

TECHNICAL METHODS FOR HIGHWAYS

**TMH7
PART 3**

**CODE OF PRACTICE
FOR THE DESIGN OF
HIGHWAY BRIDGES
AND CULVERTS
IN SOUTH AFRICA**

1989

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PREFACE

This Code is issued by the Committee of State Road Authorities as a manual to be used in the design of highway bridges and culverts in order to encourage the application of modern design philosophy and to bring about greater uniformity of standards throughout South Africa. Although some deviations to suit local requirements may be necessary, it is important that the agreed standards and design principles expounded here be adhered to.

The Code is too specific and detailed to be strictly classified as a model code, but this does not mean that it must necessarily be rigidly adhered to in all its detail. In order not to retard progress, competent designers should retain the right to exercise their discretion in applying new and improved theories or methods based on proven research, provided that this does not impair the reliability of the structure. It is the intention to update or extend this Code as and when required by issuing new editions.

For any particular application, the specific interpretation of any part of this Code shall be subject to the ruling of the responsible bridge authority. The designer shall inform the responsible authority of his intention to apply alternative or new methods not described in this Code.

TECHNICAL METHODS FOR HIGHWAYS (TMH) is a series complementing the TECHNICAL RECOMMENDATIONS FOR HIGHWAYS (TRH) series. The TRHs are intended as guides for the practising engineer and leave room for engineering judgement to be used. The TMHs, on the other hand, are intended as manuals for engineers, prescribing methods to be used in various bridge and road designs and construction procedures. It is hoped that the use of these manuals will produce uniform results and standards throughout the country.

This TMH was prepared by the Subcommittee on Bridge Loadings of the Committee of State Road Authorities (CSRA), which included a representative from SWA/Namibia. It carries the full approval of the CSRA.

SYNOPSIS

This book contains the third part of the Code of Practice for the Design of Highway Bridges and Culverts in South Africa. The Code is based on the principles of limit state design as contained in the recommendations of the *CEB-FIP Model Code for Concrete Structures* published in France in 1978. This method can be considered to be a semi-probabilistic procedure using partial safety factors (level 1) to determine the effects of design actions on the structure and the resistance of members to these effects and involves checking for the ultimate limit state and the serviceability limit state.

Part 3 of the Code defines the limit state requirements and, with reference to Part 2, gives the required combinations of actions and their application to the structure in checking for the limit states. The material properties and design assumptions for use in analysis are also given. Guidelines are provided for the detailed design of reinforced, prestressed and partially prestressed concrete as well as of precast, composite and plain concrete construction. Material strength reduction factors are specified for use in conjunction with the partial load factors given in Part 2. Explanatory notes are contained in the Appendices.

SINOPSIS

Hierdie dokument bevat die derde deel van die Gebruikskode vir die Ontwerp van Padbrûe en -duikers in Suid-Afrika. Die kode is gebaseer op die beginsels van grenstoestantontwerp soos begrepe in die aanbevelings van die *CEB-FIP Model Code for Concrete Structures* wat in 1978 gepubliseer is. Hierdie metode kan beskou word as 'n halfwaarskynlikheidsprosedure wat gedeeltelike veiligheidsfaktore (standaard 1) gebruik om die effekte van ontwerpwerking op die struktuur en die weerstand van dele daaraan te bepaal en behels die nagaan van die uiterste grenstoestand en die diensbaarheidsgrenstoestand.

Deel 3 van die Kode definieer die grenstoestandvereistes en deur na Deel 2 te verwys, die vereiste kombinasies van werkings en die toepassing daarvan op strukture by die nagaan van grenstoestande. Die materiaaleienskappe en ontwerp-aannames wat in ontleding gebruik word, word beskryf. Riglyne vir die gedetailleerde ontwerp van gewapende-, span- en gedeeltelike spanbeton asook vir voorafgegiete-, saamgestelde en ongewapende betonkonstruksie word aangegee. Reduksiefaktore vir materiaalsterkte word gespesifiseer om saam met die gedeeltelike lasfaktore in Deel 2 aangegee, gebruik te word. Die bylaes bevat verklarende aantekeninge en riglyne.

KEYWORDS:

Bridge, concrete, design, structure, reinforcement, prestressing, limit states.

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1 INTRODUCTION

1.1 Scope of Part 3

This Part deals with the design of concrete bridges. It has been largely based on Part 4 of BS 5400, although certain sections incorporate the recommendations of Volume II of the *CEB-FIP Model Code for Concrete Structures* published in *Bulletin D'Information N.124/125-E* of April 1978, as well as the *FIP Recommendations on Practical Design of Reinforced and Prestressed Concrete Structures based on the above Model Code* and published in draft form in June 1982. Various other codes and research publications have influenced the drafting of some clauses. Cognizance has been taken of the *SABS Code of Practice for the Structural Use of Concrete, Part 1: Design (SABS 0100, Part 1, of 1980)* in the assessment of certain design parameters.

After stating the objectives and requirements of design, the Code gives particular requirements for reinforced concrete, prestressed concrete and composite concrete construction and for plain concrete walls and abutments. This Code covers only limited aspects of bridge design and specialist literature should be consulted for unusual problems or structures and for structural elements not dealt with here.

The structural elements dealt with are beams, slabs, columns and walls, bases, pile caps, tension members and connections between precast concrete members.

This Part should be read in conjunction with Part 1 (General Statement) and Part 2 (Specification for loads) of this Code for an explanation of the design philosophy and the definition of terms and units. Special reference should be made to Section 6 of Part 1 for the definition and explanation of the design values for actions, action effects and resistance and the methods of application of the various partial safety factors for actions (γ_{f1}) and action effects (γ_{f3}) and the material strength reduction factors (γ_m) for resistance. The relevant values of the function ($\gamma_{fL} (= \gamma_{f1} \cdot \gamma_{f2})$) are given in Part 2. The values of γ_{f3} are given in 2.2 of this Part.

1.2 Symbols

The notation applicable to this part is explained in 1.4 of Part 1. The symbols used in this Part are as follows:

A_c	Cross-sectional area of concrete
A_{cav}	Area of section subject to axial and/or flexural compression
A_{cf}	Cross-sectional area of effective concrete flange
A_{con}	Contact area
A_o	Area of fully anchored reinforcement per unit length crossing the shear plane
A_{ef}	Cross-sectional area enclosed by median lines of effective walls
A_i	Cross-sectional area of i th layer of steel
A_o	Cross-sectional area enclosed by the median wall line
A_{ps}	Cross-sectional area of prestressing tendons
A_s	Cross-sectional area of tension reinforcement

A'_s	Cross-sectional area of compression reinforcement
A'_{s1}	Cross-sectional area of compression reinforcement in the more highly compressed face
A_{s2}	Cross-sectional area of reinforcement in other face (see 3.5.3.4)
A_{sc}	Cross-sectional area of longitudinal reinforcement (for columns)
A_{sL}	Sum of the cross-sectional areas of longitudinal reinforcement provided at a section for torsion
A_{st}	Cross-sectional area of one leg of a closed link resisting torsion
A_{sup}	Supporting area
A_{sv}	Sum of the cross-sectional areas of the legs of a link or composite link resisting shear
A_{sx}	Cross-sectional area of effective tensile reinforcement in a slab in the x-axis direction
A_{sy}	Cross-sectional area of effective tensile reinforcement in a slab in the y-axis direction
A_t	Cross-sectional area of transverse tensile reinforcement
a	Deflection
a'	Distance from compression face to point at which the crack width is being calculated
a_b	Centre-to-centre distance between bars
a_{cent}	Distance of the centroid of the concrete flange from the centroid of the composite section
a_{cr}	Distance from the point (crack) being considered to the surface of the nearest longitudinal bar
a_s	Distance of the centroid of the steel from the centroid of the net concrete section
a_v	Distance from the face of a support to the line of action or point of application of a load
b	Breadth of beam section or width of slab
b_a	Average breadth of section excluding the compression flange for non-rectangular sections
b_c	Breadth of compression face
b_{col}	Breadth of column
b_s	Width of section containing effective reinforcement for punching shear
b_{sx}	Width of a section of slab containing effective reinforcement for punching shear in the x-axis direction
b_{sy}	Width of a section of slab containing effective reinforcement for punching shear in the y-axis direction
b_t	Breadth of section at level of tension reinforcement
b_w	Breadth of web or rib of a member
b_x	Width of critical concrete section of a slab for punching shear measured in the x-axis direction
b_y	Width of critical section of a concrete slab for punching shear measured in the y-axis direction
C	Torsional constant

C_{nom}	Nominal cover
D_c	Density of concrete at time of test
d	Effective depth to tension reinforcement
d'	Depth to compression reinforcement
d_c	Depth of concrete in compression
d_e	Effective depth for a solid slab or rectangular beam, otherwise the overall depth of the compression flange
d_{ef}	Diameter of the largest circle that can be contained within U_{ef}
d_o	Depth to additional reinforcement for resisting horizontal loading
d_t	Effective depth in shear
d_x	Effective depth of slab reinforcement in x-axis direction
d_y	Effective depth of slab reinforcement in y-axis direction
d_2	Depth from the other surface to the reinforcement in that other face
E	Modulus of elasticity or equivalent modulus of elasticity of a composite section
E_c	Static secant modulus of elasticity of concrete
E_{cf}	Static secant modulus of elasticity of concrete flange
E_{cq}	Dynamic tangent modulus of elasticity of concrete
E_s	Modulus of elasticity of steel
E_{28}	Secant modulus of elasticity of the concrete at the age of 28 days
e	Eccentricity or base of Napierian logarithms
e_o	Eccentricity of axial load resulting in decompression
e_x	Resultant eccentricity of load at right angles to plane of wall or the distance in the x-direction between the edge of a load and the parallel unsupported edge of a slab
e_y	Distance in the y-direction between the edge of a load and the parallel unsupported edge of a slab
F_{bst}	Bursting tensile force
F_{bt}	Tensile force due to ultimate loads in a bar or group of bars
F_h	Maximum horizontal ultimate load
F_v	Maximum vertical ultimate load
f	Stress
f_{bs}	Local bond stress
f_c	Constant stress
f_{cav}	Average compressive stress in the flexural compressive zone
f_{ci}	Concrete strength at (initial) transfer
f_{cj}	Stress in concrete at time of application (j) of an increment of stress
f_{co}	Stress in concrete at the level of the tendon due to initial prestress (prior to losses) and dead load
f_{cp}	Compressive stress at the centroidal axis due to prestress
f_{cu}	Characteristic cube strength of concrete
f_{pb}	Tensile stress in tendons at (beam) failure
f_{ps}	Effective prestress (in tendon)
f_{pt}	Stress due to prestress
f_{pu}	Characteristic strength of prestressing tendons

f_{s2}	Stress in reinforcement in other face
f_t	Maximum principal tensile stress
f_y	Characteristic yield strength of reinforcement
f_{yc}	Design strength of longitudinal steel in compression
f_{yL}	Characteristic strength of longitudinal reinforcement
f_{yv}	Characteristic strength of link reinforcement
G	Shear modulus
h	Overall depth (thickness) of section
h_{agg}	Maximum size of aggregate
h_e	Effective thickness
h_{ef}	Thickness of effective walls
h_f	Thickness of flange
h_{max}	Larger dimension of section
h_{min}	Smaller dimension of section
h_{wo}	Wall thickness at which the stress is determined
h_x	Overall depth of the cross-section in the plane of bending M_{iy}
h_y	Overall depth of the cross-section in the plane of bending M_{ix}
I	Second moment of area or equivalent second moment of area of a composite section
i	Radius of gyration; moment in time
j	Age at application of increment of stress
j_1	Age at first loading
j_m	Number of days over which hardening takes place at $T^\circ C$
j_∞	Age at the end of the life of the structure
K	A constant depending on the type of duct, or sheath, employed
K_1	A constant depending on the shape of the bending moment diagram
k	A constant (with appropriate subscripts)
k_c	Partial coefficient; depends on the composition of the concrete
k_e	Partial coefficient; depends on the effective thickness of the member
k_j	Partial coefficient; defines the development of the deferred deformation as a function of time
k_L	Partial coefficient; depends on environmental conditions
k_m	Partial coefficient; depends on the hardening (maturity) of the concrete at the time of loading
k_{m1}	Coefficient k_m for age at first loading
k_t	Coefficient; depends on the type of tendon
k_v	Coefficient in shear equation
k_{vt}	Coefficient in torsion equation
k_1	Coefficient; depends on the concrete bond across the shear plane
k_2	Coefficient; depends on the geometric ratio, p , of longitudinal reinforcement
L_s	Breadth of shear plane
l	Distance from face of support at the end of a cantilever, or effective span of a member
l_e	Effective height of a column or wall

l_{ex}	Effective height in respect of bending about the major axis
l_{ey}	Effective height in respect of bending about the minor axis
l_o	Clear height of column between end restraints
l_{sb}	Length of straight reinforcement beyond the intersection with the link
l_t	Transmission length
M	Ultimate bending moment due to ultimate loads
M_a	Increased moment in a column, or moment at position a in a beam
M_b	Moment at position b in a beam
M_c	Moment at position c in a beam
M_{cr}	Cracking moment at section being considered
M_{cs}	Hogging restraint moment (at an internal support of a continuous composite beam-and-slab section due to differential shrinkage)
M_g	Moment due to permanent service loads
M_{ix}	Initial moment about the major axis of a slender column due to ultimate loads
M_{iy}	Initial moment about the minor axis of a slender column due to ultimate loads
M_o	Decompression moment (moment necessary to produce zero stress)
M_q	Moment due to live service loads
M_{tx}	Total moment about the major axis of a slender column due to ultimate loads
M_{ty}	Total moment about the minor axis of a slender column due to ultimate loads
M_u	Ultimate resistance moment
M_{ux}	Maximum moment capacity in a short column assuming ultimate axial load and bending about the major axis only
M_{uy}	Maximum moment capacity in a short column assuming ultimate axial load and bending about the minor axis only
M_x, M_y	Moments about the major and minor axes of a short column due to ultimate loads
M_1	Smaller initial end moment due to ultimate loads (assumed to be negative if the column is bent in double curvature)
M_2	Larger initial end moment due to ultimate loads (assumed to be positive)
m	Number of steel layers
N	Ultimate axial load at section being considered
N_{bal}	Axial load corresponding to balanced condition
N_u	Ultimate axial load
N_{uz}	Axial loading capacity of a column, ignoring all bending
n	Number
n_w	Ultimate axial load per unit length of wall
P_f	Effective prestressing force after all losses
P_h	Horizontal component of the prestressing force after all losses
P_k	Basic load in a tendon
P_o	Prestressing force in the tendon at the jacking end (or at tangent point near jacking end)

P_x	Prestressing force at distance x from jack
q	Uniformly distributed load on a beam
r	Internal radius of bend
r_b	Radius of curvature of a beam at midspan or, for cantilevers, at support
r_{cs}	Radius of shrinkage curvature
r_{ps}	Radius of curvature of a tendon
r_x	Radius of curvature of a member at point x
S_L	Spacing of longitudinal reinforcement
s_v	Spacing of links along the member
T	Torsional moment due to ultimate loads or temperature (in degrees Celsius)
T_{cd}	Torsional capacity due to concrete only
T_u	Ultimate torsional capacity
t	Time
U_{ef}	Length of mean polygonal perimeter
U_o	Perimeter of A_o
u	Perimeter
u_s	Effective perimeter of tension reinforcement
V	Shear force
V_c	Ultimate shear resistance of concrete
V_{co}	Ultimate shear resistance of a section uncracked in flexure
V_{cr}	Ultimate shear resistance of a section cracked in flexure
V_x	Applied shear due to ultimate loads for the x-x-axis
V_y	Applied shear due to ultimate loads for the y-y-axis
V_u	Ultimate shear capacity
V_{ux}	Ultimate shear capacity of a section for the x-x-axis
V_{uy}	Ultimate shear capacity of a section for the y-y-axis
V_1	Longitudinal shear force per unit length due to ultimate load
v	Shear stress
v_c	Ultimate shear stress in concrete
v_{max}	Maximum value of shear stress in beams
v_1	Ultimate longitudinal shear stress per unit area of contact surface
v_t	Torsional shear stress
v_{tmin}	Minimum ultimate torsional shear stress for which reinforcement is required
v_{tu}	Ultimate torsional shear stress
v_{xy}	Resultant shear stress
W	Concentrated load on beam
x	Neutral axis depth; distance along prestressing cable
x_1	Smaller centre-line dimension of a link
y	Distance of the fibre considered in the plane of bending from the centroid of the concrete section
y_o	Half the side of the end block
y_{po}	Half the side of loaded area
y_1	Larger centre-line dimension of a link

z	Lever arm
α_e	Modular ratio
α_i	Angle of friction for concrete interfaces
α_i	Angle between the normal to the crack (or the normal to the axis of the principal moment) and the direction of the i th steel layer
α_n	Exponent related to the ratio of the ultimate axial load on the column and its axial loading capacity
α_r	Angle of rotation of a curved tendon over a specific length
α_i	Angle between the normal to the crack (or the normal to the axis of the design moment) and the direction of the tensile reinforcement A_i resisting that moment
β	Ratio of the sum of the support moments of a beam to its midspan moment
β_{cc}	Ratio of total creep to elastic deformation
β_s	Shear factor
$\gamma_{i1}, \gamma_{i2}, \gamma_{i3}$	Partial load factors
γ_{iL}	Product of γ_{i1} and γ_{i2} ; partial safety factor
γ_m	Partial safety factor for material strength
Δ_{cc}	Concrete creep deformation (strain)
Δ_{cct}	Concrete creep strain at time t
Δ_{cs}	Concrete shrinkage deformation (strain)
Δ_{cst}	Concrete shrinkage strain at time t
Δ_{tcm}	Change in stress in concrete due to creep at time t
Δ_p	Loss of prestress due to shrinkage and creep
Δ_ϵ	Reduction of strain due to stiffening effect of the concrete in the tension zone of a beam
δ_m	Degree of hardening at time of loading
ϵ	Strain
ϵ_{cs}	Free shrinkage strain
ϵ_{c1}	Creep strain in concrete at the level of the tendon at time of loading
ϵ_{c2}	Creep strain in concrete at the centroid of the section at time of loading
ϵ_{diff}	Combined differential creep and shrinkage strains
ϵ_m	Average strain
ϵ_s	Strain in tension reinforcement
ϵ_i	Strain at level being considered
η	Relaxation coefficient
θ_c	Inclination of web compression due to shear with the centre line of the beam
θ_s	Angle between the compression face and the tension reinforcement
θ_t	Inclination of compressive force due to torsion with the centre line of the relevant beam surface
θ_v	Inclination of link with the centre line of the beam
λ_w	Coefficient for walls; dependent upon dimensions and concrete used
μ	Coefficient of friction
ξ_s	Slab or beam depth factor
ρ	Geometric ratio of reinforcement, equal to $\frac{A_s}{bd}$

ρ_o	Coefficient; dependent upon the percentage of tension and compression steel in the section
Σ	Sum of
ϕ	Size (nominal diameter) of bar or tendon
ϕ	Creep coefficient
ϕ_1	Creep coefficient for prestressed construction
ϕ_2	Creep reduction coefficient for axially loaded and symmetrically reinforced concrete members
ϕ_3	Creep reduction coefficient for axially loaded and singly reinforced concrete members
ϕ_4	Creep reduction coefficient for symmetrically reinforced concrete members subjected to bending moment
ϕ_5	Creep reduction coefficient for singly reinforced concrete members subjected to bending moment
ϕ_{t1}	Creep coefficient at time t for load applied at time

2 DESIGN: GENERAL

2.1 Limit State Requirements (See also Part 1, Section 2)

The ultimate and serviceability limit states, as specified in 2.1.1 and 2.1.2, should be considered in the design so as to ensure adequate safety and serviceability. The usual approach will be to design on the basis of the limit state that is expected to be the most critical, and then to check that the remaining limit states will not be reached and that all other requirements, such as compliance with detailing rules, will be met. In certain cases it will not be necessary to check all these criteria if experience has proved that the design does meet the criteria.

Other factors, such as deflection, fatigue and durability, might need to be considered, as referred to in 2.1.3.

2.1.1 Ultimate limit states

The structure should be sufficiently strong to withstand the design loads, taking due account of the possibility of instability caused by translation, settlement, overturning or buckling, and of the effects of sway when appropriate. (See Part 1, Section 6.)

2.1.1.1 Rupture or instability: By assessing the structure under the design loads appropriate to the ultimate limit state, the designer should be able to ensure that premature collapse does not occur as a result of the rupture of one or more critical sections, of buckling caused by the elastic or plastic instability of a single member or a combination of members, or of instability due to translation, settlement,

or overturning of parts of the structure. In particular, it shall be shown that the structure cannot suffer progressive collapse as a consequence of localized damage caused by abnormal use or an accident (see 1.4.9 of Part 2).

The engineer responsible for the design shall ensure that both the bridge as a whole and its parts are robust and stable. Where it is practically and economically feasible, redundant members shall be provided in the configuration of the structure and in the connections between parts in order to prevent catastrophic collapse owing to abnormal actions or accidents. These redundant members should be designed to take effect before or during any failure of the primary structural system.

The effects of creep and shrinkage of concrete, temperature difference and differential settlement, as well as of secondary moments in continuous prestressed members, may be reduced by 50 per cent (or as agreed by the responsible bridge authority) at the ultimate limit state. However, this may only be done if the presence of the unreduced effects does not alter the nature of the inelastic behaviour by inducing local sectional failure instead of, and before, overall structural collapse in the assumed or calculated mode of failure. Moreover, these effects must have been included in the appropriate load combinations to check the stress limitations given in 2.1.2.3 for the serviceability limit state.

2.1.1.2 Ultimate limit state requirements for vibration: If the criterion for vibration serviceability requirements (2.1.2.2) is met, it can be deemed to satisfy this clause as well.

2.1.1.3 Seismic design: In the case of structures that may be subjected to earthquake action, the design of the configuration, mass distribution and stiffness of members, connections and movement joints should be directed at reducing the internal forces that would be generated by a structural response to such seismic forces. This is best achieved by designing ductile energy-absorbing structural systems and by avoiding large force concentrations on weak but stiff members.

2.1.1.4 Bridge sabotage countermeasures: The engineer shall consult with the responsible bridge authority on the need for sabotage countermeasures before proceeding with the conceptual design. Special care shall be taken in the design of parts of the structure that could be vulnerable to such attacks.

2.1.2 Serviceability limit states

The structure should be designed to withstand local damage, which would shorten its intended life or incur excessive maintenance costs. In particular, the calculated deflections, stresses and crack widths should not exceed those permitted in 2.1.2.1.

2.1.2.1 Cracking: Cracking of concrete should not affect the appearance or durability of the structure adversely. The engineer should satisfy himself that any cracking - especially due to combinations of stresses - will not be excessive, taking into account the requirements of the particular structure and the conditions of exposure. Unless a special investigation has been done or there are alternative means of ensuring durability, the following limits should be adopted.

- (a) *Reinforced concrete:* Design crack widths, when calculated in accordance with Equation 29 as given in 3.8.8.2, should not exceed the values given in Table 1 under the loading given in 2.2.3. Special attention should be given to the effects of combinations of stresses.
- (b) *Prestressed concrete:* Prestressed concrete structures, or elements under design loads appropriate to the serviceability limit state, may be divided into three separate categories according to the flexural tensile stress limitations. Most prestressed concrete structures are stressed in one direction only and are monolithically connected with non-prestressed elements. The following requirements refer only to the prestressed components. Stresses or crack widths in transverse directions and in non-prestressed components should comply with subsection (a) above. Reference should be made to the responsible bridge authority for its classification requirements.

The categories are as follows:

Class 1: Full prestress: no tensile stress permitted.

Class 2: Limited prestress: tensile stresses permitted, in accordance with Table 26, but no visible cracking.

Class 3: Partial prestress: tensile stresses and cracking permitted, but with design crack widths limited to the values given in Table 1. These limitations shall be deemed to be satisfied if the stress increments in both the prestressing steel and the additional reinforcement near the tensile face of the concrete do not exceed the stress limits defined in 4.3.2.4(a)(iii). These stress increments are the differences between the steel stresses prevailing when the concrete is cracked under the action of the design load and those prevailing when decompression of the concrete commences at the level of the reinforcement near the tensile face.

2.1.2.2 Vibration serviceability requirements: The dynamic effects of standard traffic loading on common types of bridge are deemed to be covered in the allowance for impact that is included in the nominal live load; the requirements of Part 2 need only be applied to footbridges or to the structural components of bridges whose prime function is to carry sidewalk (footway) live loading. Special investigations may be necessary in the case of long slender wind-sensitive structures such as suspension and cable-stayed bridges, especially with regard to the cables which are prone to develop resonance under wind excitation (see 3.8.4 of Part 2).

TABLE 1 Design crack widths

Environment	Examples	Design crack width (mm)
Moderate Concrete surfaces above ground level and fully sheltered against rain and sea-spray. Concrete surfaces permanently saturated by water that is non-aggressive towards concrete	Surfaces sheltered by the bridge deck or protected by waterproofing or permanent formwork. Interior surfaces of pedestrian subways, voided superstructures or cellular abutments or piers on which condensation is unlikely Concrete permanently under water	Not greater than 0,25
Severe Concrete surfaces exposed to driving rain or alternate wetting and drying	All external surfaces not sheltered or protected from rain Bridge deck soffits and internal surfaces on which condensation is likely. Buried parts of the structure or surfaces in contact with backfill.	Not greater than 0,20
Very severe Concrete surfaces directly affected by sea-spray or a salt-laden atmosphere.	Concrete parapets, walls and all exposed surfaces of superstructures and sub-structures in terrain near to the sea.	Not greater than 0,10
Extreme* Concrete surfaces exposed to abrasive action of sea-water or water that is aggressive towards concrete.	Parts of structures in contact with sea-water or water in marshy terrain	Not greater ** than 0,10

* Class 3 prestressed concrete should be used in extreme environments only with the permission of the responsible bridge authority.

** In very extreme cases it may be necessary to use methods of protection against corrosion in addition to applying crack width limitations.

2.1.2.3 Stress limitations: To obtain a design that will preclude unacceptable deformations and cracks, stresses in concrete and steel should be calculated by linear elastic analysis for load combinations 1 to 3 where any of the following apply:

- (a) for all prestressed concrete construction;
- (b) for all composite construction;
- (c) for reinforced concrete construction where the effects of creep and shrinkage of concrete and/or temperature difference and/or differential settlement are not considered at the ultimate limit state;
- (d) where a plastic method (see 2.4.2.1) or redistribution of moments (see 3.2.2 and 4.2.2) is used for the analysis of the structure at the ultimate limit state and
- (e) under global and local axial and bending effects where these are considered separately at the ultimate limit state.

For reinforced and prestressed concrete, the compressive and tensile stress limitations are summarized in Table 2 below.

The values given in Table 2 for a triangular compressive stress distribution in both reinforced and prestressed concrete sections of approximately uniform breadth are relatively high. This is permissible because the higher stresses affect only a portion of the section, thus reducing the probability of excessive deformation due to creep if the concrete has a limit of linearity lower than the assumed value of the characteristic stress. On the other hand, with uniform compressive stress and distribution the probability of excessive creep deformation is increased, thus necessitating lower values of stress.

Lower values of stress are given for prestressed concrete than for reinforced concrete because the whole concrete cross-section is normally in compression and therefore the probability of excessive creep is greater than in reinforced concrete.

In the range of cases where interpolation is allowed, the lower values may be preferred to avoid undue complexity in the analysis.

2.1.3 Other considerations

2.1.3.1 Deflections: The deflection of the structure, or any part of it, should not be sufficient to affect the appearance or efficiency of the structure adversely.

Although the appearance and function of concrete superstructures are normally not affected, deflection calculations may be required in the following circumstances:

- (a) where minimum specified clearances may be violated;
- (b) where drainage difficulties may result;
- (c) where the method of construction may require careful control of the profile, eg at discontinuities in serial construction, and where decks consist of abutting prestressed concrete beams, or
- (d) where, due to the configuration or dimensional proportions of the structure, even normal deflection may be relevant.

TABLE 2 Stress limitations for the serviceability limit state

Material	Type of stress under design loading	Type of construction	
		Reinforced concrete	Prestressed concrete
Concrete	Triangular or near-triangular compressive stress distribution in sections of approximately uniform breadth (eg due to bending)	$0,50 f_{cu}$	$0,40 f_{cu}$
	Intermediate between triangular and uniform compressive stress distribution (eg due to combined bending and axial loading in members varying from those with sections of approximately uniform breadth to those with flanges)	Interpolate between $0,50 f_{cu}$ and $0,38 f_{cu}$	Interpolate between $0,40 f_{cu}$ and $0,30 f_{cu}$
	Uniform or near-uniform compressive stress distribution (eg in all sections due to axial loading or in wide flanges of beams and box girders due to bending)	$0,38 f_{cu}$	$0,30 f_{cu}$
	Tension	Not applicable	Class 1: 0 Class 2: $0,45 \sqrt{f_{cu}}$ pretensioned $0,36 \sqrt{f_{cu}}$ post-tensioned Class 3: Not applicable
Reinforcement	Compression	$0,75 f_y$	Not applicable
	Tension	$0,75 f_y$	Deemed to be satisfied by 4.3.2.4 (a)(iii)
Prestressing tendons	Tension	Not applicable	Deemed to be satisfied by 4.8.1

Note: See 5.3.4 for limiting flexural stresses in joints for post-tensioned segmental construction.

2.1.3.2 Fatigue: The fatigue life of bridges, where it needs to be considered, should comply with the requirements of 2.7.

2.1.3.3 Durability: The recommendation in this Code regarding concrete cover to the reinforcement and acceptable crack widths (see 2.1.2.1), together with the recommendations for cement contents given in 5.8.1.5 of *Part II of SABS 0100, Code of Practice for the Structural Use of Concrete*, should meet the durability requirements of almost all structures, but the requirements of the responsible bridge authority should be given preference. Where very severe environments are encountered, however, additional precautions may be necessary and specialist literature should be consulted.

2.2 Actions

2.2.1 General

The characteristic values of actions (including forces, loads and imposed deformations) are given in Part 2, where they are described as nominal values because of the lack of statistical data.

Creep and shrinkage of concrete, differential settlement, prestrain and prestress (including secondary effects in statistically indeterminate structures) are indirect action effects associated with the nature of the structural material being used; where they occur, they should be regarded as long-term actions, except that in ultimate limit state calculations their effects may be reduced depending on the mode of failure. In such calculations the prestress should be considered as a resisting effect (bonded prestressing steel acts as high-strength reinforcement).

2.2.2 Ultimate limit states

To check the requirements of 2.1.1, load combinations 1 to 3 should be considered.

The values of the partial safety factor γ_{iL} for the individual loadings comprising combinations 1 to 3 are given in Part 2. The value of γ_{iL} for creep and shrinkage of concrete and for differential settlement should be taken as 1,0. The values of γ_{iL} for temperature for load combinations 2 and 3 are given in 4.5 and Table 17 of Part 2. In calculating the resistance of members to vertical shear and torsion, γ_{iL} for the prestressing force should be taken as 1,15 where it adversely affects the resistance and 0,87 in other cases. In calculating secondary effects in statistically indeterminate structures, γ_{iL} for the prestressing force may be taken as 1,0. The value of γ_{i3} should be taken as 1,10 except that where plastic methods (see 2.4.2.1) are used for the analysis of the structure, γ_{i3} should be taken as 1,15.

2.2.3 Serviceability limit states

For the limitations given in 2.1.2.1(a) for reinforced concrete, it would normally be necessary to consider only load combination 1. Type NA wheel loading need not be

considered except for cantilever slabs and the top flanges in beam-and-slab, voided slab and box-beam construction. It may, however, be necessary to consider the effects of wind loading of load combinations 2 and 3 on members sensitive to wind effects.

For the limitations given in 2.1.2.1(b) for prestressed concrete, bridges should be classified, as directed by the responsible bridge authority, as being in:

either (a) Class 1 under load combination 1 (as modified above) except that live loading due to NA loading may be reduced by 50 per cent and that due to NB loading to 18 units, and

(b) Class 2 or 3, whichever is applicable, for load combinations 2 and 3;

or Class 3 for load combinations 1, 2 and 3.

For the stress limitations given in 2.1.2.3, load combinations 1 to 3 should be considered.

The values of the partial safety factor γ_{IL} for the individual loadings comprising combinations 1 to 3 are given in Part 2. The value of γ_{IL} for creep and shrinkage of concrete and for the secondary effects or prestress and differential settlement in statically indeterminate structures should be taken as 1,0.

2.2.4 Deflection and deformation

The requirements of the responsible bridge authority should be complied with. Minimum specified clearances should be maintained under the action of load combination 1.

The long-term shrinkage effects, creep deflections and subsoil settlements should not be so severe as to impair the rideability and drainage characteristics of the structure or adversely affect the appearance or reliability of the structure or its parts.

The values of γ_{IL} for the individual loads and the value of γ_{I3} should be those appropriate to the serviceability limit state.

2.3 Materials

2.3.1 General

Characteristic strength of materials: Unless otherwise stated, the term 'characteristic strength' means that value of the cube strength of concrete, f_{cu} , the yield or proof strength of reinforcement, f_y , or the breaking load of a prestressing tendon, f_{pu} , below which not more than 5 % of all possible test results may be expected to fall. The characteristic strength of concrete is generally that of the 28-day strength of cubes sampled, made and testing in accordance with SABS Methods 861 and 863 and Sections 5.8.16 to 5.8.18 of SABS 0100, Part II, of 1980. The characteristic strengths of materials are given in 3.1.4 and 4.1.4.

Characteristic stress is the value of stress at the assumed limit of linearity on the stress-strain curve for the material.

In general, in analysing a structure to determine load effects, the material properties appropriate to the characteristic strength should be used, irrespective of the limit state being considered.

For the analysis of sections, the material properties to be used for the limit states are as follows:

- (a) *Serviceability limit state*: The characteristic stresses are assumed to be $0,75 f_y$ for reinforcement, $0,5 f_{cu}$ for concrete in compression and $0,56 \sqrt{f_{cu}}$ for tension in prestressed concrete.
- (b) *Ultimate limit state*: The characteristic strengths are given in 2.3.2. These values should be verified for local materials and the necessary adjustments made.

The appropriate γ_m values are given 2.3.3.

2.3.2 Property of materials

2.3.2.1 Concrete: In assessing the strength of sections at the ultimate limit state, the design stress-strain curve for concrete of normal density may be taken from Figure 1, using the value of γ_m for concrete given in 2.3.3.1.

The modulus of elasticity E_c to be used for elastic analysis should be the mean value of the secant modulus appropriate to the characteristic cube strength of the concrete at the age considered. In the absence of special investigations or tests, it should be obtained as follows:

- (a) *Analysis of structures*: To determine the effects of permanent and transient loading, use the appropriate value given in Table 3 except that, to determine the effects of imposed deformations and for the calculation of deflections, an appropriate value intermediate between that given in Table 3 and half that value as defined below can be used.

For the analysis of dynamic effects the value could be greater than that given in Table 3 depending on the nature of the action (see Table 34 Appendix 3.B) but, unless adequate data are available and a comprehensive analysis is carried out, the value should either not be increased or should be intermediate between the static and dynamic values.

- (b) *Analysis of sections*: To determine crack widths and stresses due to the effects of permanent and transient loading and imposed deformations, use an appropriate value intermediate between that given in Table 3 and half that value as defined below.

The appropriate intermediate value of the modulus of elasticity mentioned in (a) and (b) should reflect the age of the concrete and the proportion of permanent and short-term effects in the combination. This is based on the assumption that the effect of creep under long-term loading can be allowed for by using half the values given in Table 3 for the modulus of elasticity.

Where a more precise analysis is necessary, the effects of creep and shrinkage should be assessed more accurately. Information on creep and shrinkage, based on

CEB-FIP International Recommendations, as amended in May 1972, is given in Appendix 3.A. The effects arising from construction being completed in stages should be taken into account.

Further information on the elastic deformation of concrete is given in Appendix 3.B.

TABLE 3 Modulus of elasticity of concrete under short-term loading

Characteristic cube strength of concrete at the age considered (MPa)	Mean secant modulus of elasticity of concrete (GPa)
20	25
25	26
30	28
40	31
50	34
60	36

For concrete containing low-density aggregate having an air-dry density between 1 400 kg/m³ and 2 300 kg/m³, the values given in Table 3 should be multiplied by

$$\left[\frac{D_c}{2\,300} \right]^2$$

where D_c is the density of the concrete in kg/m³. Alternatively, the value of the elastic modulus for a specific characteristic strength may be obtained by doing tests. Note that the mean elastic modulus obtained from the results of such tests should bear a relation to the characteristic cube strength of the test specimens. Concrete made from certain aggregates (such as certain types of sandstone, limestone, and granite) could have modulus of elasticity significantly lower than the values given in Table 3.

For elastic analysis, Poisson's ratio may be taken as 0,2. The coefficient of thermal expansion may be taken as $12 \times 10^{-6}/^{\circ}\text{C}$ for concrete of normal density. For concrete containing low-density aggregate and for concrete containing limestone aggregate, the values can be as low as $7 \times 10^{-6}/^{\circ}\text{C}$ and $9 \times 10^{-6}/^{\circ}\text{C}$ respectively.

2.3.2.2 Reinforcing and prestressing steel: The design stress-strain curves may be taken as follows:

- (a) for reinforcing steel from Figure 2, using the value of γ_m given in 2.3.3;
- (b) for prestressing steel, from Figure 3 or 4, using the value of γ_m given in 2.3.3.

For reinforcing steel, the modulus of elasticity may be taken as 200 GPa for both short-term and long-term loading. For prestressing steel, the short-term and long-term moduli may be taken from Figure 1 or 2 as the appropriate tangent modulus at zero load.

TABLE 4 Values of γ_m applicable to characteristic stresses for the serviceability limit state

Material	Type of stress	Type of construction	
		Reinforced concrete	Prestressed concrete
Concrete	Triangular or near-triangular compressive stress distribution in sections of approximately uniform breadth (eg due to bending)	1,00	1,25
	Intermediate between triangular and uniform compressive stress distribution (eg due to combined bending and axial loading in members varying from those with sections of approximately uniform breadth to those with flanges)	Interpolate between 1,00 and 1,33	Interpolate between 1,25 and 1,67
	Uniform or near-uniform compressive stress distribution (eg in all sections due to axial loading or in wide flanges of beams and box girders due to bending)	1,33	1,67
	Tension	Not applicable	1,25 pretensioned 1,55 post-tensioned
Reinforcement	Compression } Tension }	1,00	Not applicable
Prestressing tendons	Tension	Not applicable	Not required

It should be established that the ductility of the steel would be adequate in the event of a redistribution of stress.

For reinforcing steel this requirement can be regarded as satisfied if the unit elongation of the steel at failure, measured over a length of ten diameters*, is not less than $\epsilon_{10} = 8\%$, or measured over a length of five diameters*, is not less than $\epsilon_5 = 12\%$.

For prestressing steel this requirement may be regarded as satisfied if the plastic strain capacity is at least three times the elastic strain at yield.

2.3.3 Values of γ_m

For the analysis of sections, the values of γ_m are summarized below.

2.3.3.1 Ultimate limit state: For both reinforced concrete and prestressed concrete, the values of γ_m applicable to the characteristic strengths (see 2.3.1) are 1,50 for concrete and 1,15 for reinforcement and prestressing tendons.

2.3.3.2 Serviceability limit state: The values of γ_m applicable to the characteristic stresses defined in 2.3.1 are given in Table 4 and have been allowed for in deriving the compressive and tensile stresses given in Tables 2, 24, 25 and 26. For explanatory notes see 2.1.2.3.

2.3.3.3 Fatigue: For reinforced concrete, the value of γ_m applicable to the stress-range limitations given in 2.7 for reinforcement is 1,00.

2.4 Analysis of Structures

2.4.1 General

The requirements of methods of analysis for determining the distribution of forces and the deformations that are to be used in ascertaining that the limit state criteria are satisfied, are described in Part 1, Section 10.

2.4.2 Ultimate limit states

2.4.2.1 General: Elastic methods may be used to determine the distribution of forces and deformations throughout the structure. Stiffness constants should be based on the section properties as used for the analysis of the structure at the serviceability limit state (see 2.4.3.1). However, plastic methods of analysis (eg plastic hinge methods for beams, or yield-line methods for slabs) may be used where such methods can be shown to model accurately the combined local and global effects of all actions due to combinations 1 to 3, as given in Part 2. The use of such methods should be agreed on with the responsible bridge authority.

*For deformed bars, the effective diameter = $\sqrt{\frac{4A_s}{\pi}}$

2.4.2.2 Methods of analysis and their requirements: The application of elastic methods of analysis, in association with the design actions for the ultimate limit state in general, leads to safe lower-bound solutions; these may be refined, and made less conservative, by referring to test results or specialist literature.

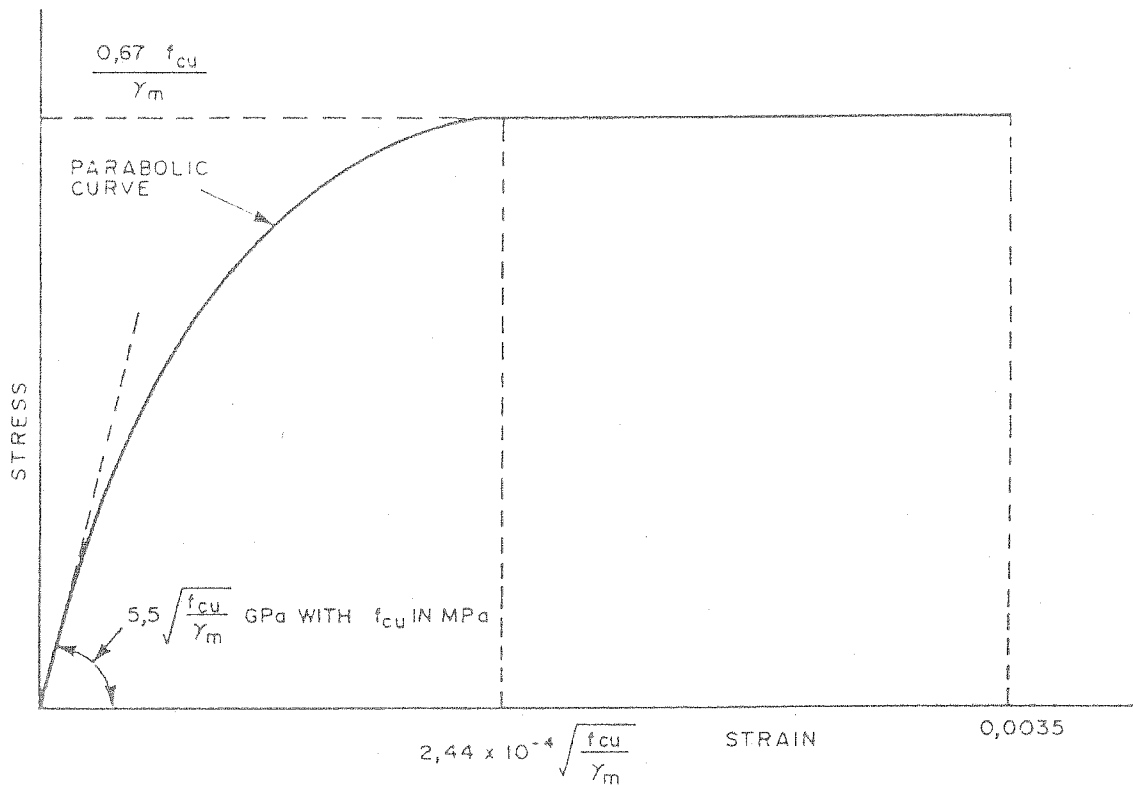


FIGURE 1
SHORT-TERM DESIGN STRESS-STRAIN CURVE FOR CONCRETE OF NORMAL DENSITY

* 0,67 takes into account the ratio between the characteristic cube strength and the bending strength in a flexural member.

The equation for the parabolic curve between $\epsilon = 0$ and

$$2,44 \times 10^{-4} \sqrt{\frac{f_{cu}}{\gamma_m}}$$

may be taken as:

$$f = \left[5\,500 \sqrt{\frac{f_{cu}}{\gamma_m}} \right] \epsilon - \left[\frac{(5\,500)^2}{2,68} \right] \epsilon^2$$

where f is the stress and
 ϵ is the strain.

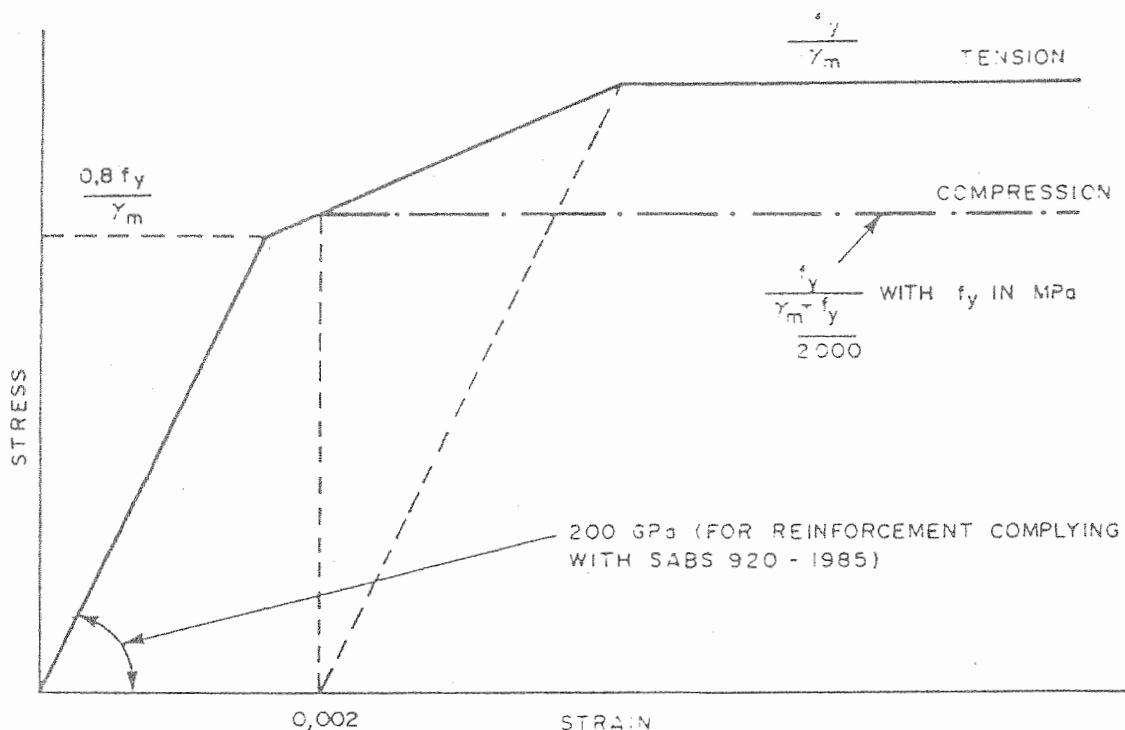


FIGURE 2

SHORT-TERM DESIGN STRESS-STRAIN CURVE FOR STEEL REINFORCEMENT

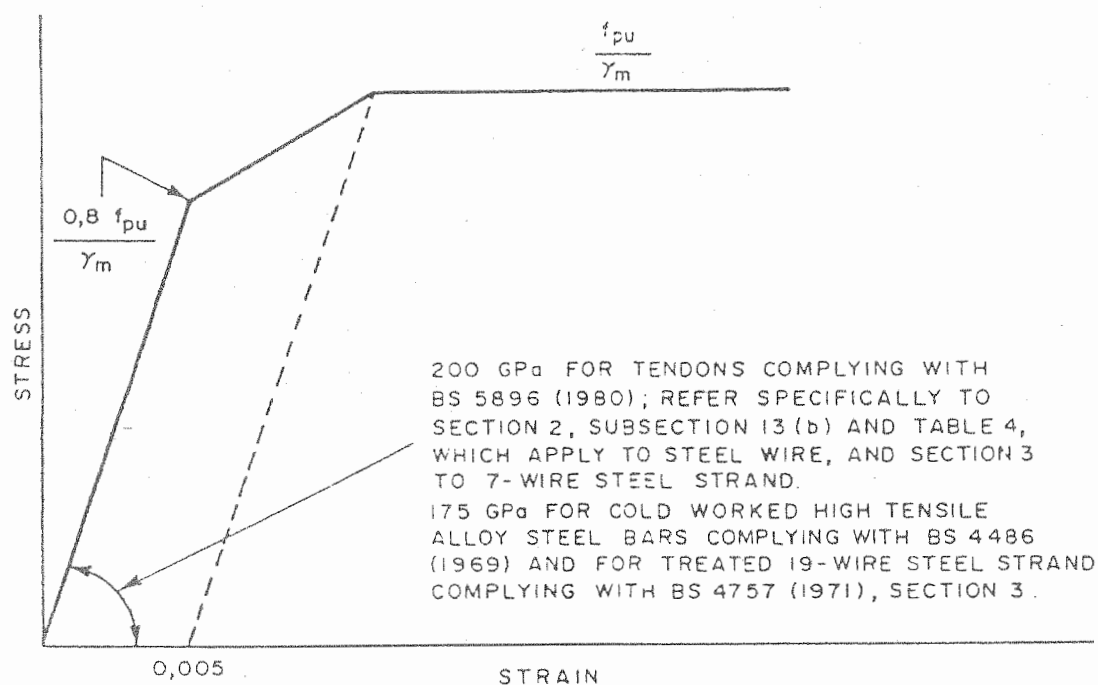


FIGURE 3

SHORT-TERM DESIGN STRESS-STRAIN CURVE FOR NORMAL AND LOW-RELAXATION STEEL PRODUCTS

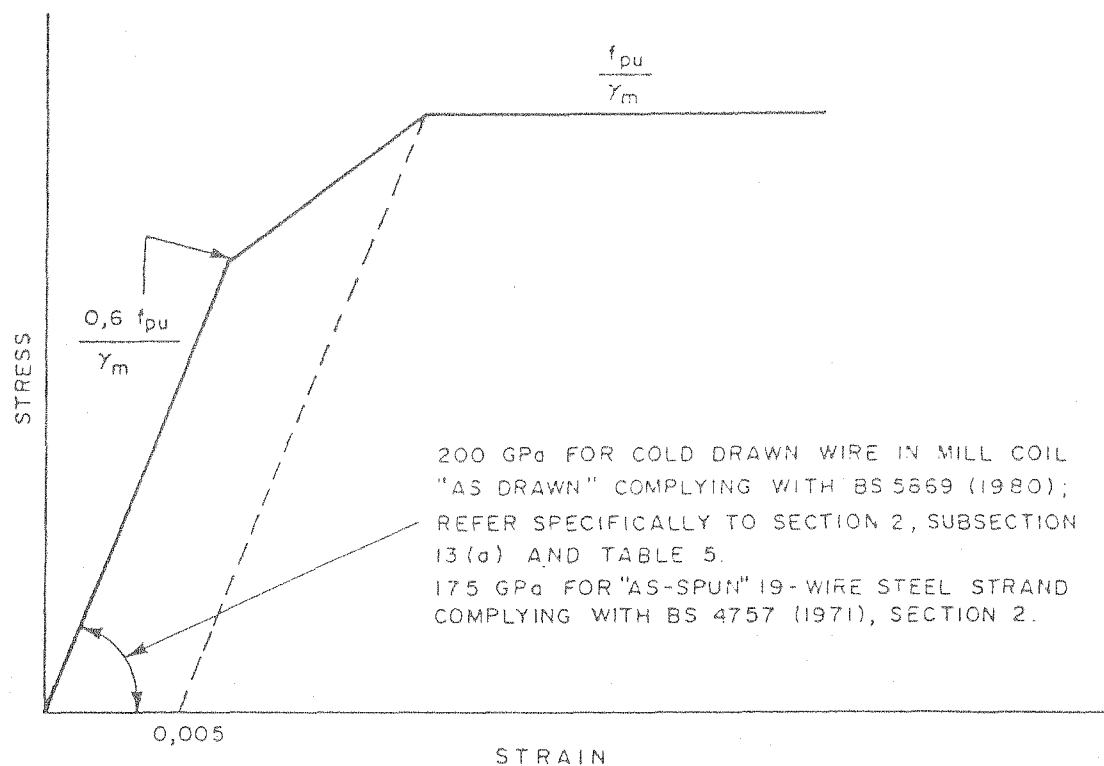


FIGURE 4
SHORT-TERM DESIGN STRESS-STRAIN CURVE FOR "AS-DRAWN" STEEL WIRE
AND "AS SPUN" STRAND

When dealing with local effects, elastic methods may be applied to derive the in-plane forces and moments due to in-plane and out-of-plane loading; alternatively yield-line methods may be used for local wheel loads. The details of the section should then be designed according to Sections 3, 4 and 5, as appropriate.

2.4.3 Serviceability limit states

2.4.3.1 General: Elastic methods of analysis should be used to determine internal forces and deformations. The flexural stiffness constants (second moment of area) for sections of discrete members or unit widths of slab elements may be based on any one of the following:

- (a) *Concrete section:* The entire cross-section of the member ignoring the presence of reinforcement.
- (b) *Gross transformed section:* The entire cross-section of the member, including the reinforcement transformed on the basis of modular ratio.
- (c) *Net transformed section:* The area of the cross-section that is in compression, together with the tensile reinforcement transformed on the basis of modular ratio.

A consistent approach should be used to the different behaviours of the various parts of the structure. Axial, torsional and shearing stiffness constants, when required by

the method of analysis, may be based on the concrete section, in which case they should be used with (a) or (b).

The values of the elasticity and shear moduli should be appropriate to the characteristic strength of the concrete. For torsional stiffness refer to 3.3.4.1 and 4.3.5.1.

2.4.3.2 Methods of analysis and their requirements: Ideally the method of analysis should take into account all the significant aspects of behaviour of a structure that govern its response to actions and imposed deformations.

In certain cases, shear lag effects may be of importance, eg in box sections and beam-and-slab decks with flanges whose breadths are considerable in relation to their effective lengths. An example is a cable-stayed superstructure with closely spaced cable supports. In such cases the specialist literature should be consulted.

The effective breadths of T, L and box beams, if determined as specified in 3.3.1.2, can be assumed to allow for shear lag effects.

2.5 Analysis of Sections

2.5.1 Design action effects

The analysis for any particular section should use the effects of the most severe combination of actions or forces derived from the analysis of the structure. For combinations not specifically covered in this Part, authoritative technical literature should be consulted.

2.5.2 Ultimate limit states

The strength of critical sections should satisfy the requirements of 2.1.1 and should be assessed in accordance with Sections 3, 4 or 5. Except for those special cases described in 2.4.3.2, in-plane shear flexibility in concrete flanges (shear lag effects) may be ignored.

2.5.3 Serviceability limit states

The material properties for both steel and concrete should be those given in 2.3.2 and 2.3.3. An elastic analysis should be carried out to ensure compliance with the requirements of 2.1.2. Shear lag effects should be allowed for; in the absence of any more accurate determinations (eg finite elements analyses), this may be done by taking an effective breadth of flange as defined in 3.3.1.2.

2.6 Deflection

Deflection should be calculated from the most unfavourable distributions of loading for the member (or slab), derived from an elastic analysis of the structure. The material properties, stiffness constants and calculation of deflections may be based on the information given in 2.3.2.1 or in Appendix 3.A.

2.7 Fatigue

The effect of repeated live loading on the fatigue strength of a bridge should be considered with respect to reinforcing bars that have been joined by mechanical couplers or subjected to welding and, in extreme cases, with respect to unwelded reinforcement and prestressing tendons if they do not comply with the limitations given below. Compliance requirements for mechanical couplers shall be approved by the responsible bridge authority.

Welding may be used to connect bars that will be subjected to fatigue loading, provided the following requirements are met:

- (a) The connections must comply with levels of workmanship approved by the responsible bridge authority.
- (b) The welded bar must not be part of a deck slab spanning between longitudinal and/or transverse members and subjected to the effect of concentrated wheel loads in a traffic lane.
- (c) The detail must have an acceptable fatigue life determined in accordance with procedures approved by the responsible bridge authority.
- (d) Lap welding must not be used.

Unless adequate test data are available and detailed fatigue calculations are performed, using principles approved by the responsible bridge authority, the following limitations should be applied to the stress range under combinations 1 to 3 for the serviceability limit state in unwelded reinforcement and prestressing tendons (see Tables 6 and 20 to 23):

- (i) Unwelded reinforcement complying with SABS 920 (1985):

Type A (grade 250)	250 MPa
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Types B and C (grade 450)	300 MPa
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- (ii) Bonded prestressing tendons in the case of class 3 prestressed concrete in which the minimum stress does not exceed 0,50 times the ultimate strength:

Tendons, not deformed	200 MPa
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Tendons, deformed	150 MPa
-------------------	---------

Strands	200 MPa
---------	---------

High-strength bars	200 MPa
--------------------	---------

The above stress limitations are based on the assumption that the probable accumulated fatigue damage caused by repeated live loading, summed by using the Palmgren-Miner concept, does not exceed that caused by 2×10^5 cycles of the maximum stress range specified above. In bridges on routes with large volumes of heavy traffic the fatigue damage may exceed this value in its useful lifetime. In such cases the above limitations to the stress ranges for the various types of reinforcement and tendons should be reduced linearly in accordance with log-log plots of

stress ranges versus n (the number of cycles) through the above stress limitations at $n = 2 \times 10^5$ cycles and 60 % thereof at $n = 2 \times 10^6$ cycles. Reference should be made to Part 10 of BS 5400 or other specialist literature if these conditions are not satisfied.

When interpreting the test data used to support an increase in the above values, the minimum stress, the radius of bends, the type of end anchorage or couples and the possibility of corrosion should be taken into account. In the case of stress reversal, the stress range shall be the algebraic difference between the extreme values.

2.8 Combined Global and Local Effects

2.8.1 General

In addition to designing individual primary and secondary elements to resist loading applied directly to them, it is necessary to consider what loading combination will produce the most adverse effects due to global and local loading where these coexist in an element.

2.8.2 Analysis of structures

Analysis of the structure may be accomplished either by a single overall analysis (eg using finite elements) or by separate analyses for global and local effects. In the latter case the forces and moments acting on the element as a result of global and local effects should be combined when appropriate.

2.8.3 Analysis of sections

Section analysis for the combined global and local effects should be carried out in accordance with 2.5 to satisfy the requirements of 2.1.

- (a) *Ultimate limit state*: The resistance of the section to direct and flexural effects should be derived from the direct strain due to global effects combined with the flexural strain due to local effects. However, in the case of a deck slab, the resistance to combined global and local effects is deemed to be satisfactory if each of these effects has been considered separately, provided that the local effects are at any time confined to a relatively small area of the slab and that the slab as a whole has a reserve of resistance to the global effects.
- (b) *Serviceability limit state*: Generally, in checking flexural and direct stress limitations, coexistent stresses acting in the same direction should be added algebraically.

For reinforced concrete elements, the total allowable crack width resulting from combined global and local effects should be determined in accordance with 3.8.8.2.

For prestressed concrete elements, coexistent stresses, acting in the direction of prestress, may be added algebraically in checking the stress limitations.

3 DESIGN AND DETAILING: REINFORCED CONCRETE

3.1 General

3.1.1 Introduction

This section gives methods of analysis and design that will in general ensure that, for reinforced concrete structures, the requirements set out in 2.1.1 and 2.1.2 are met. Other methods may be used provided they can be shown to be satisfactory for the type of structure or member considered. In certain cases the assumptions made in this section may be inappropriate and the engineer should adopt a more suitable method, having regard to the nature of the structure in question.

3.1.2 Limit state design of reinforced concrete

3.1.2.1 Basis of design: Section 3 follows the limit state philosophy set out in Sections 2 and 6 of Part 1 and Section 2 of this part. As it is not possible to assume that a particular limit will always be the critical one, design methods are given for both the ultimate and serviceability limit states.

In general, the design of reinforced concrete members is governed by the ultimate limit state, but the limitations on crack width and, where applicable, stresses at the serviceability limit state given in 2.1.2.3 should also be met.

Where a plastic method or redistribution of moments is used for analysis of the structure at the ultimate limit state, or where critical parts of the structure are subjected to the "extreme" or "very severe" category of exposure, the design is likely to be governed by the serviceability limit state of cracking.

3.1.2.2 Durability: In 3.8.2, guidance is given on the minimum cover to reinforcement that should be provided to ensure durability.

3.1.2.3 Other limit states and considerations: Section 3 does not make recommendations concerning vibration or other limit states. For these and other considerations, reference should be made to 2.1.2.2 and 2.1.3.

3.1.3 Actions and resistance

In Section 3 the design action effects or design load effects, including the effects of imposed deformations and restraints (see 6.3 of Part 1) for the ultimate and serviceability limit state, are referred to as "ultimate actions" or "ultimate loads" and "service actions" or "service loads" respectively.

The values of the "ultimate actions" or "ultimate loads" and "service actions" or "service loads" to be used in design are derived from Part 2 and 2.2 of this Part.

In Section 3, when analysing sections, the terms "strength" "resistance" and "capacity" are used to describe the design strength of the section for the ultimate limit state (see 6.4 of Part1).

3.1.4 Strength of materials

3.1.4.1 Definition of strengths: In Section 3 the design strengths of materials for the ultimate limit state are expressed in all the tables and equations in terms of the characteristic strength of the materials. *Unless specifically stated otherwise, all equations, figures and tables include allowances for γ_m , the partial safety factor for material strength.*

3.1.4.2 Characteristic strength of concrete: The characteristic 28-day cube strengths of concrete that may be specified for reinforced concrete as given in Table 5 together with their related characteristic cube strengths at other ages. The values given in Table 5 do not include any allowance for γ_m .

Design should be based on the 28-day characteristic cube strength and, if appropriate for any specific loading combination, on the characteristic strength given in Table 5 for the age at which loading takes place.

TABLE 5 Strength of concrete (ordinary Portland Cement)

Grade	Characteristic cube strength (in MPa) f_{cu} at 28 days	Characteristic cube strength (in MPa) at the age of				
		7 days	2 mths*	3 mths*	6 mths*	1 year*
20	20,0	13,5	22,0	23,0	24,0	25,0
25	25,0	16,5	27,5	29,0	30,0	31,0
30	30,0	19,0	32,0	34,0	35,0	36,0
40	40,0	27,0	42,5	44,0	46,0	48,0
50	50,0	35,0	52,5	54,0	56,0	58,0

* Increased strengths at these ages should be used only if it has been demonstrated to the satisfaction of the engineer that the materials to be used are capable of producing these higher strengths.

3.1.4.3 Characteristic strength of reinforcement: The design should be based on the appropriate characteristic strength of reinforcement given in Table 6, or a lower value if necessary to reduce deflection or control cracking. The values given in Table 6 do not include any allowance for γ_m .

TABLE 6 Strength of reinforcement, complying with SABS 920 - 1985

Designation	Nominal sizes (mm)	Specified 5% characteristic yield strength f_y (MPa)
Type A Hot-rolled mild steel bars of plain round cross-section	6, 8, 10, 12, 16, 20, 25, 32, 40	250
Type B Hot-rolled mild steel deformed bars, cold worked to increase the yield stress	As above	450
Type C Hot-rolled high-yield-stress steel deformed bars	As above	450

3.2 Structures and Structural Frames

3.2.1 Analysis of structures

Structures should be analysed in accordance with the recommendations of 2.4.

3.2.2 Redistribution of moments

Redistribution of moments obtained by rigorous elastic analysis under the ultimate limit state may be carried out, provided the following conditions are met:

- (a) Checks must be made to ensure that there is adequate rotation capacity at sections where moments are reduced, making reference to appropriate test data.

In the absence of a special investigation, the plastic rotation capacity may be taken as the lesser of:

$$(i) \quad 0,008 + 0,035 \left[0,5 - \frac{d_c}{d_s} \right]$$

or

$$(ii) \quad \frac{0,6 \phi}{d - d_c}$$

but not less than 0,0 or more than 0,015

where d_c is the calculated depth of concrete in compression at the ultimate limit state

d_o is the effective depth for a solid slab or rectangular beam, otherwise the overall depth of the compression flange

ϕ is the diameter of the smallest tensile reinforcing bar

d is the effective depth to tension reinforcement.

- (b) Proper account must be taken of the changes in transverse moments, transverse deflections and transverse shears that will be brought about by the redistribution of longitudinal moments. This is done by means of a special investigation based on a non-linear analysis.
- (c) The shears and reactions used in design must be those calculated either before or after redistribution, whichever are the greater.
- (d) Where the depth of the members or elements considered is greater than 1,0 m, the degree of distribution should be proportionally reduced, with no redistribution for depths greater than 1,5 m unless appropriate test data are available.

3.3 Beams

3.3.1 General

3.3.1.1 Effective span: The effective span of a simply supported member should be taken as the smaller of:

- (a) the distance between the centres of bearings or other supports;

or

- (b) the clear distance between supports plus the effective depth.

The effective span of a member framing into supporting members should be taken as the distance between the shear centres of the supporting members.

The effective span of a continuous member should be taken as the distance between the centres of supports, except in the case of beams on broad supports when the effect of support breadth should be included in the analysis.

The effective length of a cantilever should be taken as its length from the face of the support, plus half its effective depth or its length to the centre of the support (whichever is the smaller). Where the cantilever is an extension of a continuous beam, the length to the centre of the support should be used.

3.3.1.2 Effective breadth of flanged beams: In analysing structures and sections at the ultimate state, the effective flange breadth for a T, L or box-beam may be taken as the actual breadth of the flange except for those cases referred to in 2.4.3.2. In analysing sections at the serviceability limit state, shear lag effects should be allowed for. However, in the absence of any more accurate determination, the effective flange breadth of a continuous beam may be taken as the breadth of the web

plus one-tenth of the distance between points of zero moment in the beam (or the actual breadth of the outstand of the flange, if this is less) on each side of the web. For this purpose, in the absence of any more accurate determination, the points of zero moment in a continuous beam may be taken to be at a distance of 0,15 times the effective span from the support.

3.3.1.3 Slenderness limits for beams: To ensure lateral stability, a simply supported or continuous beam should be so proportioned that the clear distance between lateral restraints does not exceed $60b_c$ or $250b_c^2/d$, whichever is the smaller,

where b_c is the breadth of the compression face of the beam midway between restraints

where d is the effective depth measured from the centroid of the tension steel to the extreme compression fibre

For cantilevers with lateral restraint provided only at the support, the clear distance from the end of the cantilever to the face of the support should not exceed $25b_c$ or $100b_c^2/d$, whichever is the smaller.

3.3.2 Resistance moment of beams

3.3.2.1 Analysis of sections: When assessing the strength of sections at the ultimate limit state, the following assumptions should be used:

- (a) Plane sections are assumed to remain plane when the strain distribution in the concrete in compression and the strains in the reinforcement, whether in tension or compression is being derived.
- (b) The stresses in the concrete in compression can be derived from the stress-strain curve in Figure 1 with $\gamma_m = 1,5$.
- (c) The tensile strength of the concrete can be disregarded.
- (d) The stresses in the reinforcement can be derived from the stress-strain curve in Figure 2 with $\gamma_m = 1,15$.

Alternative procedures may be adopted, namely

- either* (i) The ultimate moment of resistance is determined by taking the maximum concrete strain at the outermost compression fibre at failure as 0,0035. If the ultimate moment of resistance is found to be less than 1,15 times the required value, the section should be proportioned such that the strain at the centroid of the tensile reinforcement is not less than:

$$0,002 + \frac{f_y}{E_s \gamma_m}$$

where E_s is the modulus of elasticity of the steel.

In the case of rectangular sections and in flanged, ribbed and voided sections where the neutral axis lies within the flange, assumption (b) may be adapted so that the compressive stress is taken to be equal to $0,4f_{cu}$ over the whole compression zone.

or

- (ii) the strains in the concrete and the reinforcement caused by the application of ultimate loads are calculated. The calculated strain at the outermost compression fibre of the concrete should not exceed 0,0035. In addition, the section should be proportioned such that the strain at the centroid of the tensile reinforcement is not less than

$$0,002 + \frac{f_y}{E_s \gamma_m}$$

except where the requirements for the calculated strain in the concrete, due to the application of 1.15 times the ultimate loads, can be satisfied.

In the analysis of a cross-section of a beam that has to resist a small axial thrust, the effect of the ultimate force may be ignored if the force does not exceed $0,1f_{cu}$ times the cross-sectional area.

Note: Deep beams: Deep beams are defined as having a $l:h$ ratio of less than approximately 3 for continuous beams or 2 for simply supported beams (where l is the effective span). The assumption that plane sections remain plane when the beam is subjected to bending does not hold for deep beams, so that the formulae developed for beams in this Code are not accurate. The transition to a deep beam for decreasing span-to-depth ratios is gradual and specialist literature should be consulted for methods of calculating the non-linear longitudinal and shear strain distributions, and for adequate detailing of the reinforcement. The distribution of the applied loads is significant, especially with large concentrated loads near supports.

3.3.2.2 Design charts: The design charts that form Parts 2 and 3 of the *BS Code of Practice - CP 110 (1972)* include those based on Figures 1 and 2 as well as on various assumptions that do not fully comply with the requirements of this Code. The charts may nevertheless be used for the design of beams reinforced for tension only or for tension and compression, but the calculations must be checked for compliance with the additional requirements of this Code for minimum strain in the tensile reinforcement.

3.3.2.3 Design formulae: Provided that the redistribution of the elastic ultimate moments has amounted to less than 10 %, the formulae given below may be used to calculate the ultimate moment of resistance of a solid slab or rectangular beam, or of a flanged beam, or of a ribbed or voided slab when the neutral axis lies within the flange.

For sections without compression reinforcement, the ultimate moment of resistance may be taken as the small of the values obtained from Equations 1 and 2. Equations 3 and 4 may be used for sections with compression reinforcement.

A rectangular stress block of maximum depth $0,5d$ and a uniform compressive stress of $0,4f_{cu}$ has been assumed.

$$M_u = (0,87f_y)A_s z \dots\dots\dots (1)$$

$$M_u = 0,15f_{cu}bd^2 \dots\dots\dots (2)$$

$$M_u = 0,15f_{cu}bd^2 + 0,72f_yA'_s(d - d') \dots\dots\dots (3)$$

$$(0,87f_y)A_s = 0,2f_{cu}bd + 0,72f_yA'_s \dots\dots\dots (4)$$

where M_u is the ultimate resistance moment

f_y is the characteristic yield strength of the reinforcement

A_s is the cross-sectional area of tension reinforcement

z is the lever arm

f_{cu} is the characteristic cube strength of concrete

b is the breadth of the section

d is the effective depth to the tension reinforcement

A'_s is the cross-sectional area of compression reinforcement

d' is the depth to the compression reinforcement.

When d'/d is greater than 0,2, Equation 3 should not be used and the moment of resistance should be calculated as described in 3.3.2.1.

The term $0,72f_y$ in Equations 3 and 4 is a simplification of the expression

$$\frac{f_y}{\gamma_m + \frac{f_y}{2\,000}}$$

shown in Figure 2.

The lever arm, z , in Equation 1, may be calculated from the equation:

$$z = \left[1 - \frac{1,1f_yA_s}{f_{cu}bd} \right] d \dots\dots\dots (5)$$

The value of z should not be taken as greater than $0,95d$.

The ultimate moment of resistance of a flanged beam may be taken as the lesser of the values given by Equations 6 and 7, where h_f is the thickness of the flange:

$$M_u = (0,87f_y)A_s(d - h_f/2) \dots\dots\dots (6)$$

$$M_u = 0,4f_{cu}bh_f(d - h_f/2) \dots\dots\dots (7)$$

Where it is necessary for the moment of resistance to exceed the value given in Equation 7, the section should be analysed as described in 3.3.2.1.

3.3.3 Shear resistance of beams

3.3.3.1 Shear stresses and shear reinforcement in beams (for deep beams see note in 3.3.2.1): The shear stress, v , at any cross-section in a beam should be calculated from:

$$v = \frac{V}{bd} \dots\dots\dots (8)$$

where V is the shear force due to ultimate loads

b is the minimum breadth of the section (within the effective depth d) which, for a flanged beam, should be taken as the minimum rib breadth. If the web or the rib contains bars with diameters ϕ_1 greater than $b/8$, b should be reduced by $\frac{1}{2}\Sigma\phi_1$

d is the effective depth to tension reinforcement.

Shear reinforcement in the form of links, or links combined with bent-up bars, should be provided in accordance with the following:

$$A_{sv} \geq \frac{bs_v(v - k_v\xi_s v_c)}{0,87f_{yv}(\cot \theta_c + \cot \theta_v) \sin \theta_v} \dots\dots\dots (9a)$$

provided that, to satisfy minimum reinforcement requirements, it is assumed that

$$v - k_v\xi_s v_c \leq 0,4 \text{ MPa} \dots\dots\dots (9b)$$

where v is expressed in MPa

A_{sv} is the sum of the cross-sectional areas of the legs of a link or a composite link

s_v is the spacing of the links along the beam

k_v is a coefficient that varies linearly from 1,0 for $v \leq \xi_s v_c$ to 0,0 for $v \geq 3\xi_s v_c$

v_c the ultimate shear stress, and the depth factor ξ_s are given in Tables 7 and 8 respectively

f_{yv} is the characteristic strength of the link reinforcement, which should not be taken as more than 450MPa

θ_c is the assumed inclination of web compression due to shear to the centre line of the beam, eg for a horizontal prismatic beam θ_c is the inclination to the horizontal. θ_c should be prudently chosen within limits so that $\frac{3}{5} \leq \cot \theta \leq \frac{5}{3}$ to ensure reasonable control of cracking under service conditions. θ_c may vary along the beam.

θ_v is the inclination of the link to the centre line of the beam, towards the direction of the principal tensile stresses in the web. Inclined links or stirrups are used in the range ($45^\circ \leq \theta_v \leq 90^\circ$).

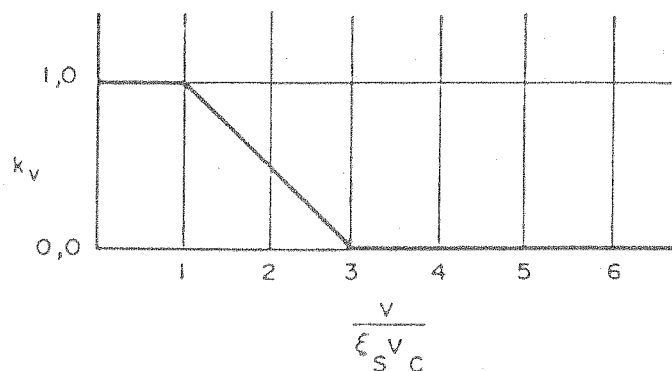


FIGURE 5
VALUES OF k_v

When links are used for shear reinforcement, the spacing of the legs in the direction of the span should not exceed $0,75d$ nor should the lateral spacing of the individual legs of the links exceed $0,75d$.

For sections within a distance $a_v < 2d$ from the face of a support, the front edge of a rigid bearing or the centre line of a flexible bearing (see Figures 6 and 7) and enhancement of shear strength, due to the local concentration of compressive stresses induced by the support reaction, may be allowed in determining the shear reinforcement.

This enhancement should take the form of an increase in the value of the term $k_v \xi_s v_c$ in Equations 9a and 9d to $\frac{2d}{a_v} \times k_v \xi_s v_c$ but should not exceed $0,75 \sqrt{f_{cu}}$ or $4,75 \text{ MPa}$, whichever is the smaller.

Where this enhancement is used, the main reinforcement at the section considered should continue to the support and be provided with an anchorage equivalent to 20 times the bar size so that it can adequately resist the horizontal component of the above-mentioned inclined compressive forces.

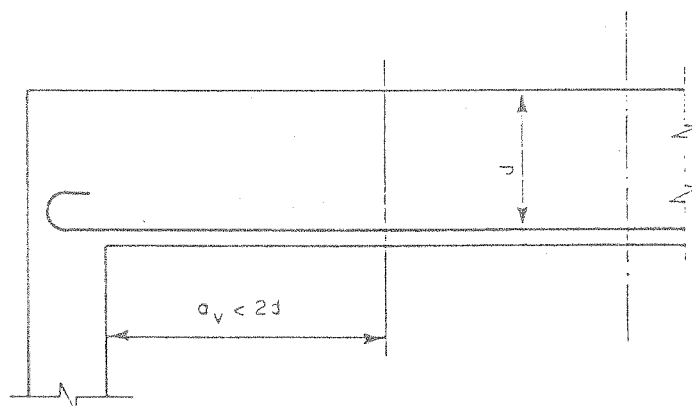


FIGURE 6
DEFINITION OF DIMENSION a_v AT A RIGID BEARING

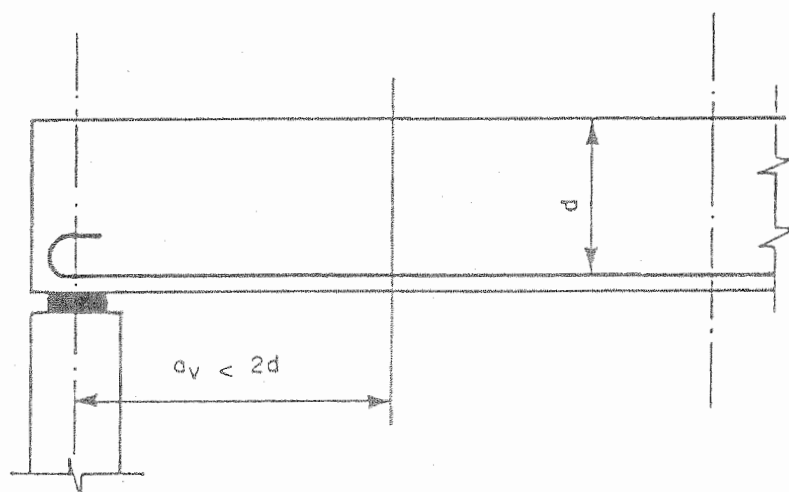


FIGURE 7
DEFINITION OF DIMENSION a_v AT FLEXIBLE BEARING

Whatever type of shear reinforcement is provided, in no case should v , calculated at all positions including the face of a support, exceed the value

$$v_{\max} \sin 2\theta_c$$

where $v_{\max} = 0,75\sqrt{f_{cu}}$ but is not greater than 4,75 MPa.

TABLE 7 Ultimate shear stress, v_c , in beams

$\frac{100A_s}{bd}$	Stress (in MPa) at concrete grades			
	20	25	30	40 or more
$\leq 0,15$	0,31	0,34	0,36	0,39
0,25	0,37	0,40	0,42	0,47
0,50	0,47	0,50	0,53	0,59
1,00	0,59	0,63	0,67	0,74
2,00	0,74	0,80	0,85	0,93
$\geq 3,00$	0,85	0,91	0,97	1,06

Table 7 is derived from the following relationship:

$$v_c = \frac{0,27}{\gamma_m} \left[\frac{100A_s f_{cu}}{bd} \right]^{\frac{1}{3}}$$

where γ_m is taken as 1,25 and f_{cu} should not exceed 40 MPa for the higher grades of concrete.

TABLE 8 Values of depth factor ξ_s

Effective depth, d	Depth factor, ξ_s
2 000 or more	0,70
1 500	0,75
1 000	0,85
500	1,00
400	1,05
300	1,15
250	1,20
200	1,25
150	1,35
≤ 100	1,50

Table 8 is derived from the following relationship:

$$\xi_s = \left(\frac{500}{d} \right)^{\frac{1}{4}} \text{ or } 0,70, \text{ whichever is the greater.}$$

The term A_s in Table 7 is that area of longitudinal tension reinforcement that continues beyond the section being considered for at least a distance equal to the effective depth, except at supports where the full area of tension reinforcement may be used provided the requirements of 3.8.7 are met. Near points of contraflexure where both top and bottom reinforcement are present, a moment associated with the shear force being considered will have been calculated; the value taken for A_s will depend on the sign of this moment which determines which reinforcement is acting as the tensile reinforcement. The shear forces associated with both the hogging bending moments and the sagging bending moments should be considered.

Where links are used, the area of additional effectively anchored longitudinal reinforcement ΔA_s provided in the tensile zone* (additional to any reinforcement required to resist bending and axial tensile forces) should be

$$\Delta A_s \geq \frac{V(\cot \theta_c - \cot \theta_v)}{2(0,87f_{yL})} \dots\dots\dots (9c)$$

where V, θ_c and θ_v are as defined above

f_{yL} is the characteristic strength of the longitudinal reinforcement which should not be taken as greater than 450 MPa.

Where links at right angles to the centre line of the beam are used (which will for horizontal beams be vertical, ie $\theta_v = 90^\circ$), together with an assumed inclination of compressive forces $\theta_c = 45^\circ$, equations 9a, 9b and 9c reduce to:

$$\text{Shear reinforcement } A_{sv} \geq \frac{bs_v(v - k_v \xi_s v_c)}{0,87f_{yv}} \dots\dots\dots (9d)$$

*see also 3.8.6.2 and 3.8.7.

In order to satisfy minimum reinforcement requirements, it is assumed that

$$v - k_v \xi_s v_c \leq 0,4 \text{ MPa} \dots\dots\dots (9e)$$

and

$$\Delta A_s \geq \frac{V}{2(0,87f_{yL})} \dots\dots\dots (9f)$$

Equations 9a, 9b and 9c should be used for special cases, in particular where combined shear and torsion are involved.

Where links combined with bent-up bars are used for shear reinforcement, not more than 50 % of the shear force $(v - k_v \xi_s v_c)bd$ should be resisted by the bent-up bars. These bars should be assumed to form the tension members of one or more single systems of lattice girders in which the concrete forms the compression members. The maximum stress in any bar should be taken as $0,87f_{yL}$. The shear resistance at any vertical section should be taken as the sum of the vertical components of the tension and compression forces at the section, which is equivalent to using Equations 9a and 9b with appropriate inclination values. Bars should be checked for anchorage (see 3.8.6.3) and bearing stress (see 3.8.6.8).

3.3.3.2 Hanger reinforcement for bottom-loaded beams: Where shear forces from supported beams are transferred at the junction with a bearing beam at a level below or partially below the neutral axis of the beam, adequate hanger steel should be provided to transfer the total shear load effectively into the compression zone of the bearing beam. The hanger steel can, however, be reduced by the ratio $\frac{h_1}{h_2}$ if the overall depth h_1 of the supported beam is smaller than the depth h_2 of the bearing beam, provided that the top surface of the two beams is at the same level. The hanger reinforcement should consist of inclined hanger bars or stirrups anchored around the main reinforcement of the bearing beam. The main reinforcement in the supported beam shall be placed above that of the bearing beams, as shown in Figure 8.

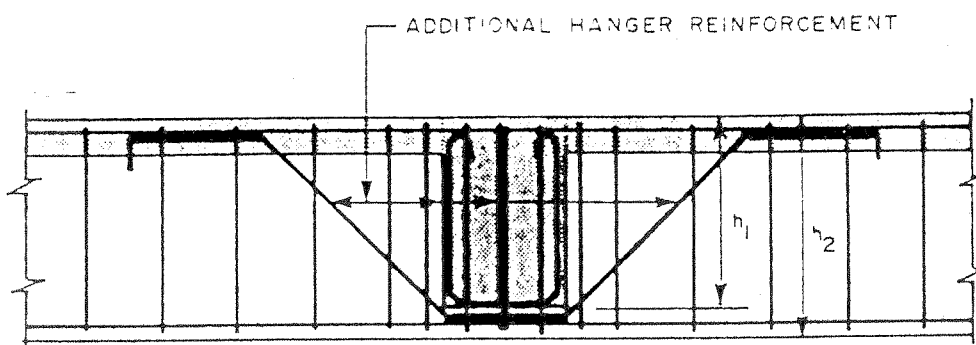


FIGURE 8
HANGER REINFORCEMENT

3.3.3.3 Members subjected to axial loads: For a member subjected to an ultimate axial compression load N (in newtons), the values in Table 7 may be multiplied by

$$\beta_s = \left[1 + 0,10 \frac{N}{A_c} \left(1 - \frac{1}{2e_0/e} \right) \right] \geq 2 \dots\dots\dots (9g)$$

where A_c is the area of the entire concrete section in mm^2

e_0 is the eccentricity of an axial load that results in zero stress (decompression) at an extreme fibre of the section under consideration, eg for a rectangular section of total depth $e_0 = \frac{h}{6}$

e is the eccentricity of the axial load N that would cause a moment equal to that due to the applied design loads at the section under consideration.

The axial compression load N is taken into account at its design value using partial load factors corresponding to the favourable effect, ie. $\gamma_{IL} = \gamma_{I3} \leq 1,0$.

For a member subjected to an axial tensile load, the value of v_c shall be zero.

For axial tensile loads less than $0,12\sqrt{f_{cu}} \cdot A_c$ (in newtons), the steel links shall be designed to take the total shear force, using Equations 9d to 9f (with $v_c = 0$). Additional longitudinal steel reinforcement (additional to that resisting the tensile load) shall be placed transversely to the legs of the links distributed through the depth of the beam to form a mesh. This additional longitudinal reinforcement shall be increased linearly to compose from not less than 20 % of the amount of shear reinforcement at a tensile load of $0,2 \times 0,12\sqrt{f_{cu}} \cdot A_c$ up to 100 % at $0,12\sqrt{f_{cu}} \cdot A_c$. In this context the amount of shear reinforcement is defined by the sum of the cross-sectional areas of the legs of the shear links within a length of beam equal to $\frac{2}{3}$ of the total depth.

For axial tensile loads greater than $0,12\sqrt{f_{cu}} \cdot A_c$, the design shall be based either on proven research or on a series of specific tests to be approved by the responsible bridge authority. The axial tensile load is taken into account at its design value using partial load factors corresponding to the adverse effect, ie $\gamma_{IL} = \gamma_{I3} \geq 1,0$.

3.3.3.4 Elements of variable effective depth: For the determination of the shear resistance of members with variable effective depth, the components of the flange forces and reinforcement forces parallel to the shear should be taken into account if their effect is unfavourable. They may also be taken into account however, if their effect is favourable.

3.3.4 Torsion

3.3.4.1 Torsional stiffness: Where the torsional resistance and stiffness of members with solid or closed hollow sections (box-sections) are taken into account

in the analysis and design of the structure, the torsional rigidity ($G \times C$) of a member may be obtained by assuming that the shear modulus G is equal to 0,4 times the modulus of elasticity of the concrete, and that the torsional coefficient C is equal to half the St Venant value for pure torsion calculated for the uncracked plain concrete section. Where torsion is resisted by open sections consisting of two or more interconnected planar elements (thin walls or slabs in separate planes, but approximately parallel to a single axis such as a deck slab and two or more thin webbed beams which do not form a closed box section), warping will take place and large deflections and/or stresses may occur. Specialist literature should be consulted.

3.3.4.2 Torsion of secondary importance: Where torsion does not decide the dimensions of members, torsional design may be carried out as a check, after the flexural design. This is particularly relevant to some members in which the maximum torsional moment does not occur under the same loading as the maximum flexural moment. In such circumstances, reinforcement in excess of that required for flexure and other forces may be used as part of the torsion reinforcement, if required. Torsional moments caused by the restraint of angular rotation by adjacent members, and which are not necessary for equilibrium, may be neglected in ultimate limit state calculations provided that experience or analysis has shown that torsion will play only a minor role in the behaviour of the structure.

3.3.4.3 Stresses caused by and reinforcement for pure torsion (St Venant torsion): Where torsion in a section substantially increases the shear stresses, the torsional shear stress should be calculated, assuming a plastic stress distribution. The following stress limits must comply with the requirements of 3.3.4.6 for combined effects. Where the torsional shear stress, v_t , exceeds the value v_{tmin} from Table 9, reinforcement should be provided. In no case should v_t exceed the value $v_{tu} \sin 2\theta_t$, where v_{tu} is given in Table 9 and θ_t is defined in 3.3.4.4. Nor, in the case of small sections ($y_1 < 550$ mm), should v_t exceed $v_{tu} y_1 / 550$, where y_1 is the larger centre-line dimension of a link. The values v_{tu} in Table 9 are derived from the formula $0,75\sqrt{f_{cu}}$, but may not be greater than 4,75 MPa.

Torsion reinforcement should consist of rectangular or polygonal links effectively closed (see also 3.8.6.5), together with longitudinal reinforcement. The closed links shall be of a type similar to those shown as Code 74, Links, in SABS 82 (1976) (see Figure 9) or those shown as Code 60 (see Figure 10), in which case the requirements of 3.8.6.5 shall apply. This reinforcement is additional to any required for shear or bending.

The member should be analysed with the assumption that the closed links and longitudinal reinforcement form a thin-walled tube in conjunction with the peripheral concrete, in which the shear stresses are balanced by longitudinal and transverse forces provided by the resistance of the reinforcement.

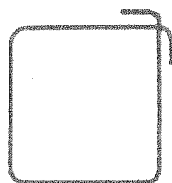


FIGURE 9

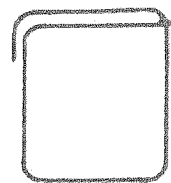


FIGURE 10

CLOSED LINKS FOR TORSION REINFORCEMENT

3.3.4.4 Treatment of various cross sections:

- (a) *Rectangular sections:* The torsional stresses should be calculated from the equation:

$$v_t = \frac{2T}{h_{\min}^2 (h_{\max} - h_{\min}/3)} \dots\dots\dots (10)$$

where T is the torsional moment due to the ultimate loads

$h_{\min}$ is the smaller dimension of the section

$h_{\max}$ is the larger dimension of the section.

Torsion reinforcement should be provided such that:

$$\frac{A_{st}}{s_v} \geq \frac{T - T_{cd}}{1,6x_1y_1(0,87f_{yv})\cot\theta_t} \dots\dots\dots (11)$$

$$A_{sL} \geq \frac{2A_{st}}{s_v} \left[\frac{f_{yv}}{f_{yL}} \right] (x_1 + y_1) \cot^2\theta_t \dots\dots\dots (12)$$

where A_{st} is the cross-sectional area of one leg of a closed link provided at a section to resist torsion

T_{cd} is the torsion resisted by concrete only and is derived from Equation 10 by substituting T_{cd} for T and equating v_t with $k_{vt}v_{tmin}$ where k_{vt} varies linearly

from 1,0 for $v_t \leq v_{tmin}$

to 0,0 for $v_t \geq 3v_{tmin}$ (see Figure 5)

s_v is the spacing of the links

x_1 is the smaller centre-line dimension of the links

y_1 is the larger centre-line dimension of the links

f_{yv} is the characteristic strength of the links

θ_t is the assumed angle of inclination of the compressive forces due to torsion with the centre line of the relevant beam surface at the section under consideration, as illustrated in Figure 11.

A_{sL} is the sum of the cross-sectional areas of longitudinal reinforcement provided at a section for torsion

f_{yL} is the characteristic strength of the longitudinal reinforcement.

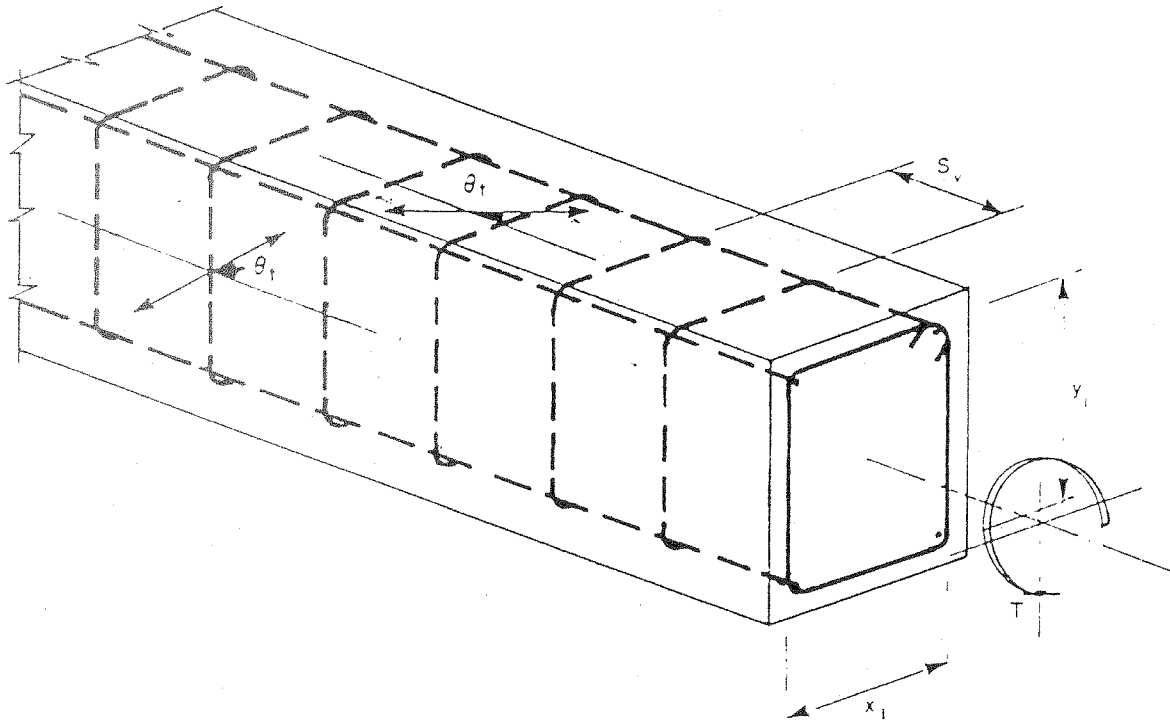


FIGURE 11

ASSUMED TORSIONAL SYSTEM SHOWING INCLINATION OF COMPRESSIVE FORCES

θ_t should be prudently chosen within limits so that $\frac{3}{5} \leq \cot \theta_t \leq \frac{5}{3}$ to ensure reasonable control of cracking under service conditions. θ_t may vary along the beam. In general, it may be assumed to be equal to 45° so that $\cot \theta_t$ reduces to 1,0.

In Equation 12, f_{yv} and f_{yL} should not be taken as greater than 450 MPa.

- (b) *T, L and I-sections*: Such sections should be divided into component rectangles for purposes of torsional design. This should be done in such a way as to maximize the function $\Sigma(h_{\max} h_{\min}^3)$ where $h_{\max}$ and $h_{\min}$ are the larger and smaller dimensions of each component rectangle. Each rectangle should then be considered to be subject to a torque:

$$\frac{T(h_{\max} h_{\min}^3)}{\Sigma(h_{\max} h_{\min}^3)}$$

Reinforcement should be so detailed as to tie the individual rectangles together. Where the torsional shear stress in a minor rectangle is less than v_{tmin} , no torsion reinforcement needs to be provided in that rectangle.

- (c) *Box sections:* Box sections may be treated in the same way as rectangular sections, except that the torsional shear stress should be calculated as:

$$v_t = T/2h_{wo}A_o \quad (10a)$$

where h_{wo} is the thickness of the wall at which the stress is determined

A_o is the cross-sectional area enclosed by the median wall line.

The reinforcement derived from Equations 11 and 12 for large thin-walled box sections may, however, be more than is actually required. In such cases Equations 11 and 12 may be modified as follows:

$$\frac{A_{st}}{s_v} \geq \frac{T - T_{cd}}{2A_o(0,87f_{yv})\cot\theta_t} \quad (11a)$$

where T_{cd} is the torsion resisted by concrete only

$$= 2k_{vt}v_{tmin}h_{wo}A_o$$

where k_{vt} varies linearly from 1,0 for $v_t \leq v_{tmin}$

to 0,0 for $vt \geq 3v_{tmin}$

$$A_{sL} \geq \frac{A_{st}}{s_v} \frac{f_{yv}}{f_{yL}} U_o \cot^2 \theta_t \quad (12a)$$

where U_o = perimeter of A_o .

The detailing requirements of 3.3.4.5 should still be observed.

See 3.3.4.6 for the combined effects of torsion and shear in flanges and webs of box-beams.

- (d) *Polygonal solid sections:* Polygonal solid sections that cannot be efficiently divided into component rectangles can be treated as equivalent hollow sections with an effective wall thickness

$$h_{ef} = \frac{d_{ef}}{6}$$

where d_{ef} is the diameter of the largest circle that can be contained within U_{ef} .

U_{ef} is the length of the mean polygonal perimeter which is defined by the median lines of the effective walls and encloses a cross-sectional area A_{ef} . Longitudinal reinforcement shall be positioned in each corner at the intersection of the median lines, on or outside the perimeter of the cross-sectional area A_{ef} , provided that minimum cover of closed links is maintained.

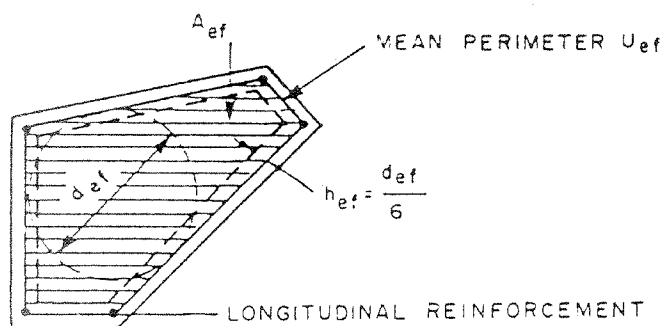


FIGURE 12
EQUIVALENT HOLLOW SECTION FOR POLYGONAL SOLID SECTION

Equations 10a, 11a and 12a apply where

$$h_{ef} = h_{wo} \text{ and } A_{ef} = A_o$$

TABLE 9 Ultimate torsional shear stress

	Stress (in MPa) at concrete grades			
	20	25	30	40 or more
V_{tmin}	0,30	0,33	0,37	0,42
V_{tu}	3,35	3,75	4,10	4,75

3.3.4.5 Detailing: Care should be taken during detailing to prevent failure caused by the diagonal compressive forces in adjacent faces of a beam spalling the corner of the section. The closed links should be detailed to have minimum cover, and a pitch less than the smallest of $(x_1 + y_1)/4$ for rectangular links (or $U_o/10$ for polygonal links), 16 longitudinal corner bar diameters or 300 mm. The longitudinal reinforcement should be positioned uniformly and in such a way that there is at least one bar at each corner of the links. The diameter of the corner bars should be not less than the diameter of the links. The designer should note that, by increasing the amount of longitudinal reinforcement, the ductility of the cross-section may be reduced under some load conditions in which the most serious torsional and bending moments do not occur simultaneously.

3.3.4.6 Combined effects

Axial and/or torsion and bending stresses: In detailing the longitudinal reinforcement to cater for torsional stresses, those areas of the cross-section that are subject to

simultaneous flexural and axial compressive stresses may be taken into account and a lesser amount of reinforcement provided. The reduction in the amount of reinforcement in the compressive zone may be taken as:

$$\text{Reduction in steel area} = \frac{f_{cav} \cdot A_{cav}}{0,87f_{yL}}$$

where f_{cav} is the average compressive stress in the axial and/or flexural compressive zone

A_{cav} is the area of section subject to axial and/or flexural compression.

In the case of beams, the depth of the compression zone used to calculate the area of section subject to axial and/or flexural compression should be taken as twice the cover to the closed links. The area of either the links or the longitudinal reinforcement may be reduced by 20 % provided the product

$$\frac{A_{sv}}{S_v} \times \frac{A_{sL}}{2(x_1 + y_1)}$$

for rectangular sections, or

$$\frac{A_{sv}}{S_v} \times \frac{A_{sL}}{U_o}$$

for other sections, remains unchanged.

In the flexural tension zone, the torsional reinforcement is additional to the reinforcement required for the axial or flexural tension.

Torsion and shear: The simultaneously applied torsion T and shear V due to ultimate loads must fulfil the condition

$$\left[\frac{T}{T_u} \right]^n + \left[\frac{V}{V_u} \right]^n \leq 1 \dots\dots\dots (13)$$

where the exponents n depend on the geometry of the cross-section, the percentage and location of the steel, and the material stresses, and are usually less than 2 but should conservatively be taken as 1 unless reliable data supporting the use of the higher value are available.

where T_u and V_u are the corresponding ultimate torsional and shear capacities respectively at the section under consideration, provided that, if

$$\frac{V_t}{k_{vt} V_{tmin}} + \frac{V}{k_v E_s V_c} > 1$$

T_{cd} the torsional capacity due to concrete only, as defined in 3.3.4.4, must be reduced by a factor

$$\frac{v_t}{v_t + v}$$

and $v_{cd} = k_v \xi_s v_c b d$, the shear capacity due to concrete only (see 3.3.3.1), must be reduced by a factor

$$\frac{v}{v_t + v}$$

and provided also that $v_t + v \leq v_{\max} \sin 2\theta_c$.

Provided that the torsion and shear reinforcements are calculated separately and added together the requirements of Equation 13 need apply only to the ultimate, torsional and shear resistances of the concrete calculated for the relevant sectional properties and the maximum allowed shear stresses v_{tu} and $v_{\max}$ respectively. (See 3.1.3 for definitions.)

3.3.5 Longitudinal shear

For flanged beams, the longitudinal shear resistance of the flange and of the flange/web junction should be checked in accordance with 5.4.2.3.

3.3.6 Deflection in beams

Deflection may be calculated in accordance with the provisions of Section 2.

3.3.7 Crack control in beams

Flexural cracking in beams should be controlled by checking crack widths in accordance with 3.8.8.2.

3.4 Slabs

3.4.1 Moments and shear forces in slabs

Moments and shear forces in slab bridges and in the top slabs of beam-and-slab, voided slab and box-beam bridges may be obtained from a general elastic analysis, or from particular elastic analyses such as those developed by Westergaard or Pucher. Alternatively, Johansen's yield-line method may be used to obtain the required ultimate moments of resistance, provided that the requirements of 2.4.2.1 are met.

The effective spans should be in accordance with 3.3.1.1.

In slabs that are well restrained against horizontal deflection at the boundaries, the augmented resistance to live loads by compressive membrane or arching action may, however, be taken into account in the analysis, provided the assumptions made are based on proven research and are approved by the responsible bridge authority.

3.4.2 Resistance moments of slabs

The ultimate resistance moment in the direction of reinforcement may be determined by the method given in 3.3.2. If reinforcement is to be provided to resist a combination of two bending moments and a twisting moment at a point in a slab, allowance should be made for the fact that the directions of the principal moment and the reinforcement do not generally coincide. Such allowance can be made by calculating the moments of resistance in the directions of reinforcements, so that adequate strength is provided in all directions.

In voided slabs, the stresses in the transverse flexural reinforcement due to transverse shear effects should be calculated by means of an appropriate analysis (eg an analysis based on the assumption that the transverse section acts as a Vierendeel frame).

3.4.3 Resistance to in-plane forces

If reinforcement is to be provided to resist a combination of in-plane direct and shear forces at a point in a slab, allowance should be made for the fact that the directions of the principal stress and reinforcement do not generally coincide. Such allowance can be made by calculating the required forces in the directions of reinforcement, so that adequate strength is provided in all directions.

3.4.4 Shear resistance of slabs

3.4.4.1 Shear stresses in solid slabs : General: The shear stress, v , at any cross-section in a solid slab should be calculated from:

$$v = \frac{V}{bd} \dots\dots\dots (14)$$

where V is the shear force due to ultimate loads

b is the width of the slab under consideration

d is the effective depth

No shear reinforcement is required when the stress, v , is less than $\xi_s v_c$, where ξ_s has the value shown in Table 8 and v_c is obtained from Table 7. An enhancement in shear strength may be allowed for sections within a distance $a_v < 2d$ from the face of a support as for beams (see 3.3.3.1). The shear stress, v , in a solid slab less than 200 mm thick should not exceed $\xi_s v_c$.

In solid slabs at least 200 mm thick, when v is greater than $\xi_s v_c$, shear reinforcement should be provided as for a beam (see 3.3.3) except that the space between links may be increased to d .

The maximum shear stress, $v_{\max}$, due to ultimate loads should not exceed $0,75\sqrt{f_{cu}}$ or 4,75 MPa, whichever is the smaller.

In calculating shear stresses in slabs, any non-uniform distribution of shear forces along the breadth being considered should be allowed for. However, the reduction in peak values due to lateral spreading of concentrated or non-uniform loads may be taken into account, provided the assumptions made are supported by a theoretical analysis or by experimental test results. The dispersal of wheel loads through surfacing should be taken only to the top surface of the concrete slab and not down to the neutral axis.

3.4.4.2 Shear stresses in solid slabs under concentrated loads (including wheel loads): The critical section for calculating shear should be taken on a perimeter $1,5d$ from the boundary of the loaded area, as shown in Figure 13, case (a), where d_x and d_y are the effective depths to the flexural tensile reinforcement in the respective directions.

Where concentrated loads are applied to a cantilever slab or near unsupported edges, the relevant portions of the critical section should be taken as the worst case from (a), (b) or (c) of Figure 13. The dimensions e_x and e_y are the distances in the X and Y directions respectively between the relevant edge of the load and the parallel unsupported edge of the slab. For a group of concentrated loads, adjacent loaded areas should also be considered singly and in combination, using the recommendations given below.

A circular loaded area should be equated with a square of equal area.

No shear reinforcement is required when the ultimate shear force, V , due to concentrated loads, is less than the ultimate shear resistance of the concrete, V_c , at the critical section, as given in Figure 13.

The overall ultimate shear resistance at the critical section should be taken as the sum of the shear resistance of each portion of the critical section. The value of

$$\frac{100A_s}{b_s d}$$

to be used in Table 7 for each portion should be derived by considering the effectively anchored flexural tensile reinforcement associated with each portion, as shown in Figure 13.

In solid slabs at least 200 mm thick, where V has a value between V_c and the maximum shear resistance (based on a maximum shear stress $v_{\max}$ given in 3.4.4.1),

Critical section for calculating shear resistance V_c . (Critical sections (a), (b) and (c) (i) are assumed to have squared corners for rectangular and circular loaded areas)	(a) LOAD AT MIDDLE OF SLAB	(b) LOAD AT EDGE OF SLAB	(c) LOAD AT CORNER OF CANTILEVER SLAB
Idealized mode of failure (only tension reinforcement shown)			
Parameters used to derive V_c from Table 7 for each portion of critical section NOTE: A_s should include only tensile reinforcement which is effectively anchored			
Shear resistance, V_c , at critical section	$\Sigma \xi_s V_c b d$ for 4 critical portions	$0,8 \Sigma \xi_s V_c b d$ for 3 critical portions	$0,8 \Sigma \xi_s V_c b d$ for 2 critical portions
			$\Sigma \xi_s V_c b (d_x + d_y) / 2$

FIGURE 13
PARAMETERS FOR SHEAR IN SOLID SLABS UNDER CONCENTRATED LOADS

an area of shear reinforcement should be provided on the critical perimeter and similar amount on a parallel perimeter at a distance of $0,75d$ inside it, such that:

$$0,4 \text{ MPa} \leq \frac{\Sigma A_{sv}(0,87f_{yv})}{\Sigma b_d} > v - k_v \xi_s v_c \dots\dots\dots (15)$$

where ΣA_{sv} is the area of shear reinforcement

f_{yv} is the characteristic strength of the shear reinforcement which should be taken as not greater than 450 MPa.

Σb_d is the area of critical section

k_v is as defined in 3.3.3.1.

Values for shear stress should be calculated on perimeters progressively $0,75d$ away from the critical perimeter and if $k_v \xi_s v_c$ continues to be exceeded, further shear reinforcement should be provided on each perimeter in accordance with Equation 15, substituting the appropriate values for v and Σb_d . Shear reinforcement should be considered effective only in those places where the slab depth is greater than or equal to 200 mm. Shear reinforcement may be provided in the form of vertical or inclined links anchored at both ends by passing around the main reinforcement. Links should not be spaced further apart than $0,75d$ and, if links at θ_v are used, the area of shear reinforcement obtained from Equation 15 should be divided by the factor

$$\{(1 + \cot \theta_v) \sin \theta_v\}.$$

When openings in slabs and footings (see Figure 14) are located at a distance less than $6d$ from the edge of a concentrated load or reaction, then that part of the periphery of the critical section that is enclosed by the radial projections of the openings to the centroid of the loaded area should be considered ineffective.

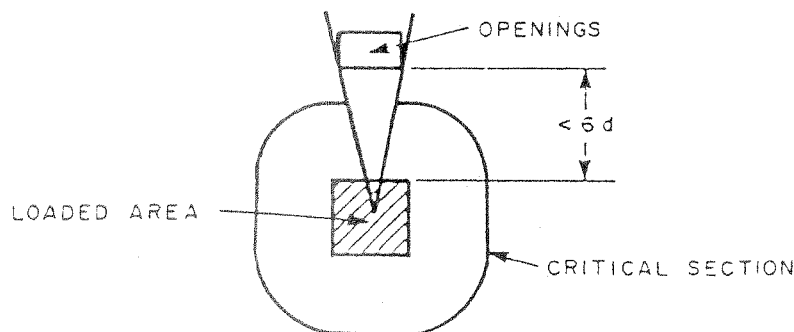


FIGURE 14
OPENINGS IN SLABS

Where one hole is adjacent to the column and its greatest width is less than one-quarter of the side of the column or one-half of the slab depth, whichever is the smaller, its presence may be ignored.

3.4.4.3 Shear in voided slabs: The longitudinal ribs between the voids should be designed as beams (see 3.3.5 and 5.4.2.3) to allow for the shear forces in the longitudinal direction, including any shear due to torsional effects (see 3.3.4).

The top and bottom flanges should be designed as solid slabs (see 3.4.4.1), each to carry a part of the global transverse shear forces and any shear forces due to torsional effects. The top flange of a rectangular voided slab should be designed to resist the punching effect of wheel loads (see 3.4.4.2). Where wheel loads may punch through the whole slab, this should also be checked.

3.4.5 Deflection of slabs

Deflections may be calculated in accordance with the provisions of Section 2.

3.4.6 Crack control in slabs

Cracking in slabs should be checked in accordance with 3.8.8.2.

3.5 Columns

3.5.1 General

The subsections in this section give approximate methods, based on a number of assumptions, for assessing the strength at the ultimate limit state of prismatic columns with symmetrical cross-sections approximating to square, rectangular or circular shapes. The methods may be used only if the assumptions are valid for the case being considered and provided the effective height is determined with commensurate accuracy.

These methods are generally conservative and the analysis may be extended and refined by using more accurate methods based on fundamental principles that take all relevant actions and effects into account. Such refinements would be necessary in the case of columns that do not comply with all the assumptions of these clauses, and in the case of columns with non-symmetrical cross-sections or varying (non-prismatic) shapes.

Where a more rigorous analysis is warranted, the long-term effects of creep and shrinkage should be assessed. For columns subject to bending moments, the serviceability limit state for cracking given in 2.1.2.1(a) should be met. Where the greater lateral dimension of the cross-section of a rectangular column is more than or equal to four times its lesser lateral dimension, the reinforcement should be designed in accordance with 3.6.

3.5.1.1 Short and slender columns : Definitions: A column should be considered as short if the ratio l_e/h in each plane of buckling is less than 12.

where l_e is the effective height in the plane of buckling under consideration and h is the depth of the cross-section in the plane of buckling under consideration.

It should otherwise be considered as slender.

3.5.1.2 Effective height of a column: The effective height, l_e , in a given plane may be obtained from Table 10 where l_o is the clear height between end restraints.

The values given in Table 10 are based on the following assumptions:

(a) The rotational restraint is at least $4(EI)_c/l_o$ for Cases 1, 2 and 4 to 6, and $8(EI)_c/l_o$ for Case 7, $(EI)_c$ being the flexural rigidity of the column cross-section, ie the integrated products of the equivalent moduli of elasticity of the material components and the moment of inertia of the corresponding component sectional areas.

(b) The lateral and rotational rigidity of elastometric bearings is zero.

Where a more accurate evaluation of the effective height is required or where the end stiffness values are less than those given in (a), the effective height should be derived from first principles.

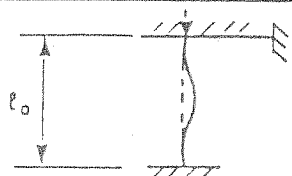
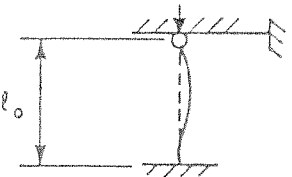
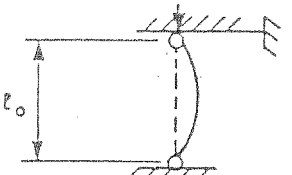
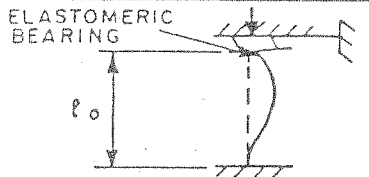
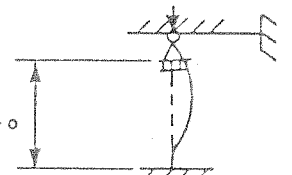
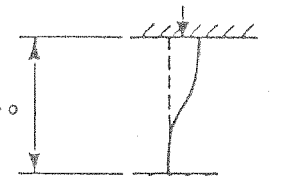
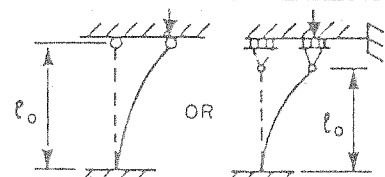
How movements under service loads are being accommodated and the method of articulation chosen for the bridge will influence the degree of restraint to be assumed for the columns. These influences should be assessed as accurately as possible using engineering principles based on elastic theory, and taking into account all the relevant factors such as foundation flexibility, type of bearings, articulation systems, etc.

3.5.1.3 Slenderness limits for columns analysed by approximate methods: In each plane of buckling the ratio l_e/h should not exceed 40, except that where the column is not restrained in position at one end, the ratio l_e/h should not exceed 30.

3.5.2 Moments and forces in columns

The moments, shear forces and axial forces in a column should be determined in accordance with 2.4, except that if the column is slender, the moments induced by deflection should also be considered. Allowance is made for these additional moments in the design recommendations for slender columns that follow. The bases as well as other members connected to the ends of such columns should also be designed to resist these additional moments.

TABLE 10 Effective height, l_e for columns

CASE	IDEALIZED COLUMN AND BUCKLING MODE	RESTRAINTS			EFFECTIVE HEIGHT, %
		LOCATION	POSITION	ROTATION	
1		TOP	FULL	FULL*	0,70
		BOTTOM	FULL	FULL*	
2		TOP	FULL	NONE	0,85
		BOTTOM	FULL	FULL*	
3		TOP	FULL	NONE	1,0
		BOTTOM	FULL	NONE	
4		TOP	NONE*	NONE*	1,3
		BOTTOM	FULL	FULL*	
5		TOP	NONE	NONE	1,4
		BOTTOM	FULL	FULL*	
6		TOP	NONE	FULL*	1,5
		BOTTOM	FULL	FULL*	
7		TOP	NONE	NONE	2,3
		BOTTOM	FULL	FULL*	

* ASSUMED VALUE (SEE 3.5.1.2)

In columns with end moments, it is generally necessary to consider the maximum and minimum ratios of moment-to-axial-load in designing reinforcement areas and concrete sections.

3.5.3 Short columns subject to axial loading and bending about the minor axis

3.5.3.1 General: A short column should be designed for the ultimate limit state in accordance with the recommendations made below, provided that the moment at any cross-section has been increased to that moment produced by considering the ultimate axial load as acting at an eccentricity equal to 0,05 times the overall depth of the cross-section in the plane of bending, but equal to not less than 20 mm. This is a nominal allowance for eccentricity to take account of construction tolerances.

3.5.3.2 Analysis of sections: When analysing a column cross-section to determine its ultimate resistance to moment and axial loading, the following assumptions should be used:

- (a) Plane sections are assumed to remain plane when the strain distribution in the concrete in compression and the compressive and tensile strains in the reinforcement are being derived.
- (b) The stresses in the concrete in compression can either be derived from the stress-strain curve in Figure 1 with $\gamma_m = 1,5$, or may be taken as equal to $0,4f_{cu}$ over the whole compression zone when this is rectangular or circular. In both cases, the concrete strain at the outermost compression fibre at failure is taken as 0,0035.
- (c) The tensile strength of the concrete can be disregarded.
- (d) The stresses in the reinforcement can be derived from the stress-strain curve in Figure 2 with $\gamma_m = 1,15$.

For rectangular and circular columns, the design methods given below, which are based on the above assumptions, may be used. For other column shapes, design methods should be derived from first principles using the above assumptions.

3.5.3.3 Design charts for rectangular and circular columns: The design charts that form Parts 2 and 3 of the *BS Code of Practice - CP 110 (1972)* include charts (based on what are given here as Figures 1 and 2 and on the assumptions listed in 3.5.3.2) that may be used for the design of rectangular and circular column sections having a symmetrical arrangement of reinforcement.

3.5.3.4 Design formulae for rectangular columns: The formulae given below (based on a concrete stress of $0,4f_{cu}$ over the whole compression zone and the assumptions in 3.5.3.2) may be used to design a rectangular column having

longitudinal reinforcement in the two faces parallel to the axis of bending, whether that reinforcement is symmetrical or not. Both the ultimate axial load N and the ultimate bending moment M should not exceed the values of N_u and M_u given by Equations 16 and 17 for the appropriate value of d_c .

$$N_u = 0,4f_{cu}bd_c + f_{yc}A's_1 + f_{s_2}As_2 \dots\dots\dots(16)$$

$$Mu = 0,2f_{cu}bd_c(h - d_c) + f_{yc}A's_1\left(\frac{h}{2} - d_1'\right) - f_{s_2}As_2\left(\frac{h}{2} - d_2\right) \dots\dots\dots(17)$$

- where M is the ultimate bending moment applied about the axis considered due to ultimate loads, including the nominal allowance for construction tolerances (see 3.5.3.1)
- N_u and M_u are the ultimate axial load and ultimate resistance moments of the section for the particular value of d_c assumed
- f_{cu} is the characteristic cube strength of the concrete
- b is the breadth of the section
- d_c is the assumed depth of concrete in compression, subject to a minimum value of $2d'$
- f_{yc} is the design compressive strength of the reinforcement, taken as

$$\frac{f_y}{\gamma_m + \frac{f_y}{2\ 000}} \text{ (in MPa)}$$
- $A's_1$ is the area of compression reinforcement in the more highly compressed face
- f_{s_2} is the stress in the reinforcement in the other face, derived from Figure 2 and taken as negative if tensile
- $A's_2$ is the cross-sectional area of reinforcement in the other face, which may be considered as being:
- (i) in compression
 - (ii) inactive, or
 - (iii) in tension
- as the resultant eccentricity of load increases and d_c decreases from h to $2d'$
- h is the depth of the section in the plane of bending
- d' is the depth from the surface to the reinforcement in the more highly compressed face.
- d_2 is the depth from the other surface to the reinforcement in the other face (see Figure 15).

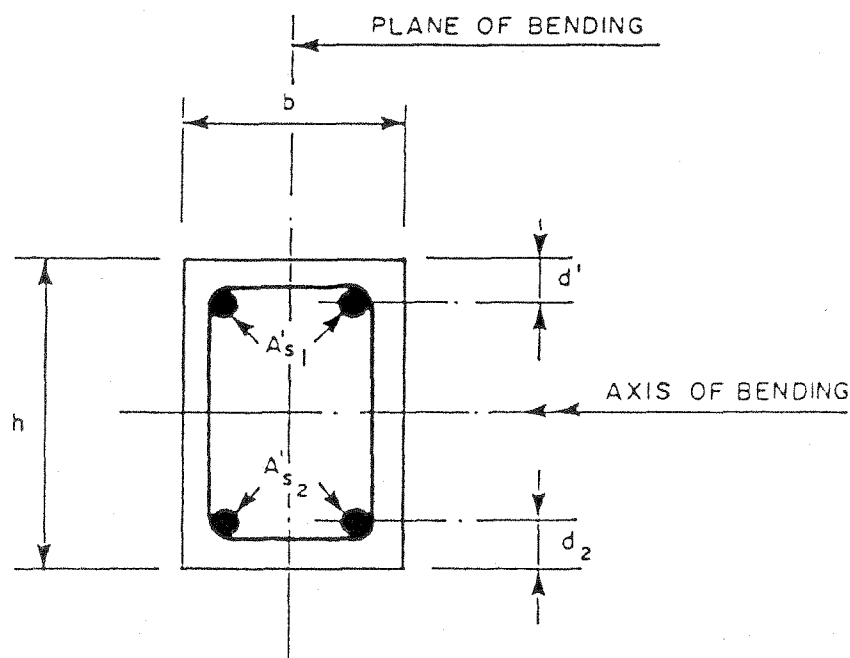


FIGURE 15
CROSS-SECTION OF A COLUMN

3.5.3.5 Simplified design formulae for rectangular columns: The following simplified formulae may be used, as appropriate, for the design of a rectangular column having longitudinal reinforcement in the two faces parallel to the axis of bending, whether that reinforcement is symmetrical or not.

- Where the resultant eccentricity $e = M/N$ does not exceed $\frac{h}{2} - d_1$ and where the ultimate axial load N does not exceed $0,45f_{cu}b(h - 2e)$, only nominal reinforcement is required (see 3.8.4.1 for minimum provision of longitudinal reinforcement) where M , N , h , d_1 , f_{cu} and b are as defined in 3.5.3.4.
- Where the resultant eccentricity is not less than $\frac{h}{2} - d_2$ the axial load may be ignored and the column section designed to resist an increased moment:

$$M_a = M + N\left(\frac{h}{2} - d_2\right) \dots\dots\dots (18)$$

where M , N , h and d_2 are as defined in 3.5.3.4. The area of tension reinforcement needed to provide resistance to this increased moment must be reduced by the amount $N/0,87f_y$.

3.5.4 Short columns subject to axial loading and to either bending about the major axis or biaxial bending

The moment about each axis due to ultimate loads should be increased by the nominal allowance for eccentricity to take account of construction tolerances, as given in 3.5.3.1.

For square, rectangular and circular columns having a symmetrical arrangement of reinforcement about each axis, the section should be designed for axial loading and bending about each axis in accordance with any one of the methods of design given in 3.5.3.2, 3.5.3.3 or 3.5.3.4, such that

$$\left[\frac{M_x}{M_{ux}} \right]^{\alpha_n} + \left[\frac{M_y}{M_{uy}} \right]^{\alpha_n} \leq 1 \quad (19)$$

where M_x and M_y are the moments about the major x-x axis and minor y-y axis respectively due to ultimate loads, including the nominal allowance for construction tolerances given in 3.5.3.1

M_{ux} and M_{uy} are the ultimate moment capacities about the major x-x axis and minor y-y axis respectively, each in association with the ultimate axial load N

α_n is related to N/N_{uz} as given in Table 11, where N_{uz} is the axial loading capacity of a column, ignoring all bending, taken as:

$$N_{uz} = 0,45f_{cu}A_c + f_{yc}A_{sc} \quad (20)$$

where f_{cu} and f_{yc} are as defined in 3.5.3.4.

A_c is the cross-sectional area of concrete

A_{sc} is the cross-sectional area of longitudinal reinforcement.

For other column sections, design should be in accordance with 3.5.3.2.

TABLE 11 Relationship of N/N_{uz} to α_n

N/N_{uz}	α_n
$\leq 0,2$	1,0
0,4	1,33
0,6	1,67
$\geq 0,8$	2,0

3.5.5 Slender columns of constant rectangular or circular cross-section

3.5.5.1 General: A cross-section of a slender column may be designed by the methods given for a short column (see 3.5.3 and 3.5.4), but in the design account should be taken of the additional moment induced in the column by its deflection. For slender columns of constant rectangular or circular cross-section having a symmetrical arrangement of reinforcement, the columns should be designed to resist the ultimate axial load N together with the moments M_{tx} and M_{ty} derived in accordance with 3.5.5.4. Alternatively, the simplified formulae given in 3.5.5.2 and 3.5.5.3 may

be used where appropriate; in this case the moment due to ultimate loads need not be increased by the nominal allowance for eccentricity to take account of construction tolerances given in 3.5.3.1. It will be sufficient to limit the minimum value of moment to not less than the nominal allowance given in 3.5.3.1. A nominal allowance for the long-term effects of creep has been made in the derivation of the additional moments incorporated in Equations 21, 23, 24 and 25.

3.5.5.2 Slender columns bent about a minor axis: A slender column of constant cross-section bent about the minor y-y axis should be designed for its ultimate axial load, N , together with the moment, M_{iy} given by:

$$M_{iy} = M_{iy} + \frac{Nh_x}{1750} \left[\frac{l_e}{h_x} \right]^2 \left[1 - 0,0035 \frac{l_e}{h_x} \right] \dots\dots\dots (21)$$

where M_{iy} is the initial moment due to ultimate loads, but may not be less than corresponding to the nominal allowance for construction tolerances given in 3.5.3.1

h_x is the overall depth of the cross-section in the plane of bending M_{iy}

l_e is the effective height either in the plane of bending or in the plane at right angles, whichever is greater.

For a column fixed in position at both ends where no transverse loads occur at any level within its height, the value of M_{iy} may be reduced to:

$$M_{iy} = 0,4M_1 + 0,6M_2 \dots\dots\dots (22)$$

where M_1 is the smaller initial end moment due to ultimate loads (assumed to be negative if the column is bent in double curvature)

M_2 is the larger initial end moment due to ultimate loads (assumed to be positive).

In no case, however, should M_{iy} be taken as less than $0,4M_2$ or may M_{iy} be less than M_2 .

3.5.5.3 Slender columns bent about a major axis: When the overall depth of its cross-section h_y is less than three times its breadth h_x , a slender column bent about the major x-x axis should be designed for its ultimate axial load, N , together with the moment, M_{ix} , given by:

$$M_{ix} = M_{ix} + \frac{Nh_y}{1750} \left[\frac{l_e}{h_x} \right]^2 \left[1 - 0,0035 \frac{l_e}{h_x} \right] \dots\dots\dots (23)$$

where l_e and h_x are as defined in 3.5.5.2

M_{ix} is the initial moment due to ultimate loads, but may not be less than that corresponding to the nominal allowance for construction tolerances given in 3.5.3.1

h_y is the overall depth of the cross-section in the plane of bending M_{ix} .

Where $h_y \geq 3h_x$ the columns should be considered to be biaxially loaded with a nominal initial moment about the minor axis.

3.5.5.4 Slender columns bent about both axes: A slender column bent about both axes should be designed for its ultimate axial load, N , together with the moments M_{ix} , about its major axis, and M_{iy} , about its minor axis, given by:

$$M_{ix} = M_{ix} + \frac{Nh_y}{1750} \left[\frac{l_{ex}}{h_y} \right]^2 \left[1 - 0,0035 \frac{l_{ex}}{h_y} \right] \dots\dots\dots (24)$$

$$M_{iy} = M_{iy} + \frac{Nh_x}{1750} \left[\frac{l_{ey}}{h_x} \right]^2 \left[1 - 0,0035 \frac{l_{ey}}{h_x} \right] \dots\dots\dots (25)$$

where h_x and h_y are as defined in 3.5.5.2 and 3.5.5.3 respectively

M_{ix} is the initial moment due to ultimate loads about the x-x axis, including the nominal allowance for construction tolerances given in 3.5.3.1

M_{iy} is the initial moment due to ultimate loads about the y-y axis, including the nominal allowance for construction tolerances given in 3.5.3.1

l_{ex} is the effective height in respect of bending about the major axis

l_{ey} is the effective height in respect of bending about the minor axis.

3.5.5.5 Adjustments to additional moments in slender columns: In each of the Equations 21, 23, 24 and 25 the second term represents the additional moment induced in the column by its deflection. This may in all cases be reduced by multiplying the factor K , where

$$K = \frac{N_{uz} - N}{N_{uz} - N_{ba1}} \leq 1$$

where N is the ultimate axial load

N_{uz} is the capacity of the cross-section under pure axial load, which may be computed from Equation 20.

N_{bal} is the axial load corresponding to the balanced condition of a maximum compressive strain in the concrete of 0,0035 and a tensile strain in the outermost layer of tension steel of 0,002.

Where columns subject to lateral loading are constrained to deflect sideways by the same amount,

$$\frac{l_e}{h} \text{ (or } \frac{l_{ex}}{h_y} \text{ or } \frac{l_{ey}}{h_x} \text{, as appropriate)}$$

for each column may be taken as the average value in the same direction for all columns.

3.5.6 Shear resistance of columns

A column subject to uniaxial shear due to ultimate loads should be designed in accordance with 3.3.3 except that the ultimate shear stress, v_c , obtained from Table 7 may be multiplied by:

$$\beta_s = \left[1 + 0,10 \frac{N}{A_c} \left[1 - \frac{1}{2} \frac{e_o}{e} \right] \right] \times 2 \dots\dots\dots (26)$$

where β_s is a shear factor

N is the ultimate axial load (in newtons)

A_c is the cross-sectional area of the entire concrete section (in mm²)

e_o is the eccentricity of an axial load on the column that results in zero stress (decompression) at an extreme fibre of the section under consideration (eg for a rectangular column bent about the major axis $e_o = \frac{h_x}{6}$)

e is the eccentricity of the axial load N which would cause a moment equal to that due to the applied loads at the section under consideration.

The axial load N is taken into account at its design value using partial load factors corresponding to the favourable effect, ie:

$$\gamma_{IL} = \gamma_{I3} \leq 1,0$$

A column subject to biaxial shear due to ultimate loads should be designed for each axis in accordance with 3.3.3 such that:

$$\left[\frac{V_x}{V_{ux}} \right]^n + \left[\frac{V_y}{V_{uy}} \right]^n \leq 1 \dots\dots\dots (27)$$

where the exponents n depend on the geometry of the cross-section on the percentage and location of the steel, and the material stresses, and are usually less than 2,0 but should conservatively be taken as 1,0 unless reliable data supporting the use of higher values are available.

V_x and V_y	are the applied shears due to ultimate loads for the x-x axis and y-y axis respectively
V_{ux} and V_{uy}	are the corresponding ultimate shear capacities of the concrete and link reinforcement for the x-x axis and y-y axis respectively, provided that the resultant shear stress v_{xy} does not exceed $v_{max} \sin 2\theta_c$ where v_{max} is equal to $0,75\sqrt{f_{cu}}$ but is not greater than 4,75 MPa.

3.5.7 Crack control in columns

A column subject to bending should be considered as a beam for the purpose of crack control (see 3.8.8.2).

3.6 Reinforced Concrete Walls

3.6.1 General

3.6.1.1 Definitions: A reinforced wall is a vertical (or near-vertical) load-bearing concrete member whose greater lateral dimension is more than four times its lesser lateral dimension, and in which the reinforcement is taken into account when its strength is being considered. Retaining walls, wing walls, abutments, piers and other similar elements subject principally to bending moments, and on which the ultimate axial load is less than $0,1f_{cu}A_c$, should, for the purpose of bending, be treated as slabs supported by interaction with other walls or supported by counterforts, or as cantilevers supported by the base, and be designed in accordance with 3.4. In other cases, the clauses given below apply.

Reinforcement must comply with the conditions given in 3.8.4. A reinforced wall should be considered as either short or slender. Similarly to columns, a wall of constant thickness may be considered as short where the ratio of its effective height does not exceed 12. It should otherwise be considered as slender. For walls with a batter, a more fundamental approach may be necessary.

3.6.1.2 Limits to slenderness: The slenderness ratio is the ratio of the effective height of the wall to its thickness. The effective height should be obtained from Table 10. When the wall is restrained in position at both ends and the reinforcement complies with the requirements of 3.8.4, the slenderness ratio should not exceed 40, unless more than 1 % of vertical reinforcement is provided, when the slenderness ratio may be up to 45. When the wall is not restrained in position at one end, the slenderness ratio should not exceed 30.

When a wall is restrained at one or both vertical ends, the permissible slenderness ratio, depending on the height-to-length ratio, may be increased provided reliable data supporting the use of such an increased value are available.

3.6.2 Forces and moments in reinforced concrete walls

These should be calculated in accordance with 2.4, except that, if the wall is slender, the moments induced by deflection should also be considered. The distribution of axial and horizontal forces along a wall from the loads on the superstructure should be determined by analysis and their points of application decided on from the nature and location of the bearings. For walls fixed to the deck, the moments should be determined similarly by elastic analysis.

The moment per unit length in the direction at right angles to a wall (ie about an axis in the plane of a wall) should be taken as not less than $0,05n_w h$, where n_w is the ultimate axial load per unit length and h is the thickness of the wall. Moments in the plane of a wall (ie about an axis normal to a wall) should be calculated for the most severe positioning of the relevant loads.

Where the axial load is non-uniform, consideration should, where necessary, be given to "deep beam effects" and the distribution of axial loads per unit length of wall.

It will generally be necessary to consider the maximum and minimum ratios of moment to axial load in designing reinforcement areas and concrete sections.

3.6.3 Short reinforced walls resisting moments and axial forces

The cross-sections of the various portions of the wall should be designed to resist the appropriate ultimate axial load and the transverse moment per unit length, calculated in accordance with 3.6.2. The assumptions made when analysing beam sections (see 3.3.2.1) apply and are also applicable when the wall is subject to significant bending only in the plane of the wall.

When the wall is subjected to significant bending both in the plane of the wall and at right angles to it, consideration should be given first to bending in the plane of the wall in order to establish a distribution of tension and compression along the length of the wall. The resulting tension and compression should then be combined with the compression due to the ultimate axial load to determine the combined axial load per unit length of wall. This may be done by an elastic analysis, assuming a linear distribution along the wall.

The bending moment at right angles to the wall should then be considered. The section should be checked for this moment and for the resulting compression or tension per unit length at various points along the length of the wall, using the assumptions given in 3.3.2.1.

3.6.4 Slender reinforced walls

The distribution of axial loads along a slender reinforced wall should be determined as for a short wall. The critical portion of the wall should then be considered as a slender column of unit width and designed as such in accordance with 3.5.5.

3.6.5 Shear resistance of reinforced walls

A wall subject to uniaxial shear due to ultimate loads should be designed in accordance with 3.4.4, except that the ultimate shear stress, v_e , obtained from Table 7 may be multiplied by β_s given in Equation 26.

A wall subject to biaxial shear due to ultimate loads should be designed for each axis in accordance with 3.4.4 and should comply with Equation 27.

3.6.6. Deflection of reinforced walls

The deflection of a reinforced concrete wall will be within acceptable limits if the recommendations given in 3.6.1 to 3.6.5 have been followed.

3.6.7 Crack control in reinforced walls

Where walls are subject to bending, design crack width should be calculated in accordance with 3.8.8.2.

3.7 Bases (including pile caps)

3.7.1 Temporary pockets (see also 5.2)

Where pockets are left for precast members, allowance should be made, when computing the flexural and shear strengths of base sections, for the effects of these pockets, unless they are to be subsequently grouted up using a cement mortar with a compressive strength not less than that of the concrete in the base.

3.7.2 Moments and forces in bases

Except where the reactions to the applied loads and moments are derived by more accurate methods, eg by an elastic analysis of a pile group or by the application of established principles of soil mechanics, the following assumptions should be made:

- (a) when the base is axially loaded, the reactions to ultimate loads may be assumed to be uniformly distributed per unit area or per pile;
- (b) when the base is eccentrically loaded, the reactions may be assumed to vary linearly across the base. For columns and walls restrained in direction of movement at the base, the moment transferred to the base should be obtained from 3.5.

The critical section in the design of the bottom reinforcement of an isolated base may be taken as being at a distance of 0,15 times the dimension of the column or wall, measured perpendicularly inwards from the face of the column or wall.

The moment at any vertical section passing completely across a base should be taken as that due to all external ultimate loads and reactions on one side of that section.

3.7.3 Design of bases

3.7.3.1 Resistance to bending: Bases should be designed as "beam and slab" or "flat slab" as appropriate. Beam-and-slab bases should be designed in accordance with 3.3. Specialist literature should be consulted for the design of "deep bases" (ie the breadth-to-depth ratio is 2 or less)(see 3.3.2.1). Flat-slab sections should be designed to resist the total moments and shears at the sections considered.

Where the breadth of the section considered is less than or equal to $1,5 (b_{co1} + 3d)$, where b_{co1} is the breadth of the column and d is the effective depth to the tension reinforcement of the base, reinforcement should be distributed evenly across the breadth of the section considered. For greater breadths, two-thirds of the area of reinforcement should be concentrated within a width of $(b_{co1} + 3d)$ centred on the column.

Pile caps may be designed by using either bending theory or space truss analogy. The "members" of the "space truss" shall be within the concrete pile cap, taking the apex of the truss under the centre of the loaded area and the "member joints" in the base of the truss at the intersections of the centre lines of the piles with the plane of the tensile reinforcement (see Figure 16), which shall be distributed as described below.

In pile caps designed as beams, the reinforcement should be uniformly distributed across any given section. In pile caps designed by truss analogy, 80 % of the reinforcement should be concentrated in strips linking the pile heads and the remainder uniformly distributed throughout the pile cape. Specialist literature should be consulted for the design of "deep pile caps" (see 3.3.2.1).

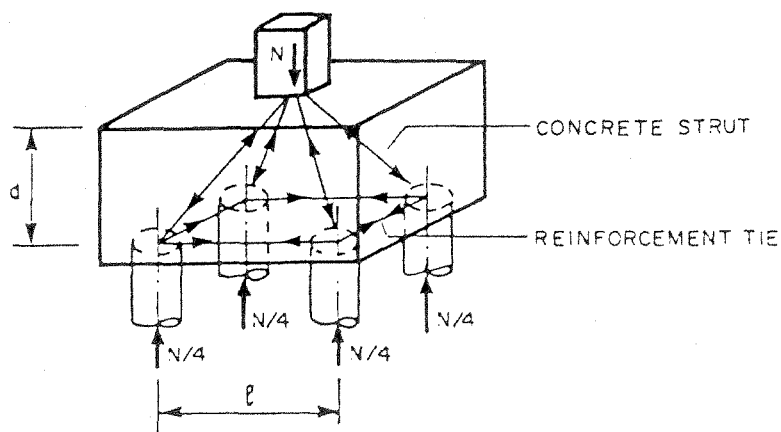


FIGURE 16
PILE CAP DESIGNED BY SPACE TRUSS ANALOGY

3.7.3.2 Shear: The design shear is the algebraic sum of all ultimate vertical loads acting on one side of, or outside, the periphery of the critical section. The shear strength of flat-slab bases in the vicinity of concentrated loads is governed by the more severe of the following two conditions:

- (a) shear along any vertical section extending across the full breadth of the base at a distance equal to the effective depth from the face of the loaded area (the provisions of 3.4.4.1 apply):
- (b) punching shear around the loaded area, in which case the provisions of 3.4.4.2 apply.

The shear strength of pile caps is governed by the more severe of the two conditions described below.

- (a) Shear along any vertical section extending across the full breadth of the cap. The provision of 3.4.4.1 apply, except that over portions of the section where the flexural reinforcement passes across the head of a pile and is fully anchored, the allowable ultimate shear stress may be increased to $(2d/a_v)k_v\xi_s v_c$

where a_v is the distance between the face of the column or wall and the critical section

d is the effective depth, to tension reinforcement, of the section.

Where a_v is taken to be the distance between the face of the column or wall and the nearer edge of the piles, it should be increased by 20 % of the pile diameter. In applying the recommendations of 3.4.4.1, the allowable ultimate shear stress should be taken as the average over the whole section.

- (b) Punching shear around loaded areas, in which case the provisions of 3.4.4.2 apply.

3.7.3.3 Bonding and anchorage: The provisions of 3.8.6 apply to reinforcement in bases. The critical sections for local bonding are:

- (a) those described in 3.8.6.2;
- (b) sections at which the depth changes or any reinforcement stops; and
- (c) those in the vicinity of piles, in which case all the bending reinforcement required to resist the pile reaction should be continued to the pile centre line and be provided with an anchorage beyond the centre line of 20 bar diameters.

3.7.4 Deflection of bases

The deflection of bases need not be considered, but the effects of differential settlement on the structure as a whole or in part shall be taken into account.

3.7.5 Crack control in bases

The recommendations of 3.8.8.2 apply as appropriate, depending on the type of bases and the design (see 3.7.3.1).

3.8 Considerations affecting Design Details

3.8.1 Construction details

3.8.1.1 Sizes of members: When deciding on the nominal overall size of a reinforced concrete member, regard should be had to the principles of dimensional co-ordination. It should be borne in mind that absolute accuracy exists only in theory and that tolerable degrees of inaccuracy have to be accepted in practice. The degree of tolerance should be as large as possible, without rendering the finished structure or any part of it unacceptable for the purpose for which it was intended.

3.8.1.2 Accuracy of position of reinforcement: In all normal cases, the design may be based on the assumption that the reinforcement is in its nominal position. However, when reinforcement is located in relation to more than one face of a member (eg a link in a beam in which the nominal cover for all sides is given), the actual concrete cover on one side may be greater and can be derived from a consideration of:

- (a) the dimensions and spacing of cover blocks, spacers and/or chairs (including the compressibility of these items and the surfaces they bear on);
- (b) the stiffness, straightness, and accuracy of cutting, bending and fixing bars of the reinforcement cage;
- (c) the accuracy of the formwork in both dimensions and plane (this includes permanent forms such as blinding or brickwork), and
- (d) the size of the structural part and the relative size of bars or the reinforcement cage.

In certain cases where bars or reinforcement cages are positioned accurately on one face of a structural member, this may affect the final position of highly stressed reinforcement at the opposite face of the member. The consequent possible reduction in effective depth to this reinforcement may exceed the percentage allowed for in the values of the partial safety factors. In the design of a particularly critical member, therefore, it may be necessary to make an appropriate adjustment to the effective depth assumed.

3.8.1.3 Construction and movement joints: When it is necessary to indicate construction joints on a drawing, careful consideration should be given to their exact location. Construction joints should generally be at right angles to the direction of the member and should take due account of the shear and other stresses. If special preparation of the joint faces is required, this should be specified.

The location of all movement joints should be clearly indicated on the drawings, both for the individual members and for the structure as a whole. Where possible, a movement joint should be in one plane through the whole structure. Reference should be made to the responsible bridge authority regarding their requirements for the design of joints.

3.8.2 Concrete cover to reinforcement

"Nominal" cover is that dimension used in design and indicated on the drawings.

The nominal cover should be not less than the size of the bar or the maximum aggregate size, plus 5 mm; in the case of a bundle of bars, (see 3.8.8.1), it should be equal to or greater than the size of a bar of equivalent area, plus 5 mm.

Cover to reinforcement should also be governed by considerations of durability under the envisaged conditions of exposure. Table 12 gives the nominal cover of dense natural-aggregate concrete which should be provided over all reinforcement, including links, when using the indicated grade of concrete under particular conditions of exposure, but subject also to the above minimum requirements. In addition, it may be necessary to specify the concrete materials and mix details to provide the required durability, such as specifying the use of sulphate-resisting cement for concrete subject to sulphate attack.

For factory-made precast components, the cover dimensions given in Table 12 may be reduced by 5 mm, but should not be less than 20 mm, except for precast piles where the cover should not be less than 30 mm. For components such as in situ piles or footings cast dry in contact with soil, the cover dimensions given in Table 12 should be increased by 25 mm; if they are cast under water in contact with soil by 30 mm; and if they are cast under water against casing, by 15 mm. In no case should the cover of in situ components cast in contact with soil or under water be less than 40 mm.

Where a surface treatment such as bush hammering cuts into the face of the concrete, the expected depth of the treatment should be added to the nominal cover.

Special care should be exercised in conditions of extreme exposure or where concrete of low density or porous aggregates are used (see 3.9.2).

3.8.3 Reinforcement: General considerations

Except where specified otherwise in these clauses, the recommendations of the *SABS Code of Practice for the Detailing of Steel Reinforcement for Concrete (SABS 0144 - 1978)* may be used as a guide where applicable.

3.8.3.1 Groups of bars: Subject to the reduction in bond stress, bars may be arranged as pairs in contact or in bundles of three or four in contact.

Bars in a bundle should terminate at different points, with the ends at least 40 times the bar size apart, except for bundles stopping at a support. Laps may be made to one bar at a time in a bundle of three, but they should be staggered so that at any cross-section there are no more than four bars in a bundle.

Bundles should not be used in a member without links.

3.8.3.2 Bending schedules: Bars should preferably be scheduled in accordance with the requirements of SABS 82 (1976 as amended). Where reinforcement is to fit between two concrete faces, the dimension on the bending schedule should be determined as the nominal dimension of the concrete less the nominal cover on each face, and less an appropriate deduction for the tolerances as specified in SABS 82.

3.8.4 Minimum areas of reinforcement in members

3.8.4.1 Minimum area of main reinforcement: The area of main tension reinforcement in a beam or slab should be not less than 0,15 % of $b_a d$ when Types B and C reinforcement (grade 450) are used or 0,25 % of $b_a d$ when Type A reinforcement (grade 250) is used, where b_a is the breadth of the section or the average breadth excluding the compression flange for non-rectangular sections, and d is the effective depth to tension reinforcement.

For a box, T or I-section, b_a should be taken as the average breadth of the concrete below the upper flange. For a voided slab, b_a should be taken as the average breadth of the concrete below the top of the voids.

For a solid concrete deck slab, the minimum effective reinforcement in each of two orthogonal directions, expressed as a percentage of the gross section, should not be less than that specified above for the main reinforcement. For other slabs, these percentages shall apply only to the main reinforcement.

The engineer shall use his discretion in specific cases in practice where the above percentages lead to unreasonable quantities of reinforcement because of the size of the member.

The minimum number of longitudinal bars provided in a column should be four in rectangular columns and six in circular columns, and their size or diameter should be not less than 12 mm. The total cross-sectional area of these bars should be not less than 1 % of the cross-section of the column, or $0,15N/f_y$, whichever is the lesser, where N is the ultimate axial load and f_y is the characteristic strength of the reinforcement.

By definition a wall cannot be considered as a reinforced concrete wall if the percentage of vertical reinforcement provided is less than 0,4 % of the gross concrete cross-sectional area, in which case it shall be considered to be a plain concrete wall (see 5.5). In reinforced concrete walls the vertical reinforcement may be in one or two layers, depending on the forces acting on the wall.

3.8.4.2 Minimum area of secondary reinforcement: In the predominantly tensile area of a solid suspended slab (other than deck slabs) or a wall, the minimum area of secondary reinforcement provided at right angles to the main reinforcement, expressed as a percentage of the gross concrete cross-sectional area, should be not less than 0,12 % of $b_t d$ when using high-yield steel, or 0,15 per cent of $b_t d$ when using mild steel.

TABLE 12 Nominal cover to reinforcement under particular conditions of exposure

Environment	Examples	Nominal cover*				
		Concrete Grade				
		20	25	30	40	50 and over
Moderate Concrete surfaces above ground level and fully sheltered against alternate wetting and drying by river water, rain and sea-spray.	Surfaces sheltered by the bridge deck, ie slab soffits, beam sides and soffits on which condensation is unlikely; surfaces protected by water-proofing or permanent formwork that will not weather or corrode; interior surfaces of pedestrian subways, voided superstructures, or cellular abutments; piers and columns on which condensation is unlikely.	50	45	40	30	25
Concrete surfaces permanently saturated by water with negligible aggressiveness** to concrete.	Concrete permanently under water.					
Severe Concrete surfaces exposed to driving rain or alternate drying and wetting by water with negligible aggressiveness** to concrete.	All external surfaces not sheltered or protected from rain; bridge-deck soffits and internal surfaces on which condensation is likely.	Concrete grade not permitted	50	45	40	35

Table 12 continued

Environment	Examples	Nominal cover*				
		Concrete Grade				
		20	25	30	40	50 and over
Concrete surfaces under ground in contact with soil.	Buried parts of structure or surfaces in contact with backfill.					
Concrete surfaces permanently saturated by flowing water with negligible aggressiveness**.	Concrete permanently under flowing water, ie abutment walls and foundations and submerged piers in rivers.					
Very severe Concrete surfaces directly affected by water that is aggressive** to concrete or by sea-spray or a salt-laden atmosphere.	Concrete parapets, walls, all exposed surfaces of super-structures and sub-structures in terrain near to the sea or in industrially polluted rivers.	Concrete grade not permitted.			60	50
Extreme Concrete surfaces exposed to abrasive action of sea-water or to water that is very aggressive** to concrete.	Parts of structures in contact with sea-water, industrially polluted water or water in marshy terrain.	Concrete grade not permitted.			65	55

* The values indicated for nominal cover are minimum values and the design engineer should round off the values used in practice to reduce the number of cover thicknesses to say three for specific works. The specific requirements of the responsible bridge authority should have preference.

** The aggressiveness of water to concrete cannot be simply defined in terms of pH values or by the degree of deficiency of calcium carbonate, as it may be passified by other constituents. Tests on suspect water should be carried out by a competent person.

In a solid slab or wall, where the main reinforcement resists compression, the area of secondary reinforcement provided should be at least 0,12 % of $b_t d$ in the case of high-yield steel and 0,15 % of $b_t d$ when using mild steel, where b_t is the width of the section and d is the effective depth to tension reinforcement. The diameter should be not less than one-quarter of the size of the main compression bars, but not less than 6 mm, and the bars should be spaced at the lesser of twelve times the size of the smallest compression bar but not exceeding 300 mm. It may also be necessary to provide links in the thickness of the wall (see 3.8.4.3).

In beams or slabs where the depth of the side face exceeds 600 mm, longitudinal reinforcement having an area of at least 0,05 % of $b_t d$ should be provided on each face, with a spacing not exceeding 300 mm. However, in flat beams or slabs the area need not exceed 0,05 % of d^2 .

In a voided slab, the amount of transverse reinforcement should exceed the lesser of the following:

- (a) in the bottom, or predominantly tensile, flange, either 1 500 mm²/m or 1,0 % of the minimum flange section;
- (b) in the top, or predominantly compressive, flange, either 1 000 mm²/m or 0,70 % of the minimum flange section.

The above-mentioned minimum flange sections required for calculating the transverse secondary reinforcement shall be taken to be in a direction parallel to that of the webs.

Additional reinforcement may be required in beams, slabs and walls to control early shrinkage and thermal cracking (see 3.8.9).

3.8.4.3 Minimum area of links: When, in a beam or column, part or all of the main reinforcement resists compression, links or ties at least one-quarter of the size of the largest compression bars should be provided at a maximum spacing of 12 times the size of the smallest compression bar. Links should be so arranged that every corner and every alternate bar or group of bars in an outer layer of reinforcement is supported by a link passing around the bar and having an included angle of not more than 135°. All other bars or groups within a compression zone should be within 150 mm of a tied bar. For circular columns, where the longitudinal reinforcement is located around the periphery of a circle, adequate lateral support is provided by a circular tie passing around the bars or groups.

When the designed percentage of reinforcement in the compression face of a wall or slab exceeds 1 %, links of at least 6 mm or one-quarter of the size of the largest compression bar, whichever is the greater, should be provided through the thickness of the member. The spacing of these links should not exceed twice the member's thickness in either of the two principal directions of the member and be not greater than 16 times the bar size in the direction of the compressive force.

In all beams, shear reinforcement should be provided throughout the span to meet the requirements given in 3.3.3.

The spacing of links should not exceed 0,75 times the effective depth of the beam (see 3.3.3.1), nor should the lateral spacing of the individual legs of the links exceed 0,75d. Links should enclose all tensile reinforcement.

3.8.5 Maximum areas of reinforcement in members

In a beam or slab, neither the area of tension reinforcement, nor the area of compression reinforcement should exceed 4 % of the gross cross-sectional area of the concrete.

In a column, the percentage of longitudinal reinforcement should not exceed 6 in vertically cast columns and 8 in horizontally cast columns, except that at laps in both types of column the percentage may be 10.

In a wall, the area of vertical reinforcement should not exceed 4 % of the gross cross-sectional area of the concrete.

3.8.6 Bond anchorage and bearing stress

3.8.6.1 Geometric classification of deformed bars: For the purposes of this Code, there are two types of deformed bar, as follows:

Type 1: A plain square twisted bar or a plain chamfered square twisted bar, each with a pitch of twist not greater than 18 times the nominal size of the bar.

Type 2: A bar with transverse ribs at a substantially uniform spacing not greater than $0,8\phi$ (and continuous helical ribs where present). The bar shall have a mean area of ribs (per unit length) beyond its core (they are being projected on a plane normal to the axis of the bar) of not less than $0,15\phi \text{ mm}^2/\text{mm}$, where ϕ is the size (nominal diameter) of the bar. The included angle of the transverse rib to the longitudinal axis of the bar shall be at least 45° .

Other bars may be classified as Types 1 or 2 from the results of performance tests. The principle of the test is to show that such bars, claimed to be equal to those satisfying the classification, will possess the specified characteristic strength in a pull-out test. The criterion of comparison is that the free-end slip of the equivalent bar is no greater than that of the geometrically defined bar. Tests shall be conducted in accordance with SABS 920, Section 6.5.

3.8.6.2 Local bond: To prevent bond failure caused by large changes in tension over short lengths of reinforcement, the local bond stress f_{bs} obtained from Equation 28 should not exceed the appropriate value obtained from Table 13.

$$f_{bs} = \frac{V \pm \frac{M}{d} \tan \theta_s}{\sum u_s d} \dots\dots\dots (28)$$

which becomes

$$f_{bs} = \frac{V}{\sum u_s d}$$

when the bars are parallel to the compression face,

where V is the shear force due to ultimate loads

M is the bending moment at the section being considered due to ultimate loads

θ_s is the angle between the compression face of the section and the tension reinforcement

$\sum u_s$ is the sum of the effective perimeters of the tension reinforcement (see 3.8.6.4)

d is the effective depth of the section.

In Equation 28 the negative sign should be used when the moment increases numerically in the same direction as the effective depth of the section.

TABLE 13 Ultimate local bond stresses

Bar type	Stress (In MPa) at concrete grades			
	20	25	30	40 or more
Plain bars	1,7	2,0	2,2	2,7
Deformed bars, Type 1	2,1	2,5	2,8	3,4
Deformed bars, Type 2	2,6	2,9	3,3	4,0

Critical sections for local bond occur at the ends of simply supported members, at points where tension bars stop and at points of contra-flexure. However, points where tension bars stop and points of contra-flexure need not be considered if the anchorage bond stresses in the continuing bars do not exceed 0,8 times the value in 3.8.6.3.

3.8.6.3 Anchorage bond: At any section of a member, appropriate embedment lengths or other end anchorages should be provided on each side of the sections for any bar in tension or compression to prevent bond failure due to ultimate loads. The anchorage bond stress is assumed to be constant over the effective anchorage length. It is taken as the force in the bar divided by the product of the

effective anchorage length and the effective perimeter of the bar or group of bars (see 3.8.6.4), and should not exceed the appropriate value obtained from Table 14.

3.8.6.4 Effective perimeter of a bar or group of bars: The effective perimeter of a single bar may be taken as 3,14 times its nominal diameter. The effective perimeter of a group of bars (see 3.8.3.1) should be taken as the sum of the effective perimeters of the individual bars, multiplied by the appropriate reduction factor given in Table 15.

3.8.6.5 Anchorage of links: A link may be considered to be fully anchored if it passes around another bar of at least its own size through an angle of 90° and continues beyond for a minimum length of eight times its own size, or through 180° and continues for a minimum length of four times its own size. In no case should the radius of any bend in the link be less than twice the radius of the test bend guaranteed by the manufacturer of the bar.

TABLE 14 Ultimate anchorage bond stresses

Bar type	Stress (in MPa) at concrete grades			
	20	25	30	40 or more
Plain bar in tension	1,2	1,4	1,5	1,9
Plain bar in compression	1,5	1,7	1,9	2,3
Deformed bar, Type 1, in tension	1,7	1,9	2,2	2,6
Deformed bar, Type 1, in compression	2,1	2,4	2,7	3,2
Deformed bar, Type 2, in tension	2,2	2,5	2,8	3,3
Deformed bar, Type 2, in compression	2,7	3,1	3,5	4,1

TABLE 15 Reduction factor for effective perimeter of a group of bars

Number of bars in a group	Reduction factor
2	0,8
3	0,6
4	0,4

3.8.6.6 Laps and joints: Continuity of reinforcement may be achieved by making a connection, using any of the following jointing methods:

- (a) lapping of bars,
- (b) butt welding (see 2.7),
- (c) sleeving (see 5.3.2.2),
- (d) threading of bars (see 5.3.2.3) or
- (e) any other method proved by tests.

Such connections should, if possible, be made away from points of high stress and should be staggered in the longitudinal direction. For joints to be considered as staggered, the distance between them must not be less than the end anchorage length for the bar.

When bars are lapped, the length of the lap should at least equal the anchorage length (derived from 3.8.6.3) required to develop the stress in the smaller of the two bars lapped.

The length of the lap provided, however, should neither be less than 25 times the size of the smaller bar plus 150 mm in tension reinforcement, nor be less than 20 times the size of the smaller bar plus 150 mm in compression reinforcement.

The lap length calculated in the preceding paragraph should be increased by a factor of 1,4 if any of the following conditions occur:

- (i) the nominal cover to the lapped bars from the top of the section as it is intended to be cast is less than twice the bar diameter;
- (ii) the clear distance between the lap and another pair of lapped bars is less than 150 mm;
- (iii) a corner bar is being lapped and the nominal cover to either face is less than twice the bar size.

Where both conditions (i) and (ii) or conditions (i) and (iii) occur, the lap length should be increased by a factor of 2,0.

The use and details of the jointing methods given in (b), (c), (d), and (e) above should be approved by the responsible bridge authority.

3.8.6.7 Hooks and bends: Hooks, bends and other reinforcement anchorages should be of such a form, dimension and arrangement as to avoid overstressing the concrete. Hooks, which should be used only to meet specific design requirements, should be of the U or L-type, as specified in SABS 82 (1976, as amended), or as approved by the responsible bridge authority.

The effective anchorage length of a hook or bend is the length of a straight bar that would be equivalent in anchorage value to that portion of the bar between the start

of the bend and a point four times the bar size beyond the end of the bend. This effective anchorage value may be taken as follows:

- (a) *For a 180° hook:* eight times the internal radius of the hook with a maximum of 24 times the bar size.
- (b) *For a 90° bend:* four times the internal radius of the bend with a maximum of 12 times the bar size.

In no case should the radius of any bend be less than twice the radius of the test bend guaranteed by the manufacturer of the bar and, in addition, the radius used should be sufficient to ensure that the bearing stress at the mid-point of the curve does not exceed the value given in 3.8.6.8.

When a hooked bar is used at a support, the beginning of the hook should be placed at least four times the bar size inside the face of the support.

3.8.6.8 Bearing stress inside the bends: In a bar that does not extend or is assumed not to be stressed beyond a point four times the bar size past the end of a bend, the bearing stress inside the bend need not be checked. In any other bar, the bearing stress inside a bend should be calculated from the equation:

$$\text{Bearing stress} = F_{bt}/r\phi$$

where F_{bt} is the tensile force due to ultimate loads in a bar or group of bars

r is the internal radius of the bend

ϕ is the size of the bar, or, in a bundle, the size of a bar of equivalent area.

The bearing stress should not exceed

$$\frac{1,5f_{cu}}{1 + \frac{2\phi}{a_b}}$$

where a_b for a particular bar or group of bars in contact should be taken as the centre-to-centre distance between bars or groups of bars perpendicular to the plane of the bend; for a bar or group of bars adjacent to the face of the member, a_b should be taken as the cover plus ϕ as defined above.

3.8.7 Curtailment and anchorage of reinforcement

3.8.7.1 General requirements for curtailment of bars: In any member subject to bending, every bar should extend (except at end supports) beyond the point at which it is no longer needed for a distance equal to the effective depth of the member, or 12 times the size of the bar, whichever is the greater. The point at which reinforcement is no longer required is that at which the resistance moment of the section, considering only the continuing bars, is equal to the required moment.

In addition, reinforcement should not be stopped within a tension zone unless one of the following conditions is satisfied:

- (a) if the bars extend for an anchorage length appropriate to their design strength ($0,87f_y$) from the point at which they are no longer required to resist bending, or
- (b) if the shear capacity at the section in which the reinforcement stops is greater than twice the shear force actually present, or
- (c) if the continuing bars at the section in which the reinforcement stops provide double the area required to resist the moment at that section.

One or more of these conditions should be satisfied for all arrangements of ultimate load considered.

At a simply supported end of a member, each tension bar should be anchored by one of the following:

- (i) an effective anchorage extending the equivalent of 12 times the bar size beyond the centre line of the support - no bend or hook should begin before the centre of the support;
- (ii) an effective anchorage extending the equivalent of 12 times the bar size plus $d/2$ from the face of the support, where d is the effective depth of the member - no bend should begin before $d/2$ from the face of the support.

3.8.8 Spacing of reinforcement

3.8.8.1 Minimum distance between bars: These recommendations are not related to bar sizes, but when a bar exceeds the maximum size of the coarse aggregate by more than 5 mm, a clear spacing smaller than the bar size should generally be avoided. A pair of bars in contact or a bundle of three or four bars in contact should be considered as a single bar of equivalent area when assessing size.

The spacing of bars should be suitable for the proper compaction of concrete. When an internal vibrator is likely to be used, sufficient space should be left between the reinforcement to enable the vibrator to be inserted. Minimum reinforcement spacing is best determined by experience or proper work tests but, in the absence of better information, the following recommendations may be used as a guide.

- (a) *Individual bars:* Except where bars form part of a pair or bundle (see (b) and (c) below), the clear distance between bars should be not less than $h_{agg} + 5$ mm, where h_{agg} is the maximum size of the coarse aggregate. Where there are two or more rows:
 - (i) the gaps between corresponding bars in each row should be in a line and
 - (ii) the clear distance between rows should be not less than h_{agg} , except for precast members when it should be not less than $\frac{2}{3}h_{agg}$.
- (b) *Pairs of bars:* Bars may be arranged in pairs either touching or closer than in (a), in which case:

- (i) the gaps between corresponding bars in each row should be in a line and of a width not less than $h_{agg} + 5 \text{ mm}$;
 - (ii) when the bars forming the pair are placed one above the other the clear distance between rows should be not less than h_{agg} , except for precast members when it should be not less than $\frac{2}{3}h_{agg}$ and
 - (iii) when the bars forming the pair are placed side by side, the clear distance between rows should be not less than $h_{agg} + 5 \text{ mm}$.
- (c) *Bundled bars*: Horizontal and vertical distances between bundles should be not less than $h_{agg} + 15 \text{ mm}$ and gaps between rows of bundles should be vertically in line.

3.8.8.2 Maximum distance between bars in tension: The maximum spacing should be not greater than 300 mm and be such that the crack widths, calculated using Equations 29 to 31 as appropriate, do not exceed the limits laid down in 2.1.2.1 under the design loadings specified in 2.2.3.

In the case of beams where the depth of the side face exceeds 600 mm, appropriate reinforcement shall be provided, in accordance with 3.8.4.2, near the faces of the web and shall be distributed in the zone of flexural tension at a vertical spacing not greater than 300 mm. Likewise, in the flanges of T-beams or box girders that are in tension, part of the main tension reinforcement shall be distributed near both faces and over the effective flange width.

- (a) For solid rectangular sections, the webs of T-beams and other solid sections shaped without re-entrant angles, the design crack width at the surface of parts of members in tension (or, where the cover to the outermost bar is greater than c_{nom} , on a surface at a distance c_{nom} from the outermost bar) should be calculated from the following equation:

$$\text{Design crack width} = \frac{3a_{cr}\epsilon_m}{1 + \frac{2(a_{cr} - c_{nom})}{(h - d_c)}} \dots\dots\dots (29)$$

where a_{cr} is the distance from the point (crack) being considered to the surface of the nearest longitudinal bar

c_{nom} is the required nominal cover to the tensile reinforcement given in Table 12 (where the cover shown on the drawing is greater than the value given in Table 12, the latter value may be used)

h is the overall depth of the section

d_c is the depth of the concrete in compression (if $d_c = 0$, the crack widths should be calculated using Equation 31)

ϵ_m is the calculated strain at the level at which cracking is being considered, allowing for the stiffening effect of the concrete in the tension zone.

A negative value of e_m indicates that the section is uncracked. e_m should be obtained from the equation:

$$\varepsilon_m = \varepsilon_1 - \Delta\varepsilon \dots \dots \dots (30)$$

where ε_1 is the calculated strain* at the level at which cracking is being considered, disregarding the stiffening effect of the concrete in the tension zone.

$$\Delta\varepsilon = \left[\frac{3,8b_1h(a' - d_c)}{\varepsilon_s A_s (h - d_c)} \right] \left[1 - \frac{M_q}{M_g} \right] 10^{-9} \text{ but may not be less than 0}$$

where b_1 is the breadth of the section at the level of the centroid of the tension steel

a' is the distance from the compression face to the point at which the crack width is being calculated

M_q is the moment at the section being considered due to live service loads

ε_s is the calculated strain in the tension reinforcement, disregarding the stiffening effect of the concrete in the tension zone

M_g is the moment at the section being considered due to permanent service loads

A_s is the cross-sectional area of tension reinforcement.

Where the axis of the design moment and the direction of the tensile reinforcement resisting that moment are not normal to each other (eg in a skew slab), the effective value of A_s should be taken as:

$$A_s = A_t \cos^4 \alpha_1$$

where A_t is the cross-sectional area of tensile reinforcement, but in a skew direction

α_1 is the angle between the normal to the axis of the design moment and the direction of the tensile reinforcement A_t resisting that moment.

Where more than one layer of reinforcement is applied in different directions:

$$A_s = \sum_{i=1}^m A_i \cos^4 \alpha_i$$

where m is the number of steel layers

A_i is the cross-sectional area of the i th layer of steel

α_i is the angle between the normal to the axis of the principle moment and the direction of the i th steel layer.

*For the appropriate modulus of elasticity, see 2.3.2.1(b).

- (b) For flanges in overall tension, including tensile zones of box beams and voided slabs, the design crack width at the surface (or at a distance c_{nom} from the outermost bar) should be calculated from the following equation:

$$\text{Design crack width} = 3a_{cr}\epsilon_m \quad (31)$$

where ϵ_m is obtained from Equation 30.

- (c) Where global and local effects are calculated separately (see 2.8.3), the value of ϵ_m may be obtained by algebraic addition of the strains calculated separately. The design crack width should then be calculated in accordance with (b) but may, in the case of a deck slab, where a global compression is being combined with a local moment, be obtained by using (a), calculating d_c on the basis of the local moment only.
- (d) The spacing of transverse bars in slabs with circular voids should not exceed twice the minimum flange thickness.

3.8.9 Reinforcement for shrinkage and thermal effects

To prevent excessive cracking due to shrinkage and thermal movement, reinforcement should be provided in the direction of any restraint to such movements. For full restraint, the area of reinforcement, calculated as a percentage of the gross concrete section at right angles to the direction of each restraint, should be not less than 0,5 % for Types B and C (grade 450) reinforcement, or 0,6 % for Type A (grade 250) reinforcement.

When the minimum percentages of reinforcement are calculated for cross-sections with dimensions exceeding 500 mm in both directions, half the area of the core of the cross-section more than 250 mm away from all concrete surfaces should be disregarded. In slabs and walls, the reinforcement should be placed near the relevant surface in both directions with adequate concrete cover, or distributed between the two surfaces as required. Reinforcement that is present for other purposes may be taken into account for the purpose of this clause. Where the restraint is partial, the reinforcement may be reduced accordingly, but shall comply with other minimum reinforcement requirements and will be subject to the approval of the responsible bridge authority. Except where permanent bending moments cause eccentric stress distributions, shrinkage and temperature reinforcement should be distributed uniformly around the perimeter of the concrete section and spaced at not more than 300 mm.

3.8.10 Arrangement of reinforcement in skew slabs

3.8.10.1 General: In all types of skew slab for which the moments and torsions have been determined by an elastic analysis, the reinforcement or prestressing tendons should be aligned as close as is practicable to the principal moment directions. In general, an orthogonal arrangement is recommended.

3.8.10.2 Solid slabs: Only for combinations of a large skew angle and a low ratio of skew width to skew span is it preferable to place reinforcement in directions perpendicular and parallel to the free edges. Usually it is more efficient to place reinforcement parallel and perpendicular to the supports, preferably in combination with bands of reinforcement positioned adjacent and parallel to the free edges.

Special attention should be given to providing adequate anchorage for bars that meet the free edge at an angle.

An alternative, but less efficient method, is to fan out the longitudinal steel, starting perpendicular to the supports and ending parallel to the free edge at the edge.

3.8.10.3 Voided slabs: The longitudinal steel will generally be placed parallel to the voids and it is recommended that the transverse steel be placed orthogonally to this steel.

3.8.10.4 Solid composite slabs: The longitudinal steel will generally be in the form of prestressing tendons placed parallel to the free edges in precast units. Ideally, the transverse reinforcement should be placed at right angles to the free edge, since this is the most efficient arrangement; however, in practice, the transverse reinforcement will often have to be placed at a different angle or parallel to the supports.

3.9 Additional Consideration in the Use of Concrete Containing Low-Density Aggregate

3.9.1 General

Concrete containing low-density aggregate may generally be designed in accordance with the recommendations of Section 2 and 3.1 to 3.8. Subsections 3.9.2 to 3.9.11 relate specifically to reinforced concrete containing low-density aggregate of grade 25 or above. Only the recommendations of 5.5 (plain concrete walls) apply to concretes below grade 25.

In considering concrete containing low-density aggregate, the properties of any particular type of aggregate can be established with far more accuracy than for most naturally occurring materials, and the engineer should therefore obtain specific data directly from the aggregate producer. The properties of concrete containing low-density aggregate to be used in design should be supported by appropriate test data and should be agreed on with the responsible bridge authority.

3.9.2 Durability

The minimum cement contents required for concrete made with ordinary aggregates also apply to concrete containing low-density aggregate. For durability, the cover to reinforcement should be 10 mm greater than the values given in Table 12.

3.9.3 Characteristic strength

Values of the characteristic strength of low-density-aggregate concretes may be taken from Table 5. When all the aggregate in the concrete is sintered pulverized fuel ash, the related cube strength at other ages may be obtained from Table 5 or, in the case of grade 15 from the values in Table 5 for grade 20 reduced by 25 %. These values apply to most other types of aggregate, but reference should be made to the producer of the particular material under consideration. With some aggregates used in rich mixes, there may be little increase in strength beyond that attained at 28 days.

3.9.4 Shear resistance of beams

The shear resistance and shear reinforcement for concrete beams containing low-density aggregate should be established in accordance with 3.3.3.1 and 3.3.3.2, except that Table 16 should be used in place of Table 7 and the maximum shear stress, v , should be limited to the values given in Table 17.

TABLE 16 Ultimate shear stress, v_c , in concrete beams containing low-density aggregate

$\frac{100A_s}{bd}$	Stress (in MPa) at concrete grades		
	25	30	40 or more
$\leq 0,25$	0,28	0,28	0,28
0,50	0,40	0,44	0,44
1,00	0,52	0,56	0,60
2,00	0,68	0,72	0,76
$\geq 3,00$	0,72	0,76	0,80

TABLE 17 Maximum value of shear stresses in concrete beams containing low-density aggregate

Maximum stress (in MPa) at concrete grades		
25	30	40 or more
3,00	3,28	3,80

3.9.5 Torsional resistance of beams

The torsional resistance and reinforcement for concrete beams containing low-density aggregate should be established in accordance with 3.3.4, except that Table 18 should be used in place of Table 9.

TABLE 18 Ultimate torsional shear stresses in concrete beams containing low-density aggregate

	Stress (in MPa) at concrete grades		
	25	30	40 or more
V_{tmin}	0,26	0,30	0,34
V_{tu}	3,00	3,28	3,80

3.9.6 Deflection of beams

Deflection of concrete beams containing low-density aggregate may be calculated using a value of E_c as described in 2.3.2.1.

3.9.7 Shear resistance of slabs

The shear resistance of and shear reinforcement for concrete slabs containing low-density aggregate should be established in accordance with 3.4.4, except that Table 16 should be used in place of Table 7 and the maximum shear stress, v , should be limited to the values given in Table 17.

3.9.8 Deflection of slabs

Deflection of concrete slabs containing low-density aggregate may be calculated using a value of E_c as described in 2.3.2.1.

3.9.9 Columns

3.9.9.1 General: The recommendations of 3.5 apply to concrete columns containing low-density aggregate, subject to the conditions in 3.9.9.2 and 3.9.9.3.

3.9.9.2 Short columns: In 3.5.1.1 the ratio of effective height l_e to thickness h for a short column should not exceed 10.

3.9.9.3 Slender columns: In 3.5.5 the divisor 1 750 in Equations 21, 23, 24 and 25 should be replaced by the divisor 1 200.

3.9.10 Local bond, anchorage bond and laps

Local bond stresses, anchorage bond stresses and lap lengths in reinforcement for concrete members containing low-density aggregate should be established in accordance with 3.8.6, except that the bond stresses for plain and deformed bars

should not exceed 50 % and 80 % respectively of those given in Table 13 and 14.

For foamed slag or similar aggregates, it may be necessary to ensure that bond stresses are kept well below the maximum values given in the preceding paragraph for reinforcement that is in a horizontal position during casting, and values should be obtained from test data.

3.9.11 Bearing and stress inside bends

The recommendations of 3.8.6.8 apply to low-density-aggregate concrete, except that the bearing stress should not exceed

$$\frac{f_{cu}}{1 + \frac{2\phi}{a_b}}$$

4 DESIGN AND DETAILING: PRESTRESSED CONCRETE, INCLUDING PARTIALLY PRESTRESSED CONCRETE

4.1 General

4.1.1 Introduction

This section gives method of analysis and design that will in general ensure that, for prestressed concrete construction of classes 1, 2 and 3, as defined in 2.1.2.1, the requirements set out in Section 2 are met. Other methods may be used provided they can be shown to be satisfactory for the type of structure or member considered. In certain cases the assumptions made in this section may be inappropriate and the engineer should adopt a more suitable method, having regard to the nature of the structure in question.

Section 4 does not apply to prestressed concrete construction using any of the following in the permanent works:

- (a) unbonded tendons;
- (b) external tendons (a tendon is considered external if, after stressing and incorporation into the permanent works, but before protection, it is outside the concrete section) and
- (c) low-density aggregate.

4.1.2 Limit state design of prestressed concrete

4.1.2.1 Basis of design: This Chapter follows the limit state philosophy set out in Sections 2 and 6 of Part 1, and Section 2 of this Part. However, as it is not possible to assume that a particular limit state will always be the critical one, design methods are given for both the ultimate and serviceability limit states.

In general, the design of Class 1 and 2 members is governed by the *cracking and concrete stress limitations* for serviceability load conditions, but the ultimate strength in flexure, shear and torsion should be checked. The design of Class 3 members is usually governed by ultimate strength conditions, but crack widths and deflections at serviceability limit states should also be checked, as specified in 4.3.2.4(iii).

4.1.2.2 Durability: Guidance is given in 4.9.2 on the minimum cover to reinforcement and prestressing tendons that should be provided to ensure durability.

4.1.2.3 Other limit states and considerations: Section 4 does not make recommendations concerning vibration or other limit states. For these and other considerations, reference should be made to 2.1.2.2 and 2.1.3.

4.1.3 Actions and resistance

In Section 4 the design action effects or design load effects, including the effects of imposed deformations and restraints (see 6.3 of Part 1), for the ultimate serviceability limit states are referred to as "ultimate actions" or "ultimate loads" and "service actions" or "service loads" respectively.

The values of the "ultimate actions" or "ultimate loads" and "service actions" or "service loads" to be used in design are derived from Section 2 and 2.2 of this Part.

Similarly, the design strength of a section is also referred to as the "ultimate resistance" or "ultimate capacity" (see 6.4 of Part 1).

Consideration should be given to the construction sequence and to the secondary effects caused by prestress (but refer also to 2.2).

4.1.4 Strength of materials

4.1.4.1 Definition of strengths: In this Section the design strengths of materials are expressed in all the tables and equations in terms of the characteristic strength of the materials. *Unless specifically stated otherwise, all equations and tables include allowances for γ_m , the partial safety factor for material strength.*

4.1.4.2 Characteristic strength of concrete: The characteristic 28-day cube strengths of concrete that may be specified for prestressed concrete are given in Table 19, together with their related cube strengths at other ages. The values given in Table 19 do not include any allowance for γ_m .

Design should be based on the 28-day characteristic cube strength and, if appropriate for any specific loading combination, on the characteristic strength given in Table 19 for the age at which loading takes place. At transfer the calculations should be based on the cube strength at transfer.

TABLE 19 Strength of concrete (ordinary Portland cement)

Grade	Character- istic cube strength f_{cu} (in MPa) at 28 days	Characteristic cube strength (in MPa) at the age of				
		7 days	2 months*	3 months*	6 months*	1 year*
30	30,0	19,0	32,0	34,0	35,0	36,0
40	40,0	27,0	42,5	44,0	46,0	48,0
50	50,0	35,0	52,5	54,0	56,0	58,0
60	60,0	43,5	62,5	64,0	66,0	68,0

* Increased strengths at these ages should be used only if it has been demonstrated to the satisfaction of the engineer that the materials to be used are capable of producing these higher strengths.

4.1.4.3 Characteristic strength of prestressing tendons: The specified characteristic strengths of prestressing tendons, f_{pu} , are given in Tables 20, 21, 22 and 23. *The values given in these tables do not include any allowance for γ_m .* When patented systems are used as tendons, the characteristic strengths shall be based on tests.

4.2 Structures and Structural Frames

4.2.1 Analysis of structures

Complete structures and complete structural frames may be analysed in accordance with the recommendations of 2.4 but, when appropriate, the methods given in 4.3 may be used for the design of individual members.

The relative stiffness of members should generally be based on that of the concrete section as described in 2.4.3.1.

4.2.2 Redistribution of moments

Redistribution of moments obtained by rigorous elastic analysis under the ultimate limit state may be carried out, provided the following conditions are met:

- (a) Checks must be made to ensure that there is adequate rotation capacity at sections where moments are reduced, making reference to appropriate test data.

In the absence of a special investigation, the plastic rotation capacity may be taken as the lesser of:

$$(i) \quad 0,008 + 0,035 \left(0,5 - \frac{d_c}{d_o} \right)$$

or

$$(ii) \quad \frac{10}{d - d_c}$$

but not less than 0,0 or more than 0,015

where d_c is the calculated depth of concrete in compression at the ultimate limit state (in mm)

d_o is the effective depth for a solid slab or rectangular beam, otherwise the overall depth of the compression flange (in mm)

d is the effective depth to tension reinforcement (in mm)

TABLE 20 Specified characteristic strengths of prestressing wire

Nominal size (mm)	Tensile range (MPa)	Specified characteristic force (kN)	Nominal cross-sectional area (mm ²)
2,65	1 400/1 550	7,72	5,51
2,65	1 550/1 700	8,55	5,51
2,65	1 700/1 850	9,38	5,51
3,00	1 400/1 550	9,9	7,06
3,00	1 550/1 700	10,9	7,06
3,00	1 700/1 850	12,0	7,06
3,15	1 400/1 550	10,9	7,79
3,15	1 550/1 700	12,0	7,79
3,15	1 700/1 850	13,2	7,79
3,66	1 400/1 550	14,7	10,5
3,66	1 550/1 700	16,3	10,5
3,66	1 700/1 850	17,8	10,5
4,00	1 400/1 550	17,5	12,5
4,00	1 550/1 700	19,4	12,5
4,00	1 700/1 850	21,3	12,5
5,00	1 400/1 550	27,4	19,6
5,00	1 550/1 700	30,4	19,6
5,00	1 700/1 850	33,3	19,6
6,00	1 400/1 550	39,5	28,2
6,00	1 550/1 700	43,8	28,2
6,00	1 700/1 850	48,0	28,2
7,00	1 400/1 550	53,8	38,4
7,00	1 550/1 700	59,6	38,4
7,00	1 700/1 850	65,4	38,4
8,00	1 400/1 550	70,3	50,2
8,00	1 550/1 700	77,9	50,2
8,30	1 400/1 550	75,7	54,1
8,30	1 550/1 700	83,8	54,1
9,00	1 400/1 550	89,0	63,6
9,00	1 550/1 700	98,6	63,6

TABLE 21 Specified characteristic strengths of prestressing strand (conventional)

Nominal size (mm)	Specified characteristic force (kN)	Minimum 0,2 % proof force (kN)	Nominal cross-sectional area (mm ²)*	No of wires
6,4	44,5	37,8	24,4	7
7,9	69,0	58,7	37,2	7
9,3	93,5	79,5	51,6	7
10,9	125	106	71,2	7
12,5	165	140	93,4	7
15,2	237	201	138,0	7

*Calculated from the nominal wire size

TABLE 22 Specified characteristic strengths of 7-HI-prestressing strand

Nominal size (mm)	Specified characteristic force (kN)	Minimum 0,2 % proof force (kN)	Nominal cross-sectional area (mm ²)*	No of wires
6,4	49,0	41,7	26,1	7
7,9	73,5	62,5	40,1	7
9,3	102	87,1	55,4	7
10,9	137	117	76,4	7
12,5	182	155	100	7
15,2	249	211	138	7

*Calculated from the nominal wire size

TABLE 23 Specified characteristic strengths of prestressing bars

Nominal size (mm)	Specified characteristic strength, $A_{ps} \times f_{pu}$ (kN)	Nominal cross-sectional area, A_{ps} (mm ²)
*20	325	314
22	375	380
*25	500	491
28	625	615
*32	800	804
35	950	961
*40	1 250	1 257

*Preferred sizes

- (b) Proper account must be taken of the changes in transverse moments, transverse deflections and transverse shears that will be brought about by the redistribution of longitudinal moments. This is done by means of an appropriate non-linear analysis.
- (c) The shears and reactions used in design must be those calculated either before or after redistribution, whichever are the greater.
- (d) Where the depth of the members or elements considered is greater than 1,0 m, the degree of redistribution should be proportionately reduced with no redistribution for depths greater than 1,5 m unless appropriate test data are available.

4.2.3 Reduction of secondary moments

The effects of secondary moments in continuous prestressed members due to the actions described in 4.3 or Part 2 may be reduced at the ultimate limit state, provided that the presence of the unreduced secondary moments does not alter the nature of the inelastic behaviour by inducing local sectional failure instead of and before overall structural collapse in the assumed or calculated mode.

4.3 Beams

4.3.1 General

4.3.1.1 Definitions: The definitions and limits of the geometric properties for the prestressed beams are given as for reinforced concrete beams in 3.3.1.

4.3.1.2 Slender beams: In addition to limiting the slenderness of a beam (see 3.3.1.3) when under load in its final position, the engineer should consider the possible instability of a prestressed beam during erection.

Members may collapse by tilting about a longitudinal axis through the lifting points. This initial tilting, which may be due to imperfections in beam geometry and in locating the lifting points, could cause lateral bending moments and these, if too high, could result in lateral instability. The problem is complex and previous experience should be relied on when dealing with a particular case. The following factors may require consideration:

- (a) beam geometry, ie type of cross-section, span: breadth: depth ratios, etc
- (b) location of the lifting points
- (c) method of lifting, ie inclined or vertical slings, type of connection between the beams and slings, etc,
- (d) tolerance in construction, eg maximum lateral bow and
- (e) location of the support points and the possibility of an unstable support condition before the deck is cast in situ.

The stresses due to the combined effects of lateral bending, dead load and prestress can be assessed and, if it is found that cracking is possible, the lifting arrangements should be changed or the beam should be provided with adequate lateral support.

4.3.2 Serviceability limit state: Flexure

4.3.2.1 Analysis of sections: The following assumptions may be used when considering the effects of design loads:

- (a) Plane sections remain plane.
- (b) The elastic modulus of concrete may be taken as that given in 2.3.2.1. The concrete is assumed to exhibit elastic behaviour up to the stresses given in 4.3.2.2 and 4.3.2.4. For Class 3 members with calculated tensile stresses above those given in Table 26, the concrete is assumed to be cracked; concrete flexural tensile stresses are then disregarded with the total tensile force being resisted by the prestressing steel and any additional reinforcement as specified in 4.3.2.4(a)(i).
- (c) In general, it may be necessary to calculate only the stresses due to the load combinations given in Part 2, immediately after the transfer of prestress and also after all losses of prestress have occurred; in both cases the effects of dead and imposed loads on the strain and force in the tendons may be disregarded up to the theoretical cracking stresses given in Table 26.

4.3.2.2 Concrete compressive stress limitation

- (a) *Under service loads:* The compressive stresses in the concrete under the loads given in 2.2 should not exceed the values in Table 24. Higher stresses are permissible for prestressed members used in composite construction (see 5.4.3.2).

TABLE 24 Compressive stresses in concrete for serviceability limit states

Nature of loading	Allowable compressive stresses
Design load in bending with triangular or near-triangular stress distribution in sections of approximately uniform breadth	$0,40f_{cu}$
Intermediate between triangular and uniform compressive stress distribution (eg combined bending and axial loading in members varying from those with approximately uniform breadth to those with flanges).	Interpolate between $0,40f_{cu}$ and $0,30f_{cu}$
Design load in direct compression with uniform or near-uniform stress distribution (eg in all sections due to axial loading or in wide flanges of beams and box girders due to bending).	$0,30f_{cu}$

TABLE 25 Allowable compressive stresses at transfer

Nature of stress distribution	Allowable compressive stresses
Triangular or near-triangular distribution of prestress	$0,50f_{ci}$ but $\leq 0,40f_{cu}$
Intermediate between triangular and uniform compressive stress distributions	Interpolate between $0,50f_{ci}$ and $0,40f_{ci}$; and $0,40f_{cu}$ and $0,30f_{cu}$ respectively
Uniform or near-uniform distribution of prestress	$0,40f_{ci}$ but $\leq 0,30f_{cu}$

- (b) *At transfer:* The compressive stresses in the concrete at transfer should not exceed the values given in Table 25, where f_{ci} is the concrete strength at transfer.

4.3.2.3 Steel stress limitations: The stress in the prestressing tendons under the loads given in 2.2 need not be checked for Class 1 and Class 2 members; the stresses and strains in Class 3 members should be checked in accordance with 4.3.2.1. The stress at transfer should be checked in accordance with 4.8.1.

4.3.2.4 Cracking

- (a) *Under service loads:* The requirements of 2.1.2.1 are deemed to be satisfied, provided that the flexural tensile stresses or stress increments of cracked sections under the loading given in 2.2.3 do not exceed the values given in (i), (ii) or (iii) below for the appropriate class (but see 5.3.4 for joints in post-tensioned segmental construction).

However, most prestressed concrete structural elements are stressed in one direction only and are thus a combination of reinforced and prestressed concrete or are monolithically connected with reinforced concrete elements such as transverse spanning slabs which form the flanges of a prestressed concrete beam. Cracking of such reinforced elements, especially for Class 1 and Class 2 members, should be limited in accordance with Table 1, so as not to affect the durability of the structure adversely.

Due allowance should be made for the cracking behaviour of concrete under biaxial or triaxial stress conditions, ie when concrete is subjected to compression, the tensile strength is reduced in the directions perpendicular to the compressive stress direction.

- (i) *Class 1 members (full prestress)*

No tensile stress, except as indicated in 4.3.2.4(b)(i).

TABLE 26 Flexural tensile stresses for Class 2 members for serviceability limit state to control cracking

	Allowable stress (in MPa) for concrete grade			
	30	40	50	60
Pretensioned members	–	2,9	3,2	3,5
Post-tensioned members	2,0	2,3	2,55	2,8

(ii) *Class 2 members (limited prestress)*

The tensile stresses should not exceed the design flexural tensile strength of the concrete which is $0,45\sqrt{f_{cu}}$ for pretensioned members and $0,36\sqrt{f_{cu}}$ for post-tensioned members. The limiting tensile stresses are given in Table 26.

(iii) *Class 3 members (partial prestress)*

The member sections shall be checked under cracked conditions to ensure that the stress increments in both the prestressing steel and any additional reinforcement near the tensile face of the concrete do not exceed 75 MPa in environments with permissible crack widths of 0,10 mm, and 150 MPa in environments with permissible crack widths of 0,20 mm or greater (see Table 1). These stress increments are the difference between the steel stresses prevailing when the concrete is cracked under the action of the design load and those prevailing when decompression of the concrete commences at the level of the reinforcement near the tensile face. In terms of the assumptions made in 4.3.2.1, the stiffening effects of the concrete in the tension zone are disregarded. This will generally lead to conservative results. Where adequate test data are available, the above-mentioned limits may be exceeded, subject to the approval of the responsible bridge authority.

The stress ranges due to repeated live loading should be limited as specified in 2.7(ii).

(b) *At transfer and during construction:* The flexural tensile stress in the concrete should not exceed the following values (but see 5.3.4 for joints in post-tensioned segmental construction):

- (i) 1 MPa for stress due solely to prestress and coexistent dead and temporary loads during erection (see Part 2).
- (ii) $0,45\sqrt{f_{c1}}$ for pretensioned members and $0,36\sqrt{f_{c1}}$ for post-tensioned members for stress due to all other load combinations. Members with pretensioned tendons should have some tendons or additional reinforcement well distributed throughout the tensile zone of the section.

Members with post-tensioned tendons should, if necessary, have additional reinforcement located near the tension faces of the member.

The engineer should use his discretion depending on the particular case, but as a guide the following is suggested:

The additional reinforcement in any part of the tensile zone that exceeds 300 mm x 300 mm in cross-section should be not less than 0,10 % when using high-yield or cold-worked steel reinforcement (grade 450) or 0,15 % when using mild steel reinforcement (grade 250). No additional reinforcement is required within a distance of 200 mm of prestressing tendons.

4.3.3 Ultimate limit state: Flexure

4.3.3.1 Analysis of sections: When assessing the strength of sections at the ultimate limit state, the following assumptions should be used:

- (a) Plane sections are assumed to remain plane when the strain distribution in the concrete in compression and the strains in bonded prestressing tendons and any additional reinforcement, whether in tension or compression, are being derived. In addition, the tendons will have an initial strain due to prestress after all losses.
- (b) The stresses in the concrete in compression can be derived from the stress-strain curve given in Figure 1 with $\gamma_m = 1,5$.
- (c) The tensile strength of the concrete can be disregarded.
- (d) The stresses in bonded prestressing tendons, whether initially tensioned or untensioned, and in additional reinforcement, can be derived from the appropriate stress-strain curves with $\gamma_m = 1,15$; the stress-strain curves for prestressing tendons are given in Figures 3 and 4, and those for reinforcement in Figure 2. An empirical approach for obtaining the stress in the tendons at failure is given in 4.3.3.3 and Table 27.

Alternative procedures may be adopted, namely
either

- (i) The ultimate moment of resistance is determined by taking the maximum concrete strain at the outermost fibre at failure as 0,0035. If the ultimate moment of resistance is found to be less than 1,15 times the required value, the section should be proportioned such that the strain in the outermost tendons is not less than:

$$0,005 + \frac{f_{pu}}{E_s \gamma_m}$$

where E_s is the modulus of elasticity of the steel

Where the outermost tendon, or layer of tendons forms less than 25 % of the total tendon area, this condition should also be met within the outermost 25 % of tendon area.

In the case of rectangular sections and in flanged, ribbed and voided sections where the neutral axis lies within the flange, assumption (b) may be adapted so that the compressive stress is taken to be equal to $0,4f_{cu}$ over the whole compression zone.

or

- (ii) The strains in the concrete, the bonded prestressing tendons and in any additional reinforcement caused by the application of ultimate loads are calculated. The calculated strain at the outermost compression fibre of the concrete should not exceed 0,0035. In addition, the section should be proportioned such that the strain at the centroid of the outermost 25 % of the cross-sectional area of the tendons is not less than

$$0,005 + \frac{f_{pu}}{E_s \gamma_m}$$

except where the requirement for the calculated strain in the concrete, due to the application of 1,15 times the ultimate load, can be satisfied.

4.3.3.2 Design charts: The design charts that form Part 3 of the *BS Code of Practice - CP110 (1972)* Part 3 include those based on Figures 1, 3 and 4, as well as on various assumptions that do not fully comply with the requirements of this Code. The charts may nevertheless be used for the design of rectangular prestressed beams, but the calculations must be checked for compliance with the requirements of this Code for minimum strain in the tendons.

4.3.3.3 Design formula: In the absence of an analysis based on the assumptions given in 4.3.3.1, the resistance of a prestressed rectangular beam, or of a flanged beam in which the neutral axis lies within the flange, may be obtained from Equation 32.

$$M_u = f_{pb} A_{ps} (d - 0,5x) \dots\dots\dots (32)$$

where M_u is the ultimate moment of resistance of the section

f_{pb} is the tensile stress in the tendons at failure

A_{ps} is the cross-sectional area of the prestressing tendons in the tension zone

d is the effective depth to the tension reinforcement

x is the neutral axis depth.

TABLE 27 Conditions at the ultimate limit state for rectangular beams with pretensioned tendons or with post-tensioned tendons having effective bond

$\frac{f_{pu}A_{ps}}{f_{cu}bd}$	Stress in tendons as a proportion of the design strength, $\frac{f_{pb}}{0,87f_{pu}}$		Ratio of depth of neutral axis to that of the centroid of the tendons in the tension zone, x/d	
	Pretensioning	Post-tensioning with effective bond	Pretensioning	Post-tensioning with effective bond
0,025	1,0	1,0	0,054	0,054
0,05	1,0	1,0	0,109	0,109
0,10	1,0	1,0	0,217	0,217
0,15	1,0	1,0	0,326	0,326
0,20	1,0	0,95	0,435	0,414*
0,25	1,0	0,90	0,542	0,480*
0,30	1,0	0,85*	0,655	0,558*
0,40	0,9	0,75	0,783*	0,653*

* The neutral axis depth in these cases is too low to provide the elongation given in 4.3.3.1. It is essential, therefore, that the strength provided should exceed that prescribed by 15 %.

Values for f_{pb} and x may be derived from Table 27 for pretensioned members and for post-tensioned members with effective bond between the concrete and the tendons, provided that the effective prestress after all losses is not less than $0,45f_{pu}$. Prestressing tendons and additional reinforcement in the compression zone are disregarded in strength calculations when using this method.

4.3.3.4 Partially prestressed (Class 3) and non-rectangular sections: All sections that are partially prestressed and/or have additional reinforcement, as well as all non-rectangular beams, should be analysed using the assumptions given in 4.3.3.1.

4.3.4 Shear resistance of beams

4.3.4.1 General: Calculations for shear are required only for the ultimate limit state. The provisions of these Clauses apply to Class 1, Class 2 and Class 3 prestressed concrete members.

At any section the ultimate shear resistance of the concrete alone, V_c , should be considered for the section both uncracked (see 4.3.4.2) and cracked (see 4.3.4.3) in flexure, and shear reinforcement provided as required in 4.3.4.4.

For a cracked section, the conditions of maximum shear with coexistent bending moment and maximum bending moment with coexistent shear should both be considered.

Within the transmission length of pretensioned members (see 4.8.4), the shear resistance of a section should be taken as the greater of the values calculated from:

- (a) 3.3.3, except that in determining the area A_s the area of tendons should be disregarded and
- (b) 4.3.4.2 and 4.3.4.4, using the appropriate value of prestress at the section considered, assuming a linear variation of prestress over the transmission length.

4.3.4.2 Sections uncracked in flexure: It may be assumed that the ultimate shear resistance of a section uncracked in flexure, V_{co} , corresponds to the occurrence of a maximum principal tensile stress, at the centroidal axis of the section, of $f_t = 0,24\sqrt{f_{cu}}$.

In the calculation of V_{co} , the value of f_{cp} should be derived from the prestressing force after all losses have occurred, multiplied by the appropriate value of γ_{il} (see 2.2.2).

The value of V_{co} is given by

$$V_{co} = 0,67bh\sqrt{(f_t^2 + f_{cp}f_t)} \dots\dots\dots (33)$$

where b^* is the breadth of the members, which for T, I and L-beams should be replaced by the breadth of the rib b_w

h is the overall depth of the member

f_{cp} is the compressive stress at the centroidal axis due to prestress, taken as positive

f_t is $0,24\sqrt{f_{cu}}$ taken as positive

In flanged members where the centroidal axis occurs within the flange, the principal tensile stress should be limited to $0,24\sqrt{f_{cu}}$ at the intersection of the flange and web; in this calculation, the algebraic sum of the stress due to the bending moment under ultimate loads and the stress due to prestress at this intersection should be used in calculating V_{co} .

For a section with inclined tendons, the component of prestressing force (multiplied by the appropriate value of γ_{il}) normal to the longitudinal axis of the member should be algebraically added to V_{co} . This component should be taken as positive when the shear resistance of the section is increased.

* Where the position of a duct coincides with the position of the maximum principal tensile stress, eg at or near the junction of a flange and a web near a support, the value of b should be reduced by the full diameter of the duct if ungrouted and by two-thirds of the diameter if grouted.

4.3.4.3 Sections cracked in flexure

- (a) *Classes 1 and 2:* The ultimate shear resistance of a section cracked in flexure, V_{cr} , may be calculated using Equation 34.

$$V_{cr} = 0,037bd\sqrt{f_{cu}} + \frac{M_{cr}}{M}V \dots\dots\dots (34)$$

provided that M is taken as not less than Vd ,

where d is the distance from the extreme compression fibre to the centroid of the tendons at the section being considered

M_{cr} is the cracking moment at the section being considered

$$M_{cr} = (0,37\sqrt{f_{cu}} + f_{pt})I/y$$

in which f_{pt} is the tensile stress due to prestress only at the extreme tensile fibre distance y from the centroid of the concrete section which has a second moment of area I ; the value of f_{pt} should be derived from the prestressing force after all losses have occurred, multiplied by the appropriate value of γ_{fl} (see 2.2.2)

V and M are the shear force and bending moment (both taken as positive) at the section being considered due to ultimate loads

V_{cr} should be taken as not less than $0,1bd\sqrt{f_{cu}}$

- (b) *Class 3:* The ultimate shear resistance of a section cracked in flexure, V_{cr} , may be calculated using Equation 35.

$$V_{cr} = (1 - 0,55 \frac{f_{pe}}{f_{pu}})v_cbd + M_o \frac{V}{M} \dots\dots\dots (35)$$

provided that M is taken as not less than Vd ,

where f_{pe} is the effective prestress in a tendon after all losses have occurred, multiplied by the appropriate values of γ_{fl} (see 2.2.2). For the purposes of this equation, f_{pe} should be not greater than $0,6f_{pu}$

v_c is obtained from Table 7

A_s (required in using Table 7) should be taken as the cross-sectional area of steel in the tension zone, irrespective of its characteristic strength

d is the distance from the compression face to the centroid of the steel area A_s

M_o is the moment necessary to produce zero stress (decompression) in the concrete at the depth d :

$$M_o = f_{pt}I/y$$

in which f_{pt} is the tensile stress due to prestress only at the depth d and distance y from the centroid of the concrete section which has a second moment of area I ; the value for f_{pt} should be derived from the prestressing force after all losses have occurred multiplied by the appropriate value of γ_{IL} (see 2.2.2).

For cases where both tensioned and untensioned steel are contained in A_s , f_{pc}/f_{pu} may be taken as

$$\frac{P_i}{A_{s(t)}f_{pu(t)} + A_{s(u)}f_{yL(u)}}$$

where P_i is the effective prestressing force after all losses

$A_{s(t)}$ is the area of tensioned steel

$A_{s(u)}$ is the area of untensioned steel

$f_{pu(t)}$ is the characteristic strength of the tensioned steel

$f_{yL(u)}$ is the characteristic strength of the untensioned steel.

V_{cr} should be taken as not less than $0,1bd\sqrt{f_{cu}}$.

The value of V_{cr} calculated using Equations 34 or 35 at a particular section may be assumed to be constant for a distance equal to $d/2$, measured in the direction of increasing moment, from the particular section.

For a section cracked in flexure and with inclined tensions, the component of prestressing force normal to the longitudinal axis of the member should be reduced by 50 % as well as being multiplied by $\gamma_{IL} = 0,87$ if it would theoretically have a beneficial effect.

4.3.4.4 Shear reinforcement: Shear reinforcement should be provided such that

$$\frac{A_{sv}}{s_v} \geq \frac{V - V_c}{0,87f_{yv}d_t(\cot \theta_c + \cot \theta_v)\sin \theta_v} \dots\dots\dots (36a)$$

provided that, to satisfy minimum reinforcement requirements, it is assumed that

$$\frac{V - V_c}{bd_t} \leq 0,4 \text{ MPa} \dots\dots\dots (37a)$$

and that, where links are used, the area of ΔA_s , the additional effectively anchored longitudinal tensile reinforcement (see 3.8.6.2 and 3.8.7) and prestressing tendons (excluding deboned tendons), should be such that

$$\Delta A_s \geq \frac{V(\cot \theta_c - \cot \theta_v)}{2(0,87f_{yL})} \dots\dots\dots (38a)$$

where A_{sv} is the sum of the cross-sectional areas of the legs of a link or composite link

s_v is the link spacing along the length of the member

V is the shear force due to ultimate loads

V_c is the shear force that can be resisted by the concrete, ie V_{co} or V_{cr}

f_{yv} is the characteristic strength of the link reinforcement, which should be taken as not greater than 450 MPa

d_l is the effective depth as defined below

θ_c is the assumed inclination of web compression to the centre line of the beam

θ_v is the inclination of the link to the centre line of the beam, towards the direction of the principal tensile stresses in the web

b is the breadth of the member, which for T, I and L-beams should be replaced by b_w , the breadth of the rib

f_{yL} is the characteristic strength of the longitudinal reinforcement, which should be taken as not greater than 450 MPa.

θ_c should be prudently chosen within limits so that $\frac{3}{5} \leq \cot \theta_c \leq \frac{5}{3}$ to ensure reasonable control of cracking under service conditions. θ_c may vary along the beam.

In rectangular beams, at both corners in the tensile zone a link should pass around a longitudinal bar, a tendon, or a group of tendons having a diameter not less than the link diameter. In this clause on shear reinforcement, the depth, d_l , should be taken as the depth from the extreme compression fibre either to these longitudinal bars or to the centroid of the tendons, whichever is greater. A link should extend as close to the tension and compression faces as possible, with due regard to cover. The links provided at a cross-section should between them enclose all the tendons and additional reinforcement provided at the cross-section and should be adequately anchored (see 3.8.6.5). Hanger reinforcement at indirect supports should comply with 3.3.3.2.

The spacing of links along a member should not exceed $0,75d_l$ nor four times the web thickness for flanged members. When V exceeds $1,8V_c$, the maximum spacing should be reduced to $0,5d_l$. The lateral spacing of the individual legs of the links provided at a cross-section should not exceed $0,75d_l$.

Where links at right angles to the centre line of the beam are used (which will for horizontal beams be vertical, ie $\theta_v = 90^\circ$), together with an assumed inclination of compression forces $\theta_c = 45^\circ$, Equations 36(a), 37(a) and 38(a) reduce to:

$$\frac{A_{sv}}{s_v} \geq \frac{V - V_c}{0,87f_{yv}d_l} \dots\dots\dots (36b)$$

provided that to satisfy minimum reinforcement requirements it is assumed that

$$\frac{V - V_c}{bd_t} \leq 0,4 \text{ MPa} \dots\dots\dots (37b)$$

and

$$\Delta A_s \geq \frac{V}{2(0,87f_{yL})} \dots\dots\dots (38b)$$

4.3.4.5 Maximum shear force: In no circumstances should the shear force V due to ultimate loads exceed $v_{\max}$ multiplied by $bd \sin 2\theta_c$, where $v_{\max} = 0,75\sqrt{f_{cu}}$ but is not greater than 5,8 MPa; b is as defined in 4.3.4.2, less either the diameter of the duct for temporarily ungrouted ducts, or two-thirds the diameter of the duct for grouted ducts; and d is the distance from the compression face to the centroid of the area of steel in the tension zone, irrespective of its characteristic strength.

4.3.4.6 Segmental construction: In post-tensioned segmental construction, the shear force at a joint due to ultimate loads should not be greater than

$$0,7\gamma_{iL}P_h \tan \alpha_i$$

where γ_{iL} is the partial safety factor for the prestressing force, to be taken as 0,87

P_h is the horizontal component of the prestressing force after all losses

α_i is the angle of friction at the joint. $\tan \alpha_i$ can vary from 0,7 for a smooth unprepared joint up to 1,4 for a castellated joint; a value greater than 0,7 should only be used when justified by tests and agreed on with the responsible bridge authority. If bonding agents such as an epoxy adhesive are used, suitable tests should be carried out.

4.3.4.7 Elements of variable effective depth: For the determination of the shear resistance of members with variable effective depth, the components of the flange forces and reinforcement forces parallel to the shear should be taken into account if their effect is unfavourable. They may also be taken into account, however, if their effect is favourable.

4.3.5 Torsion

4.3.5.1 Torsional stiffness: When the torsional resistance and stiffness of members with solid or closed hollow sections (box-sections) are taken into account in the analysis and design of the structure, the torsional rigidity ($G X C$) of a member may be obtained by assuming that the shear modulus G is equal to 0,4 times the modulus of elasticity of the concrete, and that the torsional constant C is equal to half the St Venant value for pure torsion calculated for the uncracked plain concrete section. Where torsion is resisted by open sections consisting of two or more interconnected planar elements (thin walls or slabs in separate planes, but approxi-

mately parallel to a single axis such as a deck slab and two or more thin webbed beams which do not form a closed box section) warping will take place and large deflections and/or stresses may occur. Specialist literature should be consulted.

4.3.5.2 Torsion of secondary importance: Where torsion does not decide the dimension of members, torsional design may be carried out as a check, after the flexural design. This is particularly relevant to some members in which the maximum torsional moment does not occur under the same loading as the maximum flexural moment. In such circumstances, reinforcement and prestress in excess of that required for flexure and shear may be used as part of the torsion reinforcement, if required.

Torsional moments caused by the restraint of angular rotation by adjacent members, and which are not necessary for equilibrium, may be neglected in ultimate limit state calculations, provided that experience or analysis has shown that torsion will play only a minor role in the behaviour of the structure. The provisions of this clause apply to Class 1, Class 2 and Class 3 prestressed concrete members.

4.3.5.3 Stresses caused by and reinforcement for pure torsion (St Venant torsion): Where torsion in a section substantially increases the shear stresses, the torsional shear stresses should be calculated, assuming a plastic shear stress distribution. Calculations for torsion should be in accordance with 3.3.4 with the following modifications.

When prestressing steel is used as transverse torsional steel, in accordance with Equations 11 and 11(a), or as longitudinal steel, in accordance with Equations 12 and 12(a), the stress assumed in design should be the lesser of 450 MPa or $(0,87f_{pu} - f_{pe})$. The compressive stress in the concrete due to prestress should be taken into account separately in accordance with 3.3.4.6.

In calculating $(v + v_t)$, for comparison with v_{tu} in Table 9, v should be calculated from Equation 8, regardless of whether uncracked (4.3.4.2) or cracked (4.3.4.3) sections are critical in shear.

For concrete grades above 40, the values of V_{tu} given in Table 9 may be increased to $0,75\sqrt{f_{cu}}$, but may not be greater than 5,8 MPa.

4.3.5.4 Segmental construction: When a structure that is to be constructed in segments is designed for torsion and additional torsional steel is necessary, in accordance with Equations 12 or 12(a), the distribution of this longitudinal steel, whether it is in the form of reinforcement or prestressing tendons, should comply with the requirements of 3.3.4.4. Other arrangements may be used provided that the line of action of the longitudinal elongating force is at the centroid of the steel. In precast construction, sufficient anchorage should be provided within each precast segment for unprestressed longitudinal reinforcement.

4.3.5.5 Other design methods: Alternative methods of designing members subjected to combined bending, shear and torsion may be used provided that it can be shown that they satisfy both the ultimate and serviceability limit state requirements.

4.3.6 Longitudinal shear

For flanged beams in which shear reinforcement is required to resist vertical shear, the longitudinal shear resistance of the flange and of the flange/web junction should be checked in accordance with 5.4.2.3.

4.3.7 Deflection of beams

4.3.7.1 Class 1 and 2 members: The instantaneous deflection caused by design loads may be calculated using an elastic analysis based on the concrete section properties and the value for the modulus of elasticity given in 2.3.2.1.

The total long-term deflection due to the prestressing force, dead load and any sustained imposed loading may be calculated using an elastic analysis. This should be based on the concrete section properties and on an effective modulus of elasticity obtained from the creep of the concrete per unit length per unit applied stress after the period considered (specific creep). The values for specific creep given in 4.8.2.5 may generally be used, unless a more accurate assessment is required (see Appendix 3.C). Due allowance should be made for the loss of prestress after the period considered.

4.3.7.2 Class 3 members: Where the permanent load is less than or equal to 25 % of the design imposed load, the deflection of Class 3 members may be calculated in accordance with 4.3.7.1. Where the permanent load is greater than 25 % of the design load, calculations based on the moment/curvature relationship should be made.

4.4 Slabs

The analysis of prestressed slabs should be in accordance with 3.4.1 provided that due allowance is made for moments due to prestress. The design should be in accordance with 4.3.

The design for shear should be in accordance with 4.3.4, except that shear reinforcement need not be provided if V is less than V_c .

In the treatment of shear stresses under concentrated loads, the ultimate shear resistance of a section uncracked in flexure, V_{cu} , may be taken to correspond to the occurrence of a maximum tensile stress of $f_t = 0,24\sqrt{f_{cu}}$ at the centroidal axis around the critical section. This section is assumed to have a perimeter at a distance $h/2$ from the loaded area with an effective width, b , equal to the length of the critical perimeter.

Reinforcement, if necessary, should be provided in accordance with 4.3.4.4. and 4.3.4.5.

4.5 Columns

Prestressed concrete columns, in which the mean stress in the concrete section imposed by the tendons is less than 2,5 MPa, may be analysed as reinforced columns in accordance with 3.5.; otherwise the full effects of the prestress should be considered.

4.6 Prestressed Concrete Walls

If subjected principally to bending moments as in 3.6.1, prestressed concrete walls shall be designed as slabs. In other cases, 3.6.1.1 to 3.6.7 shall apply, but with the full effects of prestressing being considered.

4.7 Tension Members

The tensile strength of tension members should be based on the design strength ($0,87f_{pu}$) of the prestressing tendons and the strength developed by any additional reinforcement. The additional reinforcement may usually be assumed to be acting at its design stress ($0,87f_y$); in special cases it may be necessary to check the stress in the reinforcement using strain compatibility.

Members subject to axial tension should also be checked at the serviceability limit state for compliance with the appropriate stress limitations of 4.3.2.3 and 4.3.2.4.

The shear resistance of tension members may be calculated in accordance with 4.3.4 by taking the reduction in the effective prestress caused by the applied tension load into account.

4.8 Prestressing Requirements

4.8.1 Maximum initial prestress

Immediately after anchoring, the force in the prestressing tendon should not exceed 70 % of the characteristic strength for post-tensioned tendons, or 75 % for pretensioned tendons. The jacking force may be increased to 80 % during prestressing provided that additional consideration is given to safety, to the stress-strain characteristics of the tendon, and to the assessment of the friction losses.

In determining the jacking force to be used, consideration should also be given to the gripping or anchorage efficiency of the anchorage.

Where deflected tendons are used in pretensioning systems, consideration should be given, in determining the maximum initial prestress, to the possible influence of the size of the deflector on the strength of the tendons. Attention should also be paid to the effect of any frictional forces that may occur.

4.8.2 Loss of prestress, other than friction losses

4.8.2.1 General: Allowance should be made when calculating the forces in tendons at the various stages in design for the appropriate losses of prestress resulting from:

- (a) relaxation of the tendons;
- (b) the elastic deformation and subsequent shrinkage and creep of the concrete;
- (c) slip or movement of tendons at anchorages during anchoring and
- (d) other causes in special circumstances as, for example, when steam curing is used with pretensioning.

If experimental evidence on performance is not available, account should be taken of the properties of the steel and of the concrete when calculating the losses of prestress from these causes. For a wide range of structures, the simple recommendations given in the subsections below should be used; it should be recognized, however, that these recommendations are necessarily general and approximate.

4.8.2.2 Loss of prestress due to relaxation of steel: The loss of force in the tendon to be allowed for in the design should be determined from the maximum relaxation after 1 000 hours duration, for a jacking force equal to that imposed at transfer.

No reduction in the value of the relaxation loss should be made for a tendon when a load greater than the relevant jacking force was applied for a short time before the tendon was anchored.

In special cases, such as when tendons are subjected to high temperatures or large lateral loads, eg deflected tendons, greater relaxation losses will occur. Specialist literature should be consulted in these cases.

4.8.2.3 Loss of prestress due to elastic deformation of the concrete: Calculation of the immediate loss of force in the tendons due to elastic deformation of the concrete at transfer may be based on the values for the modulus of elasticity of the concrete given in Table 3. The modulus of elasticity of the tendons may be obtained from 2.3.2.2.

For pretensioning, the loss of prestress in the tendons at transfer should be calculated on the basis of the modular ratio, using stress in the adjacent concrete.

For members with post-tensioning tendons that are not stressed simultaneously, there is a progressive loss of prestress during transfer due to the gradual application of the prestressing force. The resulting loss of prestress in the tendons should be calculated on the basis of half the product of the modular ratio and the stress in the concrete adjacent to the tendons, averaged along their length; alternatively, the loss of prestress may be computed exactly based on the sequence of tensioning.

In making these calculations, it may usually be assumed that the tendons are located at the centroid of the group of tendons.

4.8.2.4 Loss of prestress due to shrinkage of the concrete: The loss of prestress in the tendons due to shrinkage of the concrete may be calculated from the modulus of elasticity of the tendon, as given in 2.3.2.2, using the values for shrinkage per unit length given in Table 28.

In certain regions of South Africa the aggregates may exhibit abnormally high shrinkage characteristics. The fine-grained shales and sandstones such as from the Beaufort Formation of the Karoo Group are those most likely to lead to high dimensional changes in concrete. Expert advice should be sought when these aggregates or others of a similar type are to be used and representative tests should be performed on the concrete.

TABLE 28 Shrinkage of concrete

System	Shrinkage per unit length		
	Relative humidity		
	80 %	60 %	35 %
	Coastal towns	Most inland areas	Environments of usually low relative humidity such as Windhoek and Uplington
Pretensioning Transfer at 3 to 5 days after concreting	180×10^{-6}	310×10^{-6}	420×10^{-6}
Post-tensioning Transfer at 7 to 14 days after concreting	140×10^{-6}	250×10^{-6}	350×10^{-6}

For other ages of concrete at transfer, for other conditions of exposure, or for massive structures, some adjustment to these figures will be necessary, in which case reference should be made to specialist literature.

When it is necessary to determine the loss of prestress and the deformation of the concrete at some stage before the final shrinkage has been reached, it may be assumed for concrete containing normal aggregate that half the total shrinkage takes place during the first month after transfer and that three-quarters of the total shrinkage takes place in the first six months after transfer.

4.8.2.5 Loss of prestress due to creep of the concrete: The loss of prestress in the tendons due to creep of the concrete should be calculated on the assumption that creep is proportional to stress in the concrete for stress of up to one-third of the cube strength at transfer. The loss of prestress is obtained from the product of the modulus of elasticity of the tendon (see 2.3.2.2) and the creep of the concrete adjacent to the tendons. Usually it is sufficient to assume, in calculating this loss, that the tendons are located at the centroid of the group of tendons.

For pretensioning at between three days and five days after concreting and for humid or dry conditions of exposure where the required cube strength at transfer is greater than 40,0 MPa, the creep of the concrete per unit length should be taken as 48×10^{-6} per MPa. For lower values of cube strength at transfer, the creep per unit length should be taken as $48 \times 10^{-6} \times 40,0/f_{c1}$ per MPa.

For post-tensioning at between 7 days and 14 days after concreting and for humid or dry conditions of exposure where the required cube strength at transfer is greater than 40,0 MPa, the creep of the concrete per unit length should be taken as $36 \times 10^{-6} \times 40,0/f_{c1}$ per MPa. For lower values of cube strength at transfer, the creep per unit length should be taken as $36 \times 10^{-6} \times 40,0/f_{c1}$ per MPa.

Where the maximum stress at transfer anywhere in the section exceeds one-third of the cube strength of the concrete, the value for the creep per unit length used in calculations should be increased. When the maximum stress at transfer is half the cube strength, the values for creep are 1,25 times the values given above; at intermediate stresses, the values should be interpolated linearly.

The figures for creep of the concrete per unit length in the preceding paragraphs relate to the ultimate creep after a period of years. When it is necessary to determine the deformation of the concrete due to creep at some earlier stage, it may be assumed that half the total creep takes place in the first month after transfer and that three-quarters of the total creep takes place in the first six months after transfer.

In applying the recommendations given above, which are necessarily general, reference should be made to Appendix 3.C or to specialist literature for more detailed information on the factors affecting creep.

4.8.2.6 Loss of prestress during anchorage: In post-tensioning systems allowance should be made for any movement of the tendons at the anchorage when the prestressing force is being transferred from the tensioning equipment to the anchorage. The loss due to this movement is particularly important in short members, and for such members the allowance made by the designer should be checked on site.

4.8.2.7 Loss of prestress due to steam curing: Where steam curing is employed in the manufacture of prestressed concrete units, changes in the behaviour of the material at higher-than-normal temperatures will need to be considered. In addition, where the "long-line" method of pretensioning is used, there may be addi-

tional losses as a result of bond being developed between the tendon and the concrete when the tendon is hot and relaxed. Since the actual losses of prestress due to steam curing are a function of the techniques used by the various manufacturers, specialist advice should be sought.

4.8.3 Loss of prestress due to friction

4.8.3.1 General: In post-tensioning systems the greater part of the tendon will move relative to the surrounding duct during the tensioning operation, and, if the tendon comes into contact with either the duct or any spacers provided, friction will cause a reduction in the prestressing force as the distance from the jack increases. In addition, a certain amount of friction will develop in the jack itself and in the anchorage through which the tendon passes.

In the absence of evidence established to the satisfaction of the engineer, the stress variation likely to occur along the design profile should be assessed in accordance with 4.8.3.2 to 4.8.3.5 in order to determine the prestressing force at the critical sections that are considered in design. The extension of the tendon should be calculated, allowing for the variation in tension along its length.

4.8.3.2 Friction in the jack and anchorage: This is directly proportional to the jack pressure, but will vary considerably between systems and should be ascertained for the type of jack and the anchorage system to be used.

4.8.3.3 Friction in the duct due to unintentional variation from the specified profile: Whether the desired duct profile is straight or curved or a combination of both, there will be slight variations in the actual line of the duct. These may result in additional points of contact between the tendon and the sides of the duct, and so produce friction. The prestressing force P_x at any distance x from the jack may be calculated from:

$$P_x = P_o e^{-Kx} \dots\dots\dots (39)$$

and where $Kx \leq 0,2$, e^{-Kx} may be taken as $1 - Kx$

where P_o is the prestressing force in the tendon at the jacking end

e is the base of Napierian logarithms (2,718)

K is a constant depending on the type of duct, or sheath, employed, the nature of its inside surface, the method of forming it and the degree of vibration employed in placing the concrete.

The value of K per metre length in Equation 39 should generally be taken as not less than 33×10^{-4} , but where strong rigid sheaths or duct formers are used, closely supported so that they are not displaced during the concreting operation, the value of K may be taken as 17×10^{-4} . Other values may be used provided they have been established by tests to the satisfaction of the engineer.

4.8.3.4 Friction in the duct due to curvature of the tendon: When a tendon is curved, the loss of tension due to friction is dependent on the angle turned through and on the coefficient of friction μ between the tendon and its supports.

For a tendon with a single circular curve in a plane, the prestressing force P_x at any distance x along the curve, measured from a tangent point near the jacking end, may be calculated from:

$$P_x = P_o e^{-\mu x/r_{ps}} \dots\dots\dots (40)$$

where P_o is the prestressing force in the tendon at the tangent point near the jacking end

r_{ps} is the radius of curvature of a tendon.

Where $\mu x/r_{ps} \leq 0,2$, $e^{-\mu x/r_{ps}}$ may be taken as $1 - \mu x/r_{ps}$.

Where $(Kx + \mu x/r_{ps}) \leq 0,2$, $e^{-(Kx + \mu x/r_{ps})}$ may be taken as $1 - (Kx + \mu x/r_{ps})$.

Values of μ may be taken as:

- 0,55 for steel moving on concrete,
- 0,30 for steel moving on steel,
- 0,25 for steel moving on lead.

The value of μ may be reduced where special precautions are taken and where results are available to justify the value assumed. For example, a value of $\mu = 0,10$ has been observed for strands moving on rigid steel spaces coated with molybdenum disulphide. Such reduced values should be agreed on with the responsible bridge authority.

Where a non-circular curve is used, eg a parabola,

$$P_x = P_o e^{-\mu \Sigma \alpha},$$

where $\Sigma \alpha$ is the incremental sum of the angles of rotation over the length of the curve.

Where tendons with multiple curves are used, the prestressing force P_{x_n} at any distance x_n along the tendon from the tangent point near the jacking end may be calculated as:

$$P_{x_n} = P_o e^{-\mu \Sigma_n \alpha},$$

where $\Sigma_n \alpha$ is the increment sum of the angles of rotation, all taken as positive over the distance x_n .

Where the curvature is not confined to one plane, it shall be determined as above but by summing the maximum incremental angles of rotation along the distance x_n .

4.8.3.5 Friction in circular construction: Where circumferential tendons are tensioned by means of jacks, the losses due to friction may be calculated from the formula in 4.8.3.4, but the values of μ may be taken as:

- 0,45 for steel moving in smooth concrete,
- 0,25 for steel moving on steel bearers fixed to the concrete,
- 0,10 for steel moving on steel rollers.

4.8.3.6 Lubricants: Lubricants may be specified to ease the movement of tendons in the ducts. Lower values of μ than those given in 4.8.3.4 and 4.8.3.5 may then be used, subject to their being determined by trial. The use of lubricants and the values of μ used shall be approved by the responsible bridge authority.

4.8.4 Transmission length in pretensioned member

The transmission length is defined as the length over which a tendon must be bonded to concrete to transmit the initial prestressing force in the tendon to the concrete.

The transmission length depends on a number of variables, the most important being:

- (a) the degree of compaction of the concrete;
- (b) the strength of the concrete;
- (c) the size and type of tendon;
- (d) the deformation (eg crimp of the tendon);
- (e) the stress in the tendon; and
- (f) the surface condition of the tendon.

The transmission lengths for the tendons towards the top of a unit may be greater than those for tendons at the bottom. The sudden release of tendons may also cause a considerable increase in the transmission lengths.

Where the initial prestressing force is not greater than 75 % of the characteristic strength of the tendon, and where the concrete strength at transfer is not less than 30 MPa, the transmission length l_t (in mm) may be taken as follows:

$$l_t = \frac{K_t \phi}{\sqrt{f_{c1}}}$$

where f_{c1} is the concrete strength at transfer in MPa

ϕ is the nominal diameter of the tendon in mm

K_t is a coefficient that depends on the type of tendon, to be taken as:

600 for plain, indented and crimped wire with a total wave height of less than $0,15\phi$;

400 for crimped wire with a total wave height greater than or equal to $0,15\phi$;

360 for 7-wire drawn strand;

240 for 7-wire standard and super strand.

The development of stress from the end of the unit to the point of maximum stress should be assumed to be linear over the transmission length.

If the tendons are prevented from bonding to the concrete near the ends of the units by the use of sleeves or tape, the transmission lengths given above should be taken from the ends of the de-bonded portions.

4.8.5 End blocks

The end block (also known as the anchor block or end zone) is defined as the highly stressed zone of concrete around the termination points of a pre- or post-tensioned prestressing tendon. It extends from the point of application of prestress (ie the end of the bonded part of the tendon in pretensioned construction or the anchorage in post-tensioned construction) to that section of the member at which linear distribution of stress is assumed to occur over the whole cross-section.

The following aspect should be considered in assessing the strength of end blocks:

- (a) "bursting" forces around individual anchorages;
- (b) overall equilibrium of the end block; and
- (c) the possibility of spalling of the concrete from the loaded face around anchorages.

In considering each of the above aspects, particular attention should be given to the following factors:

- (a) shape, dimensions and positions of the anchor plates relative to the cross-section of the end block;
- (b) magnitude of the prestressing forces and sequence of prestressing;
- (c) shape of the end block relative to the general shape of the member;
- (d) layout of anchorages including asymmetry, group effects and edge distances;
- (e) influence of the support reaction; and
- (f) forces due to curved or divergent tendons.

The requirements given below are appropriate to a circular, square or rectangular anchor plate or group of plates symmetrically positioned on the end face of a square or rectangular post-tensioned member, and are followed by some guidance on other aspects. Reference should be made to specialist literature for more complex arrangements.

The bursting tensile forces in the end blocks or in regions of bonded post-tensioned members, should be assessed by applying the tendon jacking load. For temporary unbonded members the bursting tensile forces should be assessed on the basis of the tendon jacking load or the load in the tendon at the ultimate limit state, calculated from Tables 20 to 23, whichever is the greater.

The bursting tensile force, F_{bst} , in an individual square end block loaded by a symmetrically placed square anchorage or bearing plate may be derived from Table 29

where y_o is half the side of the end block

y_{po} is half the side of the loaded area

P_k is the load in the tendons assessed as above

F_{bst} is the bursting tensile force.

TABLE 29 Design bursting tensile forces in end blocks

y_{po}/y_o	0,3	0,4	0,5	0,6	0,7
F_{bst}/P_k	0,23	0,20	0,17	0,14	0,11

This force, F_{bst} , will be distributed in a region extending from $0,2y_o$ to $2y_o$ from the loaded face of the end block. Reinforcement provided to sustain the bursting tensile force may be assumed to be acting at its designed strength ($0,87f_y$), except that the stress should be limited to a value corresponding to a strain of 0,001 when the concrete cover to the reinforcement is less than 50 mm.

In rectangular end blocks, the bursting tensile forces in the two principal directions should be assessed on the basis of the formulae given in Table 29.

When circular anchorages or bearing plates are used, the size of the equivalent square area should be derived.

Where groups of anchorages or bearing plates occur, the end blocks should be divided into a series of symmetrically loaded prisms and each prism treated in the above manner. In detailing the reinforcement for the end block as a whole, it is necessary to ensure that the groups of anchorages are appropriately tied together.

Special attention should be paid to end blocks having a cross-section that differs in shape from that of the general cross-section of the beam; reference should be made to the specialist literature.

Compliance with the above requirements will generally ensure that bursting tensile forces along the load axis are provided for. Alternative methods of design that use higher values of F_{bst}/P_k and make allowance for the tensile strength of concrete may be more appropriate in some cases, particularly where large concentrated tendon forces are involved.

Consideration should also be given to the spalling tensile stresses that occur in end blocks where the anchorages or bearing plates are highly eccentric; these stresses reach a maximum at the loaded face.

4.9 Considerations affecting Design Details

4.9.1 General

The considerations in 4.9.2 to 4.9.6 are intended to supplement those for reinforced concrete given in 3.8.

4.9.2 Cover to prestressing tendons

The cover to prestressing tendons will generally be governed by considerations of durability under the envisaged conditions of exposure.

4.9.2.1 Pretensioned tendons: The recommendations of 3.8.2 concerning cover to reinforcement may be taken to be applicable. The ends of individual pretensioned tendons do not normally require concrete cover and should preferably be cut off flush with the end of the concrete member.

4.9.2.2 Tendons in ducts: The cover to any duct should be not less than 50 mm. Precautions should be taken to ensure a dense concrete cover, particularly with wide ducts or ducts of large diameter.

Recommendations for the cover to curved tendons in ducts are given in Appendix 3.D.

4.9.3 Spacing of prestressing tendons

In all prestressed members large-enough gaps should be left between the tendons or bars to allow the largest size of aggregate used to move, under vibration, to all parts of the mould.

4.9.3.1 Pretensioned tendons: The recommendations of 3.8.8.1 concerning spacing of reinforcement may be taken to be applicable. In pretensioned members, where anchorage is achieved by bond, the spacing of the wires or strands in the ends of the members should be such as to allow the transmission lengths given in 4.8.4 to be developed. In addition, if the tendons are positioned in two or more widely spaced groups, the possibility of longitudinal splitting of the member should be considered.

4.9.3.2 Tendons in ducts: The clear distance between ducts or between ducts and other tendons should be not less than the following, whichever is the greatest:

- (a) $h_{agg} + 5 \text{ mm}$, where h_{agg} is the nominal maximum size of the coarse aggregate;
- (b) in the vertical direction, the vertical internal dimension of the duct;
- (c) in the horizontal direction, the horizontal internal dimension of the duct.

Where internal vibrators are to be used, sufficient space should be left between ducts to enable the vibrator to be inserted.

Where two or more rows of ducts are used, the horizontal gaps between the ducts should be vertically in line wherever possible, for ease of construction.

Recommendations for the spacing of curved tendons in ducts are given in Appendix 3.D.

4.9.4 Longitudinal reinforcement in prestressed or partially prestressed concrete beams

Non-prestressed reinforcement may be used in prestressed concrete members either to increase the strength of sections, or to comply with the requirements of 4.3.4.4.

Any stress or strength calculation taking account of additional reinforcement should still be in accordance with 4.3.2.2 and 4.3.3.1.

Reinforcement may be necessary, particularly where post-tensioning systems are used, to control any cracking resulting from restraint on the longitudinal shrinkage of members caused by the formwork during the time before the prestress is applied.

4.9.5 Links in prestressed concrete beams

The amount and disposition of links in rectangular beams and in the webs of flanged beams will normally be governed by considerations of shear (see 4.3.4).

Links should be provided to resist the bursting tensile forces in the end zones of post-tensioned members, in accordance with 4.8.5.

Links should be provided in the transmission length of pretensioned members in accordance with the requirements of 4.3.4 and using the information given in 4.8.4.

4.9.6 Shock loading

When a prestressed concrete beam is required to resist shock loading, it should be reinforced with closed links and longitudinal reinforcement preferably of grade 250 mild steel. Other methods of design and detailing may be used, provided it can be shown that the beam will be able to develop the required ductility.

5 DESIGN AND DETAILING: PRECAST, COMPOSITE AND PLAIN CONCRETE CONSTRUCTION

5.1 General

5.1.1 Introduction

This section is concerned with the additional considerations that arise in design and detailing when precast members or precast components, including large panels, are incorporated into a structure, or when a structure is constructed entirely from precast concrete. It also covers the use of plain concrete for walls and abutments.

5.1.2 Limit State design

5.1.2.1 Basis of design: The limit state philosophy set out in Sections 2 and 6 of Part 1 and Section 2 of this Part applies equally to precast and in situ construction. Therefore, in general, the recommended methods of design and detailing for reinforced concrete given in Section 3 and those for prestressed concrete given in

Section 4 also apply to precast and composite construction. Any subsections in Sections 3 and 4 that do not apply are either specifically worded for in situ construction or are modified by this subsection.

5.1.2.2 Handling stresses: Precast units should be designed to resist without permanent damage all the stresses induced by handling, storage, transport and erection (see also 4.3.1.2). When necessary, the positions of lifting and the supporting points would be specified. Consultation at the design stage with those who will be responsible for handling is recommended. The design should take into account the effect of "snatch" lifting and placing onto supports.

5.1.2.3 Connections and joints: The design of connections is of fundamental importance in precast construction and must be carefully considered.

Joints to allow for movement due to shrinkage, thermal effects and the possible differential settlement of foundations are just as important in precast as in in situ construction. The number and spacing of such joints should be determined at an early stage in the design. In the design of beam-and-slab ends on corbels and ribs, particular care should be taken to provide overlap and anchorage in accordance with 3.8.7 for all reinforcement adjacent to the contact faces, with full regard being had for construction tolerances.

5.2 Precast Concrete Construction

5.2.1 Framed structures and continuous beams

When the continuity of reinforcement or tendons through the connections and/or the interaction between members is such that the structure will behave as a frame, or as another type of rigidly interconnected system, the analysis, redistribution of moments and the design detailing of individual members may all be in accordance with Section 3 or 4 as appropriate.

5.2.2 Other precast members

All other precast concrete members, including large panels, should be designed and detailed in accordance with the appropriate recommendations of 3.4 and 5.5 and should incorporate provision for the appropriate connections as recommended in 5.3.

Precast components intended for use in composite construction (see 5.4) should be designed as such, but should also be checked or designed for the conditions that may arise during handling, transport and erection.

5.2.3 Supports for precast members

5.2.3.1 Concrete corbels: A corbel is a short cantilever beam projecting from a column to which the principal load is applied such that the distance a_v between the

line of action of the load and the face of the supporting member is less than $0,6d$, and whose depth at the outer edge of the bearing area is not less than one-half of the depth at the face of the supporting member.

The depth at the face of the supporting member should be determined from shear conditions in accordance with 3.3.3.3, but using the modified definition of a_v given above.

In the absence of a more accurate analysis, the main tension reinforcement in a corbel may be designed, and the strength of the corbel checked, on the assumption that it behaves as a simple strut-and-tie system. The reinforcement so obtained should form not less than 0,4 % of the section at the face of the supporting member and should be adequately anchored. At the front face of the corbel, the reinforcement should be anchored by bending back the bars to form a loop; the load-bearing area should not project beyond the straight portion of the bars forming the main tension reinforcement.

When the corbel is designed to resist a stated horizontal force, additional reinforcement should be provided to transmit this force to the supporting member in its entirety; the reinforcement should be adequately anchored within the supporting member.

Shear reinforcement should be provided in the form of horizontal links distributed in the upper two-thirds of the effective depth of the corbel at the column face; this reinforcement need not be calculated but should be equal to not less than one-half of the area of the main tension reinforcement and should be adequately anchored.

The strength of the corbel should also be checked at the serviceability limit state.

5.2.3.2 Widths of supports for precast units: The width of supports for precast units should be sufficient to ensure proper anchorage of tension reinforcement in accordance with 3.8.7.

5.2.3.3 Bearing stresses: The compressive stress in the contact area should not exceed $0,4f_{cu}$ under the ultimate loads. When the members are made of concretes of different strengths, the lower concrete strength is applicable.

Higher bearing stresses may be used where suitable measures are taken to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas and additional binding reinforcement in the ends of the members. Bearing stresses due to ultimate loads should then be limited to:

$$\frac{1,5f_{cu}}{1 + 2 \sqrt{\frac{A_{con}}{A_{sup}}}} \quad \text{but no more than } f_{cu}$$

where A_{con} is the contact area

A_{sup} is the supporting area

For concrete containing low-density aggregate, the bearing stresses due to ultimate loads should be limited to:

$$\frac{f_{cu}}{1 + 2 \sqrt{\frac{A_{con}}{A_{sup}}}} \quad \text{but not more than } 0,67f_{cu}.$$

Higher bearing stresses due to ultimate loads should be used only where justified by tests and agreed on with the responsible bridge authority, eg in concrete hinges.

5.2.3.4 Horizontal forces or rotations at bearings: The presence of significant horizontal forces at a bearing can reduce the load-carrying capacity of the supporting and supported members considerably by causing premature splitting or shearing. These forces may be due to creep, shrinkage and thermal effects, or result from misalignment, being out of plumb or other causes. When they are likely to be significant, these forces should be allowed for in designing and detailing the connection by providing either:

- (a) sliding bearings (see Part 2), or
- (b) suitable reinforcement in the supporting members and in any other members where necessary.

Where, owing to large spans or for other reasons, large rotations are likely to occur at the end supports of flexural members, suitable bearings capable of accommodating these rotations should be used.

5.2.4 Joints between precast members

5.2.4.1 General: The critical sections of members close to joints should be designed to resist the work combinations of shear, axial force and bending caused by the ultimate vertical and horizontal forces. When the design of the precast members is based on the assumption that the joint between them is not capable of transmitting moment, either the design for the joint should ensure that this is so (see 5.2.3.4), or suitable precautions should be taken to ensure that if any cracking does occur it will not reduce the member's resistance to shear or axial force excessively and will not lead to premature corrosion or be unsightly.

Where a space is left between two or more precast units to be filled later with in situ concrete or mortar, the space should be large enough to allow the filling material to be placed easily and compacted sufficiently to fill the gap completely, without the necessity for abnormally high standards of workmanship or supervision. The erection instructions should contain definite information as to the stage during construction at which the gap should be filled.

Most joints will incorporate a structural connection (see 5.3) and this aspect should be considered in the design of the joint.

5.2.4.2 Halving joint: Since it is difficult to provide access to this type of joint for resetting or replacing bearings, halving joints should be used only where they are absolutely essential. For the type of joint shown in Figure 17, the maximum vertical ultimate load, F_v , should not exceed $4v_c b d_o$.

where v_c is the shear stress given by Table 7 for the full beam section

b is the breadth of the beam

d_o is the depth to additional reinforcement for resisting horizontal loading.

When the value of F_v is being determined, the method of erection and the forces involved should be considered.

The joint should preferably be reinforced with inclined links so that the vertical component of force in the links is equal to F_v , ie

$$F_v = A_{sv}(0,87f_{yv})\cos 45^\circ \text{ for links at } 45^\circ$$

where A_{sv} is the sum of the cross-sectional areas of the legs of the inclined links

f_{yv} is the characteristic strength of the inclined links.

The links and any longitudinal reinforcement taken into account should intersect the line of action of F_v .

In the compression face of the beams, the links should be anchored in accordance with 3.8.6.5. In the tension face of the beam, the horizontal load component, F_h , which for 45° links is equal to F_v , should be transferred to the main reinforcement. If the main reinforcement is continued straight on without a bend at the intersection, the links may be considered anchored if:

$$\frac{F_h}{2\sum u_s l_{sb}} < \text{the anchorage bond stress, which is given in Table 14}$$

where $\sum u_s$ is the sum of the effective perimeters of the reinforcement

l_{sb} is the length of the straight reinforcement beyond the intersection with the link, measured to the bend of the hook as shown in Figure 17.

If the main reinforcement is hooked, or bent vertically near the intersection, the inclined links should be anchored by bending them parallel to the main reinforcement; in this case, or if inclined links are replaced by bent-up bars, the bearing stress inside the bends should not exceed the value given in 3.8.6.8.

If there is a possibility of a horizontal load being applied to the joint or of restraint action, horizontal reinforcement or links should be provided to carry the load (as shown in Figure 17). Such reinforcement should be well anchored and should also be provided if there is a possibility of the inclined links being displaced so that they will cease to intersect the line of action, F_v .

Alternatively, the joint may be reinforced with vertical links, designed in accordance with 3.3.3, provided the links are adequately anchored.

The joints should also be checked at the serviceability limit state.

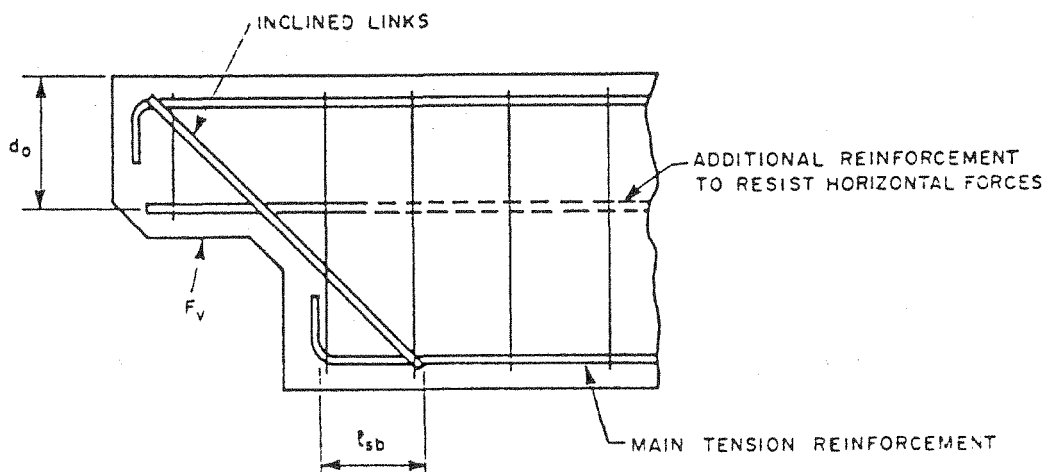


FIGURE 17
HALVING JOINT

5.3 Structural Connections between Units

5.3.1 General

5.3.1.1 Structural requirements of connections: When the connections across joints between precast members are being designed and detailed, the overall stability of the structure, including its stability during construction, must be considered.

5.3.1.2 Design method: Connections should, where possible, be designed in accordance with the general accepted methods applicable to reinforced concrete (see Section 3) prestressed concrete (see Section 4) or structural steel (See BS 5400, Part 3). Where, owing to the nature of the construction or the material used, such methods are not applicable, the efficiency of the connection should be proved by appropriate tests in accordance with Part 1.

5.3.1.3 Considerations affecting design details: In addition to ultimate strength requirements, the aspects listed below should be considered.

- (a) *Protection:* Connections should be designed to maintain the standard of protection against weather and corrosion required for the remainder of the structure.
- (b) *Appearance:* Where connections will be exposed, they should be so designed that the quality of appearance required for the remainder of the structure will not be prejudiced.
- (c) *Manufacture, assembly and erection:* Methods of manufacture and erection should be considered during design, and the following points should be given particular attention:

- (i) Where projecting bars or structural steel sections are required for the connection, they should be kept to a minimum and made as simple as possible. Such projections should be no longer than is necessary for security.
- (ii) Fragile fins and nibs should be avoided.
- (iii) Fixing devices should be located in concrete sections of adequate strength.
- (iv) The practicability of both casting and assembly should be considered.
- (v) Most connections require the introduction of suitable jointing material. Sufficient space should be allowed in the design for such material to ensure that proper filling of the joint is practicable.

5.3.1.4 Factors affecting design and construction: The strength and stiffness of any connection can be significantly affected by workmanship on site. The following points should be specified or described where appropriate:

- (a) sequence of forming the joint;
- (b) critical dimensions allowing for tolerances, eg minimum permissible bearing length or width;
- (c) critical details, eg accurate location required for a particular reinforcing bar;
- (d) method of correcting possible lack of fit in the joint;
- (e) details of temporary propping and time at which it may be removed;
- (f) general stability of the structure with details of any necessary temporary bracing;
- (g) how far the uncompleted structure may proceed above the completed and matured section;
- (h) full details of special materials; and
- (i) weld sizes, specified in full - where weld symbols are used, it should be ascertained that these will be understood on site.

5.3.2 Continuity of reinforcement

5.3.2.1 General requirements: Where continuity of reinforcement through the connection is required, the jointing method used should be such that the assumptions made in analysing the structure and critical sections are realized. The following methods may be used to achieve continuity of reinforcement:

- (a) lapping of bars;
- (b) sleeving;
- (c) threading of bars and
- (d) butt welding.

The satisfactoriness of jointing methods (b) and (c) and of any other method not listed above should be verified by tests and agreed on with the responsible bridge authority.

5.3.2.2 Sleeving: Three principal types of sleeve jointing may be used, provided that their strength and deformation characteristics, including behaviour under fatigue conditions, have been determined by tests in accordance with Part 1:

- (a) grout or resin-filled sleeves capable of transmitting both tensile and compressive forces;
- (b) sleeves that mechanically align the square-sawn ends of two bars to allow the transmission of compressive forces only; and
- (c) sleeves that are mechanically swaged to the bars and are capable of transmitting both tensile and compressive forces.

The detailed design of the sleeve, and the method of manufacture and assembly should be such as to ensure that the ends of the two bars can be accurately aligned within the sleeve. The concrete cover provided for the sleeve should be not less than that specified for normal reinforcement.

5.3.2.3 Threading: The methods given below may be used for jointing threaded bars.

- (a) The threaded ends of bars may be joined by a coupler having left and right-handed threads. This type of threaded connection requires a high degree of accuracy in manufacture in view of the difficulty of ensuring alignment.
- (b) One set of bars may be welded to a steel plate which is drilled to receive the threaded ends of the second set of bars; the second set of bars is fixed to the plate by means of nuts.
- (c) Threaded anchors may be cast into a precast unit to receive the threaded ends of reinforcement.

Where there is a risk of the threaded connection working loose, eg during vibration of concrete cast in situ, a locking device should be used.

Where there is difficulty in producing a clean thread at the end of a bar, the steel normally specified for black bolts having a minimum strength of 400 MPa in accordance with SABS 135 - 1971 (as amended) should be used.

The structural design of special threaded connections should be based on tests in accordance with Part 1, including behaviour under fatigue conditions, where relevant. Where tests have shown the threaded connection to be at least as strong as the parent bar, the strength of the joint may be based on 80 % of the specified characteristic strength of the joined bars in tension and on 100 % for bars in compression, divided in each case by the appropriate γ_m factor.

5.3.2.4 Welding of bars: The design of welded connections should be in accordance with 3.8.6.6.

5.3.3 Connections using structural steel inserts

Such joints may consist of a steel plate or rolled steel section projecting from the face of a member. The reinforcement in the ends of the supported member should be designed in accordance with Section 3.

The steel sections and any bolted or welded connections, should be designed in accordance with BS 5400, Part 3. Bearing stresses in accordance with 5.2.3.3 may be used, unless higher values can be justified as a result of tests.

The interaction and continuity of forces acting between the supported and supporting members through the steel connections shall be analysed, taking due account of localized effects.

Consideration should also be given in the design to the possibility of vertical splitting under the steel section due to shrinkage effects and localized bearing stresses, eg under a narrow steel plate.

5.3.4 Other types of connection

Any other type of connection that will be capable of carrying the ultimate loads acting on it may be used, subject to the approval of the responsible bridge authority. Amongst those suitable for resisting shear and flexure are connections made by prestressing across the joint. Special shear keys should preferably be provided to the joint faces to ensure proper alignment and to increase the shear resistance at ultimate loads.

Where tests have shown their acceptability, resin adhesives may be used to form joints subjected to compression but not for those that have to resist tension or shear.

For resin mortar joints, the flexural stresses in the joint should remain compressive under service loads. During the jointing operation at the construction stage, under service loads, the average compressive stress between the concrete surfaces to be joined should be between 0,2 MPa and 0,3 MPa measured over the total projection of the joint surface, and should be locally not less than 0,15 MPa; the difference between flexural stresses across the section should be not more than 0,5 MPa.

For cement mortar joints, the flexural stresses in the joint should remain compressive and be not less than 1,5 MP under service loads.

5.4 Composite Concrete Construction

5.4.1 General

These recommendations apply to flexural members consisting of precast concrete units acting in conjunction with added concrete, where provision has been made for the transfer of horizontal shear at the contact surface. The precast units may be of either reinforced or prestressed concrete.

In general, the analysis and design of composite concrete structures and members should be in accordance with Section 3 or 4, modified where appropriate by 5.4.2 and 5.4.3. Particular attention should be given in the design of both the component parts and the composite section to the effect, on stresses and deflections, of the method of construction and whether or not props are to be used. A check for adequacy should be made at each stage of construction. The relative stiffness of members should be based on the gross-transformed or net-transformed concrete section properties, as described in 2.4.3.1; if the concrete strengths in the two components of a composite member differ by more than 10 MPa, allowance should be made for this in assessing stiffnesses and stresses.

Differential creep and shrinkage of the added concrete and precast concrete members require consideration when analysing composite members for the serviceability limit states (see 5.4.3.4); they need not be considered for the ultimate limit state.

When precast prestressed units, having pretensioned tendons, are designed as continuous members, and continuity is obtained with reinforced concrete cast in situ over the supports, the compressive stresses due to prestress in the ends of the units may be assumed to vary linearly over the transmission length of the tendons in assessing the strength of sections.

5.4.2 Ultimate limit states

5.4.2.1 General: Where the cross-section of composite members and the applied loading increase by stages (eg a precast prestressed unit initially supporting self-weight plus the weight of added concrete, and subsequently acting compositely under live loading), the entire load may be assumed to act on the cross-section appropriate to the stage being considered.

5.4.2.2 Vertical shear: The assessment of the resistance of composite sections to vertical shear and the provision of shear reinforcement should be in accordance with 3.3.3 for reinforced concrete (except that in determining the area A_s , the area of tendons within the transmission length should be ignored) and 4.3.4 for prestressed concrete, modified where appropriate by (a), (b) or (c) below.

- (a) For I, M, T, U and box-beams precast prestressed concrete units with an in situ reinforced concrete top-slab cast over the precast units (including pseudo-box construction), the shear resistance should be based on either of the following:
- (i) the vertical shear force, V , due to ultimate loads may be assumed to be resisted by the precast unit acting alone and the shear resistance may be assessed in accordance with 4.3.4; or
 - (ii) the vertical shear force, V , due to ultimate loads may be assumed to be resisted by the composite section and the shear resistance may be assessed in accordance with 4.3.4.

In this case, the section properties should be based on those of the composite section, with due allowance for the different grades of concrete, where appropriate.

- (b) For inverted T-beam precast prestressed concrete units with transverse reinforcement placed through standard holes in the bottom of the webs of the units, completely infilled with concrete placed between and over the units to form a solid deck slab, the shear resistance and provision of shear reinforcement should be based on either of the following:

- (i) as in (a)(i) above; or
- (ii) the vertical shear force, V , due to ultimate loads may be apportioned between the infill concrete and the precast prestressed units on the basis of cross-sectional area, with due allowance for the different grades of concrete where appropriate. The shear resistance of the infill concrete section and the precast prestressed section should be assessed separately in accordance with 3.3.3 and 4.3.4 respectively.

In applying 3.3.3, the width of the infill concrete should be taken as the distance between adjacent precast webs, and the depth as the mean depth of the infill concrete, or the mean effective depth to the longitudinal reinforcement where this is provided in the infill section.

In applying 4.3.4, the breadth of the precast section should be taken as the web thickness, and the depth as the depth of the precast unit.

- (c) In applying 4.3.4.4, d_i should be derived for the composite section.

5.4.2.3 Longitudinal shear: The longitudinal shear force, V_l , per unit length of a composite or monolithic member, whether simply supported or continuous, should be calculated at the interface of a precast unit with an in situ concrete and at other potential shear planes (see Figure 18). An elastic method and the properties of the composite or monolithic concrete section (see 2.4.3.1) should be used, with due allowance for different grades of concrete where appropriate.

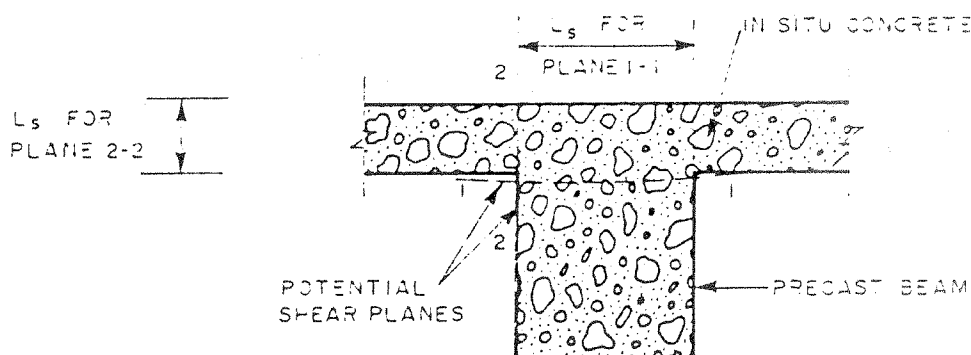


FIGURE 18
POTENTIAL SHEAR PLANES

V_1 should not exceed the lesser of either:

(a) $k_1 f_{cu} L_s$

or

(b) $v_1 L_s + 0,7 A_s f_y$

where k_1 is a constant depending on the concrete bond across the shear plane under consideration, taken from Table 30

f_{cu} is the characteristic cube strength of concrete

L_s is the breadth of the shear plane under consideration

v_1 is the ultimate longitudinal shear stress in the concrete for the shear plane under consideration, taken from Table 30

A_s is the area of fully anchored (see 3.8.6) reinforcement per unit length crossing the shear plane under consideration, but excluding reinforcement required for coexistent bending effects. Shear reinforcement crossing the shear plane and provided to resist vertical shear (see 5.4.2.2) may be included, provided it is fully anchored

f_y is the characteristic strength of the reinforcement.

TABLE 30 Ultimate longitudinal shear stress, v_1 , and values of k_1 , for composite members and monolithic construction

Type of shear plane	Longitudinal shear stress v_1 (in MPa) for concrete grade				k_1
	20	25	30	40 or more	
Monolithic construction	0,50	0,63	0,75	0,80	0,15
Surface, Type 1	0,50	0,63	0,75	0,80	0,15
Surface, Type 2	0,30	0,38	0,45	0,50	0,09

Note: For construction with concrete containing low-density aggregate, the values given above should be reduced by 25 %.

For composite or monolithic beam-and-slab construction, the minimum area of fully anchored reinforcement that should cross this surface or shear plane should be 0,15 % of the area of contact; the spacing of this reinforcement should not exceed the lesser of either:

(a) four times the minimum thickness of the in situ concrete flange;

or

(b) 600 mm.

The types of surface in composite construction are defined as follows:

Type 1: The contact surface of the concrete in the precast member is prepared as described either in (i) or (ii) below as appropriate.

- (i) Where the concrete has set but not hardened, the surface is sprayed with a fine spray of water or brushed with a stiff brush, just sufficiently to remove the outer mortar skin and expose the larger aggregate without disturbing it.
- (ii) Where the treatment in (i) has proved impracticable, the surface skin and laitance are removed by sand blasting or by using a needle-gun, but not by hacking.

Type 2: The contact surface of the concrete in the precast member is jetted with air and/or water to remove laitance and all loose material. No further roughening is carried out. (This is known as "rough as cast".)

5.4.3 Serviceability limit states

5.4.3.1 General: In addition to the recommendations made in Sections 3 and 4 with regard to the control of cracking, the design of composite construction will be affected by 5.4.3.4 and 5.4.3.5 and, where prestressed precast units are used, by 5.4.3.2 and 5.4.3.3 as well.

5.4.3.2 Compression in the concrete: For composite members comprising precast prestressed units and in situ concrete, the methods of analysis may be those given in 4.3.2. However, where ultimate failure of the composite unit would occur as a result of excessive elongation of the steel, the maximum concrete compressive stress at the upper surface of the precast unit may be increased above the values given in Table 24 by up to 25 %.

5.4.3.3 Tension in the concrete: When the composite member being considered in design consists of prestressed precast concrete units and in situ concrete, and when flexural tensile stresses are induced in the in situ concrete by sagging moments due to imposed service loads, the tensile stresses in the in situ concrete at the contact surface should be limited to the values given in Table 31. These values may, however, be increased by 50 %, provided the permissible tensile stress in the prestressed concrete unit is reduced by the same numerical amount.

Where in situ concrete in the tension zone is shown by tests to contribute to the load-distribution properties of a deck and is not taken into account in stress calculations, the notional tensile stress need not be calculated.

When the in situ concrete is not in direct contact with a precast prestressed concrete unit, the flexural tensile stresses in this concrete should be limited by cracking considerations in accordance with 3.8.8.2.

TABLE 31 Flexural tensile stresses in in situ concrete

Grade of in situ concrete	20	25	30	40	50
Maximum tensile stress (MPa)	2,8	3,2	3,6	4,4	5,0

Where the continuity is obtained with reinforced concrete cast in situ over the supports, the flexural tensile stresses in the prestressed precast units at the supports, or the stress increments in the reinforcement or prestressing steel, should be limited in accordance with 4.3.2.4.

5.4.3.4 Differential long-term deformations: The effects of differential long-term deformations* should be considered for composite concrete structures in which there is a difference between the age and quality of the concrete in the components. Differential creep and shrinkage may lead to increased stresses in the composite section and these should be investigated. Such effects are likely to be more severe when the precast component is of reinforced concrete, or of prestressed concrete with an approximately triangular distribution of stress due to prestressing. These stresses may be disregarded in inverted T-beams with solid-infill decks, provided that the difference in concrete strengths between the precast and infill components is not more than 10 MPa. For other forms of composite construction, the effects of differential creep and shrinkage should be considered in design.

In computing the tensile stresses, a value will be required for the differential shrinkage strain (the difference in total free strain between the two components of the composite member), the magnitude of which will depend on a great many variables.

For a bridge in a normal environment and in the absence of more exact data, the value of shrinkage strain given in 4.8.2.4 should be used to compute the stresses in composite construction. Reference should be made to Appendix 3.C when using concrete-containing low-density aggregate.

The effects of differential shrinkage will be reduced by creep and the reduction coefficient may be taken as 0,43. Where more exact or intermediate values of creep and shrinkage are needed, Appendix 3.C should be used.

5.4.3.5 Continuity in composite construction: When continuity is obtained in composite construction by providing reinforcement over the supports, consideration should be given to the secondary effects of differential shrinkage and creep on the

* The term "differential long-term deformations" refers to the difference in long-term deformation behaviour due to creep and shrinkage between the in situ concrete and the precast concrete.

moments in continuous beams and on the reactions at the supports. The hogging restraint moment, M_{cs} , due to differential shrinkage should be taken as:

$$M_{cs} = \epsilon_{diff} E_{cf} A_{cf} a_{cent} \phi \dots\dots\dots (41)$$

where ϵ_{diff} is the combined differential creep and shrinkage strains

E_{cf} is the modulus of elasticity of the concrete flange

A_{cf} is the cross-sectional area of the effective concrete flange

a_{cent} is the distance of the centroid of the concrete flange from the centroid of the composite section

ϕ is a reduction coefficient to allow for creep, taken as 0,43.

The resistance moment, M_{cs} , will in time be modified by creep due to dead load and creep due to any prestress in the precast units. The restraint moment due to prestress may be taken as the restraint moment that would have been set up if the composite section as a whole had been prestressed, multiplied by a reduction factor, ϕ_1 , taken as 0,87.

The information given in *CEB-FIP International Recommendations on Differential Shrinkage*, as reproduced in Appendix 3.C, should be used in determining a value for the differential shrinkage strain.

The expression given above for calculating the restraint moments due to creep and differential shrinkage is based on an assumed value of 2,0 for the ration (β_{cc}) of total creep to elastic deformation. If the design conditions are such as to make this value significantly low, then the engineer should calculate values for the reduction coefficient from the expressions:

$$\phi = \frac{(1 - e^{\beta_{cc}})}{\beta_{cc}} \dots\dots\dots (42)$$

$$\phi_1 = (1 - e^{\beta_{cc}}) \dots\dots\dots (43)$$

where e is the base of Napierian logarithms.

5.5 Plain Concrete Walls and Abutments

5.5.1 General

A plain concrete wall or abutment is a vertical load-bearing concrete member whose greatest lateral dimension is more than four times its least lateral dimensions; and is assumed to be without reinforcement when its strength is being considered.

The requirements given below refer to the design of a plain concrete wall that has a height not exceeding five times its average thickness.

5.5.2 Moments and forces in walls and abutments

The moments, shear forces and axial forces in a wall should be determined in accordance with 2.4.

The axial force may be calculated on the assumption that the beams and slabs contributing to it are simply supported.

The resultant axial force in a member may act eccentrically due to vertical loads not being applied at the centre of the member or due to the action of horizontal forces. Such eccentricities should be treated as indicated in 5.5.3 and 5.5.4.

The minimum moment in a direction at right angles to the wall (ie the moment vector in the plane of the wall) should be taken as not less than that produced by considering the ultimate axial load per unit length at an eccentricity of 0,05 times the thickness of the wall at the relevant section.

5.5.3 Eccentricity in the plane of the wall or abutment

In case of a single member, this eccentricity can be calculated from statics alone. Where a horizontal force is resisted by several members, the amount of eccentricity allocated to each member should be in proportion to its relative stiffness, provided the resultant eccentricity in any one member is not greater than one-third of its length. Where a shear connection is assumed between the vertical edges of adjacent members, an appropriate elastic analysis may be used, provided the shear connection is designed to withstand the calculated forces.

5.5.4 Eccentricity at right angles to walls or abutments

The load transmitted to a wall by a concrete deck may be assumed to act at one-third of the depth of the bearing area from the loaded face. Where there is an in situ concrete deck on either side of the member, the common bearing area may be assumed to be shared equally by each deck.

When the resultant eccentricity of the total load on a member that is unrestrained in position at any level is calculated, full allowance should be made for the eccentricity of all vertical loads and for the overturning moments produced by any lateral forces above the level of restraint.

5.5.5 Analysis of sections

Loads of a purely local nature (as at beam bearings or column bases) may be assumed to be immediately dispersed, provided the local stress under the load does not exceed that given in 5.2.3.3. Where the resultant of all the axial loads acts eccentrically in the plane of the member, the ultimate axial load per unit length of wall, n_w , should be assessed on the basis of an elastic analysis, assuming a linear distribution of load along the length of the member and no tensile resistance. Consideration should first be given to the axial force and bending moment in the plane of the wall (ie bending moment vectors normal to the wall) to determine the

distribution of tension and compression along the wall. The bending moment at right angles to the wall (ie bending moment vectors in the plane of the wall) should then be considered to determine the compression or tension per unit length at various positions along the wall. Where the axial load in the plane of the member is not eccentric, a uniform distribution of n_w may be assumed.

For short members restrained in position, the maximum axial load per unit length of a member, n_w , due to ultimate loads, should be such that:

$$n_w \leq (h - 2e_x)\lambda_w f_{cu} \dots\dots\dots (44)$$

where h is the thickness of the member

e_x is the resultant eccentricity of load at right angles to the plane of the members (see 5.5.2) (minimum value $0,05h$)

λ_w is a coefficient, taken as 0,35 for concrete grades 15 and 20, and 0,4 for concrete grades 25 and above

f_{cu} is the characteristic cube strength of the concrete.

5.5.6 Shear

The resistance to shear forces in the plane of the member may be assumed to be adequate, provided the horizontal shear force due to ultimate loads is either less than one-quarter of the vertical load, or less than the force that will produce an average shear stress of 0,45 MPa over the whole cross-section of the member in the case of concrete grade 25 or above; where grade 10, 15 or 20 concrete is used, a figure of 0,3 MPa is appropriate.

5.5.7 Bearing

Bearing stresses due to ultimate loads of a purely local nature, as at a girder bearing, should be limited in accordance with 5.2.3.3.

5.5.8 Deflection of plain concrete walls or abutments

The deflection in a plain concrete member will be within acceptable limits if the preceding recommendations have been followed.

5.5.9 Reinforcement for shrinkage and thermal effects

For plain concrete members cast in situ it is necessary to control cracking arising from shrinkage and thermal effects, including temperature rises caused by the heat of hydration released by the cement. Reinforcement should be provided in the direction of any restraint on such movement.

For full restraint, the area of reinforcement, calculated as a percentage of the gross concrete section, should be not less than 0,5% for Types B and C (grade 450) rein-

forcement, or 0,6 % for Type A (grade 250) reinforcement. In calculating these minimum percentages for cross-sections with dimensions exceeding 500 mm in both directions, half the area of the core of the cross-section more than 250 mm away from all concrete surfaces should be ignored. In slabs and walls, the reinforcement should be placed in both directions near the relevant surfaces with adequate concrete cover, or distributed between the two surfaces as required. Where the restraint is partial, the reinforcement may be reduced accordingly, but should cover not less than 0,12 % of the gross concrete section.

Reinforcement for shrinkage and thermal effects should be distributed uniformly around the perimeter of the concrete section and spaced at not more than 300 mm.

5.5.10 Stress limitations for serviceability limit states

The wall should be designed so that no tensile stresses will develop and so that the concrete compressive stresses will comply with the limitations given in Table 2.

APPENDIX 3.A

METHODS OF CHECKING FOR COMPLIANCE WITH SERVICEABILITY CRITERIA BY DIRECT CALCULATION

3.A.1 Analysis of Structures for Serviceability Limit States

In general an elastic analysis will be sufficiently accurate to assess the moments and forces in members subjected to their appropriate loadings for the serviceability limit states. Greater accuracy may be achieved in this analysis if the relative stiffnesses of the members are assessed on the basis of the properties of the cracked, transformed section of the member at its mid-span or mid-height. In assessing the second moment of area of sections, due account should be taken of the effects of any axial loads on the members, and the modulus of elasticity of the concrete should be taken as given in 2.3.2.1.

3.A.2 Calculation of Deflections

3.A.2.1 General

When the deflections of reinforced concrete members are calculated, it should be realized that there are a number of factors that may be difficult to allow for in the calculation but that can have a considerable effect on the reliability of the result. Examples are listed below.

- (a) Estimates of the restraints provided by supports are based on simplified and often inaccurate assumptions.
- (b) The precise loading, or the part of it that is of long duration, cannot be accurately determined.
- (c) Lightly reinforced members may well have a working load that is close to the cracking load for the member; considerable differences will occur in the deflections depending on whether or not the member has cracked.

The method of calculation given in 3.A.2.2 and 3.A.2.3, although not the only acceptable method, is one that will give reasonable results for both short-term and long-term loading (provided points such as those listed above have been correctly taken into account) and may be logically applied to a wide range of problems. The approach used is to assess the curvatures of sections under the appropriate moments and then calculate the deflections from the curvatures.

3.A.2.2 Calculation of curvatures

The curvature of any section may be calculated by employing whichever of the following sets of assumptions, (a) or (b), gives the larger value. (a) corresponds to the case in which the section is cracked under the loading being considered, and (b) applies to an uncracked section.

- (a) Strains are calculated on the assumptions that plane sections remain plane.

The reinforcement, whether in tension or compression, is assumed to be elastic. Its modulus of elasticity may be taken as 200 GPa.

The concrete in compression is assumed to be elastic. Under short-term loading, the modulus of elasticity may be taken as that given in 2.3.2.1. Under long-term loading, an effective modulus may be taken as having a value of $1/(1 + \phi)$ times the short-term modulus, where ϕ is the appropriate creep coefficient (see Appendix 3.C).

Stresses in the concrete in tension may be calculated on the assumption that the stress distribution is triangular, having a value of zero at the neutral axis and a value of 1 MPa at the centroid of the tension steel, instantaneously reducing to 0,55 MPa in the long term.

- (b) The concrete and the steel are both assumed to be fully elastic in tension and in compression. The elastic modulus of the steel may be taken as 200 GPa and the elastic modulus of the concrete is as specified above in (a), both in compression and in tension.

In assessing the total long-term curvature of a section, the following procedure may be adopted.

- (i) Calculate the instantaneous curvatures under the total load and under permanent load.
- (ii) Calculate the long-term curvature under the permanent load.
- (iii) Add to the long-term curvature under the permanent load the difference between the instantaneous curvatures under the total and permanent loads.

TABLE 32 Values of ρ_e for calculating shrinkage curvatures

$\frac{100A_s'}{bd}$ \ $\frac{100A_s}{bd}$	0,00	0,25	0,50	0,75	1,00	1,25	1,50	1,75	2,00
0,25	0,44	0,31	0,26	0,22	0,20	0,18	0,17	0,16	0,15
0,50	0,56	0,31	0,26	0,22	0,20	0,18	0,17	0,16	0,15
0,75	0,64	0,45	0,26	0,22	0,20	0,18	0,17	0,16	0,15
1,00	0,70	0,55	0,39	0,22	0,20	0,18	0,17	0,16	0,15
1,50	0,80	0,69	0,57	0,45	0,32	0,18	0,17	0,16	0,15
2,00	0,88	0,79	0,69	0,60	0,49	0,39	0,28	0,16	0,15
2,50	0,95	0,87	0,79	0,70	0,62	0,53	0,44	0,35	0,25
3,00	1,00	0,94	0,86	0,79	0,72	0,64	0,57	0,49	0,40
3,50	1,00	1,00	0,93	0,87	0,80	0,74	0,67	0,60	0,52
4,00	1,00	1,00	1,00	0,93	0,87	0,81	0,75	0,69	0,62

(iv) Add to the resultant curvature the shrinkage curvature calculated from the formula:

$$\frac{1}{r_{cs}} = \frac{\rho_o \epsilon_{cs}}{d} \dots\dots\dots (45)$$

where $\frac{1}{r_{cs}}$ is the shrinkage curvature

ρ_o is a coefficient that depends upon the percentages of tension and compression steel in the section (values of ρ_o can be obtained from Table 32.

ϵ_{cs} is the free-shrinkage strain, in the assessment of which allowance should be made for the shape of the section

d is the effective depth of the member

It is occasionally useful to be able to calculate the amount of creep or shrinkage curvature at some specific time after loading. This may be assessed by reference to Appendix 3.C.

3.A.2.3 Calculation of deflection from curvatures

The deflected shape of a member is related to the curvatures by the equation:

$$\frac{1}{r_x} = \frac{d^2 a}{dx^2} \dots\dots\dots (46)$$

where $\frac{1}{r_x}$ is the curvature at a point x

a is the deflection at x .

Deflections may be obtained directly from this equation by calculating the curvatures at successive points along the member and using a numerical integration technique. Alternatively, the simplified approach below may be used.

The deflection is calculated from the equation:

$$a = K_1 l^2 \frac{1}{r_b} \dots\dots\dots (47)$$

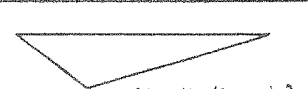
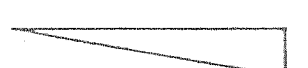
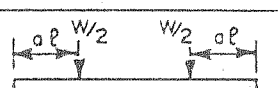
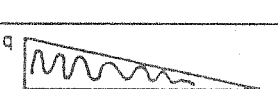
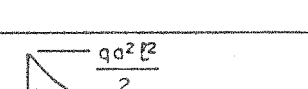
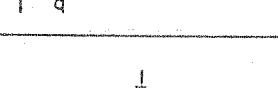
where a is the deflection

K_1 is a constant that depends on the shape of the bending moment diagram and relates the deflection at the required point to the curvature. Values of K_1 are given in Table 33.

l is the effective span of the member

$\frac{1}{r_b}$ is the curvature at mid-span or, for cantilevers, at the support section.

TABLE 33 Values of K_1 for various bending moment diagrams

LOADING	BENDING MOMENT DIAGRAM	K_1
		0,125
	 $M = Wa(1-a)l$	$\frac{4a^2 - 6a + 1}{48a}$ if $a = 1/2$ $K_1 = 1/12$
		0,0625
	 $M = \frac{Wa^2}{2}$	$0,125 - \frac{a^2}{6}$
	 $\frac{q l^2}{8}$	0,104
	 $\frac{q l^2}{15,6}$	0,102
	 M_a, M_b, M_c	$K_1 = 0,104 \left(1 - \frac{\beta}{10}\right)$ $\beta = \frac{M_a + M_b}{M_c}$
	 $W a l$	end deflection $= \frac{a(3-a)}{6}$ load at end $K_1 = 0,333$
	 $\frac{q a^2 l}{2}$	$\frac{a(4-a)}{12}$ if $a = 1$ $K_1 = 0,25$
	 M_a, M_b, M_c	$K_1 = 0,083 \left(1 - \frac{\beta}{4}\right)$ $\beta = \frac{M_a + M_b}{M_c}$
	 $\frac{q l^2}{24} (3-4a^2)$	$\frac{1}{80} \frac{(5-4a^2)^2}{3-4a^2}$

APPENDIX 3.B

ELASTIC DEFORMATION OF CONCRETE

The modulus of elasticity is primarily dependent on the crushing strength of the concrete. It is, however, influenced by the elastic properties of the aggregate and, to a lesser extent, by the conditions of curing and age of the concrete, the mix proportions and the type of cement. For concrete made with natural aggregates and having a density of 2 300 kg/m³ or more, the static or dynamic modulus of elasticity may be taken from Table 34 for concrete of various compressive strengths-(but see 2.3.2.1(a)).

If a more accurate figure is required for particular materials and a particular mix, tests should be done. Concretes made with aggregate from a few particular sources may have a modulus of elasticity substantially outside the range given in Table 34. The use of these materials may be permitted provided the appropriate value of the elastic modulus obtained from tests is used in the design calculations.

It may be convenient to use the dynamic modulus method of testing to obtain an estimated value for the static secant modulus, using the formula:

$$E_c = 1,25E_{cq} - 19 \dots\dots\dots (48)$$

Such an estimated value will generally be correct within ± 4 GPa.

For low-density-aggregate concrete having a density between 1 440 kg/m³ and 2 300 kg/m³, the values of the static modulus given in Table 34 should be multiplied by $D_c/2\,300$, where D_c is the density of the low-density-aggregate concrete in kg/m³.

Neither the equation nor the values relating to the dynamic modulus of elasticity given in Table 34 are applicable to concrete containing low-density aggregate.

TABLE 34 Elastic moduli

Compressive strength f_{cu} MPa	Static secant modulus, E_c		Dynamic modulus, E_{cq}	
	Mean value GPa	Typical range GPa	Mean value GPa	Typical range GPa
20	25	21 - 29	35	31 - 39
25	26	22 - 30	36	32 - 40
30	28	23 - 33	38	33 - 43
40	31	26 - 36	40	35 - 45
50	34	28 - 40	42	36 - 48
60	36	30 - 42	44	38 - 50

APPENDIX 3.C

SHRINKAGE AND CREEP

3.C.1 General

Shrinkage and creep affect deformation and stresses in many structures. The effect depends on the inherent shrinkage and creep of the concrete, on the size of the members, and on the amount and distribution of reinforcement.

The deformation given in 3.C.2 and 3.C.3 is essentially that given in the *CEB-FIP International Recommendations for the Design and Construction of Concrete Structures*, 1970, as amended in May 1972. However, to assess the behaviour of reinforced or prestressed concrete, the succeeding sections must be taken into account. Subject to approval by the responsible bridge authority, more recent information based on reliable research, such as Appendix E of the *CEB-FIP Model Code for Concrete Structures* of 1978, may be used. The CEB-FIP data are valid only for Portland cement concretes of normal quality, hardened under normal conditions, and subjected to service stresses equal at most to 40 % of the ultimate strength. The data provide no more than a working basis. In particular, the type of aggregate may seriously affect the magnitude of shrinkage and creep, sandstone generally leading to high, and limestone to low, deformation.

Where available, the results of tests on the materials to be used should be applied in assessing shrinkage and creep behaviour.

3.C.2 Creep

In order to evaluate the order of magnitude of time-dependent deformations due to creep under working conditions, use may be made of the theory of linear creep. For a constant stress, f_c , this theory allows calculation of the final creep deformation from the formula:

$$\Delta_{cc} = \frac{f_c}{E_{28}} \phi \dots\dots\dots (49)$$

In this formula, E_{28} is the value of the secant modulus of elasticity of the concrete at the age of 28 days, which gives an indication of the quality of the concrete and ϕ is a coefficient covering the particular working conditions envisaged. This coefficient is equal to the product of five partial coefficients (see Figures 19 to 23):

$$\phi = k_L k_m k_c k_e k_j$$

where k_L depends on the environmental conditions

k_m depends on the degree of hardening (maturity) of the concrete at the time of loading

k_c depends on the composition of the concrete

k_e depends on the effective thickness of the member

k_j defines the development of deformation as a function of time.

The value given below for ϕ , calculated from the values of these different partial coefficients, is an average value. When creep has a large influence on the limit state under consideration, an increase or reduction of the order of 15 % should be considered, so as to cover the most unfavourable case.

If the stresses producing creep are themselves influenced by creep, or if they vary continuously, it will be necessary to use iterative methods or to revert to appropriate analytical methods.

When creep has a very large effect on stresses, it may be advantageous to produce curves giving k_m and k_j from equations.

The degree of hardening of the concrete at the time of loading (k_m) exerts an influence at least as large as the environmental conditions (k_L).

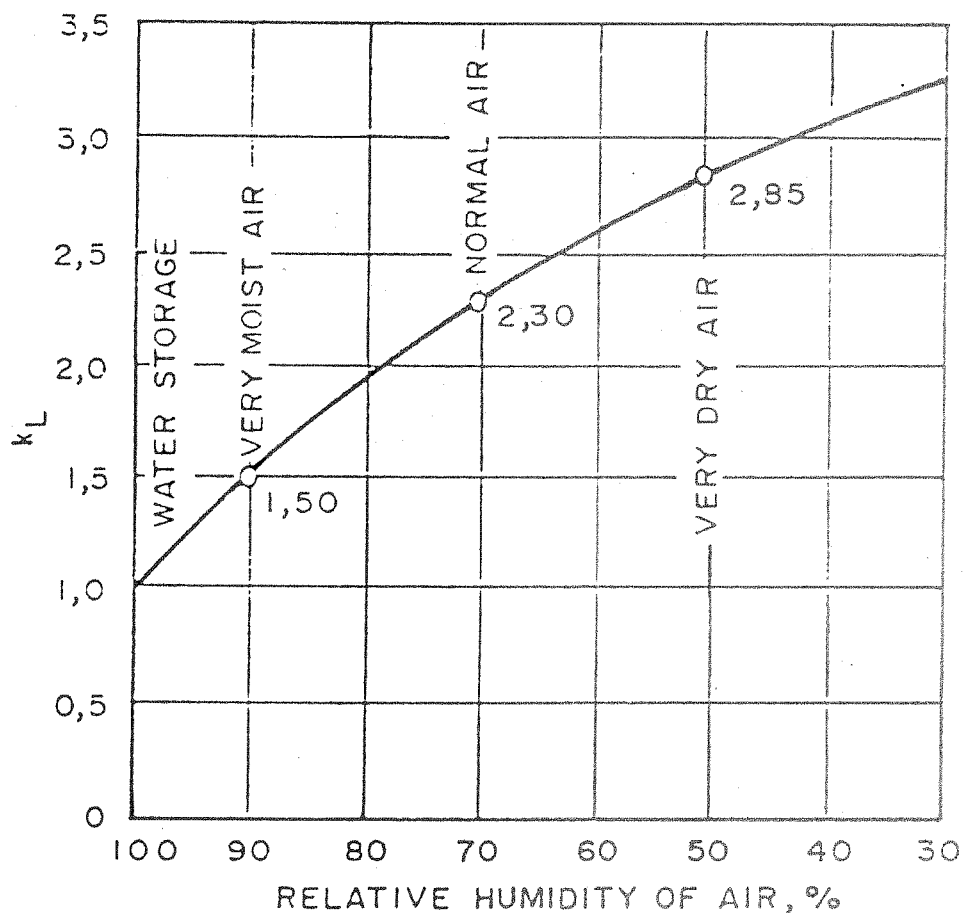


FIGURE 19
COEFFICIENT k_L (ENVIRONMENTAL CONDITIONS) FOR CREEP

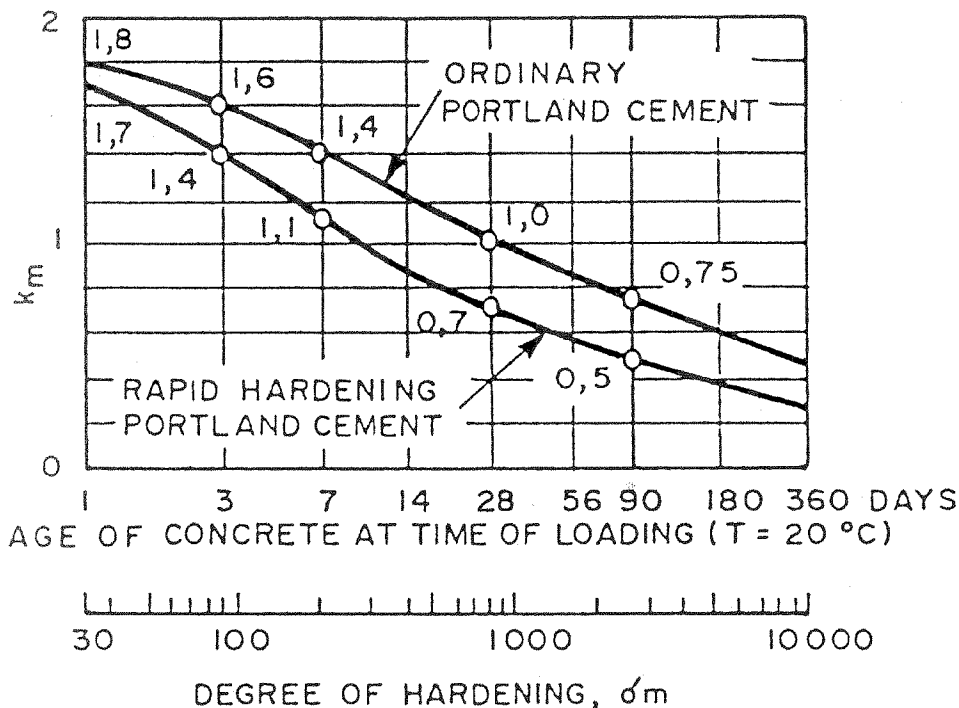


FIGURE 20
COEFFICIENT k_m HARDENING (MATURITY) AT THE TIME OF LOADING

The values in Figure 20 refer to Portland cement, hardened under normal conditions, ie at an average content temperature of 20 °C and with protection against excessive loss of moisture.

If the concrete hardens at a temperature other than 20 °C, the age at loading is replaced by the corresponding degree of hardening:

$$\delta_m = \sum j_m (T + 10 \text{ °C}) \dots\dots\dots (50)$$

where δ_m represents the degree of hardening at the time of loading

j_m represents the number of days over which hardening has taken place at T °C.

The effective thickness, h_e , is the ratio of the area of the section A to the semi-perimeter $u/2$ in contact with the atmosphere. If one of the dimensions of the section under consideration is very large compared with the other, the effective thickness will correspond approximately to the actual thickness.

If the dimensions are not constant along the member, an average effective thickness can be estimated by paying particular attention to those sections in which the stresses are highest.

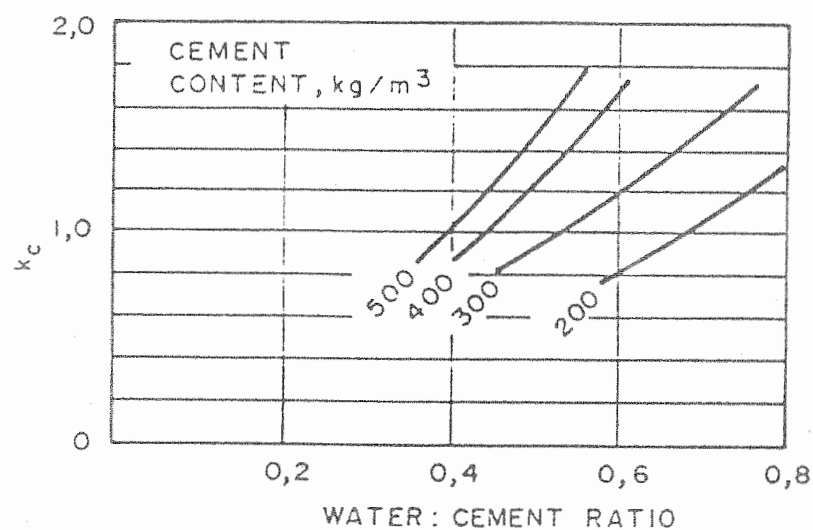


FIGURE 21
COEFFICIENT k_c (COMPOSITION OF THE CONCRETE)

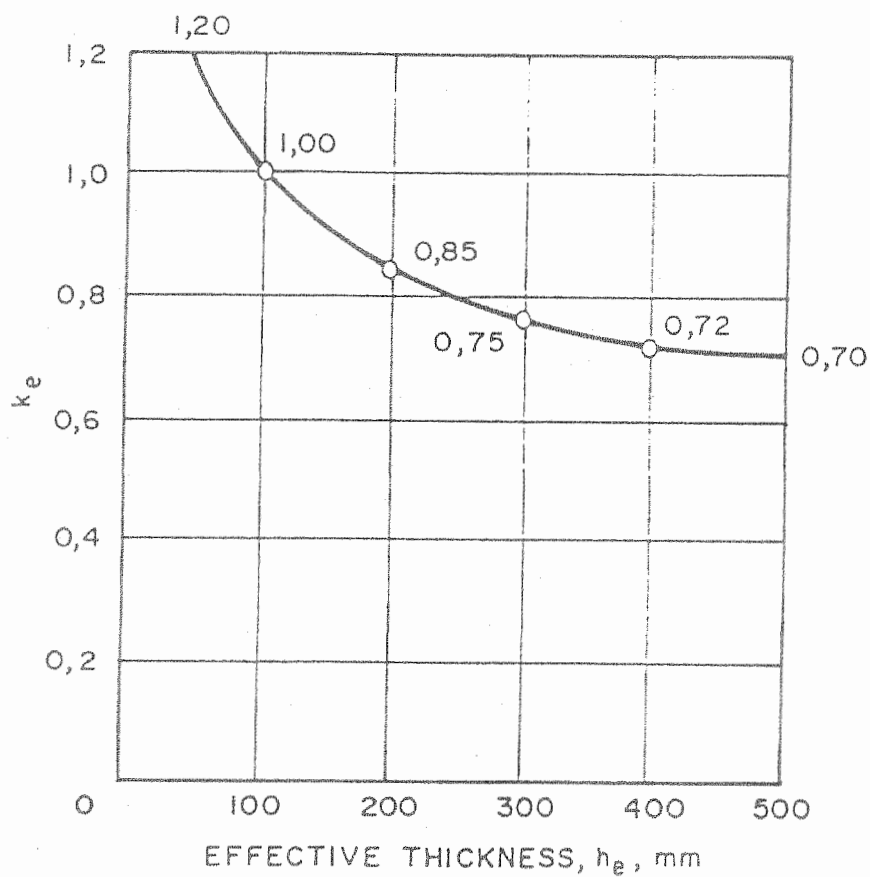


FIGURE 22
COEFFICIENT k_e (EFFECTIVE THICKNESS) FOR CREEP

In general, the final creep deformation, Δ_{cc} , for concretes containing low-density aggregate is greater than that for concretes containing normal aggregate. This difference is a little smaller for high-strength concretes than for low-strength concretes and depends on the modulus of elasticity of the aggregate. The final creep deformation should be deduced from tests. Alternatively, it may be calculated by giving E_{28} the value it would have for normal-aggregate concretes and by multiplying the result obtained by 1,6 ie

$$\Delta_{cc} = \frac{1,6f_c}{E_{28}}$$

An accelerated method for determining creep has also been developed*.

At a given time, t , after application of the loads, the influence of a stress, f_{cj} , in effect at the instant, j , and subject at any moment, i , to a variation intensity, f_{ci} , may be expressed as:

$$\Delta_{cct} = \frac{1}{E_{28}} [f_{cj}\phi_{(t-j)} + \Sigma f_{co}\phi_{(t-i)}] \dots\dots\dots (51)$$

or

$$\Delta_{cct} = \frac{k_L k_c k_e}{E_{28}} [f_{cj} k_m k_{j(t-j)} + \Sigma f_{co} k_m k_{j(t-i)}] \dots\dots\dots (52)$$

where Δ_{cct} is the resulting strain.

As shown in Equations 51 and 52, loads should always be superimposed on the assumption that the stress applied at the beginning of a particular period operates until the end of the same period. The same method may be applied for all later stress changes; in effect, the values of k_m have been determined by these hypotheses.

3.C.3 Shrinkage

The shrinkage deformation, Δ_{cs} , at any instant may be determined from the product of four partial coefficients:

$$\Delta_{cs} = k_L k_c k_e k_j$$

where k_L depends on the environmental conditions

k_c depends on the composition of the concrete

k_e depends on the effective thickness of the member

k_j defines the development of shrinkage as a function of time.

* NEVILLE, A M AND LISZKA, W Z. *Accelerated determination of creep of lightweight aggregate concrete*, Civil Engineer, June 1973, pp 515-519.

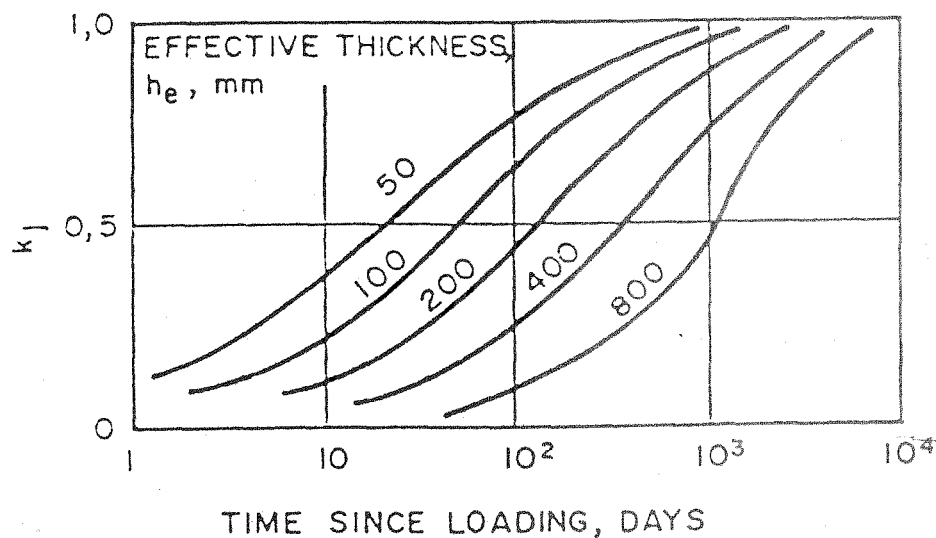


FIGURE 23
COEFFICIENT k_i (VARIATION AS A FUNCTION OF TIME)

In addition, it is possible to add a coefficient for internal restraint, k_2 , which depends on the geometric ratio of longitudinal reinforcement ρ .

As a general rule, Δ_{cs} , as a function, k_2 , gives the reduction in length of the fibre at the centre of gravity of the steel under consideration. The more detailed approach of 3.C.4 is preferable.

The average values of the partial coefficients, k_L , k_c , k_e and k_i , as functions of the parameters that define them, may be taken from the figures listed below which are valid only for concretes that have been protected from excessive losses of moisture in their early days.

For unreinforced concrete, the average values of k_L can be taken from Figure 24. Where embedded electric heating is used, values of the coefficient k_L should be based on experience.

For coefficient k_c (composition of the concrete), the same coefficients may be used as for creep (refer to Figure 21).

Figure 25 gives the values of k_e , using the same definition of effective thickness, h_e , as for creep

For coefficient k_i (variation as a function of time), the same coefficients may be used as for creep (refer to Figure 23).

Where environmental conditions remain constant, the deformation due to shrinkage in an interval of time $(t - i)$ is equal to:

$$\Delta_{cs}(t - i) = k_L k_c k_e (k_{it} - k_{ii}) \dots \dots \dots (53)$$

At early ages, the shrinkage of a protected concrete is lower than that of an unprotected concrete (this is important when trying to prevent cracking in concrete that is young and therefore of low strength). The difference decreases with time and finally vanishes. This occurrence is less noticeable in massive members.

It has been shown experimentally that the shrinkage of structural low-density concretes is between one and two times that of normal-aggregate concretes with the same compressive strength.

Some natural aggregates exhibit appreciable shrinkage; concrete made with such aggregates will undergo greater shrinkage than predicted and appropriate precautions should be built into the design.

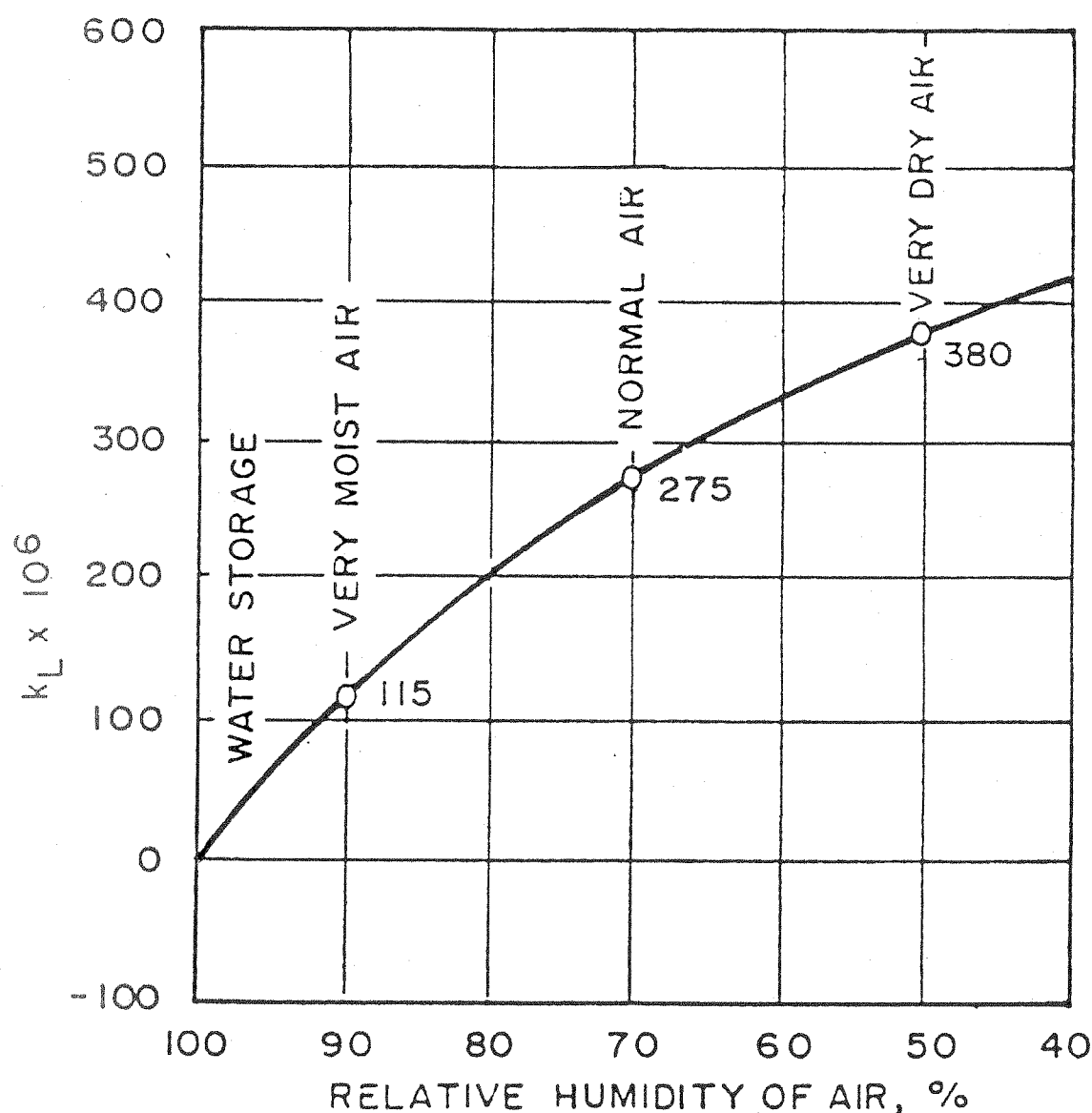


FIGURE 24
COEFFICIENT k_L (ENVIRONMENTAL CONDITIONS) FOR SHRINKAGE

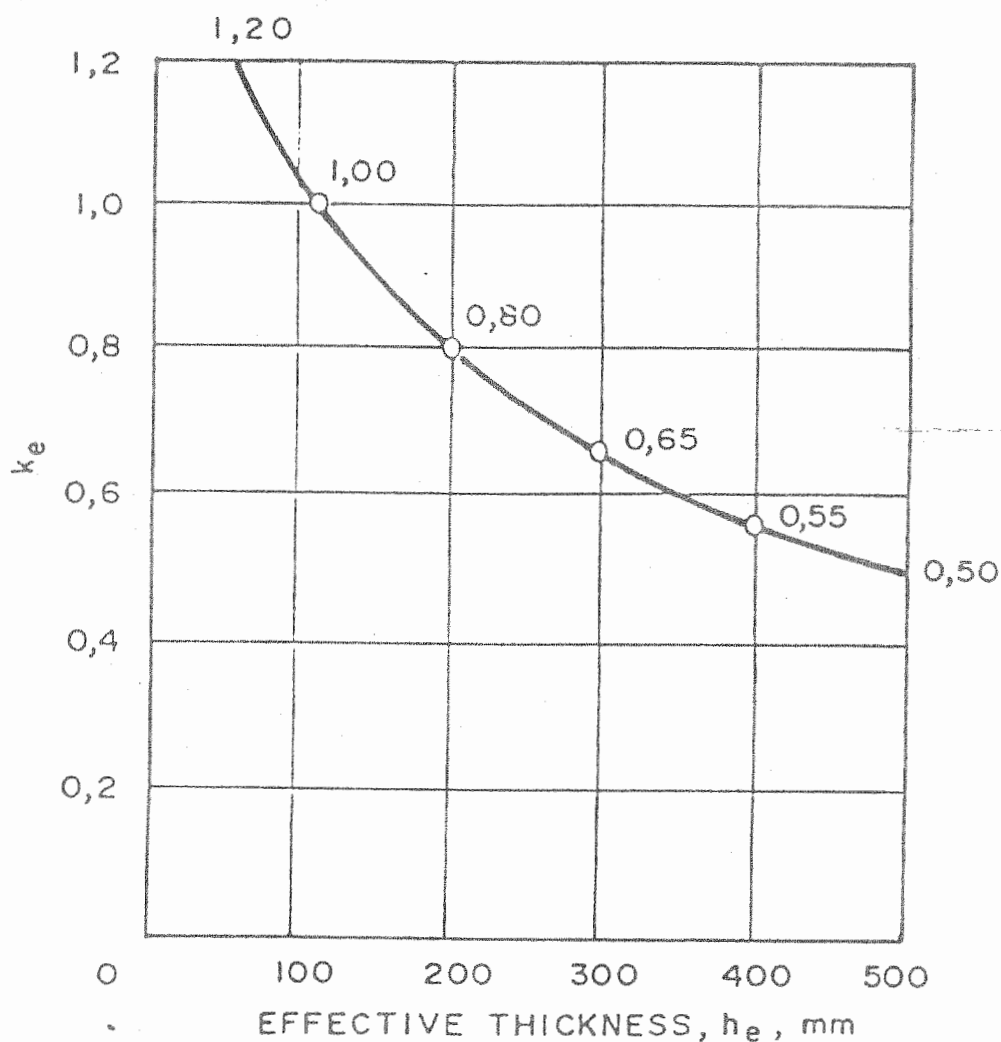


FIGURE 25
COEFFICIENT k_e (EFFECTIVE THICKNESS) FOR SHRINKAGE

3.C.4 Reinforced Concrete

The values given in 3.C.3 apply to plain (unreinforced) concrete members. For structural members containing reinforcement, the magnitude of shrinkage and creep that is likely to be realized is greatly reduced.

Various analytical methods may be used, but a practical method of computing directly the strain under a varying stress, or the stress under a constant or varying strain, is to use a relaxation coefficient:

$$\eta = \int_{j_i}^{j_{\infty}} \frac{df_{cj}}{dj} \cdot \frac{1}{\Delta f_{c\infty}} \cdot \frac{k_{mj}}{k_{mi}} dj \dots\dots\dots (54)$$

where j_{∞} is the age at the end of the life for the structure
 j_i is the age at first loading
 f_{cj} is the stress in the concrete at time of application (j) of an increment of stress
 j is the age at application of an increment of stress
 $\Delta f_{c\infty}$ is the change in stress in the concrete due to creep at time t_{∞}
 k_{mi} is the coefficient k_m for age at first loading

For various values of $\phi_n = \phi/k_m$, the value of η can be obtained from Figure 26.

To calculate axial stresses or strains due to shrinkage, the amount and position of reinforcement should be taken into account, the free shrinkage, ie that given by Δ_{cs} , being multiplied by an appropriate coefficient. For axially loaded and symmetrically reinforced members, this coefficient is:

$$\phi_2 = \frac{1}{1 + \rho \alpha_e (1 + \eta \phi)} \dots \dots \dots (55)$$

where α_e is the modular ratio at first introduction of stress
 ρ is the geometric ratio of reinforcement

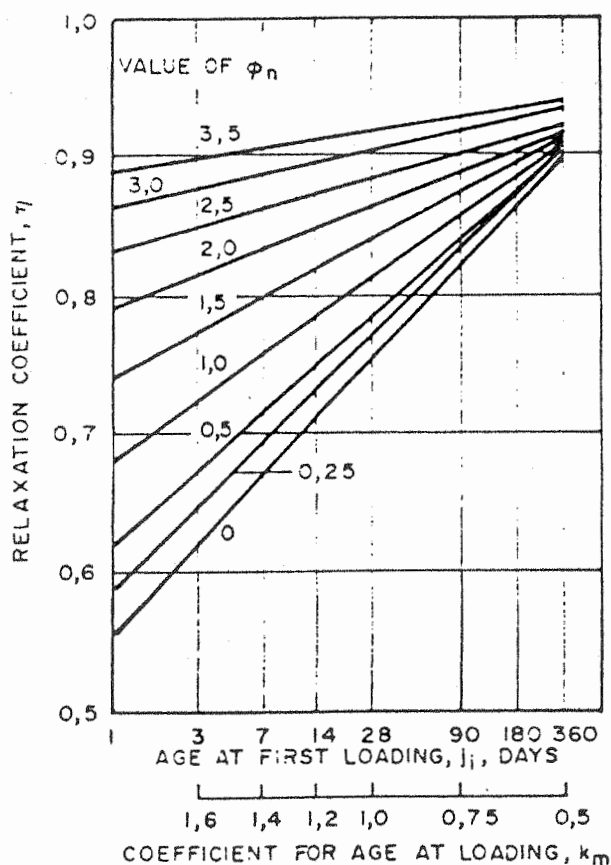


FIGURE 26
RELAXATION COEFFICIENT η

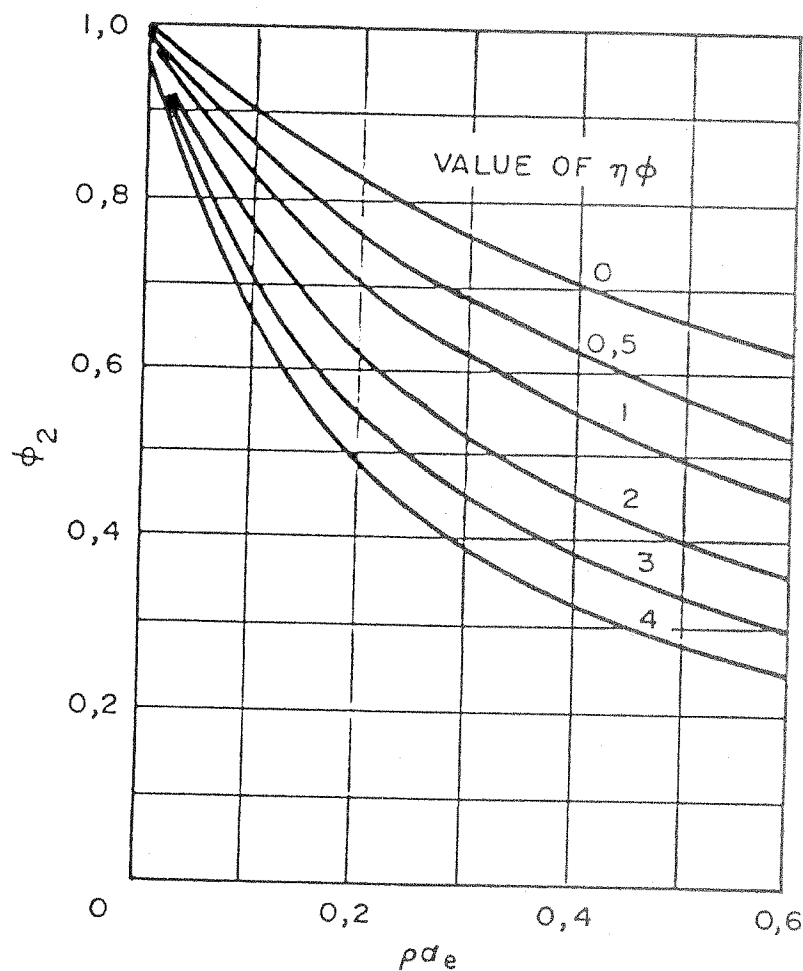


FIGURE 27
COEFFICIENT ϕ_2

The values of ϕ_2 as a function of $\rho\alpha_e$ and $\eta\phi$ are shown in Figure 27.

For singly-reinforced concrete members, the creep reduction coefficient that is applied to free shrinkage in order to calculate the shortening at the level of reinforcement is:

$$\phi_3 = \frac{1}{1 + \rho\alpha_e (1 + a_s^2/i^2)(1 + \eta\phi)} \dots\dots\dots (56)$$

where a_s is the distance of the centroid of the steel from the centroid of the net concrete section

i is the radius of gyration of the net concrete section.

The values of ϕ_3 at two levels of a_s/i , corresponding in rectangular beams to a_s/h (a_s /total depth of section) equal to 0.40 and 0.45, are shown in Figures 28 and 29.

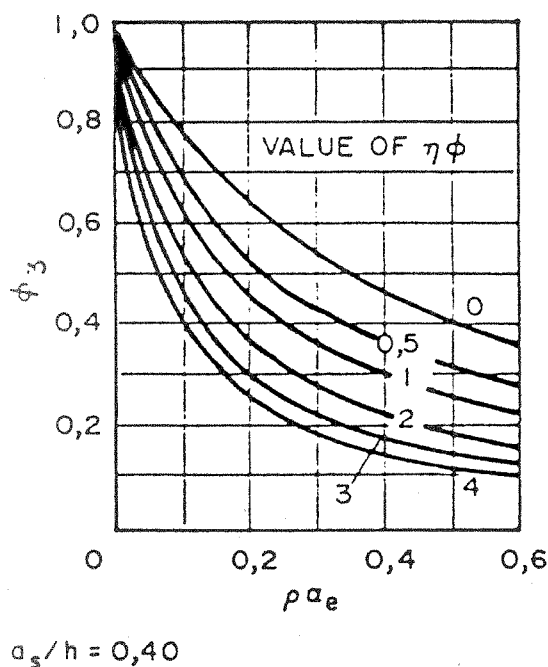


FIGURE 28
COEFFICIENT ϕ_3
 $a_s/h = 0,40$

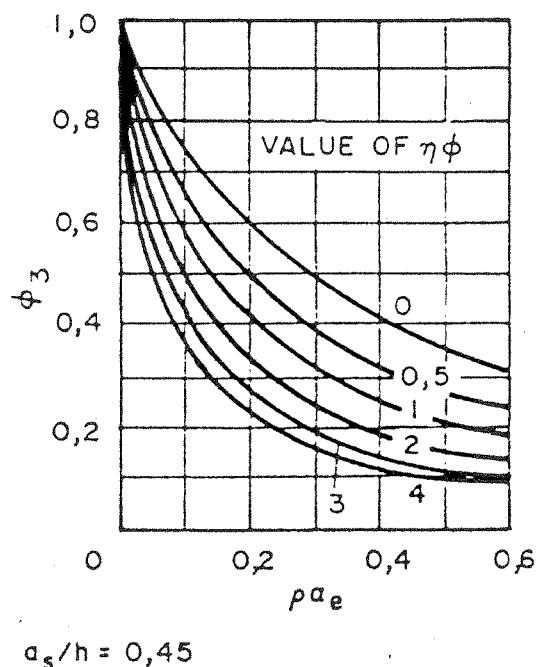


FIGURE 29
COEFFICIENT ϕ_3
 $a_s/h = 0,45$

The change in stress due to creep in a reinforced concrete beam subjected to a bending moment is that induced by the moment at the time of its application multiplied by a coefficient. For symmetrically reinforced symmetrical sections this is:

$$\phi_4 = \frac{1}{1 + \rho\alpha_s (1 + \eta\phi) a_s^2/i^2} \dots\dots\dots (57)$$

and for singly reinforced sections:

$$\phi_5 = \frac{\rho\alpha_s}{1 + \rho\alpha_s} + \frac{1 + \rho\alpha_s(1 + \eta\phi)}{1 + \rho\alpha_s (1 + a_s^2/i^2) (1 + \eta\phi) (1 + \rho\alpha_s)} \dots\dots\dots (58)$$

Other analyses have been published*.

3.C.5 Prestressed Concrete

Prestressed concrete can be considered as a more general case of reinforced concrete, but the presence of the prestress makes the effects of shrinkage and creep especially significant.

* NEVILLE, A. M. *Creep of concrete: plain reinforced and prestressed*. North Holland Publishing Co., 1970, 622 pp.

- (a) In post-tensioned members with bonded tendons, the loss of prestress due to shrinkage and creep at time t can be calculated from:

$$\Delta p = \frac{\alpha_e f_{co} \phi_{ti} + \Delta_{cst} E_s}{1 + \rho \alpha_e (1 + a_s^2/i^2) (1 + \eta \phi_{ti})} \dots\dots\dots (59)$$

where f_{co} is the stress in the concrete at the level of the tendon due to initial prestress and dead load.

ϕ_{ti} is the creep coefficient at time t for a load applied at time i

Δ_{cst} is the shrinkage at time t .

The loss of prestress due to relaxation of the steel must be added to the above loss.

In pretensioned members, there are additional losses of prestress due to the shrinkage that occurs between the setting of the concrete and the time of transfer, and due to the relaxation that occurs between the stressing and the time of transfer.

The above approach assumes a constant value of the modulus of elasticity of the concrete from the time of prestressing onward, and a shrinkage time function of the same form as the creep time function for the coefficient k_j . If these assumptions are not reasonable, a step-by-step procedure may be used.

- (b) Non-prestressed reinforcement should be taken into account when considering the effects of shrinkage and creep if the second moment of the transformed area of the non-prestressed reinforcement about the centroid of the concrete section represents a significant proportion (say 10%) of the second moment of the concrete section alone. Although the creep and shrinkage losses are reduced by the presence of the non-prestressed reinforcement, the relaxation loss is increased because the elastic and creep recoveries are smaller in the presence of the additional reinforcement.

With single-layer prestressing, the creep reduction coefficients ϕ_2 , ϕ_3 or ϕ_4 can be used to make allowance for the presence of non-prestressed reinforcement. These coefficients are determined on the basis of the non-prestressed reinforcement alone.

- (c) The change in the curvature of a prestressed member due to creep is:

$$\phi = \frac{1}{a_s} \left[\frac{1 + \rho \alpha_e (1 + \eta \phi)}{1 + \rho \alpha_e (1 + a_s^2/i^2) (1 + \eta \phi)} \epsilon_{c1} - \epsilon_{c2} \right] \dots\dots\dots (60)$$

where ϵ_{c1} is the creep strain in concrete at the level of the tendon due to initial prestress and dead load

ϵ_{c2} is the creep strain in concrete at the centroid of the section due to initial prestress and dead load.

Deflection due to creep can be calculated in the usual manner from the change in curvature.

APPENDIX 3.D

COVER AND SPACING OF CURVED TENDONS IN DUCTS FOR PRESTRESSED CONCRETE

3.D.1 General

Where curved tendons are used in post-tensioning, the positioning of the tendon ducts and the sequence of tensioning should be such as to prevent:

- (a) bursting of the side-cover perpendicular to the plane of curvature of the ducts;
- (b) bursting of the cover in the plane of curvature of the ducts and
- (c) crushing of the concrete separating the tendon ducts in the same plane of curvature.

Pending the availability of further research data, the rules given in 3.D.2 and 3.D.3 may be applied.

3.D.2 Cover

In order to prevent bursting of the cover:

- (a) perpendicular to the plane of curvature, and
- (b) in the plane of curvature, eg where the curved tendons run close to and approximately parallel to the surface of a member,

the cover should be in accordance with the values given in Table 35. Where tendon profilers or spaces are provided in the ducts, and these are of a type that will concentrate the radial force, the values given in Table 35 will need to be increased.

The cover for a given combination of duct internal diameter and radius of curvature, shown in Table 35, may be reduced pro rata with the square root of the tendon force when this is less than the value tabulated, subject to the requirements of 4.9.2.2.

When detail (b) above is used and the tendon develops radial forces perpendicular to the exposed surface of the concrete, the duct should be restrained by stirrup reinforcement anchored into the members.

3.D.3 Spacing

In order to prevent crushing of the concrete between ducts, the minimum spacing should be as given below.

- (a) *In the plane of curvature*: use the values given in Table 36 or the value required by 4.9.3.2, whichever is the greater.
- (b) *Perpendicular to the plane of curvature*: follow the requirements of 4.9.3.2. Where tendon profilers or spaces are provided in the ducts, and these are of a

type that will concentrate the radial force, the values given in Table 36 will need to be increased. If necessary, reinforcement should be provided between ducts.

The distance for a given combination of duct internal diameter and radius of curvature, shown in Table 36, may be reduced pro rata with the tendon force when this is less than the value tabulated, subject to the requirements of 4.9.3.2.

In exceptional cases, to achieve the required minimum spacing of the ducts it may be possible to tension and grout the tendon having the smallest radius of curvature first, and to allow an interval of 48 hours to elapse before tensioning the next tendon. In this case the spacing requirements given in 4.9.3.2 apply.

TABLE 35 Minimum cover to ducts perpendicular to plane of curvature (mm)

Radius of curvature of duct (m)	Duct internal diameter (mm)															
	19	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170
	Tendon force (kN)															
	296	387	960	1337	1920	2640	3360	4320	5183	6019	7200	8640	9424	10336	11248	13200
2	50	55	155	220	320	445										
4		50	70	100	145	205	265	350	420							
6			50	65	90	125	165	220	265	310	375	460				
8				55	75	95	115	150	185	220	270	330	360	395		
10				50	65	85	100	120	140	165	205	250	275	300	330	
12					60	75	90	110	125	145	165	200	215	240	260	315
14					55	70	85	100	115	130	150	170	185	200	215	260
16					55	65	80	95	110	125	140	160	175	190	205	225
18					50	65	75	90	105	115	135	150	165	180	190	215
20						60	70	85	100	110	125	145	155	170	180	205
22						55	70	80	95	105	120	140	150	160	175	195
24						55	65	80	90	100	115	130	145	155	165	185
26						50	65	75	85	100	110	125	135	150	160	180
28							60	75	85	95	105	120	130	145	155	170
30							60	70	80	90	105	120	130	140	150	165
32							55	70	80	90	100	115	125	135	145	160
34							55	65	75	85	100	110	120	130	140	155
36							55	65	75	85	95	110	115	125	140	150
38							50	60	70	80	90	105	115	125	135	150
40	50	50	50	50	50	50	50	60	70	80	90	100	110	120	130	145

Note: The tendon force shown is the maximum normally available for the given size of duct. (Taken as 80 % of the characteristic strength of the tendon.)

TABLE 36 Minimum distance between centre lines of ducts in plane of curvature (mm)

Radius of curvature of duct (m)	Duct internal diameter (mm)															
	19	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170
Tendon force (kN)																
	296	387	960	1337	1920	2640	3360	4320	5183	6019	7200	8640	9424	10336	11248	13200
2	110	140	350	485	700	960										
4	55	70	175	245	350	480	610	785	940							
6	38	60	120	165	235	320	410	525	630	730	870	1045				
8			90	125	175	240	305	395	470	545	655	785	855			
10			80	100	140	195	245	315	375	440	525	630	685	750	815	
12						160	205	265	315	365	435	525	570	625	680	800
14						140	175	225	270	315	375	450	490	535	585	785
16							160	195	235	275	330	395	430	470	510	600
18								180	210	245	290	350	380	420	455	535
20									200	220	265	315	345	375	410	480
22											240	285	310	340	370	435
24												265	285	315	340	400
26												260	280	300	320	370
28																345
30																340
32																
34																
36																
38																
40	38	60	80	100	120	140	160	180	200	220	240	260	280	300	320	340

Note 1: The tendon force shown is the maximum normally available for the given size of duct. (Taken as 80 % of the characteristic strength of the tendon.)

2: Values less than 2 x duct internal diameter are not included.

3: For non-circular ducts, the radius of the duct should be based on the equivalent area of the duct.