Draft

Code of Practice
for the
Design of Road Bridges and Culverts

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FORWARD

This document forms part of the ongoing role of the Southern Africa Transport and Communications Commission (SATCC) to coordinate the development and use of the Southern African Development Community (SADC) regional transport system.

The stated objectives for the standardisation of regional roads and bridge standards and specifications are:

- Achieve economic and technical efficiency in road transportation within the SADC region through the provision of uniform roads networks at optimal cost.
- To provide the same quality of service throughout the region.
- To improve the efficiency of the road transport system by minimising road accidents.
- To improve the efficiency of the road transport system by reducing energy consumption and hence the unnecessary use of foreign currency.

This Code is issued on behalf of the Southern Africa Transport and Communications Commission (SATCC) as a manual to be used in the design of highway bridges and culverts in order to achieve greater uniformity of standards in bridge loading and design throughout the SADC region. Whereas some deviations to suit local requirements may be necessary, it is important that the agreed standards and design principles expounded in this document be adhered to.

Although the Code is too specific and detailed to be strictly classified as a model code, this does not imply the necessity for rigid adherence to all its detail. In order not to retard progress it is desirable that competent designers should retain the right of discretion to apply new and improved theories or methods based on proven research, provided that the reliability of the structure is not impaired thereby. It is the intention that this Code shall be updated or extended as and when required by the issuing of subsequent editions.

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

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PART 1: GENERAL STATEMENT

1. INTRODUCTION

This Code in its present form covers the design of concrete highway bridges and culverts and is aimed at the achievement of acceptable levels of probability so that the structure being designed will remain fit for the required purpose during some reference period, with its intended life taken into consideration, i.e. the structures should attain the required measure of safety, serviceability and durability. The Code is based on the principles of Limit State Design outlined in ISO 2394, "General Principles for the Verification of the Safety of Structures"[1] and is modelled on Volumes I and II of the International System of Unified Standard Codes of Practice for Structures published by the Comitê Euro-International du Beton in 1978[2]. The main reference is the South African Bridge Code TMH7: Code of Practice for the design of highway bridges & culverts in South Africa[3], which in turn makes reference to BS5400, Parts 1, 2 and 4[4] and the National Building Code of Canada[5].

The application and interpretation of this Code in design shall be entrusted only to appropriately qualified and experienced Professional Engineers and the construction shall be carried out under appropriately qualified supervisors. As the Code is based on Limit State Design, it differs in principle from the previously used "Factor of Safety" or "Permissible Stress" method of design which formed the basis of its predecessors.

The load factors adopted are judged appropriate in the light of current knowledge as they have been conservatively calibrated on the basis of past experience and judgement. It is, however, anticipated that adjustments will be necessary at periodic intervals as a better understanding of structural reliability[6] and risk is gained from the results of the use of this Code in practice and from empirical calibration studies. Users of this Code should recognise the continuing need for sound engineering judgement in the transition to this new approach.

1.1 Scope

This book comprises: Part 1: **General statement** and Part 2: **Specification for loads**.

The scope of this document is restricted to the trunk road network as defined by the Southern African Transport and Communications Commission (SATCC) of the Southern Africa Department Community (SADC).

This Code applies to reinforced and/or prestressed concrete structures in which ordinary aggregates are used. The Code does not cover the structural use of concrete made with high-alumina cement. The following types of structure will at present remain outside the scope of this Code:

- Components made of plain concrete (except for plain concrete walls and abutments)
- Components reinforced with rolled-steel sections, aerated (hollow or porous) concrete or no-fines concrete components.

Additional parts comprising one or more of the following could be included in the future:

- A code of practice for the design of steel bridges
- A code of practice for the design of composite bridges (structural steel and concrete)

- A code of practice for fatigue
- A code of practice for bridge bearings
- A code of practice for the design of foundations
- Standard specification for materials and workmanship or any other relevant subjects required to form a comprehensive and unified whole.

In the interim, reference shall be made with respect to the excluded chapters, to such special design guidelines and specifications as may from time to time be issued by the relevant authority, or to the relevant Parts of BS5400[4] or applicable South African Bureau of Standards (SABS) Codes of Practice[7,8,9] or other approved codes.

The following sections of Part 1 cover a statement of the general concepts embodied in other Parts of this Code. Various terms are defined and the application of the Limit State principles explained. They include sections on analysis, foundation design and erection and make reference to general aspects of design-checking procedures and the control and acceptance of materials and products.

1.2 References

The titles of publications referred to in each Part of this Code are listed at the end of each respective Part.

1.3 Units

The units shall be those of the S.I. International System of Units based on the decimal system with seven basic units. The units recommended for the presentation of data and results are as follows:

- For loads and forces localized or distributed: kN, kN/m, kN/m²
- For density : mass per unit volume: kg/m³
- Weight per unit volume: kN/m³
- For stresses and strength: N/mm²
- For moments: kNm

1.4 Notation

The notation used complies with ISO Standard 3898[9].

A fuller description of the system of notation used is given in Appendix A of the International System of Unified Standard Codes of Practice for Structures, Volume II, CEB-FIP Model Code for Concrete Structures[2], as well as in Appendix F of BS8110: Part 1 : 1972, the Code of Practice for the Structural Use of Concrete[11].

2. DESIGN PHILOSOPHY

2.1 General

The fundamental understanding of structures has advanced considerably in the course of the last few decades. This applies not only to the safety of structures as reflected in recent developments in Structural Reliability, Theory and Risk Analysis, but also to methods of analysis and optimization procedures.

This Code is based on the CEB/FIP Model Code for Concrete Structures published in 1978[2] and incorporates the abovementioned principles. The procedures necessary for an exact treatment of the subject can be extremely complex and intractable. However, by a process of rationalisation, a practical approach known as the Limit State Design Method, using partial safety factors, has been evolved. This method is simple to apply, yet is fundamentally a great improvement on the Factor of Safety or Working Stress methods.

2.2 Limit State Design Method

Limit state design is the logical and practical procedure that has been evolved to achieve acceptable probabilities so that the structure being designed will remain fit for the required purpose during some reference period with its intended life taken into consideration. Since the consequences of attaining a given limit state (and particularly the ultimate limit state) may vary considerably, different safety margins should be accepted for the different safety classes adopted. The consequences of failure may be considered from two viewpoints:

- Risk to life or concern for public reaction (or aversion) to possible failures.
- Economic consequences due to:
    - Loss of use of the structure and all ancillary costs.
    - Need for replacement or repair.

In the design process or, more accurately, in the codification of that process, the consequences of failure may be classified as:

- Not serious - risk to life negligible and economic consequences small.
- Serious - risk to life exists and economic consequences considerable.
- Very serious - risk to life great and/or economic consequences very great.

In the design process, a calculation model should be established for the specific limit state; this model should incorporate appropriate variables allowing for the uncertainties with respect to actions, the response of the structure as a whole and the behaviour of individual elements of the structure. Further, the significance of the level of workmanship and control should be incorporated. The use of such models should permit design, for the different limit states, to acceptable degrees of safety or structural reliability.

In general, the structural analysis will be in two stages, one dealing with the analysis of the structure as a whole to assess the action effects, and the other dealing with the analysis of the individual sections of the structural elements under the assessed action effects.

Limit state design can therefore be considered as a decision-making process in which various uncertainties have to be accounted for in assessing the design variables in order to obtain an acceptable failure probability. The uncertainties need to be quantified by probability statements, these being made either on the basis of available statistical data, or, where such data are inadequate, on the basis of past experience and judgement.

2.3 Levels of Limit State Design

In the design process it is possible to identify three levels at which the structural safety may be treated and, hence, at which the design could be carried out. These are:

Level 1: A semi-probabilistic process in which the probabilistic aspects are treated specifically in defining the characteristic values of loads or actions and the strength of materials. These are then associated with partial factors, the values for which, although stated explicitly, should be derived whenever possible from a consideration of the probability aspects. Characteristic values are given as functions of mean values, coefficients of variation and distribution types or are assessed from available data and experience. Clearly Level 1 is, and should continue to be, the basis for most design codes. Simplifications within Level 1 are possible and indeed desirable in practice. This is the approach adopted in the preparation of this Code.

Level 2: A design process in which the loads or actions and the strengths of materials and sections are represented by their known or postulated distributions (defined in terms of relevant parameters such as types, mean and standard deviation), and some reliability level is accepted. It is thus a probabilistic design process. (See Appendix 2 of Volume 1, Common Unified Rules for Different Types of Construction and Material, April 1978, CEB[2]).

Level 3: Represents a design process based upon an "exact" probabilistic analysis for the entire structural system, using a full distributional approach, with safety levels based on some stated failure probability interpreted in the sense of relative frequency.

2.4 Limit States - Definition and Classification

2.4.1 General

A structure, or part of a structure, is considered unfit for use when it exceeds a particular state, called a limit state, beyond which it infringes one of the criteria governing its performance or use.

The limit states can be placed in two categories:

(i) The ultimate limit states, which are those corresponding to the maximum load-carrying capacity.

(ii) The serviceability limit states, which are related to the criteria governing normal use and durability.

2.4.2 Ultimate Limit States

The ultimate limit states applicable to this Code are:

(i) Loss of *equilibrium* of a part or the whole of the structure considered as a rigid body.
(ii) *Rupture* of critical sections of the structure, *excessive deformation or resonant vibration*.
(iii) *Transformation of the structure into a mechanism.*
(iv) A post-elastic or post-buckling state of *instability* determined by the current extent of knowledge of the ultimate behaviour of bridge structures and which, in certain cases, related only to the collapse strength of the section considered and not to the collapse strength of the whole structure.
(v) Deterioration, due to *fatigue*, to a stage where failure occurs.

2.4.3 Serviceability Limit States

The serviceability limit states applicable to this Code are:

(i) *Deformation*. The deformation of the structure, or any part of the structure, should not adversely affect the appearance or efficiency of the structure, violate minimum specified clearances, cause drainage difficulties, or give cause for public concern.
(ii) *Local damage or cracking*. Damage occurring in specific parts of the structure, e.g. caused by localized overstressing, which might entail excessive maintenance, or lead to corrosion, and hence adversely affect the appearance or efficiency of the structure, should be limited.
(iii) *Vibration*. Where there is a likelihood of the structure being subjected to excessive vibration from causes such as wind forces or moving traffic resulting in resonance, appropriate analyses should be carried out and measures taken to prevent discomfort or alarm, or impairment of its proper function.

2.5 Other Considerations

(i) *Overall stability and robustness*. The configuration of the structure and the interaction between the structural members should be such as to ensure a robust and stable design. The structure should be designed to support loads caused by normal function, but there should be a reasonable probability that it will not collapse catastrophically or suffer disproportionate damage under the effects of misuse or accident. In this respect, it is important to check that the structure has an adequate resistance to lateral actions.
(ii) *Durability*. Compliance with the relevant clauses of this Code should ensure adequate durability during the design life of the structure. Durability is normally treated by attention to the detailed aspects of design, to the specification and control of materials and workmanship, with provision for protection and routine maintenance as dictated by environmental conditions.
(iii) *Utility and aesthetic quality*. It is imperative that the abovementioned requirements of Limit State Design should not predominate to the exclusion of other important considerations such as the total utility of the bridge in terms of the overall cost-benefit to society, its aesthetic quality and effects on the environment, all of which cannot readily be quantified, but nevertheless should undergo such analyses and assessments as are feasible.

3. ACTIONS

3.1 Definitions of Actions

An action is an assembly of:

- Concentrated or distributed forces (direct actions), or
- imposed or constrained deformations (indirect actions)

applied to a structure due to a single cause.

An action is considered to be a single action if it is stochastically independent, in time and space, of any other assembly of forces, or imposed or constrained deformations, acting on the structure.

3.2 Qualitative Classification of Actions

Actions may be classified according to their variation in time, or space, or according to their dynamic nature.

3.2.1 Classification of Actions According to their Variation in Time

To define the representative values of actions and to determine rules of combinations, the actions may be classified according to their variation in time. Thus there are:

- Permanent actions
- Variable actions
- Accidental actions

<u>3.2.1.1</u> <u>Permanent Actions</u>

Permanent actions for which variations are rare (but with likely occurrences of long duration), or negligible in relation to the mean value; or those for which the variation is in one direction and the actions attain some limiting value.

Permanent actions include:

- The self-weight of structures (except possibly during phases of construction)
- The weight of any controlled superstructure
- Forces applied by earth pressure excluding the effects of moving loads applied to the ground
- Prestress
- Deformations imposed by the mode of construction
- Actions resulting from shrinkage of welds and concrete
- In certain cases, the forces resulting from water pressure.

Support settlement or heaving action and mining subsidence, all actions which tend to attain limiting values and which can be evaluated to varying degrees of accuracy in terms of available methods and information, may, in general, be regarded as permanent actions.

3.2.1.2 Variable Actions

Variable actions for which variations are frequent or continuous, or not monotonic, and not negligible in relation to the mean value. Variable actions include:

- The "working loads" (loads due to traffic)
- The self-weight of structures during certain phases of construction
- Erection loads
- All moving loads and their effects
- Forces resulting from wind, earthquakes, from water generally and temperature and its effects.

3.2.1.3 Accidental Actions

Accidental actions, the occurrence of which, in any given structure and with a significant value, is unlikely during the reference period, but the magnitude of which could be important for certain structures. They are usually assigned nominal values in assessing the resistance of the structure to them. Accidental actions include:

- Impact forces caused by vehicular collisions with elements of the structure
- Explosions
- Unpredictable subsidence of subsoil
- Avalanches of rocks
- Freak storms
- Earthquakes in regions not normally exposed to them.

They are only relevant where the estimated value of the force is neither negligible nor so large that it is unreasonable to ensure the integrity of the structure.

3.2.2 Classification of Actions According to their Variation in Space

To define the load cases to be considered, the actions are divided into two groups:

- *Fixed actions*, whose distribution over the structure is unambiguously defined by deterministic parameters and, possibly, one random parameter. The magnitude of a fixed action may change with time. A load case is defined by fixing the configuration of each of the free actions, for example by means of influence surfaces. Their random magnitude is taken into account by the application of the rules of combination.
- *Free actions*, which may have any arbitrary distribution over the structure within given limits. Free actions cannot be defined by a unique variable, without some idealization.

Actions not falling within these two groups can be considered to consist of a fixed element and a free element.

3.2.3 Classification of Actions According to their Nature

Actions may be of two types:

- *Static actions*, which do not cause significant acceleration of the structure or structural member.
- *Dynamic actions*, which cause significant acceleration of the structure.

Whether or not an action is regarded as dynamic is thus dependent on the structural response, although the dynamic character is correlated with variation in time of the action. In certain cases, the magnitude of an action is dependent on the behaviour of the structure.

In general, most actions may be considered as static actions, taking account of dynamic effects by increasing the magnitude of the actions. *Where this is not the case, a special treatment of safety may be required taking into account the dynamic response of the structure*.

3.3 Actions and Situations

In covering all phases in the life of a structure, i.e. during construction, normal use and possible misuse, a number of situations may conveniently be considered:

- A permanent situation, the duration of which is approximately the same as the reference period or life of the structure.
- Temporary situations, the durations of which are much less than the reference period or life of the structure. These may be either:
  - Transient situations, the probability of occurrence being high and the duration often being random; or
  - accidental situations, the probability of occurrence being very low and the duration being generally very short.

4. PROPERTIES OF MATERIALS

The properties of the materials and their statistical variations should be determined from tests on appropriate standard test specimens. These properties, related to the standard test specimen, should be converted to the relevant properties of the actual material in the structure by the use of conversion factors or functions. Uncertainty about the properties of the material in the structure may be derived from uncertainties about the standard test results and about the conversion factor or function.

If the strength of the material can be assessed prior to its incorporation in the structure, the characteristic strength can be evaluated on the basis of an adequate statistical evaluation of the results available.

Where this is not possible because the material will be produced on site, achievement of the characteristic strength specified should be ensured by adequate production control and acceptance procedures. In certain cases, it may be necessary to stipulate upper and lower characteristic strength values.

5. GEOMETRIC DATA

In the design, account should be taken of the possible variation of the geometric data. In most cases, the variability of the geometric data may be considered to be small, or negligible, in comparison with the variability associated with the actions and the material properties.

Hence, in general, the geometric data may be assumed to be non-random and as specified in the design.

Where the deviation of certain of the geometric data from the prescribed values may have a significant effect on the structural behaviour and the resistance of the structure, the data should be considered as random variables, the parameters of their variability being determined from the prescribed tolerance limits.

6. DEFINITIONS OF DESIGN VALUES

6.1 General

The variability of the actions on a structure is taken into account by defining the actions in terms of characteristic values. Where the necessary data are available, these characteristic values are based on a statistical interpretation of the data; where data are not available, appraisal of the values is based on experience and, possibly, forecasts of the implications of future developments.

The actions to be considered in determining the action effects, S, on the structure, are specified in Part 2 and are described throughout as nominal actions. For certain actions some statistical distributions are available but in most cases nominal values, based on judgement and experience, are given. In making such assessment, reference has also been made to statistical information obtained in other countries and the relevant values used in their codes. The nominal values adopted have been modified to suit local conditions and are considered to approximate to a 100-year return period, except where expressly defined otherwise.

The variability of the strengths and other properties of the construction materials is treated by defining characteristic strengths (and hence properties), related to some standard test specimens and procedures, on a statistical basis.

Where statistical data on the strength of materials are available, characteristic values are given in the appropriate Parts of the Code. Where such data are not available, nominal values are given to be used as characteristic values in all the computations.

For the purposes of this Code the method of Partial Coefficients (Level 1) has been adopted. Two types of partial safety factor are introduced, one for the strength of materials and/or elements, and the other for the actions or action effects.

6.2 Design Actions

The design actions, F^*, are determined from the nominal actions, $F_{k,}$ according to the relation

$$F^* = \gamma_{fL} \cdot F_k$$

where

γ_{fL} is a factor given in Part 2 for each action

γ_{fL} = function ($\gamma_{f1} \cdot \gamma_{f2}$)

where

γ_{f1} takes account of the possibility of unfavourable deviation of the actions from their nominal values; and

γ_{f2} takes account of the reduced probability that various actions acting together will all attain their nominal values simultaneously; it is thus a combination factor for actions and not a safety factor.

6.3 Design Action Effects

The design action effects, S^*, are obtained from the design actions by the relation

$$S^* = \gamma_{f3} \text{ (effects of } F^*)$$
$$= \gamma_{f3} \text{ (effects of } \gamma_{fL} \cdot F_k)$$

where
γ_{f3} is a factor that takes account of inaccurate assessment of the action effects, unforeseen stress distribution in the structure, and variations in dimensional accuracy achieved in construction and the importance of the limit state being considered. Values of γ_{f3} are given in TMH7, Part 3[12]

Where linear relationships can be assumed between actions and action effects, S^* can be determined from

$$S^* = \text{(effects of } \gamma_{f3} \cdot \gamma_{fL} \cdot f_K)$$

6.4 Design Resistance

The design resistance, R^*, may be defined as

$$R^* = \text{function } \frac{(f_k)}{(Y_m)}$$

or optionally

$$R^* = \frac{\text{function } (f_k)}{Y_m}$$

when γ_m can be explicitly treated.

where
f_k = the characteristic (or nominal) strength of the material,
γ_m = a reduction factor specified in TMH7, Part 3[12] (which = function ($\gamma_{m1} \cdot \gamma_{m2}$))

where
γ_{m1} is intended to cover the possible reductions in the strength of the materials in the structure as a whole as compared with the characteristic value deduced from the control test specimen.

γ_{m2} is intended to cover possible weaknesses of the structure arising from any cause other than the reduction in the strength of the materials allowed for in γ_{m1}, and the possible inaccurate assessment of the resistance of elements derived from the strength of the material, including the variations of the dimensional accuracy achieved in construction as they affect the resistance.

In addition to γ_f, a modifying factor, γ_n, may be introduced to adjust either γ_m or γ_f, (γ_f is the generalized symbol for γ_{f1}, γ_{f2} or γ_{f3}); γ_n is considered as a function of two factors γ_{n1} and γ_{n2}

where
γ_{n1} takes account of the type of structural failure, namely brittle or ductile, and γ_{n2} takes account of the consequences of the failure.

This factor, γ_n, takes account of the inherent structural behaviour, e.g. structures or parts of structures in which partial or complete collapse can occur without warning, and the seriousness of the consequences of failure.

It should be assumed that the normal situation implicit in existing codes is probably that associated with serious consequences and ductile failure.

γ_n *should not, however be used explicitly; it merely serves as a rational way of modifying* γ_m *or* γ_f *appropriately*.

In the case of certain materials, where γ_m can be explicitly treated, expression of function (f_k) is given in the relevant parts of the Code. In the case of other materials, expressions for R^* are given.

6.5 Modification to Design Values

For bridges in Southern Africa, no further modifications need normally be made to the design values to take account of the seriousness of attaining a limit state. For the present it is considered that the total consequences of failure of bridges of different types are the same. Clearly the consequences of failure of one large bridge will be greater than that of one small bridge. A greater number of smaller bridges are constructed, however, and in the absence of empirical data it is assumed that for the sum of the consequences, the risks are broadly the same. The factor γ_{n2}, which is intended to take account of economic consequences, danger to communities etc, has thus been allowed for in the values of γ_{fL} and γ_m. Similarly, γ_{n1} which is intended to take account of the nature of the structure and its behaviour, has been allowed for on the basis of "normal" structures with some allowance for the risks of partial or complete collapse without warning. Designs carried out in accordance with this Code should therefore not normally require the factor γ_{n1}. In cases where elements may fail without due warning, e.g. in buckling, the design equations, i.e. the values of R^*, have appropriate safety values built into them.

This does not mean that modifications should not be made for special cases which may arise and the designer should always use his discretion by assessing the risks and consequences. A relatively small increase in expenditure on a few elements may greatly reduce the risk to the total structure, thus justifying the application of additional factors γ_{n1} and γ_{n2} of values greater than unity in the design of those elements only.

6.6 Verification of Structural Adequacy

For a satisfactory design, the following relation should be satisfied:

$$R^* \geq S^*$$

i.e. function $\frac{(f_k)}{(Y_m)} \geq Y_{f3}(\text{effects of } Y_{fl}.F_k)$

or, where appropriate,

function $\frac{(f_k)}{(Y_m)} \geq Y_{f3}(\text{effects of } Y_{f3}.Y_{fl}.F_k)$

The bridge and all its elements and supports should be checked for equilibrium under the appropriate factored actions and where necessary the implications of fatigue should be investigated.

Consideration should be given to the aerodynamic stability of bridges susceptible to such effects. Vehicle-induced vibration will very rarely be a consideration, but the possibility of resonance due to the effects of moving traffic may require investigation in very special cases.

6.7 Design Life

A design life of 100 years has been assumed throughout this Code (unless otherwise stated). The design life is that length of time for which the structure will continue to be serviceable with adequate and regular inspection and maintenance. The assumption of a design life does not necessarily mean that the structure will no longer be fit for its purpose at the end of that period.

It must be emphasized that bridges, like most modern structures, require regular inspection and, when necessary, repair under competent direction.

7. ANALYSIS

7.1 General

The following general principles should be adopted in calculating the action effects arising from any assumed pattern of applied actions and in verifying the safety of the structure. Global analysis of actions should be undertaken for each of the most severe conditions appropriate to the part under consideration and for all of the action combinations prescribed in Part 2. The methods of analysis used to assess compliance with the requirements of the various limit states should be based on as accurate a representation of the behaviour of the structure as is practicable. The methods used and the degree of sophistication or refinement thereof will thus depend on the nature, purpose and configuration of the structure and the nature of the actions to which it is subjected.

7.2 Methods of Analysis

7.2.1 Categories of Structure

Attributes which may dictate intricate or elaborate methods of analysis usually relate to the importance or size of the bridge and the complexity of its structural behaviour. This Code does not explicitly indicate categories of structure and which methods of analysis (in the sense of the degree of accuracy of the mathematical simulation of behaviour) should be used for particular structures. The onus rests on the designers to ensure that they have an adequate understanding of the subject matter in order to interpret correctly the clauses of this Code so as to ensure a safe and economical design.

For structures, parts of structures or members, the nature, purpose and configuration of which are such that they require less intricate or elaborate analyses, it will be permissible to simplify some of the requirements of this Code, provided the safety of the structure is not impaired.

7.2.2 Categories of Action

In the case of certain actions or effects, there will be more than one category of method that can be applied in the analysis of the effects of traffic loads, wind and earthquake forces. These can be categorized conveniently as follows:

- Actions which do not induce a significant dynamic structural response.
- Actions which induce a significant dynamic structural response, but do not play a major role in any critical load combination.
- Actions which induce a significant dynamic structural response and which play a major role in a critical load combination.

Recommendations as to the appropriate methods to be applied are made in Part 2.

7.2.3 Ultimate Limit State

The action effects under the most adverse conditions described for the application of actions appropriate to the ultimate limit state should be calculated by a method satisfying equilibrium requirements, all action effects being shown to be in equilibrium with the applied actions. Elastic methods are acceptable as lower-bound collapse solutions; they will also lead to solutions less likely to violate serviceability criteria. Non-linear plastic or yield-line methods may be adopted when appropriate to the type of structural material and the response of the structure to actions and imposed deformations. Plastic methods, or other procedures for permitting redistribution of moments and shears, may only be used when:

- The form of construction and the materials have an adequate plateau of resistance under the appropriate ultimate conditions and are not prone to deterioration of strength due to shake-down under repeated loading.
- The development of bending plasticity does not cause an indeterminate deterioration in shear or torsional resistance, or in axial strength when relevant.
- The supports or supporting structures are capable of withstanding reactions calculated by elastic methods.
- Changes in geometry due to deflections will not influence the load effects or are fully taken into account.

It may be assumed that the first two conditions are met by each of the appropriate methods presented in the design sections of this Code.

In the analysis for the ultimate limit state, the calculation of the stiffness of the structure may be based on the nominal dimensions of the cross-sections and on the elastic moduli specified in this Code. Alternatively, the stiffness may be modified to take account of effects such as shear lag, out-of-plane deformation of plating, cracking of concrete and slip of shear connectors.

Whichever alternative is selected, it should be used consistently. In allowing for the effects of shear lag, the width of any concrete or steel deck acting compositely with a beam may be taken as the effective width determined in accordance with the appropriate clauses in this Code or such other approved codes as may be applicable.

The effective spans should be assumed to be as defined in Section 7.5.

7.2.4 Serviceability Limit State

Action effects under each of the prescribed design actions appropriate to the serviceability limit state should, where relevant, be calculated by elastic methods. Linear methods may be used when these are based on the section properties assumed in Section 7.2.3, provided that changes in geometry do not significantly influence the action effects. Non-linear methods may be adopted with appropriate allowances for loss of stiffness due to cracking, creep or other predictable forms of deformation of the structure and should be used where geometric changes significantly modify the action effects. The method used should satisfy equilibrium requirements (see Section 7.2.3) and compatibility of deformations. Due allowance should be made in both determinate and indeterminate forms of construction for any erection procedures which affect the distribution of reactions and stresses. Effective spans should be in accordance with the definitions of Section 7.5.

Where only a part of a bridge is to be analysed, the boundaries to that part should either be so idealized as to represent accurately the stiffness of the bounding parts or supports, or be sufficiently remote from the region under consideration so that errors in simulation have no significant influence on the solution. The bounding parts should be designed to carry the boundary reactions calculated from the analysis.

7.2.5 Fatigue

Global analysis of the structure for the assessment of fatigue life should, where relevant, employ linear elastic methods based on section properties without reduction of stiffness and on the short-term moduli for concrete.

7.2.6 Deflections

Analysis of the structure for deflection may employ linear elastic methods. Deflections due to structural self-weight should, however, allow for the method and sequence of construction. Account should be taken of progressive changes in stiffness due to creep and shrinkage effects in concrete during construction and of these effects after completion.

Calculations of deflections due to finishes should be based on the long-term characteristics of concrete. Calculations of deflections due to live loads should be based on the short-term characteristics of concrete and on the most unfavourable distribution of bending moments using any of the methods of analysis appropriate to the serviceability limit state (see Section 7.2.4). Where appropriate, allowance should be made for shear flexibility and/or axial deformation.

7.3 Analysis of Resistance

Methods used to calculate the ultimate resistance in axial force, shear, bending or torsion, should provide estimates of strength having a probability of at least 95 per cent of being achieved when the material characteristics are accurately known. "Lower bound" methods are therefore essential to ensure that the required reliability is obtained. The methods given in this Code may be deemed to satisfy this requirement. Other methods should be verified, either by calibration against methods in this standard or by testing in accordance with Section 7.6.

7.4 Analysis of Stresses

In the calculation of stresses for serviceability or fatigue assessment from the actions analysed in accordance with Section 7.2.4, the following should be included unless otherwise stated in the design sections of this Code:

- Stresses due to axial forces and global bending moments, both longitudinal and transverse, including the influence of shear lag (the influence of shear lag may be assumed to have been allowed for in the calculation of the properties of the cross-sections for stress analysis when using appropriate effective widths.
- Shear stresses, including those due to torsion.
- Warping stresses due to torsion and distortion of box members.
- Transverse stresses due to distortion of box members.
- Stresses due to creep, relaxation and shrinkage.
- Stresses due to membrane forces.
- Stresses occurring in the vicinity of major stress concentrations which are due to the local nature of any appropriate pattern of loading or due to structural discontinuities, particularly near supports.
- Stresses due to bending moments in members caused by deflections and changes in geometry, e.g. secondary stresses in trusses due to deformation.

The mathematical idealization of the structure should reflect the nature of its judged response. The boundaries assumed in such an idealization should either simulate accurately the stiffness of adjacent parts or be sufficiently remote from the part under consideration for the stresses in it to be insensitive to the boundary assumptions.

7.5 Effective Spans

In the determination of actions and action effects, the effective spans of beams and slabs should be assumed to be as follows:

- Simply supported members. The smaller of either:
  - the distance between the centres of bearings or other supports; or
  - the clear distance between supports plus the effective depth.
- Members framing into supporting members. The distance between the shear centres of the supporting members.
- Continuous members. The distance between centres of supports, except where, in the case of beams on wide columns, the effect of column width is included in the analysis.
- Cantilevers. The distance from the face of the support plus half its effective depth, except where it is an extension of a continuous beam when the length to the centre of the support should be used.

7.6 Model Analysis and Testing

7.6.1 Global Analysis

Model analysis and testing may be used either to define the action effects in a structure or to verify a proposed theoretical analysis for a structure. The models used should be capable of simulating the response of the structure appropriately and the results should be interpreted by engineers who have appropriate experience.

The reliability of the test results depends *inter alia* upon the accuracy or knowledge of:

a) material properties (model and prototype);
b) methods of measurement;
c) methods used to derive action effects from measurements;
d) actions and reactions.

In interpreting results, the assessed action effects to be used in the design should exceed those derived from the test data by a margin dependent upon:

e) the number of tests;
f) the method of testing;
g) an assessment of (a), (b) and (c) above.

In all cases the interpreted results should provide equilibrium.

7.6.2 Assessment of Local Resistance

Methods, other than those specified in this Code, for calculating the ultimate resistance of cross-sections should be verified by tests on a range of representative components, or models thereof, sufficient to ensure that the influence of each parameter of the physical behaviour up to collapse is demonstrated.. The reliability of such verification will depend upon (a), (d), (e) and (f) of Section 7.6.1. In such tests the ratio between the strength predicted by the chosen method (using measured properties and dimensions) and the measured value should be obtained for a number of samples and the mean and standard deviations of such ratios determined.

The partial factors for strength prescribed in this Code should then be adjusted for application with the verified method, to take into account its mean accuracy and variability.

Where prototype testing is adopted as a basis for proving the resistance of a component, the test loading should adequately reproduce the range of stress combinations to be sustained in service. A sufficient number of prototypes should be tested to enable a mean value and standard deviation of resistance to be calculated for each critical stress condition, the design value then being taken at 1.5 standard deviations below the mean.

The material strengths to be specified for construction should have mean values and coefficients of variation compatible with those in the prototypes. Tolerances and dimensions should be similarly prescribed so that constructions are compatible with the prototypes.

No results of prototype testing may be used to justify any reduction in partial safety factors. Similar tests on components may be used to verify or determine the serviceability limit loading capacity.

7.7 Checking Procedures

Errors can be mainly classified as:

- Random errors
- Systematic errors
- Gross errors.

The partial safety factors specified in this Code allow for random errors in the estimation of the magnitudes of actions and strengths of materials.

Systematic errors and especially gross errors are not necessarily covered by such factors and it is consequently imperative that these be eliminated by proper checking procedures. It is human to err and systematic methods for the checking of all stages of the design process are therefore necessary. Such checks should take the form of:

- Checking, by the members of the design team, of the validity of all assumptions, the applicability of the methods used and the reliability of data obtained, as well as repetitive checking, as the work progresses, by every designer or analyst of his own work.
- Systematic and independent checking of the work of each engineer by other competent engineers in the same design team. A comprehensive check of the whole of the work

(not necessarily of all details) by one engineer is essential in order to ensure completeness.

- In the case of major or important works, independent checking of such work by another competent team of engineers working separately from the original team. Such a team need not necessarily belong to a different organisation, but independence of work must be ensured as far as is practicable.

Particular care should be taken to guard against conceptual errors or those resulting from the uncritical acceptance of information obtained from other sources. Gross numerical errors and errors made in computation (by hand or computer) should be guarded against by doing systematic checks of the orders of magnitude of results by entirely separate and independent computations (even if they are approximate).

It is realised that checking can be a costly process if done in a comprehensive manner, but the onus rests on the designer to ensure that the probability of gross errors is reduced sufficiently so as to ensure that the required degree of reliability of the structure is not significantly reduced.

8. SOIL-STRUCTURE INTERACTION

8.1 General

The soil or rock on which the structure is founded, or which is retained by parts of it, should be considered to be an extension of the structure in order to ensure that soil-structure interaction is taken into account. The properties of the soil should be established to a degree of accuracy that is sufficient in terms of the design assumptions to achieve the required reliability. Upper and lower limits of estimated settlement, swelling or heaving should be used to establish extreme effects on the structure.

This Code does not at this stage include sections on soil-structure interaction; in addition to what is specified in Section 8.2, reference should in the interim be made to approved codes, specifications, textbooks, or technical papers on proven research dealing with this subject.

8.2 Verification of the Safety of Foundations including Piles

Foundations may in the interim, until the relevant sections are published, be assessed in accordance with the principles set out in British Code of Practice, CP2004[13] and the SABS Codes of Practice for the Design of Foundations for Buildings. SABS.0161 - 1980[9], and for Pile Foundations, SABS.088 - 1972[14], or another approved code.

The above codes have not been drafted on the basis of Limit State Design, but it will be appropriate to adopt the nominal loads specified in Part 2 with γ_{fL} and γ_{f3} = 1 as design loads for the purpose of verifying foundations in accordance with Limit State Design. In such a case the loads on foundations should be derived from the methods of analysis appropriate to the serviceability limit state. The factors of safety recommended in the appropriate code shall be complied with.

Foundations designed on embankments stabilized by rock or soil anchors, reinforced earth embankments and other techniques not covered by CP2004 or SABS.0161, shall be designed in accordance with approved procedures.

8.3 Design of Structural Foundations

Structural foundations, e.g. spread footings, pile caps or foundation beams, are designed to transmit the applied actions on the structure to the ground, or to piles or caissons depending upon the method of transfer. In each case it is necessary to know the reaction to the applied actions. The reactions, which will be bearing pressures or pile loads, should be calculated for the design actions relevant to the limit state under consideration and the structural foundations assessed for these design reactions.

9. ERECTION, CONTROL AND ACCEPTANCE

9.1 Erection

The adoption of limit state methods with partial safety factors emphasizes the necessity to assess the loads and location of erection plant and equipment accurately. The amount of the partial factor applied to these loads should be appraised for each case on its merits, making due allowance for the accuracy or otherwise of the evaluation of temporary loads. Additional material incorporated for erection purposes should also be accurately assessed and, if it is to be retained in the completed structure, its effect on this should be taken into account.

Where temporary flexible supports or suspension cables are used to suspend parts of the permanent structure during erection or construction, any interaction with the permanent structure or parts thereof, including temperature effects, should be accurately assessed, as should the effects on the permanent structure of removing such supports. The partial factors to be used in the design of all temporary works should be appraised for each case on its merits, making allowance for the risks involved and taking due account of the possible implications for the permanent structure.

9.2 Control and Acceptance of Materials and Products

The design of structures includes a specification of the requirements for the properties of the structural materials or products, for the manufacturing methods and for construction. Frequently, reference may be made to relevant national and international standards.

The properties of materials and products are subject to random variability which cannot be forecast precisely at the time of the design. Furthermore, the information on which the production process must be judged contains various uncertainties resulting from the variabilities inherent in sampling and testing, Considering these uncertainties, it is necessary to establish deciding rules which take adequate account of the requirements for the safety of structures and the cost of control.

The quality of the materials and products should be specified in terms of statistically defined values (characteristic values) in the probability distributions of one, or alternatively, several properties (e.g. strength, deformation, geometrical deviations) of the material or product. The sampling procedure, the shape and size of the samples, their treatment and the methods of testing should be specified.

The properties must be assessed by tests of attributes or measurements by sampling, either during the manufacture of the materials or products, or on site.

For the strength of materials the characteristic strength f_k, may generally be taken as the five per cent fractile.

The materials or products may be classified into different classes of quality. The number of classes should be kept as small as possible, but should allow adequate freedom in design. The upper limit to the number should be controlled by limitations of statistical methods. Materials or products made and tested in large quantities (e.g. concrete or steel) may be classified by means of their characteristic values (e.g. strength classes for concrete and steel, stress grades for timber). Somewhat different criteria should be defined for products of large size of which only a small number of specimens can be tested.

9.3 Types of Control

In principle there are three types of control:

- Initial tests: Tests for estimating the properties of the material or product.
- Production control: Tests to assess the consistency and control over the production process.
- Quality control (compliance control): Tests to decide whether the material or product satisfies the specified requirements.

These types of control should be carried out independently of each other; the responsibilities should be clearly defined.

The tests may be carried out according to demand by the manufacturers (internal control) but the result of internal control may be judged by an independent organization (external control).

9.3.1 Initial Tests

Initial tests serve to estimate the order of magnitude of the properties attainable with the chosen basic materials and, if appropriate, with the available production equipment, personnel and the relevant conditions of production. In special cases, e.g. for the testing of natural elements, the initial test is also used for classification purposes.

9.3.2 Production Control

Production control serves to judge the long-term consistency of the production process, as well as the consistency of production planning with respect to fulfilling the specified requirements.

9.3.3 Quality Control

Quality controls serve to decide whether the materials or products satisfy the specified requirements. In the quality control process a prescribed number of samples have to be taken independently and at random; these have to be treated and tested in a prescribed manner. A prescribed number of successive test results have to be set up in one or more acceptance functions. The decision on acceptance or rejection is made with the aid of a previously agreed decision rule, which contains this acceptance function as well as the appropriate acceptance limits.

In the case of testing by attributes, the permissible number (acceptance number) of defectives has to be determined on the basis of the total number of values to be tested.

In the case of negative decisions, the amount of material involved in the decision may be either refused, reclassified or submitted for further tests (re-tests). In this case, it is necessary that the characteristic tested be either the same as, or at least closely correlated with, the characteristic originally tested. After re-testing the amount of material under consideration can be finally accepted or rejected.

10. REFERENCES

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6 **Rationalization of safety and serviceability factors in structural codes**. 1970. London, UK: Construction Industry Research And Information Association CIRIA). (Report 63).

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9 **Code of practice for the design of foundations for buildings**. 1980. Pretoria, SA: South African Bureau of Standards. (SABS 0161).

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12 **Code of Practice for the design of concrete bridges in South Africa**. 1983. Pretoria, SA: Department of Transport. (Technical Methods for Highways TMH7 Part 3).

13 **Code of practice for foundations**. 1972. British Standards Institution. (CP2004).

14 **Code of practice for pile foundations**. 1972. Pretoria, SA: South African Bureau of Standards. (SABS 088).

PART 2: SPECIFICATION FOR LOADS

1. INTRODUCTION

1.1 Scope of Part 2

This Part of the Code specifies nominal actions and their application, together with the partial factors, γ_{fL}, to be used in deriving design actions. The actions and combinations of actions specified are for highway and pedestrian/cycle track bridges on the trunk road network in the SADC region. Where National, Provincial or Local Government Authorities specify different actions, modifications may be necessary. Standard railway loading is beyond the scope of this Code. In the case of railway bridges, reference should be made to the relevant railway authority.

It is advisable to read Part 1 before studying this Part. Consideration of the primary effects of the force of pre-stressing is set out in TMH7 Part 3[1] and such effects are not dealt with in this Part, except for the secondary effects as dealt with in Section 4.2 and except in so far as the force of pre-stressing may form an erection force component.

1.2 Symbols

The symbols used in this Part are as follows:

A	=	horizontal ground acceleration due to an earthquake
A_b	=	actual plan area of an elastomer bearing
A_1	=	solid area in normal projected elevation
A_2	=	solid area in projected elevation normal to the longitudinal wind direction
A_3	=	area in plan
A_4	=	projected pier area subjected to flood forces
B	=	width of gorge
C_D	=	drag coefficient
C_L	=	lift coefficient
E	=	Young's modulus
E_k	=	kinetic energy
F	=	force due to flood; pulsating point load
F_a	=	action due to differential settlement
F_{aa}	=	force due to accidental skidding
F_b	=	impact force acting on balustrade
F_{br}	=	frictional bearing restraint
F_L	=	longitudinal braking or traction force
F_c	=	centrifugal force due to traffic
F_{cc}	=	action due to creep
F_{cs}	=	action due to shrinkage
F_e	=	fetch
F_{eq}	=	horizontal force on structure due to an earthquake (Method A)
F_f	=	force due to flood action
F_{ij}	=	generalized set of equivalent static forces
F_{im}	=	impact force on bridge supports due to vehicle collision
F_k	=	force on kerb
F_{kOT}	=	nominal action causing overturning effect

F_{kR}	=	nominal action causing restoring effect
F_{pp}	=	action due to parasitic prestress
$F_{p\varepsilon}$	=	action due to prestrain
Ftr	=	action due to temperature range
F_{tg}	=	action due to temperature gradient
F_x	=	equivalent horizontal static earthquake force (Methods B and C)
G	=	shear modulus of elasticity
H_e	=	height of escarpment or depth of gorge
H_p	=	height of pier
H_s	=	mean height of superstructure
I	=	importance factor of a structure subject to earthquakes or second moment of area
K	=	constant stiffness (Section 3.9.2.2); configuration factor (Appendix C.2.5)
K_1	=	coefficient (Section 3.8.2.2)
L	=	effective loaded length; length between supports of a balustrade
L_i	=	see Appendix A.2.1.
L_p	=	see Appendix A.2.1
L_s	=	equivalent or effective span length
L_w	=	wind-loaded length of a structure
L_1L_2	=	span lengths
MM	=	modified Mercalli Scale
NA	=	normal traffic loading
NABC	=	normal plus abnormal plus super traffic loading
NB	=	abnormal traffic loading
NB36	=	36 units of NB loading
NB24	=	24 units of NB loading
NC	=	super loading
P	=	total number of separate loaded parts on all the lanes
Q_a	=	average nominal distributed NA lane loading for a loaded length $\sum_{i=1}^{p} L_i$
Q_{ap}	=	nominal distributed NA loading on pth part of length Lp
Q_b	=	equivalent type NB36 loading on culverts
Q_i	=	see Appendix A.2.1
Q_v	=	vertical point load
R^*	=	design resistance
S	=	seismic response factor
S_a	=	maximum structural response acceleration
S_{ai}	=	maximum structural response acceleration for mode i
SNABC	=	standard traffic loading
S_{OT}	=	overturning moment due to design actions
S_R	=	restoring moment due to design actions
S_1	=	funnelling factor
S_2	=	gust factor
S_3	=	exposure factor
T	=	fundamental period of vibration; time; temperature difference
T_i	=	period of mode i
U_{max}	=	strain energy; work done
V_t	=	constant speed
W_i	=	weight of mass M_i
W_L	=	equivalent static nominal longitudinal wind force
W_{LL}	=	equivalent static nominal longitudinal wind force on traffic

W_{LS}	=	equivalent static nominal longitudinal wind force on superstructure
W_t	=	equivalent static nominal transverse wind force
W_v	=	equivalent static nominal vertical wind force
X	=	distance between inner axles of Type NB vehicle
Y_s	=	static deflection
a	=	length; vertical acceleration
b	=	width
b_c	=	overall width of culvert
b_h	=	load width
b_k	=	width of kerb
b_r	=	width of carriageway
b_1	=	distance from centre line of carriageway to the retaining wall
c	=	separating or spacing distance
d	=	depth used in deriving wind forces
d_1	=	depth of deck
d_2	=	depth of deck plus solid parapet
d_3	=	depth of deck plus live load
d_L	=	depth of live load
e	=	base of natural logarithms; eccentricity
f	=	foundation factor (Section 3.10.3.1)
f_{ep}	=	earth pressure
f_w	=	water pressure
g	=	acceleration due to gravity
g_c	=	dead load of concrete
g_{di}	=	dead load of the portion "i" of a structure
g_n	=	vertical earth-fill loading on culverts (Section 2.3.3.2)
g_s	=	dead load of steel
g_{sd}	=	superimposed dead load
g_{sdj}	=	superimposed dead load of the portion "j" of the structure
h	=	height of earth-fill cover to culverts; effective height above ground level of part of a structure subjected to wind (Figure 3.7); depth of slabs or beams in temperature calculations (Figure 4.3)
h_i	=	height of portion of the structure of mass mi, above the base or pile cap
h_k	=	height of kerb
h_x	=	height of portion of the structure of mass m_x above the base or pile cap
k	=	correction factor to allow for increased effects of partial loading of base of influence line or surface
k_d	=	coefficient depending on the density of the air (Section 3.8.2.1)
k_f	=	coefficient depending on the foundation of the structure
k_h	=	factor depending on slope of fill to determine horizontal component of earth pressure (Section 2.4.2)
k_o	=	constant to determine fundamental natural frequency
k_v	=	factor depending on slope of fill to determine vertical component of earth pressure (Section 2.4.2)
l_n	=	length of part n of structure
l_h	=	load length
m	=	mass
m_i	=	mass of portion of structure at height hi
m_j	=	lumped mass free to move in direction of the degree of freedom j
m_x	=	mass of portion of structure at heigh h_x

n	=	loading sequence number of a lane; number of notional lanes; number of beams or box girders; number of railings in a balustrade; number of degrees of freedom of a structure
n_o	=	fundamental natural frequency
p	=	loading sequence number of a part of any or all the lanes
q	=	dynamic pressure head
q_{a1}, q_{a2}	=	uniformly distributed pressure on culverts due to NA loading (Section 2.6.6.2)
q_c	=	uniformly distributed pressure on culvert due to NC loading
q_p	=	distributed force acting on balustrade
r	=	radius of curvature of a lane; internal radius of bend of reinforcing bar
t	=	dimension of pier measured in direction of wind
t_1	=	total thickness of elastomer in shear
v	=	design speed of roadway; mean hourly wind speed; velocity of flow
v_c	=	maximum wind gust speed
W_1, W_2	=	unit design wind force without and with live load respectively (Section 3.8.2.2)
x	=	displacement
$\ddot{x}$	=	horizontal ground acceleration
y	=	horizontal length of escarpment
α	=	exponent (Section 2.3.3.2)
β_s	=	angle of surcharge
γ_{fL}	=	γ_{f1} x γ_{f2} = partial load factors (Section 6.2 of Part 1)
γ_{f3}	=	partial effects factor (Section 6.3 of Part 1)
γ_i	=	modal participation factor
γ_m	=	γ_{m1} x γ_{m2} = partial materials factors (Section 6.4 of Part 1)
δ	=	Logarithmic decrement of decay of vibration due to damping
δ_i	=	nominal range of movement of a bearing
η	=	shielding factor
λ	=	damping ratio
μ	=	ductility factor (Section 3.10.3.1); coefficient of friction (Section 4.5.7.3)
ξ	=	numerical factor (Section 3.10.3.1)
ρ	=	effective density of fill material
Φ_{ij}	=	normalized mode component of the its mode in the direction of jth degree of freedom
Φ_2	=	Swiss impact factor
ψ	=	dynamic response factor
ω	=	natural radial frequency

1.3 Code Definitions

For the purposes of this Code, the following definitions and classifications apply:

1.3.1 Actions

Actions are defined in Section 3 of Part 1.

1.3.2 Action Effects

Action effects are the stress resultants in the structure arising from its response to the actions.

1.3.3 Principal Actions

Principal actions are those actions that are generated by gravity and which can conveniently be subdivided into permanent and long-term loads, short-term and transient loads.

- Permanent and long-term loads include:
  - *dead load* which is the weight of the materials and parts of the structure that are structural elements, but excluding superimposed materials such as road surfacing, parapets, mains, ducts, miscellaneous furniture, etc;
  - *superimposed dead load* which is the weight of all materials forming loads on the structure that are not structural elements;
  - *earth pressure* consisting of the forces due to retained fill or soil and/lor long-term surcharge loads;
  - *water pressure* of retained or excluded water in cases where the water level is constant in the long term.
- Short-term loads include the weight of plant and equipment used during erection, construction or maintenance.
- Transient vertical loads (primary live loads considered as static loads) include the weight (increased by a factor to allow for the static equivalent of vertical impact forces) of vehicle, cycle and pedestrian traffic.
- Variable loads include water pressure in cases of varying water level.

1.3.4 Supplementary Actions

Supplementary actions are those actions generated by acceleration or deceleration of mass, such as changes in speed or direction of vehicle traffic, e.g. lurching, nosing, centrifugal, longitudinal, skidding and collision loads, or by resistance to wind or flood action and the action of earthquakes and include:

- Secondary forces due to live loads:
  - Centrifugal loads
  - Longitudinal braking and traction loads
  - Accidental skidding loads
  - Vehicle collision with bridge parapets
  - Vehicle collision with bridge supports
- Actions due to natural causes:
  - Wind action
  - Flood action
  - Earthquake action.

1.3.5 Restraint Actions

Restraint actions are those actions generated in constrained structural members by dimensional changes during or after construction. These can conveniently be subdivided into long-term and short-term actions.

- Long-term restraint actions include:
  - Creep and shrinkage effects
  - Secondary (parasitic) prestress and prestrain effects
  - Differential settlement.
- Short-term restraint actions include:
  - Temperature range, i.e. expansion or contraction effects
  - Temperature gradient effects.

1.3.6 Adverse and Relieving Areas and Effects

Where an element or structure has an influence surface (or line) consisting of both positive and negative parts, in the consideration of loading effects which are positive, the positive areas of the influence surface (line) are referred to as adverse areas and their effects as adverse effects; negative areas of the influence surface (line) are referred to as relieving areas and their effects as relieving effects. Conversely, in the consideration of loading effects which are negative, the negative areas of the influence surface (line) are referred to as adverse areas and their effects as adverse effects, and the positive areas of the influence surface (line) are referred to as relieving areas and their effects as relieving effects.

1.3.7 Total Effects

Total effects are determined by the algebraic sum of the adverse and relieving effects.

Note: Where the elements in a positive area of influence surface (line) are being considered, the total effects may be negative, in which case the equivalent positive value will be the least negative effect; where negative effects are being considered, the total effects may be positive, in which case the equivalent negative value will be the least positive effect. In either case the maximum negative or positive total effect should also be considered.

1.3.8 Dispersal

Dispersal is the spread of load through surfacing, fill, structural concrete, etc.

Note: Except for the case of culverts, no specific simple rules are given in this Code for the dispersal of load as it depends on many factors. The designer shall use his own discretion in the allowance of dispersal which shall be determined as scientifically as is practicable.

1.3.9 Highway carriageway and lanes

- Carriageway: For the purposes of this Code a carriageway is defined as that part of the running surface which includes all traffic lanes, hard shoulders, hard strips and marker strips. The carriageway width is the width between raised kerbs. In the absence of raised kerbs, it is the width between guardrails or safety fences, less the amount of set-back required for these fences, being not less than 0.6 m or more than 1.0 m from the traffic face of each fence.
- Traffic lanes: The lanes that are marked on the running surface of the bridge and are normally used by traffic.
- Notional lanes: The notional parts of the carriageway used solely for the purpose of applying the specified vehicle traffic loading in terms of the rules described in Section 2.6.2.

1.3.10 "Bridge" and "Culvert"

Whereas it may be convenient for other purposes to differentiate between bridges and culverts in terms of specific dimensions, this will not be done for the purposes of this Code and it will be left to the discretion of the authority concerned. In this Code culverts are referred to separately from bridges because of their particular soil-structure interaction.

1.3.11 Bridge components

- Superstructure: In a bridge, that part of the structure which is supported by the piers, towers and abutments. Arches and spandrel columns are considered to be part of the superstructure.
- Substructure: In a bridge, the piers, towers, abutments and wing walls that support the superstructure.
- Foundation: That part of the substructure in direct contact with, and transmitting action to the ground.

1.4 General Definitions

1.4.1 Nominal Actions

In the absence of statistical data, the nominal values of actions that are considered to approximate to a 100-year return period are given in the appropriate clauses, unless stated otherwise in the text.

1.4.2 Design Actions

Nominal actions shall be multiplied by the appropriate value of γ_{fL} to derive the design action to be used in the calculation of moments, shears, total loads and other effects for each of the limit states under consideration. Values of γ_{fL} are given in each relevant clause and also in Table 5.1 in Section 5.

1.4.3 Additional Factor γ_{f3}

Moments, shears, total loads and other effects of the design actions are also to be multiplied by γ_{f3} (see Section 6.3 of Part 1). Values of γ_{f3} are given in TMH7 Part 3[1].

1.4.4 Fatigue Actions

Values for fatigue assessments are not given in this Code. When necessary, reference shall be made to Part 10 of BS5400[2] or another approved standard.

1.4.5 Deflection and Camber

For the purposes of calculating deflection and camber, the nominal actions shall be adopted (i.e. γ_{fL} shall be taken as unity).

1.4.6 Classification of Actions

The classification of actions is given in Section 1.3, as well as in Table 5.1 in Section 5.

1.4.7 Combinations of Actions

Combinations of actions are given in Section 5.

1.4.8 Application of Actions

Each element and structure shall be examined under the effects of actions that can co-exist in each combination.

- Selection to cause most adverse effect - design actions shall be selected and applied in such a way that the most adverse total effect is caused in the element or structure under consideration.
- Removal of superimposed dead load - consideration shall be given to the possibility that the removal of superimposed dead load from part of the structure may diminish its

relieving effect. In so doing, the adverse effects of transient actions on the elements of the structure being examined may be modified to the extent that the removal of the superimposed dead load justifies this.

- Transient action shall not be considered to act on relieving areas except in the case of wind actions on traffic when the presence of light traffic is necessary to generate the wind action (see Section 3.8.7 in this part).
- Wind on relieving areas - design actions due to wind on relieving areas shall be modified in accordance with the relevant requirements of Section 3.8.3. in this part.

1.4.9 Stability

Stability of the structure and its parts shall be considered for the ultimate limit state. For example in considering overturning, the least restoring moment due to nominal actions plus the design resistances of boundary members or parts shall be greater than the greatest overturning moment due to design actions, ie

$$S_R^{min} + R^* \succ S_{OT}^{max}$$

where

S_R^{min} = the least restoring moment due to characteristic (or nominal) actions

$= \gamma'_{f3}$ (effects of $\gamma'_{fL} \cdot F_{kR}$)

where γ'_{f3} and γ'_{fL} are reduced partial factors which may be zero but not less than 1.0 for dead loads (see also Section 2.2.2 and the relevant clauses in the text on the relieving effects of various actions).

R^* = the design resistances of boundary materials, members or parts that resist the overturning moments, i.e. the resistances based on the characteristic (nominal) strength of the material divided by γ_m.

= function $\dfrac{(f_k)}{(Y_m)}$

S_{max}^{OT} = γ_{f3} (effects of γ_{fL} F_{kOT})

where

F_{kR} and F_{kOT} are the nominal actions causing restoring and overturning effects respectively; and all partial factors are for the ultimate limit state.

The requirements specified in Section 1.4.8 relating to the possible removal of superimposed dead load shall also be taken into account in considering stability (e.g. overturning).

2. PRINCIPAL ACTIONS

2.1 General

Principal actions are defined in Section 1.3.3.

2.2 Dead Loads

2.2.1 Nominal Dead Loads

The nominal dead load initially assumed shall be accurately checked with the actual weights to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies. For the purpose of initial calculations, except where other values for the constituent materials have been established in specific localities, the following values of densities, or such other applicable values determined by interpolation or extrapolation for various percentages of reinforcement and prestressing, may be assumed for nominal dead loading of concrete:

- 2 400 kg/m³ for plain concrete
- 2 600 kg/m³ for reinforced or prestressed concrete with 3½ per cent reinforcement including prestressing (by volume)

Density of steel:

- 7 850 kg/m³

Other densities shall be as set out in Appendix "B" of the SABS *Code of practice for the general procedures and loadings to be adopted for the design of buildings* - SABS 0160[3].

2.2.2 Design Dead Loads

The factor γ_{fL} to be applied to all parts of the dead load, irrespective of whether these parts have adverse or relieving effects, shall be taken for the various combinations of actions (see Section 5.2) as follows (Table 2.1), except as specified in Sections 2.2.2.1 to 2.2.2.3 inclusive.

Table 2.1: γ_{fL} for design dead loads

γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
Steel Concrete	1.1 1.2	1.0 1.05	1.1 1.2	1.0 1.0	1.0 1.0	1.0 1.0

These values for γ_{fL} assume that the nominal dead loads have been accurately assessed and that the densities of materials have been confirmed.

2.2.2.1 Approximations in Assessment of Dead Load

Any deviations from accurate assessment of nominal dead load for preliminary design or for other purposes should be accompanied by an appropriate and adequate increment in the value of γ_{fL}. Values of 1.15 for steel and 1.25 for concrete for the ultimate limit state will usually suffice to allow for the minor approximations normally made. It is not possible to specify the allowances required to be set against various assumptions and approximations, and it is the responsibility of the engineer to ensure that the absolute values specified in Section 2.2.2 are met in the completed structure.

2.2.2.2 Relieving Effects of Dead Load

Where the determination of any action in a member that is an important part of the structure is dependent on the resultant effect of dead loads at different positions, with effects of equivalent order of magnitude but acting in opposition, and where the magnitude of such resultant effect may consequently be highly dependent on the accuracy of such calculated opposing dead loads, reduced values of the factor γ_{fL} shall be applied to those parts of the dead load that have relieving effects. The reduced factors which shall depend on the risks involved shall be determined by the engineer for any particular case but need not be less than one. In conjunction therewith, the unreduced values of the factor γ_{fL} given in Section 2.2.2 shall be applied to those parts of the dead load that have adverse effects. This reduction of γ_{fL} for relieving effects relative to the values for adverse effects need not be applied generally (see Section 2.2.2.3), but may be particularly important in the case of structures that are in low stability equilibrium during or after construction. An example would be a balanced cantilever with a very high dead load to live load ratio that is stabilized by a tie or prop force of relatively small magnitude.

2.2.2.3 Alternative Load Factor

Where the structure or element under consideration is such that the application of γ_{fL} as specified in Section 2.2.2 for the ultimate limit state causes a less severe total effect (see definition of adverse and relieving areas and effects in Section 1.3) than would be the case if γ_{fL}, applied to all parts of the dead load, had been taken as 1.0, values of 1.0 shall be adopted.

2.3 Superimposed Dead Loads

2.3.1 Nominal Superimposed Dead Load

The nominal superimposed dead load initially assumed shall in all cases be accurately checked against the actual weights to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies. Where the superimposed dead load comprises filling, e.g. on spandrel-filled arches and culverts, the possibility of the fill becoming saturated shall be considered.

For the purpose of initial calculations, the following values may be assumed for nominal superimposed dead loading:

- The density of the bitumen premix carpet on the roadway of the bridge shall be taken as 2 100 kg/m³.
- For the purpose of calculating the vertical loading caused by compacted earth filling, the density of the earth shall be taken as 2 000 kg/m³.
- Densities of concrete, steel and other materials shall be as in Section 2.2.1.

2.3.2 Design Superimposed Dead Load

The factor γ_{fL} to be applied to all parts of the superimposed dead load irrespective of whether these parts have an adverse or relieving effect, shall be taken for the various combinations of actions (see Section 5.2) as follows (Table 2.2), except for the requirements of Section 1.4.8. and Sections 2.3.2.1 to 2.3.2.3.

Table 2.2: γ_{fL} for design superimposed dead load

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
Adverse effects	1.2	1.05	1.2	1.0	1.0	1.0

2.3.2.1 Increase in Superimposed Dead Load

Where there is a possibility of the superimposed dead load being increased during the life of the bridge, such as by the placement of additional layers of road surfacing, the engineer shall either make allowance for such additional weight or shall increase γ_{fL} given in Section 2.3.2 or such other values as determined in terms of Section 2.3.2.1, shall be applied to those parts of the superimposed dead load that have adverse effects.

2.3.2.2 Relieving Effects of Superimposed Dead Load

Where the determination of any action in a member that is an important part of the structure is dependent on the resultant effect of superimposed dead loads at different positions with effects of equal order of magnitude but acting in opposition, and where the magnitude of such resultant effect may consequently be highly dependent on the accuracy of such calculated opposing superimposed dead loads, reduced values of the factor γ_{fL} shall be applied to those parts of the superimposed dead load that have relieving effects. The reduced factors which will depend on the risks involved shall be determined by the engineer for any particular case. In conjunction therewith the unreduced values of the factor γ_{fL} given in Section 2.3.2, or such other values as determined in terms of Section 2.3.2.1, shall be applied to those parts of the superimposed dead load that have adverse effects.

2.3.2.3 Alternative Load Factor

Where the structure or element under consideration is such that the application of γ_{fL} as specified in Section 2.3.2 for the ultimate limit state causes a less severe total effect (see Section 1.3.6) than would be the case if γ_{fL} applied to all parts of the superimposed dead load had been taken as 1.0, values of 1.0 shall be adopted.

2.3.3 Vertical Earth Loading on Culverts

2.3.3.1 General

With due recognition of the complexity of the problem of determining loading on culverts, but also of the need for simple procedures which can be used routinely, provision is made for a three-fold approach, viz:

(i) The application of simple design rules that can be applied to rigid and flexible culverts but that require the use of increased partial safety factors which allow for the approximate nature of the formulae used.

(ii) The application of more sophisticated design theories to rigid and flexible culverts that take into account the type of culvert, the properties of the undisturbed ground and the fill materials as well as the effects of the actual width of excavation, and the positive or negative projection. These theories also allow for the use of reduced partial safety factors. (In positive and negative projecting culverts, the tops of the structures are above and below undisturbed ground level respectively.)

(iii) The application of sophisticated design theories or the design techniques based on the phenomenological approach to flexible and special types of culvert that require more accurate assessments of soil-structure interaction.

This Code covers the first approach only which is an extension of the AASHO and CPA formulae. The designer shall use his discretion in deciding on the best applicable method for any particular case and is referred to publications on the subject.

2.3.3.2 Approximate Method as Described in (i) above

Extreme values of the effect of vertical loading due to earth embankments shall be calculated for any particular case by any two of the following four relevant formulae and the applicable value shall be determined by interpolation. Negative projecting culverts are not considered and require calculation by the methods described in (ii) or (iii) in Section 2.3.3.1.

(i) *Culvert in trench on unyielding foundation with no projection*:

$$g_1 \cong 10\rho h \times 10^{-3}$$

(ii) *Culvert untrenched on yielding foundation:*

$$g_2 \cong 10\rho h \times 10^{-3}$$

(iii) *Culvert untrenched on unyielding foundation for h* > 1.7b:

$$g_3 = \rho(18.83h - 8.54) \times 10^{-3}$$

(iv) Culvert untrenched on unyielding foundation for h $\leq$ 1.7b:

$$g_4 = 25.4\rho b(e^{\alpha} - 1) \times 10^{-3}$$

where

α = 0.38 h/b
g_n = unit vertical loading due to fill in kN/m² for cases n = 1 to 4
h = height of earth-fill cover in metres
b = overall width of trench or, if untrenched, overall width of culvert in metres
ρ = effective density of fill material in kg/m³
e = base of natural logarithms = 2.7183

Consideration shall be given to the effects of the possible removal in whole or in part of the embankment filling.

2.3.3.3 Design Values for Vertical Earth Loading on Culverts

γ_{fL} to be applied to all parts of the vertical earth loading, irrespective of whether these parts have an adverse or relieving effect, shall be taken for the various combinations of actions (see Section 5.2) as follows (Table 2.3), except for the requirements of Sections 2.1.8.2, 2.3.1 and 2.3.2.1.

Table 2.3: γ_{fL} for vertical earth loading on culverts

γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
Method (i) Methods (ii) and (iii)	1.5 1.4	1.3 1.2	1.5 1.4	1.1 1.0	1.1 1.0	1.1 1.0

2.4 Earth Pressure on Retaining Structures

2.4.1 General

Earth pressure, whether caused by filling material retained by abutments or other parts of the structure and the side walls of culverts, or whether induced in time on parts of the structure cast against residual soil, shall be determined by soil mechanics principles from the properties of the filling or soil material and shall be regarded as nominal pressures. The effects of traffic load surcharge (Section 2.6) shall be taken into consideration for loading combinations (1) and (2).

In addition to a consideration of the effects of such pressures, stability checks for the ultimate limit state shall be done for the whole of and such parts of the structure that are subjected to earth pressure in terms of the requirements of Section 8.1 of Part 1 and 1.4.9 of Part 2.

The nominal pressures initially assumed shall be accurately checked with the properties of the material to be used in construction or be determined by tests on samples or in situations and, where necessary, adjustments shall be made to reconcile any discrepancies.

Consideration shall be given to the possibility that the retained materials may become saturated or may be removed in whole or in part from either side of the fill-retaining parts of the structure.

Beneficial effects of earth pressure should only be used if the lateral movement required to realise the pressure can always be achieved.

Consideration shall be given to the possibility of higher horizontal pressures approaching or exceeding that of "soil at rest" being attained, due to compaction during construction or to traffic loading in retaining structures within which the soil is totally or partially contained in opposite horizontal directions, e.g. in a cellular abutment construction or between closely spaced counter forts.

2.4.2 Approximate Theory for Earth Pressures

For the following two classifications of fill material, the equivalent fluid pressure theory may be used as an approximation with the stated equivalent fluid pressures:

Type I - coarse-grained (sands) with a low silt or clay content such that the material essentially has the properties of a "granular" material; rock fill: 5.6 kN/m² per metre depth.

Type II - fine, silty sand; granular material with a conspicuous clay content: 7.8 kN/m² per metre depth.

Where the surface of the fill behind a wall is at a slope, the horizontal and vertical components of the earth pressure shall be taken as the equivalent fluid pressure multiplied by the factors k_h and k_v

where $k_h = 1 + (9 \times 10^{-10}\ \beta_s^6)$

where $k_v = (15 \times 10^{-3})\beta_s + (65 \times 10^{-11}\ \beta_s^6)$

where β_s = angle of surcharge in degrees but not exceeding 33°.

Where the possibility exists that the retained materials may become saturated, an additional horizontal pressure of 6 kPa per metre depth below the phreatic line shall be applied to the wall.

Where fill material of poorer type than Type II is to be used, e.g. clay, the equivalent fluid pressure theory shall not be used and the calculations shall be based on soil mechanics principles and the properties of the material as determined by tests.

The surcharge pressure of highway traffic loading on the fill immediately behind the retaining wall shall be taken as acting horizontally over the full height of the wall, except where specified otherwise. In the absence of more exact calculations, the approximate nominal horizontal pressure for suitable material properly consolidated may be assumed to be:

(a)	for NA loading:	5 kN/m²	on the total length of a wall.
(b)	for 36 units of NB loading:	15 kN/m²	on a length of abutment wall of 5 m and on a length of side wall of 4 m per two axles; but reducing in intensity with depth where spreading of the load along the wall is likely to occur.
(c)	for NC loading:	12 kN/m²	on an 8 m length of abutment wall symmetrically under the centre line of a carriageway but reducing in intensity with depth where spreading along the wall is likely to occur, and on the total length of a side wall but acting only on the lower part b_1 - 3.5 m below the road level at the kerb, where b_1 is the distance from the centre line of the carriageway to the retaining wall.

Where an adequately designed reinforced concrete approach slab supported at one end by the bridge abutment is provided, no traffic load surcharge need be considered on the upper part of the wall equal to a height of two-thirds the length of the approach slab measured normally to the wall. The above pressures shall be considered to act only on the lower part.

2.4.3 Design Actions or Pressures Caused by Soil on Retaining Structures

For the design actions or pressures caused by soil acting on retaining structures, except for surcharge due to traffic, γ_{fL} shall be taken as in Table 2.4, except as defined in Section 2.4.3.2. For surcharge due to traffic, the values given in Section 2.6 shall be used.

Table 2.4: γ_{fL} for design actions for soil on retaining structures

γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
Approx. theory (see 2.4.2) More accurate theory (2.4.1)	1.5 1.4	1.3 1.2	1.5 1.4	1.1 1.0	1.1 1.0	1.1 1.0

2.4.3.1 Alternative Load Factor for Relieving Effects

Where the structure or element under consideration is such that the application of γ_{fL} as given in Section 2.4.3 for the ultimate limit state causes a less severe total effect (see Section 1.3.6) than would be the case if a reduced value of γ_{fL} had been taken, then a reduced value compatible with the practical possibilities shall be adopted. The reduced value could be as low as 0.5 for filling materials and be almost zero for residual materials that induce very small active pressures, depending on the nature of the soil-structure interaction (see Section 6.6 of Part 1).

2.5 Water Pressure of Retained or Excluded Water

2.5.1 General

The action of retained or excluded water pressure and the effects of buoyancy on the substructure due to ground or river water and the flooding action of rivers in flow, shall be considered as long-term or transient effects depending on the particular case. Rivers in high flood shall be considered as transient, thus excluding the consideration of load combination (3), but where a water level is maintained for long periods of time it shall be included in load combination (3). Where the action of water pressure has a relieving effect, the minimum water level that is assumed shall be considered.

2.5.2 Design Actions or Pressures Caused by Water

For the design actions or pressures caused by water acting on the substructures, the factor γ_{fL} shall be taken as in Table 2.5, except as defined in Section 2.5.2.1.

Table 2.5: γ_{fL} for design actions for pressures caused by water

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
	1.2	1.05	1.2	1.0	1.0	1.0

2.5.2.1 Alternative Load Factor for Relieving Effects

Where the structure or element under consideration is such that the application of γ_{fL} as given in Section 2.5.2 for the ultimate limit state causes a less severe total effect (see Section 1.3.6) than would be the case if γ_{fL} applied to all parts had been taken as 1.0, values of 1.0 shall be adopted.

2.6 Traffic Loading

2.6.1 General

2.6.1.1 Live Loading

Live loading due to Traffic on Highway Bridges (NABC) consists of three types:

(a) Normal Loading (NA) (refer to Section 2.6.3)
(b) Abnormal Loading (NB) (refer to Section 2.6.4)
(c) Super Loading (NC) (refer to Section 2.6.5)

Whereas the NC loading may be omitted on certain routes, all highway bridges shall be designed for both NA and at least NB24 loading (refer to Section 2.6.4).

2.6.1.2 Standard Traffic Loading

Standard traffic loading (SNABC) consists of:

(a) NA Loading
(b) NB Loading (Type NB36) (refer to Clause 2.6.4.1a)
(c) NC Loading (Type NC-30 x 5 x 40) (refer to Clause 2.6.5.1)

2.6.1.3 Impact Allowances and Dynamic Effects

No additional allowances need be made for the effects of wheel impact. These effects, together with an allowance for the dynamic effect of moving traffic, are included in the NA loading. Whereas the allowance for the latter effect as provided by the Swiss impact factor

$$\varphi_2 = 0.05\left(\frac{100 + L_S}{10 + L_s}\right)$$ (Where L_s is the equivalent span length in meters)

will in most cases be sufficient, it may be necessary in the case of certain structures with members whose natural frequencies of vibration correspond with the frequency of passage of vehicles or groups of vehicles, to carry out an investigation with an applied loading which simulates the NA loading reduced by a factor equal to the reciprocal of $(1 + \Phi_2)$. Likewise, when assessing the load-bearing capacity of any bridge designed for NB loading in terms of real vehicles, cognisance must be taken of the fact that the specified design loads include an allowance for impact. The impact and/or dynamic effects of the slow-moving multi-wheeled trailers simulated by the NC loadings can be considered to be negligible if the maximum speed when bridges are crossed is limited to 10 km/h. The design engineer shall investigate and satisfy himself that the effects are adequately taken care of in the design.

2.6.1.4 Application of Traffic Loading

Each element and structure shall be examined under the effects of forces which can coexist in every possible combination. Design forces shall be selected and applied in such a way that the most severe effect is caused in the elements of the structure under consideration. The NA, NB and NC loadings shall be applied separately, except as provided for in Section 2.6.5.2.

Attention is drawn to the requirement that the effects of traffic load or parts of traffic load shall not be taken into account where these are opposite in sign to the total effect or where the most severe effect on the element will be diminished by its presence, except in the case of NC loading as described in Section 2.6.5.2.

2.6.2 Width and Number of Notional Traffic Lanes to be Used in Conjunction with Type NA Loading

The width and number of notional lanes, presence of hard shoulders and central islands, are integral to the disposition of NA, NB and NC loading.

In determining the number of notional traffic lanes for the purpose of applying the provisions of Section 2.6.3, the width of the bridge deck between kerbs shall be considered and not merely the travelled width, i.e. shoulders, shall be included in the carriageway width. The number of notional lanes shall be as follows:

(i) *Bridges having a carriageway width of 4.8 m or more:*
Notional traffic lanes shall be taken to be not less than 2.4 m nor more than 3.7 m wide. The carriageway shall be divided into the least possible number of notional lanes having equal widths as follows (Table 2.6):

Table 2.6: Number of notional lanes per carriageway width

Carriageway width (m)	Number of notional lanes*
4.8 up to and including 7.4	2
above 7.4 up to and including 11.1	3
above 11.1 up to and including 14.8	4
above 14.8 up to and including 18.5	5
above 18.5 up to and including 22.2	6

* notional lanes are imaginary lanes for the application of the design loading and should not be confused with traffic lanes on the roadway

(ii) *Bridges with a carriageway width of less than 4.8 m:*
Where the carriageway on a bridge is less than 4.8 m in width, it shall be taken to have a number of notional lanes equal to the width of the carriageway in metres divided by three.

The number of notional lanes may be a non-integer number.

(iii) Should it be anticipated that at some future date sidewalks, medians, etc, are to be incorporated in the roadway width, these shall be added to the carriageway width for the purpose of determining the number of notional lanes. Otherwise the central median shall not be loaded with live load in considering the overall design of the structure, but the median shall be capable of supporting 24 units of NB loading (NB24) (refer to Section 2.6.4.1(b)).

(iv) Where dual carriageways are carried on a single superstructure, the number of notional lanes on the bridge shall be taken as the sum of the number of notional lanes in each of the single carriageways, determined as in Table 2.6. In the case where a unified substructure carries two separate superstructures of a dual carriageway, or carries multi-level superstructures, the number of notional lanes carried by the substructure shall be taken as the sum of the number of notional lanes on the superstructure.

2.6.3 Type NA Loading

<u>2.6.3.1 Definition of NA Loading</u>

Type NA loading represents normal traffic loading. The structure and its elements shall be designed to resist the most severe effects of (1) and (2) combined, or only (3).

(1) A nominal distributed lane loading, $Q_a = Q_a(L)$, acting on the whole or parts of the length of any notional lane or combination of such lanes. In the longitudinal direction it is uniformly distributed for any continuous part of a notional lane, but the intensity may be different for separate parts as explained in Section 2.6.3.2 (distributed load). The average amount of this load per linear metre of loaded notional lane shall be Q_a = 36 kN for loaded lengths up to 36 m, and for loaded lengths in excess of 36 m shall be derived from the formula

$$q_{a1} = \frac{180}{\sqrt{L}} + 6$$

where

L = the effective loaded length in metres as defined in Section 2.6.3.2 (distributed load) and

Q_a = average load per metre of notional lane in kN.

Figure 2.1 illustrates the curve of this loading.

The transverse distribution and position of the above loading shall be as explained in Sections 2.6.3.2 (distributed load - clause (b)) and 2.6.3.2. (transverse position of NA loading).

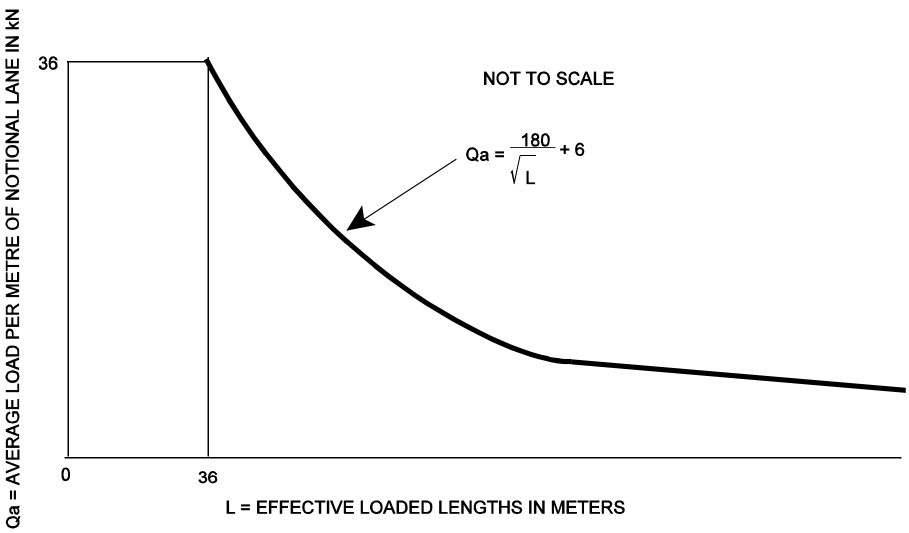


Figure 2.1: Loading curve for type NA loading

(2) One nominal axle load of $\frac{144}{\sqrt{n}}$ kN per notional lane, where n is the loading sequence number of the relevant, i.e. n = 1 for the first lane loaded with the axle load, n = 2 for the second lane, etc. This loading shall be applied in conjunction with the above loading as explained in Sections 2.6.3.2 (axle load) and 2.6.3.2 (transverse position of NA loading).

The concentration of stress under the line or point loads (1) and (2) which are obvious abstractions, need not be considered.

(3) Two nominal 100 kN wheels not less than one metre apart having circular or square contact areas of 0.1 m² each, placed anywhere in the carriageway with the axle in any direction to allow for local impact effects applied separately of (1) and (2). The contact areas, which may be circular or square, are derived by assuming a uniformly distributed effective pressure of 1.0 N/mm² (1 000 kPa).

<u>2.6.3.2 Application of NA Loading</u>

Distributed load:

(a)(i) *Longitudinal distribution.* In the longitudinal direction, the distributed part (1) of type NA loading shall be applied to those parts of any combination of notional lanes which will result in the most severe effect for the element or member under consideration. The effective loaded length L shall be taken as the aggregate length of the separate parts loaded in any single notional lane or combination of notional lanes in one or more carriageways.

The concept of a "separate part of a notional lane" as used in this context shall mean that continuous length of the notional lane that has entirely a positive (or alternatively negative) effect on the member being considered. Such lengths may be easily determined in the case of effects that are statically determinate, but are otherwise best determined by the application of influence surfaces or lines.

The total distributed load acting on any single part or any combination of parts shall be obtained by multiplying the aggregate length of such parts (i.e. the effective loaded length of the combination) by the corresponding average loading Q_a. The intensity of uniformly distributed loading acting on the separate parts may, however, vary and for any separate part may have a maximum value equal to the intensity Q_a, for an effective loaded length equal to the length of that part, provided that no combination of such parts shall have a total distributed load exceeding that obtained by multiplying the aggregate length of the relevant parts (effective loaded length of the combination of parts), by the corresponding average load Q_a, (see Appendix A, Section A.2.1).

(a)(ii) *Alternative methods of longitudinal distribution in any one lane and the adjacent notional lanes.* The following simplified but conservative method of applying NA loading is permissible as an alternative to Method a(i) above.

The uniformly distributed part (1) of type NA loading shall be applied to any one notional lane in the appropriate parts of the influence lines or surfaces for the element or member under consideration. The full or a reduced amount of this loading, which shall be not less than the values determined as described below, shall be similarly applied to all other notional lanes. The intensity of the uniformly distributed loading on separate parts of the influence line or surface in any one notional lane may for this alternative method be taken to be equal for any specific loading case, but to get the maximum effect it will be necessary to determine which of the separate parts are to be loaded.

- The loaded parts of the first loaded lane shall be fully loaded.
- The loaded parts of the second loaded lane shall be fully loaded for lengths up to 18 m, but shall have a reduced loading which varies linearly from full to two-thirds of the loading for loaded lengths varying from 18 m to 36 m respectively. For lengths in excess of 36 m the loading shall be two-thirds of the loading.
- The loaded parts of all other lanes shall similarly be fully loaded for loaded lengths up to 12 m, but shall have reduced loadings varying linearly from full to one-half of the loading for loaded lengths varying from 12 m to 36 m respectively. For lengths in excess of 36 m the loading shall be one-half of the loading.

The loaded length shall be taken as the length of the positive or negative portion, as the case may be, of the influence line or surface diagram for the element or member under consideration. Where the positive or negative portion of the influence line or surface consists of separate parts (as for continuous construction), the loaded length shall be taken as the aggregate of that number of positive parts or negative parts which has the most severe effect, using the loading appropriate to the length or to the total combined length of the loaded portions. Similarly, where the most severe effect is caused by locating the portions of loaded length on one side of the structure over one portion of carriageway and on the other side in a longitudinally adjacent portion of the carriageway, this shall be taken into consideration, using the loading appropriate to the combined length of the loaded portions (see Appendix A, Section A.2.3).

(b) *Transverse distribution.* The transverse distribution of the above loading shall be in the form of two equal and parallel line loads of Q_a /2 kN per metre to represent the effect of two longitudinal lines of wheels spaced 1.9 m apart in such a transverse position as to cause the most severe effect on the member being analysed, subject to the requirements of Section 2.6.3.2 (transverse position of NA loading).

The line loads on separate parts of any single lane may act in different transverse positions, but always parallel to the longitudinal direction of the lane. The two line loads acting on a separate part shall be of equal length and they shall terminate at each end on a line orthogonal to the direction of the lane.

Where the transverse distribution in a lane has no significant effect on the element being considered, the loading Q_a may be applied as a uniformly distributed loading over the full width of the notional lane.

Axle load:

The axle load (2) equal to $\frac{144}{\sqrt{n}}$ kN per notional lane, acting in conjunction with the distributed line loadings mentioned above, shall be applied in the form of two equal point loads to represent the effect of two wheel loads spaced 1.9 m apart. Only one axle load shall be applied to any single lane and it shall act transversely with the longitudinal direction of the lane so that the point loads coincide with the transverse positions of the line loads. The loading sequence "n" for the different lanes causing the most severe effect shall be selected. The axle loads in adjacent notional lanes do not necessarily act in a single line and shall be taken as acting at such positions as to cause the most severe effect on the member being analysed.

Where the transverse distribution in a lane has no significant effect on the element being considered, the axle loading may be applied as a knife-edge load uniformly distributed over the full width of the notional lane.

Transverse position of NA loading:
Loadings (1) and (2) occupy one or more notional lanes. The loading on any specific lane acting in conjunction with the loading on adjacent lanes shall be confined within the width of this notional lane and, except for the case described in Sections 2.6.3.2 (distributed load (b)) and 2.6.3.2 (axle load) where it is uniformly distributed, it shall be not closer than 0.5 m to the adjacent loading, or 0.25 m to the face of a kerb. Where an element can be more severely affected by the lateral translation of the loading, the loading on two notional lanes shall be considered to straddle two or three notional lanes, i.e. the loading on two notional lanes may occupy any position within three lanes, subject to the requirements of Section 2.6.3.2 (distributed load (b)) and the above minimum clearances. When such loading of two lanes straddles two or three lanes, no other lanes shall be loaded.

Narrow bridges:
In the case of bridges having a carriageway width between 2.4 m and 4.8 m, two loading cases shall be considered:

(a) One pair of line loads and one pair of point loads (axle load) subject to the provisions of 2.6.3.2 (distributed load (b))

(b) The latter loading multiplied by the non-integral factor defined in Section 2.6.2(ii) and uniformly distributed across the width of the bridge.

2.6.3.3 Design NA Loading

For design NA loading, γ_{fL} shall be taken as follows (Table 2.7):

Table 2.7: γ_{fL} for design NA loading

γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
	1.5	1.3	-	1.0	1.0	-

2.6.4 Type NB Loading

2.6.4.1 Definition of NB loading

Nominal NB loading is a unit loading representing a single abnormally heavy vehicle.

(1) Figure 2.2 shows the plan and axle arrangement for one unit of this loading. The weight factors for each of the four axles shall be multiplied by an appropriate number of units to give the applicable NB loading as specified in Sections 2.6.4.1(a) and (b).

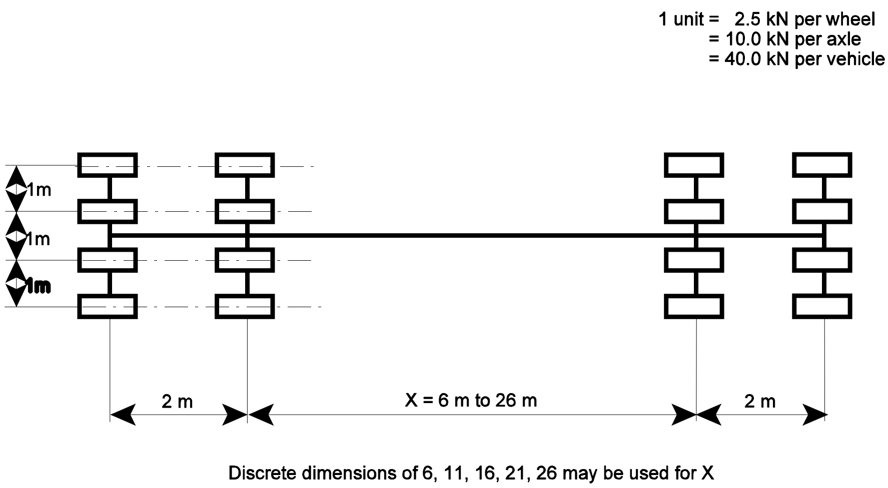


Figure 2.2: Type NB loading

(2) No allowance shall be made for impact.

(3) The design wheel loads shall be applied to a circular or square contact area derived by assuming a uniformly distributed effective pressure of 1.0 N/mm² (1000 kPa).

(4) The dimension X shall be selected between the limits of 6 m and 26 m to have the most severe effect. Discrete dimensions of 6, 11, 16, 21 and 26 m may be used for X.

Magnitude of NB loading:

(a) For *Type NB36 Loading* referred to in 2.6.1.2, 36 units of type NB loading shall be applied, which equals an axle loading of 360 kN, i.e. 90 kN per wheel. The effective contact area is accordingly defined by a circle of 340 mm diameter or a square having a 300 mm side.

(b) For *Type NB24 Loading* referred to in Sections 2.6.1.1 and 2.6.2, 24 units of type NB loading shall be applied which equals an axle loading of 240 kN, i.e. 50 kN per wheel. The effective contact area is accordingly defined by a circle of 276 mm diameter or a square having a 245 mm side.

2.6.4.2 Application of NB Loading

The NB vehicle shall be taken to occupy any transverse position within the length of the carriageway it is in and to be within 0.6 m from the face of a kerb, except that when the distance between the kerb and the balustrade exceeds 0.6 m it shall be placed up to within 0.15 m from the face of the kerb. No other traffic loading shall be considered to act in conjunction with this loading. In positioning the NB vehicle, account shall be taken of future incorporation of sidewalks into the roadway.

2.6.4.3 Design NB Loading

For design NB loading, γ_{fL} shall be taken as follows (Table 2.8):

Table 2.8: γ_{fL} for design NB loading

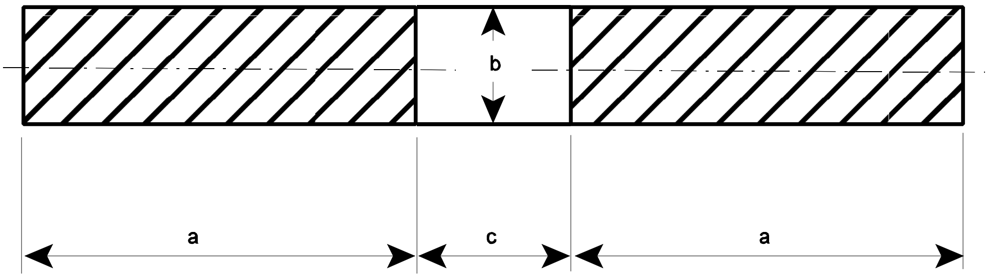
γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
	1.2	1.1	-	1.0	1.0	-

2.6.5 Type NC Loading

2.6.5.1 Definition of NC Loading

Nominal NC loading is a loading representing multi-wheeled trailer combinations (or self-propelled multi-wheel vehicles) with controlled hydraulic suspension and steering intended to transport very heavy indivisible payloads.

(1) Figure 2.3 shows the plan arrangement of Standard Type NC-30 x 5 x 40 loading. The loading is uniformly distributed over the area shown with an intensity of 30 kN/m², e.g. 150 kN/mm if b is 5.0 m.



'a' varies from 5 to 20 m   Discrete lengths of 5, 10, 15 and 20 m may be used for 'a'
'b' varies from 3 to 5 m   Discrete lengths of 3, 4 and 5 m may be used for 'b'
'c' varies from 0 to 25 m   Discrete lengths of 0, 5, 10, 15, 20 and 25 m may be used for 'c'

Figure 2.3: Type NC-36 X 5 X 40 loading

(2) No allowance shall be made for impact.

(3) The dimensions "a", "b" and "c" shall be selected, between the limits shown, to have the most severe effect. Discrete lengths may be used as follows:

for "a":	5, 10, 15 and 20 m
for "b":	3, 4 and 5 m
for "c":	0, 5, 10, 15, 20 and 25 m.

Note that this loading excludes the effects of separate mechanical horses; these have been omitted for the sake of simplicity. Where in practice this problem is critical for an existing bridge and cannot be overcome by using extended towbars, the permissible load on the trailers shall be determined by analysis of the effects of the actual complete train, in comparison with NC loading.

The responsible authorities may amend the configuration and loading intensity of type NC loading for a specific bridge.

2.6.5.2 Application of NC Loading

Type NC loading shall be directed along the centre line of any carriageway unless otherwise dictated by road geometrics, with an allowance for moving offline by 1.0 m in either direction and in such a position as to cause the most severe effect on the member being analysed. Subject to the above restriction of movement, the loading may be placed hard up against a kerb, but it shall not, however, be placed closer than 0.45 m from a balustrade. For loading on the line of travel as defined above, the dimensions "a", "b",and "c" in Figure 2.3 shall, however, be varied to cause the most severe effect on any structural member. Discrete lengths may be used as given. No other traffic loading shall be considered to act in conjunction with this loading in any single carriageway, but where dual carriageways are carried by a single superstructure or where a unified substructure carries separate superstructures of a dual carriageway or carries multi-level superstructures, an additional loading case shall be considered consisting of NC loading only on any one carriageway with two-thirds of the intensity of NA loading on the whole or parts of the other carriageways. Contrary to Section 2.6.1.4 the effects of NC loading that are opposite in sign may be taken into account, but the engineer shall use his discretion depending on the case.

2.6.5.3 Design NC and (NC + b NA) Loading

For design NC and (NC + b NA) loading, γ_{fL} shall be taken as follows (Table 2.9):

Table 2.9: γ_{fL} for Design NC and (NC + b NA) loading

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
	1.2	1.1	-	1.0	1.0	-

2.6.6 Primary Traffic Loading on Culverts

2.6.6.1 General

Culverts shall be designed to resist traffic loading in accordance with the principles described in Section 2.3.3. Types NA, NB and NC loading shall be applied where necessary in terms of the requirements of Sections 2.6.1 to 2.6.5. The approximate methods given below may be used. The assumption is made that the perimeter of the area over which the pressure acts spreads out at approximately 45° with an increase of depth in directions normal to the perimeter, but that the pressure remains uniformly distributed.

2.6.6.2 Approximate Methods for Determining the Effects of Traffic Loading on Rigid Culverts

The following methods are based on the procedure described in Section 2.3.3.2(i):

(a) *Type NA loading applied to rigid culverts:*

Nominal NA loading applied to culverts shall consist of:

(i) a uniformly distributed pressure of intensity

$$q_{a1} = \frac{36n}{b_r + 2h} \text{ kN/m}^2$$

where

n = number of notional lanes (see Section 2.6.2)

b_r = width of carriageway in metres

h = minimum soil cover to culvert in metres

The pressure shall be applied to the top surface of the culvert and shall extend over all or parts of the surface area so as to cause the maximum adverse effects.

(ii) a strip of uniformly distributed pressure

$$q_{a2} = \frac{\sum_{i=1}^{n} \frac{120}{\sqrt{i}}}{(b_r + 2h)2h} \text{ kN/m}^2 \text{ of width } 2h \text{ metres}$$

superimposed on (i) and extending for a length equal to (b_r + 2h) in the direction of the centre line of the culvert, where the symbols are as for (i) above.

The pressure shall be applied to the total width or parts of the width of the culvert so as to cause the maximum adverse effects.

(iii) two nominal loads of 100 kN not less than 1.0 m apart applied separately of loads (i) and (ii) and distributed over two circular areas equal to $\pi(h + 0.2)^2$ m² applied anywhere to the top surface of the culvert so as to cause the maximum adverse effect, where the symbol h is as in (i). Where the circular areas overlap, the pressures are additive.

(b) *The equivalent type NB36 loading applied to culverts:*
The equivalent NB36 loading shall consist of a single load

$$Q_b = 1.25(90 + 12L_s^{1.8})$$

L_s = effective span of the culvert in metres

The load shall be distributed over a contact area of 300 x 300 mm² at such positions on the surface of the road as to cause the maximum adverse effects and spread through the depth of fill only, taken under the conditions of minimum cover, at an angle of 45°, i.e. $b_h = l_h = (0.3 + 2h)$, where b_h and l_h are respectively the load width and load length at the level of the top of the culvert. Further:

(i) In the case of rigid monolithic culverts and flexible culverts, the load width, measured in the direction of the centre line of the culvert, shall be increased by 0.7 (L_s - b_h + 0.3) and 0.35 (L_s - b_h + 0.3) respectively for $b_h < (L_s + 0.3)$.

(ii) In the case of interconnected non-monolithic box culverts which permit adequate shear transference between units, the load width shall be increased by 0.5 (L_s - b_h + 0.3) for $b_h < (L_s + 0.3)$.

(iii) In the case of unconnected non-monolithic box culverts and pipe culverts where the load width is less than the width of the unit, the load width shall be increased by 0.35 of the span or to the width of the unit, whichever is smaller.

(iv) Where the load length, measured at right angles to the centre line of the culvert, exceeds the outside span of the culvert, the load length shall be reduced by one half the excess.

The strength of rigid box and pipe culverts shall not be less than would result from 0.1 m and 0.4 m earth cover, dead and live loading conditions respectively. For reduced NB loadings, the equivalent single load can be reduced proportionately.

Note: This loading approximately simulates local maximum bending moments and shears up to spans of 6.0 m but does not simulate the total load on the culvert.

(c) *Type NC loading applied to culverts*:
Type NC-30 x 5 x 40 loading applied to culverts shall consist of a uniformly distributed loading of intensity

$$q_c \cong \frac{150}{5 + 2h} \text{ kN/m}^2$$

applied over the full span and a length of (5 + 2h) metres of the culvert. The pressure shall be applied to the top surface of the culvert in such a position as to cause the most adverse effects.

For any depth of embankment filling greater than 0.1m over the culvert top, the strength of the culvert shall be not less than that which would result from dead and live loading due to NB24 or b equivalent type NB36 (see (b) above) on a similar culvert with 0.1 m earth cover.

2.6.6.3 Design Traffic Loading on Culverts
For design traffic loading on culverts, γ_{fL} shall be taken as follows (Table 2.10):

Table 2.10: γ_{fL} for design traffic loading on culverts

γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
NA	1.5	1.3	-	1.0	1.0	-
NB	1.2	1.1	-	1.0	1.0	-
NC	1.2	1.1	-	1.0	1.0	-

2.6.6.4 Horizontal Effects of Primary Traffic Loadings on Culverts
The horizontal effects of primary traffic loading on culverts shall be determined in accordance with the principles of soil mechanics or, in the absence of such an analysis, shall be estimated in accordance with Section 2.4.2.

2.7 Sidewalk and Cycle Track Live Loading

2.7.1 Bridges Supporting Sidewalks and Cycle Tracks only

2.7.1.1 Nominal Live Load
The nominal live load on elements supporting sidewalks and cycle tracks only shall be uniformly distributed and the load intensity determined as follows:

(a) For loaded lengths of 25 m and under, 5.0 kN/m²
(b) For loaded lengths "L" in excess of 25 m, $25/\sqrt{L}$ kN/m², but not less than 1.5 kN/m².

Special consideration shall be given to the intensity of the live load to be adopted on loaded lengths in excess of 25 m where exceptional crowds may be expected (e.g. where a pedestrian bridge serves a sports stadium).

2.7.1.2 Design Loading for Live Load on Sidewalks and Cycle Tracks

For design live loading on sidewalks and cycle tracks, γ_{fL} shall be taken as follows (Table 2.11):

Table 2.11: γ_{fL} for live loads on sidewalks and cycle tracks

γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
	1.5	1.3	-	1.0	1.0	-

2.7.1.3 Vibration Serviceability

Consideration shall be given to vibration that can be induced in pedestrian or cycle track bridges by resonance with the movement of users. The structure shall be deemed to be satisfactory where its response, as calculated in accordance with Appendix C, complies with the limitations specified therein.

2.7.2 Elements Supporting Sidewalks or Cycle Tracks and a Highway

2.7.2.1 Nominal Live Load

On sidewalks and cycle tracks carried by elements that also support highway loading, the nominal live load shall be taken as 0.8 of the value specified in Section 2.7.1.1(a) or (b), as appropriate, provided that the contribution by the highway load to the load effect being considered is at least equal to that of the sidewalk/cycle track load. Where the sidewalk, or sidewalk and cycle track together, is wider than 2.0 m, these intensities may be further reduced by 15 per cent on the first metre in excess of 2.0 m and by 30 per cent on the second metre in excess of 2.0 m. No further reduction shall be made for widths exceeding 4.0 m. These intensities may be averaged and applied as a uniform intensity over the full width of the sidewalk.

Where a main structural member supports two or more highway traffic lanes, the sidewalk/cycle track loading to be carried by the main member may be reduced by 0.5 of the values specified in Section 2.7.1.1(a) or (b) as appropriate.

However, where crowd loading is expected, special consideration shall be given to the intensity of live loading to be adopted.

In addition, where the sidewalk or cycle track is not protected from highway traffic by an effective barrier, any four wheels of 24 units of type NB loading (see Section 2.6.4) acting alone in any position shall be considered.

The nominal traffic loading on the carriageway to be associated with sidewalk or cycle track loading shall be type NA loading (see Section 2.6.3).

Where highway traffic can encroach onto cycle track areas, such areas shall be loaded with standard traffic loading (see Section 2.6.1.2).

2.7.2.2 Design Loading for Live Load on Sidewalks and Cycle Tracks Carried by Elements that also Support Highway Loading

The factor γ_{fL} to be applied to the nominal loads shall be as follows (Table 2.12):

Table 2.12: γ_{fL} for live loads on sidewalks and cycle tracks on highway support elements

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
Sidewalk and cycle track	1.5	1.3	-	1.0	1.0	-
NA	1.5	1.3	-	1.0	1.0	-
NB	1.2	1.1	-	1.0	1.0	-

2.8 Erection Loads

2.8.1 General

For the ultimate limit state, erection loads shall be considered in accordance with Sections 2.8.2 to 2.8.5.

For the serviceability limit state, nothing shall be done during erection that will cause damage to the permanent structure or will alter its response in service from that considered in design.

2.8.2 Temporary (short-term) Erection Loads

2.8.2.1 Nominal Loads

The total weight of all temporary materials, plant and equipment to be used during erection shall be taken into account. This shall be accurately assessed to ensure that the loading is not underestimated.

2.8.2.2 Design Loads Caused by Temporary Loads

The factor γ_{fL} to be applied to all parts of the temporary erection loads shall be as follows (Table 2.13), except as specified below (relieving effects):

Table 2.13: γ_{fL} for loads caused by temporary loads

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
	-	1.15	-	-	1.0	-

Relieving effects; Where any temporary materials have a relieving effect, and have not been introduced specifically for this purpose, they shall be considered not to be acting. Where, however, they have been so introduced, precautions shall be taken to ensure that they are not inadvertently removed during the period for which they are required. The weight of these materials shall also be accurately assessed to ensure that the loading is not overestimated. This value shall be taken as the design load.

2.8.3 Permanent Loads

2.8.3.1 Nominal Loads

All dead and superimposed dead loads affecting the structure at each stage of erection shall be taken into account. The effects of the method of erection of permanent materials shall be considered and due allowance shall be made for impact loading or shock loading.

2.8.3.2 Design Loads due to Permanent Erection Loads

The design loads due to permanent erection loads for the combinations (1) and (2) shall be as specified in Sections 2.2.2 and 2.3.2 respectively.

2.8.4 Disposition of Permanent and Temporary Loads

The various dispositions of all permanent and temporary loads at all stages of erection shall be taken into consideration and due allowance shall be made for possible inaccuracies in their location. Precautions shall be taken to ensure that the assumed dispositions are maintained during erection.

2.8.5 Wind and Temperature Effects

Wind and temperature effects shall be considered in accordance with Sections 3.8 and 4.5 respectively.

3. SUPPLEMENTARY ACTIONS

3.1 General

Supplementary actions are defined in Section 1.3.4.

The forces given below are applicable to bridges. The only supplementary forces applicable to culverts are those given in Sections 3.5 and 3.6.

3.2 Centrifugal Forces

On curved bridges, point forces shall be applied in any two notional lanes at 50 m centres, acting in radial direction at the surface of the road and parallel to it.

3.2.1 Nominal Centrifugal Force

The nominal centrifugal force F_c shall be taken as

$$F_c = \frac{30\ 000}{r + 150} \text{ kN}$$

where
r = the radius of curvature of the lane (in m).

Each force F_c shall be either taken as a single force or subdivided into two parts of one-third F_c and two-thirds F_c at 5.0 m centres longitudinally.

3.2.2 Associated Nominal Primary Live Load

With each centrifugal force there shall also be considered a vertical live load of 300 kN, distributed uniformly over the notional lane for a length of 5.0 m.

Where the centrifugal force is subdivided, the vertical live load shall be subdivided in the same proportions. The 100 kN portion shall be considered to act as a point load coincident with the one-third F_c force, and the 200 kN portion shall be considered as a distributed load applied uniformly over the notional lane for a length of 1.0 m and coincident with the two-thirds F_c force.

3.2.3 Force Combination

Centrifugal force shall be considered in combination (2) only, and need not be taken as being coexistent with other secondary forces caused by live loads.

3.2.4 Design Force

For the centrifugal forces and primary live load, γ_{fL} shall be taken as follows (Table 3.1):

Table 3.1: γ_{fL} for centrifugal forces and primary live load

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
	-	1.5	-	-	1.0	-

3.3 Longitudinal Force

The longitudinal force resulting from traction or braking of vehicles shall be taken as the more severe of Sections 3.3.1 or 3.3.2, applied at the road surface and parallel to it in one notional lane only.

3.3.1 Nominal Force for Type NA

The nominal force for NA shall be 3 kN/m of loaded length plus 100 kN, subject to a maximum of 400 kN, applied to an area one notional lane wide multiplied by the loaded length.

3.3.2 Nominal Force for Type NB

The nominal force for NB shall be 20 per cent of the total nominal NB load adopted, applied as equally distributed between the eight wheels of the two axles of the vehicle, 2.0 m apart (see Section 2.6.4).

3.3.3 Nominal Force for NC-36 x 5 x 40 Loading

On the assumption that the maximum velocity of the vehicles represented by this loading will not exceed 10 km/h when crossing bridges, it may be assumed that the effects of the braking or tractive forces will not exceed those caused by NB loading and need not be considered.

3.3.4 Associated Nominal Primary Live Load

Type NA or NB load, applied in accordance with Sections 2.6.3.2 or 2.6.4.2 shall be considered to act with the longitudinal load as appropriate.

3.3.5 Force Combination

Longitudinal forces shall be considered in combination (2) only, and need not be taken as being coexistent with other secondary forces caused by live loads.

In assessing the reaction forces, the dynamic behaviour of the structure and all other coexistent factors contributing to the reaction shall be considered. If significant, the longitudinal force shall be considered to act for an effective period of four seconds (e.g. it builds up to its peak value in one second, maintains this for two seconds, and dies off in one second).

3.3.6 Design Forces

For the longitudinal forces and primary live load, γ_{fL} shall be taken as follows (Table 3.2):

Table 3.2: γ_{fL} for longitudinal forces and primary live load

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
NA NB	- -	1.25 1.1	- -	- -	1.0 1.0	- -

3.4 Accidental Force caused by Skidding

On straight and curved bridges a single point force shall be considered in one notional lane only, acting in any direction parallel to the surface of the highway.

3.4.1 Nominal Force

The nominal force shall be taken as 250 kN.

3.4.2 Associated Nominal Primary Live Load

Type NA loading, applied in accordance with Section 2.6.3.2 shall be considered to act with the accidental skidding force.

3.4.3 Force Combination

Accidental forces caused by skidding shall be considered in combination (2) only, and need not be taken as being coexistent with other secondary forces caused by live loads.

3.4.4 Design Force

For the skidding primary live load, γ_{fL} shall be taken as follows (Table 3.3):

Table 3.3: γ_{fL} for skidding primary live load

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
	-	1.25	-	-	1.0	-

3.5 Forces caused by Vehicle Collisions with Balustrades

3.5.1 Nominal Forces

<u>3.5.1.1 Forces on Elements of the Structure</u>

In the design of the elements of the structure that support the balustrades, the actual forces, moments or shears required to bring about the collapse of the balustrade or the collapse of the connection to the element supporting the balustrade (whichever is the greater) shall be regarded as the nominal forces, moments of shears applied to the element.

3.5.1.2 Forces on Balustrades

In the design of balustrades, the following forces, or such other forces as the responsible authority may specify, are to be considered. Traffic railings should provide a smooth, continuous face of rail on the traffic side with posts preferably set back from the face of the rail. It is essential that the railing members be structurally continuous, including the anchorages at the ends.

(a) *On kerbs:* Where sidewalk, kerb and balustrade form an integral system, the forces shall be as set out in (b) and (c), as applicable. Where kerbs are not integral, the force shall be 7 kN per linear metre of kerb and shall be applied laterally at the top of the kerb (see Figure 3.1).

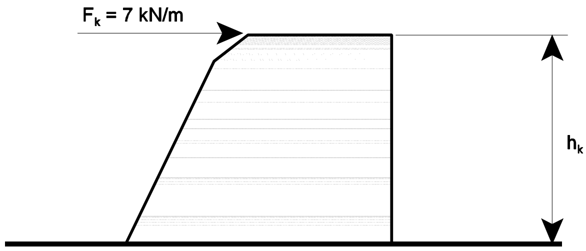


Figure 3.1: Forces on kerbs

(b) *On Class I balustrades, viz. those required to resist impact by vehicles on highway bridges*: The force F_b shall be 50 kN and shall be applied laterally, at or below a limiting height of 700 mm above the level of the road adjacent to the kerb for a kerb width of 150 mm, increasing linearly to a limiting height of 900 mm for a kerb width of 450 mm. For kerb widths greater than 450 mm, the limiting height shall remain at 900 mm (Figure 3.2).

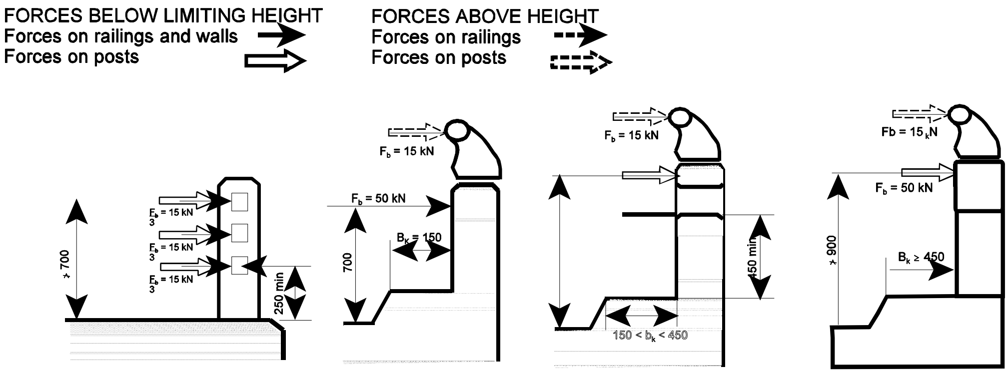


Figure 3.2: Forces on Class I balustrades

The forces to be applied to elements of the balustrade below the aforementioned limiting heights shall be as follows:

(i) Force F_b shall be divided equally between longitudinal railings and the characteristic moment for each of these members at the centre of the panel and at the posts shall be $F_bL/6n$ where n is the number of railings and L the length between supports.

(ii) A transverse force of F_b/n acting simultaneously with a longitudinal force of $F_b/2n$ shall be applied at each post at the level of each longitudinal railing. Where the tensile strength of the rail members is maintained through a series of post spaces, the longitudinal force may be divided along as many as four posts in this continuous length.

(iii) For concrete parapet walls, force F_b shall act at the level of the aforementioned limiting heights and shall be spread longitudinally over a length not exceeding 300 mm, and the effect shall be spread at an angle of 45° to the horizontal for the purpose of determining the length of wall resisting the force (Figure 3.3).

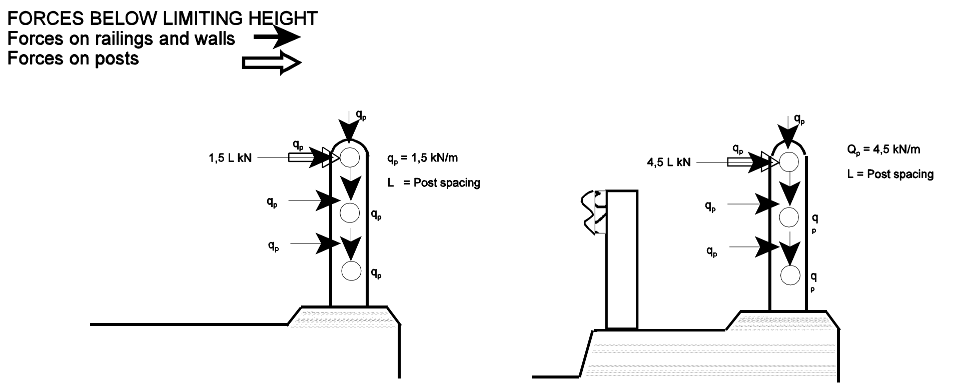


Figure 3.3: Forces on Class II balustrades

The forces to be applied to elements of the balustrade above these limiting heights shall be as specified in paragraphs (i), (ii) and (iii) above, excepting that the force F_b shall be reduced to $F' = 15kN_b$ or such other force as the responsible authority may specify.

This force shall act independently of the specified 50 kN force.

(c) *On Class II balustrades, viz. those required to contain only pedestrians on highway or pedestrian bridges* (Figure 3.3):

(i) In the case of balustrades on pedestrian bridges, the distributed force q_p shall be 1.5 kN per linear metre acting transversely, together with a simultaneous vertical force of 1.5 kN/m acting vertically, on each longitudinal member. A transverse force of 1.5 L kN, where L is the post spacing in metres at the centre of the upper rail, shall act on each post independently of the longitudinal member forces specified. The characteristic moment for longitudinal railings shall be taken as 0.1 q_pL^2 at the centre of the panel and at the posts.

(ii) In the case of balustrades on highway bridges, the forces shall be identical to (i) above, excepting that the forces shall be increased from 1.5 to 4.5 kN/m for longitudinal members and from 1.5 L to 4.5 L for posts (Figure 3.3). For concrete parapet walls, a transverse force of 15 kN shall act at the top of the wall and shall be spread longitudinally at an angle of 45° to the horizontal for the purpose of determining the length of wall resisting the force (Figure 3.4).

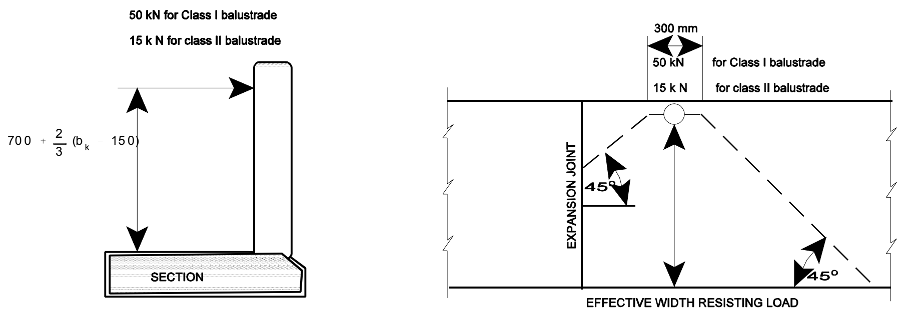


Figure 3.4: Forces on concrete parapet walls

3.5.2 Associated Nominal Primary Live Load

Any four wheels of 24 units of NB loading (see Section 2.6.4) shall be considered in whatever position they will have the most adverse effect on the element; where their application has a relieving effect they shall be ignored.

3.5.3 Force Combination

Forces due to vehicular collision with balustrades shall be considered in combination (2) only, and need not be taken as coexistent with other secondary forces caused by live loads.

3.5.4 Design Forces

For the force transmitted by the collapse of the balustrade or its connection to the element supporting the balustrade, for the force acting on the balustrades, and for the primary live load, γ_{fL} shall be taken as follows (Table 3.4):

Table 3.4: γ_{fL} for balustrades

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
	-	1.25	-	-	1.0	-

3.6 Supplementary Forces Applicable to Culverts

The only supplementary forces that require consideration in the design of culverts are those given in Section 3.5 (if balustrades are used), and those discussed below.

3.6.1 Nominal Longitudinal Force Resulting from Traction and Braking of Highway Vehicles

A nominal horizontal force F_L shall be considered to act at the level of the top of the culvert over a width equal to (3 + h) metres in a direction parallel to the direction of the carriageway where F_L is the greater of

$$\left\{ 100 \left(\frac{1 - h}{b_c} \right) \right\} \text{kN}$$

or $\{100 - 10(1 + h)h\}$ kN

where

h = the minimum depth of soil cover in metres

b_c = overall width of culvert (i.e. outside span measured parallel to the direction of the carriageway)

If both functions are negative, no longitudinal force need be applied.

3.6.2 Associated Nominal Primary Live Loads

Type NA or NB loads (see Section 2.6.6.2(a) and (b)) shall be considered to act with longitudinal force.

3.6.3 Force Combination

Longitudinal forces shall be considered in combination (2) only.

3.6.4 Design Forces

For the longitudinal forces and primary live loads, γ_{fL} shall be taken as follows (Table 3.5):

Table 3.5: γ_{fL} for longitudinal forces and primary live loads

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
NA	-	1.25	-	-	1.0	-
NB	-	1.1	-	-	1.0	-

3.7 Impact Forces on Bridge Supports

3.7.1 General

Bridge supports shall be considered to be vulnerable to impact force damage by highway vehicles where they are located within a distance of 8.0 m from the travelled way and where adequate *rigid* barrier protection is not provided. The analysis of the effects of impact forces on bridge supports is extremely complex. The designer must in addition to the provisions given herein use his judgement when designing for impact on bridge supports. The forces given below are applied as static forces and therefore cannot correctly simulate the effects of impact forces, and at best serve only to ensure a certain degree of robustness. In the design of the structure, this fact shall be given due consideration. Collision forces on the supports of highway bridges over railways or navigable water shall be as agreed with the appropriate authority.

3.7.2 Nominal Forces

The nominal forces caused by highway vehicles to be considered on vulnerable bridge supports are as follows:

(i) A horizontal longitudinal force of 3 v kN subject to a minimum force of 200 kN in a direction parallel to the direction of the underpassing road, where v = design speed for the underpassing roadway in km/h.

(ii) A horizontal transverse force of 120 kN in a direction normal to the direction of the underpassing roadway.

Only one such pair of forces acting simultaneously at 1.4 m above the shoulder breakpoint level of the adjacent road shall be applied on each support in turn. The transverse force shall be distributed longitudinally over the length of a support, or over 1.5 m, whichever is less.

Nevertheless, vulnerable reinforced-concrete bridge supports shall have a cross-sectional area of at least 0.45 m² and a thickness of at least 600 mm longitudinally and transversely.

3.7.3 Associated Nominal Primary Live Load

No primary live load need be considered on the bridge.

3.7.4 Force Combination

Impact forces shall be considered in combination (3) only, and need not be taken as coexistent with other supplementary actions or primary live loads.

3.7.5 Design Force

For impact forces on bridge supports, γ_{fL} shall be taken as follows (Table 3.6):

Table 3.6: γ_{fL} for impact forces on bridge supports

γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
	-	-	1.25	-	-	1.0

3.8 Wind Action

3.8.1 General

Wind action and its effects on a bridge depend on the bridge's geographical location, the local topography, the height of the relevant bridge member above ground, and the horizontal and cross-sectional dimensions of the bridge or element under consideration. The maximum pressures or suctions are due to gusts that cause local and transient fluctuations about the mean values. The natural frequency of the bridge or section of bridge under consideration can, in large flexible structures, also have an influence on the wind action. The accurate determination of wind effects is extremely complex.

Reference should be made to suitable texts on the structure of wind, and its effects on and interaction with various types of structural element[3-7].

This Code allows the application of one of two approaches for the evaluation of equivalent static wind forces and suggests a third dynamic method of analysis based on the statistical approach. The applicability of a particular approach will depend on the susceptibility of the bridge to wind actions and the importance of wind relative to the other actions that can be imposed on the bridge. Method A is applicable to structures for which wind forces are relatively unimportant and Method B to structures for which wind forces are important but for which dynamic effects are negligible. Method C applies to structures that are susceptible to dynamic effects.

3.8.2 Wind Action Assessment: Method A

3.8.2.1 General

This approximate method, in which equivalent static wind forces are determined, may be used for bridges with structural members on which the effects of wind action are small in relation to the total action effects and which members are not susceptible to dynamic effects. The types of bridge generally belonging to this category are low to normal level concrete bridges (of the order of 10 m effective height above ground level or less), of robust construction and small to medium spans which have substructures and/or superstructures of relatively high lateral (horizontal) stiffness. A structure or any of its members is unlikely to be susceptible to the dynamic effects of wind action if $n_o L_w > 8v$ (approximately),

where

n_o	=	fundamental natural frequency of the structure or member being considered
L_w	=	wind-loaded length of the structure or member being considered
v	=	mean hourly wind speed given in Figure 3.5. In cases where the required data is not available in Figure 3.5, an appropriate estimate of the mean hourly wind speed based on local information should be taken.

Refer to Appendix D for the evaluation of natural frequencies.

3.8.2.2 Nominal Transverse Design Forces Caused by Wind Acting on Structures

The effective transverse design forces caused by wind acting on structures shall be determined from the following resultant unit forces:

w1 = 1.5kN/m² on substructures and superstructures without live load, corresponding to a wind gust speed of approximately 40m/s; or

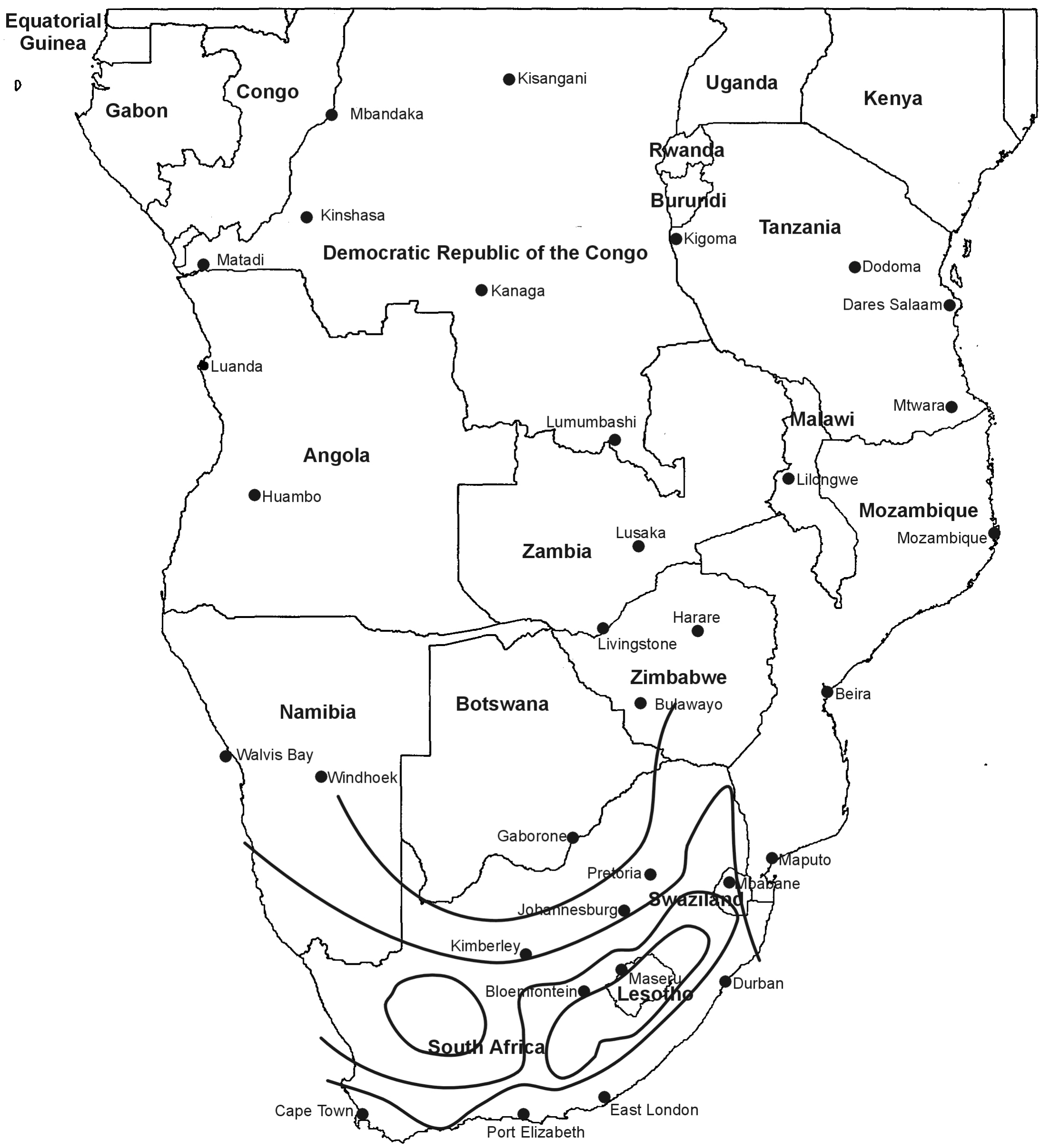


Figure 3.5: Isotachs of estimated mean hourly wind speeds in m/s appropriate to a height above ground level of 10 m in open level country and a 100-year return period

w2 = 1.0kN/m² on substructures and superstructures with live load, corresponding to a wind gust speed of approximately 32 m/s;

in combinations (3) and (2) respectively.

These forces shall act horizontally on such parts of the effective exposed area and in such a direction on the loaded or unloaded structure as to cause the most adverse effects on the member under consideration.

The effective exposed area shall be measured as follows:

(a) Substructures .... 1.0 A_1
(b) Superstructures:

- Slabs and solid superstructures with a ratio of width to total solid depth (including solid parapets) greater than 10 .... 1.5 A_1
- All members with a width to solid depth ratio of less than 2 .... 2.5 A_1

For intermediate ratios a linear interpolation shall be applied.

where
A_1 = nett exposed area of the structure,
= area of all solid parts projected on a vertical plane normal to the wind direction, less those areas where the load would have relieving effects.

3.8.2.3 Nominal Vertical Wind Force
The following additional nominal force shall be taken into account:

an upward or downward nominal vertical force of 1.0 kN/m² of deck and sidewalk plan area applied at the windward or leeward quarter point of the transverse superstructure width respectively.

3.8.2.4 Nominal Design Forces Caused by Wind on Live Loads
For the structure with live loading, additional forces caused by wind on the live loading shall be considered as follows:

- Structures supporting sidewalk live loading only: The nominal force shall be 0.75 kN per linear metre of exposed length of live load and shall act horizontally in the direction of the wind at a height of 1.0 m above sidewalk level.
- Structures supporting highway live loading with or without sidewalk loading: The nominal force shall be 1.5 kN per linear metre of exposed length of live load and shall act horizontally in the direction of the wind at a height of 2.0 m above road level.

3.8.3 Wind Action Assessment: Method B

3.8.3.1 General
For this method the determination of the equivalent static wind forces requires consideration of the effects of exposure, funnelling, the type and shape of the structure and the maximum probable gusting velocity in the expected lifetime of the bridge. It is applicable to medium to large-size reinforced concrete bridge structures on which the wind has a substantial effect in relation to the total action effects, but the members of which are not very susceptible to dynamic effects. In general this method may be applicable to structures or structural members for which

$4 v < n_o L_w < 8 v$ or $n_o > 0.5$ c/s (approximately)

where
v = mean hourly wind speed from Figure 3.5. In cases where the required data is not available in Figure 3.5, an appropriate estimate of the mean hourly wind speed based on local information should be taken.

n_o = fundamental natural frequency of the structure or member being considered (Refer to Appendix D for the evaluation of natural frequencies. However, if n_o < 0.5 c/s, the structure is sensitive to dynamic wind action and Method C should be used.)

L_w = wind-loaded length of the structure or member.

3.8.3.2 Nominal Transverse Design Forces Caused by Wind Acting on Structures

The equivalent static transverse wind forces shall be considered to act horizontally at the centroids of the appropriate areas and shall be derived from

$$W_1 = qA_1C_D$$

where

q = the dynamic pressure head acting on the relevant part of the structure (see below - dynamic pressure head)

A_1 = the solid area (in m²) (see below - Area A)

C_D = a coefficient which takes the shape of the relevant part of the structure, and the effects of frictional drag thereon, into account (see below - drag coefficient C_D for erection stages for beams and girders).

The dynamic pressure head shall be determined from

$$q = k_d v_c^2 \text{ in kN/m}^2 \text{ with } v_c \text{ in m/s}$$

where

v_c = the maximum wind gust speed (see below - maximum wind gust speed v_c on bridges without live load and with live load)

k_d = a coefficient which depends on the density of the air and thus on the site altitude above sea-level. It varies furthermore with temperature and atmospheric pressure. A temperature of 20°C has been selected as appropriate for southern Africa and the variation of mean atmospheric pressure with altitude is allowed for in Table 3.7. Intermediate values of "k_d" may be obtained by linear interpolation.

The following are the values of "k_d" for a range of site altitudes (Table 3.7):

Table 3.7: Values of "k_d" for a range of site altitudes

Site altitude above sea-level (m)	k_d
0	0.00060
500	0.00056
1 000	0.00053
1 500	0.00050
2 000	0.00047

Maximum wind gust speed v_c on bridges without live load: The maximum wind gust speed v_c on those parts of the bridge or its elements on which the application of wind loading increases the effect being considered, shall be equal to the function given below. For the remaining parts of the bridge or elements on which the application of wind loading gives relief to the effects under consideration, a reduced coexistent wind gust speed $v_c \cong b\ v_c$ shall be taken.

$$v_c = K_1 S_1 S_2 S_3 v$$

where

v = the mean hourly wind speed appropriate to a height above ground level of 10 m in open level country and a 100-year return period for which values in m/s for the location of the bridge may be obtained from the map of isotachs shown in Figure 3.5. In cases where the required data is not available in Figure 3.5, an appropriate estimate of the mean hourly wind speed based on local data should be taken.

Figure 3.5 is based on observations at isolated locations and does not necessarily indicate maximum wind speeds in all localities. Local records that go back for a sufficient period of time to make more reliable extreme value predictions should therefore be given preference. For sites that have a record of very severe winds, a special study may be required.

K_1 = a wind coefficient related to the return period which shall be taken as 1.0 for highway and pedestrian/cycle track bridges for a return period of 100 years. For pedestrian/cycle track bridges, subject to the agreement of the appropriate authority, a return period of 50 years may be adopted and K_1 shall be taken as 0.94. During erection, the value of K_1 may be taken as 0.82 corresponding to a return period of 10 years. Where a particular erection will be completed in approximately two days or less, and for which reliable wind speed forecasts are available, this predicted wind speed may be used as a mean hourly wind speed v, in which case the value of K_1 shall be taken as 1.0.

Should the values for other return periods be required, these may be obtained from Figure 3.6.

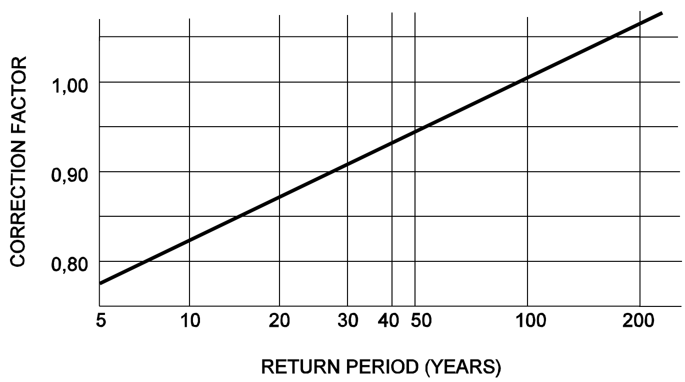


Figure 3.6: Coefficient K_1 related to the return period

S_1 = a funnelling factor. In general the funnelling factor shall be taken as 1.0 in valleys or gorges where local funnelling of the wind occurs, or where a bridge is sited to the lee of a range of hills or on an escarpment causing local acceleration of wind, a value of not less than 1.1 shall be taken.

S_2 = the gust factor dependent on the size and horizontal length or the height of a member to which the wind loading is applied, and which shall have the following values:

(a) $$S_2 = \left[1.6 - \frac{L_W^{0.4}}{40}\left(\frac{5}{H_S}\right)^{0.1}\right]$$

for the superstructure

where

L_w = horizontal wind-loaded length of the superstructure in metres (see Section 1.3.10)

H_s = average height in metres of the wind-loaded length of the superstructure, measured from the datum level shown in Figure 3.7.

(b) $$S_2 = 1.6 - \frac{H_p^{0.2}}{40}$$

for the vertical members of the substructure such as towers and piers

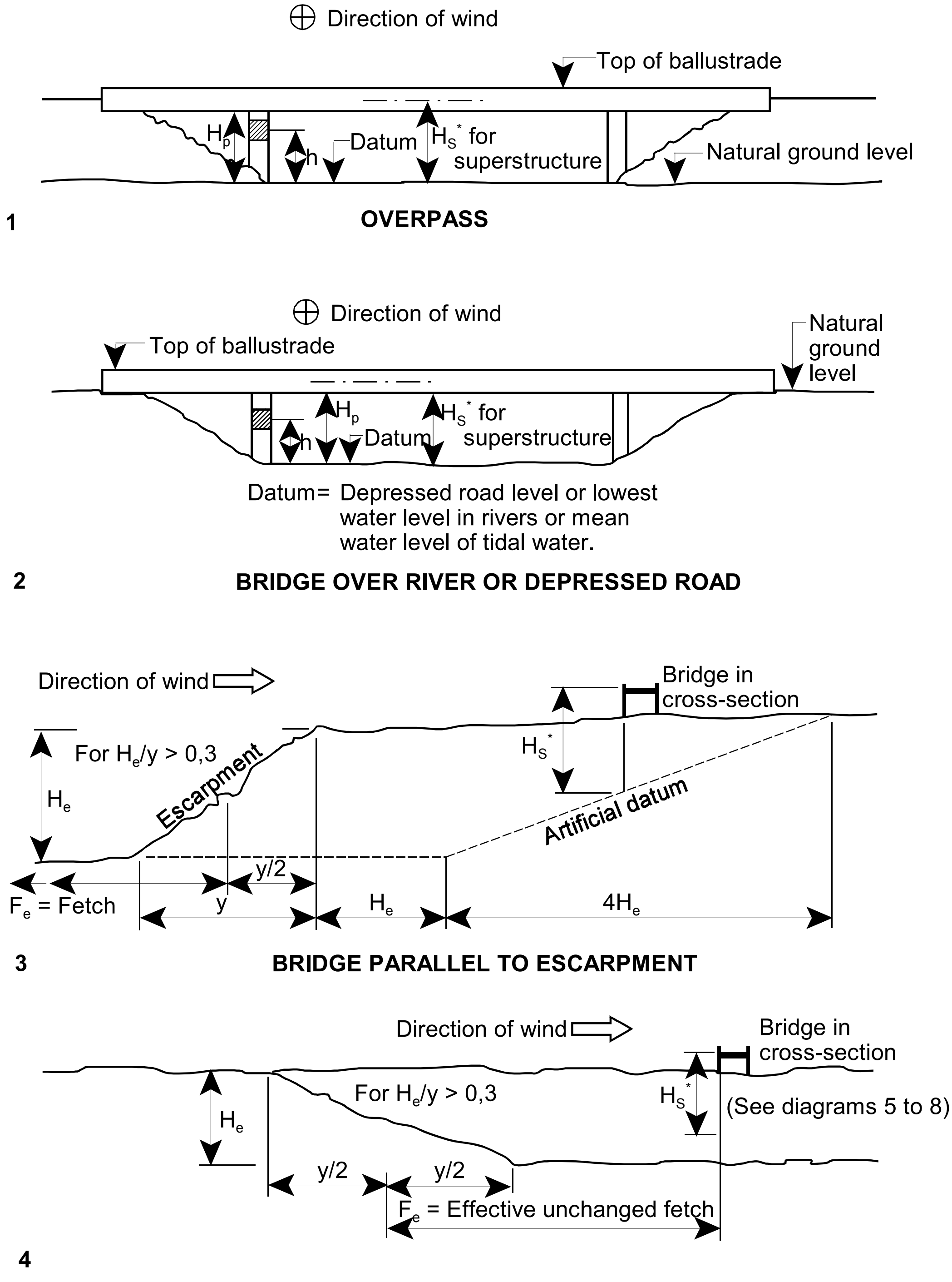
where

H_p = exposed height in metres of pier or tower.

(c) S_2 = 1.6 for small elements forming parts of the above members.

S_3 = the exposure factor which depends on the surface roughness of the terrain, the degree of shielding or exposure, and the effective height "h" in m, of the structure or member above ground level. The terrain is classified into three categories, viz.:

Category 1: Bridges over rivers or gorges, and grade separation structures in exposed open terrain or near the coast, with little or no protection from forests or artificial obstructions.

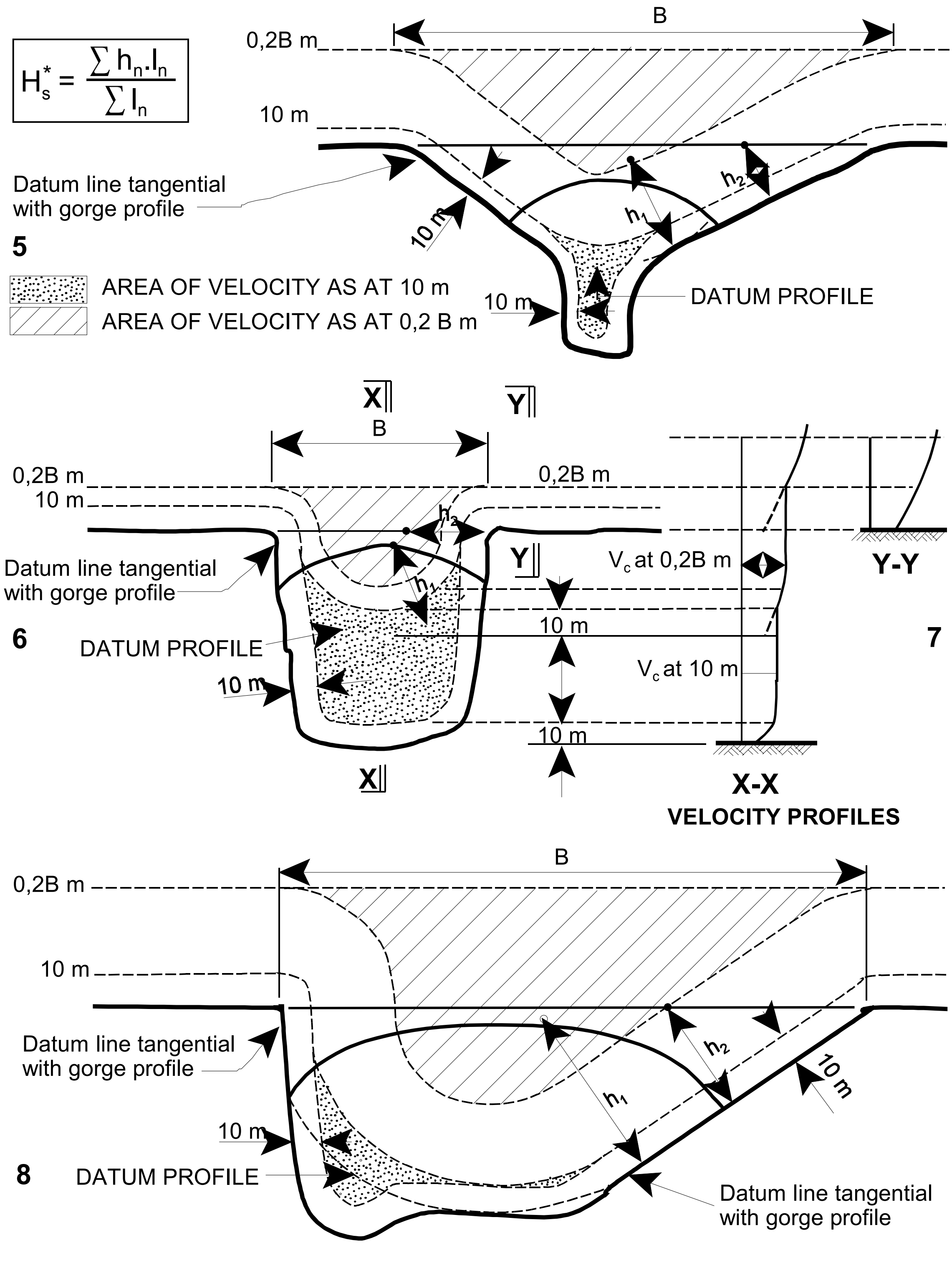


* The values indicated are the effective heights for winds transverse to the structure assuming that the ground profile (fetch) is largely unchanged in the upwind directions for a distance $F_e = H_s^2 / 10$.

For shorter unchanged fetches, or effects due to longitudinal and inclined wind directions, the values may be adjusted to take into account shielding by embarkments and surface roughness effects as a result of other topographical features.

Figure 3.7: Effective heights above ground level

$$H_s^* = \frac{\sum h_n . l_n}{\sum l_n}$$



5 - 8 BRIDGE OVER GORGE

FIGURE 3.7 (cont): Effective heights above ground level

Category 2: Bridges over rivers or gorges, and grade separation structures in open terrain with well-scattered obstructions having heights generally between 1.5 m and 10 m. This category includes terrain with bushes or sparsely distributed trees or buildings. It is the category on which the mean hourly wind speed is based.

Category 3: Bridges over rivers or gorges, and grade separation structures in mountainous or built-up terrain with numerous closely spaced obstructions such as buildings in cities or suburbs, towns and industrial areas, or in well-wooded areas that provide substantial reduction in the wind effect.

Terrain Categories 2 and 3 should not be used unless the appropriate terrain roughness persists in the upwind direction for at least 1.5 km; the factor S_3 should be varied according to the terrain if the roughness differs from one direction to another.

Values of S_3 for the three categories may be derived from the following formulae:

$$\text{Category 1} : S_3 = 1.36 \left(\frac{h}{250}\right)^{0.08} \text{ but } \nless 1.0$$

$$\text{Category 2} : S_3 = 1.36 \left(\frac{h}{300}\right)^{0.1} \text{ but } \nless 0.9$$

$$\text{Category 3} : S_3 = 1.36 \left(\frac{h}{400}\right)^{0.15} \text{ but } \nless 0.7$$

where h is the effective height in metres above ground level of the part of the bridge being considered, which shall be determined as shown in Figure 3.7.

Theoretically S_3 cannot exceed 1.36 which is the value at the gradient height, at which height surface roughness has no effect on the wind speed.

The height of vertical elements such as piers and towers shall be divided into convenient units, and the maximum wind gust speed shall be derived for the centroid of each unit.

Note: The above formulae apply only to winds originating in mature large-scale storms, i.e. extensive pressure system winds with short-period gusts of 3 to 10 seconds, and not to winds in localized storms such as thunder-storms which are a primary source of extreme gusts in certain areas. In such areas the maximum wind gust speed v_c, determined as above, shall be increased by an additional factor of 1.3.

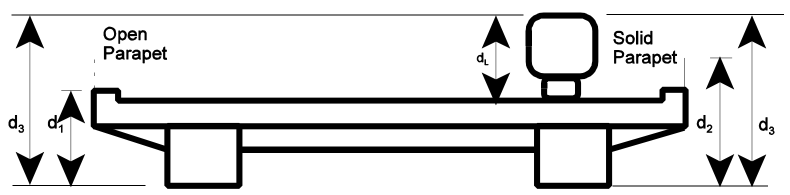
Maximum wind gust speed v_c on bridges with live load: The maximum wind gust speed on those parts of the bridge or its elements on which the application of wind loading increases the effect being considered, shall be as specified above (maximum wind gust speed v_c on bridges without live load), but not exceeding 35 m/s. For the remaining parts of the bridge or elements on which the application of wind loading gives relief to the effects under consideration, the lesser of a reduced coexistent wind gust speed $v_{\acute{c}} \leq b\ v_{\acute{c}}$ or 25 m/s shall be taken.

Area A: The area of the structure or element under consideration shall be the area of all solid parts projected on a vertical plane normal to the direction of the wind, derived as described below:

(a) *Erection stages for all bridges*:
The area A_1, at all stages of construction, shall be the appropriate unshielded solid area of the structure or element.

(b) *Highway bridge superstructures with solid elevation*:
For superstructures with or without live load, the Area A_1 shall be derived using the appropriate value of d as given in Table 3.8.

Table 3.8: Depth to be used in deriving A_1



Parapet	Unloaded bridge	Live loaded bridge
Open	$d = d_1$	$d = d_3$
Solid	$d = d_2$	$d = d_2$ or d_3 (whichever is greater)

d_L = 2.5 m above the roadway, or
3.7 m above the rail level, or
1.25m above sidewalk or cycle track

(i) *Superstructures without live load*: W_t shall be derived separately for the areas of the following elements:

- For superstructures with open parapets:
  - The superstructure, using depth d_1 from Table 3.8
  - The windward parapet or safety fence
  - The leeward parapet or safety fence.

Where there are more than two parapets or safety fences, irrespective of the width of the superstructure, only those two elements having the greatest unshielded effect shall be considered.

For superstructures with solid parapets: the superstructure, using depth d_2 from Table 3.8, which includes the effects of the windward and leeward parapets. Where there are safety fences or additional parapets, W_1 shall be derived separately for the solid areas of the elements above the top of the solid parapets.

(ii) *Superstructures with live load:* W_t shall be derived for the area A_1, as given in Table 3.8 which includes the effects of the superstructure, the live load and the windward and leeward parapets. Where there are safety fences or leeward parapets higher than the live load depth d_L, W_t shall be derived separately for the solid areas of the elements above the live load.

(iii) *Superstructures separated by an air gap:* Where two generally similar superstructures are separated transversely by a gap not exceeding 1.0 m, the nominal force on the windward structure shall be calculated as if it were a single structure, and that on the leeward superstructure shall be taken as the difference between the forces calculated for the combined and the windward structures (see Note 7 to Figure 3.8).

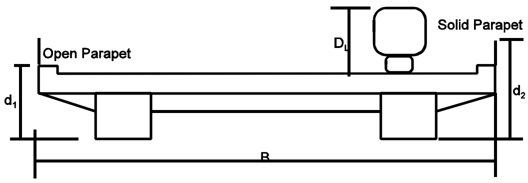
Where the superstructures are dissimilar or the air gap exceeds 1.0 m, each superstructure shall be considered separately without any allowance for shielding.

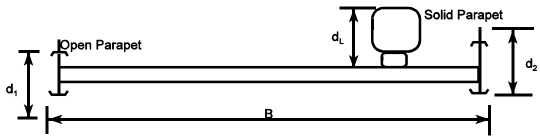
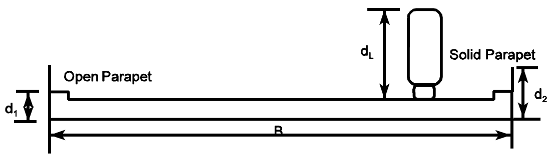
(c) *Pedestrian/cycle bridge superstructures with solid elevation*:

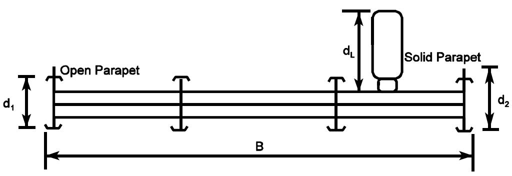
(i) *Superstructures without live load:* Where the ratio b/d as derived from Table 3.9 is greater than, or equal to 1.1, the area A_1 shall comprise the solid area in normal projected elevation of the windward exposed face of the superstructure and parapet only. W_t shall be derived for this area, the leeward parapet being disregarded.

Where b/d is less than 1.1, the area A_1 shall be derived as specified in (b).

Table 3.9: Depth d to be used in deriving C_D

<table>
<tr><th></th><th>Parapet</th><th>Superstructures without live load</th><th>Superstructures with live load</th></tr>
<tr><td rowspan="2">(a) Superstructures where the depth of the superstructure (D_1 or D_2) exceeds D_L



</td><td>Open</td><td>$d = d_1$</td><td>$d = d_1$</td></tr>
<tr><td>Solid</td><td>$d = d_2$</td><td>$d = d_2$</td></tr>
<tr><td rowspan="2">(b) Superstructures where the depth of the superstructure (D_1 or D_2)is less than D_L



</td><td>Open</td><td>$d = d_1$</td><td>$d = d_L$</td></tr>
<tr><td>Solid</td><td>$d = d_2$</td><td>$d = d_L$</td></tr>
</table>

(ii) *Superstructures with live load:* Where the ratio b/d as derived from Table 3.9 is greater than, or equal to 1.1, the area A_1 shall comprise the solid area in normal projected elevation of the deck, the live load depth (taken as 1.25 m above the sidewalk) and the parts of the windward parapet more than 1.25 m above the sidewalk. W_t shall be derived for this area, the leeward parapet being disregarded.

Where b/d is less than 1.1, W_t shall be derived for the area A_1 as specified in (b).

(d) *All truss-girder bridge superstructures*:

(i) *Superstructures without live load:* The area A_1 for each truss, parapet, etc, shall be the solid area in normal projected elevation. The area A_1 for the deck shall be based on the full depth of the deck.

W_t shall be derived separately for the areas of the following elements:

- Windward and leeward truss girders
- Deck
- Windward and leeward parapets

except that W_t need not be considered on projected areas of:

- Windward parapet screened by the windward truss, or vice versa
- Deck screened by the windward truss, or vice versa
- Leeward truss screened by the deck
- Leeward parapet screened by the leeward truss, or vice versa.

(b) *Superstructures with live load:* The area A_1 for the deck parapets, trusses, etc, shall be as for the superstructure without live load. The area A_1 for the live load shall be derived using the appropriate live load depth d_L as given in Table 3.8.

W_t shall be derived separately by the areas of the following elements:

- Windward and leeward truss girders
- Deck
- Windward and leeward parapets
- Live load depth

except that W_t need not be considered on projected areas of:

- Windward parapet screened by the windward truss, or vice versa
- Deck screened by the windward truss, or vice versa
- Live load screened by the windward truss or the parapet
- Leeward truss screened by the live load and the deck
- Leeward parapet screened by the leeward truss and the live load
- Leeward truss screened by the leeward parapet and the live load.

(e) *Parapets and safety fences*:
For open and solid parapets and fences, W_t shall be derived for the solid area, in normal projected elevation, of the element under consideration.

(f) *Piers:*
W_t shall be derived for the solid area, in normal projected elevation, of each pier. No allowance shall be made for shielding.

Drag coefficient C_D for erection stages for beams and girders: In clauses (a) to (e) below, requirements are specified for discrete beams or girders before deck construction or other infilling (e.g. shuttering).

(a) *Single beam or box girder*:
C_D shall be derived from Figure 3.8 in accordance with the ratio b/d.

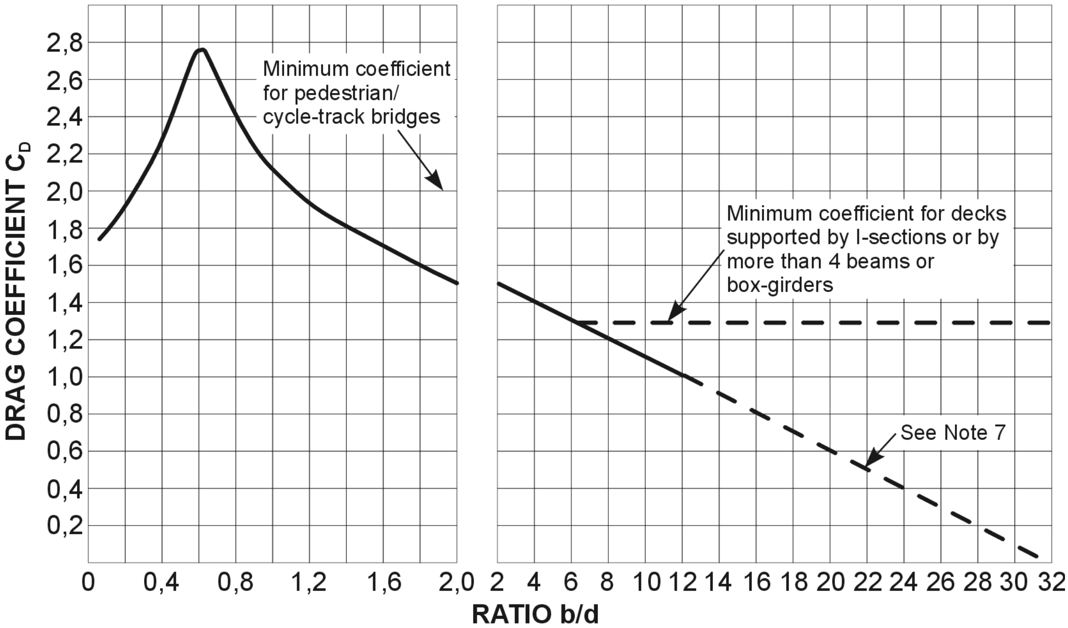


Figure 3.8: Drag coefficient C_D for superstructures with solid elevation

Notes to Figure 3.8

Note 1: These values are given for vertical elevations and for horizontal wind.

Note 2: Where the windward face is inclined to the vertical, the drag coefficieint C_D may be reduced by 0.5 per cent per degree of inclination from the vertical, subject to a maximum reduction of 30 per cent.

Note 3: Where the windward face consists of a vertical and a sloping part or two sloping parts inclined at different angles, C_D shall be derived as follows: For each part of the face, the depth shall be taken as the total vertical depth of the face (i.e. over all parts), and values of C_D derived in accordance with Notes 1 and 2. These separate values of C_D shall be applied to the appropriate area of the face.

Note 4: Where a superstructure is superelevated, C_D shall be increased by 3 per cent per degree of inclination to the horizontal, but not by more than 25 per cent.

Note 5: Where a superstructure is subject to inclined wind not exceeding 5° inclination, C_D shall be increased by 15 per cent. Where the angle of inclination exceeds 5°, the drag coefficient shall be derived from tests.

Note 6: Where the superstructure is superelevated and also subject to inclined wind, the drag coefficient C_D shall be specially investigated.

Note 7: Where two generally similar superstructures are separated transversely by a gap not exceeding 1m, the drag coefficient for the combined superstructure shall be obtained by taking b as the combined width of the superstructure. In assessing the distribution of the transverse wind load between the two separate superstructures (see above - Area A subclause b(iii)), the drag coefficient C_D for the windward superstructure shall be taken as that of the windward superstructure alone, and the drag coefficient C_D of the leeward superstructure shall be the difference between that of the combined superstructure and that of the windward superstructure. For the purposes of determining this distribution, if b/d is greater than 12, the broken line in Figure 3.8 shall be used to derive C_D. The load on the leeward structure is generally opposite in sign to that on the windward superstructure. Where the gap exceeds 1.0 m, C_D for each superstructure shall be derived separately, without any allowance being made for shielding.

(b) *Two or more beams or box girders*:
C_D for each beam or box shall be derived from Figure 3.8 without any allowance for shielding. Where the combined beams or boxes are required to be considered, C_D shall be derived as described below:

- Where the ratio of the clear distance between the beams or boxes to the depth does not exceed seven, C_D for the combined structure shall be taken as 1.5 times C_D, derived as specified in clause (a) above for the single beam or box.
- Where this ratio is greater than seven, C_D for the combined structure shall be taken as n times the value derived as specified in clause (a) for the single beam or box, where n is the number of beams or box girders.

(c) *Single plate girder*:
C_D shall be taken as 2.2.

(d) *Two or more plate girders:*
C_D for each girder shall be taken as 2.2 without any allowance for shielding. Where the combined girders are required to be considered, C_D for the combined structure shall be taken as 2(1 + c/20d), but not more than four, where c is the distance centre to centre of adjacent girders, and d is the depth of the windward girder.

(e) *Truss girders:*
The discrete stages of erection shall be considered in accordance with the requirements below (drag coefficient C_D for all truss-girder superstructures).

Drag coefficient C_D for all superstructures with solid elevation: See Figure 3.9. For superstructures with or without live load, C_D shall be derived from Figure 3.8 in accordance with the ratio b/d as derived from Table 3.9. Where designs are not in accordance with Table 3.9, and for those types of superstructure illustrated in Figure 3.10, drag coefficients shall be ascertained from wind tunnel tests, failing which conservative estimates shall be made.

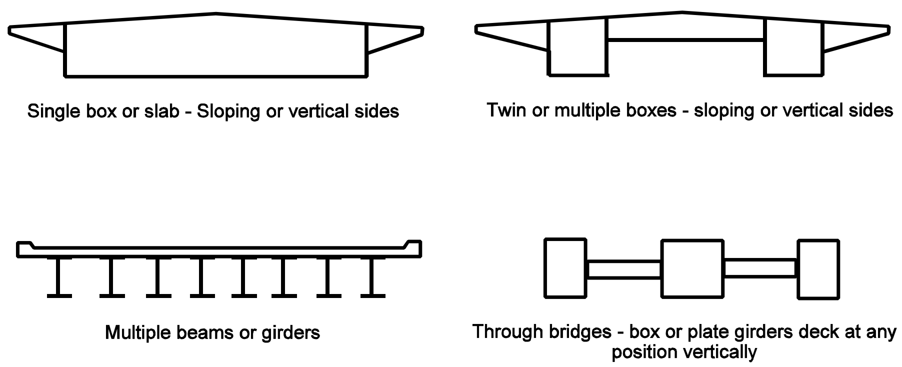


Figure 3.9: Typical bridge cross-sections to which Figure 3.8 applies

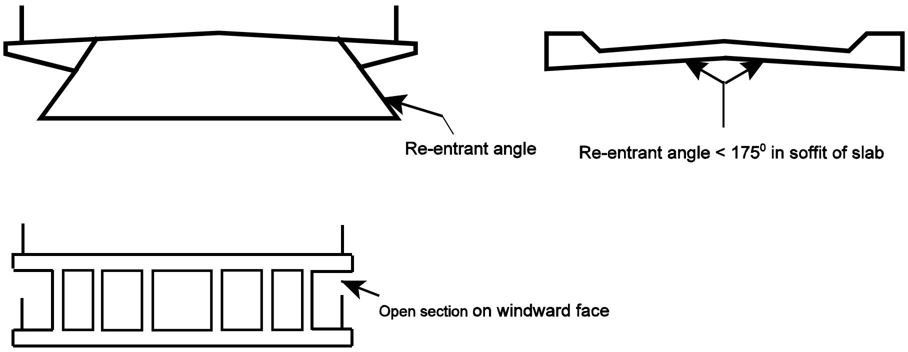


Figure 3.10: Typical bridge cross-sections that require wind tunnel tests

Drag coefficient C_D for all truss-girder superstructures:

(a) *Superstructures without live load:* The drag coefficient C_D for each truss and for the deck shall be derived as follows:

(i) For a windward truss, C_D shall be taken from Table 3.10. The solidity ratio of the truss is the ratio of the net area to the overall area of the truss.

Table 3.10: Drag coefficient C_D for a single truss

Solidity ratio	For flat-sided members	For round members where d is diameter of member	
		dvc < 6m²/s	dvc ≤ 6 m²/s
0.1	1.9	or 1.2	or 0.7
0.2	1.8	dv'c 1.2	dv'c 0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8

(ii) For the leeward truss of a superstructure with two trusses, the drag coefficient shall be taken as ηC_D.

Values of η are given in Table 3.11. The spacing ratio is the distance between centres of trusses divided by the depth of the windward truss.

Table 3.11: Shielding factor η

Spacing ratio	Value of η for solidity ratio of:				
	0.1	**0.2**	**0.3**	**0.4**	**0.5**
Less than 1	1.0	0.90	0.80	0.60	0.45
2	1.0	0.90	0.80	0.65	0.50
3	1.0	0.95	0.80	0.70	0.55
4	1.0	0.95	0.85	0.70	0.60
5	1.0	0.95	0.85	0.75	0.65
6	1.0	0.95	0.90	0.80	0.70

(iii) Where a superstructure has more than two trusses, the drag coefficient for the truss adjacent to the windward truss shall be derived as specified in (ii).

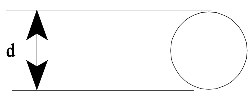
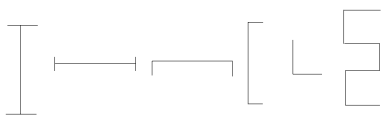
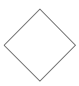
The coefficient for all other trusses shall be taken as equal to this value.

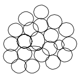
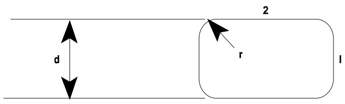
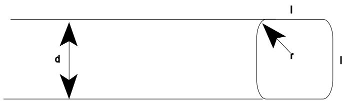
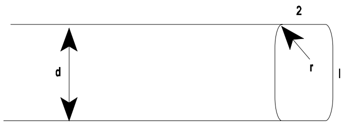
(iv) For the deck construction, the drag coefficient C_D shall be taken as 1.1.

(b) *Superstructures with live load:* The drag coefficient C_D for each truss and for the deck shall be as for the superstructure without live load. C_D for unshielded parts of the live load shall be taken as 1.45.

Drag coefficient C_D for parapets and safety fences: For the windward parapet or fence, C_D shall be taken from Table 3.12.

Table 3.12: Drag coefficient C_D for parapets and safety fences

Item	Description	C_D
	Circular sections $dv_c < 6$ $dvc \geq 6$ (where v_c is in m/s and d is in m) **Note:** On relieving areas use $v_{\acute{c}}$ instead of v_c	1.2 0.7
	Flat member with rectangular corner, crash barrier rail and solid parapets	2.2
	Square members diagonal to wind	1.5

	Circular stranded cables	1.2
	Rectangular members with circular corners - r > d/12	1.1*
	Rectangular members with circular corners - r > d/12	1.5*
	Rectangular members with circular corners - r > d/24	2.1
* For sections with intermediate proportions, C_D may be obtained by interpolation		

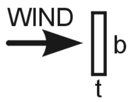
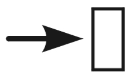
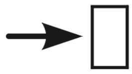
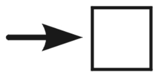
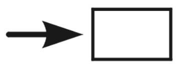
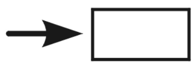
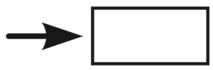
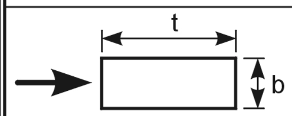
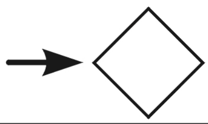
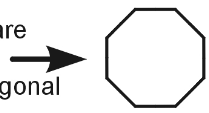
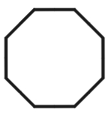
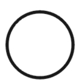
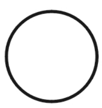
Where there are two parapets or fences on a bridge, the value of C_D for the leeward element shall be taken as equal to that of the windward element. Where there are more than two parapets or fences, the values of C_D shall be taken from Table 3.12 for the two elements having the greatest unshielded effect.

Where parapets have mesh panels, consideration shall be given to the possibility of the mesh becoming filled with ice. In these circumstances, the parapet should be considered as solid.

Drag coefficient C_D for piers: The drag coefficient shall be taken from Table 3.13. For piers with cross-sections dissimilar to those given in Table 3.13, wind tunnel tests shall be carried out, failing which conservative estimates shall be made.

C_D shall be derived for each pier, without reduction for shielding.

Table 3.13: Drag coefficient C_D for piers

PLAN SHAPE	$\frac{t}{b}$	C_D FOR PIER $\frac{\text{HEIGHT}}{\text{BREADTH}}$ RATIOS OF						
		1	2	4	6	10	20	40
	< $^{1}/_{4}$	1,3	1,4	1,5	1,6	1,7	1,9	2,1
	$^{1}/_{3}$ $^{1}/_{2}$	1,3	1,4	1,5	1,6	1,8	2,0	2,2
	$^{2}/_{3}$	1,3	1,4	1,5	1,6	1,8	2,0	2,2
	1	1,2	1,3	1,4	1,5	1,6	1,8	2,0
	$1^{1}/_{2}$	1,0	1,1	1,2	1,3	1,4	1,5	1,7
	2	0,8	0,9	1,0	1,1	1,2	1,3	1,4
	3	0,8	0,8	0,8	0,9	0,9	1,0	1,2
	≥ 4	0,8	0,8	0,8	0,8	0,8	0,9	1,1
 Square or Octogonal 		1,0	1,1	1,1	1,2	1,2	1,3	1,4
 12-Sided polygon		0,7	0,8	0,9	0,9	1,0	1,1	1,3
 Circle with smooth surface where $tv_c \geq 6$ m²/s.		0,5	0,5	0,5	0,5	0,5	0,6	0,6
 Circle with smooth surface where $tv_c < 6$ m²/s. Also circle with rough surface or projections.		0,7	0,7	0,8	0,8	0,9	1,0	1,2

Notes for Table 3.13

Note 1:	After erection of the superstructure, C_D shall be derived for a height breadth ratio of 40.
Note 2:	For a rectangular pier with radiused corners, the value of C_D shall be multiplied by (1 - 1.5r/b) or 0.5, whichever is greater.
Note 3:	For a pier with triangular nosings, C_D shall be derived as for the rectangle encompassing the outer edges of the pier.
Note 4:	For a pier tapering with height, C_D shall be derived for each of the unit heights into which the support has been subdivided (see Section 3.8.3.2 - maximum wind gust speed v_c on bridges without live load (S_3)). Mean values of t and b for each unit height shall be used to evaluate t/b. The overall pier height and the mean breadth of each unit height shall be used to evaluate height-breadth

3.8.3.3 Nominal Longitudinal Wind Force

The nominal longitudinal wind force W_L (in kN), taken as acting at the centroids of the appropriate areas, shall be the more severe of either;

(a) the nominal longitudinal wind force on the superstructure, W_{Ls}, alone; or

(b) the sum of the nominal longitudinal wind force on the superstructure W_{Ls}, and the nominal longitudinal wind force on the live load W_{LL}, derived separately as specified below.

All superstructures with solid elevation:

$$W_{Ls} = 0.25\, q\, A_1\, C_D$$

where

q is as defined in Section 3.8.3.2, the appropriate value of v_c for superstructures with or without live load being adopted.

A_1 is as defined in (b) and (c) of Section 3.8.3.2 (Area A) for the superstructure alone.

C_D = the drag coefficient for the superstructure (excluding reduction for inclined webs) as defined in Section 3.8.3.2 (Drag coefficient C_D for all superstructures with solid elevation) but not less than 1.3.

All truss-girder superstructures:

$$W_{Ls} = 0.5\, q\, A_1\, C_D$$

where

q is as defined above (all superstructures with solid elevation)

A_1 is as defined in D(a) of Section 3.8.3.2 (Area A)

C_D is as defined in Section 3.8.3.2 (Drag coefficient C_D for all truss-girder superstructures (a)(i) to (iv)), inclusive, ηC_D being adopted where appropriate.

Live load on all superstructures:

$$W_{LL} = 0.5\, q\, A_1\, C_D$$

where

q	is as defined above (all superstructures with solid elevation)
A_1	is the area of live load derived from the depth d_L as given in Table 3.8 and the appropriate horizontal wind-loaded length as defined for S_2 in Section 3.8.3.2 (Minimum gust speed v_c on bridges without live load).
C_D	= 1.45

Parapets and safety fences:

(a) With vertical infill members $W_L = 0.8\ W_t$
(b) With two or three horizontal rails only, $W_L = 0.4\ W_t$
(c) With mesh panels, $W_L = 0.6\ W_t$

where W_t is the appropriate nominal transverse wind load on the element.

Cantilever brackets extending outside main girders or trusses: W_L is the force derived from a horizontal wind acting at 45° to the longitudinal axis on the area of each bracket not shielded by a fascia girder or adjacent bracket. The drag coefficient C_D shall be taken from Table 3.12.

Piers: The force derived from a horizontal wind acting along the longitudinal axis of the bridge shall be taken as:

$$W_l = q\ A_2\ C_D$$

where

q	is as defined above (all superstructures with solid elevation)
A_2	is the solid area in projected elevation normal to the longitudinal wind direction (in m²)
C_D	is the drag coefficient, taken from Table 3.13, with values of b and t interchanged.

Nominal vertical wind force: An upward or downward nominal vertical wind force W_v (in N), acting at the centroids of the appropriate areas, for all superstructures shall be derived from

$$W_v = qA_3CL$$

where

q	is as defined above (all superstructures with solid elevation)
A_3	is the area in plan (in m²)
C_L	is the lift coefficient as derived from Figure 3.11 for superstructures where the angle of super-elevation is less than 1°.

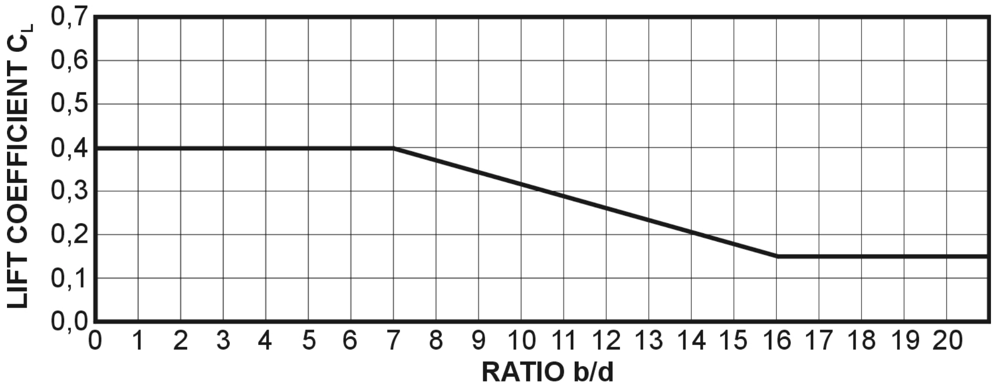


Figure 3.11: Lift coefficient CL

Where the angle of super-elevation of a superstructure exceeds 1° and 5°C, C_L shall be taken as ± 0.75.

Where the angle of super-elevation of a superstructure exceeds 5°, the value of C_L shall be determined by testing.

Where inclined wind may affect the structure, C_L shall be taken as ± 0.75 for wind inclinations up to 5°. The angle of inclination in these circumstances shall be taken as the sum of the angle of inclination of the wind and that of the super-elevation of the bridge. The effects of wind inclinations in excess of 5° shall be investigated by testing.

3.8.4 Dynamic Wind Action Assessment: Method C

In the case of long slender wind-sensitive structures such as suspension and cable-stayed bridges, reference should be made to the texts listed in Section 6. It may be necessary to do a dynamic analysis based on the statistical approach to determine the wind forces and the aero-elastic structural response. The possibility of aerodynamic instability as a result of wind-excited oscillations should be investigated. Special wind tunnel tests or other experimental methods may be required to supplement the theoretical analyses.

3.8.5 Associated Nominal Primary Live Load

For the effect of wind on highway live loading, the live loading shall consist of NA loading in such positions as to produce the most severe effects.

3.8.6 Force Combinations

3.8.6.1 Method A

For Method A the wind forces on the structure and live loading shall be considered in combination with the other loads in Combinations (2) or (3), as appropriate.

3.8.6.2 Method B

For Method B the wind forces W_t , W_L and W_v shall be considered in combination with the other loads in Combinations (2) and (3) indicated in Section 3.8.7, taking four separate cases:

(a) W_1 alone
(b) W_t in combination with ± W_v

(c) W_L alone
(d) 0.5 W_t in combination with $W_L \pm 0.5\ W_v$

3.8.7 Design Forces

For the design wind forces, γ_{fL} shall be taken as follows (Table 3.14):

Table 3.14: γ_{fL} for wind forces

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
(a) With erection loads	-	1.1	-	-	1.0	-
(b) With permanent principal actions only	-	-	1.4	-	-	1.0
(c) With permanent and transient principal actions	-	1.1	-	-	1.0	-
(d) Relieving effects of wind	-	1.0	1.0	-	1.0	1.0

3.8.8 Overturning Effects

Where overturning effects are being investigated, the wind load shall also be considered in combination with vertical traffic live load. Where the vertical traffic live load has a relieving effect, this load shall be limited to one notional lane only and shall be not more than 6 kN/m of bridge.

Load factor relieving vertical live load: For a live load producing a relieving effect γ_{fL} for both ultimate limit states and serviceability limit states shall be taken as 1.0.

3.9 Flood Action

3.9.1 General

The action of flowing water on bridge structures over rivers or estuaries shall be derived in accordance with the principles of modern theory of hydraulics based on the best data available for the site. The design flood shall be determined in consultation with the responsible authority preferably based on sensitivity studies and risk analysis and the partial load factors γ_{fL} given in Section 3.9.3 adjusted accordingly.

3.9.2 Flood Forces

3.9.2.1 Method of Calculation

The following approximate method will suffice for most bridge structures, but the engineer shall, in conjunction with the responsible authority, use his discretion to decide whether further analysis or experimentation is required.

3.9.2.2 Approximate Method

For bridge piers parallel to the direction of stream flow, or within 5° thereof, the force F shall be calculated from the equation:

$$F = KA_4\, v^2 \text{ kN}$$

where

K	=	a coefficient, being 0.7 per square pier ends, and 0.35 for circular and angle ends where the angle is 30° or less.
A_4	=	projected pier area in m²
v	=	velocity of flow in m/s.

Where the angle of attack exceeds 5°, the equation furnished may underestimate the force considerably and a more accurate approach shall be made.

Where the river carries debris, the effect of this shall be taken as an additional horizontal force on each pier. The force shall act at flood level elevation and shall be selected between the values of 90 and 180 kN depending on the anticipated degree of debris build-up. Allowance shall be made for the effects of overtopping where this is probable.

Due allowance shall be made for buoyancy and the effects of scour of the river bed.

3.9.3 Design Forces

For the design forces resulting from flood action, γ_{fL} shall be taken as follows (Table 3.15):

Table 3.15: γ_{fL} for forces resulting from flood action

γ_{fL}	Ultimate Limit State			Serviceability Limit State		
Combinations	1	2	3	1	2	3
	-	1.05	1.3	-	0.85	1.0

3.10 Earthquake Action

3.10.1 General

The probability of the occurrence of earthquakes of a particular magnitude varies considerably within the SADC region. This Code allows the application of four different methods for the evaluation of earthquake effects, but sets a minimum requirement that shall be applicable to all bridges (Method A). The applicability of a particular method will depend on the expected seismic intensity level and on the susceptibility or vulnerability of the bridge to earthquake actions. The classifications given in Table 3.16 may be used as an approximate guide. The appropriate authority shall be consulted in the decision as to the method and the relevant parameters that are to be used for the design of a specific structure.

Table 3.16: Modified Mercalli classification

Modified Mercalli Intensity at epicentre (MM)	Maximum ground acceleration (A) at epicentre (g)
ii - iii	0.003
iv - v	0.01
vi	0.03
vii - viii	0.1
ix	0.3
x - xi	1.0

Methods B and C require an estimation of the expected intensity levels, expressed in accordance with Modified Mercalli Scale (MM) classification, that have a 90 per cent probability of not being exceeded in 100 years. Data on earthquake intensity in southern Africa is not readily available. A seismic intensity map, based on a preliminary map drawn up by the Geophysics Division of the South African Department of Mines using a statistical analysis of the limited amount of earthquake action is shown in Figure 3.12. The maximum horizontal ground accelerations corresponding to the MM classification are given in Table 3.16. The map is, of necessity, approximate and can only be improved if and when new earthquake data become available. The expected peak horizontal ground accelerations are to be determined by interpolation between the boundary lines.

The structural response of a bridge structure depends not only on the seismic characteristics of any particular earthquake, but also on the natural frequencies of the structure. The nature and location of bridge bearings can also have a critical effect on the structural response. The accurate determination of earthquake effects can therefore be highly complex. Reference should be made to suitable texts on the nature and effects of earthquakes, some of which are listed in Section 6[9-12].

Figure 3.12: Earthquake intensity zones in terms of modified Mercalli scale

3.10.2 Assessment of Earthquake Effects: Method A

This method represents the minimum requirement for any bridge structure. It is a static method which does not accurately simulate the dynamic effects of an earthquake, but may be used to ensure that the bridge has a nominal capability of resisting earthquake effects.

<u>3.10.2.1 Nominal Earthquake Forces</u>

A total nominal horizontal force F_x to be applied to the portion m_x of the bridge structure at a height h_x above the base of pile cap, can be represented by the following equation:

$$F_{eq} = k_f \left[\sum_{i=1}^{n} g_{di} + \sum_{j=1}^{n} g_{sdi} \right]$$

shall be applied to the structure or a part of the structure

where

k_f	=	0.02 for structures founded on rock or firm subsoil rated at not less than 400 kPa bearing capacity
	=	0.04 for structures founded on material rated between 400 kPa and 100 kPa
	=	0.06 for structures founded on piles in very soft upper layers or on material rated below 100 kPa.
g_{di}	=	dead load of the portion "i" of the structure
g_{sdj}	=	superimposed dead load of the portion "j" of the structure

The horizontal force caused by a portion of the superstructure shall be applied at the height of the centre of gravity thereof and be distributed along its length in terms of the distribution of its mass. The resultant horizontal forces from the substructure shall be applied at a point two-thirds above the base or in accordance with a triangular distribution, i.e. increasing linearly in the vertical direction. These forces shall be applied separately in the transverse and the longitudinal directions.

The effects of bridge bearings and the forces thereon shall be given due consideration.

The effects of transient live loads and short-term and supplementary actions need not be considered in combination.

3.10.2.2 Partial Safety Factors

Partial safety factors for the design actions are given in Section 3.10.6.

3.10.3 Assessment of Earthquake Effects: Method B

Method B is an approximate equivalent static method applicable to a range of bridge structures varying from those with a low susceptibility to earthquake actions and situated in zones (see Figure 3.12) with a 90 per cent probability that Class viii seismic intensity, in accordance with the Modified Mercalli Scale defined in Table 3.16, will not be exceeded in 100 years, to those structures that are vulnerable and situated in zones in which Class vi seismic activity will not be exceeded. Structures with a low susceptibility to earthquake action are those with periods shorter than 0.05 seconds and longer than 3.0 seconds.

The basis of this approach, which is a modification of one of the methods in the National Building Code of Canada 1980[8], is the assessment of the equivalent static horizontal forces which, when applied to the structure, would induce stress effects of a magnitude similar to those that would be induced by a real earthquake. The equivalent horizontal forces are assumed to act separately in the direction of the transverse and longitudinal axes of the bridge.

For the purpose of determining these forces, the bridge is subdivided into portions in such a manner as to give a realistic distribution of the forces. Horizontally the forces are assumed to be distributed in accordance with the distribution of the relevant masses, or to be concentrated at a point in vertical alignment with the centre of gravity of the relevant mass. Vertically the distribution shall be determined as defined in Section 3.10.3.1.

3.10.3.1 Nominal Earthquake Forces

The equivalent horizontal static earthquake force F_x to be applied to the portion m_x of the bridge structure at a height h_x above the base or pile cap, can be represented by the following equation:

$$F_x = .S.A.\xi.I.f.m_x.h_x \left[\frac{\sum_{i=1}^{n} m_i}{\sum_{i=1}^{n} m_i\ h_i} \right]$$

where

A	=	horizontal ground acceleration (see Table 3.16)
S	=	the seismic response factor for the structure which shall be equal to $0.5/\sqrt{T}$ but need not exceed 1.0

where

T	=	fundamental period of vibration of the structure in seconds in the direction under consideration as defined below. Except for structures that are vulnerable or complex, the fundamental period T may be determined according to the approximate method in Appendix D.
ξ	=	numerical factor that reflects the structural configuration and the mechanical properties of the material including damping, ductility and/or energy-absorptive capacity of the structure as given in Table 3.17.
I	=	importance factor of the structure which shall vary from 1.0 to 1.3 depending on the importance of the structure.
f	=	foundation factors which are given in Table 3.18. The product 1.3 need not exceed 1.0.
$\sum_{i=1}^{n} m_i$	=	mass of the dead load plus the mass of the superimposed dead load of the structure or part of the structure being considered, conveniently subdivided into n portions.
h_x	=	the height above base level of the portion m_x of the structure being considered.
i	=	0; refers to base level.
i	=	1....n refers to the relative levels of the n portions of the structure considered.

The equivalent static analysis can now be performed by loading the structure with the forces F_x. The effects of bridge bearings and the forces thereon shall be given due consideration.

Table 3.17: Numerical factors reflecting structural configuration and mechanical properties

Case	Type of arrangement of resisting elements	Value of ξ (Method B)	Structural ductility factor μ (Method C)
1	Structural steel unbraced bridges, piers and superstructures adequately designed to resist the total lateral forces by bending of the members in accordance with their relative rigidities considering the interaction of the various parts.	0.8	4
2	Structures with braced flexural piers of structural steel, or substructures consisting of slender reinforced concrete columns and superstructures in braced structural steel, or in reinforced or prestressed concrete, adequately designed to resist the total lateral force in accordance with their relative rigidities considering the interaction of the various parts.	1.0	3
3	Structures with piers or abutments of shear wall proportions and superstructures in monolithic reinforced or prestressed concrete adequately designed to resist the total lateral force in accordance with their relative rigidities considering the interaction of the various parts.	1.3	2

Table 3.18: Foundation factors affecting nominal earthquake forces

Type and depth of soil	f
Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils; compact coarse-grained soils, and firm and stiff fine-grained soils from 0 to 15 m deep.	1.0
Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15 m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15 m deep	1.3
Very loose and loose coarse-grained soils and very soft and soft fine-grained soils with depths greater than 15 m	1.5

3.10.3.2 Partial Safety Factors

Partial safety factors for the design actions are given in Section 3.10.6.

3.10.4 Assessment of Earthquake Effects: Method C

This method of analysis, based on the National Building Code of Canada, 1980[8], is a quasi-dynamic simulation of the response of single-degree-of-freedom structures to the base excitation caused by earthquakes. It is applicable to a range of bridge structures varying from those with a low susceptibility to earthquake actions and situated in zones (see Figure 3.12) with a 90 per cent probability that Class viii seismic intensity, in accordance with the Modified Mercalli Scale defined in Table 3.16, may be exceeded in 100 years, to those structures that are vulnerable and situated in zones in which Class vi seismic activity may be exceeded. Structures with a low susceptibility to earthquake action are those with periods shorter than 0.05 seconds and longer than 3.0 seconds. This method can be used as a manual method on small structures or as a computerized method on larger structures. In a multi-degree-of-freedom system that has independent or uncoupled modes of deformation, each mode responds to the base motion or excitation as an independent single-degree-of-freedom system. Generally, only the response in the fundamental or lowest natural response frequency is of interest to the designer.

For the purpose of seismic design of strictures, an average response spectrum approach is recommended here. The properties of the average response spectrum were derived so that its general shape matches the spectra of a large number of recorded earthquakes. The recommended average response spectrum agrees particularly well with the response spectra of these recorded earthquakes in the short-period range, but is not as satisfactory in the long-period range. This variation can, in part, be attributed to the presence of different amounts of surface wave energy in the earthquakes that were recorded. The recommended average spectrum bound in the long-period range is, therefore, chosen conservatively.

The recommended average response spectrum is calculated from the peak ground-motion bounds. The peak ground-motion bounds in turn are linked to the seismic risk level by way of the peak horizontal ground acceleration that can be expected. Values of A that correspond with the various seismic intensities shown in Figure 3.12 are given in Table 3.16. The intensity levels shown have a 90 per cent probability of not being exceeded in 100 years.

3.10.4.1 Design Method

The peak ground-motion bounds and the elastic average response spectrum shown in Figure 3.13 are normalized to a ground acceleration of 1.0 g. For a single-degree-of-freedom structure the design spectrum is entered with the natural period of the structure. The maximum acceleration S_a (in units of gravitational acceleration) is then read off at the respective damping ratio, which is defined as the ratio of the damping coefficient to the critical damping coefficient for a single-degree-of-freedom oscillator or a normal mode. Critical damping means the minimum value of damping that will allow a displaced oscillator to return to its initial position without oscillation. Typical damping ratios are listed in Table 3.19. For intermediate values of damping, linear interpolation may be used for response obtained from the design spectrum. The required response modification due to elastic-plastic behaviour is explained in Section 3.10.4.2. If the elastic average response spectrum is applied without modification, the results will generally be two to six times higher than those obtained from Method B. This is due to the fact that Method B is based on field observations in which elastic-plastic behaviour has occurred.

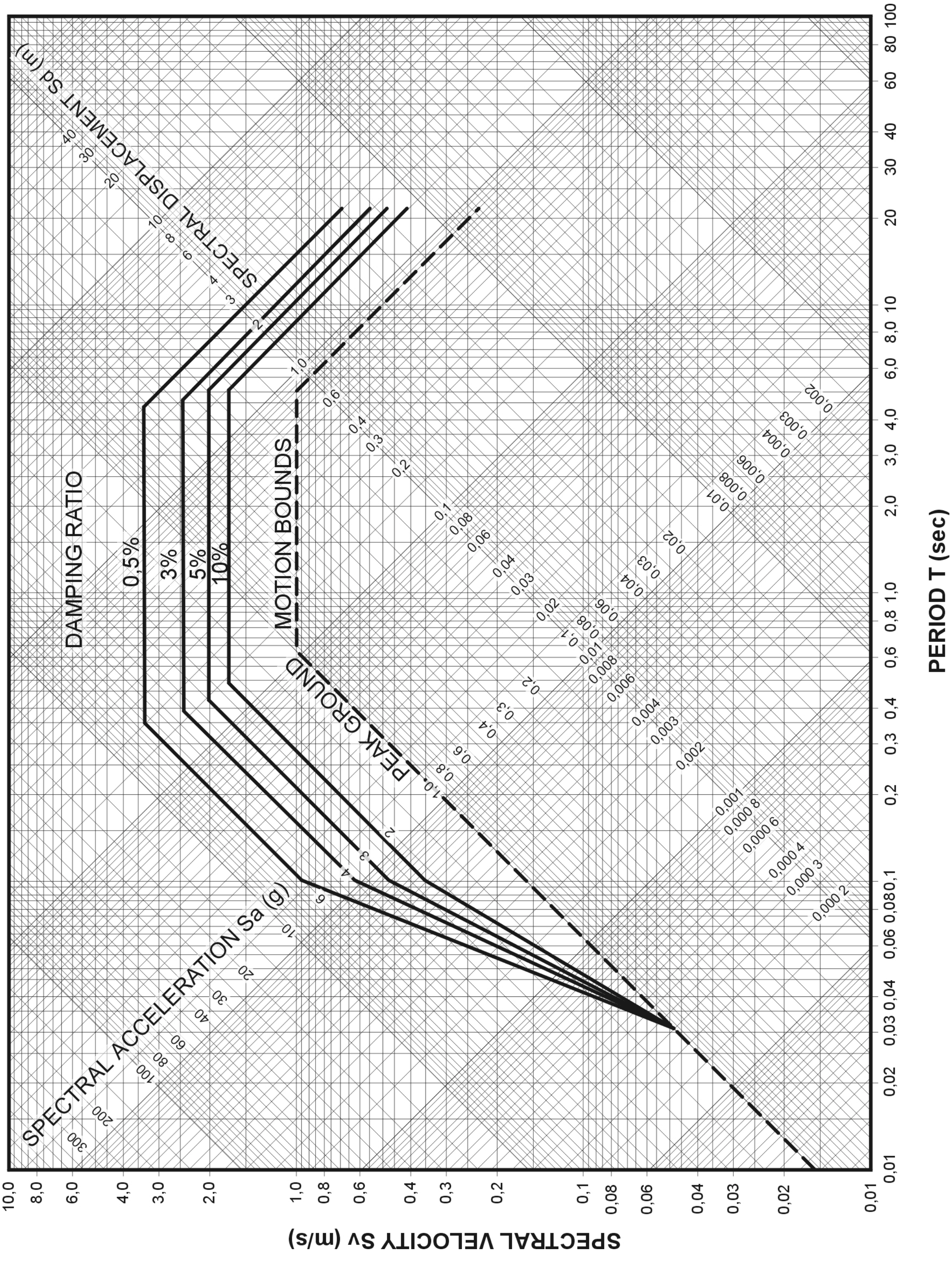


Figure 3.13: Peak ground motion bounds and average elastic response spectrum

The equivalent static load to which the structure is subjected can be calculated from:

$F_x = A.\ell.f.S_a.m_x$

where

m_x = the mass of the structure or part of the structure considered.

The above parameters, A, ℓ and f, relating to the structure and its location have been defined in Section 3.10.3.

In this way the dynamic earthquake loading is effectively reduced to an equivalent static loading.

A similar procedure is followed for multi-degree-of-freedom structures as described in Section 3.10.4.3.

Table 3.19: Typical damping ratios

Stress level	Type and condition of structure	Damping ratio (λ)
Working stress no more than about half yield	Welded steel, prestressed concrete, well-reinforced concrete (only slight cracking).	0.02
	Reinforced concrete with considerable cracking	0.03 to 0.05
	Bolted and/or rivetted steel	0.05 to 0.07
At or just below yield point	Welded steel, prestressed concrete under full prestress (slight cracking)	0.05
	Prestressed concrete in cracked state	0.07
	Reinforced concrete	0.07 to 0.10
	Bolted and/or rivetted steel	0.10 to 0.15

3.10.4.2 Response Modification Due to Elastic-plastic Behaviour

For a structure designed to undergo elastic-plastic deformation under earthquake action, the average elastic response spectrum may be modified as follows in order to obtain the inelastic design spectrum.

The elastic spectrum, for any given damping ratio, is modified along the displacement bound region by multiplying the bound line by a factor $1/\mu$, and along the acceleration bound region by multiplying the bound line by a factor:

$$\frac{1}{\sqrt{2\mu - 1}}$$

A typical conversion is shown in Figure 3.14. Note that the period at the corner points is unchanged. The ductility factor μ of the structure is defined as the

$$\frac{\text{total elastic-plastic deformation}}{\text{total elastic deformation at yield}}$$

Table 3.17 gives values of μ for typical structural configurations.

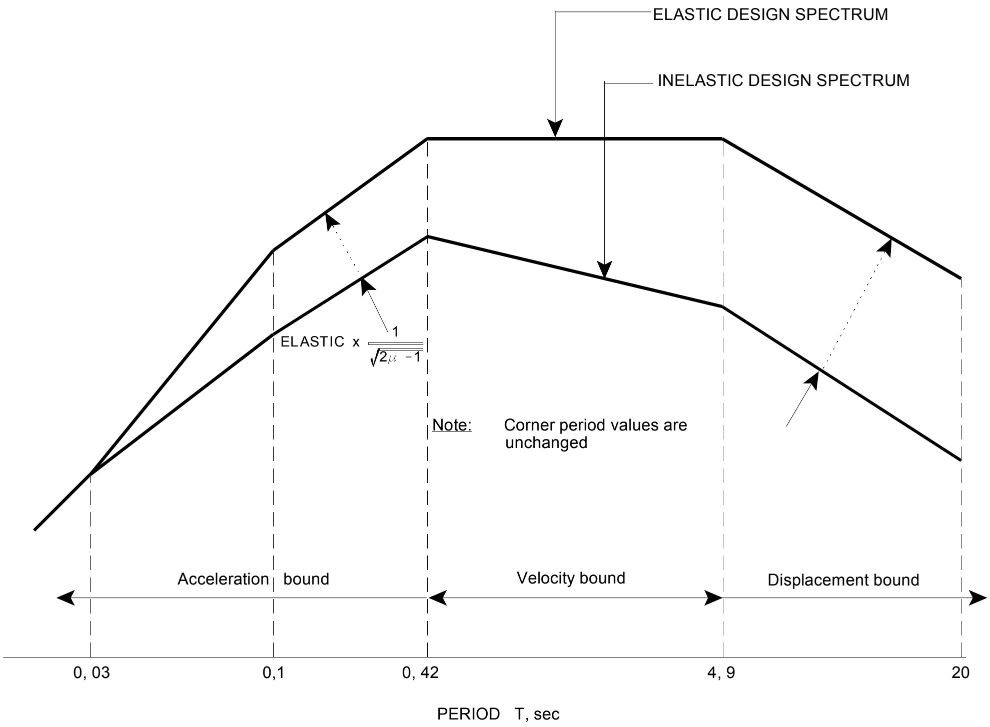


Figure 3.14: Transformation from elastic to elastic - plastic design spectrum

The maximum acceleration S_a is then read off at the modified spectrum bound and used in the equation F_x - A.ℓf.S_a.m_x as before. The equivalent static analysis can be performed by loading the structure with the forces F_x. The sectional forces obtained are the true elastic-plastic response of the structure. The elastic-plastic displacements can be obtained from the analysis by multiplying the displacements by the ductility factor μ.

The maximum structural ductility factor μ given in Table 3.17 must be used, except where tests and calculations demonstrate that higher values are justified.

For structural ductility factors μ equal to or greater than five, this recommended procedure should not be used, but a time series analysis with realistic material behaviour should be performed (refer to Method D). It should be noted that in order to achieve a given structural ductility, the ductility factors of some individual members, particularly those of girders, may need to be greater than the overall structural ductility.

3.10.4.3 Multi-degree-of-Freedom Structures

General requirements: The preparation and application of the design spectrum proceeds in the same way as described in Sections 3.10.4.1 and 3.10.4.2. The general provisions of 3.10.3 also apply.

For symmetrical structures the influence of seismic disturbances is to be considered along both principal axes of symmetry. For non-symmetrical structures, seismic effects may be considered along two arbitrarily chosen orthogonal axes. For both symmetrical and non-symmetrical structures, the seismic effects along the two orthogonal axes may be considered independently of one another. Where it is deemed necessary to consider the influence of vertical seismic motions, the average vertical design spectrum may be taken as two-thirds the average horizontal seismic response spectrum (see Figure 3.13). It should be noted that the relevant resonance frequencies of the structure and its components in the vertical direction are generally quite different from those in the horizontal direction. The response caused by vertical excitation shall be combined with the response caused by horizontal excitation as specified below (computation of structural response).

The structure is first analysed to determine its natural modes of vibration and the corresponding natural frequencies (or periods). This generally involves the solution of an eigenvalue formulation of the structure of the form

$$\left[[m] - \left[\frac{1}{\omega^2} \right] [K] \right] \{x\} = 0$$

where

$[m]$	=	diagonal matrix of lumped masses representative of the dead and superimposed dead load imposed on the structure
$[K]$	=	stiffness matrix of the structure
$\{x\}$	=	general displacement vector of all degrees of freedom of the structure
$\left[\frac{1}{\omega^2} \right]$	=	the inverse square vector of the natural radial frequencies ω.

Computation of structural response: Each mode is first assumed to behave independently in the earthquake response. The structural response acceleration S_{ai} (in units of gravitational acceleration) can thus be evaluated independently for each mode i with period T_i in the way shown in Sections 3.10.4.1 and 3.10.4.2. Each mode will participate in the total combined response by an amount $\gamma_i \Phi_{ij} S_{ai}$. The "modal participation factor" γ_i is evaluated for the ith mode as follows:

$$\gamma_i = \frac{\Sigma m_i \varphi_{ij}}{\Sigma m_j (\varphi_{ij})^2}$$

where

$$\Sigma \equiv \sum_{j=1}^{n}$$

m_j	=	lumped mass free to move in the direction of the degree of freedom j
m_j	=	m_j when the direction of the ground motion is parallel to the direction of the degree of freedom j, otherwise
m_j	=	0
Φ_{ij}	=	normalized mode component of the ith mode in the direction of the jth degree of freedom.
n	=	number of degrees of freedom of the structure.

The generalized set of equivalent static forces acting on the structure can thus be written as:

$$F_{ij} = A.\ell.f.\gamma_i \Phi_{ij} S_{ai} m_j$$

For a schematic diagram of forces acting on a bridge structure, see Figure 3.15.

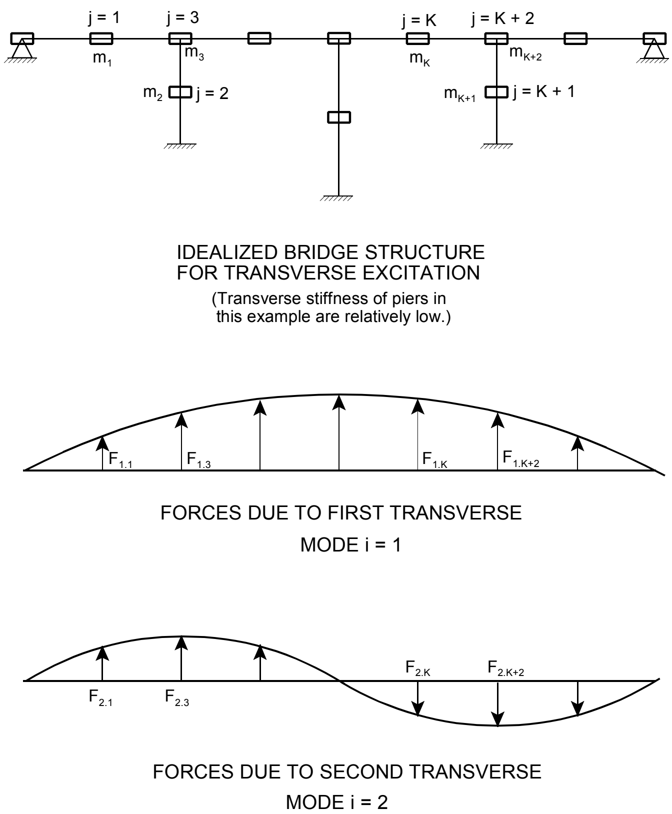


Figure 3.15: Forces acting on a bridge

Combination of modes: By the process described in Section 3.10.4.2, the maximum response of each mode of the multi-degree-of-freedom structure can be found. The maximum values of combined effects are obtained by taking the square root of the sum of the squares of the effects from each mode. The first three natural modes of vibration of the structure are normally sufficient to obtain maximum combined effects.

Partial safety factors for the design actions are given in Section 3.10.6.

3.10.5 Assessment of Earthquake Effects: Method D

Method D is a more rigorous dynamic analysis applicable to exceptional or major structures that are very vulnerable to earthquake effects. This method requires a dynamic analysis of the structure subjected to typical accelerograms of recorded earthquakes scaled down for the relevant zone. Due to the lack of recordings of strong-motion earthquakes in southern Africa, a set of accelerograms of earthquakes recorded on other continents, such as El Centro, 18th

May 1940 in California; Pacoima Dam, 9th February 1971 in San Fernando, California; Taft, 21st July 1952 in California; Aomori, 1968 in Japan; Ocurido, 23rd October 1972 in Managua, Nicaragua, suitably scaled down is recommended[11]. A thorough understanding of dynamic analysis and structural response to earthquakes is required before this method is attempted. More than one possible ground motion (accelerogram) should be used to reduce the likelihood of incorporating into the design a particular bias that may be inherent in any one excitation characteristic. Use of a suitable computer program should be resorted to in order to perform either a mode superposition analysis or a direct integration analysis on the structure, or to perform other acceptable forms of dynamic analysis.

The design accelerograms referred to above have to be scaled to the required peak horizontal ground accelerations applicable to the seismic zone in which the structure is located. The structure to be analysed must be broken up into a sufficient number of members so that the mass can be realistically allocated to the respective internodes. The parameters given for Methods B and C may be used where relevant.

3.10.6 Design Actions

For the design forces resulting from earthquakes and acting in combination (3) only, γ_{fL} shall be taken as follows (Table 3.20):

Table 3.20: γ_{fL} for earthquakes

γ_{fL}	**Ultimate Limit State**			**Serviceability Limit State**		
Combinations	1	2	3	1	2	3
	-	-	1.3	-	-	1.0

4. RESTRAINT ACTIONS

4.1 General

The effects considered above all result from body forces or externally applied actions. Restraint actions, on the other hand, are caused by induced dimensional changes or deformations which generate reactions in constrained structural members. The stress effects caused by these induced deformations depend on the stiffness of the relevant structural members and the rigidity of the restraints. Because of the non-linear behaviour of concrete as the ultimate limit state is approached, and because of resultant reduction in the effective stiffness of the relevant members, the restraint effects constitute a larger portion of the stress effects for a serviceability limit state condition than for an ultimate limit state condition. Since this Code recommends that analysis should in general be performed for service actions imposed on structures consisting of members whose properties are evaluated on the basis of elastic material behaviour, a large over-estimation of restraint action effects may be made in the case of certain ultimate combinations. An example of this is the general practice of evaluating equivalent loads due to cross-sectional temperature gradients on the basis of elastic member behaviour and to use the resultant restraint actions developed by the structure (as obtained from an elastic analysis) in an ultimate load combination. However, until the state of the art of structural behaviour has advanced sufficiently to give a more realistic assessment of restraint actions under ultimate conditions, it is proposed that conservative ultimate load factors be applied to the restraint effect portion of ultimate effect combinations.

4.2 Creep and Shrinkage Effects, Residual Stresses, etc.

4.2.1 General

Where it is necessary to take into account the effects of shrinkage or creep in concrete, requirements are specified in Appendix E. The requirements for stresses in steel caused by rolling, welding, or lack of fit, variations in the accuracy of bearing levels and similar sources of strain arising from the nature of the material or its manufacture or from circumstances associated with fabrication and erection, should be obtained from other approved sources as indicated in Section 2 of Part 1.

4.2.2 Design Actions of Creep and Shrinkage etc.

For all combinations, γ_{fL} shall be taken as 1.0 for both the ultimate and serviceability limit states.

4.3 Secondary (Parasitic) Prestress and Prestrain Effects

4.3.1 General

Structural members that are prestressed shorten and bend if unrestrained. Statically indeterminate structures accordingly generate reactions at points of restraint if the cables are not concordant, which in turn impose additional stresses along the members of the structure. These stresses are defined as the parasitic prestress effects which must be allowed for in the design calculation. Prestraining any member of a structure will also impose stresses.

4.3.2 Design Effects of Parasitic Prestress and Prestrain Effects

For all combinations, γ_{fL} shall be taken as 1.0 for both the ultimate and serviceability limit states.

4.4 Differential Settlement Effects

4.4.1 General

Where differential settlement of the foundations is likely to affect the structure in whole or in part, the effects of this shall be taken into account. The magnitude of likely settlement shall be assessed as accurately as practicable in accordance with the principles of soil mechanics based on such relevant information about the nature of the founding media as can economically be obtained. In assessing the amount of differential movement to be provided for, the engineer shall take into account the probability of its effect being observed and remedied before damage ensues and the fact that a partial safety factor of 1.0 is applied. The effects of differential settlement on the structure will depend on the time-dependent nature of the settlement and on the elasto-plastic response of the structure with time.

4.4.2 Design Actions of Differential Settlement

For all combinations, γ_{fL} shall be taken as 1.0 for both the ultimate limit state and the serviceability limit state.

4.5 Temperature Effects

4.5.1 General

Daily and seasonal fluctuations in shade air temperature, solar radiation, re-radiation, etc, cause the following:

(a) Changes in the overall temperature of the bridge, referred to as the effective bridge temperature. Over a prescribed period, there will be a minimum and a maximum, together with a range of effective bridge temperatures, resulting in actions and/or action effects within the bridge due to:

   (i) restraint of associated expansion or contraction by the form of construction (e.g. portal frame, arch, flexible pier, elastomeric bearings) referred to as temperature restraint; and

   (ii) friction at roller or sliding bearings where the form of the structure permits associated expansion and contraction, referred to as frictional bearing restraint.

(b) Differences in temperature between the top surface and other levels through the depth of the superstructure, referred to as temperature difference or gradient and resulting in associated actions and/or action effects within the structure.

Effective bridge temperatures are derived from the isotherms of shade air temperature and the available data from Member States are shown in Figures 4.1 and 4.2. These figures will be updated as and when further data becomes available. In cases where data is not presented, maximum and minimum shade air temperatures should be obtained from appropriate local services. These shade air temperatures are appropriate to mean sea-level in open country and a fifty-year period.

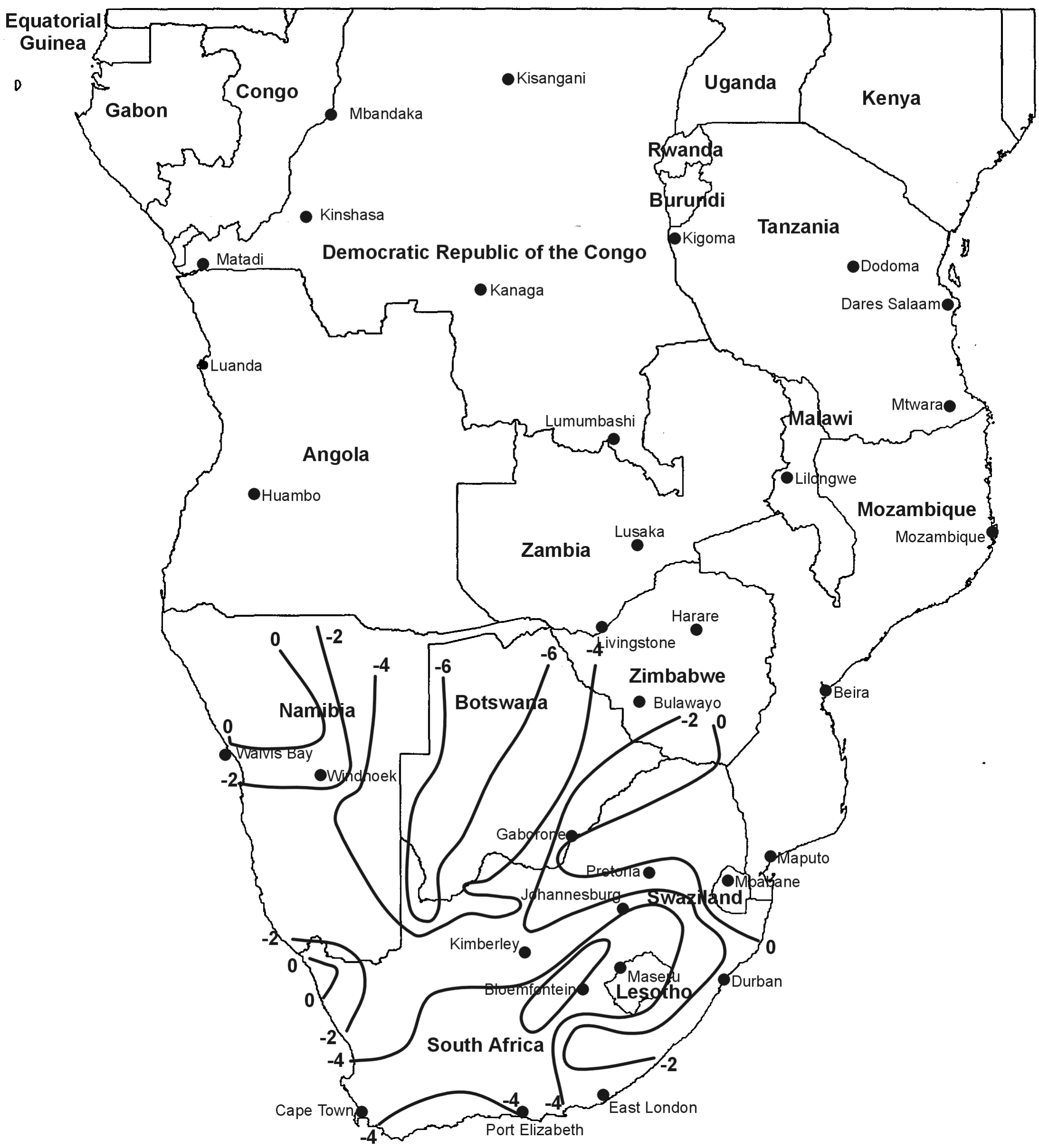


Figure 4.1: Isotherms of minimum shade temperature (in °C) for 50 year return period

Note: Isotherms in Figures 4.1 and 4.2 are to be adjusted for height above mean sea level.

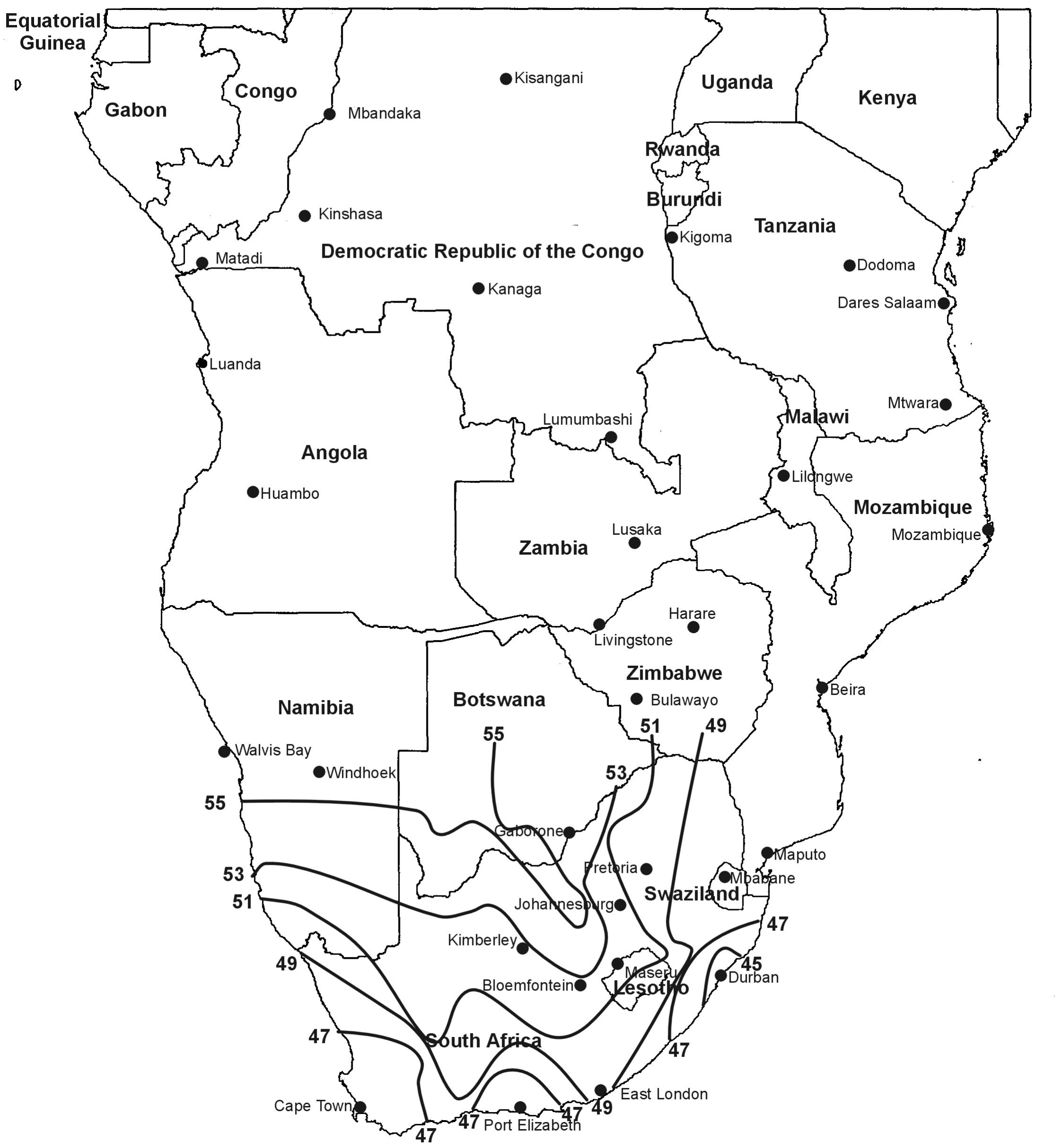


Figure 4.2: Isotherms of minimum shade temperature (in °C) for 50 year return period

4.5.2 Minimum and Maximum Shade Air Temperatures

For all bridges, extremes of shade air temperature for the location of the bridge shall be obtained from the maps of isotherms for a 50-year return period shown in Figures 4.1 and 4.2 except where the appropriate authority may prescribe otherwise. These values shall be adjusted for height above mean sea-level as described in Section 4.5.2.1.

Where a particular erection will be completed within a period of one or two days for which reliable shade air temperature and temperature range predictions can be made, these may be adopted.

Carriageway joints and similar equipment that will be replaced during the life of the structure may also be designed for temperatures related to the relevant shorter return periods.

4.5.2.1 Adjustment for Height Above Mean Sea-level

The values of shade air temperature shall be adjusted for height above sea-level by subtracting 0.5°C per 100 m height for minimum shade air temperatures and 1°C per 100 m height for maximum shade air temperatures.

4.5.2.2 Divergence from Minimum Shade Air Temperature

There are locations where the minimum values diverge from the values given in Figure 4.1, as, for example, in frost pockets and sheltered low-lying areas where the minimum may be higher than that indicated. These divergences shall be taken into consideration.

4.5.3 Minimum and Maximum Effective Bridge Temperatures

The minimum and maximum effective bridge temperatures for different types of construction shall be derived from the minimum and maximum shade air temperatures by reference to Tables 4.1 and 4.2 respectively, except where the appropriate authority may prescribe otherwise. The different types of construction are as shown in Figure 4.3.

Table 4.1: Minimum effective bridge temperature

Minimum shade air temperature (°C)	Minimum effective bridge temperature (°C)		
	Type of superstructure		
	Group 1 and 2	Group 3	Group 4
-15	-18	-13	- 9
-14	-17	-12	- 9
-13	-16	-11	- 8
-12	-15	-10	- 7
-11	-14	-10	- 6
-10	-12	- 9	- 6
- 9	-11	- 8	- 5
- 8	-10	- 7	- 4
- 7	- 9	- 6	- 3
- 6	- 8	- 5	- 3
- 5	- 7	- 4	- 2
- 4	- 6	- 3	- 1
- 3	- 5	- 2	0
- 2	- 4	- 1	0
- 1	- 3	- 1	1
- 0	- 2	0	1

Table 4.2: Maximum effective bridge temperature

Maximum shade air temperature (°C)	Maximum effective bridge temperature (°C)		
	Type of superstructure		
	Group 1 and 2	Group 3	Group 4
30	44	36	32
31	44	36	32
32	44	37	33
33	45	37	33
34	45	38	34
35	46	39	35
36	46	39	36
37	46	40	36
38	47	40	37
39	47	41	38
40	48	42	38
41	48	42	39
42	48	43	39
43	49	43	40
44	49	44	40
45	49	44	41
46	50	45	41
47	50	45	42
48	50	46	42
49	51	46	43
50	51	46	44
51	51	47	44
52	52	47	45
53	52	48	45
54	52	48	46
55	52	48	46
56	53	49	47
57	53	49	47
58	53	49	47
59	53	50	48
60	53	50	48

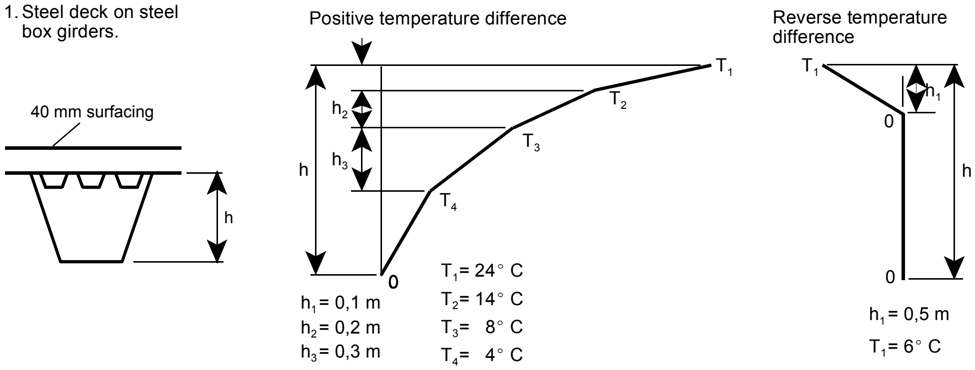
4.5.3.1 Adjustment for Thickness of Surfacing

The effective bridge temperatures are dependent on the depth of surfacing on the bridge deck, and the values given in Tables 4.1 and 4.2 assume depths of 40 mm for groups 1 and 2 and 100 mm for groups 3 and 4. Where the depth of surfacing differs from these values, the minimum and maximum effective bridge temperatures may be adjusted by the amounts given in Table 4.3.

4.5.4 Range of Effective Bridge Temperatures

In determining action effects due to temperature restraint, the effective bridge temperature at the time the structure is effectively restrained shall be taken as the datum in calculating expansion up to the maximum effective bridge temperature, and contraction down to the minimum effective bridge temperature.

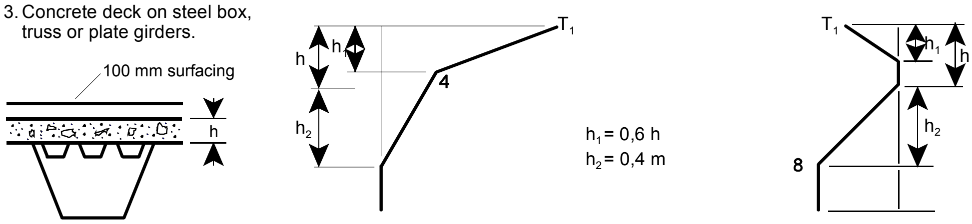
1. Steel deck on steel box girders.

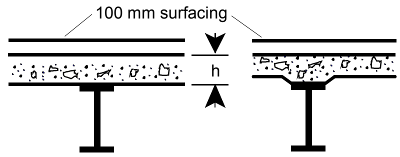


2. Steel deck on steel truss or plate girders.

Use differences as for group 1.

3. Concrete deck on steel box, truss or plate girders.

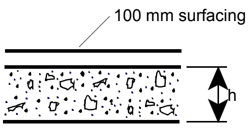


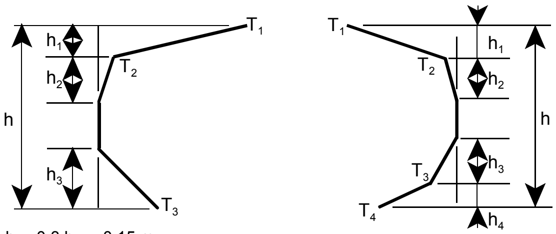


h	T_1
m	°C
0,2	13
0,3	16

h	T_1
m	°C
0,2	3,5
0,3	5,0

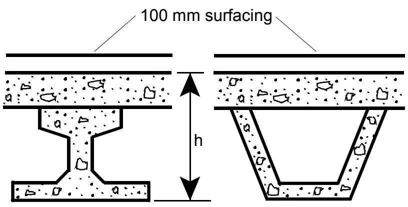
4. Concrete slab or concrete deck on concrete beams or box girders.





$h_1 = 0,3\,h \le 0,15$ m
$h_2 = 0,3\,h \ge 0,10$ m
$\ge 0,25$ m
$h_3 = 0,3\,h \le$ (0,1 m + surfacing depth in metres)
(for thin slabs, h_3 is limited by $h - h_1 - h_2$)

$h_1 = h_4 = 0,2\,h \le 0,25$ m
$h_2 = h_3 = 0,25\,h \le 0,2$ m



h	T_1	T_2	T_3
m	°C		
≤ 0,2	8,5	3,5	0,5
0,4	12,0	3,0	1,5
0,6	13,0	3,0	2,0
≥ 0,8	13,5	3,0	2,5

h	T_1	T_2	T_3	T_4
m	°C			
≤ 0,2	2,0	0,5	0,5	1,5
0,4	4,5	1,4	1,0	3,5
0,6	6,5	1,8	1,5	5,0
0,8	7,6	1,7	1,5	6,0
1,0	8,0	1,5	1,5	6,3
≥ 1,5	8,4	0,5	1,0	6,5

Figure 4.3: Temperature difference for different types of construction

Table 4.3: Adjustment to effective bridge temperature for deck surfacing

Deck Surface	Addition to minimum effective bridge temperature (°C)			Addition to maximum effective bridge temperature (°C)		
	Groups 1 & 2	Group 3	Group 4	Groups 1 & 2	Group 3	Group 4
Unsurfaced	0	- 3	- 1	+ 4	0	0
Waterproofed	0	- 3	- 1	-	+ 4	+ 2
40 mm surfacing *	0	- 2	- 1	0	+ 2	+ 1
100 mm surfacing *	-	0	0	-	0	0
200 mm surfacing *	-	+ 3	+ 1	-	- 4	- 2
* Surfacing depths include waterproofing						

4.5.5 Temperature Difference or Gradient

Effects of temperature differences within the superstructure shall be derived from Section 4.5.5.3.

Positive temperature differences occur when conditions are such that solar radiation and other effects cause a gain in heat through the top surface of the superstructure. Conversely, reverse temperature differences occur when conditions are such that heat is lost from the top surface of the bridge deck as a result of re-radiation and other effects.

4.5.5.1 Adjustment for Thickness of Surfacing

Temperature differences are sensitive to the thickness of surfacing, and the data given in Figure 4.3 assume depths of 40 mm for groups 1 and 2 and 100 mm for groups 3 and 4. For other depths of surfacing different values will apply. Values for other thicknesses of surfacing are given in Section 4.5.5.3.

4.5.5.2 Combination with Effective Bridge Temperatures

Maximum positive temperature differences shall be considered to coexist with effective bridge temperatures at above 25°C (groups 1 and 2) and 15°C (groups 3 and 4). Maximum reversed temperature differences shall be considered to coexist with effective bridge temperatures up to 8°C below the maximum for groups 1 and 2, up to 4°C below the maximum for group 3, and up to 2°C below the maximum for group 4.

4.5.5.3 Temperature Differences T for Various Surfacing Depths

The values of T given in Figure 4.3 are for 40 mm surfacing depths for groups 1 and 2 and 100 mm surfacing depths for groups 3 and 4. For other depths of surfacing, the values given in Tables 4.4 to 4.6 may be used. These values are based on the temperature difference curves given in the report, *Temperature difference in bridges*[13], by the Transport Research Laboratory (TRL). Methods of computing temperature difference are to be found in the TRL Report *The calculation of the distribution of temperature in bridges*[14].

Table 4.4: Values of T for Groups 1 and 2

Surfacing thickness (mm)	Positive temperature difference (°C)				Reverse temperature difference (°C)
	T_1	T_2	T_3	T_4	T_1
Unsurfaced	30	16	6	3	8
20	27	15	9	5	6
40	24	14	8	4	6

Table 4.5: Values of T for Group 3

Depth of slab h (m)	Surfacing thickness (mm)	Positive temperature difference (°C)	Reverse temperature difference (°C)
		T_1	T_1
0.2	unsurfaced	16.5	5.9
	waterproofed	23.0	5.9
	50	18.0	4.4
	100	13.0	3.5
	150	10.5	2.3
	200	8.5	1.6
0.3	unsurfaced	18.5	9.0
	waterproofed	26.5	9.0
	50	20.5	6.8
	100	16.0	5.0
	150	12.5	3.7
	200	10.0	2.7

Table 4.6: Values of T for Group 4

Depth of slab (h) (m)	Surfacing thickness (mm)	Positive temperature difference (°C)			Reverse temperature difference (°C)			
		T_1	T_2	T_3	T_1	T_2	T_3	T_4
≤0.2	unsurfaced	12.0	5.0	0.1	4.7	1.7	0.0	0.7
	waterproofed	19.5	8.5	0.0	4.7	1.7	0.0	0.7
	50	13.2	4.9	0.3	3.1	1.0	0.2	1.2
	100	8.5	3.5	0.5	2.0	0.5	0.5	1.5
	150	5.6	2.5	-0.2	1.1	0.3	0.7	1.7
	200	3.7	2.0	-0.5	0.5	0.2	1.0	1.8
0.4	unsurfaced	15.2	4.4	1.2	9.0	3.5	0.4	2.9
	waterproofed	23.6	6.5	1.0	9.0	3.5	0.4	2.9
	50	17.2	4.6	1.4	6.4	2.3	0.6	3.2
	100	12.0	3.0	1.5	4.5	1.4	1.0	3.5
	150	8.5	2.0	1.2	3.2	0.9	1.4	3.8
	200	6.2	1.3	1.0	2.2	0.5	1.9	4.0

Depth of slab (h) (m)	Surfacing thickness (mm)	Positive temperature difference (°C)			Reverse temperature difference (°C)			
		T_1	T_2	T_3	T_1	T_2	T_3	T_4
0.6	unsurfaced	15.2	4.0	1.4	11.8	4.0	0.9	4.6
	waterproofed	23.6	6.0	1.4	11.8	4.0	0.9	4.6
	50	17.6	4.0	1.8	8.7	2.7	1.2	4.9
	100	13.0	3.0	2.0	6.5	1.8	1.5	5.0
	150	9.7	2.2	1.7	4.9	1.1	1.7	5.1
	200	7.2	1.5	1.5	3.6	0.6	1.9	5.1
0.8	unsurfaced	15.4	4.0	2.0	12.8	3.3	0.9	5.6
	waterproofed	23.6	5.0	1.4	12.8	3.3	0.9	5.6
	50	17.8	4.0	2.1	9.8	2.4	1.2	5.8
	100	13.5	3.0	2.5	7.6	1.7	1.5	6.0
	150	10.0	2.5	2.0	5.8	1.3	1.7	6.2
	200	7.5	2.1	1.5	4.5	1.0	1.9	6.0
1.0	unsurfaced	15.4	4.0	2.0	13.4	3.0	0.9	6.4
	waterproofed	23.6	5.0	1.4	13.4	3.0	0.9	6.4
	50	17.8	4.0	2.1	10.3	2.1	1.2	6.3
	100	13.5	3.0	2.5	8.0	1.5	1.5	6.3
	150	10.0	2.5	2.0	6.2	1.1	1.7	6.2
	200	7.5	2.1	1.5	4.8	0.9	1.9	5.8
≥1.5	unsurfaced	15.4	4.5	2.0	13.7	1.0	0.6	6.7
	waterproofed	23.6	5.0	1.4	13.7	1.0	0.6	6.7
	50	17.8	4.0	2.1	10.6	0.7	0.8	6.6
	100	13.5	3.0	2.5	8.4	0.5	1.0	6.5
	150	10.0	2.5	2.0	6.5	0.4	1.1	6.2
	200	7.5	2.1	1.5	5.0	0.3	1.2	5.6

4.5.6 Coefficients of Thermal Expansion

For the purpose of calculating temperature effects, the coefficient of thermal expansion for structural steel and for concrete may be taken as 12 x 10^{-6}/°C, except where more accurate values have been experimentally determined for the concrete to be used.

4.5.7 Nominal Values

4.5.7.1 Nominal Range of Movement

For the purpose of designing expansion joints, etc, the effective bridge temperature at the time the structure is attached to those parts permitting movement shall be taken as the datum, and the nominal range of movement shall be calculated for expansion up to the maximum effective bridge temperature and for contraction down to the minimum effective bridge temperature.

4.5.7.2 Nominal Action for Temperature Restraint

The force due to temperature restraint of expansion or contraction for the appropriate effective bridge temperature range (see Section 4.5.4) shall be taken as the nominal force.

Where temperature restraint is accompanied by elastic deformations in flexible piers and elastomeric bearings, the nominal force shall be derived as specified below (flexure of piers and elastomeric bearings).

Flexure of piers: For flexible piers pinned at one end and fixed at the other, or fixed at both ends, the force required to displace the pier by the amount of expansion or contraction for the appropriate effective bridge temperature range (see Section 4.5.4) shall be taken as the nominal force.

Elastomeric bearings: For temperature restraint accommodated by shear in an elastomer, the force required to displace the elastomer by the amount of expansion or contraction for the appropriate effective bridge temperature range (see Section 4.5.4) shall be taken as the nominal force.

The nominal force shall be taken as

where

A_b	=	the actual plan area of elastomer bearing
G	=	the shear modulus of elasticity
δ_1	=	the nominal range of movement of the bearing
t_1	=	the total thickness of elastomer in shear

4.5.7.3 Nominal Force for Frictional Bearing Restraint

The nominal force caused by the frictional bearing restraint shall be derived from the nominal dead load (Section 2.2.1) and the nominal superimposed dead load (Section 2.3.1) using the appropriate coefficient of friction given as follows (Table 4.7):

Table 4.7: Coefficient of friction for bearing restraints

Bearing type	Detail		Coefficient of friction (μ)
For roller bearings with bearing plates at the circumference of the rollers	one or two rollers		0.03
	three or more rollers		0.05
For sliding bearings, steel on polytetra-fluoroethylene (PTFE)	For average contact pressure of:	10 N/mm^2	0.06
		20 N/mm^2	0.04
		30 N/mm^2	0.03

The above coefficients of friction are for bearings that comply with the requirements of Appendix F of Part 2 of BS5400[2].

For other bearings, the coefficient of friction shall be derived from tests, unless adequate data supporting the value adopted are available. The test procedures given in Appendix F of Part 2 of BS5400[2] are recommended.

The above values of the coefficients of friction are assumed to be 99 per cent characteristic values. Where the forces transmitted through bearings to piers, columns or abutments are significant in the design of these members, consideration shall be given to increasing the coefficients to allow for possible malfunction. The amount of such increase shall be based on the risks involved and the cost implications, but could be as high as 100 per cent.

4.5.7.4 Nominal Effects of Temperature Difference

The effects of temperature difference shall be regarded as nominal values.

4.5.8 Design Values

4.5.8.1 Design Range of Movement

For the purpose of designing expansion joints, etc, the design range of movement shall be taken as 1.3 times the appropriate nominal value for the ultimate limit state and 1.0 times the nominal value for the serviceability limit state.

For the purpose of this clause, the ultimate limit state shall be regarded as a condition where expansion or contraction beyond the serviceability range up to the ultimate range would cause collapse or substantial damage to main structural members. Where expansion or contraction beyond the serviceability range will not have such consequences, only the serviceability range need be provided for.

4.5.8.2 Design Force for Temperature Restraint

For combinations (2) and (3), γ_{fL} shall be taken as follows (Table 4.8):

Table 4.8: γ_{fL} for temperature restraint

γ_{fL}	Ultimate limit state			Serviceability limit state		
Combinations	1	2	3	1	2	3
	-	1.0	1.3	-	0.7	1.0

4.5.8.3 Design Force for Frictional Bearing Restraint

For combination (3), γ_{fL} shall be taken as follows (Table 4.9):

Table 4.9: γ_{fL} for frictional bearing constraint

γ_{fL}	Ultimate limit state			Serviceability limit state		
Combinations	1	2	3	1	2	3
	-	-	1.3	-	-	1.0

Associated vertical design load. The design dead load (see Section 2.2.2) and design superimposed dead load (see Section 2.3.2) shall be considered in conjunction with the design force due to frictional bearing restraint.

4.5.8.4 Design Effects of Temperature Difference or Gradient

For combinations (2) and (3), γ_{fL} shall be taken as follows (Table 4.10):

Table 4.10: γ_{fL} for temperature difference or gradient

γ_{fL}	Ultimate limit state			Serviceability limit state		
Combinations	1	2	3	1	2	3
	-	0.8	1.0	-	0.6	0.8

5. COMBINATIONS OF ACTIONS

5.1 General Requirements of Combinations

The actions to which the bridge structure may be subjected are to be applied in various combinations as specified below. Reduced partial load factors are used where the probability of two or more actions attaining their nominal values simultaneously is less than that for a single nominal action.

Supplementary actions classified under Combinations (2) and (3) act separately in combination with their associated principal actions and the restraint actions.

5.2 Combination Description and Tabulation

Three combinations of actions defined in Section 1.3 are to be used, viz:

Combination 1 includes permanent, long-term and transient or variable principal actions, plus long-term restraint actions only.

Combination 2 includes permanent, long-term and transient or variable principal actions, plus supplementary actions (but excluding impact on bridge supports), plus long and short-term restraint actions.

Combination 3 includes permanent and long-term principal actions, plus supplementary actions (excluding secondary forces caused by traffic on the bridge, but including impact on bridge supports), plus long and short-term restraint actions.

The above described combinations and the relevant partial action factors are tabulated in Table 5.1 on the following page. **Reference should, however, be made to the relevant specification clauses for fuller descriptions.**

Type of action			Nominal action F_k			Notation	Clause No	Limit state	$\gamma_{fL} = \gamma_{f1} \times \gamma_{f2}$ to be considered in combinations: 1	2	3
Direct actions	Principal actions	Permanent and long term	Dead loads: concrete			g_c	2.2.2	ULS SLS	1.2 1.0	1.05 1.0	1.2 1.0
			Dead loads: steel			g_s	2.2.2	ULS SLS	1.1 1.0	1.0 1.0	1.1 1.0
			Superimposed dead loads			g_{sd}	2.3.2	ULS SLS	1.2 1.0	1.05 1.0	1.2 1.0
			Reduced load for dead and superimposed dead load where this has a more severe effect			$g_{\acute{s}d}$	2.2.2.2 2.3.2.2	ULS SLS	1.0 1.0	1.0 1.0	1.0 1.0
			Vertical earth loading on culverts	Method (i)		g_n	2.3.3.3	ULS SLS	1.5 1.1	1.3 1.1	1.5 1.1
				Methods (ii) & (iii)		g_n	2.3.3.3	ULS SLS	1.4 1.0	1.2 1.0	1.4 1.0
			Earth pressure due to retained fill	Approximate theory		f_{ep}	2.4.3	ULS SLS	1.5 1.1	1.3 1.1	1.5 1.1
				More accurate theory		f_{ep}	2.4.3	ULS SLS	1.4 1.0	1.2 1.0	1.4 1.0
			As above but causing relieving effect			$f_{\acute{e}p}$	2.4.3.1	ULS SLS	See Clause 2.4.3.1		
		Permanent and long term / Transient and variable	Water pressure of retained or excluded water			f_w	2.5.2	ULS SLS	1.2 1.0	1.05 1.0	1.2 1.0
			As above but causing relieving effect			f_w	2.5.2	ULS SLS	1.0 1.0	1.0 1.0	1.0 1.0
		Transient and variable	Primary live	Vehicle traffic loading and surcharge	NA Load	Q_a	2.6.3.3	ULS SLS	1.5 1.0	1.3 1.0	-
					NB Load	Q_b	2.6.4.3	ULS SLS	1.2 1.0	1.1 1.0	-
					NC + b NA on separate carriageways	Q_c	2.6.5.3	ULS SLS	1.2 1.0	1.1 1.0	-
			Sidewalk and cycle track loading			Q_p	2.7.1.2	ULS SLS	1.5 1.0	1.3 1.0	-
		Short term	Erection loads			Q_e	2.8.2.2	ULS SLS	-	1.15 1.0	-
	Supplementary actions (each acting separately in combination with associated principal actions)	Transient — Secondary forces caused by primary live loads	Centrifugal forces			F_c	3.2.4	ULS SLS	-	1.5 1.0	-
			Longitudinal braking and traction forces	NA Load		F_L	3.3.6 3.6.4	ULS SLS	-	1.25 1.0	-
				NB Load		F_L	3.3.6 3.6.4	ULS SLS	-	1.1 1.0	-
			Accidental skidding			F_{as}	3.4.4	ULS SLS	-	1.25 1.0	-
			Impact due to vehicle collision with bridge balustrades or parapets			F_b	3.5.4	ULS SLS	-	1.25 1.0	-
			Impact due to vehicle collision with bridge supports			F_{im}	3.7.5	ULS SLS	-	-	1.25 1.0
		Transient — Forces due to natural causes	Wind action	With erection loads		W	3.8.7	ULS SLS		1.1 1.0	-
				With permanent principal actions only		W	3.8.7	ULS SLS		-	1.4 1.0
				With permanent and transient principal actions		W	3.8.7	ULS SLS		1.1 1.0	-
				Relieving effects of wind		W	3.8.7	ULS SLS		1.0 1.0	1.0 1.0
			Flood action			F_f	3.9.8	ULS SLS		1.05 0.85	1.3 1.0
			Earthquake action			F_{eq}	3.10.5	ULS SLS		-	1.3 1.0
Indirect actions	Imposed deformations or restraint actions	Long - term	Creep and shrinkage			F_{cc}, F_{cs}	4.2.2	ULS SLS	1.0 1.0	1.0 1.0	1.0 1.0
			Parasitic prestress and prestrain			F_{pp} $F_{p\varepsilon}$	4.3.2	ULS SLS	1.0 1.0	1.0 1.0	1.0 1.0
			Differential settlement			F_a	4.4.2	ULS SLS	1.0 1.0	1.0 1.0	1.0 1.0
		Short - term	Temperature range			F_{tr}	4.5.8	ULS SLS	-	1.3 1.0	1.3 1.0
			Temperature gradient			F_{tg}	4.5.8	ULS SLS	-	1.0 0.8	1.0 0.8
			Frictional bearing restraint			F_{br}	4.5.8	ULS SLS	-	-	1.3 1.0

* ULS - Ultimate limit state SLS - Serviceability limit state

6. REFERENCES

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APPENDIX A

EXPLANATORY NOTES ON TRAFFIC LOADING

APPENDIX A: EXPLANATORY NOTES ON TRAFFIC LOADING

A.1 NA Loading (Refer to Section 2.6.3)

NA Loading is a formula loading representing normal traffic consisting of the most severe arrangements of legal vehicles that are probable. It is based on South African legal loadings and an allowance for impact based on the Swiss (SIA Norm 160 - 1970) Highway Impact Factor

$$\Phi = 0.05\left(\frac{100 + L_s}{10 + L_s}\right)$$

where L_s is the equivalent span length in metres.

Cognisance has been taken of the results derived from theoretical analyses based on the assumption that normal traffic behaviour is a quasi-random phenomenon, and statistical information obtained from field studies of the distribution of vehicles along highways. However the so-called "credibility" approach which is admittedly intuitive and rather arbitrary, has been preferred for the reasons given below.

It is considered that insufficient statistical information is at present available about traffic in general and specifically traffic in the SADC region. Traffic behaviour is not a simple random phenomenon, but is heavily conditioned by the characteristics of a particular route, the type of traffic and such effects as the tendency for a queue to form behind a heavy vehicle which in turn causes other heavy vehicles, which cannot overtake as readily as lighter vehicles, to accumulate. Furthermore, traffic behaviour is subject to human direction and manipulation which can result in heavy convoys. Although design standards cannot provide for every possibility, the so-called "economic" probability of failure does not appear at this stage to be acceptable in terms of judgements in our courts, and it is accordingly necessary to provide for a non-zero but very low probability of failure for any single bridge.

It is generally accepted and can readily be shown that, except in the very small span range, the worst loading condition occurs under congested (bumper to bumper) conditions caused by a traffic blockage and that the dispersion of traffic at speed caused by increased vehicular interspacing, more than off-sets the effects of impact. In assessing the equivalent uniformly distributed line loads and axle loads, simply supported spans only have been considered, but in determining maximum bending moments and shear forces, the sequence of vehicle types has been rearranged to give the most severe loading. For the very short span range referred to above, on which one or more vehicles at speed may cause the most severe effect, the Swiss impact formula has been applied to vehicles complying with the abovementioned legal axle loadings. For spans of less than 6.1 m the former HA loading of BS153 (1954) increased rapidly as the spans diminished in order to cater for the effects of heavy wheels on short spans; this increase has now been abandoned. The application of the loading caused by a single NB vehicle as specified in Sections 2.6.1 and 2.6.4 has now become obligatory. This will produce a more realistic design of short span lengths and will be advantageous in rigorous distribution analysis. In particular, it will free designers from the arbitrary edge-stiffening rules for slabs and allow rigorous analysis to be applied to any shape and width of slab. There has also been some concern about rogue abnormal loads and this arrangement serves as a check in this respect.

A.2 Application of NA loading (Refer to Section 2.6.3.2)

A.2.1 Longitudinal Distribution of NA Distributed Loading in Parts of Lanes

Contrary to the practice of codes such as BS5400[2] in which all the loadable parts of two lanes of a bridge are fully loaded at a single intensity corresponding to the aggregate loaded length of the parts of one lane, and in which an arbitrarily reduced loading of one third of the abovementioned intensity is applied to the loaded parts of the rest of the lanes, this specification provides for a procedure whereby the loading intensities are based on the assumption that the total loading is dependent on the aggregate loaded length of all the notional lanes being considered. The intensities of the uniformly distributed loadings in these separate parts need not, however, be equal. To determine the maximum effect on any structural member in terms of this specification, an approximate procedure may be adopted whereby the sequence of loading is determined by the ranking of the average influence values of the above mentioned parts as follows.

That part of any notional lane which has the maximum average influence value (positive or negative as the case may be), is loaded at an intensity determined by the NA uniformly distributed loading formula for that loaded length.

Thereafter, that part of the same or any other notional lane with the next highest average influence value of similar sign is loaded at an intensity such that the total load on the two loaded parts corresponds to the formula loading for a loaded length equal to the sum of the two loaded lengths.

This procedure is continued until all the parts of equal sign are loaded.

If $\sum_{i=1}^{p} L_i$ is the sum of all loaded lengths up to and including the p^{th} parts, the intensity of loading Q_{ap} on the p^{th} parts of length Lp is:

$$Q_{ap} = \left[Q_a \sum_{i=1}^{p} L_i - \sum_{i=1}^{p-1} Q_i L_i \right] / L_p$$

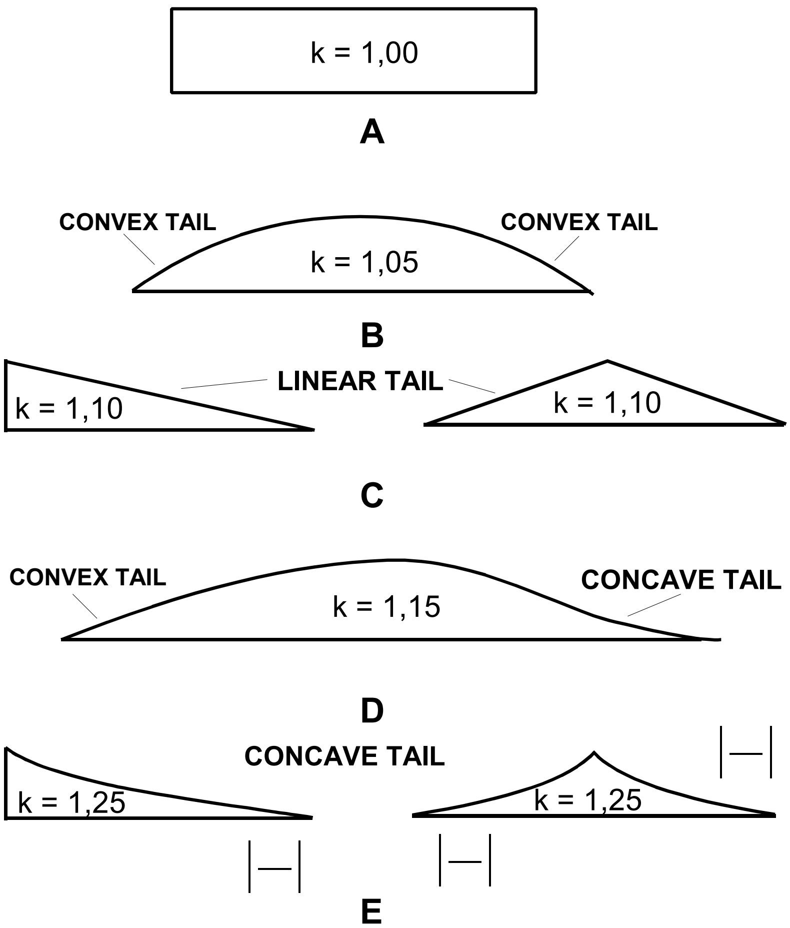
where

Q_a	=	the intensity of loading obtained from the curve in Figure 2.1 for $\sum_{i=1}^{p} L_i$ length
Q_i	=	the intensity of loading applied to any previously calculated base length portion i;
L_i	=	the dimension of any previously calculated base length portion i.

In this procedure Q_{ap} reduces with increase of p in accordance with the above formula and has no limiting value.

A.2.2 Partial Loading of Parts of Influence Lines (Refer to Section 2.6.3.2)

A further complication arises from the fact that partial loading of an influence line may result in a more severe effect than that caused by loading the whole base of the relevant part of the influence line with a correspondingly lower intensity of uniformly distributed load. These effects can be readily calculated. However, since traffic intensities are difficult to predict with great accuracy, it will be satisfactory if full loading of influence lines (either positive or negative) is applied, provided that the approximate correction factors (k) for the various types of influence line given in Figure A.1 are applied to the formula loading in the case of all loaded parts for which the total loading divided by the total loaded length is in the range of decreasing values of Q_a, provided that the product does not exceed 36 kN/m. Note that the classification of the influence lines shown depends on the curvature of the tails. The factors given are approximate in that they can vary considerably depending on the loaded lengths and the loading sequence of lanes. The influence lines shown in Figure A.1 do not cover all cases. An accurate calculation or conservative values of the k factors may be preferable in critical cases.



For a concave tail where $\left|\frac{dy}{dx}\right|$ >0, interpolate approximately between B and C and E

Figure A.1: Correction factors for typical shapes of influence lines

A.2.3 Alternative Methods of Longitudinal Distribution in Lanes (Refer to Section 2.6.3.2)

The alternative method has been introduced as a permissible simplification and although it will generally give more conservative results than Method A(i), this is not always the case. Although different intensities of loading on separate parts in a lane need not be applied, the correction factors (k) for the effects of partial loading given in Figure A.1 must be used.

A.2.4 Axle Loading

The "axle load" (or knife-edge load) has been expressed as a function 144 / √n kN per notional lane, irrespective of the loaded lengths, where n is the loading sequence number of a lane. This does not represent an actual axle load, but its application as a concentrated loading superimposed on the uniformly distributed loading is necessary in order to obtain a sufficiently accurate simulation of the effects of groupings of axle loads as caused by actual vehicular traffic. The probability of the requirement of a simultaneous application of the "axle loads" at specific positions in each of the notional lanes is assumed to be independent of the loaded lengths, but is a function of the number of lanes. The above function is admittedly an over-simplication of the problem, but can be shown to be a reasonable estimate of the probable maximum effect.

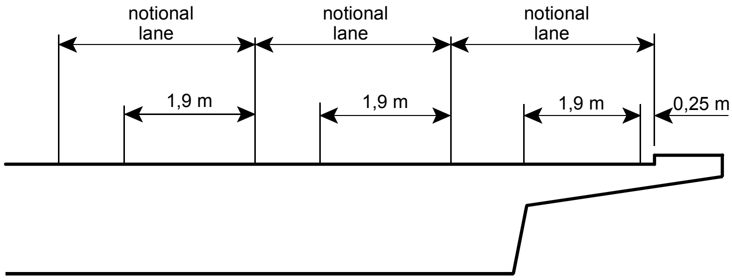
A.2.5 Transverse Distribution and Position of NA Loading in Lanes (Refer to Section 2.6.3.2)

Contrary to the practice in the United Kingdom where the HA and HB loadings are applied simultaneously, this specification requires that the NA and NB loadings be applied separately. This simplifies the analysis. However, in order to cover all the probable effects of normal loading, it is consequently necessary to apply the uniformly distributed and knife-edge loadings in the form of two equal *uniformly distributed line loads* and two equal *point* loads respectively. The application of these loadings as specified in Section 2.6.3.2 (Distributed load - transverse distribution), will result in a more realistic simulation of maximum probable transverse eccentricities of load and concentration of loading on single longitudinal members in certain span ranges that are not adequately covered by the NB loading. Typical positions of the line loads to obtain maximum effects are shown in Figure A.2.

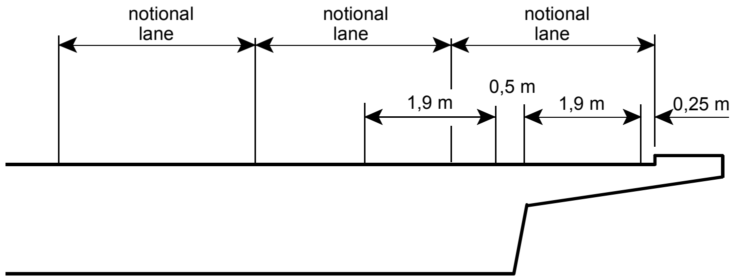
Where the abovementioned transverse effects are not significant, the line and point loadings may however be applied as uniformly distributed loads in the transverse direction and knife edge loads respectively.

The line loading cannot, however, apply to non-integral numbers of lanes for bridge width less than 4.8 m as defined in Section 2.6.2(ii), which necessitates the application of both a single lane loading in the above form and an alternative non-integral lane loading uniformly distributed across the width.

Where parts of the influence surfaces are of opposite sign to the total effect, the distributed loading on these parts shall be omitted in terms of Section 2.6.1.4.



(a) EXTREME TRANSVERSE POSITIONS FOR NA LOADING ON THREE LANES.



(b) EXTREME TRANSVERSE POSITIONS FOR NA LOADING OF TWO LANES STRADDLING TWO LANES.

Figure A.2: Straddling of lanes

A.2.6 Skew Bridges

The two equal and parallel line loads of $\frac{Q_a}{2}$ kN/m for a particular notional lane of equal length, terminating at each end on a line orthogonal to the direction of the lane (refer to Section 2.6.3.2 (Distributed load - transverse distribution)). These requirements are illustrated for the case of a skew bridge in Figure A.3. The loading shown is for the parts fully loaded. Application of the correction factor k is consequently necessary to allow for the effects of partial loading.

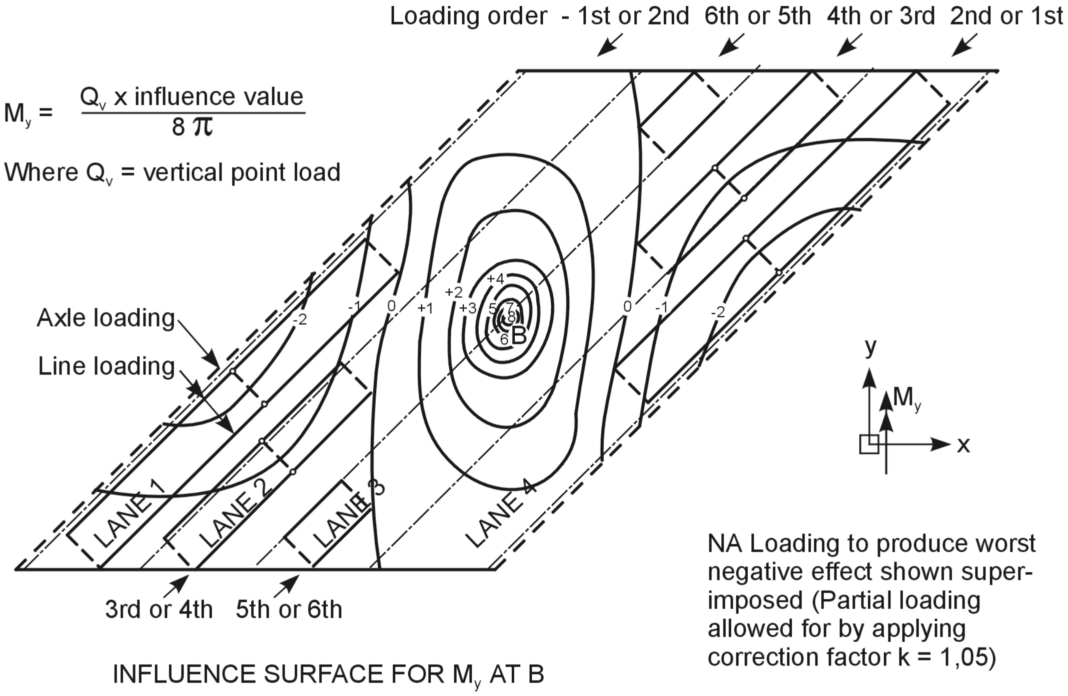


Figure A.3: NA line and axle loading on skew bridge

APPENDIX B

RECOMMENDATIONS FOR THE PROTECTION OF PIERS BY SAFETY FENCES OR GUARDRAILS

APPENDIX B: RECOMMENDATIONS FOR THE PROTECTION OF PIERS BY SAFETY FENCES OR GUARDRAILS

The space available for deflection determines the arrangement of the safety fences. If the clearance between the pier and the guardrail is 0.6 m or more, the guardrail should be mounted on posts to form a free-standing fence. If the clearance is less than 0.6m, the guardrail should be mounted on the traffic face of the member by means of energy-absorbing brackets. Whatever the arrangement, the protection afforded should be such that when a car of 1.5 t strikes the safety fence at 110 km/h and at an angle of 20°, the wheels of the car will only just reach the member.

APPENDIX C

VIBRATION SERVICEABILITY REQUIREMENTS FOR PEDESTRIAN AND CYCLE TRACK BRIDGES

APPENDIX C: VIBRATION SERVICEABILITY REQUIREMENTS FOR PEDESTRIAN AND CYCLE TRACK BRIDGES

C.1 General

For superstructures where n_o (the fundamental natural frequency of vibration for the unloaded bridge) exceeds 5 c/s, the vibration serviceability requirement is deemed to be satisfied.

For superstructures where n_o is equal to or less than 5 c/s, the maximum vertical acceleration of any part of the superstructure shall be limited to $0.5\sqrt{n_o}$ m/s². The maximum vertical acceleration shall be calculated in accordance with the methods given in Sections C.2 or C.3, as appropriate.

C.2 Simplified Method for Deriving Maximum Vertical Acceleration

This method is valid only for single-span, or two or three-span continuous, symmetric superstructures, of constant cross-section and supported on bearings that may be idealized as simple supports.

The maximum vertical acceleration a (in m/s²) shall be taken as:

$$a = 4\pi^2 n_o^2 Y_s K \Psi$$

where

n_o	=	the fundamental natural frequency (in c/s) (see Section C.2.3)
Y_s	=	the static deflection (in m) (see Section C.2.4)
K	=	the configuration factor (see Section C.2.6)
ψ	=	the dynamic response factor (see Section C.2.6)

For values of no greater than 4 c/s, the calculated maximum acceleration may be reduced by an amount varying linearly from zero reduction at 4 c/s to 70 per cent reduction at 5 c/s.

C.2.1 Modulus of Elasticity

In calculating the values of n_o and Y_s, the short-term moduli of elasticity shall be used for concrete and steel.

C.2.2 Second Moment of Area

In calculating the values of n_o and Y_s, the second moment of area for sections of discrete concrete members may be based on the entire uncracked concrete section ignoring the presence of reinforcement. The effects of shear lag need not be taken into account in steel and concrete bridges.

C.2.3 Fundamental Natural Frequency n_o

The fundamental natural frequency n_o is evaluated for the bridge including superimposed dead load but excluding pedestrian live loading.

The stiffness of the parapets shall be included where they contribute to the overall flexural stiffness of the superstructure (see Appendix D).

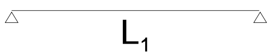
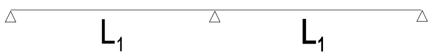
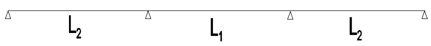
C.2.4 Static Deflection Y_s

The static deflection Y_s is taken at the midpoint of the main span for a vertical concentrated load of 0.7kN applied at this point. For three-span superstructures the centre span is taken as the main span.

C.2.5 Configuration Factor K

Values of K shall be taken from Table C.1.

Table C.1: Configuration factor K

<table>
<tr><th>Bridge Configuration</th><th>Ratio L_2 / L_1</th><th>K</th></tr>
<tr><td>
</td><td>-</td><td>1.0</td></tr>
<tr><td>
</td><td>-</td><td>0.7</td></tr>
<tr><td>
</td><td>1.0
0.8
0.6 or less</td><td>0.6
0.8
0.9</td></tr>
</table>

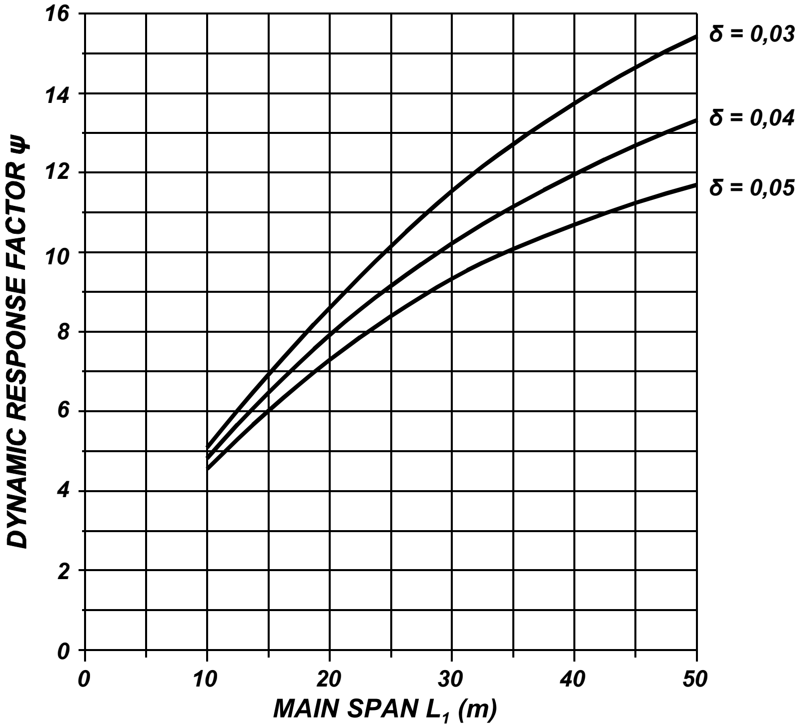
For three-span continuous bridges, intermediate values of K may be obtained by linear interpolation.

C.2.6 Dynamic Response Factor ψ

Values of ψ are given in Figure C.1. In the absence of more precise information, the values of δ (the logarithmic decrement of the decay of vibration due to structural damping) given in Table C.2 should be used.

Table C.2: Logarithmic decrement of decay of vibration δ

Bridge superstructure	δ
Steel with asphalt or epoxy surfacing	0.03
Composite steel/concrete	0.04
Prestressed and reinforced concrete	0.05



NOTE 1. ***Main span L_1 is shown in Table 18***

NOTE 2. ***Values of δ for different types of construction are given in Table 19.***

Figure C.1: Dynamic response factor

C.3 General Method for Deriving Maximum Vertical Acceleration

For superstructures other than those specified in Section C.2 the maximum vertical acceleration should be calculated assuming that the dynamic loading applied by a pedestrian can be represented by a pulsating point load F, moving across the main span of the superstructure at a constant speed V_1 as follows:

$F = 180 \sin 2\pi n_o T$ (in N), where T is the time (in s)

$V_1 = 0.9\, n_o$ (in m/s)

For values of n_o greater than 4 c/s, the calculated maximum acceleration may be reduced by an amount varying linearly from zero reduction at 4 c/s to 70 per cent reduction at 5 c/s.

C.4 Damage from Forced Vibration

Consideration should be given to the possibility of permanent damage to a superstructure by a group of pedestrians deliberately causing resonant oscillations of the superstructure. As a general precaution therefore, the bearing should be of robust construction with adequate provision to resist upward or lateral movement.

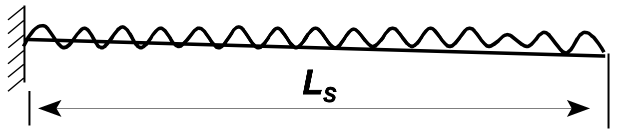
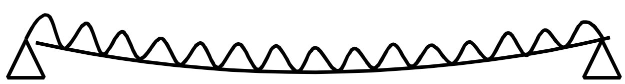
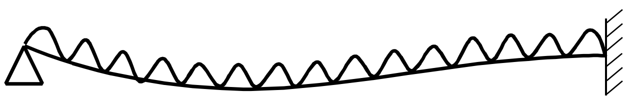
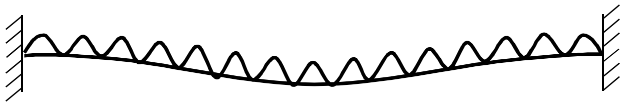
In a prestressed concrete construction, resonant oscillation may result in a reversal in excess of 10 per cent of the static live load bending moment. Provided that sufficient unstressed reinforcement is available to prevent gross cracking, no further consideration need be given to this effect.

APPENDIX D

APPROXIMATE EVALUATION OF FUNDAMENTAL NATURAL FREQUENCIES OF BRIDGE STRUCTURES

APPENDIX D: APPROXIMATE EVALUATION OF FUNDAMENTAL NATURAL FREQUENCIES OF BRIDGE STRUCTURES

Many structures or elements of structures can be approximately simulated by one of the equivalent systems shown in Figure D.1.

	k_o
	*0,56*
	*1,57*
	*2,45*
	*3,56*

n_o = fundamental natural frequency

= $k_o \sqrt{EI/mL_s^4}$

where

EI	=	flexural rigidity of the equivalent member
m	=	mass per unit length
L_s	=	span of the equivalent member

Structures which cannot be simulated by any of the above systems can be solved by applying an extension of the energy method developed by Rayleigh[9,10].

The procedure of this method is as follows:

(i) Sketch out the general shape of the expected mode of vibration.

(ii) Subdivide the structure in such a way that representative portions of mass can be lumped at nodes along the structural members. Generally it is not required to have more than five nodes per span (including the support nodes).

(iii) Apply the weight W_i of each mass m_i as a load in the direction of deflection of the mode of vibration sketched out in (i), and perform a static analysis. The structure will displace at the nodes by the amount δ_i.

(iv) The maximum kinetic energy can be summed as follows:

$$E_k = ½ \frac{\omega^2}{g} \Sigma W_i \delta_i^2$$

where

ω = radial frequency

and the maximum strain energy (work done) can be summed as:

U_{max} = $½_{\Sigma wi\delta i}$

By equating the two energies, the radial frequency is found as:

$$\omega^2 = \frac{g \ \Sigma W_i \ \delta_i}{\Sigma W_i \ \delta_i^2}$$

Hence the fundamental natural frequency is:

$$n_o = \frac{1}{2\pi} \sqrt{\frac{g \Sigma W_i \ \delta_i}{\Sigma W_i \ \delta_i^2}}$$

and the natural period $T = \dfrac{1}{n_o}$

Note that consistent units must be used, e.g.

W = m.g if m is in tonnes and g in m/second²
W will be in kN, therefore the elastic modulus used in the analysis should be given in kN/m².

More rigorous computer analysis should be resorted to when more than only the fundamental (lowest) natural frequency needs to be evaluated.

APPENDIX E

SHRINKAGE AND CREEP

APPENDIX E: SHRINKAGE AND CREEP

E.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 Sections E.2 and E.3 is essentially that given in the *CEP-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-FIB 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 per cent 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.

E.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:

$$\triangle_{cc} = \frac{f_c}{E_{28}} \Phi$$

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 E.1 to E.5):

$$\Phi = k_L k_m k_c k_e k_j$$

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 per cent 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).

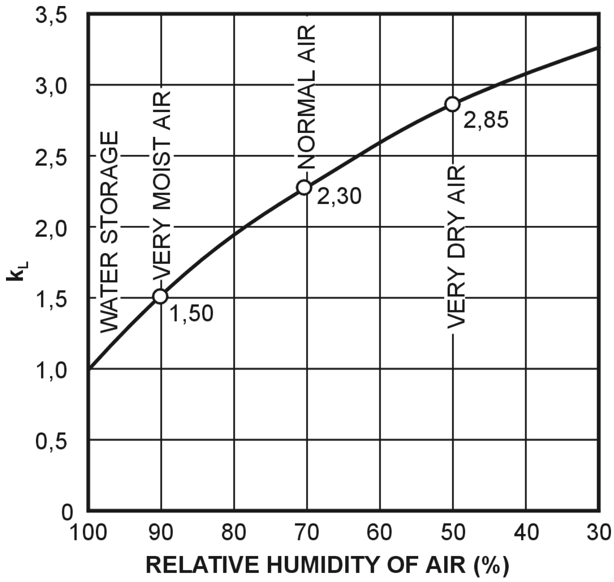


Figure E.1: Coefficient k_L (environmental conditions) for creep

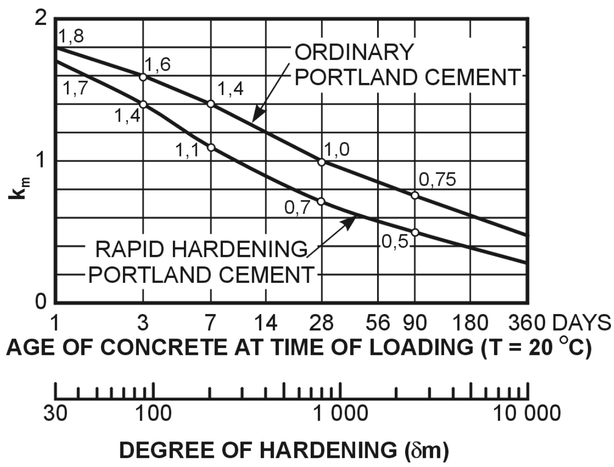


Figure E.2: Coefficient k_m hardening (maturity) at the time of loading

The values in Figure E.2 refer to Portland cement, hardened under normal conditions, i.e. at an average content temperature other than 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:

$$\delta_m = \Sigma j_m (T + 10°C)$$

where

δ_m = the degree of hardening at the time of loading

j_m = the number of days over which hardening has taken place at T°C.

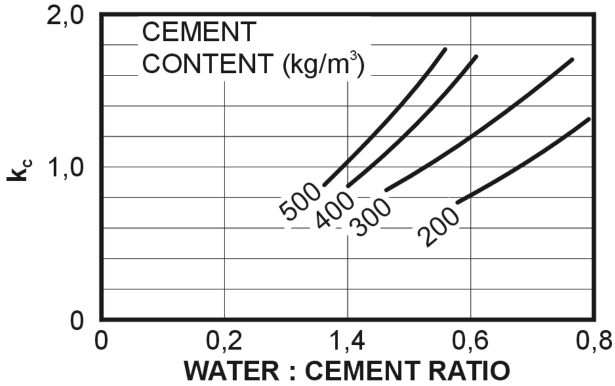


Figure E.3: Coefficient k_c (composition of the concrete)

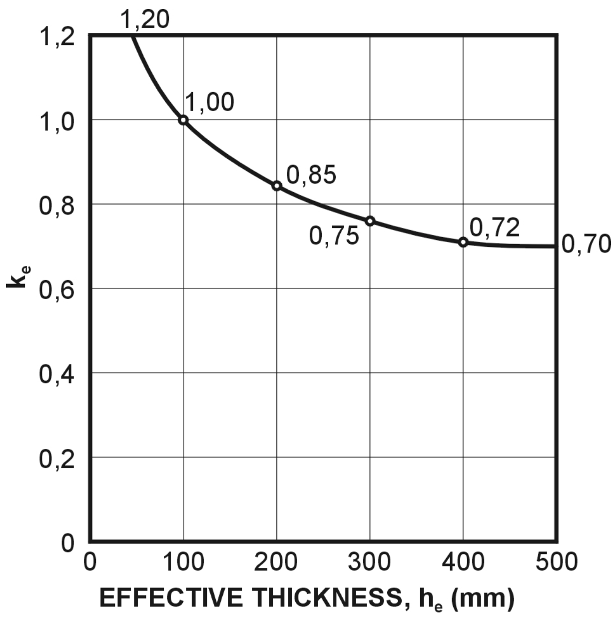


Figure E.4: Coefficient k_e (effective thickness) for creep

The effective thickness, k_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.

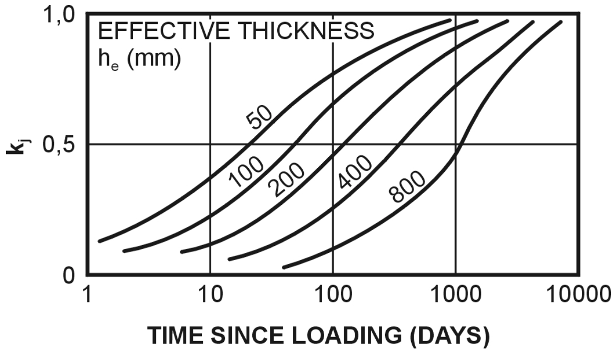


Figure E.5: Coefficient k_j (variation as a function of time)

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, i.e.:

$$\Delta_{cc} = \frac{1.6 f_c}{E_{28}}$$

An accelerated method for determining creep has also been developed[14].

At a given time, t, after the 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_{cj}, may be expressed as:

$$\Delta_{cct} = \frac{1}{E_{28}} [f_{cj}\Phi_{(t-j)} + \Sigma f_{co}\Phi_{(t-i)}]$$

or

$$\Delta_{cct} = \frac{k_L k_c k_e}{E_{28}} [f_{cj} k_{mj} k_{j(t-j)} + \Sigma f_{co} k_{mi} k_{j(t-i)}]$$

where

Δ_{cct} = the resulting strain.

As shown in the above two equations, loads should always be superimposed on the assumption that 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.

E.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 deformation 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 of k_2, gives the reduction in length of the fibre at the centre of gravity of the steel under consideration. The more detailed approached of Section E.3.4 is preferable.

The average values of the partial coefficients, $k_{L,}$ k_c, k_e and k_j, 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 E.6. Where embedded electric heating is used, values of the coefficient K_L should be based on experience.

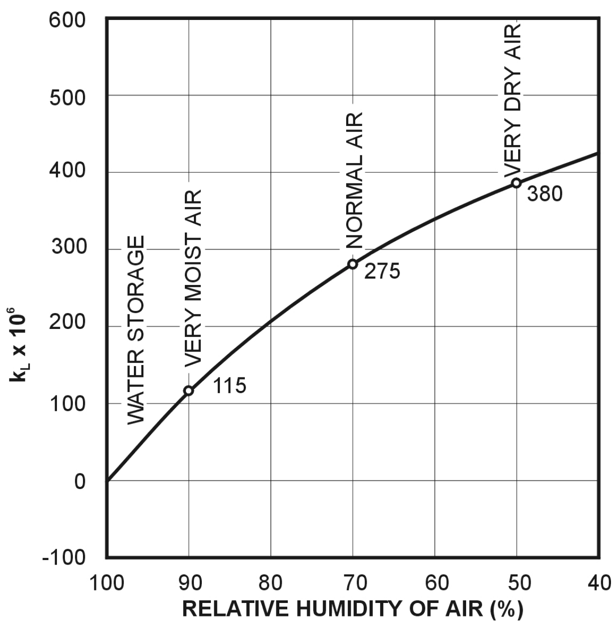


Figure E.6: Coefficient k_L (environmental conditions for shrinkage)

For coefficient k_c, (composition of the concrete), the same coefficients may be used for creep (refer to Figure E.3).

Figure E.7 gives the values of k_e, using the same definition of effective thickness, k_e, as for creep.

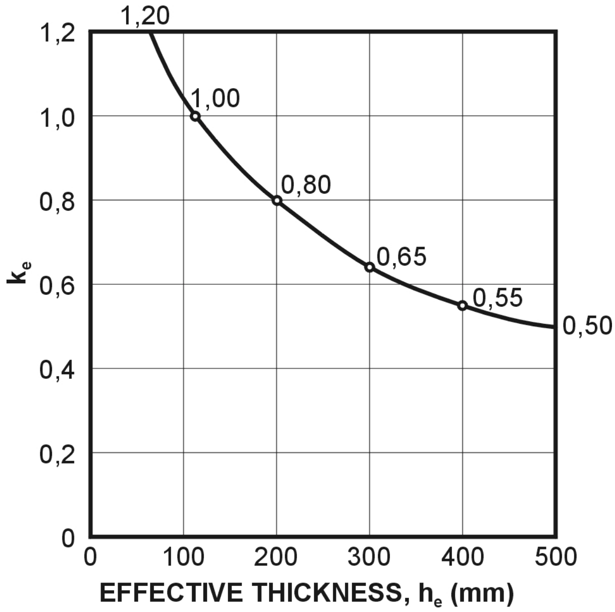


Figure E.7: Coefficient k_e (effective thickness) for shrinkage

For coefficient k_j, (variation as a function of time), the same coefficients may be used as for creep (refer to Figure E.5).

Where environmental conditions remain constant, the deformation due to shrinkage in an interval of time (t - i) is equal to:

$$\triangle_{cs}(t - i) = k_L k_c k_e (k_{jt} - k_{ji})$$

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.

E.4 Reinforced Concrete

The values given in Section E.3 apply to plain (unreinforced) concrete members. For structural members containing reinforcement, the magnitude of shrinkage and creep that is likely to be realised 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:

$$\eta = \int_{j_i}^{j_\infty} \frac{df_{cj}}{dj} \frac{1}{\Delta f_{c\infty}} \frac{k_{mj}}{k_{mi}} \, dj$$

where

j_∞	=	the age at the end of the life for the structure
j_i	=	the age at first loading
f_{cj}	=	the stress in the concrete at time of application (j) of an increment of stress
j	=	the age at application of an increment of stress
$\Delta f_{c\infty}$	=	the change in stress in the concrete due to creep at time t_∞
K_{mi}	=	the coefficient k_m for the age at first loading

For various values of $\Phi_n = \Phi/k_m$, the value of η can be obtained from Figure E.8.

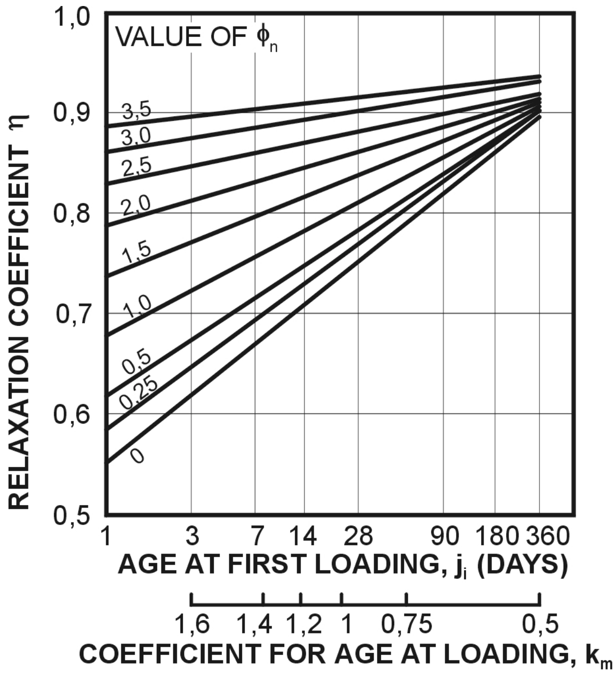


Figure E.8: Relaxation coefficient η

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:

$$\Phi_2 = \frac{1}{1 + \rho\alpha_e(1 + \eta\Phi)}$$

where

α_e = the modular ratio at first introduction of stress

ρ = the geometric ratio of reinforcement

The values of Φ_2 as a function of $\rho\alpha_e$ and $\eta\Phi$ are shown in Figure E.9.

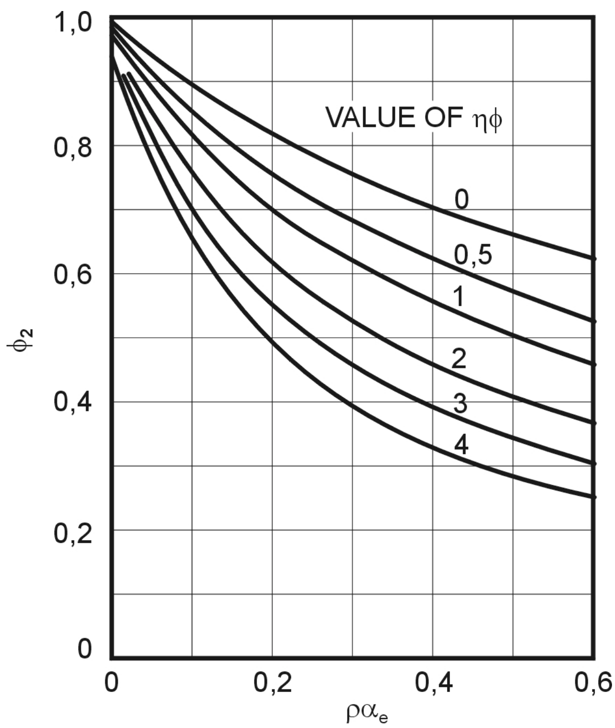


Figure E.9: Coefficient Φ_2

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:

$$\Phi_3 = \frac{1}{1 + \rho\alpha_e(1 + a_s^2 / i^2)(1 + \eta\Phi)}$$

where

a_s = the distance of the centroid of the steel from the centroid of the net concrete section

i = the radius of gyration of the net concrete section.

The values of Φ_3 at two levels 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 E.10 and E.11.

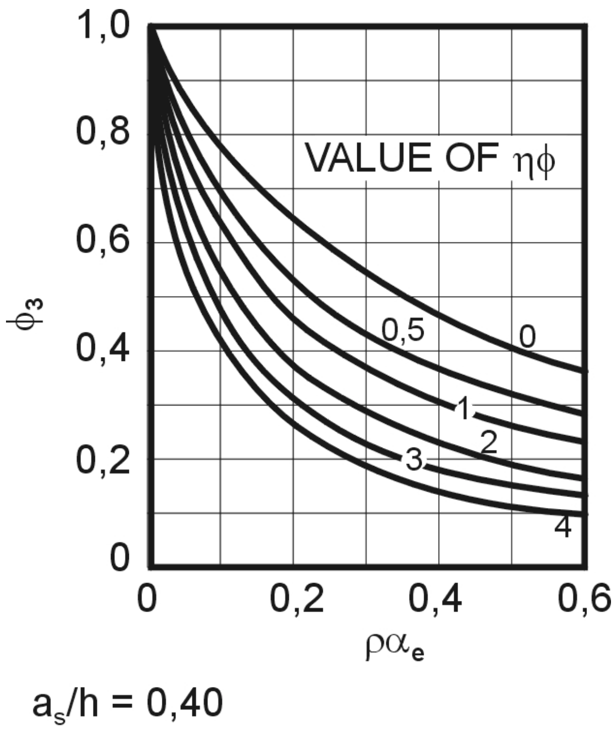


Figure E.10: Coefficient Φ_3 a_s/h = 0.40

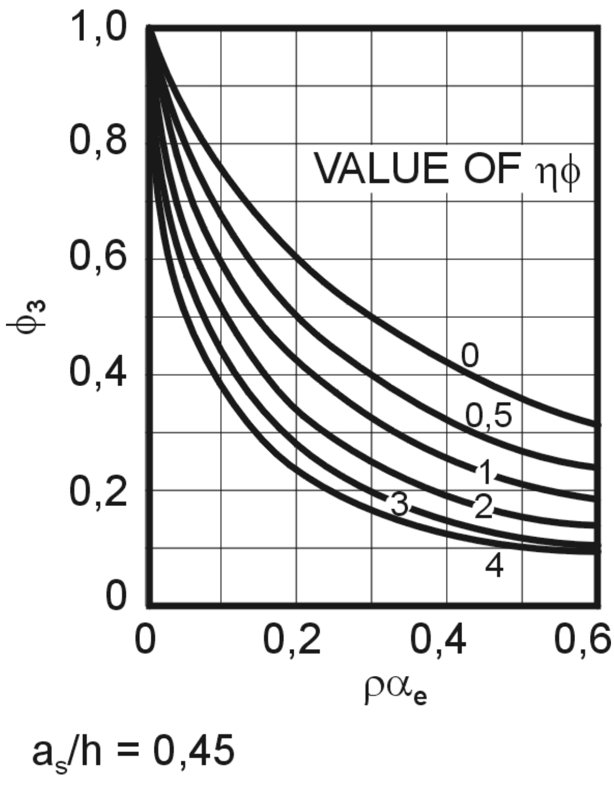


Figure E.11: Coefficient Φ_3 a_s/h = 0.45

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:

$$\Phi_4 = \frac{1}{1 + \rho\alpha_e(1 + \eta\Phi)a_s^2/\, i^2}$$

and for singly reinforced sections:

$$\Phi_5 = \frac{\rho\alpha_e}{1 + \rho\alpha_e} + \frac{1 + \rho\alpha_e(1 + \eta\Phi)}{1 + \rho\alpha_e(1 + a_s^2/\, i^2)(1 + \eta\Phi)(1 + \rho\alpha_e)}$$

Other analyses have been published[15].

E.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.

(a) In post-tensioned members with bonded tendons, the loss of prestress due to shrinkage and creep at time t can be calculated from:

$$\Delta\rho = \frac{\alpha_e f_{co}\Phi_{ti} + \Delta_{cst}E_s}{1 + \rho\alpha_e(1 + a_s^2/\, i^2)(1 + \eta\Phi_{ti})}$$

where

f_{co}	=	the stress in the concrete at the level of the tendon due to initial prestress and dead load.
Φ_{ti}	=	the creep coefficient at time t for a load applied at time i
Δ_{cst}	=	the shrinkage at time t.

The loss of prestressed due to relaxation of the steel must be added to the above loss.

In pretensioned members, there are additional losses of prestresses 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 per cent) 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:

$$\Phi = \frac{1}{a_s}\left[\frac{1 + \rho\alpha_e(1 + \eta\Phi)}{1 + \rho\alpha_e(1 + a_s^2/ i^2)(1 + \eta\Phi)}\varepsilon_{c1} - \varepsilon_{c2}\right]$$

where

ε_{c1} = the creep strain in concrete at the level of the tendon due to initial prestress and dead load

ε_{c2} = the creep strain in concrete at the centroid of the section due to initial prestresses and dead load.

Deflection due to creep can be calculated in the usual manner from the change in curvature.