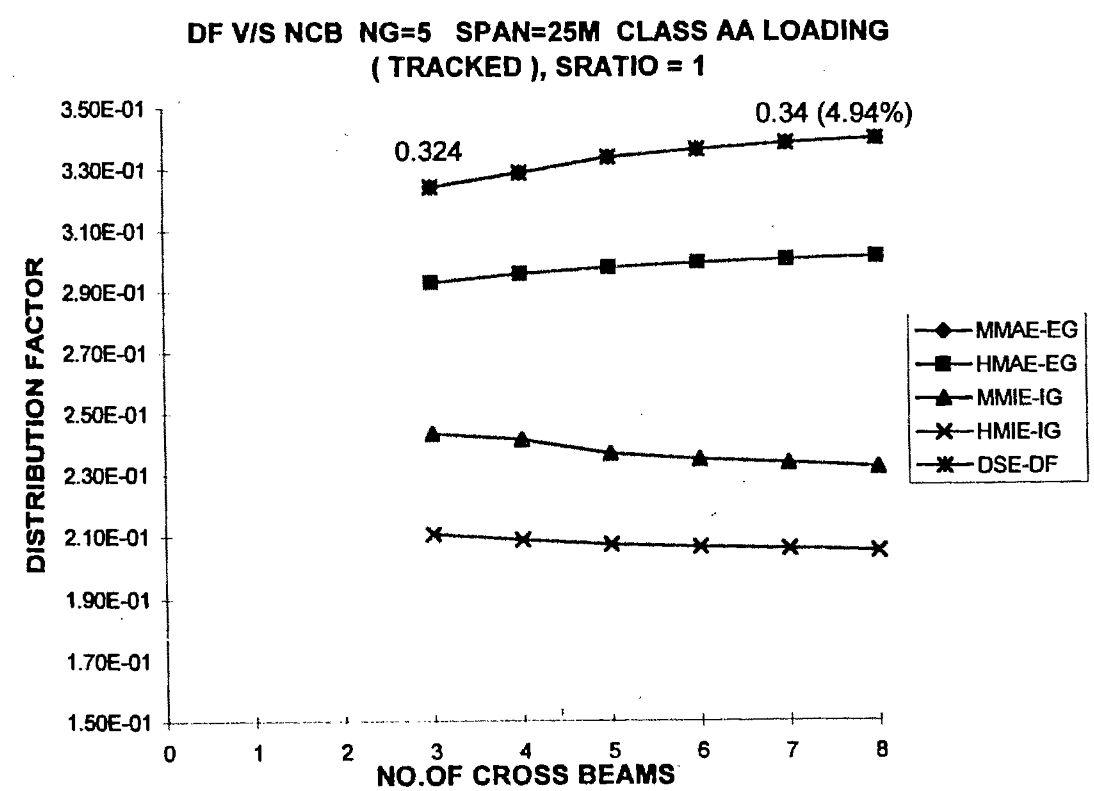

DF V/S NCB NG=5 SPAN=25M CLASS AA LOADING (TRACKED), SRATIO = 1
DISTRIBUTION FACTOR
3.50E-01
3.30E-01
3.10E-01
2.90E-01
2.70E-01
2.50E-01
2.30E-01
2.10E-01
1.90E-01
1.70E-01
1.50E-01
0.324
0.34 (4.94%)
0
1
2
3
4
5
6
7
8
NO.OF CROSS BEAMS
MMAE-EG
HMAE-EG
MMIE-IG
HMIE-IG
DSE-DF

Fig. 16

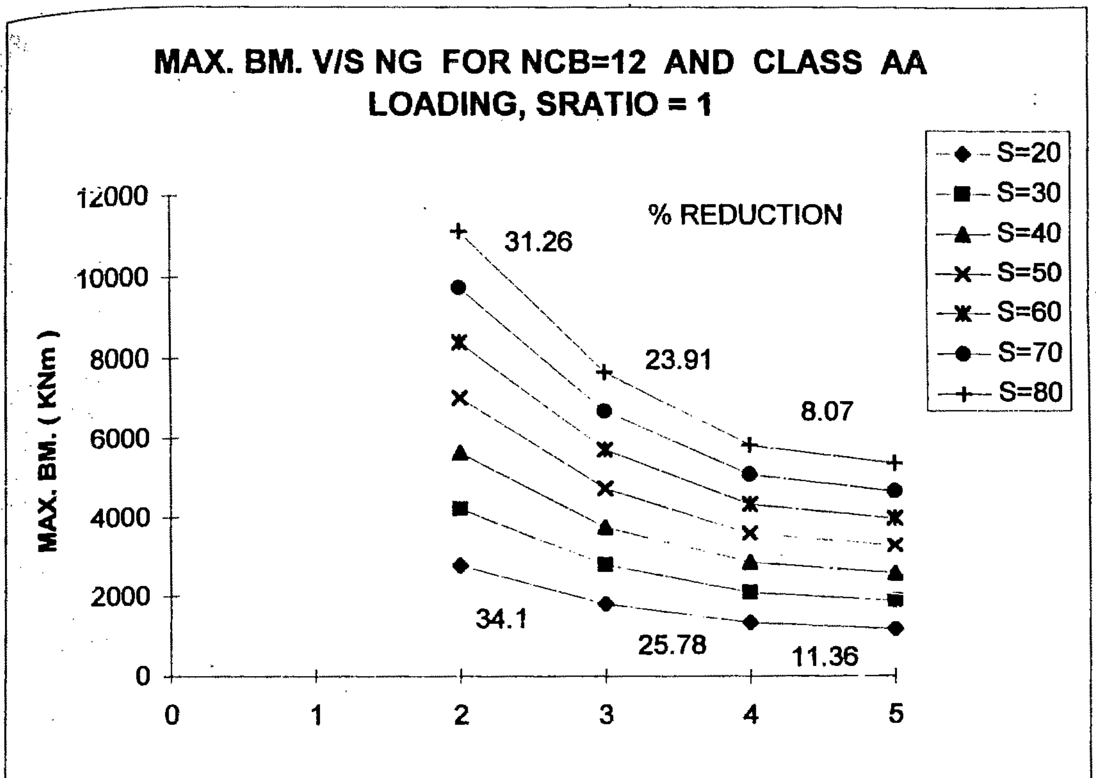

MAX. BM. V/S NG FOR NCB=12 AND CLASS AA LOADING, SRATIO = 1
MAX. BM. (KNm)
NO. OF GIRDERS
% REDUCTION
31.26
23.91
8.07
34.1
25.78
11.36
S=20
S=30
S=40
S=50
S=60
S=70
S=80
12000
10000
8000
6000
4000
2000
0
0
1
2
3
4
5

FIG.17

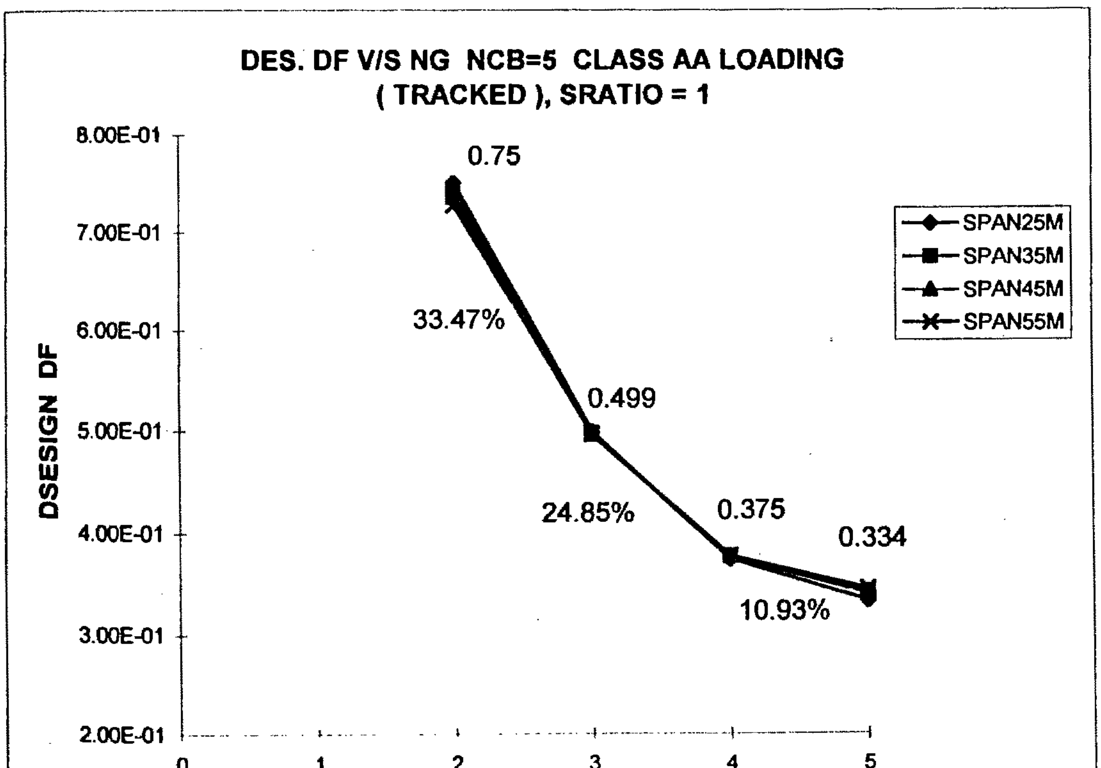

DES. DF V/S NG NCB=5 CLASS AA LOADING (TRACKED), SRATIO = 1
DSESIGN DF
NO. OF GIRDERS
0.75
33.47%
0.499
24.85%
0.375
0.334
10.93%
SPAN25M
SPAN35M
SPAN45M
SPAN55M
8.00E-01
7.00E-01
6.00E-01
5.00E-01
4.00E-01
3.00E-01
2.00E-01
0
1
2
3
4
5

Fig. 18

But this change is only 10.93% when the number of girders changes from 4 to 5. Thus, the rate of reduction of bending moment with the increase in number of girders should be higher in the smaller range of number of girders than in higher range of number of girders.

III) Maximum bending moment v/s span of the bridge

From Fig. 19 it may be observed that percentage increase in the magnitude of the maximum bending moment over a given range of the span is higher for larger number of longitudinal girders in comparison to the smaller number of longitudinal girders. For two and five girders system under IRC class AA loads, the maximum bending moment increases to 4.805 and 5.155 times respectively when the span increases from 20m to 80m. Fig. 20 shows the variation of design distribution factor with the increase in span of the bridge. This difference in increase in the value of the maximum bending moment is because for the two and three girders systems, the value of the distribution factor reduces with the increase in the span whereas for four and five girders systems the distribution factor increases with the increase in the span.

In the shorter span bridges, the value of the maximum bending moment in the girder is critical under IRC class AA loads whereas for longer span bridges it is under IRC class A loads. This is because over a span range of 20 to 80m, the total load on the span under IRC class AA loads remains unchanged whereas under IRC class A loads it increases with the increase in span length.

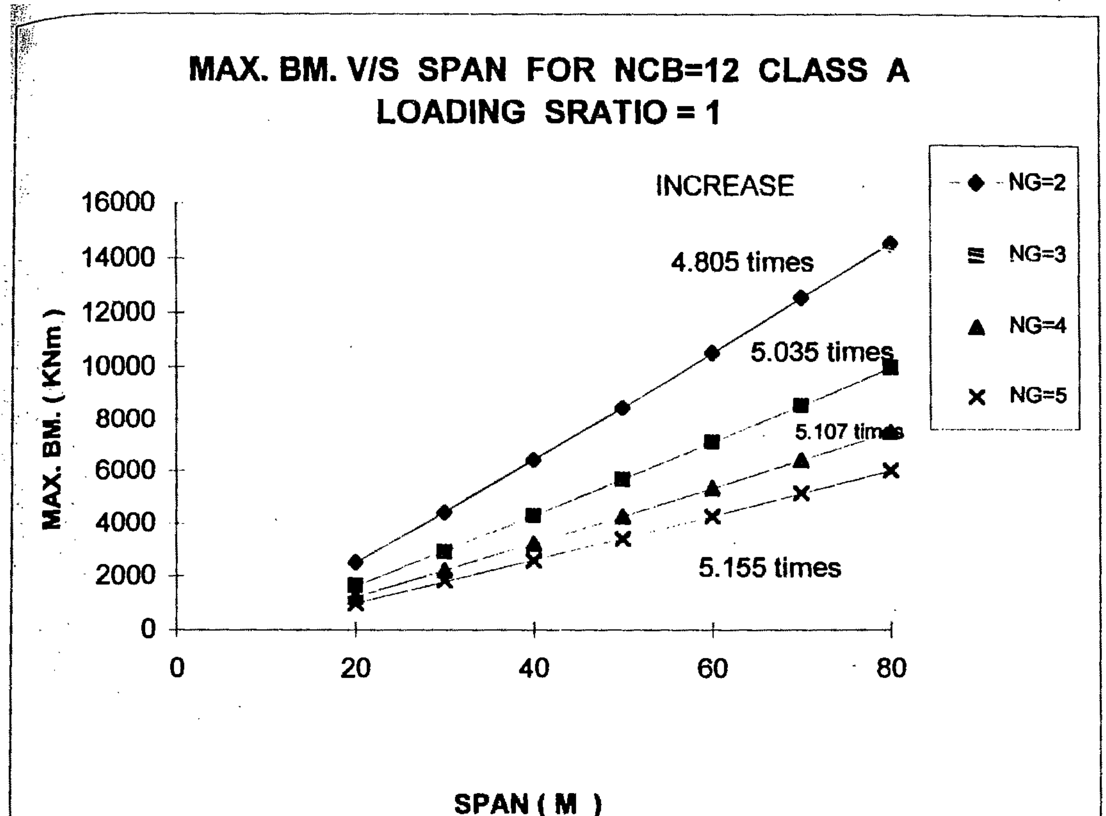

MAX. BM. V/S SPAN FOR NCB=12 CLASS A
LOADING SRATIO = 1
INCREASE
4.805 times
5.035 times
5.107 times
5.155 times
NG=2
NG=3
NG=4
NG=5
MAX. BM. (KNm)
16000
14000
12000
10000
8000
6000
4000
2000
0
0
20
40
60
80
SPAN (M)

FIG.19

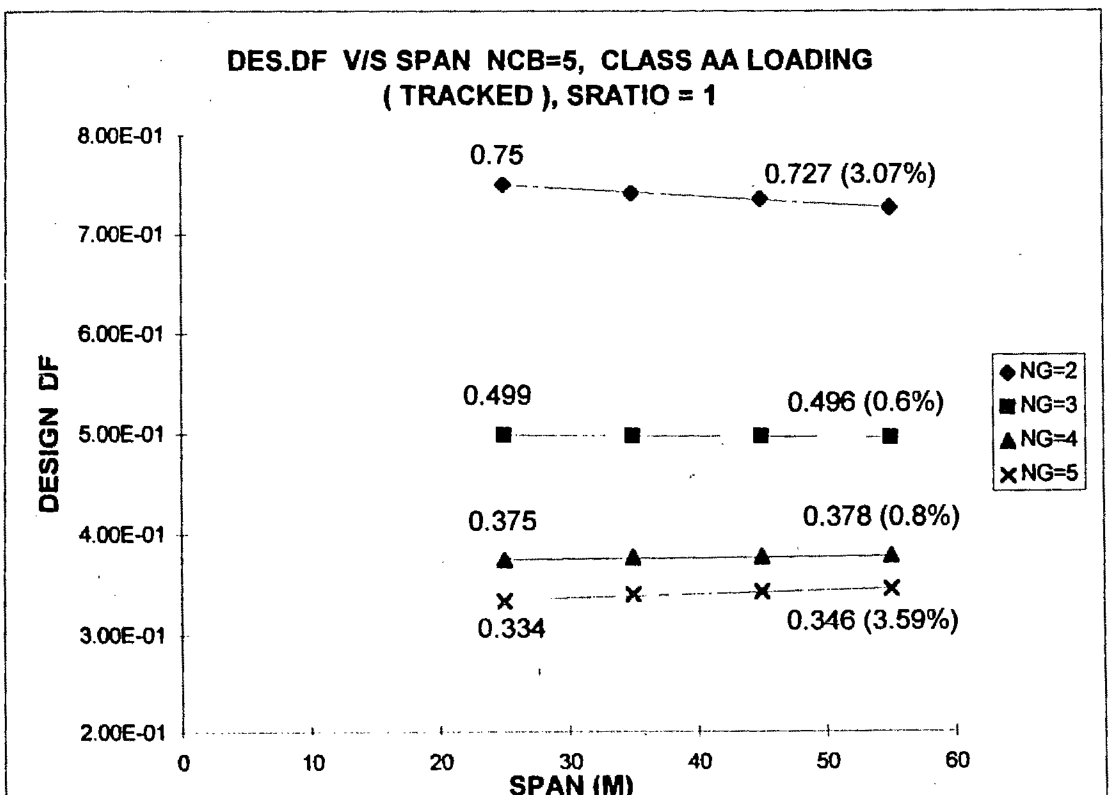

DES.DF V/S SPAN NCB=5, CLASS AA LOADING
(TRACKED), SRATIO = 1
0.75
0.727 (3.07%)
0.499
0.496 (0.6%)
0.375
0.378 (0.8%)
0.334
0.346 (3.59%)
NG=2
NG=3
NG=4
NG=5
DESIGN DF
8.00E-01
7.00E-01
6.00E-01
5.00E-01
4.00E-01
3.00E-01
2.00E-01
0
10
20
30
40
50
60
SPAN (M)

Fig. 20

IV) Maximum bending moment v/s SRATIO

From Fig. 21 it may be observed that the value of the maximum bending moment in the longitudinal girders reduces with the increase in the value of SRATIO. It may also be observed that decrease in BM in longitudinal girders with the increase in the value of SRATIO is of higher magnitude in the longer span bridges with smaller number of longitudinal girders. The longitudinal flexural stiffness per unit width of the bridge increases with the increase in the value of SRATIO. This increase in longitudinal flexural stiffness per unit width of the bridge is of higher magnitude when the number of the girders is small. The increased longitudinal flexural stiffness per unit width of the bridge causes the better load distribution among the girders and hence the reduction in the maximum bending moment in longitudinal girders.

V) Equivalent UDL for maximum bending moment

The calculated equivalent UDL for maximum bending moment in the girder under IRC class AA and class A loads and for different combinations of the various parameters of the bridge superstructure have been tabulated in the present investigation. A few of these tables are given in the Appendix F.

The variation of the equivalent UDL for maximum bending moment with various bridge superstructures (except the span of the bridge) is similar to the variation of the maximum bending moment. From the comparison of Figs. 19 and 22 it may be observed that the equivalent UDL for maximum bending moment reduces whereas the maximum bending increases with the increase in the span.

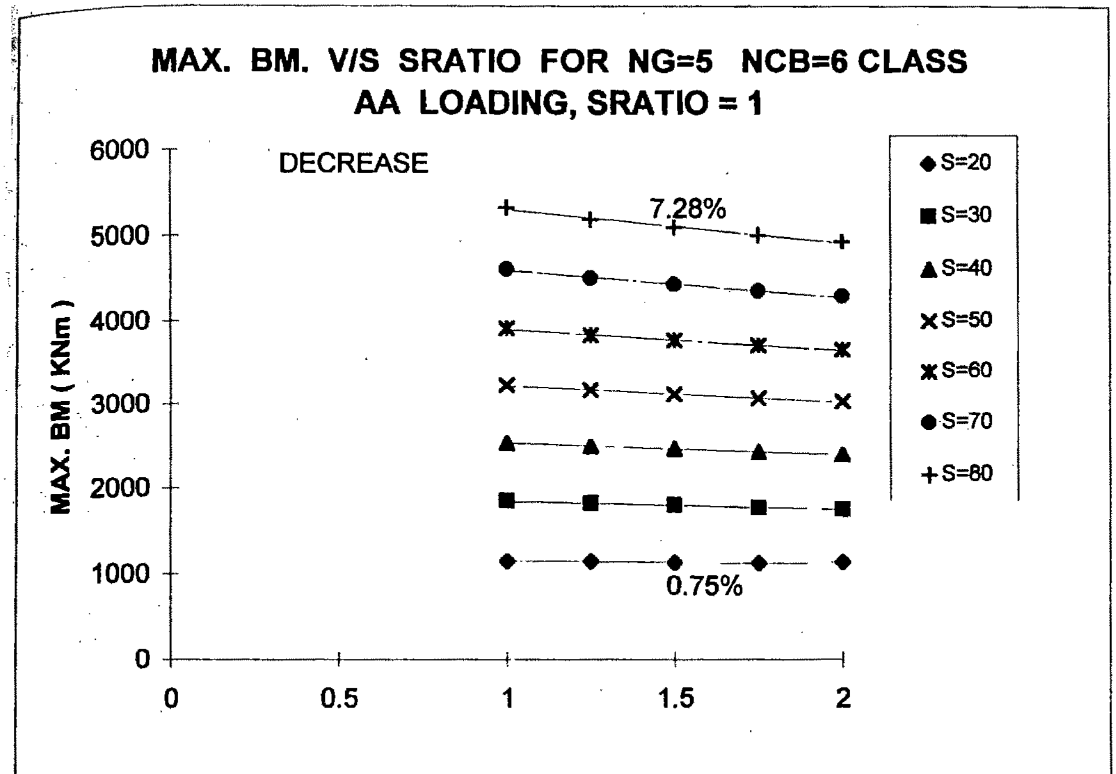

MAX. BM. V/S SRATIO FOR NG=5 NCB=6 CLASS AA LOADING, SRATIO = 1
DECREASE
7.28%
0.75%
MAX. BM (KNm)
XC / XK (SRATIO)
S=20
S=30
S=40
S=50
S=60
S=70
S=80

Fig. 21

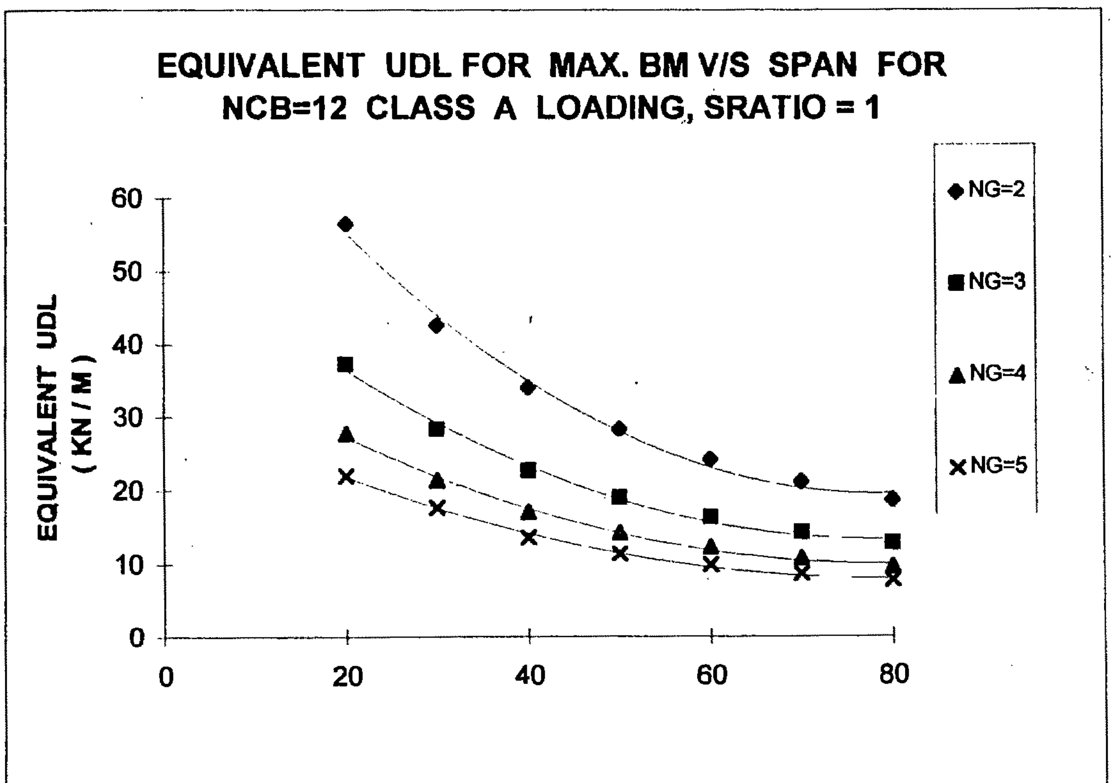

EQUIVALENT UDL FOR MAX. BM V/S SPAN FOR NCB=12 CLASS A LOADING, SRATIO = 1
EQUIVALENT UDL (KN / M)
SPAN (M)
NG=2
NG=3
NG=4
NG=5

Fig. 22

5.2.1.2 Variation of maximum shear force

I) Maximum shear force v/s number of cross brams

The comparison of Figs. 12 and 23 shows that similar to the bending moment, the maximum shear force in the girders is little dependent on the number of cross beams.

II) Maximum shear force v/s number of girders

Figs. 24 and 25 show the variation of the maximum shear force in longitudinal girder in the region outside 5.5m from the supports with the increase in number of longitudinal girders. The comparison of Figs. 17 and 24 shows that the variation of the maximum shear force in longitudinal girder is similar to the variation of the maximum bending moment with the increase number of longitudinal girders. The rate of variation of the maximum shear force with the number of longitudinal girders reduces as the span increases. Similarly, the rate of variation of maximum shear force with the number of longitudinal girders is higher in the smaller range of the number of longitudinal girders than in the higher range of the number of longitudinal girders.

From Fig. 26 it may be observed that the variation of the maximum shear force in the longitudinal girder in the region within 5.5m from the supports of the girder under IRC class AA and class A loads with the variation in the number longitudinal girders is very much different from the variation of the maximum bending moment in the longitudinal girder under similar conditions. The maximum shear force in the longitudinal girder under IRC class AA loads is of higher

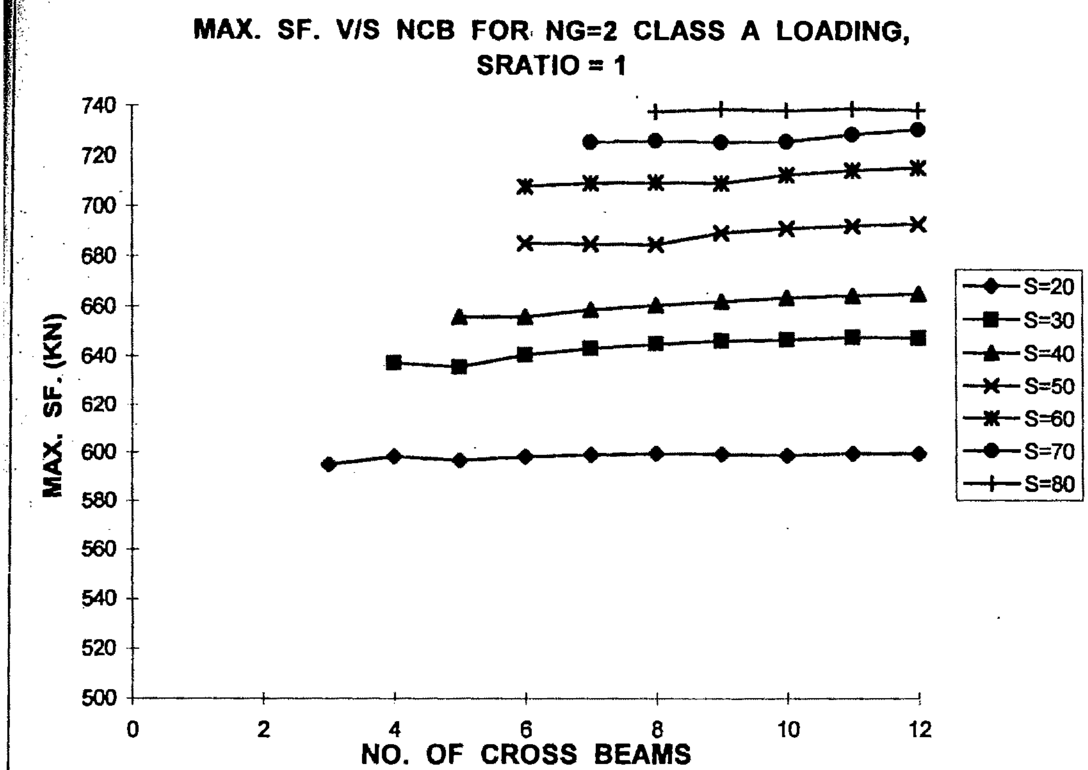

MAX. SF. V/S NCB FOR NG=2 CLASS A LOADING, SRATIO = 1
MAX. SF. (KN)
740
720
700
680
660
640
620
600
580
560
540
520
500
0
2
4
6
8
10
12
NO. OF CROSS BEAMS
S=20
S=30
S=40
S=50
S=60
S=70
S=80

Fig. 23

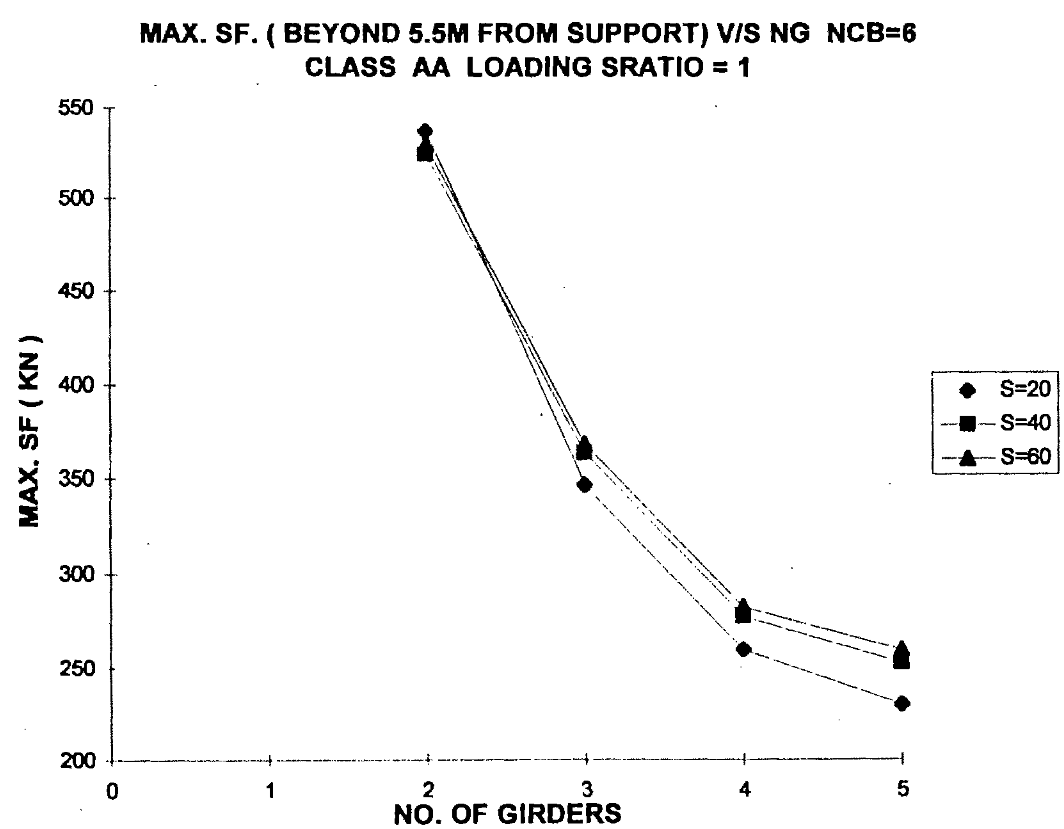

MAX. SF. (BEYOND 5.5M FROM SUPPORT) V/S NG NCB=6 CLASS AA LOADING SRATIO = 1
MAX. SF (KN)
550
500
450
400
350
300
250
200
0
1
2
3
4
5
NO. OF GIRDERS
S=20
S=40
S=60

Fig. 24

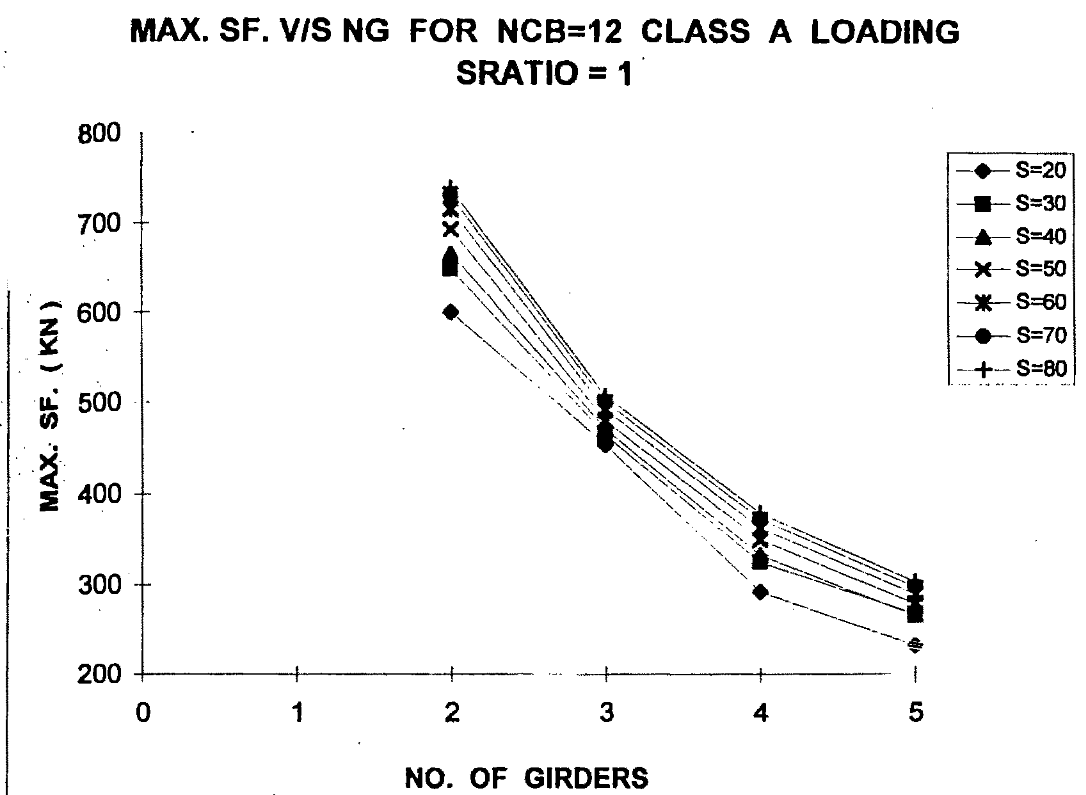

MAX. SF. V/S NG FOR NCB=12 CLASS A LOADING
SRATIO = 1
MAX. SF. (KN)
800
700
600
500
400
300
200
0
1
2
3
4
5
NO. OF GIRDERS
S=20
S=30
S=40
S=50
S=60
S=70
S=80

FIG. 25

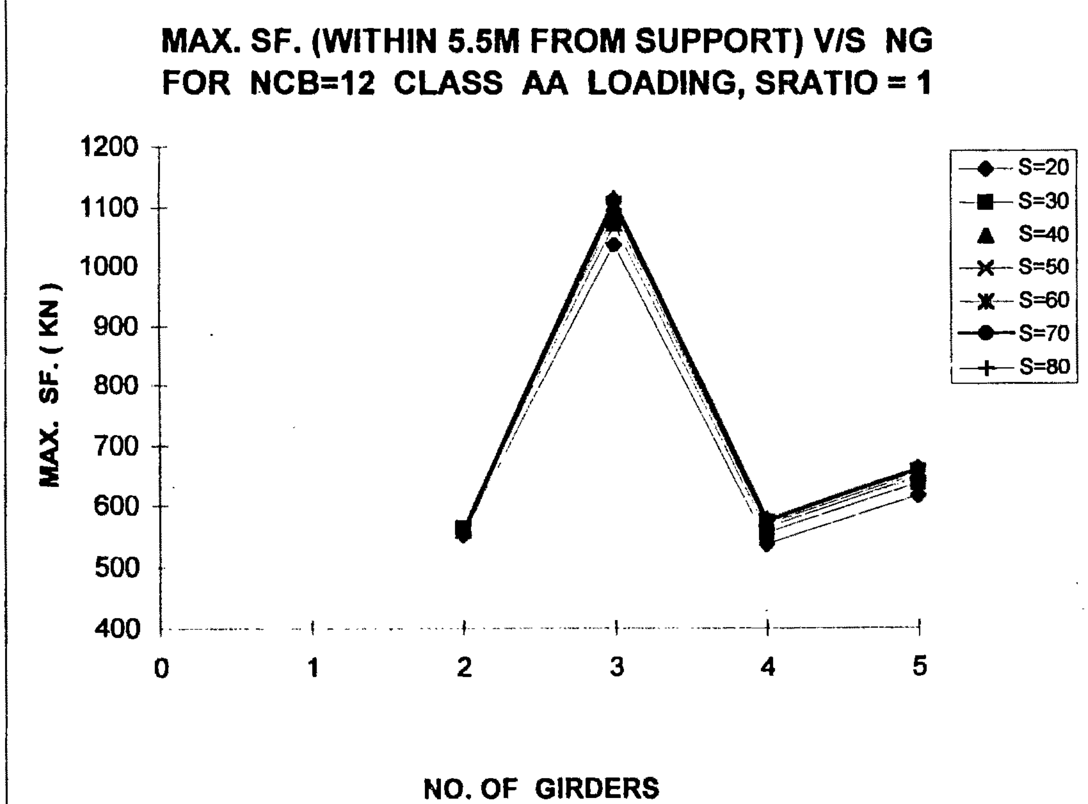

MAX. SF. (WITHIN 5.5M FROM SUPPORT) V/S NG
FOR NCB=12 CLASS AA LOADING, SRATIO = 1
MAX. SF. (KN)
1200
1100
1000
900
800
700
600
500
400
0
1
2
3
4
5
NO. OF GIRDERS
S=20
S=30
S=40
S=50
S=60
S=70
S=80

Fig.26

magnitude when the number of longitudinal girders is an odd number in comparison to when it is an even number.

The clause no. 305.9.2 of IRC:21 - 1987 (1997) is more effective in case of the IRC class AA loads as compared to the IRC class A loads. This is because of the shorter length of distribution of IRC class AA loads as compared to IRC class A loads. Thus maximum shear force in the longitudinal girder in the region within 5.5m from the supports under IRC class AA loads mainly depends upon the reactions at the girder locations assuming the deck slab to be continuous over the longitudinal girders. Due to the distribution of IRC class A loads over a longer length, the maximum shear force in the girder mainly depends on the distribution factor. It is critical of the factors of the Morrice - Little and Hendry - Jaeger methods. From Fig. 25 it is seen that maximum shear force in the girder under IRC class A loads reduces with the increase in the number of girders.

III) Maximum shear force v/s span of the bridge

From Figs. 27 & 28 it can be observed that there is little variation in the maximum shear force in the longitudinal girder under IRC class AA loads with the variation in the span of the bridge. However under IRC class A loads, there is appreciable increase in the maximum shear force with the increase in the span of the bridge. This difference is because over the span range of 20 to 80m, the total load on the bridge under IRC class AA loads remains constant whereas under IRC class A loads it increases with the span. The change in the value of the distribution factor with the increase in the span also adds to the magnitude of the variation of the maximum shear force with the span.

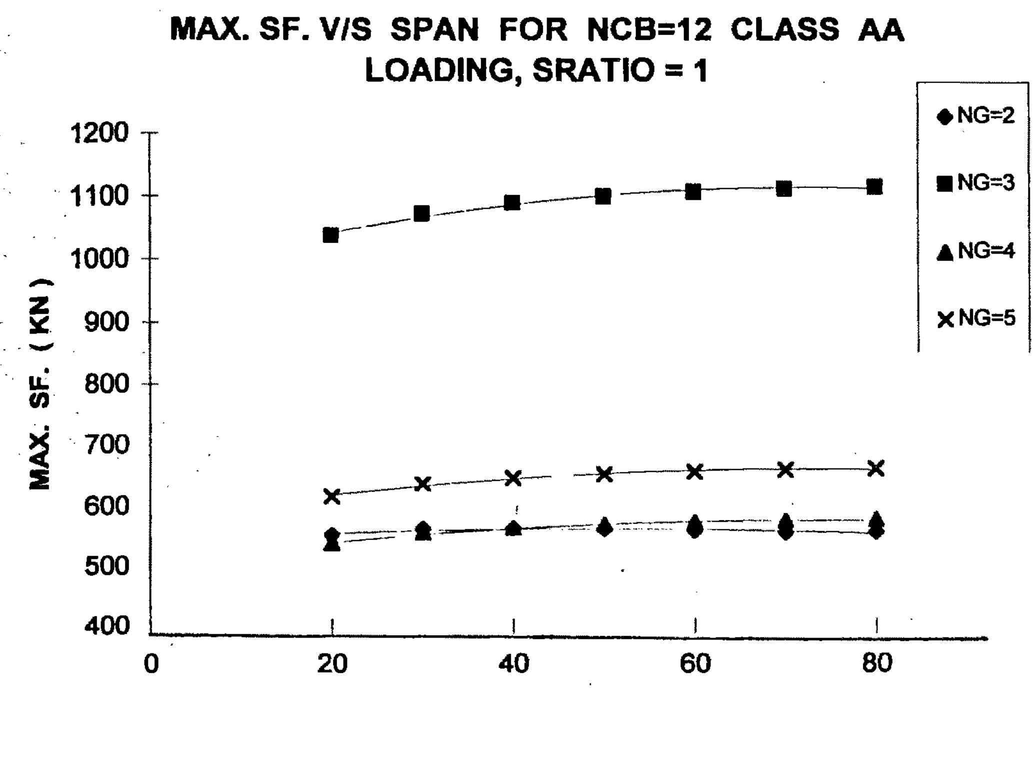

MAX. SF. V/S SPAN FOR NCB=12 CLASS AA LOADING, SRATIO = 1
NG=2
NG=3
NG=4
NG=5
1200
1100
1000
900
800
700
600
500
400
MAX. SF. (KN)
0
20
40
60
80
SPAN (M)

Fig. 27

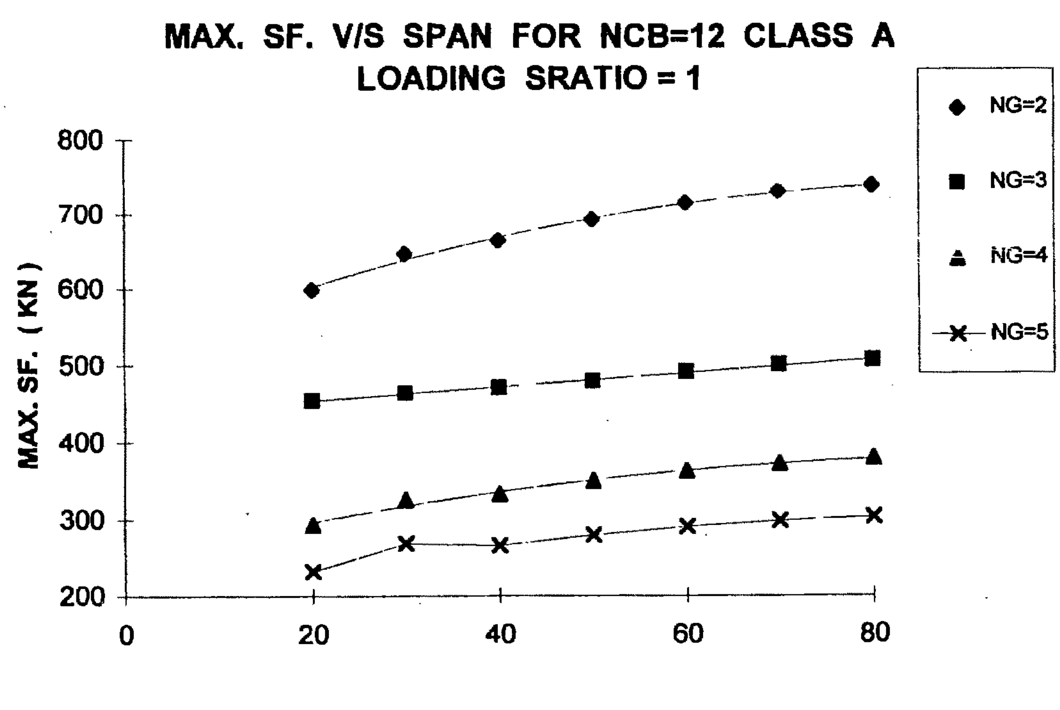

MAX. SF. V/S SPAN FOR NCB=12 CLASS A LOADING SRATIO = 1
NG=2
NG=3
NG=4
NG=5
800
700
600
500
400
300
200
MAX. SF. (KN)
0
20
40
60
80
SPAN (M)

Fig. 28

(iv) Maximum shear force v/s SRATIO

From Fig. 29 it is seen that the maximum shear force in the longitudinal girder under IRC class AA loads reduces appreciably with the increase in the value of the SRATIO. The variation in the value of SRATIO from 1 to 2 causes the maximum shear force to reduce by about 21.6%.

The maximum shear force in the girder is generally induced near its supports. Due to the clause no. 305.9.2 of IRC:21 - 1987 (1997), this maximum shear force near the supports is equal to the reaction at the longitudinal girder locations assuming the deck slab to be continuous over the longitudinal girders. When the value of the SRATIO reduces, the spacing between the girders also reduces. Thus, shear force developed due to the fixed end moments at the intermediate supports of the deck slab also decreases. In case of IRC class AA loads, the maximum part of the loads may be placed in the region within 5.5m from the support . Hence, with the increase in the value of the SRATIO, a large reduction in the value of the maximum shear force in the girder under IRC class AA loads takes place.

Fig. 30 shows the variation of the maximum shear force in the longitudinal girder under IRC class A load with the increase in the value of SRATIO. In contrast to the IRC class AA loads, there is only slight change (about 6%) in the value of the maximum shear force in the girder under IRC class A loads with the increase in the value of SRATIO. The IRC class A loads occupy a length longer than 5.5m. Hence, the value of the maximum shear force in the girders depends on the reactions over the girder supports as well as on the distribution factor. The change in the value of the distribution factor with the increase in the value of

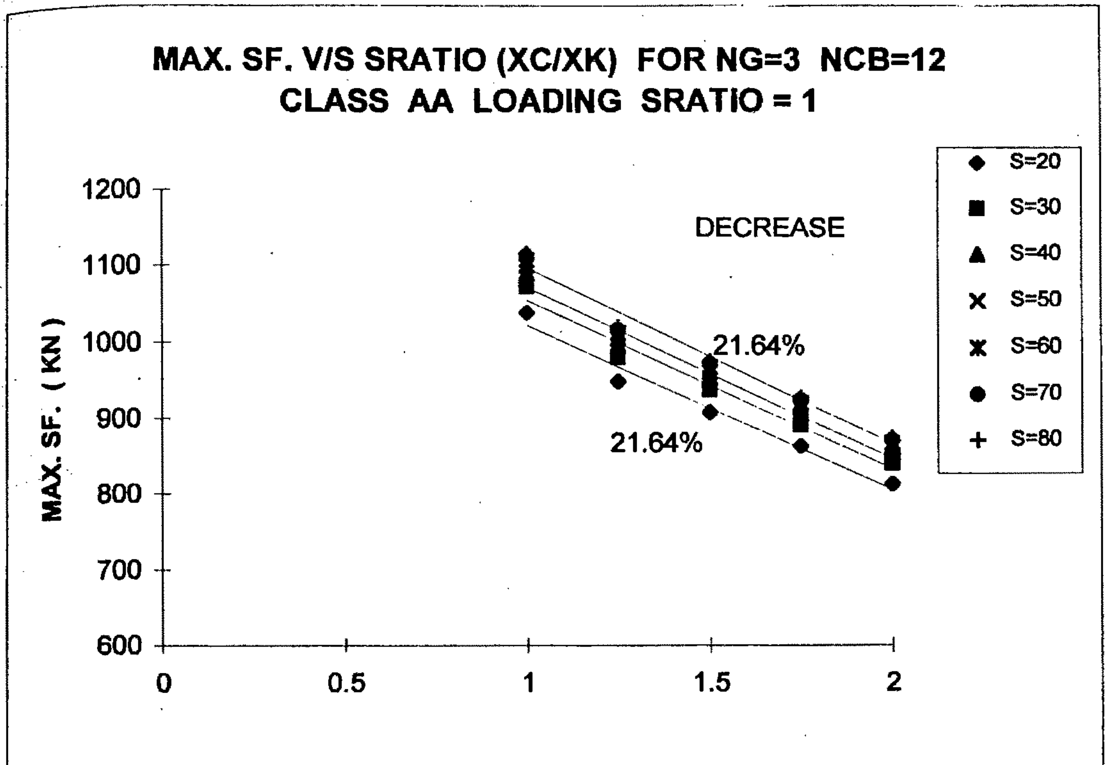

MAX. SF. V/S SRATIO (XC/XK) FOR NG=3 NCB=12
CLASS AA LOADING SRATIO = 1
DECREASE
21.64%
21.64%
MAX. SF. (KN)
1200
1100
1000
900
800
700
600
0
0.5
1
1.5
2
XC/XK (SRATIO)
S=20
S=30
S=40
S=50
S=60
S=70
S=80

Fig. 29

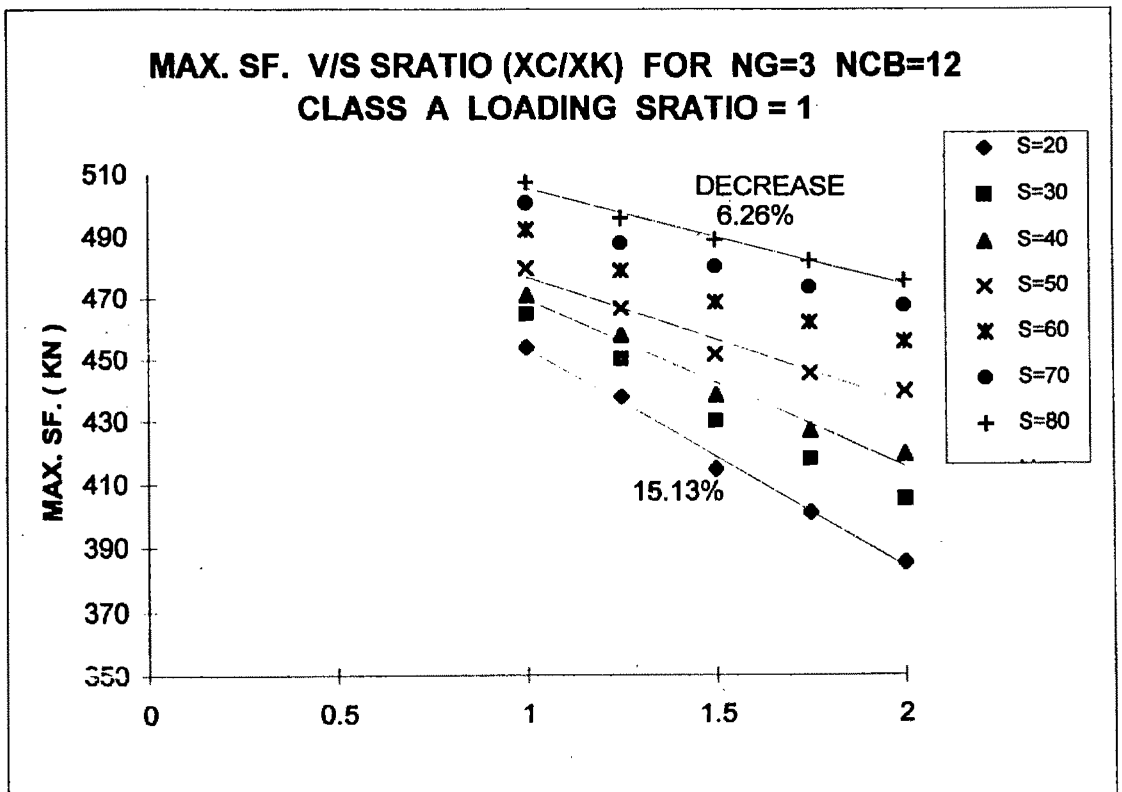

MAX. SF. V/S SRATIO (XC/XK) FOR NG=3 NCB=12
CLASS A LOADING SRATIO = 1
DECREASE
6.26%
15.13%
MAX. SF. (KN)
510
490
470
450
430
410
390
370
350
0
0.5
1
1.5
2
XC/XK (SRATIO)
S=20
S=30
S=40
S=50
S=60
S=70
S=80

Fig. 30

SRATIO is generally very small. Thus variation in the maximum shear force in longitudinal girder under IRC class A loads is expected to be smaller compared to IRC class AA loads.

5.2.2 Variation of deck slab thickness

The bending moment and shear force induced in the deck slab are discussed in terms of its thickness required to resist these forces. The trend of the variation of the deck slab thickness may vary from increasing to decreasing trend and vice versa if by the variation in the parameters of the bridge superstructure;

A) the deck slab behaviour changes from one way to two way slab action and vice versa, and

B) the direction of the shorter span of the deck slab changes from transverse to the longitudinal direction of the bridge and vice versa.

The variation of deck slab thickness with various bridge superstructure parameters is shown in Figs. 31 to 35.

I) Deck slab thickness v/s number of girders

From Fig. 31 it may be observed that the longer span bridges under IRC class AA loads are subjected to higher variations in the deck slab thickness with the variation in the number of longitudinal girders as compared to shorter span bridges. This variation under IRC class A load is similar to the IRC class AA and class A loads. The effective (shorter) span of the deck slab panel reduces with the increase in the number of girders if the span of the bridge is so large that the effective span of the slab panel is always along the transverse direction of the

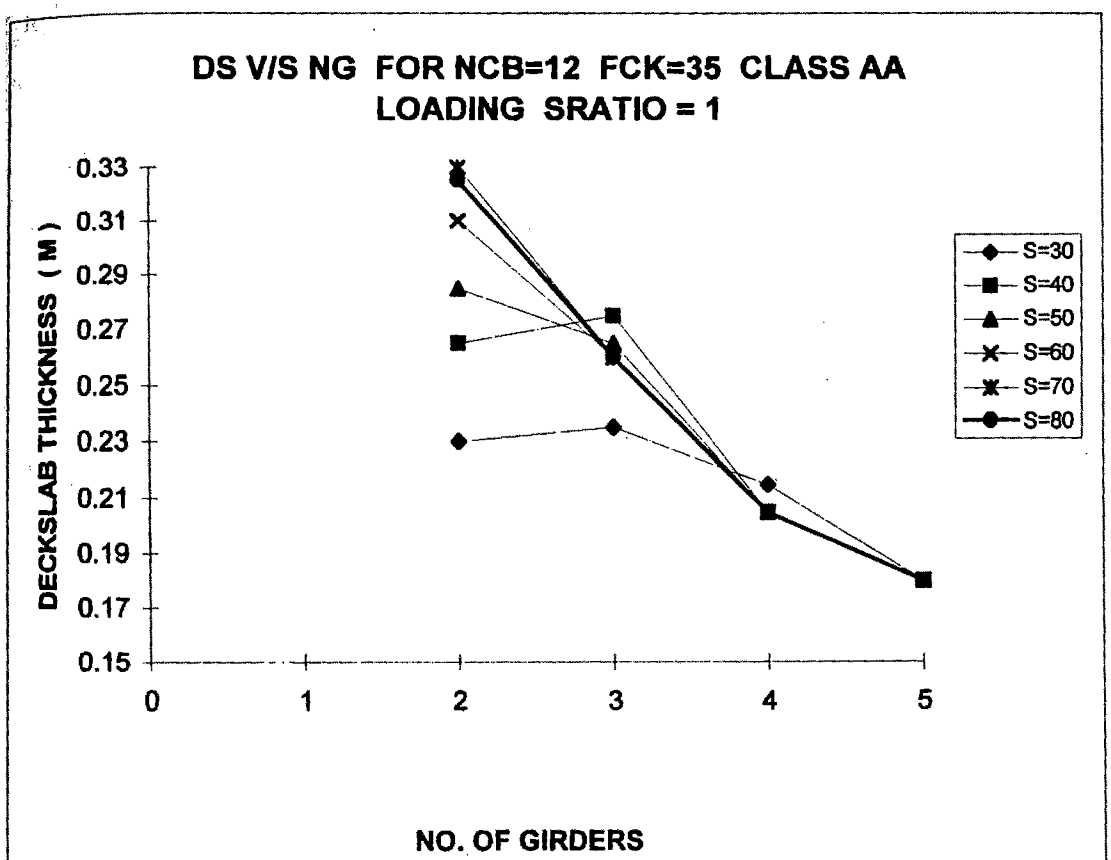

DS V/S NG FOR NCB=12 FCK=35 CLASS AA LOADING SRATIO = 1
DECKSLAB THICKNESS (M)
0.33
0.31
0.29
0.27
0.25
0.23
0.21
0.19
0.17
0.15
0
1
2
3
4
5
S=30
S=40
S=50
S=60
S=70
S=80
NO. OF GIRDERS

Fig. 31

bridge. Thus with the increase in the number of longitudinal girders, the effective span of the slab panel decreases but the total loads on it remains practically constant. Hence the magnitude of the maximum bending moment induced in the deck slab panel reduces and results in the decrease in its thickness. If the span of the bridge is such that the effective span of the deck slab panel is along the length of the bridge (possible for smaller number of the girders), the increase in the number of girders (i.e. decrease in the longer dimension of the panel) reduces the region of the longer dimension of the slab which is not covered by the loads i.e. load per unit width of the slab panel increases. So long as this longer dimension of the slab panel remains along the transverse direction of the bridge, any reduction in the length of the longer dimension of the slab panel causes an increase in the thickness of the slab panel. As soon as the dimension of the slab panel along the transverse direction of the bridge becomes its shorter dimension, further reduction of this dimension of the slab panel causes a reduction in the deck slab thickness.

II) Deck slab thickness v/s span of the bridge

Fig. 32 depicts the variation of deck slab thickness with the span of the bridge. It may be observed from this figure that the variation in the deck slab thickness with the span for the given number of cross beams and longitudinal girders is higher when the number of the longitudinal girders is small. However the deck slab thickness is seen to be practically constant when the number of the girders is four or more. For the higher number of the girders, the shorter span of the slab panel is always in the transverse direction of the bridge superstructure

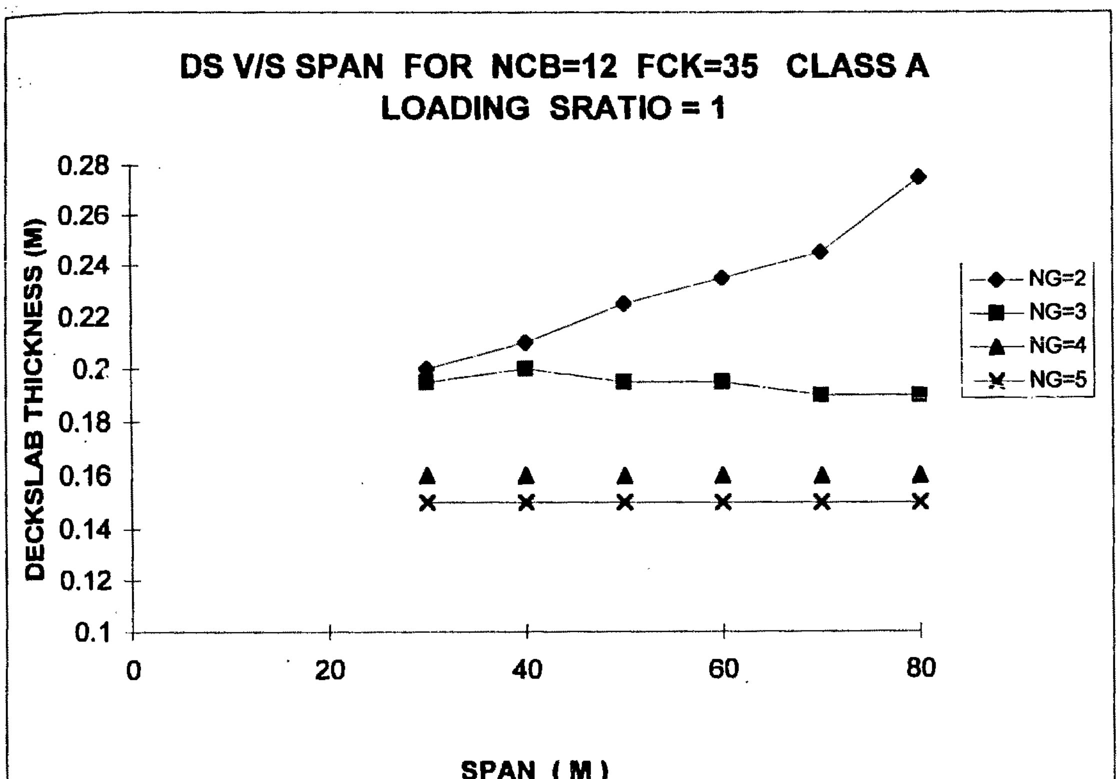

DS V/S SPAN FOR NCB=12 FCK=35 CLASS A
LOADING SRATIO = 1
DECKSLAB THICKNESS (M)
0.28
0.26
0.24
0.22
0.2
0.18
0.16
0.14
0.12
0.1
0
20
40
60
80
NG=2
NG=3
NG=4
NG=5
SPAN (M)

Fig. 32

and it always behaves as one way slab. With the increase in the span of the bridge, the slab behaviour remains unchanged i.e. it still behaves as one way slab. Hence, with the increase in the span of the bridge the deck slab thickness remains practically constant.

It may further be observed from Fig. 32 that if the shorter dimension of the slab panel is along the length of the bridge and the increase in this shorter dimension of the panel is accompanied by an increase in the total load on the panel, the deck slab thickness increases with the increase in the span.

III) Deck slab thickness v/s SRATIO

The effect of SRATIO on deck slab thickness is shown in Fig. 33. It is observed from this figure that the pattern of the variation of the deck slab thickness with SRATIO is different for varying combinations of the number of longitudinal girders and the cross beams. The variation of the deck slab thickness depends upon the direction of the shorter span of the slab panel. The variation of the deck slab thickness with SRATIO is similar to its variation with the number of the girders because in both the cases, the dimension of the slab panel along the transverse direction of the bridge changes.

IV) Deck slab thickness v/s number of cross beams

A comparison of Figs. 33 and 34 clearly shows that the variation of the deck slab thickness with the number of cross beams for the given number of the longitudinal girders and the span is similar to the variation with the change in span for the given number of the cross beams and the longitudinal girders. In both

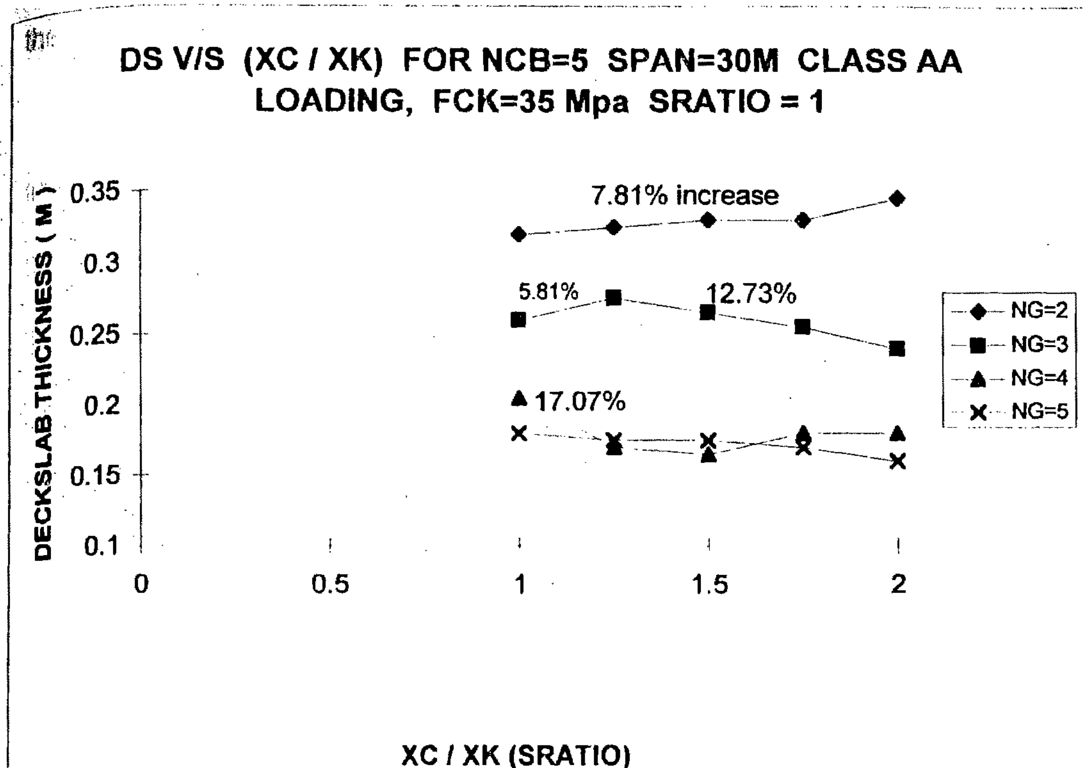

DS V/S (XC / XK) FOR NCB=5 SPAN=30M CLASS AA LOADING, FCK=35 Mpa SRATIO = 1
7.81% increase
5.81%
12.73%
17.07%
DECKSLAB THICKNESS (M)
0.35
0.3
0.25
0.2
0.15
0.1
0
0.5
1
1.5
2
NG=2
NG=3
NG=4
NG=5
XC / XK (SRATIO)

Fig. 33

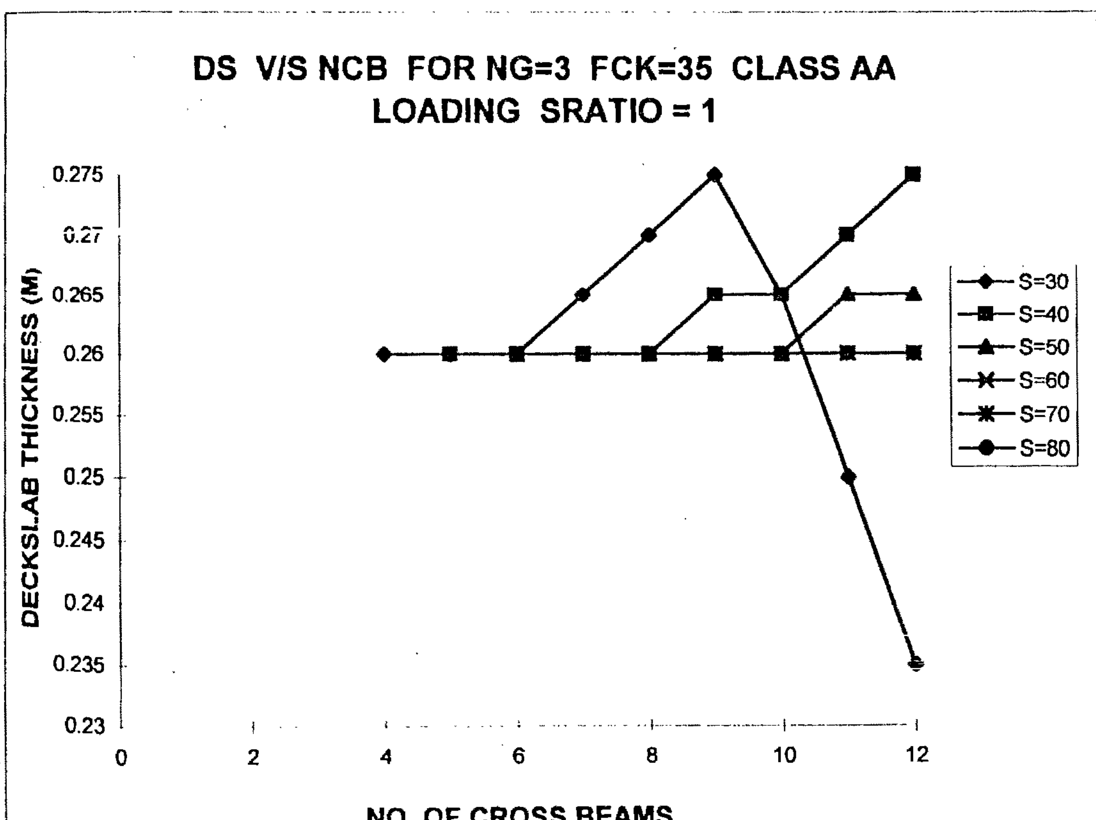

DS V/S NCB FOR NG=3 FCK=35 CLASS AA LOADING SRATIO = 1
DECKSLAB THICKNESS (M)
0.275
0.27
0.265
0.26
0.255
0.25
0.245
0.24
0.235
0.23
0
2
4
6
8
10
12
S=30
S=40
S=50
S=60
S=70
S=80
NO. OF CROSS BEAMS

Fig. 34

the cases, the dimension of the panel along the length of the bridge changes. The variation of the deck slab thickness with the increase in number of cross beams is appreciable when the number of the girders is small. For number of girders equal to five, the deck slab thickness remains practically constant over a wide range of number of cross beams. It may also be observed from Fig. 34 that the variation of the deck slab thickness with number of cross beams is more significant in the shorter span bridges.

V) Deck slab thickness v/s deck slab concrete grade

Fig. 35 shows the variation of deck slab thickness with deck slab concrete grade. It may be observed that deck slab thickness decreases with the increase in the concrete strength irrespective of the bridge span and the number of the longitudinal girders. If the number of longitudinal girders is four or more, the rate of variation of the deck slab thickness with the strength of the concrete is practically independent of the span of the bridge.

5.3 Need of Amendments in IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997)

The results obtained using the computer programme show that the provisions of the IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) not only discourage the use of high-strength concrete in highway bridges by rendering its use uneconomical in terms of its higher cost of production but also, in some of the cases, makes the bridge even structurally costlier.

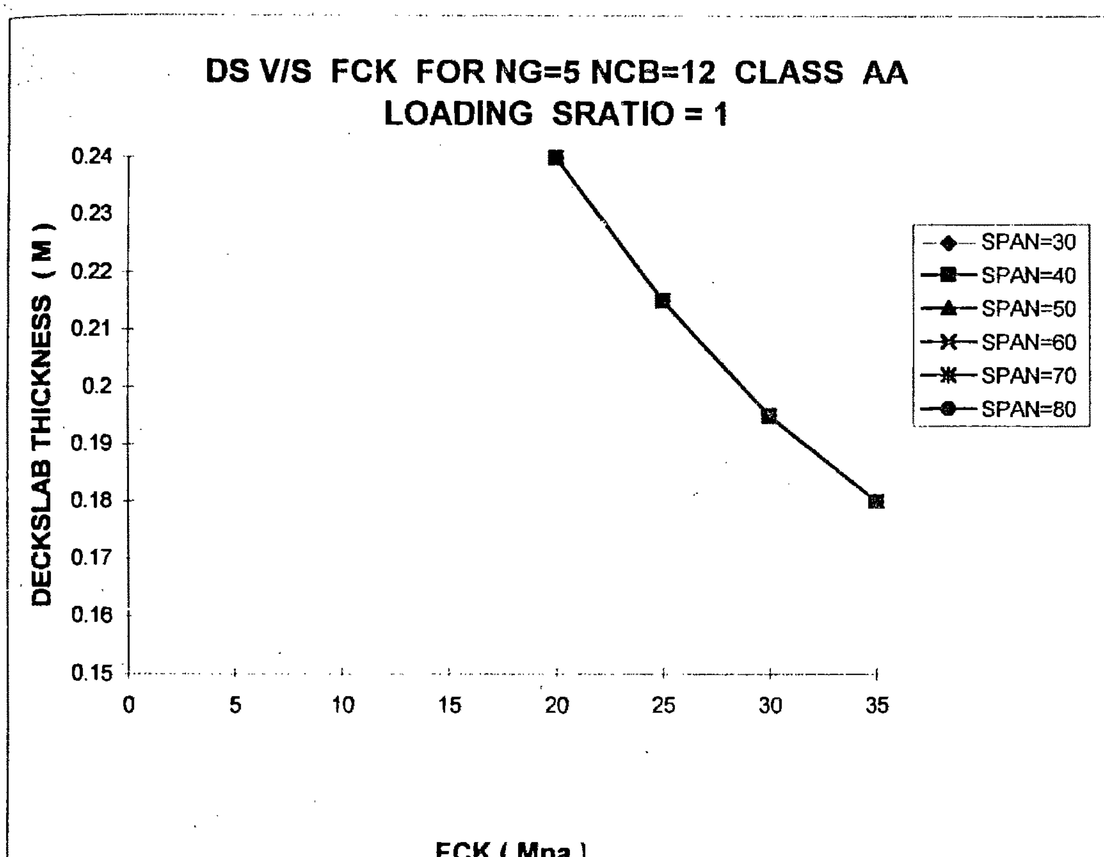

DS V/S FCK FOR NG=5 NCB=12 CLASS AA
LOADING SRATIO = 1
DECKSLAB THICKNESS (M)
0.24
0.23
0.22
0.21
0.2
0.19
0.18
0.17
0.16
0.15
0
5
10
15
20
25
30
35
SPAN=30
SPAN=40
SPAN=50
SPAN=60
SPAN=70
SPAN=80
FCK (Mpa)

Fig. 35

5.3.1 Bridge superstructure cost v/s longitudinal girder concrete strength

The cost of the bridge superstructure per meter of the bridge length is normalised with respect to the cost per MT of 30 MPa concrete. In general, as span increases, the concrete grade corresponding to the minimum cost of the superstructure should also increase. But the results of the present investigation on high-strength concrete superstructure do not follow this trend. From Figs. 36-38, it can be observed that the minimum cost of the bridge superstructure over the span range from 18 to 34m and for depth of girder section varying from 1500mm to 2030mm corresponds to the concrete strength range of 40 to 60 MPa. The use of higher strength concrete increase the cost of even the longer span bridges.

This anomaly may be due to the fact that IRC:18 - 1985 (1997) does not allow an increase in the permissible stress at transfer of prestress with the increase in the concrete strength. The IRC:21 - 1987 (1997) specifications overestimate the value of the secant modulus of high-strength concrete. The overestimation of the secant modulus reduces the elastic shortening losses and hence increases the effective prestress. This increased effective prestress throws up two possibilities:

I) If the girder is safe under the service loads and at the transfer of prestress, the cost of superstructure increases due to the higher cost of production of high-strength concrete.

II) If the girder fails at the transfer of prestress due to increased stress in the concrete caused by the increased effective prestress , the superstructure may become safe if the number of longitudinal girders is allowed to increase. This increased number of the girders reduces the stresses in it due to loads and hence

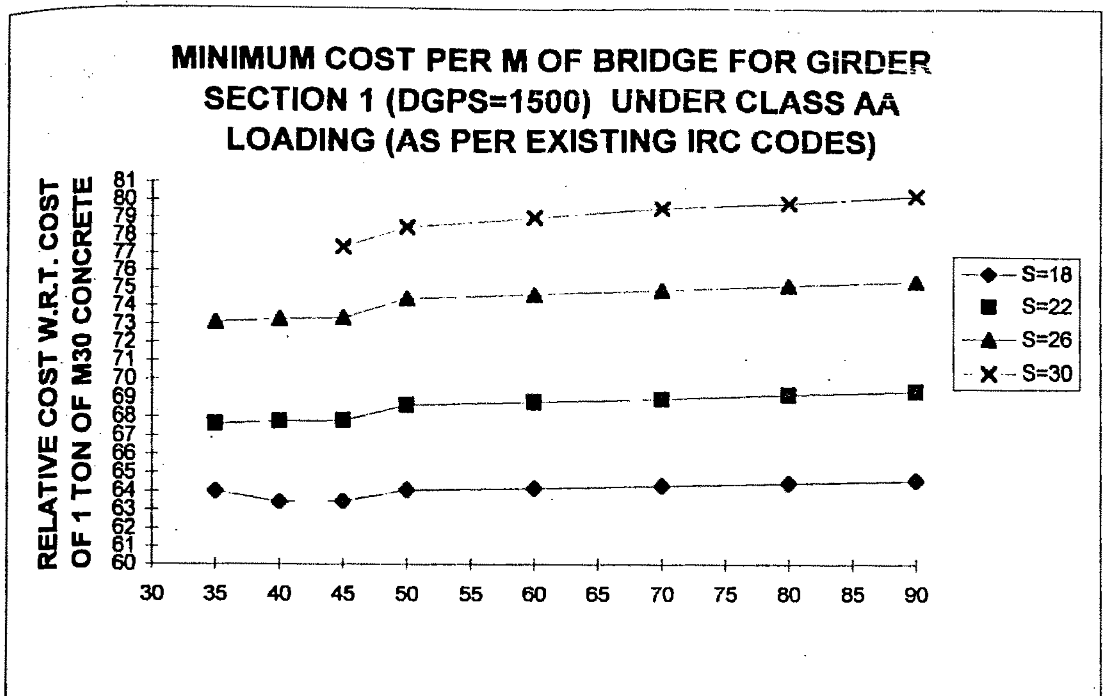

MINIMUM COST PER M OF BRIDGE FOR GIRDER SECTION 1 (DGPS=1500) UNDER CLASS AA LOADING (AS PER EXISTING IRC CODES)
RELATIVE COST W.R.T. COST OF 1 TON OF M30 CONCRETE
GIRDER CONCRETE GRADE (MPa)
S=18
S=22
S=26
S=30

Fig. 36

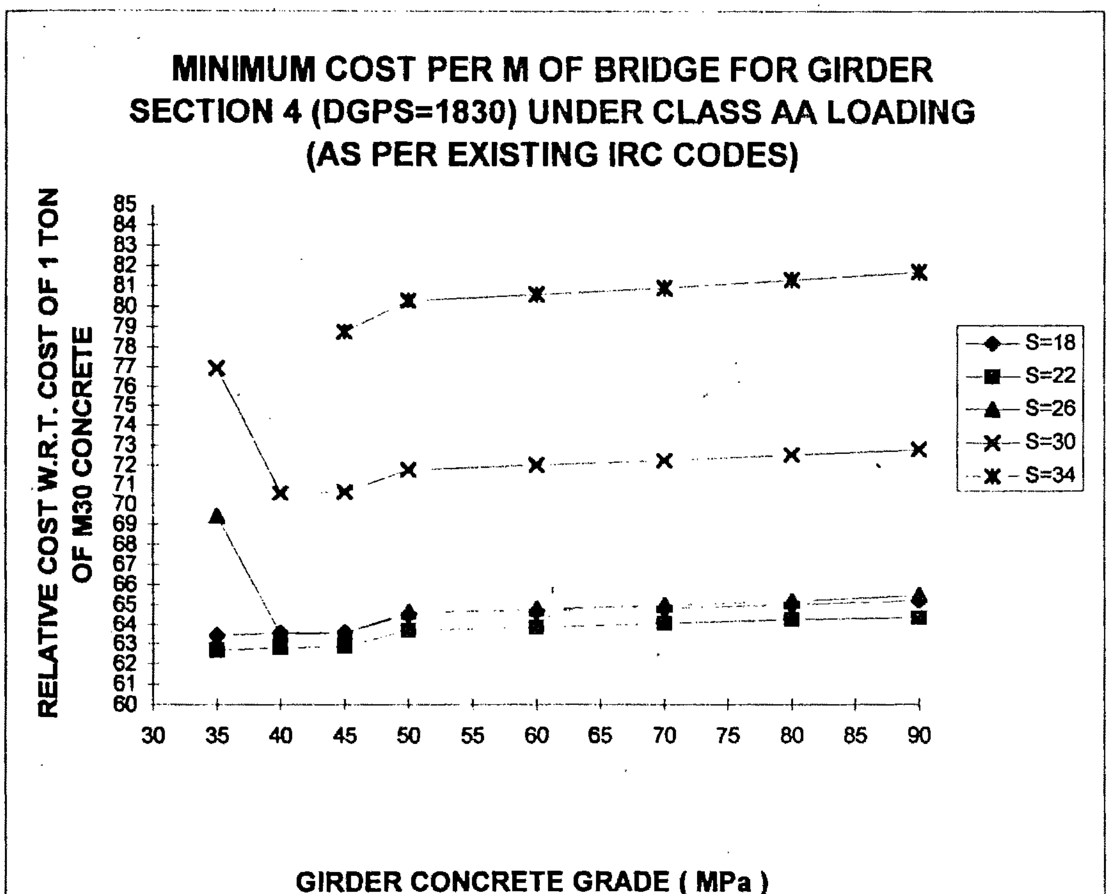

MINIMUM COST PER M OF BRIDGE FOR GIRDER SECTION 4 (DGPS=1830) UNDER CLASS AA LOADING (AS PER EXISTING IRC CODES)
RELATIVE COST W.R.T. COST OF 1 TON OF M30 CONCRETE
GIRDER CONCRETE GRADE (MPa)
S=18
S=22
S=26
S=30
S=34

Fig. 37

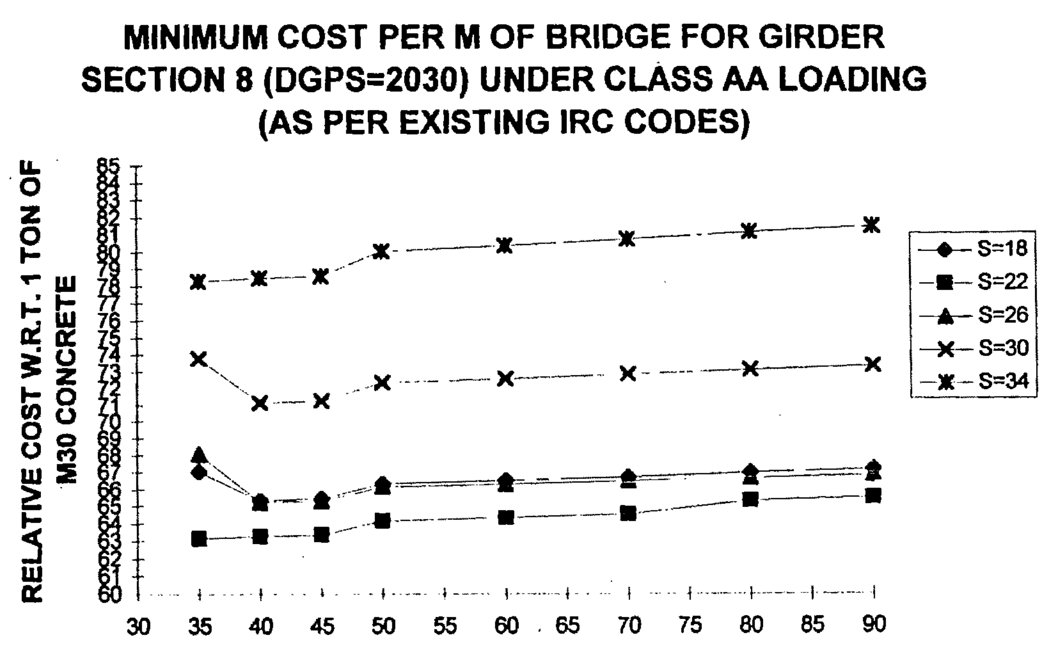

MINIMUM COST PER M OF BRIDGE FOR GIRDER SECTION 8 (DGPS=2030) UNDER CLASS AA LOADING (AS PER EXISTING IRC CODES)
RELATIVE COST W.R.T. 1 TON OF M30 CONCRETE
60
85
30
35
40
45
50
55
60
65
70
75
80
85
90
S=18
S=22
S=26
S=30
S=34
GIRDER CONCRETE GRADE (MPa)

Fig. 38

it may become safe at lower prestress level. Due to the increase in the longitudinal girders, the cost of the superstructure also increases.

Figs. 39 - 41 are based on the results obtained after including the proposed amendments. It can be observed that the concrete grade corresponding to the minimum cost of the superstructure increases with the increase in the span of the bridge. Thus higher span bridges could be economically designed by using high-strength concrete.

5.3.2 Girder span capability v/s girder concrete strength

From Figs. 42 & 43 it may be observed that there is no advantage of using high-strength concrete in the longitudinal girders because it does not increase the maximum span capability of the longitudinal girders. This is not in agreement with the findings of the research work of Jobse and Moustafa (1984). They showed that shallower sections had more potential for an increase in the span capability with an increase in the concrete strength. The influence of the maximum available prestress force was reported to have limited the increase in the span capability of deeper longitudinal girder sections.

The increase in the span capability of various girder sections could not be observed in the present investigation because the specifications of the IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) do not allow continuous increase in concrete strength with the increased grade of concrete.

Figs. 44 & 45 are based on the results obtained after including the proposed amendments in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997). These results are similar to the results reported by Jobse and Moustafa (1984) and other

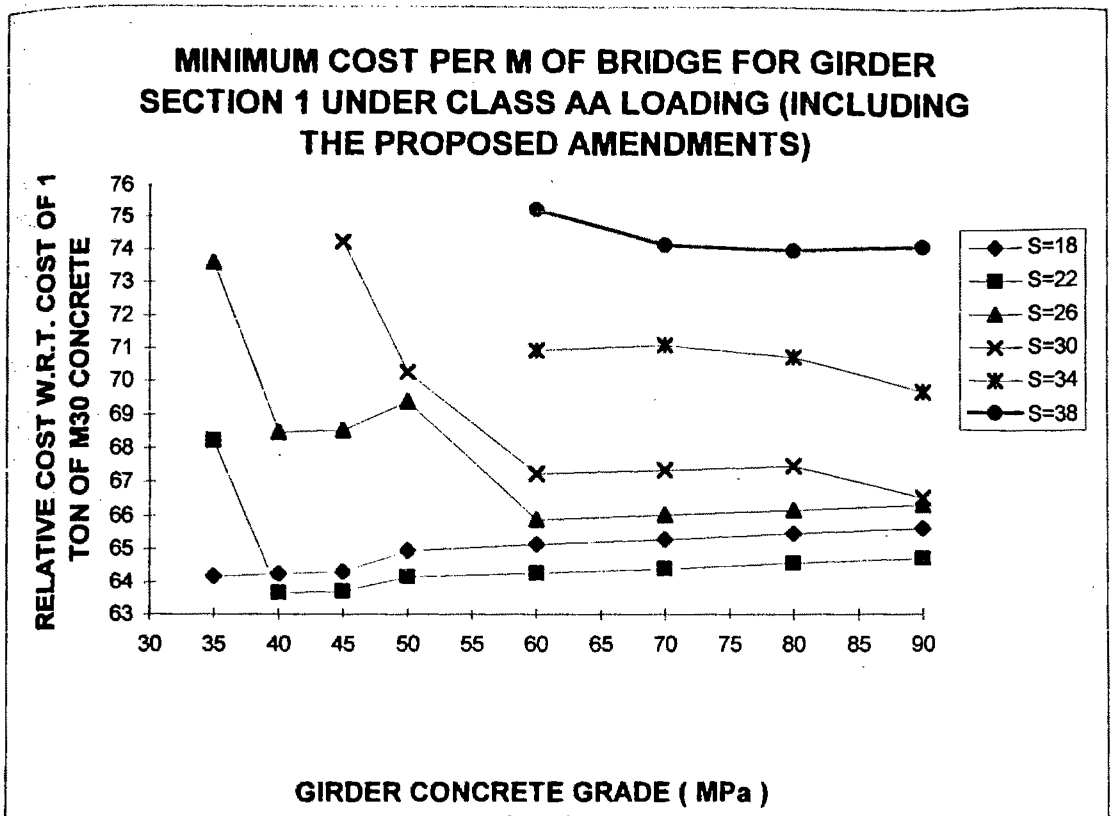

MINIMUM COST PER M OF BRIDGE FOR GIRDER SECTION 1 UNDER CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST W.R.T. COST OF 1 TON OF M30 CONCRETE
76
75
74
73
72
71
70
69
68
67
66
65
64
63
30
35
40
45
50
55
60
65
70
75
80
85
90
S=18
S=22
S=26
S=30
S=34
S=38
GIRDER CONCRETE GRADE (MPa)

Fig. 39

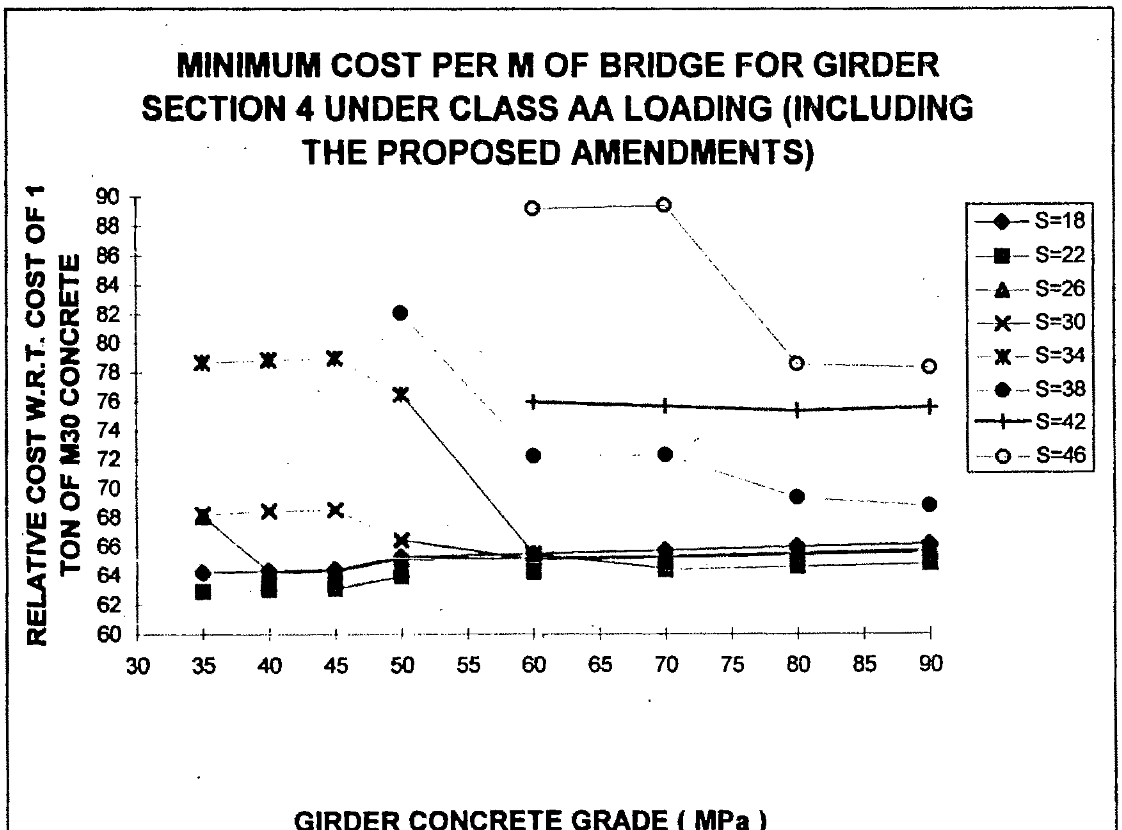

MINIMUM COST PER M OF BRIDGE FOR GIRDER SECTION 4 UNDER CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST W.R.T. COST OF 1 TON OF M30 CONCRETE
90
88
86
84
82
80
78
76
74
72
70
68
66
64
62
60
30
35
40
45
50
55
60
65
70
75
80
85
90
S=18
S=22
S=26
S=30
S=34
S=38
S=42
S=46
GIRDER CONCRETE GRADE (MPa)

Fig. 40

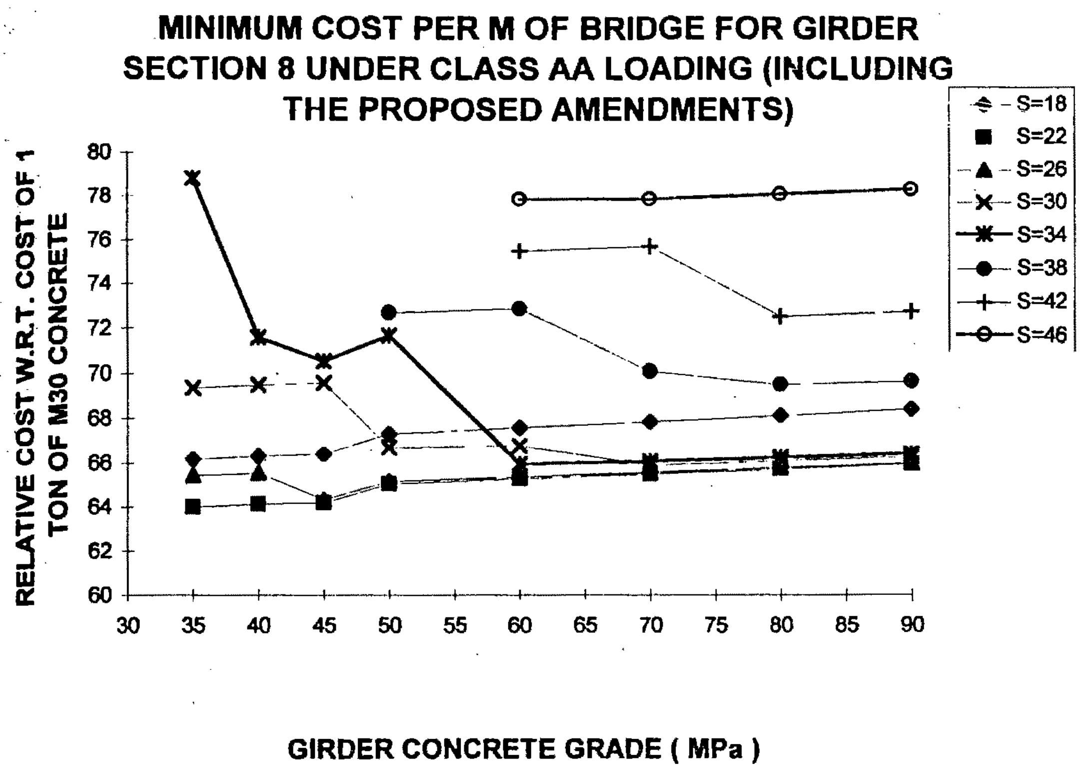

MINIMUM COST PER M OF BRIDGE FOR GIRDER SECTION 8 UNDER CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST W.R.T. COST OF 1 TON OF M30 CONCRETE
GIRDER CONCRETE GRADE (MPa)
S=18
S=22
S=26
S=30
S=34
S=38
S=42
S=46

Fig. 41

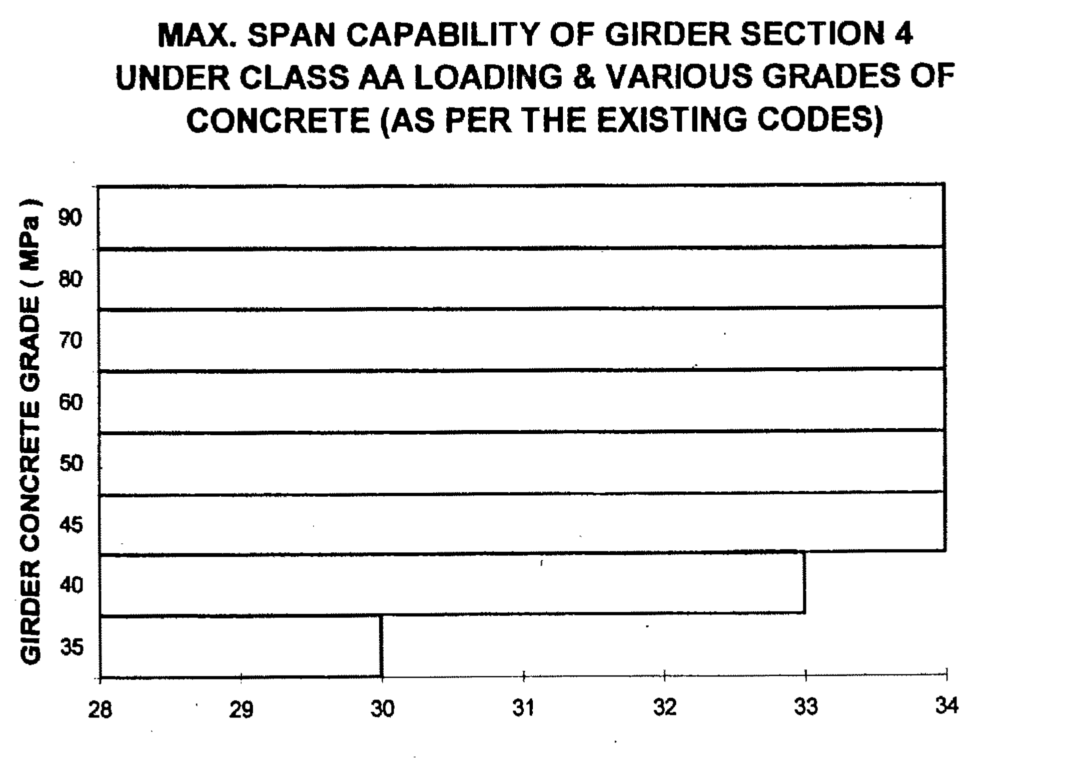

MAX. SPAN CAPABILITY OF GIRDER SECTION 4 UNDER CLASS AA LOADING & VARIOUS GRADES OF CONCRETE (AS PER THE EXISTING CODES)
GIRDER CONCRETE GRADE (MPa)
MAXIMUM SPAN (M)

Fig. 42

MAX. SPAN CAPABILITY OF GIRDER SECTION 8 UNDER CLASS AA LOADING & VARIOUS GRADES OF CONCRETE (AS PER THE EXISTING IRC CODES)

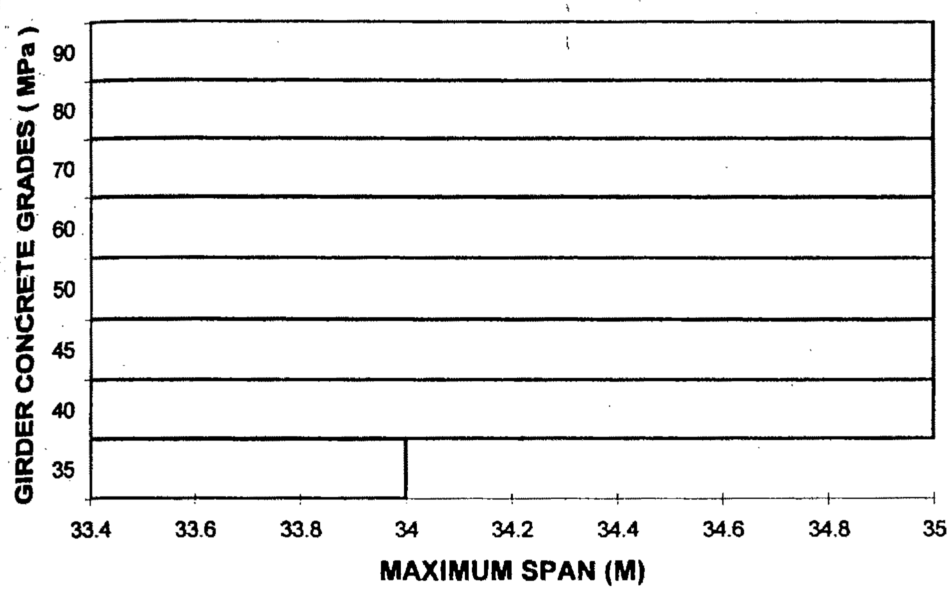


Fig. 43

MAX. SPAN CAPABILITY OF GIRDER SECTION 1 UNDER CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)

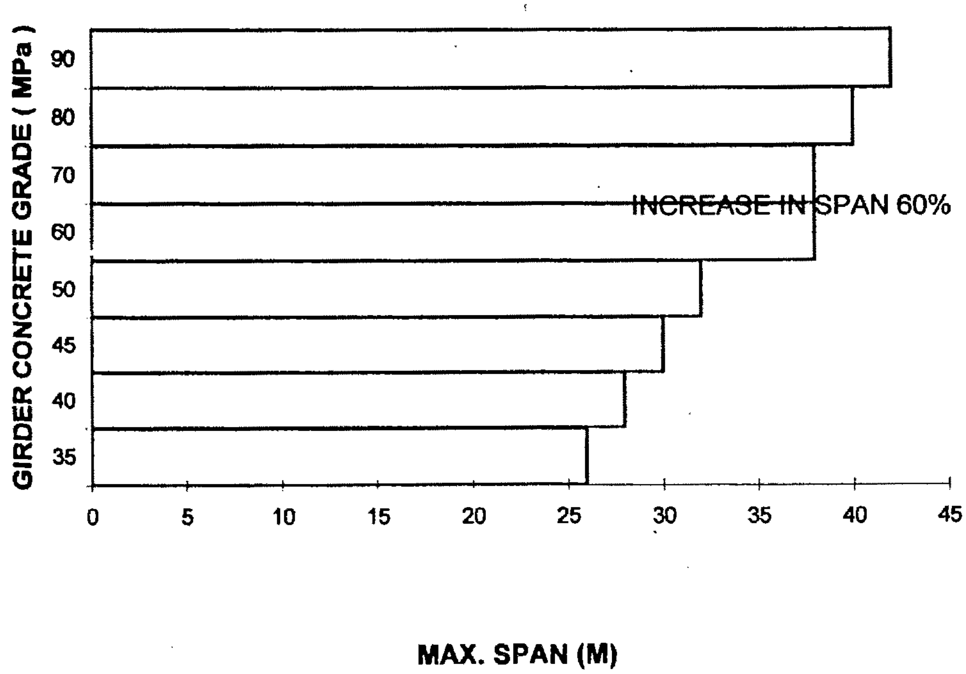


Fig. 44

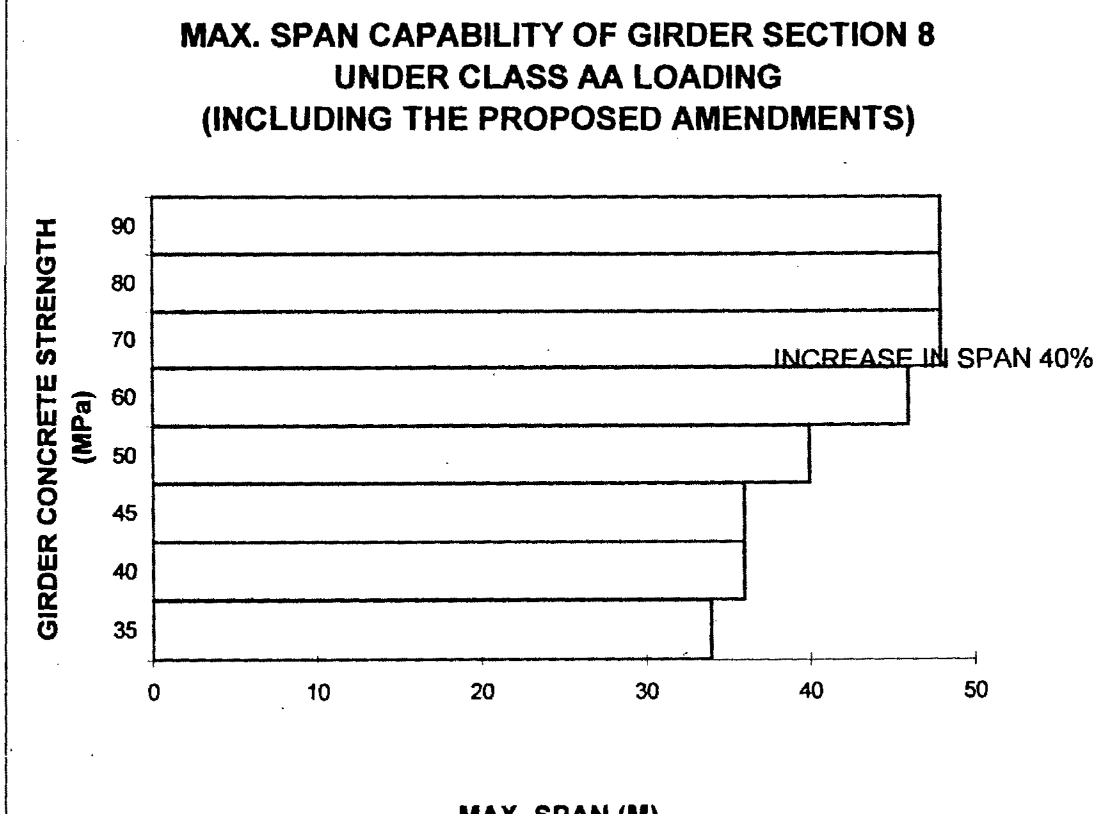

MAX. SPAN CAPABILITY OF GIRDER SECTION 8
UNDER CLASS AA LOADING
(INCLUDING THE PROPOSED AMENDMENTS)
GIRDER CONCRETE STRENGTH (MPa)
90
80
70
60
50
45
40
35
INCREASE IN SPAN 40%
0
10
20
30
40
50
MAX. SPAN (M)

Fig. 45

researchers. The span capability of the shallower sections increases continuously with the increase in the concrete grades. A 60% increase in the maximum span capability is observed for the shallowest girder (girder section 1, depth = 1500mm). As the depth of the girder increases, the maximum available prestress force restricts the advantage of high-strength concrete beyond a certain grade of the concrete. It is also observed from Figs. 44 and 45 that the span capability of the deepest section (girder section 8, depth = 2030mm) does not increase with the increase in the concrete strength beyond 70 MPa and only 40% increase in the span capacity is observed when concrete strength increases from 35 to 70 MPa.

5.3.3 Relative span capability of various longitudinal girder sections

Fig. 46 shows the maximum span under IRC class AA load that can be achieved by various girder sections when number of the girders varies from 2 to 5, concrete strength in the girders varies from 35 to 90 MPa and that in the deck slab from 30 to 50 MPa.

Jacques (1971) reported that the girder sections 1 and 8 may be used up to a maximum span of 35 and 45m respectively. From Fig. 46 it is observed that the maximum span under IRC class AA load attained by the longitudinal girder sections 1 and 8 are only 90% and 77% respectively of that reported by Jacques (1971). This is due to the fact that the maximum permissible stress at the transfer of prestress as permitted by IRC:18 - 1985 (1997) is 20 MPa. This value of the permissible stress at the transfer of prestress corresponds to the concrete characteristic strength of 45 MPa. Thus higher strength concretes may not utilise

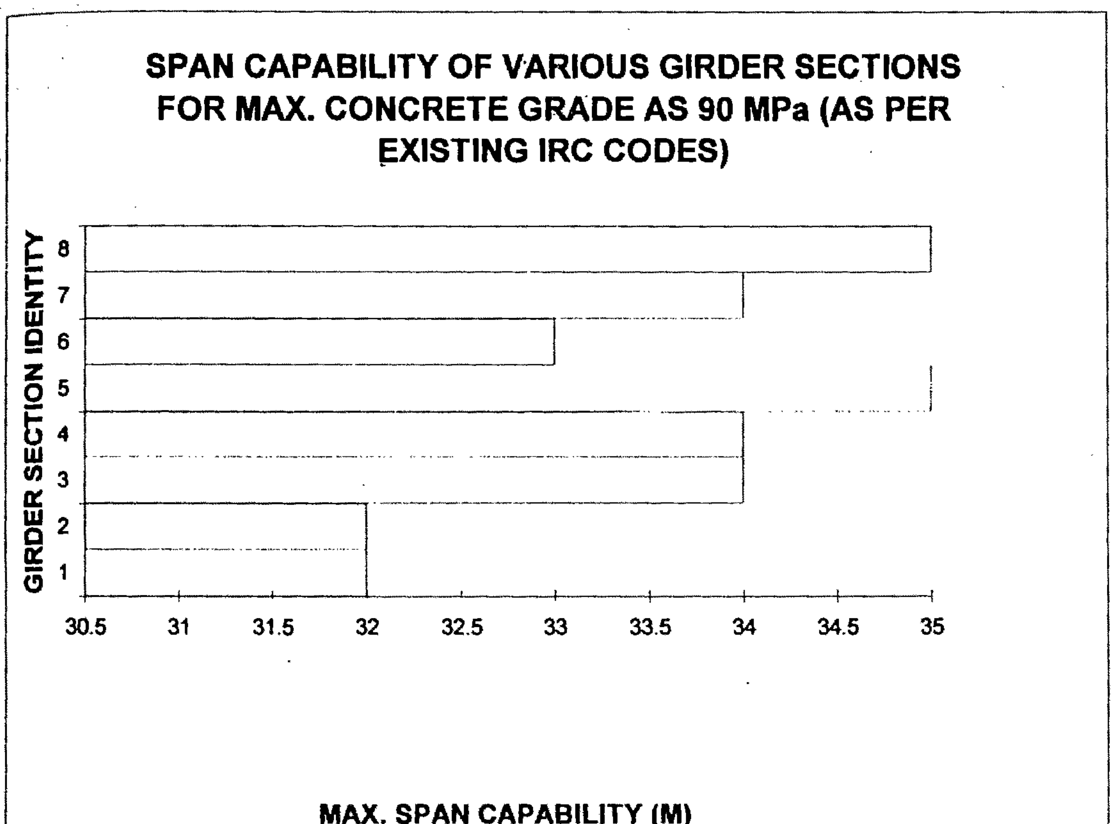
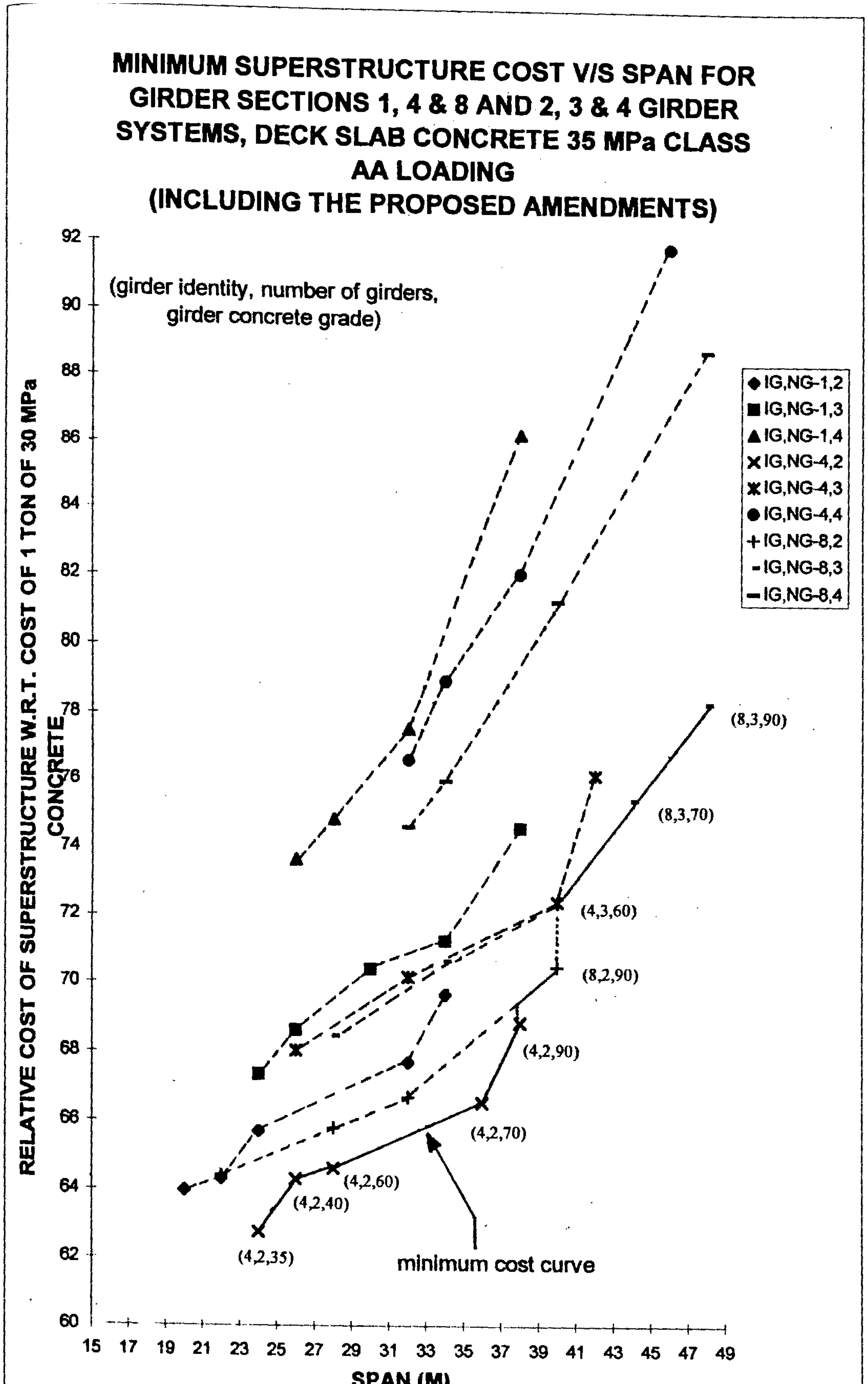

SPAN CAPABILITY OF VARIOUS GIRDER SECTIONS FOR MAX. CONCRETE GRADE AS 90 MPa (AS PER EXISTING IRC CODES)
GIRDER SECTION IDENTITY
8
7
6
5
4
3
2
1
30.5
31
31.5
32
32.5
33
33.5
34
34.5
35
MAX. SPAN CAPABILITY (M)

Fig. 46


MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR GIRDER SECTIONS 1, 4 & 8 AND 2, 3 & 4 GIRDER SYSTEMS, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING
(INCLUDING THE PROPOSED AMENDMENTS)
(girder identity, number of girders, girder concrete grade)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF 30 MPa CONCRETE
SPAN (M)
IG,NG-1,2
IG,NG-1,3
IG,NG-1,4
IG,NG-4,2
IG,NG-4,3
IG,NG-4,4
IG,NG-8,2
IG,NG-8,3
IG,NG-8,4
(8,3,90)
(8,3,70)
(4,3,60)
(8,2,90)
(4,2,90)
(4,2,70)
(4,2,60)
(4,2,40)
(4,2,35)
minimum cost curve
92
90
88
86
84
82
80
78
76
74
72
70
68
66
64
62
60
15
17
19
21
23
25
27
29
31
33
35
37
39
41
43
45
47
49
Fig. 72

their full potential at the transfer of prestress and, hence, the full span capability of the various girder sections could not be achieved.

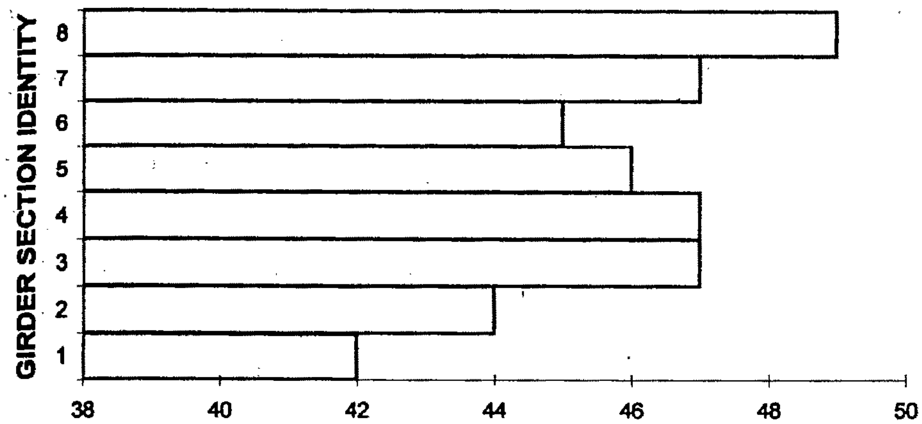
Fig. 47 is based on the results obtained after including the proposed amendments in IRC 18 (1985) and IRC 21(1987). From these figures it is observed that the maximum span capability of the longitudinal girder sections 1 and 8 under IRC class AA and class A loads is 42 and 49m respectively corresponding to maximum grade of concrete as 90 MPa. These values are higher than the corresponding values reported by Jacques (1971). For maximum concrete strength of 60 MPa, the maximum span capability of the longitudinal girder sections 1 and 8 are 38 and 46m respectively. These values are very close to the values of 35m and 45m reported by Jacques (1971).

5.3.4 Number of longitudinal girders v/s girder concrete strength

In the present study, in general, no reduction in the number of longitudinal girders with the increase in the girder concrete grade is observed. On the contrary, in some of the cases, even the increase in the number of longitudinal girders is observed. This trend is clearly reflected in Figs. 48 & 49.

For the given span and number of longitudinal girders, the stresses induced due to load do not change with the concrete grade. With the increase in the concrete grade, the effective prestress increases. Thus the longitudinal girder may fail at the transfer of prestress. If the prestress force is decreased, it is possible that the girder may fail at the transfer of prestress as well as at the ultimate loads because of the reduction in its flexural resistance due to the decrease in the prestress force. If the prestress force increases and the girder

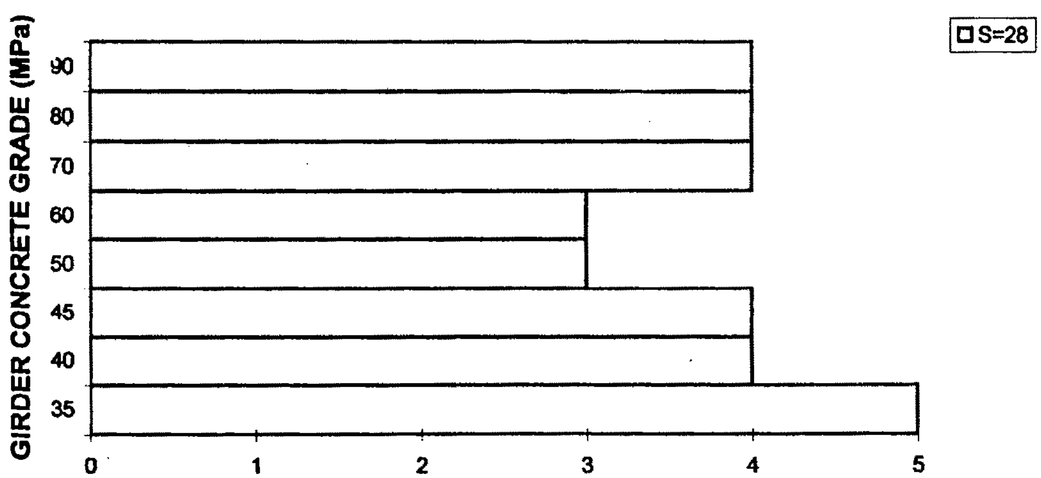
MAX. SPAN CAPABILITY OF VARIOUS GIRDER SECTIONS FOR MAX. GIRDER CONCRETE GRADE AS 90 Mpa (INCLUDING THE PROPOSED AMENDMENTS)



MAX. SPAN (M)

Fig. 47

GIRDER CONCRETE GRADE V/S NO. OF GIRDERS GIRDER SECTION 6 DECK SLAB CONCRETE GRADE 35 MPa CLASS AA LOADING (AS PER EXISTING IRC CODES)



NO. OF GIRDERS

Fig. 48

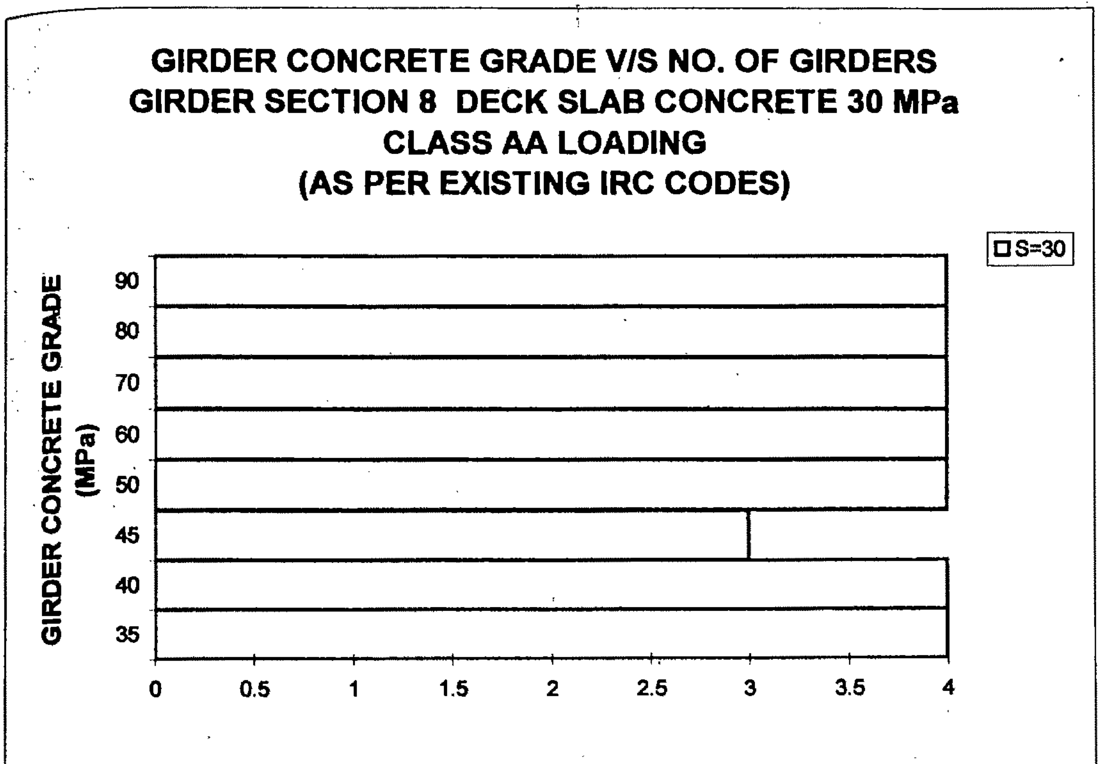

GIRDER CONCRETE GRADE V/S NO. OF GIRDERS
GIRDER SECTION 8 DECK SLAB CONCRETE 30 MPa
CLASS AA LOADING
(AS PER EXISTING IRC CODES)
S=30
GIRDER CONCRETE GRADE (MPa)
90
80
70
60
50
45
40
35
0
0.5
1
1.5
2
2.5
3
3.5
4
NO. OF GIRDERS

Fig. 49

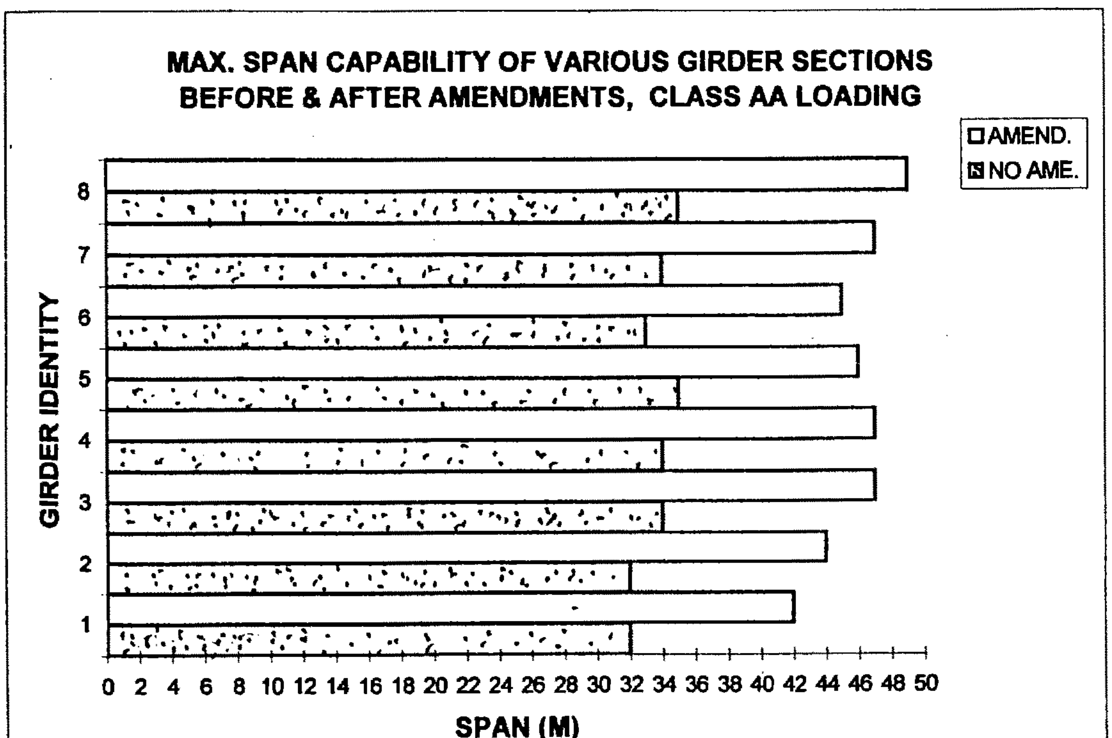

MAX. SPAN CAPABILITY OF VARIOUS GIRDER SECTIONS
BEFORE & AFTER AMENDMENTS, CLASS AA LOADING
AMEND.
NO AME.
GIRDER IDENTITY
8
7
6
5
4
3
2
1
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50
SPAN (M)

Fig. 50

remains safe at the transfer of prestress, no reduction in the number of girders takes place. The other possibility is that the number of girders may increase if the number of girders already used are less than the maximum permitted. This increased number of longitudinal girder reduces the stresses in longitudinal girder due to the loads thereby requiring less prestress force and hence the girder may become safe. If the number of girders already used is the maximum permitted, then design with higher grade of concrete is not possible.

5.3.5 High-strength concrete in deck slab

The appreciable advantage is not observed if higher strength concrete is used in the deck slab. This is due to the fact that IRC:21 - 1987 (1997) does not allow any increase in the permissible stress in flexural as well as in shear beyond the concrete strength of 35 MPa.

5.4 Economic Studies

As already pointed out earlier, the present study does not pertain to any particular region of the country. The cost of various raw materials depends on their local availability. If they are not locally available, the transportation cost plays an important role while optimizing the cost function. Thus, it becomes necessary to tabulate all the structurally possible designs along with their costs so that a particular design with minimum cost and feasible with local materials may be adopted. The transportation cost may be added to the tabulated cost if the materials are to be transported and the decision may be taken accordingly. A few tables of such structurally possible designs according to existing provisions and provisions incorporating the suggested amendments in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) are given in Appendices G and H respectively.

The existing provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) render the high-strength concrete bridges uneconomical. Thus economic studies were undertaken in two ways:

I) Analysis and cost effective design of the slab-on-girder bridges according to existing provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) using normal-strength concrete having 28days compressive strength less than 60 MPa.

II) Analysis and cost effective design according to the provisions including the proposed amendments in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) using the normal- as well as high-strength concrete.

5.4.1 Economic studies according to the existing provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997)

The existing provisions of these codes do not allow any advantage of using high-strength concrete in highway bridges. Thus, high-strength concrete has not been considered in economic studies according to the existing provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997).

5.4.1.1 Bridge superstructure cost v/s SRATIO

The parametric study have shown that SRATIO affected the magnitude of the internal forces developed in both the girders and the deck slab. Thus it is expected that the cost of the bridge superstructure should vary with the

SRATIO. Table 20 contains the results from Appendix G-3 which clearly show the variation of the cost of the superstructure with the SRATIO.

The present investigation has shown that the value of SRATIO corresponding to minimum cost of the bridges reduces as the number of girders increases. ~~reduces~~. Similar results were also reported by Hassanain and Loov (1999).

5.4.1.2 Bridge superstructure cost v/s girder concrete strength

From Figs. 36-38 it can be observed that for smaller span bridges, the minimum cost of superstructure corresponds to the concrete strength in the vicinity of 40 MPa. For longer span bridges, the concrete strength corresponding to minimum cost is about 45 MPa. If the concrete strength is higher than 50 MPa, the cost of the superstructure continuously increases. This is due to the fact that IRC:18 - 1985 (1997) does not allow the continuous increase in the allowable stresses at transfer of prestress with the increase in its strength beyond 45 MPa.

5.4.1.3 Cost comparison of the existing bridge and the optimized designs

U. P. State Bridge Corporation Limited, Lucknow provided the design of the bridge (25m span, under IRC class A 2-lane loads) constructed by them at Raebareli. The cost of this bridge was calculated after modifying the dimensions of the girder and the deck slab to satisfy the provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997). The cost of the bridge was calculated according to the unit cost of various materials adopted in the present investigation. Table 21 shows the cost comparison of the bridge at Raebareli and the optimized designs obtained in the present investigation.

TABLE - 20
BRIDGE SUPERSTRUCTURE COST FOR DIFFERENT VALUES OF SRATIO

Sr. no.	IPS	Span (m)	NCB	IG	ICON	ICON1	NG	SRATO	TCOST
1	2	25	4	8	1	1	3	1.00	71.0524
2	4	25	12	8	1	1	2	1.25	71.7693
3	4	25	12	8	1	1	2	1.50	71.1892
4	2	25	4	8	1	1	3	1.75	68.9575
5	3	25	4	8	1	1	2	2.00	62.4771
6	3	25	4	8	1	1	2	2.25	62.1580*
7	3	25	4	8	1	1	2	2.50	62.6639
8	3	25	4	8	1	1	2	2.75	63.5057
9	3	25	4	8	1	1	2	3.00	64.4454

TABLE - 21
COST COMPARISON OF THE EXISTINF BRIDGE AND THE OPTIMIZED DESIGNS

Span = 25 m, Load = IRC class A 2-lane loads

Sr. No.	Girder Identity	Depth of girder	NG	NCB	IPS	ICON	ICON1	SRATIO	TCOST
1	UPSBC Lucknow	1700	2	5	3	1	2	1.52	62.6676
2	2	1600	2	4	4	2	2	2.25	62.2124
3	8	2030	2	4	3	1	2	2.25	62.0615

From the Table 21 it is clear that the bridge could have been constructed at a lesser cost by using the girders shallower than or deeper than the one used by the Bridge Corporation. The shallower section should be preferred as it is lighter and may further reduce the cost due to its lower cost of transportation and erection.

5.4.2 Economic studies incorporating the proposed amendments in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997)

5.4.2.1 Bridge superstructure cost v/s longitudinal girder concrete strength

Figs. 39 - 41 show the variation of the cost of a given span of the bridge with the longitudinal girder concrete strength. It can be observed that longitudinal girder concrete strength corresponding to minimum cost of superstructure increases with the span. These curves may be used in preliminary design to select the concrete grade and the girder depth corresponding to the minimum cost of the superstructure for a given span.

5.4.2.2 Maximum girder span v/s girder concrete strength

Figs. 51-53 show the variation of the maximum span under IRC class AA load that can be attained by the two, three and four girders systems of a particular grade of concrete. The variation of the maximum span under IRC class A load is similar to as shown in Figs. 51-53. From these figures it is clear that the maximum grade of concrete that can be used in the shallower sections reduces as the number of girders increases. For two and three girders system of girder section 1, the maximum attainable span length becomes constant beyond the girder

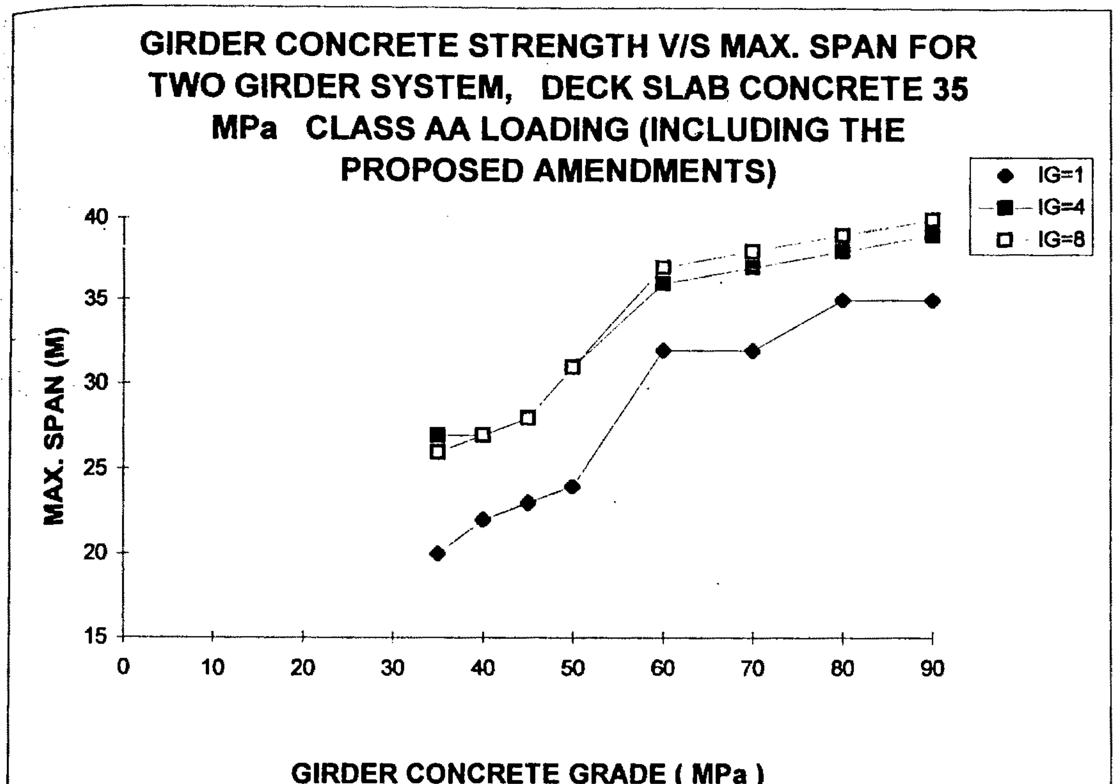

GIRDER CONCRETE STRENGTH V/S MAX. SPAN FOR TWO GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)
IG=1
IG=4
IG=8
MAX. SPAN (M)
40
35
30
25
20
15
0
10
20
30
40
50
60
70
80
90
GIRDER CONCRETE GRADE (MPa)

Fig. 51

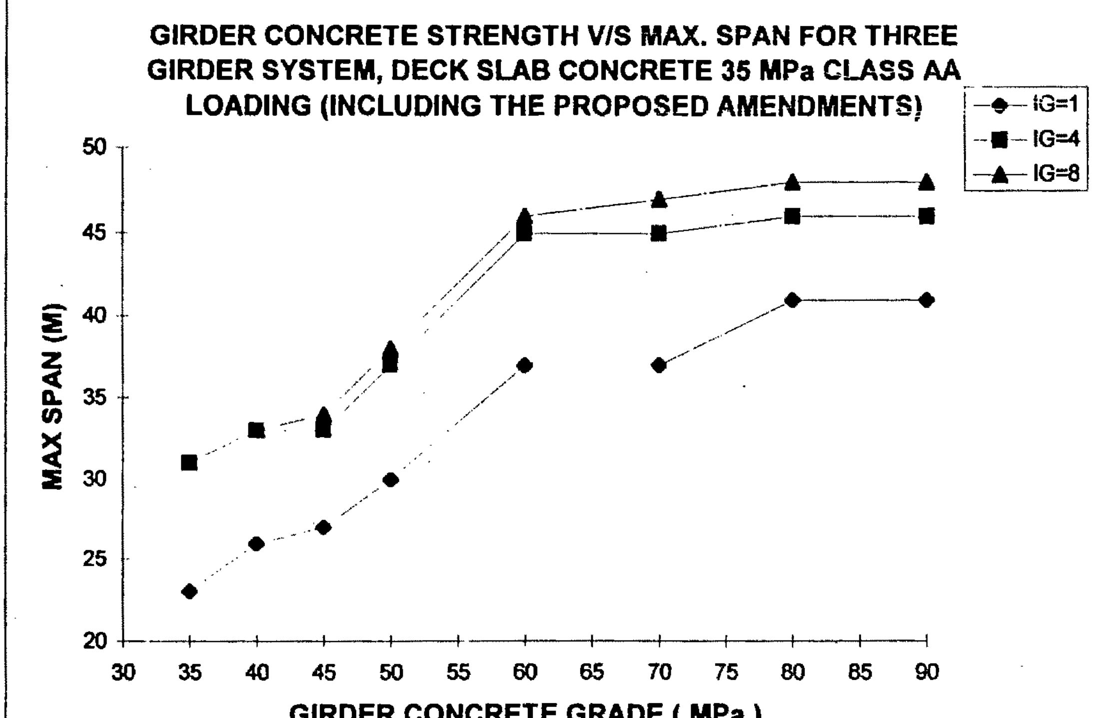

GIRDER CONCRETE STRENGTH V/S MAX. SPAN FOR THREE GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)
IG=1
IG=4
IG=8
MAX SPAN (M)
50
45
40
35
30
25
20
30
35
40
45
50
55
60
65
70
75
80
85
90
GIRDER CONCRETE GRADE (MPa)

Fig. 52

GIRDER CONCRETE STRENGTH V/S MAX. SPAN FOR FOUR GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)

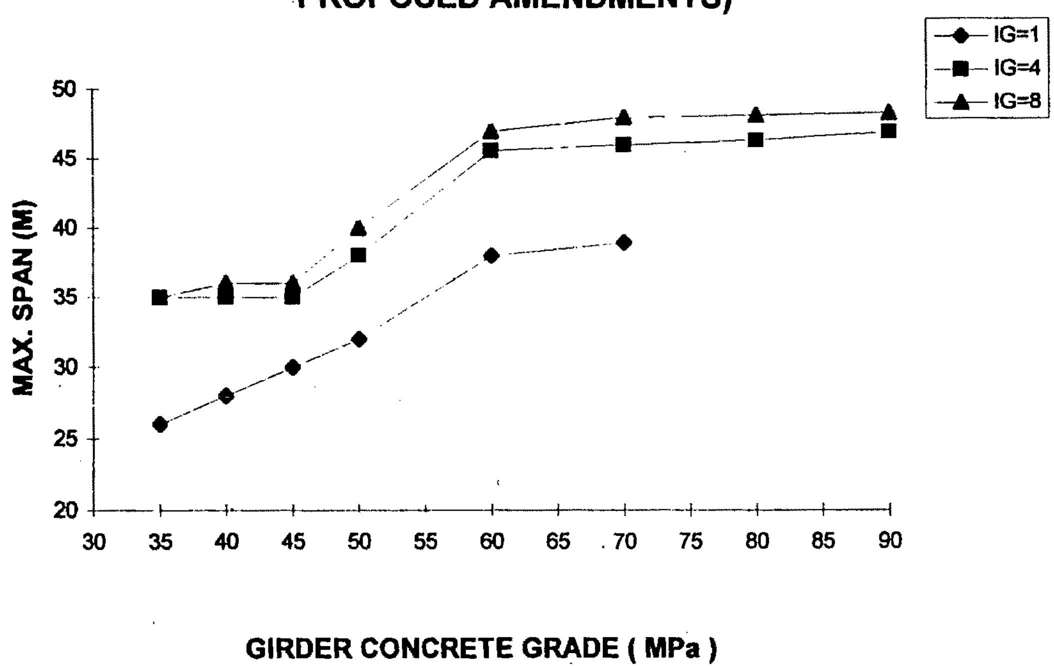


Fig. 53

concrete strength of 80 MPa. The maximum span corresponding to three girder system is longer than for two girders system. The concrete grade beyond 80 MPa is not advantageous in two and three girders system because of the failure of the girders due to excessive deflection. From Fig. 52 it may be observed that in a four girder system of shallowest section (section - 1), the use of concrete strength beyond 70 MPa is not advantageous and the maximum span attained is only 38m. Thus the maximum span and the maximum strength of the concrete that can be used by a four girder system is less than the corresponding values for a three girders system. This is due to the earlier failure of the girders due to excessive deflection in four girder system in comparison to three girder system. The effectiveness of the composite T-beam in a four girder system becomes less than in the three girder system because of the following reasons:

I)The deck slab thickness reduces as number of girders increases.

II) The effective flange width reduces as the number of girders increases.

III) The rate of reduction of the maximum bending moment in longitudinal girder with the increase in number of longitudinal girders reduces as the number of longitudinal girders becomes higher and higher.

From Fig. 51 it may be observed that the use of deep section may avoid the failure due to excessive deflection and higher strength concrete may be used effectively. For the two girders system of deeper sections, there is continuous increase in the span with the increase in the girder concrete strength. As number of girders is increased, the maximum attainable span also increases. For three and four girders systems of deeper sections, the rate of increase of the maximum span with concrete strength becomes very small (i.e. cost effectiveness reduces) beyond concrete strength of 70 MPa. The full advantage of high-strength concrete in three and four girders systems is not obtained due to the limitation of maximum available prestress force. Figs. 44 and 45 show the maximum span capability of girder sections 1 & 8 respectively for different grades of longitudinal girder concrete strength. The maximum span capability of the girder section 1 (the shallowest section) increases by 60% with the increase in girder concrete strength from 35 to 90 MPa. In case of the longitudinal girder section 8 (deepest section), the span increases by only 40% when girder concrete strength increases from 35 to 70 MPa. Beyond 70 MPa, the maximum girder span remains constant

5.4.2.3 Girder concrete strength v/s girder span corresponding to minimum superstructure cost

The results of the optimal design solutions for two, three and four girders systems under IRC class AA load corresponding to longitudinal girder sections 1, 4 and 8 are shown in Tables 22, 23 and 24 respectively. Tables 25,26 and 27 show the optimal design solutions for two, three and four girders systems respectively under IRC class A load corresponding to longitudinal girder sections 1, 4 and 8.

Figs. 54-56 and 57-59 show the variation of the span corresponding to the minimum cost with the variation of the longitudinal girder concrete strength for two, three and four girders systems under IRC class AA and class A loads respectively. These figures can be used by the designers at the preliminary design stage to determine the maximum achievable span with minimum cost for a particular girder section and given concrete strength.

TABLE - 22

MAXIMUM SPAN OF TWO GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS AA LOAD CORRESPONDING TO MINIMUM COST OF SUPERSTRUCTURE FOR DIFFERENT GRADES OF GIRDER CONCRETE STRENGTH

Sr. No.	FCK	Girder Section 1 Depth = 1500 mm		Girder Section 4 Depth = 1830 mm		Girder Section 8 Depth = 2030 mm	
		Span (m)	Cost	Span (m)	Cost	Span (m)	Cost
1	35	20	63.98	24	62.78	22	64.42
2	40			26	64.34		
3	45	22	64.33			28	65.83
4	50						
5	60	24	65.72	28	64.66	32	66.74
6	70	32	67.74	36	66.60		
7	80						
8	90	34	69.70	38	68.91	40	70.54

TABLE - 23

MAXIMUM SPAN OF THREE GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS AA LOAD CORRESPONDING TO MINIMUM COST OF SUPERSTRUCTURE FOR DIFFERENT GRADES OF GIRDER CONCRETE STRENGTH

Sr. No.	FCK	Girder Section 1 Depth = 1500 mm		Girder Section 4 Depth = 1830 mm		Girder Section 8 Depth = 2030 mm	
		Span (m)	Cost	Span (m)	Cost	Span (m)	Cost
1	35	24	67.37	26	68.07	28	68.49
2	40	26	68.66				
3	45			32	70.21	34	70.69
4	50	30	70.44				
5	60					40	72.61
6	70	34	71.28			44	75.51
7	80			40	72.50		
8	90	38	74.67	42	76.24	48	78.4

TABLE - 24

MAXIMUM SPAN OF FOUR GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS AA LOAD CORRESPONDING TO MINIMUM COST OF SUPERSTRUCTURE FOR DIFFERENT GRADES OF GIRDER CONCRETE STRENGTH

Sr. No.	FCK	Girder Section 1 Depth = 1500 mm		Girder Section 4 Depth = 1830 mm		Girder Section 8 Depth = 2030 mm	
		Span (m)	Cost	Span (m)	Cost	Span (m)	Cost
1	35	26	73.69	32	76.68	32	74.68
2	40			34	79.02	34	76.05
3	45	28	74.89				
4	50	32	77.63	38	82.11	40	81.32
5	60	38	86.29				
6	70			46	91.87	48	88.82
7	80						
8	90						

TABLE - 25

MAXIMUM SPAN OF TWO GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS A LOAD CORRESPONDING TO MINIMUM COST OF SUPERSTRUCTURE FOR DIFFERENT GRADES OF GIRDER CONCRETE STRENGTH

Sr. No.	FCK	Girder Section 1 Depth = 1500 mm		Girder Section 4 Depth = 1830 mm		Girder Section 8 Depth = 2030 mm	
		Span (m)	Cost	Span (m)	Cost	Span (m)	Cost
1	35	20	60.96	22	61.06	24	61.46
2	40	22	61.38	28	62.52		
3	45						
4	50						
5	60	24	61.39	34	63.26	28	63.25
6	70					38	66.54
7	80						
8	90	36	68.63	38	65.74	40	68.72

TABLE - 26

MAXIMUM SPAN OF THREE GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS A LOAD CORRESPONDING TO MINIMUM COST OF SUPERSTRUCTURE FOR DIFFERENT GRADES OF GIRDER CONCRETE STRENGTH

Sr. No.	FCK	Girder Section 1 Depth = 1500 mm		Girder Section 4 Depth = 1830 mm		Girder Section 8 Depth = 2030 mm	
		Span (m)	Cost	Span (m)	Cost	Span (m)	Cost
1	35	24	66.45	30	68.45	28	67.95
2	40					32	69.16
3	45	26	65.91	34	69.83	34	70.06
4	50			38	71.99	36	71.29
5	60	36	69.73			40	72.03
6	70	38	73.11	40	71.95		
7	80			44	77.95	42	73.99
8	90	40	77.89	46	77.90	48	77.65

TABLE - 27

MAXIMUM SPAN OF FOUR GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS A LOAD CORRESPONDING TO MINIMUM COST OF SUPERSTRUCTURE FOR DIFFERENT GRADES OF GIRDER CONCRETE STRENGTH

Sr. No.	FCK	Girder Section 1 Depth = 1500 mm		Girder Section 4 Depth = 1830 mm		Girder Section 8 Depth = 2030 mm	
		Span (m)	Cost	Span (m)	Cost	Span (m)	Cost
1	35	26	73.80	34	79.04	34	79.20
2	40	28	75.25			36	79.63
3	45	30	77.86				
4	50					40	81.81
5	60			44	91.47		
6	70			46	91.38		
7	80					48	89.28
8	90						

GIRDER CONCRETE STRENGTH V/S SPAN CORRESPONDING TO MINIMUM COST OF TWO GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)

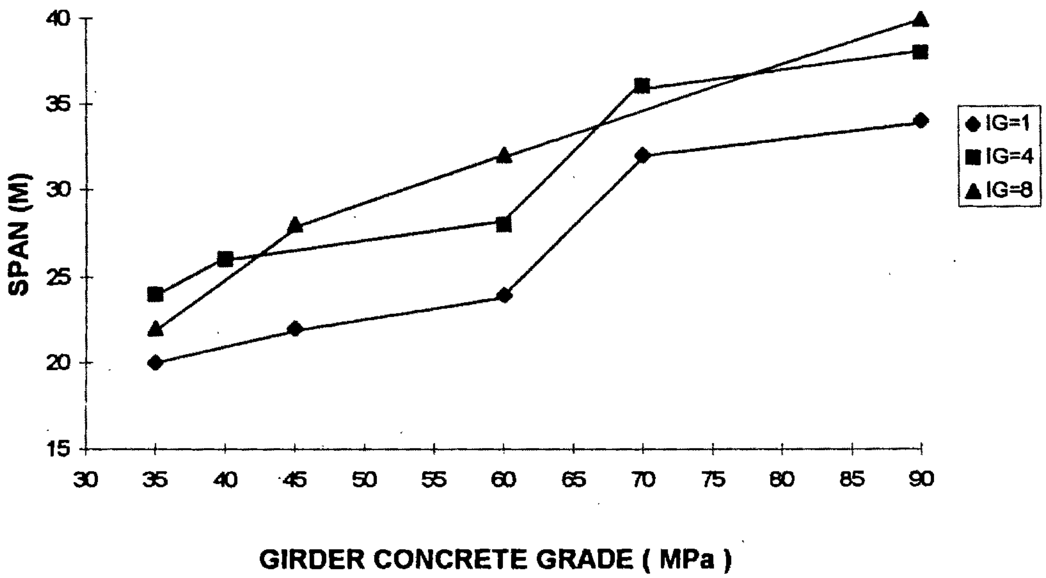


Fig. 54

GIRDER CONCRETE STRENGTH V/S SPAN CORRESPONDING TO MINIMUM COST OF THREE GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)

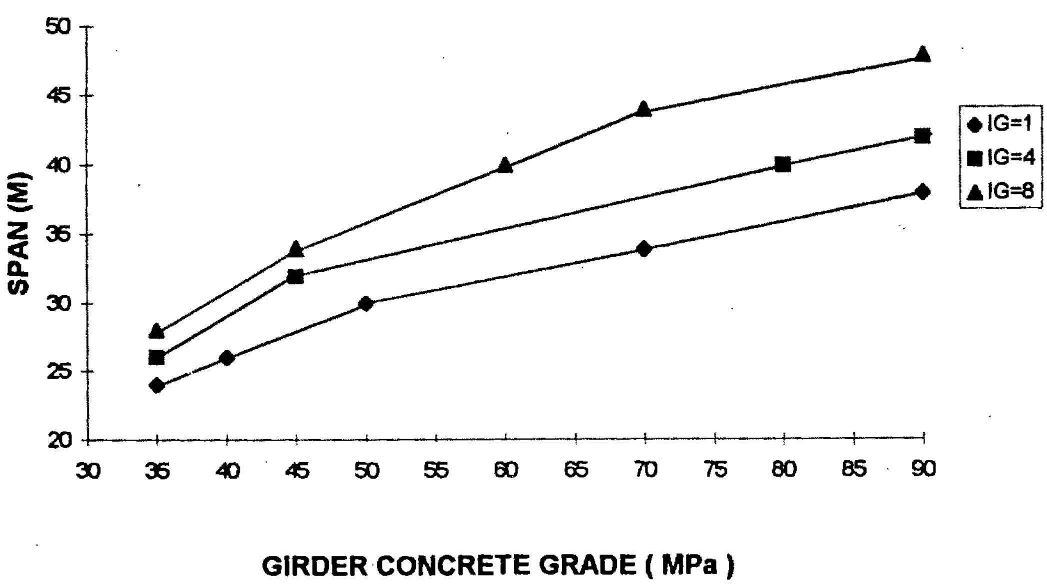


Fig. 55

GIRDER CONCRETE STRENGTH V/S SPAN CORRESPONDING TO MINIMUM COST OF FOUR GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)

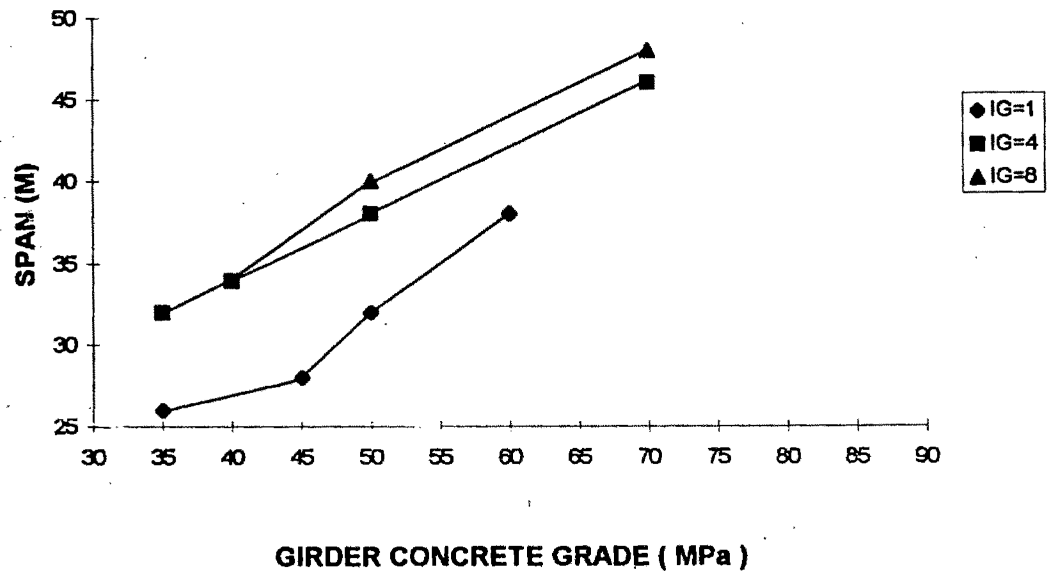


Fig. 56

GIRDER CONCRETE STRENGTH V/S SPAN CORRESPONDING TO MINIMUM COST OF TWO GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS A LOADING (INCLUDING THE PROPOSED AMENDMENTS)

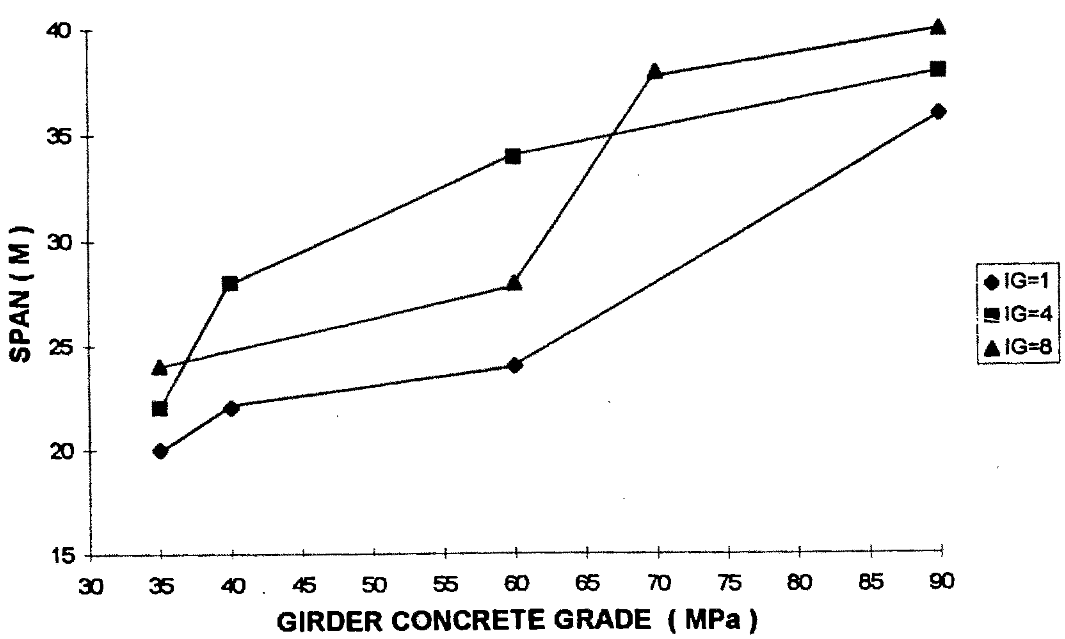


Fig. 57

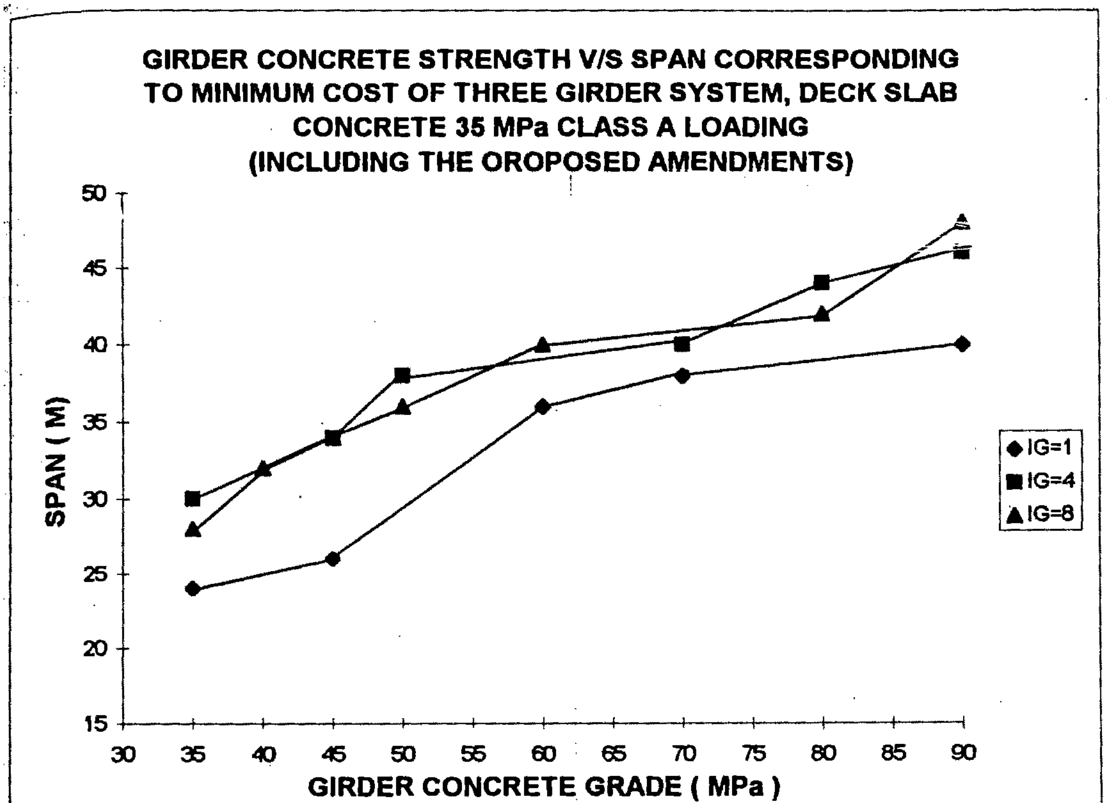

GIRDER CONCRETE STRENGTH V/S SPAN CORRESPONDING TO MINIMUM COST OF THREE GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS A LOADING (INCLUDING THE OROPOSED AMENDMENTS)
SPAN (M)
GIRDER CONCRETE GRADE (MPa)
IG=1
IG=4
IG=8

Fig.58

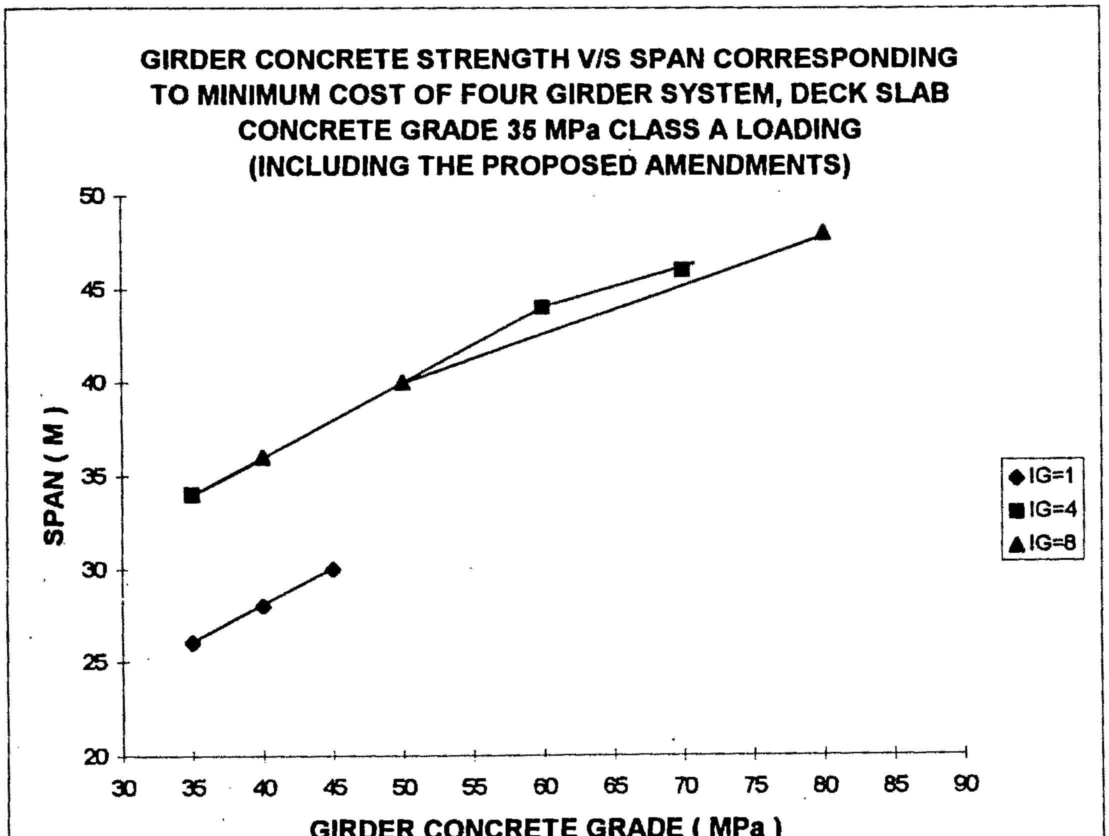

GIRDER CONCRETE STRENGTH V/S SPAN CORRESPONDING TO MINIMUM COST OF FOUR GIRDER SYSTEM, DECK SLAB CONCRETE GRADE 35 MPa CLASS A LOADING (INCLUDING THE PROPOSED AMENDMENTS)
SPAN (M)
GIRDER CONCRETE GRADE (MPa)
IG=1
IG=4
IG=8

Fig. 59

5.4.2.4 Optimal design of bridges

I) No restriction on maximum available concrete strength

A preliminary optimal design of slab-on-girder bridge of the given span is possible by using Figs. 60 to 65 and 66 to 71 for IRC class AA and class A loads respectively These figures are for two, three and four girders bridges covering the concrete strength up to 90 MPa.

For a given span, Figs. 60 to 62 and 66 to 68 may be used for deciding the type (depth of the girder) and the number of the girders corresponding to minimum cost of the bridge superstructure under IRC class AA and class A loads respectively. Corresponding to this minimum cost of the superstructure, the concrete grade may be chosen from Figs. 63 to 65 and 69 to 71 for two, three and four girders bridges under IRC class AA and class A loads respectively. If concrete grade corresponding to minimum cost falls in between the grades used in study, then two possibilities are there:

I) Use concrete grade just higher to the one corresponding to the minimum cost. The details of the parameters of the bridge superstructure may be taken from the tables given in Appendix G and H.

II) For the strength of the concrete corresponding to the minimum cost, the computer programme developed in the present study may be run and the values of the different parameters of the bridge superstructure may be found.

Figs. 60 to 71 are based on the deck slab concrete strength of 35 MPa. Similar curves for the deck slab concrete strength from 30 to 50 MPa are available with the author.

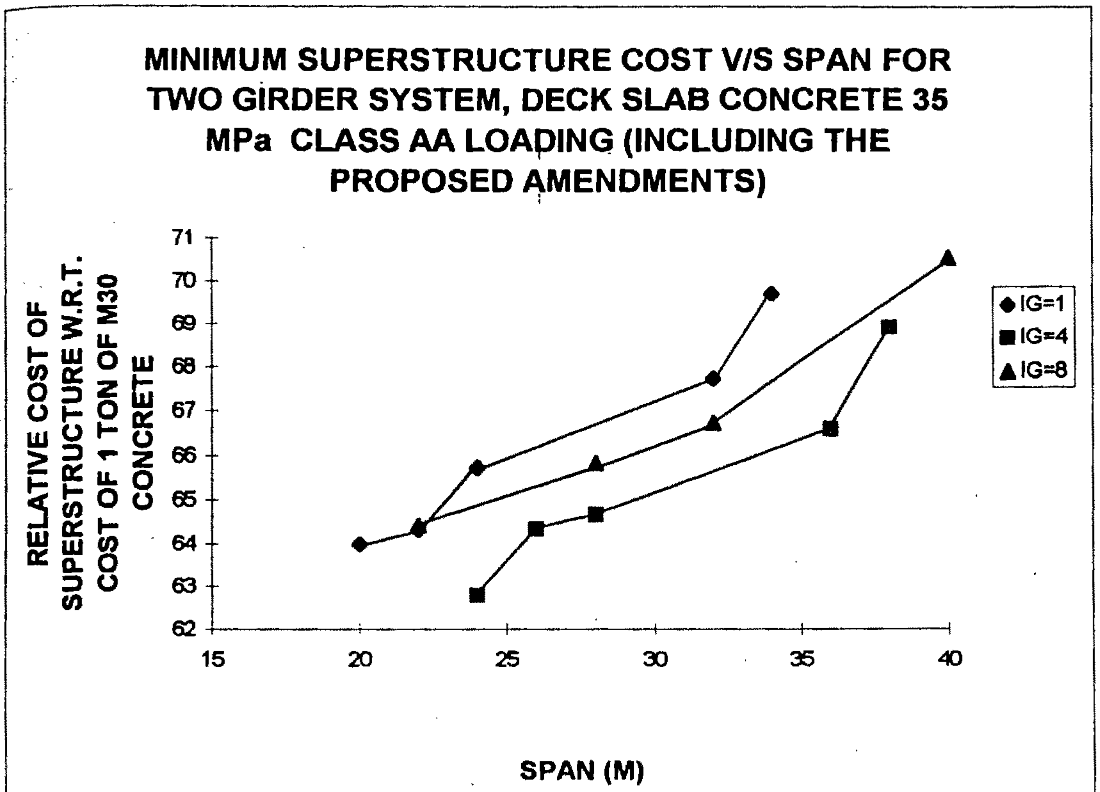

MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR TWO GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF M30 CONCRETE
71
70
69
68
67
66
65
64
63
62
15
20
25
30
35
40
IG=1
IG=4
IG=8
SPAN (M)

Fig. 60

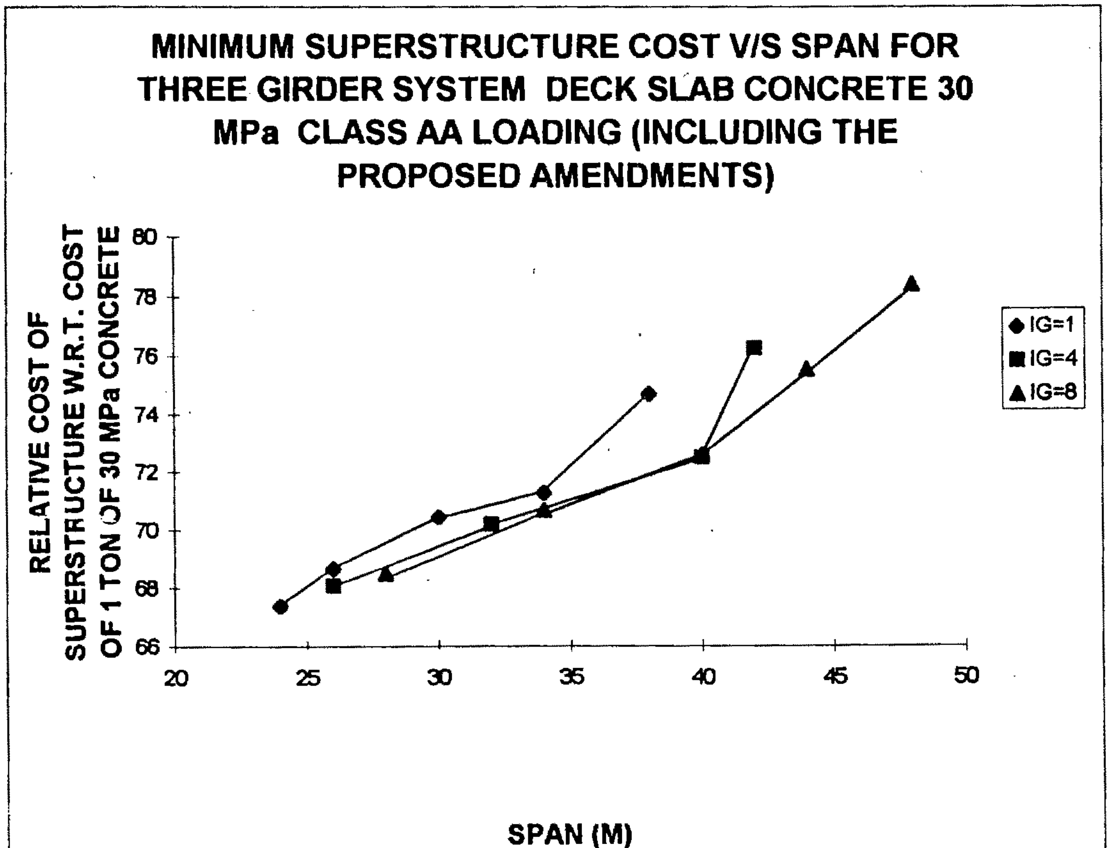
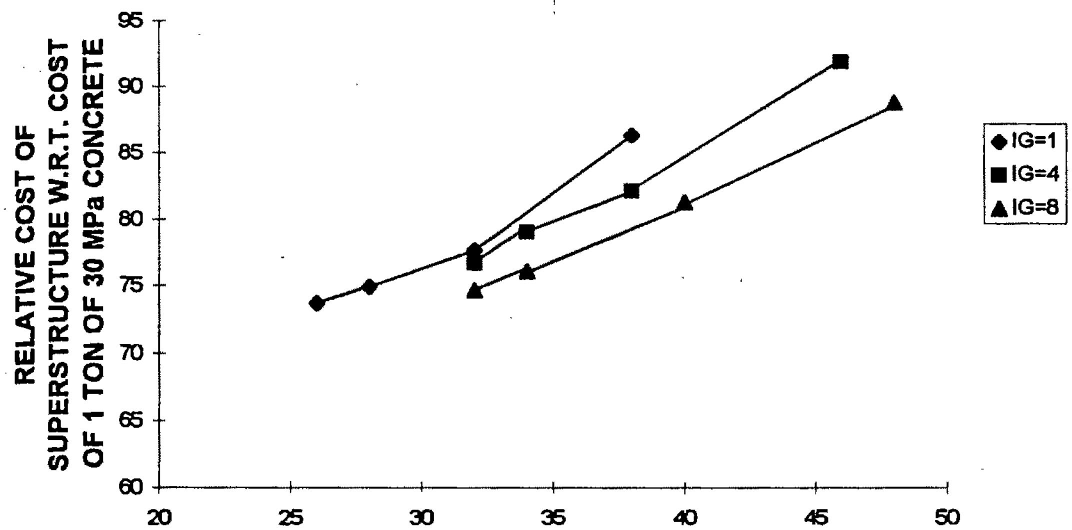

MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR THREE GIRDER SYSTEM DECK SLAB CONCRETE 30 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF 30 MPa CONCRETE
80
78
76
74
72
70
68
66
20
25
30
35
40
45
50
IG=1
IG=4
IG=8
SPAN (M)

Fig. 61

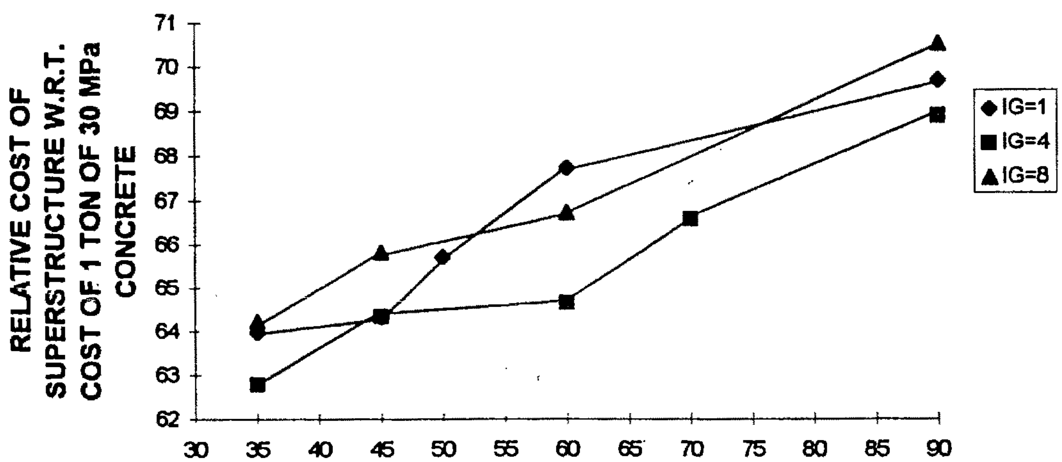
MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR FOUR GIRDER SYSTEM DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)



SPAN (M)

Fig. 62

GIRDER CONCRETE STRENGTH V/S MINIMUM COST OF SUPERSTRUCTURE OF TWO GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa, CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)



GIRDER CONCRETE GRADE (MPa)

Fig. 63

GIRDER CONCRETE STRENGTH V/S MINIMUM SUPERSTRUCTURE COST FOR THREE GIRDER SYSTEM, DECK SLAB CONCRETE 35 MPa, CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)

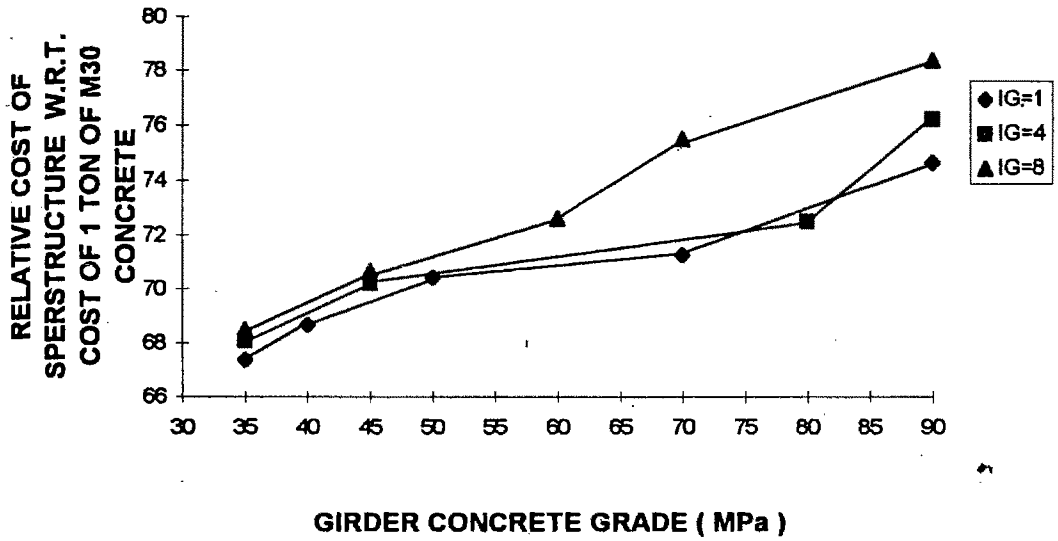


Fig. 64

GIRDER CONCRETE STRENGTH V/S MINIMUM SUPERSTRUCTURE COST FOR FOUR GIRDER SYSTEM, DECK SLAB CONCRETE GRADE 35 MPa, CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)

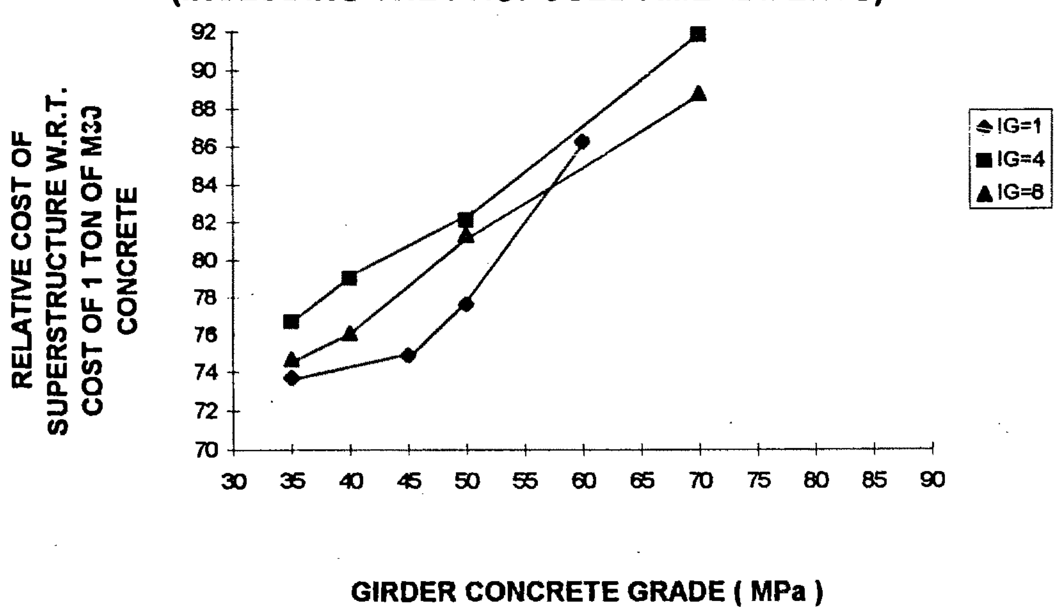


Fig. 65

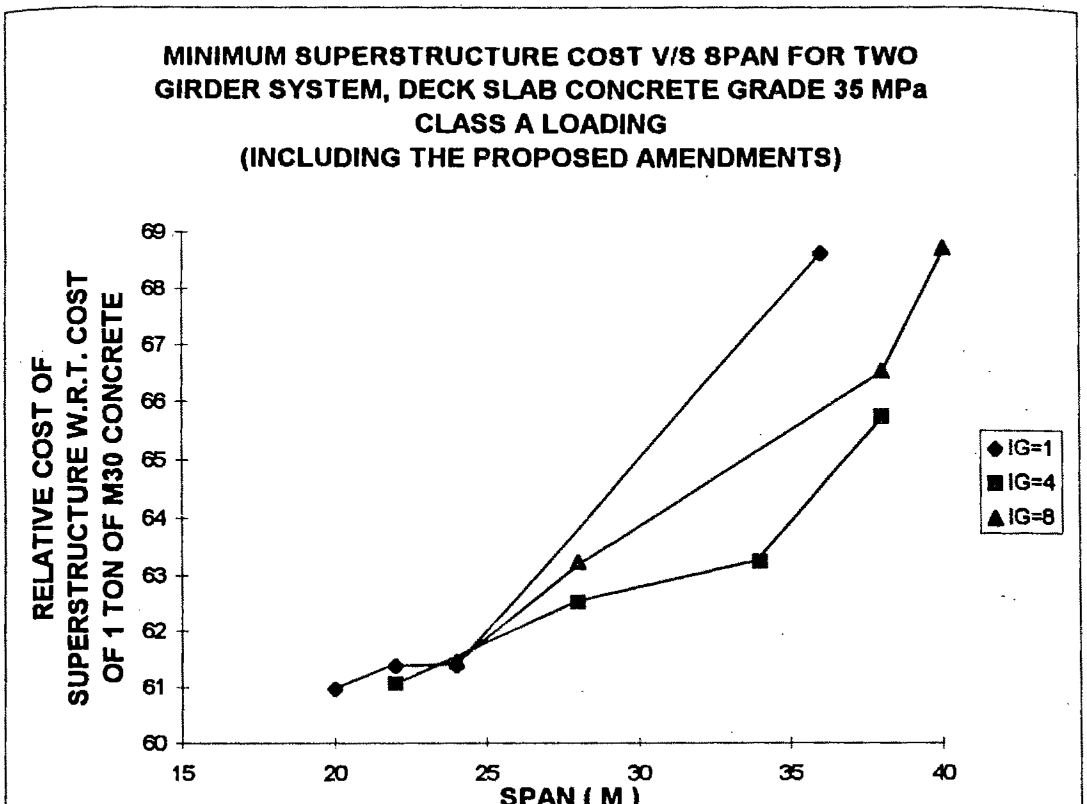

MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR TWO GIRDER SYSTEM, DECK SLAB CONCRETE GRADE 35 MPa
CLASS A LOADING
(INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF M30 CONCRETE
60
61
62
63
64
65
66
67
68
69
15
20
25
30
35
40
SPAN (M)
IG=1
IG=4
IG=8

Fig.66

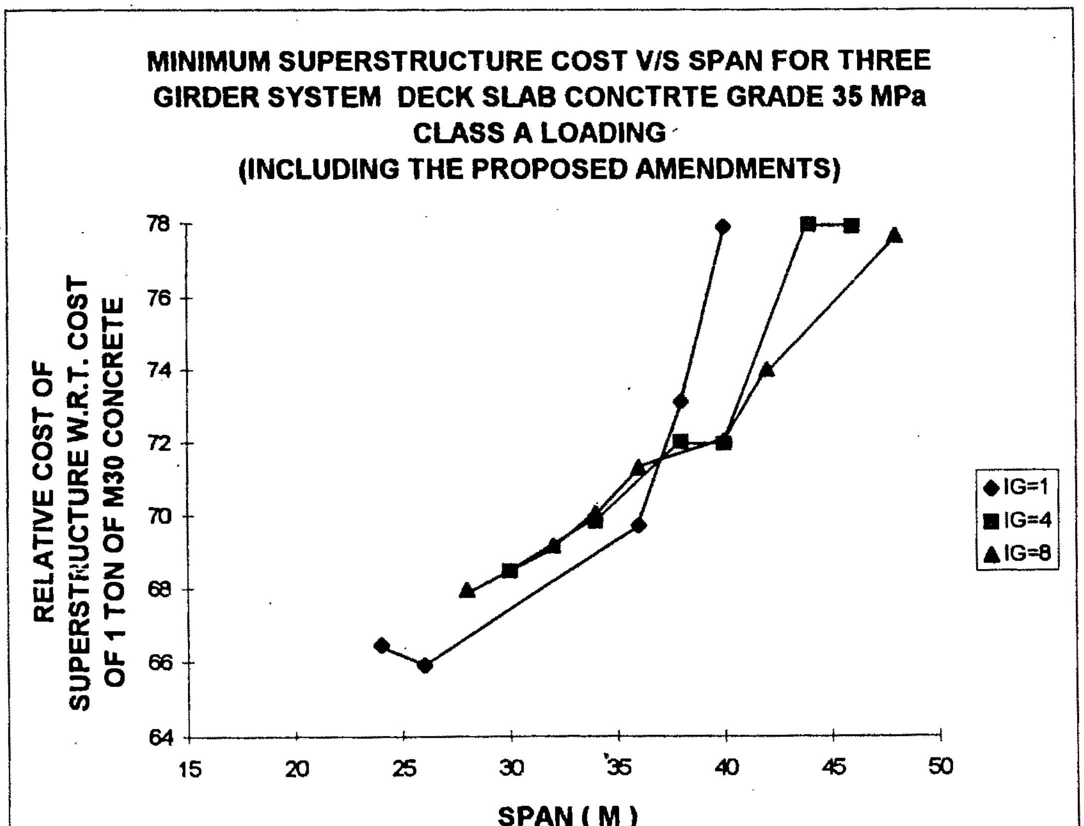

MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR THREE GIRDER SYSTEM DECK SLAB CONCTRTE GRADE 35 MPa
CLASS A LOADING
(INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF M30 CONCRETE
64
66
68
70
72
74
76
78
15
20
25
30
35
40
45
50
SPAN (M)
IG=1
IG=4
IG=8

Fig. 67

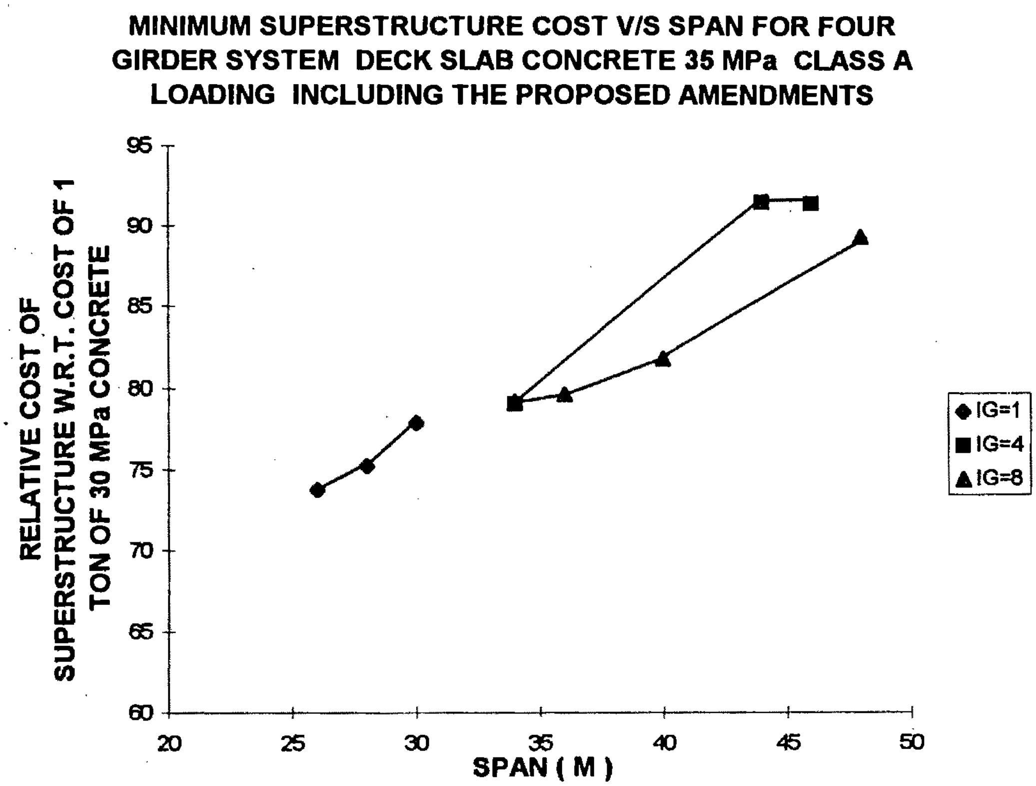

MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR FOUR GIRDER SYSTEM DECK SLAB CONCRETE 35 MPa CLASS A LOADING INCLUDING THE PROPOSED AMENDMENTS
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF 30 MPa CONCRETE
95
90
85
80
75
70
65
60
20
25
30
35
40
45
50
SPAN (M)
IG=1
IG=4
IG=8

Fig.68

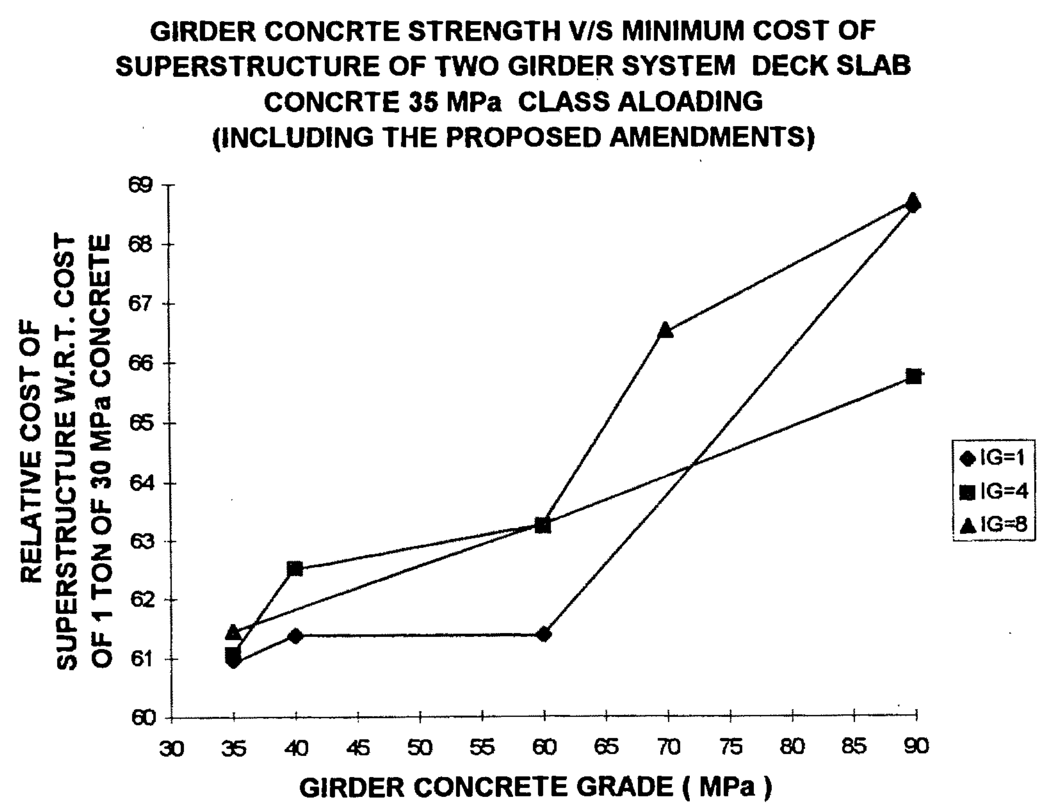

GIRDER CONCRTE STRENGTH V/S MINIMUM COST OF SUPERSTRUCTURE OF TWO GIRDER SYSTEM DECK SLAB CONCRTE 35 MPa CLASS ALOADING (INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF 30 MPa CONCRETE
69
68
67
66
65
64
63
62
61
60
30
35
40
45
50
55
60
65
70
75
80
85
90
GIRDER CONCRETE GRADE (MPa)
IG=1
IG=4
IG=8

Fig.69

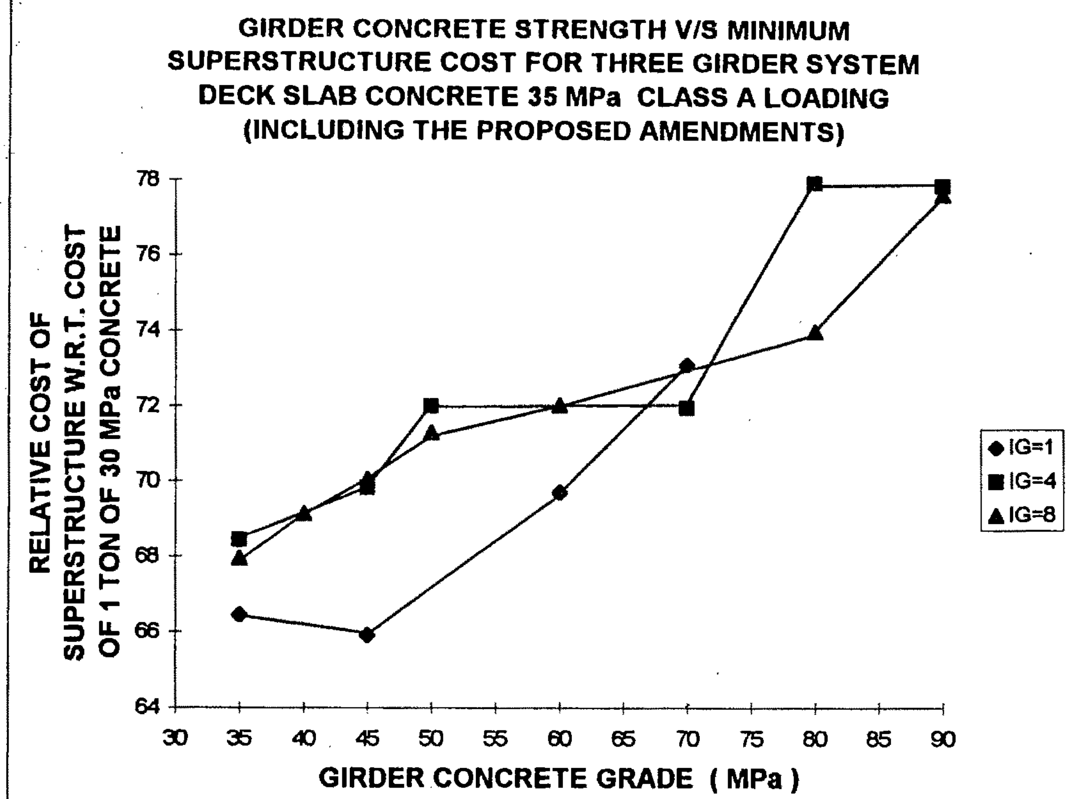

GIRDER CONCRETE STRENGTH V/S MINIMUM SUPERSTRUCTURE COST FOR THREE GIRDER SYSTEM DECK SLAB CONCRETE 35 MPa CLASS A LOADING (INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF 30 MPa CONCRETE
64
66
68
70
72
74
76
78
30
35
40
45
50
55
60
65
70
75
80
85
90
GIRDER CONCRETE GRADE (MPa)
IG=1
IG=4
IG=8

Fig. 70

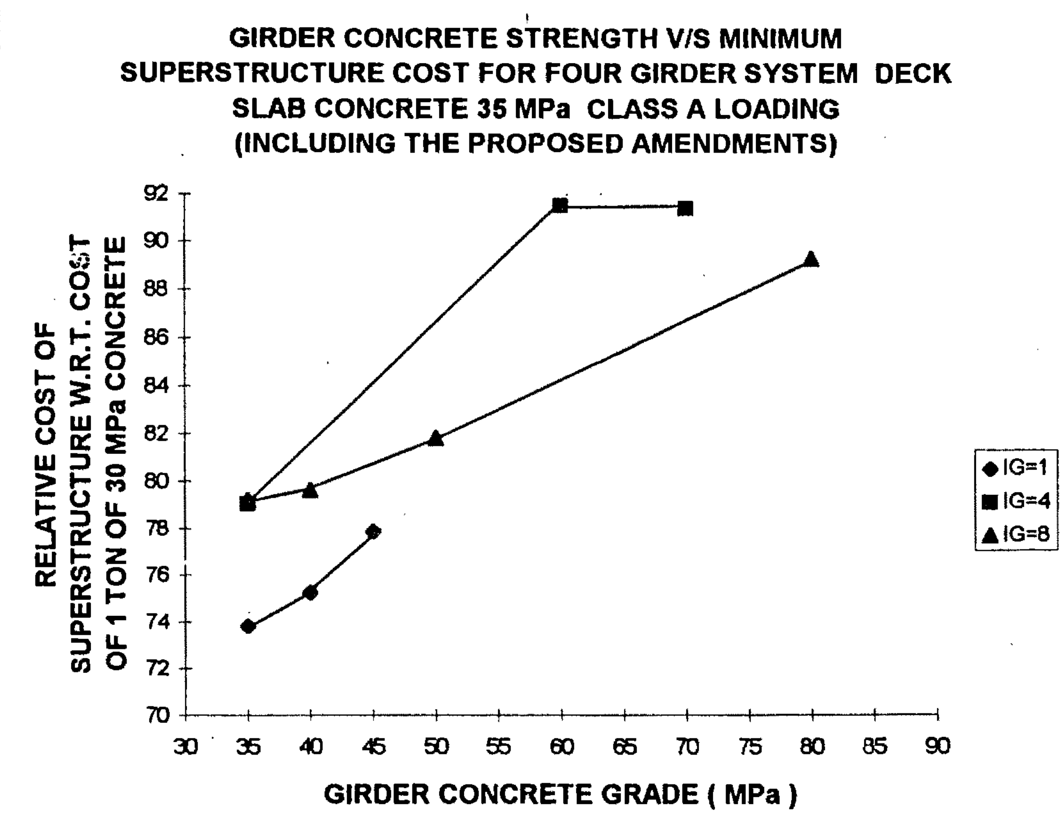

GIRDER CONCRETE STRENGTH V/S MINIMUM SUPERSTRUCTURE COST FOR FOUR GIRDER SYSTEM DECK SLAB CONCRETE 35 MPa CLASS A LOADING (INCLUDING THE PROPOSED AMENDMENTS)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF 30 MPa CONCRETE
70
72
74
76
78
80
82
84
86
88
90
92
30
35
40
45
50
55
60
65
70
75
80
85
90
GIRDER CONCRETE GRADE (MPa)
IG=1
IG=4
IG=8

Fig. 71

II) Concrete grades in the deck slab and the girders are predetermined

If there is restriction on the maximum possible strength of the concrete that may be economically produced using local materials, the maximum concrete grade to be used in the design should be selected taking into account the additional cost of transportation of the materials for producing higher grades of concrete. Two possibilities may arise out of this situation:

I) The preliminary design may be done for the strength of the concrete corresponding to the minimum cost for the given span. If the concrete grade corresponding to minimum cost of bridge superstructure is higher than possible to produce using local materials, the cost of transportation must be added to the calculated cost to find the actual cost of the superstructure.

II) The design tables given in Appendices G and H may be used to find the most economical design corresponding to the minimum grade of concrete possible to produce using the local materials.

The above two discussed possible designs for the given span should be compared and the economical one is to be adopted as preliminary design of the bridge.

5.4.2.5 Optimal concrete strength for two, three and four girders bridges

Figs. 72 and 73 show the minimum cost curves for slab-on-girder bridges under IRC class AA and class A loads respectively. The girder depth may vary from 1500 mm (IG = 1) to 2030 mm (IG = 8) and girder concrete strength varying in the range from 35 to 90 MPa.

MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR GIRDER SECTIONS 1, 4 & 8 AND 2, 3 & 4 GIRDER SYSTEMS, DECK SLAB CONCRETE 35 MPa CLASS AA LOADING (INCLUDING THE PROPOSED AMENDMENTS)

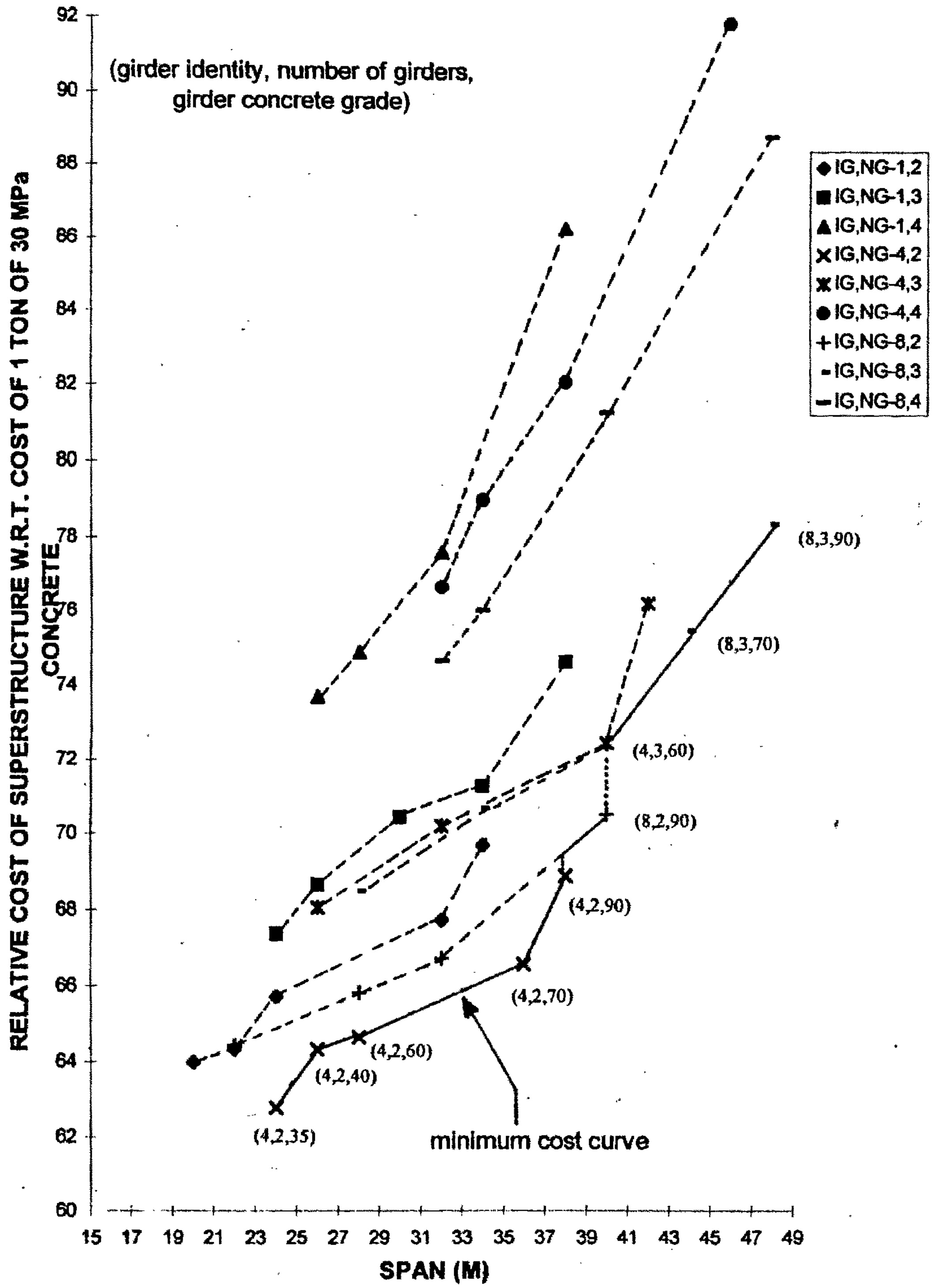


Fig. 72

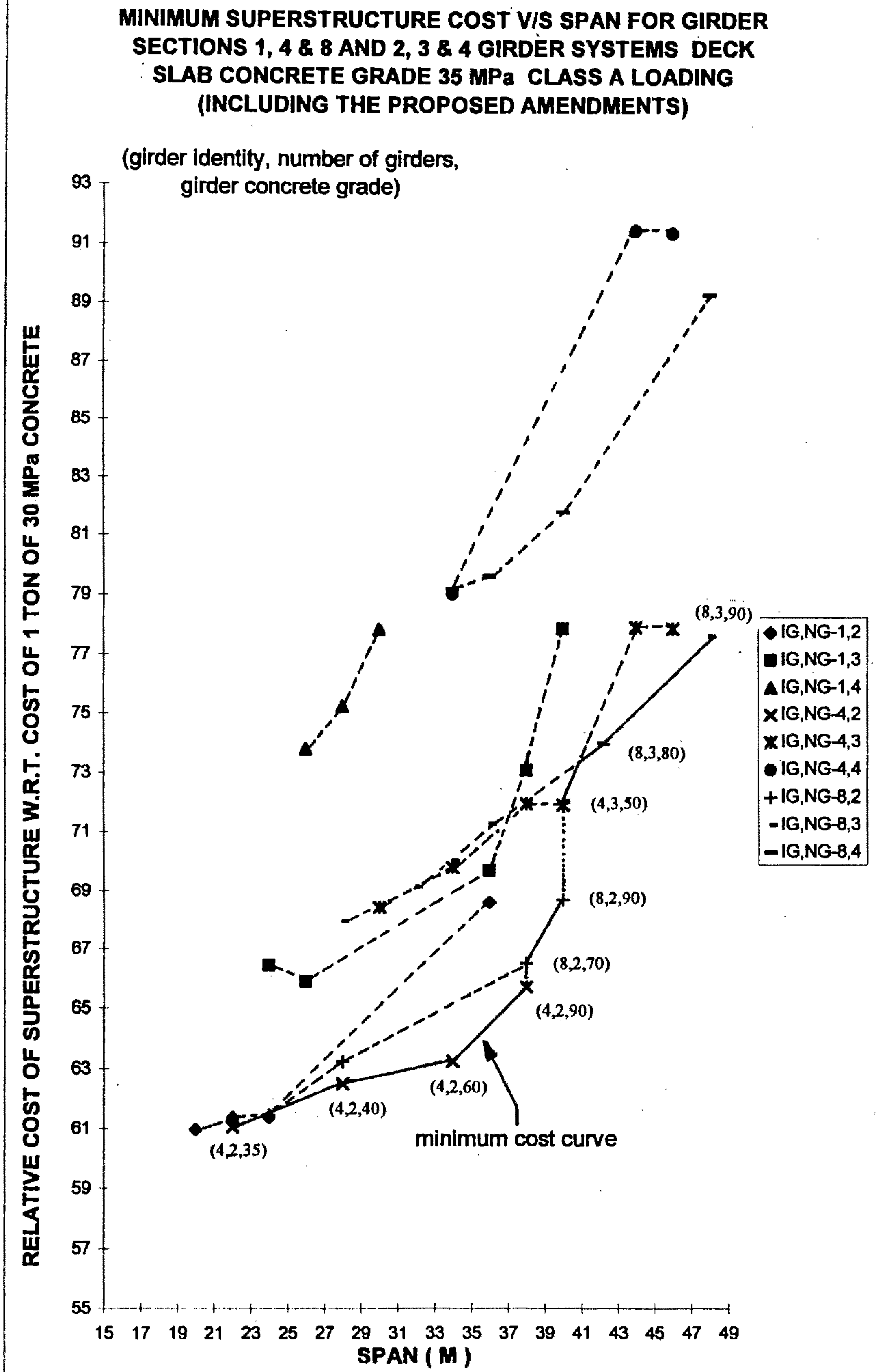

MINIMUM SUPERSTRUCTURE COST V/S SPAN FOR GIRDER SECTIONS 1, 4 & 8 AND 2, 3 & 4 GIRDER SYSTEMS DECK SLAB CONCRETE GRADE 35 MPa CLASS A LOADING (INCLUDING THE PROPOSED AMENDMENTS)
(girder identity, number of girders, girder concrete grade)
RELATIVE COST OF SUPERSTRUCTURE W.R.T. COST OF 1 TON OF 30 MPa CONCRETE
SPAN (M)
(8,3,90)
(8,3,80)
(4,3,50)
(8,2,90)
(8,2,70)
(4,2,90)
(4,2,60)
(4,2,40)
(4,2,35)
minimum cost curve
IG,NG-1,2
IG,NG-1,3
IG,NG-1,4
IG,NG-4,2
IG,NG-4,3
IG,NG-4,4
IG,NG-8,2
IG,NG-8,3
IG,NG-8,4

Fig.73

TABLE - 28

OPTIMAL CONCRETE STRENGTH FOR TWO GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS AA LOAD

Sr. No.	Span Limit (m)	Optimal girder concrete strength, f_{ck} (MPa)
1	to 24	35
2	to 26	40
3	to 28	60
4	to 36	70
5	to 40	90

TABLE - 29

OPTIMAL CONCRETE STRENGTH FOR THREE GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS AA LOAD

Sr. No.	Span Limit (m)	Optimal girder concrete strength, f_{ck} (MPa)
1	to 28	35
2	to 34	45
3	to 40	60
4	to 44	70
5	to 48	90

TABLE - 30

OPTIMAL CONCRETE STRENGTH FOR FOUR GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS AA LOAD

Sr. No.	Span Limit (m)	Optimal girder concrete strength, f_{ck} (MPa)
1	to 32	35
2	to 40	50
3	to 48	70

TABLE - 31

OPTIMAL CONCRETE STRENGTH FOR TWO GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS A LOAD

Sr. No.	Span Limit (m)	Optimal girder concrete strength, f_{ck} (MPa)
1	to 22	35
2	to 28	40
3	to 34	60
4	to 38	70
5	to 40	90

TABLE - 32

OPTIMAL CONCRETE STRENGTH FOR THREE GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS A LOAD

Sr. No.	Span Limit (m)	Optimal girder concrete strength, f_{ck} (MPa)
1	to 24	35
2	to 26	45
3	to 36	60
4	to 40	70
5	to 42	80
6	to 48	90

TABLE - 33

OPTIMAL CONCRETE STRENGTH FOR FOUR GIRDERS SLAB-ON-GIRDER BRIDGES UNDER IRC CLASS A LOAD

Sr. No.	Span Limit (m)	Optimal girder concrete strength, f_{ck} (MPa)
1	to 34	35
2	to 36	40
3	to 40	50
4	to 48	80

The optimal concrete strength for different spans is summarised in Tables 28 to 30 and 31 to 33 for two, three and four girders bridges under IRC class AA and class A loads respectively . From Figs. 72 and 73 it may be observed that there is little advantage of using four girders system if there is limitation on the adaptability and availability of the maximum prestress force.

5.5 Conclusions

From the parametric study it is observed that the deck slab thickness and bending moment and shear force in the longitudinal girders vary significantly with the variation in the values of different parameters of the bridge superstructure. The reduction in the girder spacing up to a certain value reduces the cost to the minimum. However, further reduction in the girder spacing causes the increase in the superstructure cost.

The existing provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) are not suitable for high-strength concrete application in highway bridges. There is the need of amendments in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) so that high-strength concrete may be used in highway bridges with full economical and structural advantages.

The results obtained using the proposed amendments in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) are very much in agreement with the results of similar investigations available in the literature.

The optimized designs obtained using the methodology and computer programme developed in the present investigation are more economical as compared to the one adopted for an existing bridge.

6. CONCLUSIONS AND FUTURE SCOPE

6.1 Introduction

The present investigation has successfully examined the following facts on composite prestressed concrete slab-on-girder bridges:

I) The influence of the variation in the values of various parameters of the bridge superstructure on the internal forces in the deck slab and the longitudinal girders. For the purpose of parametric study, complete superstructure and actual IRC loads are considered so that the effect of the interaction among the various parameters of the bridge superstructure and the magnitude and placing of the IRC loads is also reflected.

II) The various provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) are critically reviewed with reference to their applicability to high-strength concrete. The results of the investigation are critically discussed. The inability of the provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) to use high-strength concrete in highway bridges advantageously is explained in chapter 5.

III) The mechanical properties of high-strength concrete are very much different from those of normal-strength concrete. Based on this fact, amendments are proposed in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997). The same slab-on-girder bridges are analysed and designed as per existing provisions and including the proposed amendments in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997). The proposed amendments enable the codes to permit the advantageous use of high-strength concrete in highway bridges. The results of the present investigation are comparable to the results of the similar investigations by other researchers.

IV) A computer programme in FORTRAN has been developed for the analysis and optimal design of the slab-on-girder bridges of a given span and specified grades of concrete in the deck slab and the longitudinal girders. The optimal designs obtained in the investigation are compared with the design of an existing bridge.

6.2 Conclusions

The in-depth and extensive analytical work carried out in this study on prestressed concrete simply supported slab-on-girder bridges has led to the following conclusions:

6.2.1 Parametric studies

1) The maximum bending moment in the longitudinal girders due to the IRC loads (class AA and class A) remains practically unchanged with the variation in the number of cross beams over a wide range of 3 to 12.

2) An increase in the number of the longitudinal girders beyond a certain number (depending up on the span of the bridge) does not produce appreciable benefits in terms of the magnitude of the internal forces developed in the longitudinal girders.

3) The use of higher number of longitudinal girders is more beneficial for longer span bridges compared to the smaller span bridges.

4) Over a given range of the bridge spans, the percentage increase in the bending moment in the longitudinal girders is higher for a larger number of longitudinal girders compared to the smaller number of longitudinal girders.

5) The maximum bending moment in the longitudinal girder of the shorter span bridges is critical due to IRC class AA loads, whereas for longer span bridges it is critical due to IRC class A loads.

6) The maximum bending moment in the longitudinal girders reduces with the increase in the value of SRATIO. The effect of SRATIO is more significant in the longer span bridges and when the number of the longitudinal girders is small.

7) For the purpose of calculation of bending moment in the longitudinal girders, the complicated IRC loads may be replaced by an equivalent uniformly distributed load (EUDL). The shear force may be computed by the EUDL associated with a point load acting at the center of the span. A few tables giving equivalent uniformly distributed load (EUDL) for various combinations of the different parameters of bridge superstructure are shown in Appendix F.

8) Within 5.5m from the either supports, the maximum shear force due to IRC class AA loads is very high when the longitudinal girders are odd in number as compared to when the number of longitudinal girders is an even number.

9) There is very little change in the maximum shear force in the longitudinal girders under IRC class AA loads as the span of the bridge changes. However, under IRC class A loads there is an appreciable increase in the maximum shear force as the span of the longitudinal girder increases.

10) The maximum shear force in the longitudinal girder under IRC class AA loads reduces appreciably with the increase in the value of SRATIO. In contrast to this,

in case of IRC class A loads there is very little change in the maximum shear force in the longitudinal girder with the increase in the value of SRATIO over a range from 1 to 2.

11) For a given number of cross beams, the deck slab thickness changes appreciably in the longer span bridges with the change in number of the longitudinal girders as compared to the smaller span bridges.

12) For a given span and number of cross beams, the variation of the deck slab thickness with the number of longitudinal girders is higher when number of longitudinal girders is smaller as compared to the higher number of longitudinal girders.

13) The deck slab thickness may increase or decrease with SRATIO depending upon the different combinations of the number of longitudinal girders and cross beams.

6.2.2 Need of amendments in IRC:18-1985 (1997) and IRC:21-1987 (1997)

The existing provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) do not encourage the use of high-strength concrete in highway bridges. The following results of high-strength concrete application in highway bridges led to the necessity of amendments in these codes.

1) In general, the concrete grade for the longitudinal girders of the bridge, corresponding to the minimum cost of the bridge superstructure, increases with the increase in the span of the bridge. But the results of this investigation [based on the provisions of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997)] do not follow this trend. The minimum cost of the superstructure for various spans ranging from

18 to 36m corresponds to the longitudinal girder concrete strength in the range of 40 to 60 MPa.

2) No enhancement in the maximum span capability of the longitudinal girders is observed when high-strength concrete is used in the longitudinal girders.

3) In the present investigation, generally no reduction in the number of longitudinal girders is observed with an increase in the longitudinal girder concrete strength. In some of the cases even the number of longitudinal girders increases with the increase in the concrete strength.

4) The maximum spans attained by the longitudinal girder cross-sections 1 and 8 under IRC class AA and A loads, variation of number of longitudinal girder and several grades of concrete are only 90% and 77% of that reported by Jacques (1971).

5) IRC:21 - 1987 (1997) gives no advantage of application of concrete strength higher than 35 MPa in the deck slab.

6) The various recommendations of IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) are not in agreement with the findings of ACI Committee 363 (1984), Attard and Setunge (1996) and Iravani (1996).

6.2.3 Proposed amendments in IRC:18-1985 (1997) and IRC:21-1987 (1997)

Basic mechanical properties of high-strength concrete like, secant modulus, split cylinder strength, modulus of rupture strength and stress-strain relationship etc. are very much different (in general higher) than those of normal-strength concrete. Thus following amendments are proposed in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) with due considerations to various associated aspects like, permissible stress at transfer of prestress and service loads, minimum flexural and shear reinforcement and the ultimate strength expressions.

6.2.3.1 Proposed revisions in IRC:21-1987 (1997)

The revisions proposed in this code are basically related with the deck slab of the composite slab-on-girder bridges. The maximum strength of the concrete used in deck slab in the present investigation is 50 MPa as use of higher strength concrete did not result in advantage. However, the proposed revisions are recommended for concrete strength up to 60 MPa.

1) Clause number 303.1

The permissible compressive stress in flexure be taken as 33% of 28 day cube strength up to 60 MPa.

Secant modulus of elasticity of normal weight concrete in the strength range of 20 to 100 MPa should be taken as

$$Ec = 8910 \times \sqrt[0.325]{fck} \quad \text{MPa}$$

2) Clause number 304.7.2

The maximum permissible shear stress of concrete for the deck slabs, the concrete strength ranging from 20 to 60 MPa, shall be given by the following expression

$$\tau_{\max} = 0.3675 \sqrt{fck}$$

3) Clause number 304.7.3

The permissible shear stress of concrete (τ) for the deck slabs in the strength range of 20 to 60 MPa shall be taken as

$$\tau_{co} = 0.0706\sqrt{f_{ck}} \quad \text{MPa}$$

4) Clause number 305.19

The minimum area of reinforcement in the deck slab should be taken as 0.2% and 0.3% of the gross sectional area for S-415 and S-140 grades of reinforcing steel respectively for concrete strengths up to 60 MPa.

6.2.3.2 Proposed revisions in IRC:18 - 1985 (1997)

The revisions proposed in this code are basically related with the post-tensioned longitudinal girders of the slab-on-girder bridges. The maximum grade of concrete used in the longitudinal girders in the present investigation is 90 MPa. Hence, the revisions proposed in this code are recommended for concrete strength from 30 MPa to 100 MPa.

1) Clause number 7.1

The permissible temporary stress at full transfer should be given by the following expressions:

I) Compressive stress

$$= 0.45 f_{cj} \quad \text{for} \quad f_{cj} \langle 60\, MPa$$

$$= 0.60 f_{cj} \quad \text{for} \quad f_{cj} \geq 60\,MPa \quad \text{up to 100 MPa}$$

II) Tensile stress

$= 0.447\sqrt{f_{cj}}$ MPa (where f_{cj} is the cube strength at j^{th} day)

2) Clause number 7.2

The following expressions should be used for permissible stress in concrete during service loads.

I) Compressive stress

$$= 0.33 f_{ck} \quad \text{for} \quad fck \langle 60\,MPa$$

$$= 0.45 f_{ck} \quad \text{for} \quad 100 \geq fck \geq 60\,MPa$$

II) Tensile stress

$$= 0.3714\sqrt{f_{ck}}\,MPa \quad \text{for} \quad f_{ck} \langle 60\,MPa$$

$$= 0.5571\sqrt{f_{ck}} - 1.44\,MPa \quad \text{for} \quad 100 \geq f_{ck} \geq 60\,MPa$$

3) Clause number 10.2

The secant modulus of elasticity of concrete in the strength range of 20 to 100 MPa should be taken as

$$Ec = 8910 \times {}^{0.325}\sqrt{fck} \quad \text{MPa}$$

4) Clause number 11.2

The specific creep strain of concrete of strength 60 MPa to 100 MPa should be taken equal to 20% of the values specified in the clause. The recommended values of specific creep strain are summarised below.

Maturity of concrete at the time of stressing as a percentage of f_{ck}	Creep strain per 10 MPa
40	1.88×10^{-4}
50	1.66×10^{-4}
60	1.44×10^{-4}
70	1.22×10^{-4}
75	1.12×10^{-4}
80	1.02×10^{-4}
90	0.88×10^{-4}
100	0.80×10^{-4}
110	0.72×10^{-4}

5) Clause number 13

The flexural strength of the composite T-beam section corresponding to compression failure should be given by the following expression:

$$M_{ult} = 0.176 b d^2 Ef_{ck} + 0.533(B_f - b)\left(d_b - t/2\right) \times t \times Df_{ck}$$

where $Efck = \left\{Dfck \times t + fck\left(xu - t\right)\right\} \div xu$

6) Clause number 14.1.2

The maximum permissible tensile stress (f_t), used in the expression for calculating the shear strength of the section uncracked in flexure should be taken as

$$f_t = 0.24\sqrt{fck}\,\ldots\ldots\ldots\ldots fck \langle 60\,MPa$$

$$f_t = 0.352\sqrt{fck}\,\ldots\ldots\ldots\ldots fck \geq 60\,MPa$$ up to 100 MPa

7) Clause number 14.1.3

The modulus of rupture strength (f_r) for calculating the cracking moment used in the expressions for shear strength of the section cracked in flexure should be taken as

$$f_r = 0.37\sqrt{fck}\,\ldots\ldots\ldots fck \langle 60\,MPa$$

$$f_r = 0.25 \times {}^{2/3}\sqrt[3]{fck}\,\ldots\ldots\ldots fck \geq 60\,Mpa$$ up to 100 MPa

8) Clause number 14.1.4

The minimum shear reinforcement in the longitudinal girder should be calculated using the following expression:

$$\left(A_{sv}\right)\min = \frac{0.575V_c \times S_v}{f_{yv} \times d_t} \geq \frac{0.689 b_w \times S_v}{0.87 f_{yv}}$$

9) Clause number 14.1.5

The maximum permissible shear stress in the longitudinal girders should be calculated using the following expression:

$$\tau_{\max} = 0.75\sqrt{f_{ck}}$$

10) Clause number 15.3

The minimum area of tension reinforcement in the longitudinal girders should be provided according to the following expression (as % of the gross sectional area)

I) For S415 grade steel

$$\left(A_{st}\right)\min = 0.2\% \ldots\ldots\ldots\ldots f_{ck} \leq 40\,MPa$$

$$\left(A_{st}\right)\min = 0.2 \times \sqrt{\frac{f_{ck}}{40}}\% \ldots\ldots\ldots\ldots f_{ck} \rangle 40\,MPa \text{ (up to 100 MPa)}$$

II) For S240 grade steel

$$\left(A_{st}\right)\min = 0.3\% \ldots\ldots\ldots\ldots f_{ck} \leq 40\,MPa$$

$$\left(A_{st}\right)\min = 0.3 \times \sqrt{\frac{f_{ck}}{40}}\%.........f_{ck}\rangle 40\,MPa \text{ (up to 100 MPa)}$$

11) The proposed amendments related to the properties of concrete and design considerations in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997) remove the deficiencies in these codes and permit the application of high-strength concrete in highway bridges. The results of the investigation based on the proposed amendments are comparable to the results of the similar investigations reported in the literature.

6.2.4 Economic studies

Based on the findings of the economic studies undertaken in the present investigation, the following conclusions may be drawn:

1) The optimization method and the computer programme developed and adopted in the present investigation can be effectively applied for cost effective preliminary design of the slab-on-girder prestressed concrete simply supported bridges, thus permitting the designers to participate in the creative process.

2) The tables given in Appendices G and H can be used effectively for preliminary design of the bridges, if there is a restriction on the maximum strength of the concrete that may be produced using local materials.

3) The alternate designs generated using the computer programme and the optimization method of the present investigation, are found to be more cost effective compared to the one actually used for the SAI Bridge (25m span and IRC class A loads) at Raebareli District, U. P. [UPSBC Ltd. Lucknow (1989)].

4) The variation in the value of SRATIO over the range of 1.0 to 3.0 (with increments of 0.25) resulted in an appreciable variation in the cost of the bridge superstructure. The value of the SRATIO corresponding to the minimum cost of the superstructure depends on the number of the longitudinal girders and the cross beams and the concrete strength of the deck slab and the longitudinal girders. In general the value of SRATIO corresponding to the minimum cost reduces as the number of longitudinal girders increases.

5) The maximum grade of concrete that can be economically used in shallow longitudinal girder sections reduces as number of longitudinal girders increases i.e. there is a maximum number of longitudinal girders that can be used effectively for a given longitudinal girder depth and the maximum strength of concrete in longitudinal girders.

6) If there is a limitation on the availability and applicability of the maximum prestress force, it is economical to use shallower sections of high-strength concrete.

7) For economical use of high-strength concrete in deeper longitudinal girder sections, the higher prestress force and hence special end block (which may resist excessively high bursting stresses) are necessary.

8) For the optimal design of the bridge superstructure of a given span and concrete strengths ranging from 35 to 90 MPa, Figs. 60 to 65 and 66 to 71 may be used for IRC class AA and class A loads respectively. The longitudinal girder depth in these designs may vary from 1500mm to 2030mm.

9) The optimal concrete strength and the corresponding maximum span for two, three and four longitudinal girders systems under IRC class AA and class A loads

may be obtained from Tables 28 to 30 and 31 to 33 respectively. The depth of the longitudinal girders varying from 1500mm to 2030mm.

10) The maximum recommended span of a longitudinal girder section can be exceeded economically by using a higher strength concrete. The span capability of a shallower section (depth = 1500mm) increases up to 60% by using the concrete strength up to 90 MPa. In case of a deeper section (depth = 2030 mm) under IRC class AA and class A loads, the span capability increase by about 40% if concrete strength increases up to 70 MPa and 80 MPa respectively. No advantage was obtained by using concrete strength exceeding 70 MPa and 80 MPa in the deeper longitudinal girder sections under IRC class AA and class A loads respectively because of the limitation of the maximum prestressing force.

11) The optimum longitudinal girder type and the number of longitudinal girders are affected marginally by the variation of the concrete strength in the deck slab. The higher concrete strength in the deck slab could only affect the properties of the composite longitudinal girder section due to change in the deck slab thickness.

12) A shallow longitudinal girder of high-strength concrete can be more economical than a deeper longitudinal girder section of lower strength concrete.

13) This study verifies that for a given span of the bridge it is more economical to use fewer number of longitudinal girders of high-strength concrete than larger number of longitudinal girders of normal-strength concrete. Thus by using high-strength concrete a larger longitudinal girder spacing may be achieved which makes the bridge substructure more economical. This economical advantage of

high-strength concrete application in highway bridges is more visible in longer span bridges.

14) For bridge spans up to 40 m, the two longitudinal girder system results in the lowest superstructure cost. The optimal concrete strength varies from 35 to 90 MPa.

15) If three longitudinal girders system is allowed, concrete strength up to 90 MPa will be optimal. The maximum span that can be achieved is 48m. Up to the span of about 40m, three longitudinal girders bridge is costlier than the two longitudinal girders bridge system. In case of three longitudinal girders system lower superstructure cost was achieved as compared to the same span with four longitudinal girders.

16) If four longitudinal girders are needed, the concrete from 35 to 70 MPa and 35 to 80 MPa is optimal for IRC class AA and class A loads respectively, depending up on the span length. Due to the limitation of the maximum prestress force, the deeper longitudinal girder sections and the bridge superstructure using larger number of longitudinal girder remain no longer feasible and economical.

6.3 Scope for Future Research and Application

There is considerable scope for future research in this area. The following points need immediate detailed analytical and experimental investigation for promoting high-strength concrete application in highway bridges.

1) There is no maximum allowable deflection criterion in IRC:18 - 1985 (1997). The application of high-strength concrete in highway bridges allows the use of more and more slender longitudinal girder sections. Thus, maximum deflection

criterion becomes more important. A detailed analytical and experimental investigation is needed to asses the deflection response of high-strength concrete slender longitudinal girders.

2) The clause no. 305.9.2 of IRC:21 - 1987 (1997) recommends that shear force in the longitudinal girder due to load within 5.5m from either supports also depends up on the reactions at the longitudinal girders. The deck slab is assumed simply supported or continuous over the longitudinal girders as unyielding support. The use of high-strength concrete requires shallower sections to bridge the same span. The shear behaviour of the longitudinal girders largely depends on the shear span - depth ratio. The deflection response of high-strength concrete members is different from that of normal strength concrete members. Thus, the distance '5.5 m' should be a function of the ratio of the longitudinal girder span to its depth and the strength of the concrete. This clause needs detailed investigation before extending its application for high-strength concrete. It has been shown that for odd number longitudinal girders system under IRC class AA load, the shear force induced in longitudinal girders is of much higher magnitude in comparison to when the number of longitudinal girders is an even number.

3) The cost of the two longitudinal girders system may be further reduced if SRATIO is allowed to take values higher than 3. It will result in the cantilever length of the deck slab as 2.25m and more. In the present investigation the maximum cantilever length of the deck slab is limited to 2.25 m (as suggested by the U. P. State Bridge Corporation Ltd. Lucknow). For bridges with two longitudinal girders system, experimental and theoretical verification is needed for a cantilever length of more than 2.25m.

4) In case of deeper longitudinal girder sections and the bridge system with four or more number longitudinal girders, there is no advantage of using concrete of strength higher than 70 MPa and 80 MPa for IRC class AA and class A loads respectively. This is due to the limitation of feasibility and applicability of maximum prestress force. The application of very high prestress force on the slender longitudinal girder sections may require specially designed end blocks.

5) The clause no. 9.3.1.4 of IRC:18 - 1985 (1997) requires that in slab-on-girders bridges, the number of longitudinal girders should not be less than three. The present investigation has shown that the two longitudinal girders system is most advantageous as far as application of high-strength concrete in highway bridges is concerned. This advantage reduces with the increase in the number and the depth of the longitudinal girders. This clause will further reduce the scope of applicability of this code to high-strength concrete highway bridges. Hence, this clause needs a detailed study in view of the increasing use of high-strength concrete in highway bridges.

6) At present very limited information on the creep of high-strength concrete is available. This area needs more exhaustive and long duration study so that the proper assessment of creep losses in prestressed concrete longitudinal girders may become feasible.

7) The behaviour of high-strength concrete in bridges under dynamic loads needs to be investigated analytically as well as experimentally. This is because the basic stress-strain relationship of high-strength concrete is very much different from the normal-strength concrete.

High-strength concrete has been successfully and advantageously used primarily in compression members. The present investigation has shown that high-strength concrete might be used advantageously in highway bridges. But the application of high-strength concrete in highway bridges is not possible unless very much desired amendments are incorporated in IRC:18 - 1985 (1997) and IRC:21 - 1987 (1997). The absence of the clear cut recommendation in these IRC codes regarding their applicability to high-strength bridges is an obstacle.

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APPENDIX - A

VARIATION OF K 1×K 2 (INCLUDING THE PROPOSED REVISIONS)

APPENDIX - A-1 SLAB UNDER IRC CLASS AA LOADS

SPAN (M)	IG	DS	NG	NCB	ICON1	PRODUCT	TAUMAX	TAUC	TAU
30	1	0.325	2	4	1	0.9475	2.0129	0.3664	0.4683
30	1	0.305	2	4	2	0.9615	2.1742	0.4016	0.4985
30	1	0.285	2	4	3	0.9755	2.3243	0.4356	0.5339
30	1	0.275	2	4	4	0.9825	2.4653	0.4653	0.5539
30	1	0.26	2	4	5	0.993	2.5986	0.4957	0.5876
30	1	0.29	2	5	1	0.972	2.0129	0.3759	0.5398
30	1	0.275	2	5	2	0.9825	2.1742	0.4104	0.5702
30	1	0.26	2	5	3	0.993	2.3243	0.4434	0.605
30	1	0.245	2	5	4	1.0035	2.4653	0.4753	0.6452
30	1	0.235	2	5	5	1.0105	2.5986	0.5045	0.6756
30	1	0.275	2	6	1	0.9825	2.0129	0.3799	0.579
30	1	0.26	2	6	2	0.993	2.1742	0.4148	0.6198
30	1	0.245	2	6	3	1.0035	2.3243	0.4481	0.6669
30	1	0.235	2	6	4	1.0105	2.4653	0.4786	0.7027
30	1	0.225	2	6	5	1.0175	2.5986	0.508	0.7425
30	1	0.27	2	7	1	0.986	2.0129	0.3813	0.5551
30	1	0.255	2	7	2	0.9965	2.1742	0.4162	0.5963
30	1	0.24	2	7	3	1.007	2.3243	0.4496	0.644
30	1	0.23	2	7	4	1.014	2.4653	0.4802	0.6803
30	1	0.22	2	7	5	1.021	2.5986	0.5097	0.7209
30	1	0.255	2	8	1	0.9965	2.0129	0.3853	0.564
30	1	0.24	2	8	2	1.007	2.1742	0.4206	0.6102
30	1	0.23	2	8	3	1.014	2.3243	0.4528	0.6453
30	1	0.22	2	8	4	1.021	2.4653	0.4835	0.6845
30	1	0.21	2	8	5	1.028	2.5986	0.5132	0.7288
30	1	0.245	3	4	1	1.0035	2.0129	0.388	0.5493
30	1	0.23	3	4	2	1.014	2.1742	0.4235	0.5962
30	1	0.22	3	4	3	1.021	2.3243	0.4559	0.6322
30	1	0.21	3	4	4	1.028	2.4653	0.4869	0.6729
30	1	0.2	3	4	5	1.035	2.5986	0.5167	0.7192
30	1	0.245	3	5	1	1.0035	2.0129	0.388	0.5493
30	1	0.23	3	5	2	1.014	2.1742	0.4235	0.5962
30	1	0.22	3	5	3	1.021	2.3243	0.4559	0.6322
30	1	0.21	3	5	4	1.028	2.4653	0.4869	0.6729
30	1	0.2	3	5	5	1.035	2.5986	0.5167	0.7192
30	1	0.235	3	6	1	1.0105	2.0129	0.3908	0.5797
30	1	0.225	3	6	2	1.0175	2.1742	0.425	0.6137
30	1	0.21	3	6	3	1.028	2.3243	0.459	0.6729
30	1	0.2	3	6	4	1.035	2.4653	0.4902	0.7192
30	1	0.2	3	6	5	1.035	2.5986	0.5167	0.7192
30	1	0.23	3	7	1	1.014	2.0129	0.3921	0.6105
30	1	0.215	3	7	2	1.0245	2.1742	0.4279	0.6676
30	1	0.205	3	7	3	1.0315	2.3243	0.4606	0.7121
30	1	0.2	3	7	4	1.035	2.4653	0.4902	0.7366
30	1	0.2	3	7	5	1.035	2.5986	0.5167	0.7366

SPAN (M)	IG	DS	NG	NCB	ICON1	PRODUCT	TAUMAX	TAUC	TAU
30	1	0.215	3	8	1	1.0245	2.0129	0.3962	0.6806
30	1	0.205	3	8	2	1.0315	2.1742	0.4308	0.7259
30	1	0.2	3	8	3	1.035	2.3243	0.4621	0.7509
30	1	0.2	3	8	4	1.035	2.4653	0.4902	0.7509
30	1	0.2	3	8	5	1.035	2.5986	0.5167	0.7509
30	1	0.205	4	4	1	1.0315	2.0129	0.3989	0.5452
30	1	0.2	4	4	2	1.035	2.1742	0.4323	0.5637
30	1	0.2	4	4	3	1.035	2.3243	0.4621	0.5637
30	1	0.2	4	4	4	1.035	2.4653	0.4902	0.5637
30	1	0.2	4	4	5	1.035	2.5986	0.5167	0.5637
30	1	0.205	4	5	1	1.0315	2.0129	0.3989	0.5452
30	1	0.2	4	5	2	1.035	2.1742	0.4323	0.5637
30	1	0.2	4	5	3	1.035	2.3243	0.4621	0.5637
30	1	0.2	4	5	4	1.035	2.4653	0.4902	0.5637
30	1	0.2	4	5	5	1.035	2.5986	0.5167	0.5637
30	1	0.2	4	6	1	1.035	2.0129	0.4002	0.5637
30	1	0.2	4	6	2	1.035	2.1742	0.4323	0.5637
30	1	0.2	4	6	3	1.035	2.3243	0.4621	0.5637
30	1	0.2	4	6	4	1.035	2.4653	0.4902	0.5637
30	1	0.2	4	6	5	1.035	2.5986	0.5167	0.5637
30	1	0.2	4	7	1	1.035	2.0129	0.4002	0.5637
30	1	0.2	4	7	2	1.035	2.1742	0.4323	0.5637
30	1	0.2	4	7	3	1.035	2.3243	0.4621	0.5637
30	1	0.2	4	7	4	1.035	2.4653	0.4902	0.5637
30	1	0.2	4	7	5	1.035	2.5986	0.5167	0.5637
30	1	0.2	4	8	1	1.035	2.0129	0.4002	0.5637
30	1	0.2	4	8	2	1.035	2.1742	0.4323	0.5637
30	1	0.2	4	8	3	1.035	2.3243	0.4621	0.5637
30	1	0.2	4	8	4	1.035	2.4653	0.4902	0.5637
30	1	0.2	4	8	5	1.035	2.5986	0.5167	0.5637
30	1	0.2	5	4	1	1.035	2.0129	0.4002	0.468
30	1	0.2	5	4	2	1.035	2.1742	0.4323	0.468
30	1	0.2	5	4	3	1.035	2.3243	0.4621	0.468
30	1	0.2	5	4	4	1.035	2.4653	0.4902	0.468
30	1	0.2	5	4	5	1.035	2.5986	0.5167	0.468
30	1	0.2	5	5	1	1.035	2.0129	0.4002	0.468
30	1	0.2	5	5	2	1.035	2.1742	0.4323	0.468
30	1	0.2	5	5	3	1.035	2.3243	0.4621	0.468
30	1	0.2	5	5	4	1.035	2.4653	0.4902	0.468
30	1	0.2	5	5	5	1.035	2.5986	0.5167	0.468
30	1	0.2	5	6	1	1.035	2.0129	0.4002	0.468
30	1	0.2	5	6	2	1.035	2.1742	0.4323	0.468
30	1	0.2	5	6	3	1.035	2.3243	0.4621	0.468
30	1	0.2	5	6	4	1.035	2.4653	0.4902	0.468
30	1	0.2	5	6	5	1.035	2.5986	0.5167	0.468
30	1	0.2	5	7	1	1.035	2.0129	0.4002	0.468
30	1	0.2	5	7	2	1.035	2.1742	0.4323	0.468
30	1	0.2	5	7	3	1.035	2.3243	0.4621	0.468

SPAN (M)	IG	DS	NG	NCB	ICON1	PRODUCT	TAUMAX	TAUC	TAU
30	1	0.2	5	7	4	1.035	2.4653	0.4902	0.468
30	1	0.2	5	7	5	1.035	2.5986	0.5167	0.468
30	1	0.2	5	8	1	1.035	2.0129	0.4002	0.468
30	1	0.2	5	8	2	1.035	2.1742	0.4323	0.468
30	1	0.2	5	8	3	1.035	2.3243	0.4621	0.468
30	1	0.2	5	8	4	1.035	2.4653	0.4902	0.468
30	1	0.2	5	8	5	1.035	2.5986	0.5167	0.468

APPENDIX - A-2 SLAB UNDER IRC CLASS A 2 LANE LOADS

SPAN (M)	IG	DS	NG	NCB	ICON1	PRODUCT	TAUMAX	TAUC	TAU
30	1	0.255	2	4	1	0.9965	2.0129	0.3853	0.5107
30	1	0.24	2	4	2	1.007	2.1742	0.4206	0.5487
30	1	0.225	2	4	3	1.0175	2.3243	0.4543	0.5934
30	1	0.215	2	4	4	1.0245	2.4653	0.4852	0.6277
30	1	0.205	2	4	5	1.0315	2.5986	0.5149	0.6665
30	1	0.23	2	5	1	1.014	2.0129	0.3921	0.6195
30	1	0.215	2	5	2	1.0245	2.1742	0.4279	0.6734
30	1	0.205	2	5	3	1.0315	2.3243	0.4606	0.7152
30	1	0.2	2	5	4	1.035	2.4653	0.4902	0.7382
30	1	0.2	2	5	5	1.035	2.5986	0.5167	0.7382
30	1	0.23	2	6	1	1.014	2.0129	0.3921	0.5505
30	1	0.215	2	6	2	1.0245	2.1742	0.4279	0.6013
30	1	0.205	2	6	3	1.0315	2.3243	0.4606	0.6408
30	1	0.2	2	6	4	1.035	2.4653	0.4902	0.6626
30	1	0.2	2	6	5	1.035	2.5986	0.5167	0.6626
30	1	0.23	2	7	1	1.014	2.0129	0.3921	0.5197
30	1	0.215	2	7	2	1.0245	2.1742	0.4279	0.5691
30	1	0.205	2	7	3	1.0315	2.3243	0.4606	0.6075
30	1	0.2	2	7	4	1.035	2.4653	0.4902	0.6286
30	1	0.2	2	7	5	1.035	2.5986	0.5167	0.6286
30	1	0.22	2	8	1	1.021	2.0129	0.3948	0.5249
30	1	0.205	2	8	2	1.0315	2.1742	0.4308	0.5794
30	1	0.2	2	8	3	1.035	2.3243	0.4621	0.6
30	1	0.2	2	8	4	1.035	2.4653	0.4902	0.6
30	1	0.2	2	8	5	1.035	2.5986	0.5167	0.6
30	1	0.2	3	4	1	1.035	2.0129	0.4002	0.5133
30	1	0.2	3	4	2	1.035	2.1742	0.4323	0.5133
30	1	0.2	3	4	3	1.035	2.3243	0.4621	0.5133
30	1	0.2	3	4	4	1.035	2.4653	0.4902	0.5133
30	1	0.2	3	4	5	1.035	2.5986	0.5167	0.5133
30	1	0.2	3	5	1	1.035	2.0129	0.4002	0.5133
30	1	0.2	3	5	2	1.035	2.1742	0.4323	0.5133
30	1	0.2	3	5	3	1.035	2.3243	0.4621	0.5133
30	1	0.2	3	5	4	1.035	2.4653	0.4902	0.5133
30	1	0.2	3	5	5	1.035	2.5986	0.5167	0.5133
30	1	0.2	3	6	1	1.035	2.0129	0.4002	0.5133
30	1	0.2	3	6	2	1.035	2.1742	0.4323	0.5133
30	1	0.2	3	6	3	1.035	2.3243	0.4621	0.5133
30	1	0.2	3	6	4	1.035	2.4653	0.4902	0.5133
30	1	0.2	3	6	5	1.035	2.5986	0.5167	0.5133
30	1	0.2	3	7	1	1.035	2.0129	0.4002	0.5219
30	1	0.2	3	7	2	1.035	2.1742	0.4323	0.5219
30	1	0.2	3	7	3	1.035	2.3243	0.4621	0.5219
30	1	0.2	3	7	4	1.035	2.4653	0.4902	0.5219
30	1	0.2	3	7	5	1.035	2.5986	0.5167	0.5219
30	1	0.2	3	8	1	1.035	2.0129	0.4002	0.5288
30	1	0.2	3	8	2	1.035	2.1742	0.4323	0.5288
30	1	0.2	3	8	3	1.035	2.3243	0.4621	0.5288
30	1	0.2	3	8	4	1.035	2.4653	0.4902	0.5288
30	1	0.2	3	8	5	1.035	2.5986	0.5167	0.5288

SPAN (M)	IG	DS	NG	NCB	ICON1	PRODUCT	TAUMAX	TAUC	TAU
30	1	0.2	4	4	1	1.035	2.0129	0.4002	0.4002
30	1	0.2	4	4	2	1.035	2.1742	0.4323	0.4002
30	1	0.2	4	4	3	1.035	2.3243	0.4621	0.4002
30	1	0.2	4	4	4	1.035	2.4653	0.4902	0.4002
30	1	0.2	4	4	5	1.035	2.5986	0.5167	0.4002
30	1	0.2	4	5	1	1.035	2.0129	0.4002	0.4002
30	1	0.2	4	5	2	1.035	2.1742	0.4323	0.4002
30	1	0.2	4	5	3	1.035	2.3243	0.4621	0.4002
30	1	0.2	4	5	4	1.035	2.4653	0.4902	0.4002
30	1	0.2	4	5	5	1.035	2.5986	0.5167	0.4002
30	1	0.2	4	6	1	1.035	2.0129	0.4002	0.4002
30	1	0.2	4	6	2	1.035	2.1742	0.4323	0.4002
30	1	0.2	4	6	3	1.035	2.3243	0.4621	0.4002
30	1	0.2	4	6	4	1.035	2.4653	0.4902	0.4002
30	1	0.2	4	6	5	1.035	2.5986	0.5167	0.4002
30	1	0.2	4	7	1	1.035	2.0129	0.4002	0.4002
30	1	0.2	4	7	2	1.035	2.1742	0.4323	0.4002
30	1	0.2	4	7	3	1.035	2.3243	0.4621	0.4002
30	1	0.2	4	7	4	1.035	2.4653	0.4902	0.4002
30	1	0.2	4	7	5	1.035	2.5986	0.5167	0.4002
30	1	0.2	4	8	1	1.035	2.0129	0.4002	0.4002
30	1	0.2	4	8	2	1.035	2.1742	0.4323	0.4002
30	1	0.2	4	8	3	1.035	2.3243	0.4621	0.4002
30	1	0.2	4	8	4	1.035	2.4653	0.4902	0.4002
30	1	0.2	4	8	5	1.035	2.5986	0.5167	0.4002
30	1	0.2	5	4	1	1.035	2.0129	0.4002	0.3274
30	1	0.2	5	4	2	1.035	2.1742	0.4323	0.3274
30	1	0.2	5	4	3	1.035	2.3243	0.4621	0.3274
30	1	0.2	5	4	4	1.035	2.4653	0.4902	0.3274
30	1	0.2	5	4	5	1.035	2.5986	0.5167	0.3274
30	1	0.2	5	5	1	1.035	2.0129	0.4002	0.3274
30	1	0.2	5	5	2	1.035	2.1742	0.4323	0.3274
30	1	0.2	5	5	3	1.035	2.3243	0.4621	0.3274
30	1	0.2	5	5	4	1.035	2.4653	0.4902	0.3274
30	1	0.2	5	5	5	1.035	2.5986	0.5167	0.3274
30	1	0.2	5	6	1	1.035	2.0129	0.4002	0.3274
30	1	0.2	5	6	2	1.035	2.1742	0.4323	0.3274
30	1	0.2	5	6	3	1.035	2.3243	0.4621	0.3274
30	1	0.2	5	6	4	1.035	2.4653	0.4902	0.3274
30	1	0.2	5	6	5	1.035	2.5986	0.5167	0.3274
30	1	0.2	5	7	1	1.035	2.0129	0.4002	0.3274
30	1	0.2	5	7	2	1.035	2.1742	0.4323	0.3274
30	1	0.2	5	7	3	1.035	2.3243	0.4621	0.3274
30	1	0.2	5	7	4	1.035	2.4653	0.4902	0.3274
30	1	0.2	5	7	5	1.035	2.5986	0.5167	0.3274
30	1	0.2	5	8	1	1.035	2.0129	0.4002	0.3274
30	1	0.2	5	8	2	1.035	2.1742	0.4323	0.3274
30	1	0.2	5	8	3	1.035	2.3243	0.4621	0.3274
30	1	0.2	5	8	4	1.035	2.4653	0.4902	0.3274
30	1	0.2	5	8	5	1.035	2.5986	0.5167	0.3274

APPENDIX - B

STRESSES DEVELOPED IN LONGITUDINAL GIRDERS AT VARIOUS STAGES OF LOADING

APPENDIX B - 1

MAXIMUM COMPRESSIVE STRESSES GOVERNING THE DESIGN (IRC CLASS AA LOADS)

CASE - 1

NG = 2, IG = 1, SPAN = 18M, NCB = 7, ICON = 4, ICON1 = 1, IPS = 4

stage1	section x=0	section X=L/8	section X=L/4	section X=3L/8	section X=L/2
snetb	3.4425	5.2521	7.3317	7.6259	7.7082
snett	1.7056	0.9393	1.4702	1.4626	1.4013
stage2					
snetb	3.249	3.9219	4.8896	4.7782	4.6619
snett	1.6064	1.9653	3.656	4.1544	4.3169
stage3					
snetb	12.8671	17.7561	23.9594	23.8425*	23.4081
snett	-0.4921	-2.7909*	-2.7871	-2.1511	-1.8887
stage4					
snetb	12.8671	15.164	18.8661	17.7915	16.9538
snett	-0.4921	-0.1139	2.8244	4.6683	5.3853
stage5					
snetb	12.6249	14.9023	18.5567	17.5116	16.6773
snett	0.341	0.7394	3.7189	5.56	6.2763
snettm	0.9426	0.9676	1.0193	1.0124	1.0109
stage6					
snetb	13.4381	16.3272	20.8329	19.9842	19.1181
snett	0.562	0.8769	3.8579	5.682	6.396
snettm	1.055	0.869	0.7665	0.7035	0.7051
stage7					
snetb	13.4381	15.9525	20.0461	19.0193	18.0888
snett	0.562	0.9733	4.0181	5.8818	6.6091
snettm	1.055	1.0519	1.1003	1.1168	1.146
stage8					
snetb	11.4756	13.2874	16.3514	15.2788	14.3328
snett	0.4102	0.8999	3.8957	5.7677	6.4982
snettm	1.2351	1.4536	1.6328	1.6675	1.7033
stage9					
snetb	11.4756	10.6467	10.8068	8.4786	7.0793
snett	0.4102	1.5796	5.0249	7.1756	7.9999
snettm	1.2351	2.7421	3.9856	4.5802	4.8102

Deflection = 19.3680mm Permissible deflection = 45.0000mm

Permissible compressive stress (during transfer of prestress) = 22.5 MPa

Permissible tensile stress (during transfer of prestress) = 3.161 MPa

CASE - 2

NG = 4, IG = 4, SPAN = 38M, NCB = 5, ICON = 4, ICON1 = 1, IPS = 7

stage1	section x=0	section X= L/8	section X = L/4	section X = 3L/8	section X =L/2
snetb	4.0253	8.5948	8.5086	7.3657	6.3008
snett	3.3085	5.0654	5.2919	5.6792	6.0164
stage2					
snetb	3.7656	6.3787	5.2736	3.4878	2.1897
snett	3.0927	6.166	7.3025	8.2365	8.7722
stage3					
snetb	17.3797	35.6544	35.5178	32.8123	30.2281
snett	1.4062	0.7085	0.2782	1.0456	1.8907
stage4					
snetb	17.3797	33.3708	31.8411	28.2163	25.3258
snett	1.4062	2.614	3.4054	4.9547	6.0604
stage5					
snetb	17.2192	33.5209	32.3435	28.9391	26.1253
snett	1.7606	2.8884	3.512	4.9581	6.0278
snettm	0.3961	0.2651	0.0414	-0.0972	-0.1455
stage6					
snetb	17.8987	36.6747*	36.4587	33.2261	30.2297
snett	2.6268	3.3225	3.4968	4.7593	5.8336
snettm	1.2827	0.402	-0.4253	-0.7863	-.8095*
stage7					
snetb	17.8987	35.773	34.9893	31.3894	28.2705
snett	2.6268	3.7345	4.1851	5.6196	6.7513
snettm	1.2827	0.9575	0.4988	0.3688	0.4226
stage8					
snetb	13.4869	26.032	24.5255	20.4543	17.2607
snett	2.1248	3.8457	4.7722	6.4275	7.5941
snettm	1.2079	2.1455	2.2937	2.4601	2.5608
stage9					
snetb	13.4869	23.3788	20.2017	15.0496	11.4957
snett	2.1248	5.058	6.7974	8.959	10.2944
snettm	1.2079	3.7802	5.0128	5.859	6.1862

Deflection = 86.5718mm Permissible deflection = 95.0000mm

Permissible compressive stress (during transfer of prestress) = 22.5 MPa

Permissible tensile stress (during transfer of prestress) = 3.161 MPa

CASE - 3

NG = 5, IG = 8, SPAN = 38M, NCB = 7, ICON = 4, ICON1 = 1, IPS = 7

stage1	section x=0	section X = L/8	section X = L/4	section X = 3L/8	section X = L/2
snetb	3.1605	8.1788	8.6894	7.9481	7.0173
snett	3.8354	5.9077	5.6302	5.721	6.0397
stage2					
snetb	2.9507	5.8126	5.2119	3.7192	2.5293
snett	3.5781	7.5149	8.4947	9.4021	10.0025
stage3					
snetb	14.9402	33.7577	34.2906	31.9952	29.5636
snett	2.1647	0.5078	-0.5201*	.1078	1.108
stage4					
snetb	14.9402	31.6134	30.8599	27.7067	24.9893
snett	2.1647	2.7965	3.209	4.7692	6.0802
stage5					
snetb	14.7603	31.6167	31.1987	28.2472	25.6004
snett	2.7271	3.4207	3.6479	5.0963	6.3682
snettm	0.6093	0.6393	0.4043	0.2616	0.2117
stage6					
snetb	15.2809	34.5564	35.1144*	32.3381	29.5161
snett	3.6732	3.9046	3.6042	4.8377	6.1162
snettm	1.5972	0.8812	-0.0295	-0.4255	-0.451
stage7					
snetb	15.2809	33.8784	34.0074	30.9544	28.0401
snett	3.6732	4.2728	4.2177	5.6046	6.9342
snettm	1.5972	1.3525	0.7535	0.5533	0.593
stage8					
snetb	11.4799	24.8866	24.251	20.7277	17.7394
snett	3.0902	4.4211	4.8988	6.5462	7.9168
snettm	1.3313	2.4013	2.4629	2.5952	2.6874
stage9					
snetb	11.4799	22.5245	20.3947	15.9074	12.5977
snett	3.0902	5.7035	7.0359	9.2176	10.7663
snettm	1.3313	4.0427	5.1905	6.0047	6.3242

Deflection = 87.2585mm Permissible deflection = 95.0000mm

Permissible compressive stress (during transfer of prestress) = 22.5 MPa

Permissible tensile stress (during transfer of prestress) = 3.161 MPa

APPENDIX B - 2
MAXIMUM TENSILE STRESSES GOVERNING THE DESIGN (IRC CLASS AA LOADS)

CASE - 1

NG = 2, IG = 4, SPAN = 18M, NCB = 7, ICON = 4, ICON1 = 1, IPS = 1

stage1	section x=0	section X = L/8	section X = L/4	section X = 3L/8	section X = L/2
snetb	0.8566	0.881	1.1276	1.0943	1.0371
snett	0.5215	0.7828	1.2868	1.4554	1.5049
stage2					
snetb	0.8226	0.1936	-0.2532	-0.5677	-0.754
snett	0.499	1.3128	2.3612	2.7818	2.9409
stage3					
snetb	3.3676	4.0564	5.4661	5.2605	4.9956
snett	0.1039	0.2563	0.9032	1.3521	1.5292
stage4					
snetb	3.3676	2.4865	2.2568	1.3962	0.8737
snett	0.1039	1.6045	3.5963	4.6389	5.0351
stage5					
snetb	3.3302	2.4499	2.2013	1.3654	0.8458
snett	0.8158	2.3426	4.4007	5.443	5.8385
snettm	0.7696	0.7967	0.8685	0.8655	0.8643
stage6					
snetb	3.5063	2.8483	2.9167	2.173	1.6373
snett	0.8946	2.3772	4.4211	5.4543	5.8492
snettm	0.8338	0.7766	0.7844	0.7572	0.7577
stage7					
snetb	3.5063	2.5756	2.3216	1.4359	0.8511
snett	0.8946	2.4616	4.5659	5.6332	6.0401
snettm	0.8338	0.9146	1.0404	1.0737	1.0954
stage8					
snetb	3.3029	2.2749	1.8743	0.973	0.3852
snett	0.8781	2.4616	4.5652	5.6337	6.0413
snettm	0.8454	0.9599	1.1068	1.1439	1.1668
stage9					
snetb	3.3029	0.4832	-2.0353	-3.8699	-4.7804*
snett	0.8781	3.0161	5.5166	6.8093*	7.2952
snettm	0.8454	1.8669	2.7887	3.2239	3.3854

Deflection = 1.4987mm Permissible Deflection = 45.0000mm

Permissible compressive stress (under service load) = 16.5 MPa

Permissible tensile stress (under service loads) = 2.626 MPa

CASE - 2

NG = 2, IG = 4, SPAN = 28M, NCB = 7, ICON = 4, ICON1 = 1, IPS = 4

stage1	section x=0	section X = L/8	section X = L/4	section X = 3L/8	section X = L/2
snetb	2.1519	3.3136	4.0325	3.6791	3.2021
snett	1.5969	2.1889	2.9633	3.18	3.3533
stage2					
snetb	2.054	1.7984	1.3919	0.4092	-0.3054
snett	1.5222	3.2601	4.9412	5.6931	6.0685
stage3					
snetb	8.944	13.9718	16.9895*	15.6291	14.4224
snett	0.6033	0.4475	1.3326	1.9599	2.4531
stage4					
snetb	8.944	8.9196	7.8768	4.2383	2.2721
snett	0.6033	4.7515	9.0834	11.6484	12.7875
stage5					
snetb	8.7686	8.7147	7.6868	4.0657	2.1059
snett	1.4111	5.6064	9.9604	12.5227	13.6608
snettm	0.9029	0.956	0.9769	0.9705	0.9681
stage6					
snetb	9.2632	9.9948	9.6662	6.1469	4.1198
snett	1.5989	5.7412	10.0764	12.61	13.7446
snettm	1.0346	0.8811	0.7517	0.6929	0.6987
stage7					
snetb	9.2632	9.3055	8.3292	4.4755	2.3371
snett	1.5989	5.8702	10.2851	12.871	14.023
snettm	1.0346	1.1599	1.2435	1.3076	1.3543
stage8					
snetb	8.4593	7.9246	6.5474	2.6258	0.4763
snett	1.5191	5.8093	10.2175	12.814	13.9677
snettm	1.0874	1.3407	1.4897	1.5787	1.6295
stage9					
snetb	8.4593	4.9658	0.8082	-4.5482	-7.1759*
snett	1.5191	6.363	11.1136	13.9341	15.1625
snettm	1.0874	2.5374	3.6004	4.2172	4.4439

Deflection = 11.2645mm   Permissible deflection = 70.0000mm

Permissible compressive stress (under service load) = 16.5MPa

Permissible tensile stress (under service loads) = 2.626MPa

CASE - 3

NG = 4, IG = 4, SPAN = 38M, NCB = 7, ICON = 4, ICON1 = 1, IPS = 4

stage1	section X = 0	section X = L/8	section X = L/4	section X = 3L/8	section X = L/2
snetb	2.1016	2.0895	0.6285	-0.8821	-1.7373
snett	1.6873	4.5725	5.9124	6.8965	7.3315
stage2					
snetb	2.0245	0.3707	-2.0337	-4.1728	-5.2441*
snett	1.6233	5.7847	7.957	9.4756	10.0944
stage3					
snetb	8.9207	15.1747*	13.2336	10.6291	8.9086
snett	0.7563	3.0179	4.4093	5.8452	6.6203
stage4					
snetb	8.9207	12.8912	9.5569	6.0332	4.0063
snett	0.7563	4.9234	7.5366	9.7543	10.79
stage5					
snetb	8.9555	13.4972	10.566	7.2971	5.3561
snett	1.1238	5.1688	7.5842	9.6826	10.678
snettm	0.3893	0.1831	-0.0795	-0.2397	-0.2937
stage6					
snetb	9.295	15.0727	12.6218	9.4387	7.4064
snett	1.5565	5.3857	7.5766	9.5832	10.581
snettm	0.8322	0.2515	-0.3126	-0.584	-0.6254
stage7					
snetb	9.295	14.1711	11.1526	7.6022	5.4475
snett	1.5565	5.7976	8.2647	10.4434	11.4985
snettm	0.8322	0.807	0.6113	0.571	0.6065
stage8					
snetb	8.3276	12.035	8.8583	5.2041	3.0327
snett	1.4458	5.8212	8.3926	10.6199	11.6828
snettm	0.8152	1.0667	1.0039	1.0288	1.0748
stage9					
snetb	8.3276	9.3643	4.5061	-0.2361	-2.7702
snett	1.4458	7.0415	10.4311	13.1681	14.4009
snettm	0.8152	2.7121	3.7409	4.45	4.7241

Deflection = 18.6043mm Permissible deflection = 95.0000mm

Permissible compressive stress (during transfer of prestress) = 22.5 MPa

Permissible tensile stress (during transfer of prestress) = 3.161 MPa

APPENDIX - C

MOMENT OF RESISTANCE OF COMPOSITE T-BEAM SECTIONS CORRESPONDING TO FCK, DFCK AND EFCK

CASE 1

IG =1, SPAN = 38M, NCB = 7, DFCK = 35 MPa, IRC CLASS AA LOADS, SRATIO = 1, SECTION X = L/2

	NG = 2			NG = 5		
FCK (MPa)	35	50	90	35	50	90
Bult (KNm)	33615.54	33615.54	33615.54	13711.94	13711.94	13711.94
Bmr2dfck(KNm)	57497.79	57497.79	57497.79	11485.5	11485.5	11485.5
Bmr2fck(KNm)	67080.75	95829.64	172493.4	13399.75	19142.51	34456.52
Bmr2efck(KNm)	57904.18	59123.35	62374.47	11878.93	13059.19	16206.57

CASE 2

IG =4, SPAN = 38M, NCB = 7, DFCK = 35 MPa, IRC CLASS AA LOADS, SRATIO = 1, SECTION X = L/2

	NG = 2			NG = 5		
FCK (MPa)	35	50	90	35	50	90
Bult (KNm)	36179	36179	36179	14803.26	14803.26	14803.26
Bmr2dfck(KNm)	75707.63	75707.63	75707.63	13788.52	13788.52	13788.52
Bmr2fck(KNm)	88325.56	126179.4	227122.9	16086.61	22980.87	41365.56
Bmr2efck(KNm)	76337.06	78225.38	83260.88	14394.39	16212.01	21059

CASE 3

IG =8, SPAN = 38M, NCB = 7, DFCK = 35 MPa, IRC CLASS AA LOADS, SRATIO = 1, SECTION X = L/2

	NG = 2			NG = 5		
FCK (MPa)	35	50	90	35	50	90
Bult (KNm)	35467.39	35467.39	35467.39	14847.98	14847.98	14847.98
Bmr2dfck(KNm)	79300.01	79300.01	79300.01	17040.3	17040.3	17040.3
Bmr2fck(KNm)	92516.68	132166.7	237900	19880.35	28400.5	51120.91
Bmr2efck(KNm)	80086.55	82446.18	88738.52	17801.03	20083.2	26168.98

APPENDIX - D

SHEAR RESISTANCE OFFERED BY THE GIRDER SECTIONS (WITHOUT SHEAR REINFORCEMENT)

1) SECTIONS FAILING DUE TO EXCESSIVE SHEAR

CASE - 1

IG = 1, IRC CLASS AA LOADS, SPAN 38M, NG = 2, NCB = 7, ICON1 = 5, ICON = 8, IPS = 7

Section	x=0	x= L/8	x= L/4	x=3L/8	x= L/2
Shearmax (MPa)	4.3394	7.825	6.1178*	3.989	2.0823
Shearmb (MPa)	1.4413	2.2026	1.3216	1.2903	1.2857
Bult (KNm)	-2.2087	13357.19	22867.03	28581.58	30500.85
Bmr1 (KNm)	25847.58	29966.48	32048.03	32826.03	32942.39
Bmr2 (KNm)	49860.63	53397.27	57403.26	59079.43	59330.7

Maximum permissible shear stress = 5.5 MPa

CASE - 2

IG = 4, IRC CLASS AA LOADS, SPAN 38M, NG = 2, NCB = 7, ICON1 = 5, ICON = 8, IPS = 7

Section	x=0	x= L/8	x= L/4	x=3L/8	x= L/2
Shearmax (MPa)	3.8842	7.0591*	5.4464	3.4709	1.7391
Shearmb (MPa)	1.5208	2.516	1.3117	1.271	1.2648
Bult (KNm)	-3.1387	14353.19	24558.3	30691.59	32753.09
Bmr1 (KNm)	28816.87	35072.54	38183.36	39405.96	39598.61
Bmr2 (KNm)	59578.37	66274.29	72599.55	75383.22	75823.33

Maximum permissible shear stress = 5.5 MPa

CASE - 3

IG = 8, IRC CLASS AA LOADS, SPAN 38M, NG = 2, NCB = 7, ICON1 = 5, ICON = 8, IPS = 7

Section	x=0	x= L/8	x= L/4	x=3L/8	x= L/2
Shearmax (MPa)	3.5711	6.3812*	4.9214	3.1474	1.6207
Shearmb (MPa)	1.546	2.5873	1.3051	1.262	1.2554
Bult (KNm)	-3.2342	14168.46	24233.46	30285.07	32323.3
Bmr1 (KNm)	30961.29	38379.11	41914.63	43346.89	43572.47
Bmr2 (KNm)	65456.66	73864.98	81145.16	84464.98	84989.85

Maximum permissible shear stress = 5.5 MPa 266

2) MINIMUM SHEAR REINFORCEMENT MAY NOT PROVIDE SUFFICIENT RESERVE STRENGTH

$$v - v_c \leq 0.4\,MPa \quad \text{or} \quad v \leq 0.5 \times v_c$$

and

$$v_c \div 2 \rangle 0.4\,MPa$$

CASE - 1

IG = 1, IRC CLASS AA LOADS, SPAN 38M, NG = 5, NCB = 7, ICON1 = 5, ICON = 4, IPS = 4

Section	x=0	x= L/8	x= L/4	x=3L/8	x= L/2
Shearmax (MPa)	2.1956	3.6194	2.7899	1.8316	0.9736*
Shearmb (MPa)	1.0876	0.9991	0.9385	0.9143	0.9107*
Bult (KNm)	-2.2087	5914.466	10104.17	12621.19	13465.52
Bmr1 (KNm)	11572.22	13629.83	14669.67	15058.32	15116.45
Bmr2 (KNm)	14519.12	15303.39	16628.92	17218.04	17306.72

Maximum permissible shear stress = 5.303 MPa

CASE - 2

IG = 4, IRC CLASS AA LOADS, SPAN 38M, NG = 5, NCB = 7, ICON1 = 5, ICON = 4, IPS = 4

Section	x=0	x= L/8	x= L/4	x=3L/8	x= L/2
Shearmax (MPa)	2.013	3.2875	2.4894	1.5902	0.7876*
Shearmb (MPa)	1.1513	1.1729	0.9296	0.8986	0.8938*
Bult (KNm)	-3.1387	6493.44	11078.65	13831.9	14753.21
Bmr1 (KNm)	13005.91	16130.94	17684.95	18295.7	18391.95
Bmr2 (KNm)	17814.88	19255.23	21369.3	22376.95	22537.19

Maximum permissible shear stress = 5.303 MPa

CASE - 3

IG = 8, IRC CLASS AA LOADS, SPAN 38M, NG = 5, NCB = 7, ICON1 = 1, ICON = 4, IPS = 4

Section	x=0	x= L/8	x= L/4	x=3L/8	x= L/2
Shearmax (MPa)	1.8317	2.9546	2.2385	1.4329	0.7345*
Shearmb (MPa)	1.1685	1.001	0.9238	0.8912	0.8863*
Bult (KNm)	-3.2342	6429.475	10960.71	13683.76	14598.67
Bmr1 (KNm)	14077.16	17782.74	19548.92	20264.4	20377.09
Bmr2 (KNm)	12134.76	13206.1	14696.78	15440.32	15558.63

Maximum permissible shear stress = 5.303 MPa

APPENDIX - E

VARIATION OF MORRICE-LITTLE FLEXURAL PARAMETER (θ)

NG	SPAN=25M		SPAN=35M		SPAN=45M		SPAN=55M		SPAN=65M		SPAN=75M		SPAN=85M	
	NCB=4	NCB=12	NCB=4	NCB=12	NCB=5	NCB=12	NCB=6	NCB=12	NCB=7	NCB=12	NCB=8	NCB=12	NCB=4	NCB=12
2	0.3024	0.2367	0.2317	0.1808	0.1786	0.1476	0.1452	0.1255	0.1225	0.1095	0.1060	0.0973	0.0937	0.0874
3	0.3244	0.2495	0.2487	0.1892	0.1920	0.1552	0.1563	0.1324	0.1319	0.1160	0.1140	0.1036	0.1004	0.0939
4	0.3508	0.2683	0.2692	0.2050	0.2078	0.1678	0.1692	0.1430	0.1427	0.1254	0.1234	0.1120	0.1087	0.1015
5	0.3692	0.2834	0.2837	0.2160	0.2189	0.1766	0.1782	0.1506	0.1503	0.1319	0.1299	0.1179	0.1144	0.1068

APPENDIX - F

QUIVALENT SIMPLIFIED LOADS (UNIFORMLY DISTRIBUTED LOAD) FOR IRC CLASS AA AND CLASS A LOADS

(A) IRC CLASS AA LOADS

NCB	NG	SPAN	DESF1	DESF2	DESF3	EUDLGS1	EPONTL1	EUDLGS2	EPONTL2	EUDLGS3	EPONTL3	DEBM1	DEBM2	DEBM3	EUDLGB1	EUDLGB2	EUDLGB3
3	2	20	541.453	0	0	29.75	487.903	0	0	0	0	2707.267	0	0	54.145	0	0
4	2	20	538.271	0	0	29.575	485.036	0	0	0	0	2691.357	0	0	53.827	0	0
5	2	20	544.065	0	0	29.894	490.256	0	0	0	0	2720.324	0	0	54.406	0	0
6	2	20	547.361	0	0	30.075	493.226	0	0	0	0	2736.804	0	0	54.736	0	0
7	2	20	549.097	0	0	30.17	494.791	0	0	0	0	2745.485	0	0	54.91	0	0
8	2	20	551.042	0	0	30.277	496.543	0	0	0	0	2755.21	0	0	55.104	0	0
9	2	20	551.42	0	0	30.298	496.884	0	0	0	0	2757.102	0	0	55.142	0	0
10	2	20	551.9	0	0	30.324	497.316	0	0	0	0	2759.499	0	0	55.19	0	0
11	2	20	552.34	0	0	30.348	497.713	0	0	0	0	2761.698	0	0	55.234	0	0
12	2	20	552.366	0	0	30.35	497.736	0	0	0	0	2761.828	0	0	55.237	0	0

NCB	NG	SPAN	DESF1	DESF2	DESF3	EUDLGS1	EPONTL1	EUDLGS2	EPONTL2	EUDLGS3	EPONTL3	DEBM1	DEBM2	DEBM3	EUDLGB1	EUDLGB2	EUDLGB3
3	3	20	344.364	1038.345	0	18.921	310.306	90.558	265.53	0	0	1721.82	1473.36	0	34.436	29.467	0
4	3	20	347.214	1038.345	0	19.078	312.874	90.801	260.662	0	0	1736.069	1446.35	0	34.721	28.927	0
5	3	20	349.709	1038.345	0	19.215	315.122	91.03	256.085	0	0	1748.544	1420.96	0	34.971	28.419	0
6	3	20	351.861	1038.345	0	19.333	317.061	91.214	252.406	0	0	1759.304	1400.54	0	35.186	28.011	0
7	3	20	353.546	1038.345	0	19.426	318.58	91.308	250.527	0	0	1767.732	1390.11	0	35.355	27.802	0
8	3	20	355.886	1038.345	0	19.554	320.688	91.374	249.211	0	0	1779.428	1382.81	0	35.589	27.656	0
9	3	20	357.424	1038.345	0	19.639	322.074	91.436	247.971	0	0	1787.118	1375.93	0	35.742	27.519	0
10	3	20	358.88	1038.345	0	19.719	323.387	91.486	246.98	0	0	1794.402	1370.43	0	35.888	27.409	0
11	3	20	359.484	1038.345	0	19.752	323.931	91.537	245.949	0	0	1797.421	1364.71	0	35.948	27.294	0
12	3	20	359.9	1038.345	0	19.775	324.306	91.574	245.218	0	0	1799.502	1360.66	0	35.99	27.213	0

NCB	NG	SPAN	DESF1	DESF2	DESF3	EUDLGS1	EPONTL1	EUDLGS2	EPONTL2	EUDLGS3	EPONTL3	DEBM1	DEBM2	DEBM3	EUDLGB1	EUDLGB2	EUDLGB3
3	4	20	252.58	537.653	0	13.878	227.599	42.966	215.983	0	0	1262.9	1198.44	0	25.258	23.969	0
4	4	20	257.464	537.653	0	14.146	232.001	43.166	211.987	0	0	1287.322	1176.26	0	25.746	23.525	0
5	4	20	260.056	537.653	0	14.289	234.336	43.256	210.185	0	0	1300.279	1166.27	0	26.006	23.325	0
6	4	20	261.059	537.653	0	14.344	235.24	43.279	209.726	0	0	1305.293	1163.72	0	26.106	23.274	0
7	4	20	262.709	537.653	0	14.435	236.727	43.364	208.019	0	0	1313.546	1154.25	0	26.271	23.085	0
8	4	20	264.109	537.653	0	14.511	237.988	43.43	206.7	0	0	1320.546	1146.93	0	26.411	22.939	0
9	4	20	264.885	537.653	0	14.554	238.687	43.467	205.971	0	0	1324.424	1142.88	0	26.488	22.858	0
10	4	20	265.526	537.653	0	14.589	239.265	43.498	205.347	0	0	1327.629	1139.42	0	26.553	22.789	0
11	4	20	266.527	537.653	0	14.644	240.167	43.51	205.097	0	0	1332.636	1138.04	0	26.653	22.761	0
12	4	20	267.104	537.653	0	14.676	240.688	43.53	204.699	0	0	1335.522	1135.82	0	26.71	22.717	0

NCB	NG	SPAN	DESF1	DESF2	DESF3	EUDLGS1	EPONTL1	EUDLGS2	EPONTL2	EUDLG3	EPONTL3	DEBM1	DEBM2	DEBM3	EUDLG1	EUDLGB2	EUDLGB3
3	5	20	217.458	616.696	444.937	11.948	195.952	51.903	195.335	35.716	175.555	1087.292	1083.87	974.114	21.746	21.677	19.482
4	5	20	223.813	616.696	444.937	12.297	201.677	51.99	193.598	35.884	172.199	1119.063	1074.23	955.495	22.381	21.485	19.11
5	5	20	227.66	616.696	444.937	12.509	205.144	52.045	192.489	35.968	170.513	1138.299	1068.08	946.142	22.766	21.362	18.923
6	5	20	230.01	616.696	444.937	12.638	207.262	52.075	191.886	36.016	169.561	1150.05	1064.73	940.859	23.001	21.295	18.817
7	5	20	231.018	616.696	444.937	12.693	208.17	52.077	191.846	36.04	169.075	1155.089	1064.51	938.16	23.102	21.29	18.763
8	5	20	232.354	616.696	444.937	12.767	209.374	52.087	191.649	36.081	168.255	1161.768	1063.41	933.609	23.235	21.268	18.672
9	5	20	233.908	616.696	444.937	12.852	210.774	52.102	191.347	36.133	167.209	1169.539	1061.74	927.808	23.391	21.235	18.556
10	5	20	235.235	616.696	444.937	12.925	211.97	52.116	191.074	36.179	166.303	1176.176	1060.22	922.78	23.524	21.205	18.456
11	5	20	236.099	616.696	444.937	12.972	212.749	52.126	190.88	36.205	165.779	1180.495	1059.14	919.871	23.61	21.183	18.397
12	5	20	236.761	616.696	444.937	13.009	213.345	52.134	190.704	36.224	165.393	1183.804	1058.17	917.727	23.676	21.163	18.355

(B) IRC CLASS A LOADS

NCB	NG	SPAN	DESF1	DESF2	DESF3	EUDLGS1	EPONTL1	EUDLGS2	EPONTL2	EUDLGS3	EPONTL3	DEBM1	DEBM2	DEBM3	EUDLGB1	EUDLGB2	EUDLGB3
5	2	40	655.733	0	0	21.191	479.33	0	0	0	0	6336.01	0	0	33.547	0	0
6	2	40	655.751	0	0	21.191	479.343	0	0	0	0	6336.185	0	0	33.548	0	0
7	2	40	658.513	0	0	21.281	481.362	0	0	0	0	6362.874	0	0	33.69	0	0
8	2	40	660.374	0	0	21.341	482.723	0	0	0	0	6380.86	0	0	33.785	0	0
9	2	40	662.001	0	0	21.393	483.912	0	0	0	0	6396.58	0	0	33.868	0	0
10	2	40	663.431	0	0	21.439	484.957	0	0	0	0	6410.392	0	0	33.941	0	0
11	2	40	664.254	0	0	21.466	485.559	0	0	0	0	6418.345	0	0	33.983	0	0
12	2	40	664.985	0	0	21.49	486.093	0	0	0	0	6425.408	0	0	34.021	0	0

NCB	NG	SPAN	DESF1	DESF2	DESF3	EUDLGS1	EPONTL1	EUDLGS2	EPONTL2	EUDLGS3	EPONTL3	DEBM1	DEBM2	DEBM3	EUDLG1	EUDLGB2	EUDLGB3
5	3	40	438.868	473.359	0	14.182	320.805	17.012	266.224	0	0	4240.558	3383.47	0	22.452	17.915	0
6	3	40	440.221	472.805	0	14.226	321.794	17.005	265.423	0	0	4253.632	3372.22	0	22.522	17.855	0
7	3	40	441.271	472.369	0	14.26	322.562	16.999	264.794	0	0	4263.773	3363.37	0	22.575	17.808	0
8	3	40	442.114	472.014	0	14.287	323.178	16.994	264.281	0	0	4271.926	3356.17	0	22.619	17.77	0
9	3	40	442.584	471.715	0	14.303	323.522	16.99	263.848	0	0	4276.463	3350.08	0	22.643	17.738	0
10	3	40	443.177	471.461	0	14.322	323.955	16.986	263.482	0	0	4282.192	3344.94	0	22.673	17.71	0
11	3	40	443.684	471.241	0	14.338	324.326	16.983	263.165	0	0	4287.091	3340.47	0	22.699	17.687	0
12	3	40	443.909	471.046	0	14.345	324.49	16.98	262.882	0	0	4289.267	3336.50	0	22.71	17.666	0

NCB	NG	SPAN	DESF1	DESF2	DESF3	EUDLGS1	EPONTL1	EUDLGS2	EPONTL2	EUDLGS3	EPONTL3	DEBM1	DEBM2	DEBM3	EUDLGB1	EUDLGB2	EUDLGB3
5	4	40	330.52	315.66	0	10.681	241.605	10.506	211.079	0	0	3193.65	2756.72	0	16.909	14.596	0
6	4	40	329.976	315.559	0	10.664	241.207	10.505	210.934	0	0	3188.39	2754.68	0	16.882	14.585	0
7	4	40	330.832	315.436	0	10.691	241.833	10.503	210.755	0	0	3196.658	2752.18	0	16.925	14.572	0
8	4	40	331.517	315.335	0	10.713	242.333	10.502	210.61	0	0	3203.277	2750.13	0	16.96	14.561	0
9	4	40	332.08	315.25	0	10.732	242.745	10.5	210.487	0	0	3208.719	2748.40	0	16.989	14.552	0
10	4	40	332.553	315.177	0	10.747	243.091	10.499	210.382	0	0	3213.286	2746.92	0	17.013	14.544	0
11	4	40	332.956	315.113	0	10.76	243.385	10.498	210.289	0	0	3217.182	2745.62	0	17.034	14.537	0
12	4	40	333.304	315.056	0	10.771	243.64	10.498	210.207	0	0	3220.546	2744.46	0	17.052	14.531	0

NCB	NG	SPAN	DESF1	DESF2	DESF3	EUDLGS1	EPONTL1	EUDLGS2	EPONTL2	EUDLGS3	EPONTL3	DEBM1	DEBM2	DEBM3	EUDLGB1	EUDLGB2	EUDLGB3
5	5	40	271.537	248.642	249.039	8.775	198.489	8.007	178.684	8.509	157.705	2623.727	2356.70	2043.263	13.892	12.478	10.818
6	5	40	265.476	245.7	248.731	8.579	194.059	7.924	174.435	8.505	157.26	2565.159	2296.98	2037.011	13.582	12.162	10.785
7	5	40	263.93	244.727	248.815	8.529	192.929	7.911	173.03	8.506	157.382	2550.224	2277.22	2038.719	13.503	12.057	10.794
8	5	40	264.589	244.731	248.621	8.55	193.41	7.911	173.036	8.504	157.102	2556.583	2277.31	2034.788	13.536	12.058	10.774
9	5	40	265.132	244.734	248.46	8.568	193.807	7.911	173.04	8.501	156.869	2561.833	2277.36	2031.505	13.564	12.058	10.756
10	5	40	265.59	244.735	248.322	8.583	194.142	7.911	173.042	8.499	156.67	2566.257	2277.39	2028.707	13.588	12.058	10.741
11	5	40	265.982	244.735	248.202	8.595	194.428	7.911	173.042	8.498	156.497	2570.044	2277.4	2026.282	13.608	12.058	10.729
12	5	40	266.322	244.735	248.097	8.606	194.677	7.911	173.041	8.496	156.346	2573.33	2277.39	2024.153	13.625	12.058	10.717

APPENDIX - G

STRUCTURALLY FEASIBLE DESIGNS AS PER EXISTING IRC: 18 - 1985 (1997) AND IRC: 21-1987 (1997)

APPENDIX - G1 (UNDER IRC CLASS AA LOADS)

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	STATIO
3	4	3	1	1	4	4	26	0.2	0.2	69.3937	1060.958	1009.714	0	6166.017	4867.455	0	2.75
3	4	3	2	1	4	4	26	0.2	0.2	69.555	1060.958	1009.714	0	6166.017	4867.455	0	2.75
3	4	3	3	1	4	4	26	0.2	0.2	69.7819	1060.958	1009.714	0	6166.017	4867.455	0	2.75
3	4	3	4	1	4	4	26	0.2	0.2	69.9105	1060.958	1009.714	0	6166.017	4867.455	0	2.75
3	4	3	5	1	4	4	26	0.2	0.2	71.1825	1060.958	1009.714	0	6166.017	4867.455	0	2.75
2	9	4	1	2	4	3	26	0.29	0.29	67.7586	1410.299	0	0	9007.717	0	0	2
2	4	4	2	2	4	3	26	0.25	0.25	63.5279	1430.756	0	0	8927.336	0	0	3
2	4	4	3	2	4	3	26	0.25	0.25	63.7781	1430.756	0	0	8927.336	0	0	3
2	4	4	4	2	4	3	26	0.25	0.25	63.9197	1430.756	0	0	8927.336	0	0	3
2	4	4	5	2	4	3	26	0.25	0.25	65.3451	1430.756	0	0	8927.336	0	0	3
2	9	4	1	3	4	3	26	0.29	0.29	67.8353	1410.299	0	0	9007.717	0	0	2
2	4	4	2	3	4	3	26	0.25	0.25	63.5838	1430.756	0	0	8927.336	0	0	3
2	4	4	3	3	4	3	26	0.25	0.25	63.8354	1430.756	0	0	8927.336	0	0	3
2	4	4	4	3	4	3	26	0.25	0.25	63.9769	1430.756	0	0	8927.336	0	0	3
2	4	4	5	3	4	3	26	0.25	0.25	65.4021	1430.756	0	0	8927.336	0	0	3
2	9	4	1	4	4	3	26	0.29	0.29	68.8403	1410.299	0	0	9007.717	0	0	2
2	4	4	2	4	4	3	26	0.275	0.275	65.2027	1437.538	0	0	8988.502	0	0	2.75
2	4	4	3	4	4	3	26	0.25	0.25	64.6509	1430.756	0	0	8927.336	0	0	3
2	4	4	4	4	4	3	26	0.25	0.25	64.7926	1430.756	0	0	8927.336	0	0	3
2	4	4	5	4	4	3	26	0.25	0.25	66.2178	1430.756	0	0	8927.336	0	0	3
3	4	3	1	5	4	4	26	0.2	0.2	71.08	1060.958	1009.714	0	6166.017	4867.455	0	2.75

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	SRATIO
2	4	4	2	5	4	3	26	0.275	0.275	65.372	1437.538	0	0	8988.502	0	0	2.75
2	4	4	3	5	4	3	26	0.25	0.25	64.815	1430.756	0	0	8927.336	0	0	3
2	4	4	4	5	4	3	26	0.25	0.25	64.9592	1430.756	0	0	8927.336	0	0	3
2	4	4	5	5	4	3	26	0.25	0.25	66.3852	1430.756	0	0	8927.336	0	0	3
3	4	3	1	6	4	4	26	0.2	0.2	71.3349	1060.958	1009.714	0	6166.017	4867.455	0	2.75
2	4	4	2	6	4	3	26	0.275	0.275	65.5414	1437.538	0	0	8988.502	0	0	2.75
2	4	4	3	6	4	3	26	0.25	0.25	64.9822	1430.756	0	0	8927.336	0	0	3
2	4	4	4	6	4	3	26	0.25	0.25	65.1263	1430.756	0	0	8927.336	0	0	3
2	4	4	5	6	4	3	26	0.25	0.25	66.5539	1430.756	0	0	8927.336	0	0	3
3	4	3	1	7	4	4	26	0.2	0.2	71.6293	1060.958	1009.714	0	6166.017	4867.455	0	2.75
2	4	4	2	7	4	3	26	0.275	0.275	65.739	1437.538	0	0	8988.502	0	0	2.75
2	4	4	3	7	4	3	26	0.25	0.25	65.1772	1430.756	0	0	8927.336	0	0	3
2	4	4	4	7	4	3	26	0.25	0.25	65.3213	1430.756	0	0	8927.336	0	0	3
2	4	4	5	7	4	3	26	0.25	0.25	66.7488	1430.756	0	0	8927.336	0	0	3
3	4	3	1	8	4	4	26	0.2	0.2	71.8853	1060.958	1009.714	0	6166.017	4867.455	0	2.75
2	4	4	2	8	4	3	26	0.275	0.275	65.9149	1437.538	0	0	8988.502	0	0	2.75
2	4	4	3	8	4	3	26	0.275	0.275	66.1783	1437.538	0	0	8988.502	0	0	2.75
2	4	4	4	8	4	3	26	0.25	0.25	65.4903	1430.756	0	0	8927.336	0	0	3
2	4	4	5	8	4	3	26	0.25	0.25	66.9178	1430.756	0	0	8927.336	0	0	3

APPENDIX - G2 (UNDER IRC CLASS A LOADS)

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	SRATIO
3	4	2	1	1	2	4	25	0.2	0.26	67.2899	930.754	850.187	0	5020.728	4750.901	0	2.25
3	4	2	2	1	2	4	25	0.2	0.245	67.3658	930.754	850.187	0	5020.728	4750.901	0	2.25
3	4	2	3	1	2	4	25	0.2	0.245	67.5895	930.754	850.187	0	5020.728	4750.901	0	2.25
3	4	2	4	1	2	4	25	0.2	0.245	67.7158	930.754	850.187	0	5020.728	4750.901	0	2.25
3	4	2	5	1	2	4	25	0.2	0.245	68.9942	930.754	850.187	0	5020.728	4750.901	0	2.25
2	4	4	1	2	2	3	25	0.23	0.26	62.451	1303.295	0	0	7531.282	0	0	2.25
2	4	4	2	2	2	3	25	0.21	0.245	62.2124	1277.122	0	0	7367.016	0	0	2.25
2	4	4	3	2	2	3	25	0.21	0.245	62.4433	1277.122	0	0	7367.016	0	0	2.25
2	4	4	4	2	2	2	25	0.21	0.245	62.5722	1277.122	0	0	7367.016	0	0	2.25
2	4	4	5	2	2	2	25	0.21	0.245	63.8762	1277.122	0	0	7367.016	0	0	2.25
2	4	4	1	3	2	3	25	0.23	0.26	62.5076	1303.295	0	0	7531.282	0	0	2.25
2	4	4	2	3	2	3	25	0.21	0.245	62.2666	1277.122	0	0	7367.016	0	0	2.25
2	4	4	3	3	2	3	25	0.21	0.245	62.4992	1277.122	0	0	7367.016	0	0	2.25
2	4	4	4	3	2	2	25	0.21	0.245	62.6279	1277.122	0	0	7367.016	0	0	2.25
2	4	4	5	3	2	2	25	0.21	0.245	63.9319	1277.122	0	0	7367.016	0	0	2.25
2	4	4	1	4	2	3	25	0.23	0.26	63.2848	1303.295	0	0	7531.282	0	0	2.25
2	4	4	2	4	2	3	25	0.21	0.245	63.0427	1277.122	0	0	7367.016	0	0	2.25
2	4	4	3	4	2	3	25	0.21	0.245	63.2752	1277.122	0	0	7367.016	0	0	2.25
2	4	4	4	4	2	2	25	0.21	0.245	63.4056	1277.122	0	0	7367.016	0	0	2.25
2	4	4	5	4	2	2	25	0.21	0.245	64.7095	1277.122	0	0	7367.016	0	0	2.25
2	4	4	1	5	2	4	25	0.205	0.59	64.6177	1262.458	0	0	7214.447	0	0	2.75
2	4	4	2	5	2	3	25	0.21	0.245	63.2078	1277.122	0	0	7367.016	0	0	2.25
2	4	4	3	5	2	3	25	0.21	0.245	63.4402	1277.122	0	0	7367.016	0	0	2.25
2	4	4	4	5	2	2	25	0.21	0.245	63.5719	1277.122	0	0	7367.016	0	0	2.25
2	4	4	5	5	2	2	25	0.21	0.245	64.8775	1277.122	0	0	7367.016	0	0	2.25
2	4	4	1	6	2	4	25	0.205	0.59	64.7838	1262.458	0	0	7214.447	0	0	2.75
2	4	4	2	6	2	3	25	0.21	0.245	63.375	1277.122	0	0	7367.016	0	0	2.25
2	4	4	3	6	2	3	25	0.21	0.245	63.6074	1277.122	0	0	7367.016	0	0	2.25
2	4	4	4	6	2	2	25	0.21	0.245	63.739	1277.122	0	0	7367.016	0	0	2.25
2	4	4	5	6	2	2	25	0.21	0.245	65.0452	1277.122	0	0	7367.016	0	0	2.25
2	4	4	1	7	2	4	25	0.205	0.59	64.9749	1262.458	0	0	7214.447	0	0	2.75
2	4	4	2	7	2	3	25	0.21	0.245	63.5679	1277.122	0	0	7367.016	0	0	2.25

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	SRATIO
2	4	4	3	7	2	3	25	0.21	0.245	63.8002	1277.122	0	0	7367.016	0	0	2.25
2	4	4	4	7	2	3	25	0.21	0.245	63.9331	1277.122	0	0	7367.016	0	0	2.25
2	4	4	5	7	2	2	25	0.21	0.245	65.238	1277.122	0	0	7367.016	0	0	2.25
2	4	4	1	8	2	4	25	0.205	0.59	65.141	1262.458	0	0	7214.447	0	0	2.75
2	4	4	2	8	2	3	25	0.21	0.245	63.734	1277.122	0	0	7367.016	0	0	2.25
2	4	4	3	8	2	3	25	0.21	0.245	63.968	1277.122	0	0	7367.016	0	0	2.25
2	4	4	4	8	2	3	25	0.21	0.245	64.1008	1277.122	0	0	7367.016	0	0	2.25
2	4	4	5	8	2	2	25	0.21	0.245	65.4057	1277.122	0	0	7367.016	0	0	2.25

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	SRATIO
2	4	3	1	1	8	2	25	0.245	0.26	62.158	1355.665	0	0	7822.924	0	0	2.25
2	4	3	2	1	8	1	25	0.225	0.245	62.0615	1329.386	0	0	7658.083	0	0	2.25
2	4	3	3	1	8	1	25	0.225	0.245	62.2938	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	1	8	1	25	0.225	0.245	62.4244	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	1	8	1	25	0.225	0.245	63.7607	1329.386	0	0	7658.083	0	0	2.25
2	4	3	1	2	8	2	25	0.245	0.26	62.2877	1355.665	0	0	7822.924	0	0	2.25
2	4	3	2	2	8	1	25	0.225	0.245	62.1913	1329.386	0	0	7658.083	0	0	2.25
2	4	3	3	2	8	1	25	0.225	0.245	62.4236	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	2	8	1	25	0.225	0.245	62.554	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	2	8	1	25	0.225	0.245	63.8902	1329.386	0	0	7658.083	0	0	2.25
2	4	3	1	3	8	2	25	0.245	0.26	62.352	1355.665	0	0	7822.924	0	0	2.25
2	4	3	2	3	8	1	25	0.225	0.245	62.2558	1329.386	0	0	7658.083	0	0	2.25
2	4	3	3	3	8	1	25	0.225	0.245	62.4879	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	3	8	1	25	0.225	0.245	62.6183	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	3	8	1	25	0.225	0.245	63.9544	1329.386	0	0	7658.083	0	0	2.25
2	4	3	1	4	8	2	25	0.245	0.26	63.2102	1355.665	0	0	7822.924	0	0	2.25
2	4	3	2	4	8	1	25	0.225	0.245	63.1128	1329.386	0	0	7658.083	0	0	2.25
2	4	3	3	4	8	1	25	0.225	0.245	63.3448	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	4	8	1	25	0.225	0.245	63.4751	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	4	8	1	25	0.225	0.245	64.8111	1329.386	0	0	7658.083	0	0	2.25
2	4	3	1	5	8	2	25	0.245	0.26	63.3934	1355.665	0	0	7822.924	0	0	2.25
2	4	3	2	5	8	1	25	0.225	0.245	63.2961	1329.386	0	0	7658.083	0	0	2.25

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	SRATIO
2	4	3	3	5	8	1	25	0.225	0.245	63.528	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	5	8	1	25	0.225	0.245	63.6581	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	5	8	1	25	0.225	0.245	64.994	1329.386	0	0	7658.083	0	0	2.25
2	4	3	1	6	8	2	25	0.245	0.26	63.579	1355.665	0	0	7822.924	0	0	2.25
2	4	3	2	6	8	2	25	0.225	0.245	63.4838	1329.386	0	0	7658.083	0	0	2.25
2	4	3	3	6	8	1	25	0.225	0.245	63.7134	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	6	8	1	25	0.225	0.245	63.8434	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	6	8	1	25	0.225	0.245	65.1792	1329.386	0	0	7658.083	0	0	2.25
2	4	3	1	7	8	2	25	0.235	0.405	64.2721	1333.202	0	0	7684.841	0	0	2.5
2	4	3	2	7	8	2	25	0.225	0.245	63.695	1329.386	0	0	7658.083	0	0	2.25
2	4	3	3	7	8	1	25	0.225	0.245	63.9273	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	7	8	1	25	0.225	0.245	64.0572	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	7	8	1	25	0.225	0.245	65.3929	1329.386	0	0	7658.083	0	0	2.25
2	4	3	1	8	8	2	25	0.235	0.405	64.4521	1333.202	0	0	7684.841	0	0	2.5
2	4	3	2	8	8	2	25	0.225	0.245	63.8779	1329.386	0	0	7658.083	0	0	2.25
2	4	3	3	8	8	1	25	0.225	0.245	64.1122	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	8	8	1	25	0.225	0.245	64.2432	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	8	8	1	25	0.225	0.245	65.5788	1329.386	0	0	7658.083	0	0	2.25

APPENDIX - G3

(UNDER IRC CLASS A LOADS, RESULTS FOR ALL VALUES OF SRATIO)

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	SRATIO
3	4	2	1	1	8	1	25	0.215	0.215	71.0524	1073.645	1128.799	0	6130.58	5657.745	0	1
2	12	4	1	1	8	3	25	0.24	0.24	71.7693	1497.951	0	0	8718.249	0	0	1.25
2	12	4	1	1	8	9	25	0.24	0.24	71.1892	1481.414	0	0	8616.309	0	0	1.5
3	4	2	1	1	8	1	25	0.2	0.23	68.9575	982.37	975.937	0	5465.898	5112.527	0	1.75
2	4	3	1	1	8	2	25	0.24	0.245	62.4771	1359.278	0	0	7842.038	0	0	2
2	4	3	1	1	8	2	25	0.245	0.26	62.158	1355.665	0	0	7822.924	0	0	2.25
2	4	3	1	1	8	2	25	0.235	0.405	62.6639	1333.202	0	0	7684.841	0	0	2.5
2	4	3	1	1	8	2	25	0.225	0.59	63.5057	1342.712	0	0	7656.552	0	0	2.75
2	4	3	1	1	8	2	25	0.215	0.76	64.4454	1296.761	0	0	7420.88	0	0	3
3	4	2	2	1	8	1	25	0.2	0.2	70.9683	1057.117	1111.929	0	6027.002	5552.332	0	1
2	12	4	2	1	8	9	25	0.225	0.225	71.6729	1478.106	0	0	8593.887	0	0	1.25
2	4	3	2	1	8	1	25	0.24	0.24	63.3993	1380.763	0	0	7968.755	0	0	1.5
2	4	3	2	1	8	1	25	0.23	0.23	62.9086	1356.962	0	0	7823.415	0	0	1.75
2	4	3	2	1	8	1	25	0.22	0.23	62.4324	1333.047	0	0	7677.458	0	0	2
2	4	3	2	1	8	1	25	0.225	0.245	62.0615	1329.386	0	0	7658.083	0	0	2.25
2	4	3	2	1	8	1	25	0.215	0.405	62.567	1306.822	0	0	7519.45	0	0	2.5
2	4	3	2	1	8	1	25	0.21	0.555	63.2738	1322.574	0	0	7532.104	0	0	2.75
2	4	3	2	1	8	1	25	0.2	0.565	63.0551	1276.621	0	0	7296.398	0	0	3
3	4	2	3	1	8	1	25	0.2	0.2	71.1807	1057.117	1111.929	0	6027.002	5552.332	0	1
2	12	4	3	1	8	3	25	0.225	0.225	71.8954	1478.106	0	0	8593.887	0	0	1.25
2	12	4	3	1	8	9	25	0.225	0.225	71.3329	1461.548	0	0	8491.83	0	0	1.5
2	4	3	3	1	8	1	25	0.23	0.23	63.1372	1356.962	0	0	7823.415	0	0	1.75
2	4	3	3	1	8	1	25	0.22	0.23	62.659	1333.047	0	0	7677.458	0	0	2
2	4	3	3	1	8	1	25	0.225	0.245	62.2938	1329.386	0	0	7658.083	0	0	2.25
2	4	3	3	1	8	1	25	0.215	0.405	62.8215	1306.822	0	0	7519.45	0	0	2.5
2	4	3	3	1	8	1	25	0.21	0.555	63.5555	1322.574	0	0	7532.104	0	0	2.75
2	4	3	3	1	8	1	25	0.2	0.565	63.343	1276.621	0	0	7296.398	0	0	3

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	SRATIO
2	9	4	4	1	8	9	25	0.21	0.21	69.6502	1477.009	0	0	8601.311	0	0	1
2	11	4	4	1	8	9	25	0.22	0.22	70.8091	1458.099	0	0	8476.336	0	0	1.25
2	12	4	4	1	8	3	25	0.225	0.225	71.4588	1461.548	0	0	8491.83	0	0	1.5
2	4	3	4	1	8	1	25	0.23	0.23	63.2654	1356.962	0	0	7823.415	0	0	1.75
2	4	3	4	1	8	1	25	0.22	0.23	62.7863	1333.047	0	0	7677.458	0	0	2
2	4	3	4	1	8	1	25	0.225	0.245	62.4244	1329.386	0	0	7658.083	0	0	2.25
2	4	3	4	1	8	1	25	0.215	0.405	62.9652	1306.822	0	0	7519.45	0	0	2.5
2	4	3	4	1	8	1	25	0.21	0.555	63.7152	1322.574	0	0	7532.104	0	0	2.75
2	4	3	4	1	8	1	25	0.2	0.565	63.5066	1276.621	0	0	7296.398	0	0	3
2	8	4	5	1	8	9	25	0.215	0.215	70.1544	1467.718	0	0	8529.537	0	0	1
2	11	4	5	1	8	3	25	0.22	0.22	72.0744	1458.099	0	0	8476.336	0	0	1.25
2	11	4	5	1	8	9	25	0.22	0.22	71.5818	1442.389	0	0	8379.107	0	0	1.5
2	4	3	5	1	8	1	25	0.23	0.23	64.5834	1356.962	0	0	7823.415	0	0	1.75
2	4	3	5	1	8	1	25	0.22	0.23	64.0891	1333.047	0	0	7677.458	0	0	2
2	4	3	5	1	8	1	25	0.225	0.245	63.7607	1329.386	0	0	7658.083	0	0	2.25
2	4	3	5	1	8	1	25	0.215	0.405	64.4191	1306.822	0	0	7519.45	0	0	2.5
2	4	3	5	1	8	1	25	0.21	0.555	65.3164	1322.574	0	0	7532.104	0	0	2.75
2	4	3	5	1	8	1	25	0.2	0.565	65.1378	1276.621	0	0	7296.398	0	0	3

STRUCTURALLY FEASIBLE DESIGNS INCORPOTING THE PROPOSED AMENDMENTS IN IRC: 18-1985 (1997) AND IRC: 21-1987 (1997)

APPENDIX - H1 (UNDER IRC CLASS AA LOADS)

ng	ncb	ips	icon1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	STATIO
5	5	3	1	1	1	18	27	0.215	0.215	85.2946	689.713	832.176	705.665	4220.923	4071.157	3906.737	1.75
5	5	3	2	1	1	18	27	0.21	0.21	85.5204	682.894	919.266	716.189	4225.053	4055.931	3881.651	1.5
4	5	3	3	1	1	18	27	0.205	0.205	75.7505	757.158	766.514	0	4487.812	4000.053	0	2.75
4	5	3	4	1	1	18	27	0.205	0.205	75.9875	732.8	804.364	0	4411.964	4081.522	0	2.5
4	5	3	5	1	1	18	27	0.2	0.2	76.9993	754.847	764.206	0	4472.21	3984.562	0	2.75
4	4	2	1	2	1	4	27	0.225	0.225	73.4436	762.866	773.038	0	4520.045	4040.958	0	2.75
4	4	2	2	2	1	4	27	0.215	0.215	73.3034	758.274	768.42	0	4489.072	4010.063	0	2.75
4	4	2	3	2	1	4	27	0.205	0.205	73.2219	753.665	763.802	0	4457.964	3979.13	0	2.75
3	4	3	4	2	1	1	27	0.225	0.245	68.0599	1022.667	1031.326	0	6223.727	4878.593	0	3
3	4	3	5	2	1	1	27	0.22	0.235	69.272	1019.51	1028.166	0	6203.234	4857.342	0	3
3	5	4	1	3	1	9	27	0.285	0.285	72.2894	1154.748	1262.432	0	6941.186	5452.095	0	2.5
3	4	3	2	3	1	1	27	0.25	0.275	68.3122	1038.455	1047.128	0	6325.091	4984.749	0	3
3	4	3	3	3	1	1	27	0.235	0.255	68.1938	1028.983	1037.647	0	6264.486	4921.073	0	3
3	4	3	4	3	1	1	27	0.225	0.245	68.1154	1022.667	1031.326	0	6223.727	4878.593	0	3
3	4	3	5	3	1	1	27	0.22	0.235	69.3286	1019.51	1028.166	0	6203.234	4857.342	0	3
3	4	3	1	4	1	1	27	0.265	0.295	69.4313	1047.927	1056.612	0	6385.109	5048.372	0	3
3	4	3	2	4	1	1	27	0.25	0.275	69.1612	1038.455	1047.128	0	6325.091	4984.749	0	3
3	4	3	3	4	1	1	27	0.235	0.255	69.0423	1028.983	1037.647	0	6264.486	4921.073	0	3
3	4	3	4	4	1	1	27	0.225	0.245	68.9636	1022.667	1031.326	0	6223.727	4878.593	0	3
3	4	3	5	4	1	1	27	0.22	0.235	70.1769	1019.51	1028.166	0	6203.234	4857.342	0	3
2	4	5	1	5	1	3	27	0.4	0.4	66.5538	1550.28	0	0	10052.48	0	0	3
2	4	5	2	5	1	3	27	0.37	0.37	66.0824	1506.54	0	0	9775.291	0	0	3
2	4	5	3	5	1	3	27	0.345	0.345	65.8164	1470.09	0	0	9544.148	0	0	3
2	4	5	4	5	1	3	27	0.33	0.33	65.6542	1448.22	0	0	9405.318	0	0	3
2	4	5	5	5	1	3	27	0.315	0.315	66.9446	1426.35	0	0	9266.325	0	0	3
2	4	5	1	6	1	2	27	0.4	0.4	66.6558	1550.28	0	0	10052.48	0	0	3
2	4	5	2	6	1	2	27	0.37	0.37	66.1847	1506.54	0	0	9775.291	0	0	3
2	4	5	3	6	1	2	27	0.345	0.345	65.9193	1470.09	0	0	9544.148	0	0	3

ng	nc b	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	STATIO
2	4	5	4	6	1	2	27	0.33	0.33	65.758	1448.22	0	0	9405.318	0	0	3
2	4	5	5	6	1	2	27	0.315	0.315	67.0491	1426.35	0	0	9266.325	0	0	3
2	4	5	1	7	1	2	27	0.4	0.4	66.781	1550.28	0	0	10052.48	0	0	3
2	4	5	2	7	1	2	27	0.37	0.37	66.3102	1506.54	0	0	9775.291	0	0	3
2	4	5	3	7	1	2	27	0.345	0.345	66.0453	1470.09	0	0	9544.148	0	0	3
2	4	5	4	7	1	2	27	0.33	0.33	65.8847	1448.22	0	0	9405.318	0	0	3
2	4	5	5	7	1	2	27	0.315	0.315	67.1767	1426.35	0	0	9266.325	0	0	3
2	4	5	1	8	1	2	27	0.4	0.4	66.8906	1550.28	0	0	10052.48	0	0	3
2	4	5	2	8	1	2	27	0.37	0.37	66.4269	1506.54	0	0	9775.291	0	0	3
2	4	5	3	8	1	2	27	0.345	0.345	66.1673	1470.09	0	0	9544.148	0	0	3
2	4	5	4	8	1	2	27	0.33	0.33	66.001	1448.22	0	0	9405.318	0	0	3
2	4	5	5	8	1	2	27	0.315	0.315	67.2864	1426.35	0	0	9266.325	0	0	3
ng	nc b	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	STATIO
4	5	5	1	4	7	6	38	0.225	0.225	81.0311	984.984	1031.375	0	8460.579	7806.833	0	2.5
4	5	5	2	4	7	6	38	0.215	0.215	80.8807	978.111	1024.476	0	8395.312	7741.585	0	2.5
4	5	5	3	4	7	12	38	0.2	0.2	80.6269	992.168	970.21	0	8397.124	7497.304	0	2.75
4	5	5	4	4	7	11	38	0.2	0.2	80.7411	967.775	1014.127	0	8297.127	7643.637	0	2.5
4	5	5	5	4	7	6	38	0.2	0.2	82.0202	967.775	1014.127	0	8297.127	7643.637	0	2.5
3	5	5	1	5	7	4	38	0.265	0.265	72.558	1352.547	1404.275	0	11773.09	9685.72	0	2.75
3	5	5	2	5	7	4	38	0.24	0.275	72.0746	1287.233	1273.323	0	11209.54	9230.089	0	3
3	5	5	3	5	7	4	38	0.23	0.255	71.9904	1278.805	1264.681	0	11129.46	9148.088	0	3
3	5	5	4	5	7	4	38	0.22	0.245	71.903	1270.34	1256.041	0	11049.05	9066.068	0	3
3	5	5	5	5	7	4	38	0.21	0.235	73.0172	1261.837	1247.4	0	10968.27	8984.031	0	3
3	5	5	1	6	7	4	38	0.255	0.295	72.5861	1299.813	1286.284	0	11329.03	9353.092	0	3
3	5	5	2	6	7	4	38	0.24	0.275	72.2781	1287.233	1273.323	0	11209.54	9230.089	0	3
3	5	5	3	6	7	4	38	0.23	0.255	72.1939	1278.805	1264.681	0	11129.46	9148.088	0	3
3	5	5	4	6	7	4	38	0.22	0.245	72.1066	1270.34	1256.041	0	11049.05	9066.068	0	3
3	5	5	5	6	7	4	38	0.21	0.235	73.221	1261.837	1247.4	0	10968.27	8984.031	0	3
2	5	7	1	7	7	1	38	0.385	0.385	70.3687	1993.859	0	0	18324.85	0	0	3
2	5	7	2	7	7	1	38	0.36	0.36	70.0028	1942.559	0	0	17857.07	0	0	3
2	5	7	3	7	7	1	38	0.335	0.335	69.7367	1891.259	0	0	17388.7	0	0	3
2	5	7	4	7	7	1	38	0.32	0.32	69.5696	1860.479	0	0	17107.28	0	0	3
2	5	7	5	7	7	1	38	0.305	0.305	70.8284	1829.699	0	0	16825.51	0	0	3
2	5	7	1	8	7	1	38	0.385	0.385	70.5028	1993.859	0	0	18324.85	0	0	3
2	5	7	2	8	7	1	38	0.36	0.36	70.1371	1942.559	0	0	17857.07	0	0	3
2	5	7	3	8	7	1	38	0.335	0.335	69.8722	1891.259	0	0	17388.7	0	0	3
2	5	7	4	8	7	1	38	0.32	0.32	69.7058	1860.479	0	0	17107.28	0	0	3
2	5	7	5	8	7	1	38	0.305	0.305	70.9654	1829.699	0	0	16825.51	0	0	3

APPENDIX - H2 (UNDER IRC CLASS A LOADS)

ng	ncb	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	STATIO
3	4	2	1	1	6	1	25	0.2	0.295	67.3546	920.677	869.617	0	5024.774	4730.173	0	2.25
3	4	2	2	1	6	1	25	0.2	0.275	67.4121	920.677	869.617	0	5024.774	4730.173	0	2.25
3	4	2	3	1	6	1	25	0.2	0.265	67.5888	920.677	869.617	0	5024.774	4730.173	0	2.25
3	4	2	4	1	6	1	25	0.2	0.25	67.6346	920.677	869.617	0	5024.774	4730.173	0	2.25
3	4	2	5	1	6	1	25	0.2	0.24	68.8552	920.677	869.617	0	5024.774	4730.173	0	2.25
2	4	4	1	2	6	15	25	0.28	0.295	63.5663	1358.902	0	0	7884.888	0	0	2.25
2	4	4	2	2	6	15	25	0.26	0.275	63.2877	1333.043	0	0	7722.327	0	0	2.25
2	4	4	3	2	6	15	25	0.245	0.265	63.2273	1313.595	0	0	7600.111	0	0	2.25
2	4	3	4	2	6	1	25	0.21	0.635	63.1402	1249.701	0	0	7144.652	0	0	3
2	4	3	5	2	6	1	25	0.2	0.565	64.1142	1236.281	0	0	7061.795	0	0	3
2	4	4	1	3	6	1	25	0.28	0.295	63.6119	1358.902	0	0	7884.888	0	0	2.25
2	4	4	2	3	6	1	25	0.26	0.275	63.3338	1333.043	0	0	7722.327	0	0	2.25
2	4	4	3	3	6	1	25	0.245	0.265	63.2738	1313.595	0	0	7600.111	0	0	2.25
2	4	3	4	3	6	1	25	0.21	0.635	63.1896	1249.701	0	0	7144.652	0	0	3
2	4	3	5	3	6	1	25	0.215	0.405	63.3743	1264.831	0	0	7298.163	0	0	2.5
2	4	4	1	4	6	1	25	0.28	0.295	64.3428	1358.902	0	0	7884.888	0	0	2.25
2	4	4	2	4	6	1	25	0.26	0.275	64.0639	1333.043	0	0	7722.327	0	0	2.25
2	4	4	3	4	6	1	25	0.245	0.265	64.0032	1313.595	0	0	7600.111	0	0	2.25
2	4	3	4	4	6	1	25	0.225	0.405	62.7776	1277.927	0	0	7380.345	0	0	2.5
2	4	3	5	4	6	1	25	0.215	0.405	64.0936	1264.831	0	0	7298.163	0	0	2.5
2	4	3	1	5	6	1	25	0.28	0.295	62.6375	1358.902	0	0	7884.888	0	0	2.25
2	4	3	2	5	6	1	25	0.26	0.275	62.3648	1333.043	0	0	7722.327	0	0	2.25
2	4	3	3	5	6	1	25	0.245	0.265	62.3086	1313.595	0	0	7600.111	0	0	2.25
2	4	3	4	5	6	1	25	0.235	0.25	62.2476	1300.6	0	0	7518.473	0	0	2.25
2	4	3	5	5	6	1	25	0.225	0.24	63.4533	1287.579	0	0	7436.693	0	0	2.25
2	4	3	1	6	6	1	25	0.28	0.295	62.7971	1358.902	0	0	7884.888	0	0	2.25
2	4	3	2	6	6	1	25	0.26	0.275	62.5059	1333.043	0	0	7722.327	0	0	2.25
2	4	3	3	6	6	1	25	0.245	0.265	62.4465	1313.595	0	0	7600.111	0	0	2.25
2	4	3	4	6	6	1	25	0.235	0.25	62.386	1300.6	0	0	7518.473	0	0	2.25
2	4	3	5	6	6	1	25	0.225	0.24	63.5924	1287.579	0	0	7436.693	0	0	2.25
2	4	3	1	7	6	1	25	0.28	0.295	62.9958	1358.902	0	0	7884.888	0	0	2.25
2	4	3	2	7	6	1	25	0.26	0.275	62.7043	1333.043	0	0	7722.327	0	0	2.25

ng	ncb	ips	icon1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	STATIO
2	4	3	3	7	6	1	25	0.245	0.265	62.6347	1313.595	0	0	7600.111	0	0	2.25
2	4	3	4	7	6	1	25	0.235	0.25	62.567	1300.6	0	0	7518.473	0	0	2.25
2	4	3	5	7	6	1	25	0.225	0.24	63.7642	1287.579	0	0	7436.693	0	0	2.25
2	4	3	1	8	6	1	25	0.28	0.295	63.1697	1358.902	0	0	7884.888	0	0	2.25
2	4	3	2	8	6	1	25	0.26	0.275	62.8779	1333.043	0	0	7722.327	0	0	2.25
2	4	3	3	8	6	1	25	0.245	0.265	62.8081	1313.595	0	0	7600.111	0	0	2.25
2	4	3	4	8	6	1	25	0.235	0.25	62.7403	1300.6	0	0	7518.473	0	0	2.25
2	4	3	5	8	6	1	25	0.225	0.24	63.9374	1287.579	0	0	7436.693	0	0	2.25
ng	ncb	ips	icon1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	STATIO
2	4	3	1	1	7	1	25	0.27	0.405	62.4966	1355.027	0	0	7845.952	0	0	2.5
2	4	3	2	1	7	1	25	0.26	0.275	61.9327	1351.316	0	0	7819.138	0	0	2.25
2	4	3	3	1	7	1	25	0.245	0.265	61.8732	1331.769	0	0	7696.383	0	0	2.25
2	4	3	4	1	7	1	25	0.235	0.25	61.8105	1318.709	0	0	7614.396	0	0	2.25
2	4	3	5	1	7	1	25	0.225	0.24	63.0141	1305.626	0	0	7532.276	0	0	2.25
2	4	3	1	2	7	1	25	0.28	0.295	62.3234	1377.311	0	0	7982.437	0	0	2.25
2	4	3	2	2	7	1	25	0.26	0.275	62.0464	1351.316	0	0	7819.138	0	0	2.25
2	4	3	3	2	7	1	25	0.245	0.265	61.9873	1331.769	0	0	7696.383	0	0	2.25
2	4	3	4	2	7	1	25	0.235	0.25	61.9247	1318.709	0	0	7614.396	0	0	2.25
2	4	3	5	2	7	1	25	0.225	0.24	63.1286	1305.626	0	0	7532.276	0	0	2.25
2	4	3	1	3	7	1	25	0.28	0.295	62.377	1377.311	0	0	7982.437	0	0	2.25
2	4	3	2	3	7	1	25	0.26	0.275	62.1006	1351.316	0	0	7819.138	0	0	2.25
2	4	3	3	3	7	1	25	0.245	0.265	62.0418	1331.769	0	0	7696.383	0	0	2.25
2	4	3	4	3	7	1	25	0.235	0.25	61.9795	1318.709	0	0	7614.396	0	0	2.25
2	4	3	5	3	7	1	25	0.225	0.24	63.1836	1305.626	0	0	7532.276	0	0	2.25
2	4	3	1	4	7	1	25	0.28	0.295	63.161	1377.311	0	0	7982.437	0	0	2.25
2	4	3	2	4	7	1	25	0.26	0.275	62.8838	1351.316	0	0	7819.138	0	0	2.25
2	4	3	3	4	7	1	25	0.245	0.265	62.8244	1331.769	0	0	7696.383	0	0	2.25
2	4	3	4	4	7	1	25	0.235	0.25	62.7616	1318.709	0	0	7614.396	0	0	2.25
2	4	3	5	4	7	1	25	0.225	0.24	63.9652	1305.626	0	0	7532.276	0	0	2.25
2	4	3	1	5	7	1	25	0.28	0.295	63.2617	1377.311	0	0	7982.437	0	0	2.25
2	4	3	2	5	7	1	25	0.26	0.275	62.9702	1351.316	0	0	7819.138	0	0	2.25
2	4	3	3	5	7	1	25	0.245	0.265	62.9006	1331.769	0	0	7696.383	0	0	2.25
2	4	3	4	5	7	1	25	0.235	0.25	62.8351	1318.709	0	0	7614.396	0	0	2.25
2	4	3	5	5	7	1	25	0.225	0.24	64.0419	1305.626	0	0	7532.276	0	0	2.25

ng	nc b	ips	icon 1	icon	ig	iprseq	span (m)	ds (m)	dcs (m)	tcost	desf1	desf2	desf3	debm1	debm2	debm3	STATIO
2	4	3	1	6	7	1	25	0.28	0.295	63.4504	1377.311	0	0	7982.437	0	0	2.25
2	4	3	2	6	7	1	25	0.26	0.275	63.1586	1351.316	0	0	7819.138	0	0	2.25
2	4	3	3	6	7	1	25	0.245	0.265	63.0889	1331.769	0	0	7696.383	0	0	2.25
2	4	3	4	6	7	1	25	0.235	0.25	63.0211	1318.709	0	0	7614.396	0	0	2.25
2	4	3	5	6	7	1	25	0.225	0.24	64.2182	1305.626	0	0	7532.276	0	0	2.25
2	4	3	1	7	7	1	25	0.28	0.295	63.6639	1377.311	0	0	7982.437	0	0	2.25
2	4	3	2	7	7	1	25	0.26	0.275	63.3718	1351.316	0	0	7819.138	0	0	2.25
2	4	3	3	7	7	1	25	0.245	0.265	63.3018	1331.769	0	0	7696.383	0	0	2.25
2	4	3	4	7	7	1	25	0.235	0.25	63.2339	1318.709	0	0	7614.396	0	0	2.25
2	4	3	5	7	7	1	25	0.225	0.24	64.4308	1305.626	0	0	7532.276	0	0	2.25
2	4	3	1	8	7	1	25	0.28	0.295	63.829	1377.311	0	0	7982.437	0	0	2.25
2	4	3	2	8	7	1	25	0.26	0.275	63.5368	1351.316	0	0	7819.138	0	0	2.25
2	4	3	3	8	7	1	25	0.245	0.265	63.4668	1331.769	0	0	7696.383	0	0	2.25
2	4	3	4	8	7	1	25	0.235	0.25	63.3989	1318.709	0	0	7614.396	0	0	2.25
2	4	3	5	8	7	1	25	0.225	0.24	64.5958	1305.626	0	0	7532.276	0	0	2.25

VITA

The author, Virendra Kumar, was born on 25th may 1961 in town ORAI of JALAUN district, Uttar Pradesh. He completed his school education from Government Inter College, ORAI. He passed the intermediate examination conducted by U. P. board Allahabad in 1978. He completed his B.Sc. (Engg.) in Civil Engineering in 1985 from Regional Engineering College, Kurukshetra.

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Presently, the author is working as Assistant Professor in Civil Engineering Department, College of Technology, G. B. Pant University, Pantnagar.

ABSTRACT

NAME: VIRENDRA KUMAR ID. NO. 17775

Semester and year of admission: II - 1993-94 Degree: Ph.D.

Major: Structural Engineering Department: Civil Engineering

Thesis Title: "ANALYTICAL STUDIES ON HIGH-PERFORMANCE CONCRETE T-BEAM BRIDGES"

Advisor : Dr. V. P. Bhargava

This project is a analytical investigation on composite prestressed high-performance concrete simply supported slab-on-girder bridges. The main thrust of the investigation is on the following areas:

1) The influence of the various parameters of the bridge superstructure in increasing its structural efficiency and reducing the cost.

2) The limitations of the IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997) codes to advantageous application of high-strength concrete in highway bridges.

3) Proposal of necessary amendments in IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997) codes to remove the obstacles against the economic use of high-strength concrete in highway bridges designed according to these codes.

4) Development of computer programme in FORTRAN and a method of optimal design of high-performance concrete slab-on-girder bridges.

Analytical study was made with the following as main parameters:

I) Prestressing force

II) Number of longitudinal girders (2-5)

III) Number of cross beams (3-12)

IV) Transverse spacing of longitudinal girders

V) Deck slab concrete grade (30-50 MPa)

VI) Girder concrete grade (35-90 MPa)

VII) Type of girder (8 standard girder sections)

The importance of various parameters of bridge superstructure on its structural efficiency and cost has been clearly established in the present investigation. The results obtained have shown that provisions of IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997) can not be extended without modification to high-strength concrete. The modifications proposed in IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997) yielded the results which are comparable to the results of the similar investigations undertaken by other researchers. The results of the study are reported in the form of tables and graphs which may be used in optimal preliminary design of simply supported prestressed concrete slab-on-girder bridges. The areas of further research have been identified in the present investigation.

There appears to be a considerable advantage in extending the application of high-strength concrete in highway bridges. But, this is possible only when the much needed amendments are incorporated in the IRC: 18 - 1985 (1997) and IRC: 21 - 1987 (1997).

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