COMITE EURO-INTERNATIONAL DU BETON

CEB-FIP MODEL CODE 1990

DESIGN CODE

Thomas Telford

T. Ullaga

Published by Thomas Telford Services Ltd, Thomas Telford House, 1 Heron Quay, London E14 4JD, for the Comité Euro-International du Béton, EPF Lausanne, Case Postale 88, CH 1015 Lausanne, Switzerland

First published 1993

Final draft published 1991 by the Comité Euro-International du Béton as Bulletins d'Information 203-205

Classification
Availability: Unrestricted
Content: Recommendations based on research and best current practice
Status: Committee guided
User: Specialist Designers

A catalogue record for this book is available from the British Library.

ISBN: 0 7277 1696 4

Typeset in Great Britain by Alden Multimedia

Printed in Great Britain by Redwood Books, Trowbridge, Wiltshire

PREFACE

The CEB/FIP Model Code for Concrete Structures was published in 1978 following approval by the Euro-International Committee for Concrete (CEB) at its 19th Plenary Session in Granada in September 1977—the publication was associated with the 8th Congress of the International Federation for Prestressing (FIP) in London in May 1978.

Since that time, the Model Code has had a considerable impact on the National Codes in many countries and, more particularly, on the harmonization of the codification process, as exemplified by the activities of the Commission of the European Communities (CEC), the Eastern Countries, the Nordic Building Regulations Committee (NKB) and members of the European Free Trade Association (EFTA). Indeed Eurocode 2 'Design of Concrete Structures, Part 1: General Rules and Rules for Buildings' used as its basic reference document the Model Code of 1978; this Eurocode was produced under the auspices of the CEC and EFTA members through the Comité Européen de Normalisation (CEN). The CEB activities in its Commissions also contributed to this work.

Naturally the work of the CEB, in synthesizing research findings and technical information with a view to translating them into practice, has continued and, at a certain stage, it became apparent that a revision of the Model Code could, with advantage, be undertaken. Thus the target of establishing the first complete draft of the Model Code 1990 was set and the CEB, together with FIP, worked towards that goal. A Committee for the Model Code (CMC) was set up under the Chairmanship of Professor T.P. Tassios, having an Editorial Board of E. Skettrup, U. Litzner, M. Miehlbradt, J. Perchat, and E. Siviero and with a supporting secretariat, originally in Athens and then in Copenhagen. Membership of the CMC comprised the Chairmen of the various Permanent Commissions and certain General Task Groups of CEB, designated Chairmen of FIP Committees and a number of invited experts.

The first complete draft of the Model Code 1990 was presented for consideration during the 11th Congress of the FIP in Hamburg in June 1990 and for consideration and approval at the 27th Plenary Session of the CEB in Paris in September 1990. Subsequently, the Committee for the Model Code reviewed all the comments received and, acting on the basis of the Technical Resolutions of the 27th Plenary Session, has produced this definitive version of Model Code 1990 for ratification during the 28th Plenary Session of CEB in Vienna in September 1991.

The Model Code 1990, in its drafting, has already influenced the codification work going on concurrently in the National and International fields—a natural effect of the inherent dissemination process occurring within and between international and professional scientific Assocations—and will certainly influence the future codification process, which is a common aim of the CEB and FIP.

On behalf of our two Associations we must thank all those concerned with this work for their sustained efforts to bring the Model Code 1990 to a successful conclusion. These thanks go particularly to those who bore the main burden of the work—the Chairmen and Editorial Board of the CMC: their enthusiasm for, and dedication to, the task were notable.

Finally, both CEB and FIP commend the Model Code 1990 for study and use to all those concerned with, and about, the design and construction of concrete structures which are appropriate to our time and effective, efficient and economic in performance and use.

Roy E. Rowe, President of CEB
René Walther, President of FIP
June 1991

INTRODUCTION

Nature of the Model Code

This document synthesizes scientific and technical developments over the past decade in the safety, analysis and design of concrete structures. It is intended to serve as a basis for the design of buildings and civil engineering works in structural concrete using normal-weight aggregates. Some of the detailed provisions given are only applicable under specified conditions. This Model Code does not attempt to cover particular types of civil engineering works (bridges, reservoirs, off-shore structures) nor does it include provisions against certain actions (seismic, impact or fire), these subjects being treated elsewhere in specific CEB Bulletins and FIP Publications.

By virtue of its international character, this document is more general than most national Codes and, since it is also a Model Code, it provides more detail to aid the drafters of those national Codes in their task of simplifying within their known constraints.

Nevertheless, it is meant to be operational with or without such further simplifications. In this regard the following breakdown of its content will be useful.

- Chapters 1–3 contain basic information serving both as a foundation for the subsequent chapters and as a source of data generally applicable in fields not directly covered by the Model Code. They thus provide an essential data bank for the designer and a basis for further development.
- Chapters 4–10 form the main operational part including specific provisions for the design of concrete structures.
- Chapters 11–14 relate primarily to the construction phase although additional sections on the design of precast structures are also included here.
- The appendices are technical sections associated with the application of this Model Code or for general use.

While this Model Code may be used as a basis for future harmonization of regulatory documents, some of its more innovative contents may, after further calibration, need additional elaboration and this activity will be continuing within CEB, with the outcome being appropriately published.

It is to be noted that the treatment of plain concrete is not fully given.

The format of the Model Code follows the CEB-FIP tradition.

- *On the right-hand side, the text*—the main provisions are presented in the logical sequence of topics. Structural requirements are stated followed by the relevant design criteria, i.e. appropriate engineering models and/or design rules; their application is intended to achieve the satisfaction of the relevant structural requirement. However, the fulfilment of the requirement may alternatively be achieved by means of models other than those given in the Code, provided that the designer is satisfied that adequate substantiating and well founded evidence exists for them in the international literature.
- *On the left-hand side, the comments*—explanations of provisions, particular sketches, alternative simplified rules, indicative numerical values, short justifications of options on the right-hand side, cross references to clauses of this Code or other relevant documents.

  In this connection the Model Code makes use of the following reference documents:

CEB-Bulletin d'Information 124/125 (1978): International System of Unified Standard Codes of Practice in Structures, Volume I: Common Unified Rules for Different Types of Construction and Materials (Joint Committee on Structural Safety–JCSS)
CEB-Bulletin d'Information 191 (1988): General Principles on Reliability for Structures – A commentary on ISO 2394 —approved by the Plenum of the Joint Committee on Structural Safety.

For the application of this Model Code it is assumed that an adequately high quality assurance level is maintained over the design and construction process; thus:

— design is carried out by appropriately qualified and experienced personnel; depending on the importance and complexity of the structure being designed, additional qualifications and/or design checks may be needed;
— construction materials and components are produced and used as specified in the standards and recommendations;
— construction is carried out by appropriately qualified and experienced personnel; depending on the importance and complexity of the structure, the Quality Management System will define required skills and experience of staff, supervision procedures etc;
— the structure, during its intended life, is used as foreseen in the design documents and it is appropriately maintained.

Background

The synthesis of experience and research output and its translation into practical documents for design has been the vocation of the CEB since its establishment; the long tradition of publication of such guidance (Bulletins, Guides, Manuals) periodically culminated in the production of Code-like Recommendations, 1964 and 1970, and, particularly, with the CEB-FIP Model Code 1978.

The impact of MC 78 was very considerable including its direct application, in one form or other, in some twenty-five national and regional Codes. Since 1978, the Model Code has aided the harmonization of the codification process as exemplified by the activities of the Commission of the European Communities (CEC), the Eastern Countries, the Nordic Building Regulations Committee (NKB) and members of the European Free Trade Association (EFTA). Indeed Eurocode 2 'Design of Concrete Structures, Part 1: General Rules and Rules for Buildings' used as its basic reference document the Model Code 1978.

With further developments in the understanding of the performance of concrete and the means of improving analytical and design techniques to reflect that understanding, the continuing work of CEB led naturally to the need to consider revising the existing Model Code. The procedure to be adopted was defined after much preliminary work, and following debates in the CEB Administrative Council and Plenary Sessions, which culminated in the setting up of a 'Committee for the Model Code 1990', with a remit and time scale, under the Chairmanship of Professor T.P. Tassios. This was in May 1987 with the Committee constituted as follows:

J. Appleton, T. Balogh, J. Calavera, J. Eibl, M. Fardis, H. Hilsdorf, M. Kavyrchine, G. König, F. Levi, U. Litzner, G. Macchi, H. Mathieu, M. Miehlbradt, H.R. Müller, J. Perchat, U. Quast, P. Regan, S. Rostam, E. Siviero, E. Skettrup, G. Somerville, G. Thielen, B. Westerberg, M. Wicke, *Ex officio members*: R.E. Rowe,

Y. Saillard, R. Molzahn (until October 1989), R. Tewes (since November 1989).

Experts invited by the Council who participated were

M.A. Chiorino, R. Eligehausen, R. Favre, E. Grasser, P. Matt, M. Menegotto, J. Schlaich, J. Walraven, E. Wölfel.

The work of CMC 90 and its Editorial Board led to the first complete draft of MC 90 (CEB-Bulletins d'Information 195 and 196), which was circulated for comments by the National Delegations and individual members of the CEB. These comments were compiled and, together with the initial reactions of the CMC to them, appeared in Bulletin 198, as an Addendum to the First Draft. All of this material was discussed at the 27th Plenary Session in Paris in 1990.

The Plenary Session approved the Model Code subject to certain agreed modifications and instructed the CMC to finalize the document under the authority of the Administrative Council. All the comments have been carefully considered by the CMC, at its meetings and those of its Editorial Board, and the final text has been achieved on a consensus basis within the Committee.

Thus, after five years of continuous effort, the final version of CEB-FIP Model Code 1990 was published in Bulletins 203, 204 and 205 for ratification at the 28th Plenary Session in Vienna, September 1991.

It is worth noting that with the publication of both Eurocode 2 and the Model Code 1990 at about the same time, the latter will be of special interest and value in commenting on the use of the former during the trial period.

Main innovative aspects

Broader field of application. The CEB-FIP MC 90 has certain characteristics.

a) In its concept and generality, it covers different types of structure as well as buildings. The general content of its chapters has a more fundamental character and is performance orientated.
b) The fundamental character of the contents covers
   i) the description of the mechanical behaviour of reinforced concrete, the materials and their composite behaviour;
   ii) a coherent framework for the subsequent chapters with appropriate simplifications of the basic models;

   and enables designers of exceptional structures, or coping with design situations not covered by this Code, to apply these basic models with confidence but, obviously, with appropriate judgement.

Hence, MC 90 is intended to be operational for normal design situations and structures.

Extensive presentation of properties of concrete. A comprehensive and detailed compilation of the mechanical and other properties of concrete is given. This provides a scientifically sound data base, directly applicable in modern designs, and of general validity.

Generalized behaviour models of reinforced concrete. A series of fundamental behaviour models is included in the Code, as a basis for the dimensioning criteria used in subsequent chapters where, when necessary, further simplifications and adaptations are made.

Broader structural analysis provisions. All types of structural analysis are foreseen. Emphasis is, however, given to linear and non-linear analysis. Additional information is included on the analysis of two-dimensional elements.

Continuity and consistency of models for dimensioning. While the resistance model for axial action effects remains practically the same as in MC 78, a clearly more rational approach is adopted for dimensioning.

Instead of critical 'cross-sections' and separate verifications for axial (M, N) and shear (V, T) action effects, the concept of critical regions is introduced and their global resistance (against M, N, and V, T) is sought. To this end, continuous fields of compressive and tensile forces are considered; their direct effects on the extreme chords and the web of the building elements are examined. Thus shear resistance modelling becomes more rational, and the M, N or M/V and T 'interactions' have a better treatment.

'Discontinuity' critical regions or entire two-dimensional structural elements (plates), where plane sections do not remain plane, are treated in a rational way.

On buckling, several modifications and improvements are introduced such as the 10% criterion, the slenderness bounds, the definition of eccentricity, the simplified calculation of second order effects, and the lateral buckling of beams.

Computational verification against fatigue. Several possible levels of the treatment of the ULS of fatigue are included.

Improvements in Serviceability Limit States verifications. Crack-width control remains as a criterion. In general, crack width limitation is achieved by appropriate detailing based upon stress control. Calculations are only required for special conditions of sensitive steels and corrosive environments; improved modelling is used for this purpose and extended to prestressed concrete. Control of deflections is achieved using methods consistent with the application of the basic model of previous chapters. However, simplified rules are given for most practical cases.

Broader handling of prestressed concrete. Compared with Model Code 1978, the present Code contains more information on prestressed concrete and presented in a more systematic way.

Design for durability. A separate chapter is given on the design of durable concrete structures, reflecting the very considerable emphasis that must be placed on this aspect by designers and, indeed, all those associated with the creation and use of structures.

Design of precast structures. Extensive provisions, both conceptual and computational, for the design of precast structures are given together with certain construction requirements.

Construction aspects. There is a systematic consideration of the construction stage and its interaction with the design stage. More detailed recommendations on the execution of concrete works are now included.

Design by testing. Special attention has been given to this aspect of design and operational provisions are included in an appendix.

Position of the Model Code

The position of the Model Code in the broad spectrum of national and international regulatory documents should be clearly understood.

It is a developing 'model' intended for the widest possible use in the design and construction of concrete structures directly by designers for individual structures or, more often, as a guide to those responsible for drafting the various national and international Codes. As such, it goes into considerable detail, addresses a wider audience with many different traditions and technical practices and thus, necessarily, adopts a more theoretical approach to achieve the optimum clarity for ease of understanding. Also, to this end, it offers a detailed Commentary and several Appendices.

Thus MC 90 should provide a sound background, or source document, with which to refine, improve and, possibly, extend the existing national and international documents.

Future action

Following the practice adopted with MC 78, it is intended to carry out a series of trial calculations for a limited range of structures to show applications of the Model Code; these should be of particular value in comparison with the results obtained using other documents. Equally, in time, Manuals and Bulletins on specific aspects will be produced since the vocation of the CEB leads naturally to these.

R.E. Rowe, President of CEB

T.P. Tassios, Chairman of CMC 90

June 1991

CONTRIBUTORS

The various technical committees of CEB and FIP have contributed over the years to the input to the Model Code. A complete list of those who contributed in the various Commissions and Task Groups was given in CEB-News 81. The constitution of Commissions and Task Groups having a direct input from 1987 to the time of the production of MC 90 was as follows

MODEL CODE EDITORIAL BOARD (MCEB)
Chairman: T.P. Tassios
Members: U. Litzner, M. Miehlbradt, R. Molzahn (until October 1989), J. Perchat, E. Siviero, E. Skettrup, R. Tewes (since November 1989).

COMMITTEE FOR THE MODEL CODE 90 (CMC)
Chairman: T.P. Tassios
Members and invited experts: J. Appleton, T. Balogh, J. Calavera, M.A. Chiorino, J. Eibl, R. Eligehausen, M. Fardis, R. Favre, E. Grasser, H. Hilsdorf, M. Kavyrchine, G. König, F. Levi, U. Litzner, G. Macchi, H. Mathieu, P. Matt, M. Menegotto, M. Miehlbradt, H.R. Müller, J. Perchat, U. Quast, P. Regan, S. Rostam, J. Schlaich, E. Siviero, E. Skettrup, G. Somerville, G. Thielen, J. Walraven, B. Westerberg, M. Wicke, E. Wölfel. *Ex officio members*: R.E. Rowe, Y. Saillard, R. Molzahn (until October 1989), R. Tewes (since November 1989).

Special acknowledgement is due to the collaborators of the Technical Secretariats of Model Code 90:

in Athens:	M. Chronopoulos, K. Trezos, S. Tsoukantas, E. Vintzeleou
and in Copenhagen:	S. Pedersen, E. Skettrup

CEB Commissions and Related Task Groups

Commission I: RELIABILITY AND QUALITY ASSURANCE
Reporter: H. Mathieu, *Co-reporters*: L. Östlund (until 1989), G. König
Members: J. Anton Corrales, W. Cranston, A. Giuffre, G. König, B. Lewicki, T. Monnier, J. Nielsen, R. Rackwitz, R. Suter
Associate Members: R. Baus, W. Bosshard, F.L.B. Carneiro, S. Chatterjee, D. Dekanovic, J. Ferry Borges, G. Grattesat, M. Kersken-Bradley, M. Leporati, A.G. Meseguer, J.F.A. Moore, Y. Ozaka, K. Szalai, M. Tichy, E. Thorenfeldt, B. Pilon

TG I/3: Quality Assurance for Building. A. van den Beukel, H. Blaut, J.M. Bueno, J. Buchanan, N. Marsellos, M. Matousek, A.G. Meseguer (*Reporter*), W.H.A. Weiss

Commission II: STRUCTURAL ANALYSIS
Reporter: G. Macchi, *Co-reporter*: J. Eibl
Members: J.P. Boutin, E. Cansado de Carvalho, V. Cervenka, G. Creazza, H.B. Gould, V. Kalevras, P. Lenkei, A. Mari, M. Miehlbradt, T. Monnier, A. Recuero, S. Sørensen
Associate Members: M. Acic, J. Appleton, A. Cauvin, M.A. Chiorino, M.Z. Cohn, J.L. Costaz, H. Duddeck, R. Eligehausen, H. Falkner, J.P. Guitierrez, S. Krylov, W. Kuczynski, G. Mehlhorn, P. Regan, K. Schäfer, M. Vaz de Silva

Commission III: BUCKLING AND INSTABILITY
Reporter: U. Quast, *Co-reporter*: F. Moran
Members: K. Aas-Jakobsen, W. Cranston, F. Mola, U. Litzner, E. Skettrup, E.A. Tchistiakov, M. Virlogeux
Associate Members: G. Creazza, H. Corres, E. Dulacska, B. Fouré, J.G. MacGregor, S.Ľ. Groot, K. Kordina, W. Kukulski, M. Menegotto, Z. Prascevic, K.S. Virdi

Commission IV: MEMBER DESIGN
Reporter: P. Regan, *Co-reporter*: E. Grasser
Members: J. Bobrowski, M.W. Braestrup, Y. Goushcha, G. Mancini, P. Marti, J. Mathivat, W. Moosecker, K. Reineck, P. Ritz, F.J. Rui Wamba, M. Teutsch, J.C. Walraven
Associate Members: R. Chaussin, M.P. Collins, H. Corres, K. Dabrowski, P. Gambarova, F.K. Garas, R. de Keyser, J. Korda, L. Lima, Z. Maric, F. Motta, J. Niwa, A. Plakas, J. Prochazka, A. Sarja, H. Seiler, L. Shehata, R. Skrucany, P. Spyropoulos, I. Tertea

TG IV/1: Shear in Prestressed Concrete Members. P. Gergely, R. De Keyser, J. Mathivat, W. Moosecker, J. Niwa, K.H. Reineck, J.C. Walraven (*Reporter*), F. Watanabe

TG IV/3: Punching. J. Bobrowski, M.W. Braestrup, W. Dilger, S. Kinnunen, E. Lima, G. Madaras, A. Muttoni, D. Nölting, P. Regan (*Reporter*), K.H. Reineck, H. Seiler

Commission V: SERVICEABILITY
Reporter: M. Wicke, *Co-reporter*: Å. Holmberg
Members: A.W. Beeby, R. Favre, A. Kudzis, P. Marro, D. van Nieuwenburg, P. Schießl, W.B. Wilson, P. Xercavins
Associate Members: T. Brøndum-Nielsen, E. Cosenza, G. Deak, M. Dragosavic, H. Falkner, F.K. Garas, A. Giovambattista, E. Giuriani, R. Green, J. Hajek, M. Ivkovic, G. König, M. Koprna, V. Monteiro, Z. Perisic, F.S. Rostasy, R. Skrucany, H.T. Solanki, J.A. Torroja, E. Wölfel, P. Yannopoulos

TG V/5: Minimum Reinforcement. A.W. Beeby (*Reporter*), Å. Holmberg, J.P. Jaccoud, P. Marro, Z. Perisic, P. Schießl

TG V/6: Tension Stiffening. R. Favre, G. Fritsche, E. Giuriani, G. Hartl, Å. Holmberg (*Reporter*), M. Koprna, D. van Nieuwenburg, Z. Perisic, M. Wicke, P. Yannopoulos

TG V/7: Deformations. A.W. Beeby, P.G. Debernardi, H. Falkner, R. Favre (*Reporter*), J. Fernandez Gomez, G. Fritsche, M. Koprna, G. Thielen

Commission VI: DETAILING
Reporter: J. Perchat, *Co-reporter*: G. Somerville
Members: A. Broucke, T. Hagberg, B. Chr. Jensen, A. Migliacci, G. Rehm, F.J. Rui-Wamba, J. Schlaich, R. Tepfers, J. Zielinski
Associate Members: V. Acanski, B.K. Bardhan Roy, J. Bradac, E. Giuriani, G. Gross, H. Heimerdinger, Å Holmberg, S. Jasman, J. Mathivat, C. Modena, S. Morita, K. Otsuka, J. Rodriguez, R. Tewes, G. Toniolo, V. Urban, D. Weischede, R. Whittle, A. Windisch, L. Zikeev

TG VI/1: Anchorage Zones. G. Balazs, J. Cairns, R. Eligehausen, P. Gergely, E. Giuriani, J.O. Jirsa, P. Gambarova, A. Skudra, S. Stöckl, G. Tassi, R. Tepfers (*Reporter*), J.A. den Uijl, V. Urban, E. Vintzeléou, A. Windisch, J.L. Zielinski

Commission VII: REINFORCEMENT: TECHNOLOGY AND QUALITY CONTROL
Reporter: J. Calavera, *Co-reporter*: G. Rehm
Members: K. Bay, L. Bodo, B. Creton, A. Delibes, A. Ghio, R.I. Lancaster, B. Törnwall
Associate Members: R. Eligehausen, J.D. Holst, D. Jungwirth, E.J. van Koot, G. Kulessa, I. Podhorski, D. Russwurm, A. Zingali

TG VII/5: Ductility Requirements for Reinforcing Steel. R. Eligehausen (*Reporter*), F.B.J. Gijsbers, T. Hagberg, F. Herkommer, J.D. Holst, J. Leon, G. Macchi, K. Schäfer, G. Thielen

TG VII/6: Recommendations for Mechanical Splices. L. Bodo, G.M. Bowmer, A. Delibes (*Reporter*), R.I. Lancaster, H. Martin, H. Ortega, V. Rios, J.B. Ripoll, R. Tepfers

TG VII/7: Spacers and Covers of Reinforcement. Å. Holmberg, J.D. Holst, R.I. Lancaster (*Reporter*), D. Jungwirth

Commission VIII: CONCRETE: TECHNOLOGY AND QUALITY CONTROL
Reporter: H.K. Hilsdorf, *Co-reporter*: H. Lambotte
Members: A. Delibes, P. Dutron, O.E. Gjørv, L. Goffi, B.G. Hellers, V. Hermann, Z. Kormos, K. Newman, P. Nischer, S. Rostam, A. Short, D.C. Spooner, J. Trinh, V. Volta, J. Znidaric
Associate Members: G. Tognon, M. Valente

CEB General Task Groups
GTG 9: Evaluation of Time-Dependent Behaviour of Concrete. P. Acker, M.A. Chiorino, H. Hilsdorf (*Reporter*), J.M. Illston, D. Jungwirth, P. Lenkei, H.S. Müller, L.F. Nielsen, C.D. Pomeroy, H. Trost

GTG 13: Design Value Format. M. Diamantidis, M. Kersken-Bradley, G. König (*Reporter*), H. Madsen, L. Östlund, P.E. Pinto, R. Rackwitz, J. Schneider, K. Skov, V. Solana, G. Thielen

GTG 14: Concrete Structures under Impact and Impulsive Loading. W. Ammann, P. Barr, C. Berriaud, J. Bobrowski, K. Brandes, D. Chauvel, J. Eibl (*Reporter*), F.K. Garas, I. Holand, S.H. Perry, H.W. Reinhardt, M. Romano, E. Rüdiger, G. Spaethe, G. Thielen

GTG 15: Fatigue of Concrete Structures. J.A.W. Cornelissen, R.P. Frey, K. Gylltoft, G. König, J.M. Lovegrove, D.G. Owen, A. Pitonak, J. Sharp, H. Stemland, R. Sturm, R. Tepfers, G. Thielen, G.P. Tilly (*Reporter*)

GTG 16: Structure and Development of Future Technical Guidance Documents. J. Ferry-Borges, F. Levi, H.U. Litzner, P. Lüchinger, H. Mathieu, T. Monnier, H. Motteu, J.F.A. Moore, L. Östlund, J. Perchat, P.E. Pinto, P. Spehl, G. Thielen (*Reporter*)

GTG 17: Design of Prestressed Structures. P. Acker, J.J. Arenas, T. Balogh, B.K. Bardhan Roy, L. Bodo, T. Brøndum-Nielsen, J.J. Eberwijn, C. Freyermuth, M. Kavyrchine (*Reporter*), M. Lorrain, G. Mancini, M. Miehlbradt, P. Ritz, E. Segre, L. Taerwe, J. Trinh, M. Virlogeux, E. Wölfel

GTG 18: Prefabricated Structures. A. Van Acker, G.S.T. Armer, M. Catania, B. Engstrom, N.O. Karstoft, M. Kavychine, F. Krueger, B. Lewicki (*Reporter*), G. Menditto, M. Menegotto (*Reporter*), O. Olesen, A. Pakvor, S. Tsoukantas, A. Tydeman

GTG 19: Diagnosis and Assessment of Concrete Structures. F. Baquedano, C. Bois, R.D. Browne, R.E. Carrier, P. Engelke, J. Ferry-Borges, G.

Frohnsdorff, H. Geymayer, H.H. Gotfredsen, N. Hutzen-Andersen (*Reporter*), M. Ladner, G.P. Mallett, D.G. Manning, M. Mele, Th. Monnier, A. Samuelli Ferretti, E. Gonzales Valle

GTG 20: Durability and Service Life. M. Adam, M.C. Andrade, R.F.M. Bakker, A.W. Beeby, G. Hartl, H.O. Madsen, R. Marino, D. van Nieuwenburg, S. Rostam (*Reporter*), P. Schießl, L. Sentler, M. Valente

GTG 21: Re-Design of Concrete Structures. J. Appleton, H.H. Christensen, M. Chronopoulos, J. Korda, V.A. Klevzov, T. Monnier (*Reporter*), P. Napoli, D. Poineau, E.F. Radogna, J.L. Ramirez, R. Suter

FIP Commissions

FIP 2 on Prestressing Materials and Systems. MC 90 working group: G. Hampejs, M. Miehlbradt, U. Morf, H.R. Müller (*Reporter*), F.S. Rostásy

FIP 3 on Practical Design. MC 90 working group: J. Almeida, J. Appleton (*Reporter*), T. Friedrich, H.-R. Ganz, M. Miehlbradt, B. Westerberg

FIP 4 on Practical Construction. MC 90 working group: J. Groenveld, J. Lindgren, G.-S. Littlejohn, P. Matt, M. Miehlbradt (*Reporter*), W. Podolny Jr., A. Windisch, E. Wölfel

FIP 5 on Prefabrication. MC 90 working group: A. van Acker, B. Engström, M. Menegotto (*Reporter*)

CONTENTS

PART I DESIGN INPUT DATA

1. BASIS OF DESIGN

1.1. REQUIREMENTS AND CRITERIA

1.1.1. General requirements

Structures should with appropriate degrees of reliability, during their construction and whole intended lifetime, perform adequately and more particularly

- withstand all actions and environmental influences, liable to occur
- withstand accidental circumstances without damage disproportionate to the original events (this is called the insensitivity requirement).

Their intended lifetime should be specified by the client.

Particular requirements liable to be codified in other guidance documents supplementing this Model Code may refer to

- the particular kind of the structure; an example is a requirement on tightness of a tank
- some hazards; an example is paraseismic requirements; more generally dynamic not quasi-static actions should be covered by such documents
- some particular techniques used for the structure; examples are segmental construction and construction with launching.

General and particular requirements may be supplemented by other requirements specified by the authority, e.g. for the impact of the structure or of its erection on the environment, and by the client, e.g. for economy or for aesthetic aspects.

1.1.2. Criteria

The criteria of compliance with the requirements comprise two categories of measures:

- appropriate design procedures, including measures to facilitate inspection and maintenance of vital structural elements during the lifetime of the structure
- quality assurance measures intended to prevent and eliminate human errors.

The intended lifetime is directly considered for measures relating to durability. For the main dimensioning and for reliability verifications, lifetimes are for practical purposes substituted by reference periods t_R.

In the design procedures, various design situations should be identified as relevant, by distinguishing

- persistent situations
- transient situations
- accidental situations.

The various types of design situations are defined by clause 3.2.2 of CEB Bulletin 191.

Generally a reference period of 50 years is adopted for persistent situations, 1 year for transient situations, and accidental situations are considered to be instantaneous.

In many cases judgement is necessary to supplement codified provisions, in order to identify those design situations that are to be taken into account for a particular structure.

The insensitivity requirement is defined in section 2.1 of CEB Bulletin 191. Clause 3.2.3 of this Bulletin gives some guidance on the choice of a design procedure appropriate to limit damages liable to result from identified or unidentified hazards.

1.1.3. Degrees of reliability

In principle a degree of reliability should correspond to a statistically determined rate of failures with regard to the whole set of requirements (for a set of similar structures under similar conditions). For a given structure this rate should be represented by an assessed probability referable to a given period of time. In the present state of knowledge, degrees of reliability can only correspond to individual verifications and can be associated with nominal probabilities which do not account for gross errors; therefore, they do not represent any actual rate of failures.

The degree of reliability is a function of the design procedures (models and values of actions included) and quality assurance measures associated with the design and construction process.

The design data given in this Model Code are only partial. To supplement them on actions it is possible to refer either to

- Appendices 2 and 3 to Volume 1 of Model 78, or
- to regional or national codes.

It is at present commonly agreed that for the intended degrees of reliability accepted in Western Europe, the 200 years mean return period of characteristic variable actions (envisaged in the Appendices of CEB Bulletin 191) should be reduced to approximately 100 years for the most common climatic actions, in conjunction with this Model Code.

By acting on design and/or quality assurance, degrees of reliability can be indirectly differentiated to some extent, on the basis of criteria related either to the construction works or to the structural member (e.g. because the expected consequences of a failure would be unequal), or to the causes of a possible failure (e.g. a degree of reliability with regard to earthquake or to permanent loads cannot be the same).

In some cases only reliability differentiation measures are defined in this Model Code (see subsection 1.6.1, which is limited to reliability differentiation by the design). Quality assurance measures are still little codified at the international level, but very significant complementary differentiations already result from different quality assurance measures (commonly more or less supplemented or substituted by common practices), adopted at the national level and possibly distinguished depending on the type of building or civil engineering works. For more details, see CEB Bulletin 202.

1.1.4. Quality assurance measures

In order that the properties of the completed structure be consistent with the requirements and the assumptions made during the planning and the design, adequate quality assurance measures shall be taken.

The philosophy of quality assurance is presented in section 2.5 of CEB Bulletin 191. Fundamentally quality assurance implies 'thinking in advance' for the whole building process.

The possibilities to codify quality assurance are limited. However ISO 8402 and 9001 to 9004 define terminologies and concepts and provide guidelines intended for a series of contractual situations. In construction activities various contractual situations between clients, contractors, subcontractors and suppliers are met; the choice of the relevant standard should be made accordingly.

These standards are essentially intended for industrial mass production; some amendments are necessary to use them for construction activities.

Quality assurance measures are both technical and organizational. Some common cases excepted, they should be specified in a general Quality Assurance Plan, which shall identify the key elements necessary to provide fitness of the structure and the means by which they are to be provided and measured, with the overall purpose to provide confidence that the realized project will work satisfactorily in service—fulfilling intended needs.

Each party involved in the realization of a project should establish and implement a quality assurance plan for its participation in the project. Suppliers' and subcontractors' activities shall be covered. The individual Quality Assurance Plans shall fit into the general Quality Assurance Plan.

A Quality Assurance Plan shall define the tasks and responsibilities of all persons involved, adequate control and checking procedures and the organization and filing of an adequate documentation of the building process and its results.

The extent of Quality Assurance Plans should depend on the type and size of the project. In common cases, where current good practice of participants is enough to assure quality, specific Quality Assurance Plans may be unnecessary.

The Quality Assurance Plans and quality controls are dealt with in chapter 12 of this Model Code. The assessments of aspects pertaining to human behaviour (motivation of participants and possibly their skill, qualification, etc.) are widely subjective and cannot be codified. For guidance, refer to CEB Bulletins 157 and 184.

Checklists may be a useful tool for establishing and implementing a Quality Assurance Plan. See examples in CEB Bulletin 184.

1.1.5. Design procedures

Design procedures consist of

- calculations based on analytical models and possibly on experimental models
- applying complementary rules dealing with detailing, limit measures and special provisions.

Detailing, limit measures and special provisions supplement the use of models for various purposes

- to avoid superfluous calculations, e.g. for alternate moment areas
- to satisfy the insensitivity requirement (see sections 2.1 and 3.2.3 of CEB Bulletin 191) with regard to unidentified or hardly quantified hazards; examples are minimum resistance to lateral forces, multiple load paths and ties between structural elements
- to ensure the validity of calculation models, e.g. by minimum ratios of reinforcement
- to ensure good execution and/or durability, e.g. by maximum ratio of reinforcement and by rules for concrete cover.

1.2. LIMIT STATES

1.2.1. General

Limit states are states of the structure defining unfitness for use. They are generally classified as

- ultimate limit states
- serviceability limit states.

Some of them, in turn, may be further subdivided as mentioned in the following clauses.

The limit states either refer to the entire structure, to structural elements or to local regions of elements.

This is only a simplified classification. In very particular cases some intermediate limit states should be considered. For more details, see section 3.1 of CEB Bulletin 191.

1.2.2. Ultimate limit states

Attainment of the bearing capacity of a structural part or of the structure as a whole is classified as an ultimate limit state.

The ultimate limit states considered in this Code are distinguished by the consequences of their exceedance

- loss of static equilibrium (see subsection 1.6.5) when a structural part or the whole structure (considered as rigid bodies) is overturned, is lifted or slides
- exceedance of resistance of one or more critical regions of the structure.

The criteria for resistance refer to the structural behaviour of one or several isolated critical regions of the structure. Under certain conditions, the behaviour of a set of critical regions is considered globally, as a transformation of the structure into a mechanism (see clause 1.6.2.2).

In practice each criterion generally refers to damaging events which occur once only (see clause 5.4.1.4) and result (see subsections 1.6.2 and 6.3.2 to 6.3.5) in a combination of

- axial action effects (bending moments and axial forces)
- tangential action effects (shear forces, torsional moments), including effects on bond and anchorage (section 6.9)

or, in some cases, in each of them separately.

Where second-order effects are to be taken into account in a simple way, it is customary to consider the specific ULS of buckling (sections 1.6.3 and 6.6). Under essentially repetitive loading, the particular ULS of fatigue (sections 1.6.4 and 6.7) is considered.

1.2.3. Serviceability limit states

The serviceability limit states associated with the general requirements refer to

- limited local structural damage such as excessive cracking or excessive compressive stresses, producing irreversible strains and microcracks; see sections 1.6.6, 7.3 and 7.4
- deformations which produce unacceptable damage in non-structural elements or excessively affect the use or appearance of structural or non-structural elements; see sections 1.6.6 and 7.5
- vibrations resulting in discomfort, alarm or loss of utility; see sections 1.6.6 and 7.6.

Large and repeated exceedance of such limit states may result in ULS; however such an exceedance is usually covered by the other verifications.

Such damage reduces the durability of the structure and may also affect its efficient use (tanks, pipes, canals) or the appearance. In many cases, the risk of damage is indirectly covered by ultimate limit state verifications or by detailing.

The conditions to be fulfilled for deformation verifications are associated with the type of building or of civil engineering works. They are often as a simplification, substituted by rough approximations.

Although such limit states may be characterized by the magnitude of the vibrations, they are commonly indirectly covered by limiting the fundamental period of vibrations of the structure (or some of its members), in comparison to the expected period of the cause of the vibrations.

Other serviceability limit states may be associated with particular requirements (see subsection 1.1.1).

Rare, frequent and quasi-permanent combinations of actions defined in clause 1.6.6.5 are associated with the relevant limit states.

In some cases, this model may be based on experimental tests made for the particular design or on a combination of testing and analytical calculations (see Appendix c).

Variables taking into account chemical and biological influences on material properties are defined in subsection 1.5.2 as exposure classes.

These reliability margins, defined in section 1.4, seem to cover the whole set of uncertainties; however, a part of the model uncertainties is commonly directly covered by the codified model itself.

This does not exclude that some actions (e.g. shrinkage) can be negligible in particular cases. What is to be considered as one individual action is defined in the corresponding standard and explained in clause 4.2.1 of Bulletin 191. For prestress, see subsection 1.4.3 of this Model Code.

For these fundamental geometrical quantities, tolerances should be carefully fixed (see subsection 1.4.5) and controlled. For the other geometrical quantities, tolerances generally reflect usual practice. For all geometrical quantities it would not be realistic to specify tolerances less than twice the mean deviation expected or minimum attainable. As a consequence, tolerances may, according to the case, be either the basis for the design or necessary complements to the design.

See sections 4.1 and 6.1 of Bulletin 191. To identify and select the other relevant fundamental variables is one of the major responsibilities of a designer who faces a problem having some unusual aspects.

1.3. DESIGN METHODS

For each relevant limit state verification, a design model should be set up from

- an appropriate description of the structure, of its constitutive materials and of its environment
- corresponding behaviour models for the whole or parts of the structure, related to the relevant limit states
- models describing the actions and how they are imposed.

In the model, variables are represented by design values for the relevant limit state.

For some variables, designated as fundamental basic variables, design values include reliability margins.

For other variables, whose dispersion may be neglected or is covered by a set of partial factors, they are normally taken equal to their most likely values.

In this Model Code the following are considered as fundamental basic variables

- actions (F), unless otherwise specified in particular clauses
- some geometrical quantities (a), as listed in clause 1.4.5.1
- strengths (f), unless otherwise specified, and other material properties (e.g. creep and friction coefficients) where specified.

Occasionally other variables should be considered as fundamental variables. This may be the case for the numbers of repetitions of loads in fatigue verifications.

Although limit state equations representing different limit state conditions are various, the corresponding design conditions may often be written in the general form

$$S < R \quad \text{or} \quad s \subset R^* \tag{1.3-1}$$

where R^* defines a safe domain in which the vector s should be included; the assessment of S may be referred to as overall analysis, while the assessment of R may be referred to as local analysis.

See clause 3.2.1 of Bulletin 191 and the various clauses of section 1.6.

According to the limit state under consideration, the limit state equations may have to be formulated

- either in the space of internal and external moments and forces and directly presented as in eq. (1.3–1), or
- in the space of forces, as

$$F_S < F_R \tag{1.3-2}$$

(F_R being for example a carrying capacity), or

- in the space of stresses as

$$\sigma < \alpha f \tag{1.3-3}$$

- or in the space of geometrical quantities, as

$$e < C \tag{1.3-4}$$

(C being for example a deflection or a crack width).

1.4. METHOD OF PARTIAL FACTORS

1.4.1. General

The partial factor format separates the treatment of uncertainties and variabilities originating from various causes. In the verification procedure defined in this Model Code the design values of the fundamental basic variables (defined in section 1.3) are expressed as follows.

This separation is theoretically not correct, and in practice not complete, because the various factors are not mutually independent. Hence, constant values given in partial factors should be considered as approximations having limited fields of validity. This approximation of using constant values for partial factors may not apply in the following cases

- non-linear limit state equations
- mutually correlated variables
- design by testing.

(a) Design values of actions are generally expressed as

$$F_d = \gamma_F F_{rep}$$

where

F_{rep} are representative values of actions, defined in subsection 1.4.2

γ_F are partial safety factors.

However some actions (e.g. non-closely bounded hydraulic actions) should be expressed in another way, as mentioned in section 4.1 of Bulletin 191. Furthermore for verifications relating to fatigue and vibrations, the format is generally different (see clauses 1.6.4. and 1.6.6.2(c)).

As explained in sections 6.2 and 6.6 of Bulletin 191, γ_F may in some cases be substituted by two partial factors: γ_{Sd} applicable to the action-effect and γ_f applicable to F_{rep}.

For practical applications see clause 1.6.2.4.

(b) Design values of strengths are generally directly expressed as

$$f_d = f_k/\gamma_M$$

where

f_k are characteristic values of strengths, defined in subsection 1.4.4

γ_M are partial safety factors.

For material properties other than strengths (e.g. modulus of elasticity, creep, friction coefficients) see the relevant parts of chapters 2, 3 and 4.

Numerical values of γ_M may be different in various parts of the limit state equation, especially for the calculations of S and R; for example (see clause 1.6.2.4(b)) γ_M may be reduced for the assessment of S by a non-linear analysis.

For concrete and steel, γ_M usually covers the deviations of structural dimensions not considered as fundamental variables and includes a conversion factor η converting the strength obtained from test specimens to the strength in the actual structure. For practical application, see clause 1.6.2.4(b).

Other factors, applied to f_d or implicitly included in design formulae, take into account the variations of strength due to non-standardized loading conditions. For practical application, see section 3.5.

As explained in sections 6.3 and 6.6 of Bulletin 191, γ_M may in some cases be substituted by

- one or two partial factors γ_{Rd} applicable to the resistance,
- and a partial factor γ_m applicable to f_k.

(c) Design values of geometrical quantities to be considered as fundamental basic variables are generally directly expressed by their design values a_d.

Liquid levels representing hydraulic actions should in some cases be expressed as $a_k + \Delta a$, where a_k is a characteristic level and Δa an additive or reducing reliability margin.

1.4.2. Representation of actions

1.4.2.1. Definitions and classifications

Actions should be classified as

- direct or indirect
- permanent, variable or accidental
- fixed or free
- static, quasi-static or dynamic
- closely bounded or non-closely bounded.

For practical classifications of the most common actions, see the relevant Appendices to ISO 2394 and Bulletin 191.

Permanent actions, selfweight included, although usually classified as fixed, may have to be considered as partially free where the effects are very sensitive to their variation in space, e.g. for static equilibrium and analogous verifications (see subsection 1.6.5).

Dynamic (not quasi-static) actions are not dealt with in this chapter.

Reactions, mainly on supports, should also be distinguished from directly imposed actions. Although they are taken into account like actions for some verifications, they are in reality effects of actions and may need specific reliability measures in design.

Soil reactions, e.g. soil pressure underneath foundation slabs or footings, are strongly influenced by soil-structure interaction. They should be determined by analysis; but the result should commonly be considered widely uncertain, especially the distribution in space.

1.4.2.2. Representative values

(a) Permanent actions

Each permanent action is represented by a single representative value G if at least one of the following conditions is satisfied

- the variability of the action in time and with regard to the design is small
- the influence of the action on the total effect of the actions is small
- it is evident that one of the two representative values (the upper or the lower) governs all parts of the structure.

In the first two cases, G is considered as a mean value and should be calculated from nominal dimensions. In the third case it is defined as G_{sup} or G_{inf}.

The difference between G_{sup} or G_{inf} and G_m should not exceed $0.1\,G_m$. For some types of prestressed structures this maximum acceptable difference may have to be reduced to $0.05\,G_m$.

In the other cases, two representative values (upper and lower, G_{sup} and G_{inf}) should be defined, taking into account variations which can be foreseen.

This case is mainly applicable to finishes and equipment. G_{sup} and G_{inf} may normally be defined as corresponding to 0.95 and 0.05 fractiles plus (or minus) the expected variation in time of G_m.

Nominal numerical values of densities are given in subsection 2.1.2 for plain, reinforced and prestressed concrete, and in ISO 9194 for other materials. For future possible permanent equipment an upper value should be specified.

The representative values of the prestress are defined in subsection 1.4.3.

(b) Variable actions

Each variable action may be represented by

- characteristic value Q_k
- combination value $\Psi_o Q_k$
- frequent value $\Psi_1 Q_k$
- quasi-permanent value $\Psi_2 Q_k$.

For the most common variable actions these values are given in standards or codes associated with the same γ_F values as in this Model Code. It is also possible to refer to Appendices 2 and 3 to Volume I of Model Code 78 (see 1.1.3 above). Ψ values depend on the model of the action.

In particular cases (e.g. temperature and possibly hydraulic actions) upper and lower values should be distinguished for the three first representative values.

Besides, for some variable actions, specific representative values are defined for fatigue verifications.

These values are associated with the methods of verification defined in clause 1.6.4.2(d).

(c) Accidental actions

Each accidental action can be given by a single representative value, which is usually the design value A_d.

These values are normally defined by the competent public authority or by the client and correspond to the values beyond which a high probability of integrity of the structure can no longer be assured.

1.4.2.3. Load arrangements

For each free action, different load arrangements should be defined.

See subsection 4.2.3 of CEB Bulletin 191.

These load arrangements are sometimes defined in the load standards. If several actions are free, the load cases (fixing the arrangements of all actions by taking into account their compatibility) are sometimes defined in the same documents.

1.4.3. Representation of prestress

Precamber, i.e. a permanent deformation of the concrete structure imposed by jacks during execution, is not treated in this Model Code. Preflexed steel profiles are not covered either.

Stays, i.e. cables in which the tension is mainly due to the permanent weight of the stucture, are not treated in this Model Code.

For more details on the content of this subsection and for its background refer to the report 'Reliability problems associated with uses of prestress' in CEB Bulletin 202.

1.4.3.1. Definition and classification

A tendon or a set of tendons is prestressed if it is subjected to permanent tensile stresses and strains, due to external forces purposely exerted on it.

Generally, during prestressing, the external forces are imposed and the associated elongations of the tendons are controlled.

As defined in the following chapters, the various parts of the calculations (e.g. for the assessment of losses of prestress, for structural analysis and for verifications with regard to various limit states) have to refer to different indicators consistent with these parts of the calculations. These indicators may be, e.g. force or prestrain, with or without the effect of permanent actions. See especially subsection 4.2.1. An example is given in clause 1.4.3.3.

The various types of prestress and anchorages are listed in section 4.1.

In the most common cases the action effects due to prestress can be classified as

- local, i.e. where only the action effects due to prestress are significant
- isostatic (in statically determinate structures)
- isostatic and hyperstatic (in statically indeterminate structures).

Mainly in the vicinity of anchorages and at points where the tendons change their direction.

The distinction between isostatic and hyperstatic effects is not possible for slabs and shells.

1.4.3.2. Representative values of prestress considered as an action

At given time t and abscissa x (or arc length) the prestressing force $P(x, t)$ is equal to the total force $P(0,0)$ imposed at an active end of the set of tendons, minus losses $\Delta P(x, t)$. Losses can be classified as immediate, i.e. $\Delta P(x, 0)$, and long-term, resulting in final total losses $\Delta P(x, t_R)$ at the end of a reference time t_R. A strain $\varepsilon_p(x, t)$ can be associated with $P(x, t)$.

Even where prestress has to be considered as an action, a prestrain $\varepsilon_p(x, t)$ has commonly also to be considered in some parts of the calculations, especially in verifications with regard to ULS. Where only immediate losses are considered $\varepsilon_p(x, t)$ is deduced from $P(x, t)$ by dividing it by the product $E_p A_p$. Where also long-term losses are considered, this simple division may have to be supplemented by a correction transforming the relaxation of the tendon into a variation of strain.

Length and angular deviation may be considered small if the ratio $\Delta P_m(x, t)/P(0, 0)$ is not, at any time t, greater than 0.30. In other cases see subsection 4.6.2.

They may also depend on whether possible corrective measures are specified in a Quality Assurance Plan.

This general rule, necessary for practical reasons, may result in non-negligible uncertainties on the action effects if several, not correlated, prestresses intervene in the same verification. Besides, attention is drawn, especially for serviceability verifications, to the fact that the variability of the hyperstatic effects is not fully represented by P_k and special care would be necessary in cases where such effects are greater than usual.

In the vast majority of cases, when characteristic values shall be considered, it is sufficient to consider $P_{k\,\sup}$ for the initial persistent situation and $P_{k\,\inf}$ for the final persistent situation.

Losses are numerically defined, as mean values $\Delta P_m(x, t)$, in chapter 4 assuming that the structure is submitted to the quasi-permanent combination of actions defined in clause 1.6.6.5a.

For a given set of tendons, considered in the same calculation of losses, the mean value of the prestressing force is defined as $P_m(x, t) = P(0, 0) - \Delta P_m(x, t)$ (ΔP in absolute value).

Two characteristic values of the prestressing force are also defined.

In the cases where the length and angular deviation of the tendons are not exceptionally large, the following formulae, although conservative if the angular deviation is small, may be used as acceptable approximations.

(a) Bonded tendons

$$P_{k\,\sup}(x, t) = 1.1\, P_m(x, t)$$

$$P_{k\,\inf}(x, t) = 0.9\, P_m(x, t)$$

(b) Unbonded tendons

$$P_{k\,\sup}(x, t) = 1.05\, P_m(x, t)$$

$$P_{k\,\inf}(x, t) = 0.95\, P_m(x, t)$$

The representative values of prestressing forces to be used in various verifications are defined in the relevant chapters (4, 6 and 7).

As a general rule, the same representative value P(P_m or $P_{k\,\sup}$ or $P_{k\,\inf}$) is used for all sets of tendons involved in one verification (e.g. a verification with respect to maximum shear at a given abscissa).

In many cases the following design situations shall be distinguished for the structure as a whole

- transient situations during erection, during which losses are calculated for the relevant values of t
- persistent situation (initial situation) defined for prestress by the losses at the beginning of the lifetime of the completed structure (time t_0)
- persistent situation (final situation) defined for prestress by the final total losses.

The values of γ_P are specified in the relevant clauses of this Model Code (especially in 1.6.2.4(a). In many cases γ_P is equal to 1.0. In some cases, instead of introducing a γ_P factor different from 1.0, it may be suitable to use an upper limit of the bearing capacity of the tendon.

γ_P mainly covers the variability of the effects of prestress (as the other γ_F do) and the variability of prestress in extreme conditions (i.e. beyond the characteristic values). It does not cover the stress variations due to actions not taken into account in ΔP_m.

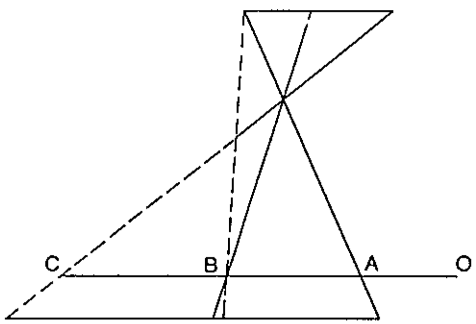
For SLS verification γ_P, systematically equal to 1, is not considered.

For calculations with regard to ultimate limit states, the design value of the prestress is deduced from the relevant representative values of $P(x, t)$ by multiplying by a partial factor γ_F, denoted $\gamma_P(\gamma_{P\,sup}$ or $\gamma_{P\,inf}$ if relevant).

When a part of a calculation refers to an indicator other than the value of P defined in clause 1.4.3.2, e.g. to the difference of strain between the tendon or the adjacent concrete, the steel stress variations are calculated in two steps: a first part $\Delta\varepsilon_1$ corresponding to the difference between $\varepsilon_p(x, t)$ and the indicator, and the second part $\Delta\varepsilon_2$ corresponding to the rest of the design values of these actions. As an example see Fig. 1.4.1.

1.4.3.3. Stress variations of prestressing steels

Steel stress variations due to actions, the effect of which on $P(x,t)$ is not included in P_m are calculated on the basis of the design values of these actions. No supplementary creep is taken into account in the assessment of these variations.



- B corresponds to the indicator defined as the difference of strain between a tendon and the adjacent concrete (see subsection 6.1.1)
- AO represents $\varepsilon_P(x, t)$ (this value is used in clause 4.6.2)
- AB represents $\Delta\varepsilon_1$ (before cracking at the level of the tendon)
- BC represents $\Delta\varepsilon_2$ (after cracking)
- CO represents the design strain used for the assessment of the resistance (see subsection 6.2.4)

Fig. 1.4.1.

The design value of the yield stress, to be considered in this case, would be $\gamma_s f_{ptk,\,sup}$, which in practice never can be exceeded.

If unbonded prestress is used with tendons set within the external outline of the structure, the stress variations $\Delta\sigma_p$ may be neglected. If tendons are partially set out of this external outline, a more refined analysis of deformations is necessary to calculate $\Delta\sigma_p$.

Practically, where the simplified stress–strain diagram defined in Fig. 2.3.2 (clause 2.3.4.3), but transformed to a design diagram by dividing $0.9 f_{ptk}$ by γ_s, is used, the method can be developed as follows.

The stress variations $\Delta\sigma_p$, due to design external actions, are first calculated for every critical cross-section.

A first verification then consists of checking whether the total stress of all tendons is less than their design yield stress $0.9 f_{ptk}/\gamma_s$. If this condition is satisfied, prestress and stress variations are considered, as in (a), for the rest of the verification for the same critical regions.

This condition should always be satisfied for SLS verifications (with in this case $\gamma_s = 1$). In practice it is covered by the verifications for ULS, or relating to reinforcing steels, considered in this Model Code.

If the condition is not satisfied, the isostatic effect of prestress of these tendons and their stress variations are globally substituted by $0.9 f_{ptk}/\gamma_s$ for the rest of the verification for the same critical region, as if the tendons were considered to be passive.

(a) Where $P_{k\,sup}$, (or $\gamma_{P\,sup} P_m$ with $\gamma_{P\,sup} > 1$) is considered, which implies that an increase of prestress would be considered to be unfavourable, no reference is made to any yield design strength limiting the stress, even if external actions result in stress increases in the tendons. The design prestressing force is then considered as acting, i.e. as an external action, to assess the total action effects applied to the structure, and the stress variations of the tendons are either neglected or taken into account as the stresses of reinforcing steels for the assessment of the structural response.

(b) Where $P_{k\,inf}$ (or $\gamma_{P\,inf} P_m$ with $\gamma_{P\,inf} \ngtr 1$) is considered, which generally implies that an increase of prestress would be favourable, reference is made to the design stress–strain diagram for the prestressing tendons (with γ_s for ULS verification). The prestressing force is in principle considered as acting. The stress variation $\Delta\sigma_p$ is calculated by shifting the origin of the design stress–strain diagram for the prestressing tendon by an amount corresponding to the pre-elongation ($\varepsilon_{Pk\,inf}$ or ε_{Pm}) (Fig. 1.4.2) associated with the relevant indicator—and is taken into account separately as is stress of reinforcing steels (generally for the assessment of the resistance of the critical region).

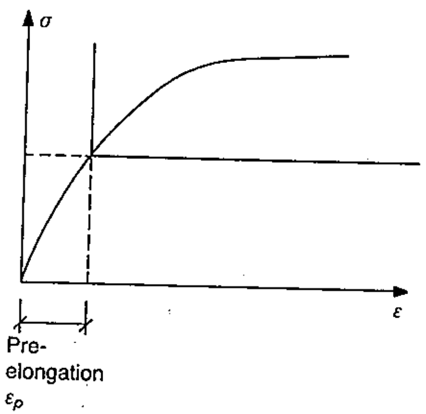


Fig. 1.4.2. Transformation of stress strain diagram

1.4.4. Representative values of material properties

Strengths and other material properties to be considered as fundamental basic variables are represented by their characteristic values f_k (or X_k) or by their mean values.

The significance of these values is shown in clause 6.3 of Bulletin 191. In exceptional cases, where an increase of the strength results in a decrease in reliability, upper characteristic values and specific γ_M values (smaller than 1) should be used.

Some formulae given in section 2.1 make it possible to assess the representative values of some material properties from strengths, with which they are correlated (e.g. f_{ct} and E_c from f_c). These formulae should be used for assessing the mean values and the characteristic values, but not the design values.

Where strengths and other material properties are not considered fundamental variables in limit state equations (see sections 1.3 and 1.6), they should be represented by mean values f_m (or X_m), which usually are the most likely values of f, and not other fractiles taken out of the same statistical populations as f_k values. However, these may generally be substituted by characteristic values f_k, as an approximation acceptable for such verifications.

1.4.5. Geometrical quantities

1.4.5.1. Representative values

Unintentional eccentricities, inclinations and parameters defining curvatures affecting columns and walls and the depth of reinforcement in members thinner than 100 mm, are the unique geometrical quantities defined in this Model Code to be taken into account as fundamental basic variables.

The other geometrical quantities are given the numerical values specified in the drawings of the design.

The fundamental geometrical variables are directly fixed as design values in the chapters where the relevant limit states are treated.

In this clause, only geometrical quantities representing the structure are considered (for geometrical quantities representing some actions, refer to subsections 1.4.2 and 1.4.3). For most of the quantities, their deviations within the specified tolerances should be considered as statistically covered by γ_{Sd} and γ_{Rd}, i.e. by γ_F and γ_M factors. Only those quantities, which might in some verifications be one of the main variables, should, in those verifications only, be taken as fundamental.

The depths of reinforcement in thin members are taken into account by modifying their nominal values by additive reliability margins.

1.4.5.2. Tolerances

The possible deviations in the geometry of the concrete elements, of the cover, or of the position of steel shall not alter significantly the SLS nor the ULS performance of the relevant elements.

As a general rule for these geometrical fundamental variables, the corresponding specified tolerances may be taken equal to their design values of the deviations divided by 1.2 and should be controlled accordingly.

Larger than intended dimensions in slabs may significantly increase the selfweight, whereas smaller dimensions and/or lever arms of steel bars may significantly reduce resistances. Similarly, smaller than nominal values of concrete cover may endanger the durability or the anchorage resistance of steel bars. An unintended inclination of columns may disproportionately increase their action effects.

For the other geometrical variables, the values of the materials partial safety factors included in this Code, are meant to cover small reductions of performance (resistances, mainly) which may result from their deviations.

Because of the complicated nature of the related phenomena, no explicit figure of general validity can be given on the amount of such performance reduction; however, it is considerably less than 4%.

In the absence of a more justified set of tolerances, the following limitations apply.

(a) *Table 1.4.1. Tolerances for concrete sectional dimensions*

Elements and dimension (mm)	Tolerances (mm) $\Delta a = (a_{nom} - a_{act})$
Beams; columns; walls	
$a \leqslant 200$	$\|\Delta a\| < 5$
$200 < a \leqslant 2000$	$\|\Delta a\| < 3.5 + 0.008\,a$
$2000 < a$	$\|\Delta a\| < 17.5 + 0.001\,a$
Slabs	
$a \leqslant 200$	$-10 < \Delta a < 6$
$200 < a \leqslant 2000$	$-20 < \Delta a < 4 + 0.010\,a$
$2000 < a$	$-30 < \Delta a < 20 + 0.002\,a$

(b) *Table 1.4.2. Tolerances for the position of passive or active reinforcement*

Structural depth d (mm)	Tolerances (mm) $\Delta d = (d_{nom} - d_{act})$
$d < 1000$	$\Delta d < 10$
$1000 < d < 2000$	$\Delta d < 0.01\,d$
$2000 < d$	$\Delta d < 20$

(c) Tolerance of cover: $c_{nom} - c_{act} < 10$ mm (see 8.4.3(c)).

(d) Tolerance of unintentional inclination of columns and walls: $\alpha < 1\%$.

The above tolerance values are valid for the final condition of the structure, after compaction and hardening of the concrete.

Depending on the quality assurance scheme applicable, relevant tolerance values should be respected for each category of possible deviations under well specified conditions of measurements and evaluations. Possible higher deviations foreseen, should lead to additional design steps taking into account all the consequences of deviations in exceedance of the specified tolerances.

1.5. PRINCIPLES OF DESIGN VS. DURABILITY

1.5.1. General

Concrete structures shall be designed, constructed and operated in such a way that, under the expected environmental influences, they maintain their safety, serviceability and acceptable appearance during an explicit or implicit period of time, without requiring unforeseen high costs for maintenance and repair.

If a structure is designed, executed and maintained according to the requirements of the Model Code, there is a high probability that it will withstand the expected conditions of use for a long period of time, say 50 years or more.

Designing for durability implies a slow-down of the process of deterioration of the parts of the structure that are critical in this respect. This normally implies a multistage strategy, which may often be based on successive protective barriers against corrosion.

The service life concept leads to an integral treatment of the

- planning and design phase
- execution phase
- period of use.

However, it is not the intention of this Code to impose any legal obligations to third parties, but only to make clear the whole working environment in which the designer is acting.

1.5.2. Exposure classes

Environmental conditions mean those chemical and physical actions to which the concrete is exposed and which result in effects that are not considered as loads or action effects in structural design. In the absence of a more specific study, these environmental conditions may be classified in the exposure classes given in Tables 1.5.1 and 1.5.2.

Table 1.5.1. Exposure classes related to environmental conditions

Exposure class	Environmental conditions
1. Dry environment	E.g. interior of buildings for normal habitation or offices*
2. Humid environment (a) Without frost	E.g. • interior of buildings where humidity is high† • exterior components • components in non-aggressive soil and/or water
(b) With frost	E.g. • exterior components exposed to frost • components in non-aggressive soil and/or water and exposed to frost • interior components when the humidity is high and exposed to frost
3. Humid environment with frost and de-icing agents	E.g. interior and exterior components exposed to frost and de-icing agents
4. Sea-water environment (a) Without frost	E.g. • components partially immersed in sea-water or in the splash zone • components in saturated salt air (coastal area)
(b) With frost	E.g. • components partially immersed in sea-water or in the splash zone and exposed to frost • components in saturated salt air and exposed to frost
5. Aggressive chemical environment‡ (a)	E.g. • slightly aggressive chemical environment (gas, liquid or solid) • aggressive industrial atmosphere
(b)	Moderately aggressive chemical environment (gas, liquid or solid)
(c)	Highly aggressive chemical environment (gas

* This exposure class is valid only as long as during construction the structure or some of its components is not exposed to more severe conditions over a period of several months.

† E.g. in commercial laundries.

‡ May occur alone or in combination with classes 1–4.

For more information, see d.6.3.

The treatment of corrosion of the reinforcement itself may require additional restrictions.

Table 1.5.2. Limiting values of deleterious substances in water of predominantly natural composition for the assessment of the severity of chemical attack

	Degree of severity		
	Slight	Moderate	High
pH-value	6.5–5.5	5.5–4.5	< 4.5
Carbonic acid dissolving lime $(CO)_2$ in mg/l determined by marble test according to Heyer	15–40	40–100	> 100
Ammonium (NH_4^+)	15–30	30–60	> 60
Magnesium (Mg^{2+}) in mg/l	300–1000	1000–3000	> 3000
Sulphate (SO_4^{2-}) in mg/l	200–600	600–3000	> 3000

For more information, see d.6.6.4.

1.5.3. Durability design criteria

In order to satisfy the requirement of subsection 1.5.1, a combination of criteria may be used as defined in this Model Code and developed in chapter 8 (including structural form, skin-concrete quality, detailing), as well as in chapter 7 for nominal crack width control under specified conditions. Additional protective coatings may be envisaged under very specific conditions.

1.6. BASIC DESIGN RULES

1.6.1. General

These basic design rules differ according to the limit state under consideration.

These numerical values are considered as appropriate for the socio-economic conditions in the Western European countries. In some countries (and possibly depending on the type of building or civil engineering works) where different conditions prevail, γ factors may be reduced.

The basic design rules applicable to the various types of limit states listed in section 1.2 are presented in subsections 1.6.2 to 1.6.6.

The numerical values of γ factors given in these clauses are applicable to buildings and civil engineering works not subject to variable actions having an exceptional variability.

See clause 1.1.3 concerning reliability degrees and reliability differentiation measures.

However, the $\gamma_{G\,sup}$ and γ_Q values given in subsection 1.6.2 may be reduced respectively to 1.2 and 1.35 for reliability differentiation in the following cases:

one-storey buildings (ground floor plus roof) with spans not exceeding 9 m, that are only occasionally occupied (storage buildings, sheds, green-houses, small silos and buildings for agricultural purposes), floors resting directly on the ground, light partition walls, lintels, sheeting and ordinary lighting masts, provided that these reductions are not associated with a reduced quality assurance level.

If the basic set of γ factors given in this section is adopted, any increase of the reliability degree is normally limited to the consideration of supplementary hazards or higher values of accidental actions, and more refined analyses.

Some γ_M factors may however have to be increased in cases where quality measures, considered normal in the actual case, would not be expected, but this is intended to maintain the reliability degree, not to modify it.

In principle all relevant limit states should be considered, as well as all relevant design situations, load arrangements and load cases and combinations of actions.

In some cases, defined in other chapters, some limit state calculations may be substituted by detailing rules or special provisions.

1.6.2. ULS of resistance of critical regions

1.6.2.1. Definition

These limit states have been defined in 1.2.2.

1.6.2.2. Design principle

It should be verified that either condition (a) is satisfied or conditions (b) and (c) are satisfied.

(a) In any cross-section, chord, strut or tie

$S_d < R_d$ if a one-component action-effect is to be considered,
$S_d \subset R_d^*$ if a multi-component action-effect is to be considered,

where

S_d denotes a design action-effect,
R_d denotes a design resistance (and R_d^* a design resistance domain).

(*a*): for example normal force and bending, or shear and torsion.

Models of resistance used in such verifications are denoted 'first yield' models in clause 5.4.1.2 and in chapter 6.

OR

(b) The number and extent of plastic hinges is such that no equilibrium would be possible in the model if the actions were increased.

Such a model can be physically envisaged only if elastic-plastic or rigid-plastic behaviour is assured for the reinforcement.

(*b*): only in some cases in which a statically indeterminate structure is considered.

This is not the only physical condition (see chapter 5). For example such a limit state cannot be reached at the fixed end of a cantilever slab eccentrically loaded. In any case this model is usable only for verifications with regard to bending; tangential limit states (shear, punching) are verified according to (*a*).

In this case all indirect actions are neglected. The location of the plastic hinges is identified. Then the design resistances of the plastic hinges are calculated and it shall be verified that a stable equilibrium is statically possible without exceeding these resistances if the loads take their design values

Unless the location of the plastic hinges be unconditionally codified, it should be verified that this location is the most unfavourable.
Hence the verification is made by comparing global design loads to maximum proportional loads compatible with design resisting moments.

AND

(c) Using the same models as in (b) (see clause 5.4.1.2), the limit plastic rotation θ_{pl} is not exceeded at any critical section before the mechanism of plastic hinges is fully developed.

The ULS may be reached before the formation of all plastic hinges.

1.6.2.3. Types and models for overall and local analysis

These are defined

- in chapter 5 for overall analysis (of the whole structure)
- in chapter 6 for local analysis (e.g. of cross-sections).

In case (b) of clause 1.6.2.2, only plastic analysis can be used; hyperstatic effects of prestress (see clause 1.4.3.1) are ignored.

These may be chosen separately if their compatibility is sufficiently ensured.
The various types and models may result in safety degrees. These differences should be limited either by corrective factors or (in most cases) by limiting the fields of application of some types and models.

1.6.2.4. Partial factors and ways to introduce them into the calculations

(a) γ_F factors

The general content of γ_F factors is defined in subsection 6.2.2 of Bulletin 191.

a1. Persistent and transient situations. The numerical values applicable to non-particular actions are given in the following table and clauses.

An example of particular actions is that of some hydraulic actions (see Bulletin 201).

Table 1.6.1. Partial γ_F factors: basic values

Actions, γ_F	Unfavourable effect (γ_{sup})	Favourable effect (γ_{inf})
Permanent, γ_G (P excluded)	1.35	1.0
Prestress, γ_P	1.1	1.0
Variable, γ_Q	1.5	Usually neglected

Basic values given in Table 1.6.1 are in some cases conservative. See subsections 1.1.3 and 1.6.1 (and Bulletin 202) for reliability differentiation and clause 1.6.2.5(c) for possible refinements (especially for $\gamma_{G\,sup}$).
If, as envisaged in subsection 1.4.1 and in (*c*) later, each of these factors is substituted by two partial factors γ_{Sd} and γ_f (respectively γ_g, γ_p and γ_q), the following approximations usually may be accepted.

- γ_{Sd} should generally be assumed to be equal to 1.15 for permanent action and 1.10 for variable actions; taking $\gamma_{Sd} = 1.125$ for both is generally acceptable.
- γ_f (i.e. γ_g, γ_p or γ_q) $= \dfrac{\gamma_F \text{ (i.e. } \gamma_G, \gamma_P \text{ or } \gamma_Q)}{\gamma_{Sd}}$

This rule is not applicable for the limit state of equilibrium.

A more refined method is defined in CEB Bulletin 128 par. 9.433.

The reduction of γ_F is for example applicable to a favourable normal force, independent of or little correlated with the unfavourable bending moment. The rule is equivalent to the application of a factor γ_{Sd} equal to 0.95 instead of 1.12 to this component. It is conservative in cases where the components are partially correlated and should ensure against the premature cut-off of bars in columns of a multistorey frame in which the assessment of the favourable normal forces cannot be precise.

In the most common cases one of $\gamma_G(\gamma_{G\,sup}$ or $\gamma_{G\,inf})$ may be applied globally to all permanent actions (unfavourable or not), prestress excepted. The other cases should be identified by judgement.

Action-effects sensitive to random spatial variation of actions (permanent or variable), usually classified as fixed (see clause 1.4.2.1), should be calculated with the assumption that a part ξ of each of these actions is considered a free action. ξ should be determined on the basis of a study of the spatial variability of the actions. If no such study is made, $\xi = 0.1$ may be used.

For bi-component action effects, if a component is favourable, the γ_F factors associated with this component should be divided by 1.2 if the two components are not correlated or little correlated. In other cases (e.g. for the normal force and the moment due to prestress) the same γ_F factors shall be applied to both components.

For imposed deformations (permanent or variable), γ_F values given in Table 1.6.1 are applicable in the case (a) of 1.6.2.2 if the deformations themselves, and not corresponding forces, are introduced in the structural analysis. Commonly, depending on their origin or effect, imposed deformations may not be taken into account for the ultimate limit state. If they are, in the case of linear analysis with or without redistribution, the partial factors γ_F applicable to them should be between 1 and 1.2.

Unless differently specified in particular clauses, the conditions of use of γ_P are the following.

- The upper value of γ_P (i.e. $\gamma_{P\,sup}$) given in Table 1.6.1 shall be associated with the characteristic value $P_{k\,sup}$ and is applicable for general effects (i.e. where the effects of other actions have the same order of magnitude as the effects of P). It shall be increased up to 1.3 for local effects. If for simplification $P_{k\,sup}$ is substituted by P_m, $\gamma_{P\,sup}$ shall be increased by 10% if prestress is bonded, by 5% if unbonded.
- The lower value of γ_P (i.e. $\gamma_{P\,inf}$) given in Table 1.6.1 is normally applicable to $P_{k\,inf}$. It may generally be kept if applied to P_m, especially for verifications with regard to bending; however, in some cases, especially for some verifications with regard to shear, the more safe-sided value 0.9 may be preferred if applied to P_m.

Generally the same γ_P should be applied to all prestress involved in the same verification, and it is sufficient to associate $\gamma_{P\,inf}$, applied to $P_{k\,inf}$ or P_m, with $\gamma_{G\,sup}$, and to associate $\gamma_{P\,sup}$, applied to $P_{k\,sup}$ or P_m, with $\gamma_{G\,inf}$.

For closely bounded variable actions or low variability actions identified in standards the value 1.5 of γ_Q (unfavourable action-effect) should be reduced to 1.35.

a2. Accidental situations. The values of γ_F applicable to all actions are equal to 1.

Safety is normally ensured by the design values of the accidental action or of the other parameters describing the accidental situation.

(b) γ_M factors
The numerical values of γ_M to be used for calculating R_d are given in Table 1.6.2.

The general content of γ_M factors is defined in subsection 6.3.2 of Bulletin 191.

As a simplification a conversion factor η is included in γ_c.

Table 1.6.2. Partial factors-γ_M

Fundamental basic variable	Design situation	
	Persistent/transient	Accidental
Concrete		
Compressive strength (f_{cck}), γ_c	1.5	1.2
Tensile strength (f_{ctk}), γ_{ct}	*	*
Reinforcing or prestressing steel		
Tensile strength (f_{stk}), γ_s	1.15	1.0
Compressive strength (f_{sck}), γ_{sc}	1.15	1.0

* See relevant clauses.

The values of γ_c and γ_s given in Table 1.6.2 should be increased if the geometrical tolerances given in clause 1.4.5.2 are not fulfilled. Conversely they might be reduced by 0.1 and 0.05 respectively, at the maximum, if these tolerances are reduced by 50% and are strictly controlled (see e.g. subsection 14.1.3).

A variation of γ_c or γ_s, according to the degree of control of f_{ck} (without making the specimen more severe), does not seem to be justified, because the variation of the control can more rationally be taken into account by the compliance criteria included in the control itself. In any case, it cannot be numerically fixed independently of the control criteria. Besides, even if a better quality, characterized by a lower coefficient of variation of the strength, is ensured for a given characteristic strength, this would not justify reducing the γ_M-values, because this would imply also a lower mean strength. However, see subsection 1.6.1 for abnormal cases. See also subsection 14.1.3 for the cases where the conversion factor η included in γ_c may be reduced.

The γ_M factors applicable to other fundamental variables are given in the relevant clauses.

Numerical values given in Table 1.6.2 include conversion factors η, which, for some applications (see clauses in which these factors should not be taken into account) may be assumed to be equal to 1.1 for γ_c and to 1.0 for γ_s.

If, as envisaged in subsection 1.4.1, γ_c factors are substituted by γ_{Rd} and γ_m factors, the following approximation may usually be accepted:

- γ_m should be taken equal to γ_c divided by a partial factor γ_{Ra} equal to 1.1 taking mainly into account the consequences of an imperfect position of the reinforcement;
- γ_{Rd} should include γ_{Ra}.

Strengths may intervene in S_d via stiffnesses and the spatial distribution throughout the structure. They may generally be favourable as well as unfavourable and are not to be considered as fundamental variables.

These rules shall be amended for accidental situations (see (*a*) of clause 1.6.2.5) and if possible simplifications or refinements defined in (*b*) and (*c*) of clause 1.6.2.5 are applied.

Formula (1.6–2) is the more general. Particular cases are mainly those where

- S_d is an under-proportional function of the actions (or the principal of them); in these cases eq. (1.6–1) may be unsafe; or
- the effects of some actions have a sense opposite to the effects of the other actions and are of the same order of magnitude; in these cases eq. (1.6–1) may be too conservative (this may be the case for the isostatic effects of prestress).

This rule (not splitting γ_M into γ_M and γ_{Rd}) is not applicable in design by testing.

For the definition of individual actions, refer to subsections 1.2.1 and 6.2.1 of Bulletin 191.

Prestress should be added in the symbolic combinations, if relevant.

For the application of the γ factors, see clause 1.6.2.4(c). Besides, for $\gamma_{G\,sup}$, $\gamma_{G\,inf}$, $\gamma_{P\,sup}$ and $\gamma_{P\,inf}$, refer to clause 1.6.2.4 a1.
For the Ψ factors, refer to clause 1.4.2.2(b).

Whenever strengths intervene in the value of the action-effect S_d the associated γ_M values should be taken equal to 1. This rule is not applicable to buckling verifications (see clause 1.6.3.4(c)), in which strengths are important favourable fundamental variables.

(c) Introduction of the partial coefficients into the calculations
In most cases γ_F factors should be applied globally as follows

$$S_d = S\left\{\gamma_G G + \gamma_P P + \gamma_Q\left(Q_{1k} + \sum_{i>1} \Psi_{oi} Q_{ik}\right)\right\} \quad (1.6\text{–}1)$$

In particular cases, defined in the relevant clauses of other chapters or to be identified by judgement, for persistent or transient situations, this formula may be substituted by

$$S_d = \gamma_{Sd} S\left\{\gamma_g G + \gamma_P P + \gamma_q\left(Q_{1k} + \sum_{i>1} \Psi_{oi} Q_{ik}\right)\right\} \quad (1.6\text{–}2)$$

where the partial factors should be taken by referring to a1 above.
These two formulae are partially symbolic and should be applied by following in detail the combination rules given in clause 1.6.2.5.
γ_M factors should generally be applied globally.

1.6.2.5. Combinations of actions

(a) General rules
The combinations of design values to be taken into account for applying the equations (1.6–1) and (1.6–2) above are as follows, in symbolic presentation

- fundamental combinations applicable for persistent and transient situations

$$\gamma_{G\,sup} G_{sup} + \gamma_{G\,inf} G_{inf} + \gamma_{Q1} Q_{1k} + \sum_{i>1} \gamma_{Qi} \Psi_{oi} Q_{ik} \quad (1.6\text{–}3)$$

- accidental combinations, applicable for accidental situations

$$G_{sup} + G_{inf} + (A_d \text{ or } 0) + \Psi_{11} Q_{1k} + \sum_{i>1} \Psi_{2i} Q_{ik} \quad (1.6\text{–}4)$$

In these combinations

- G_{sup} and G_{inf} refer to the unfavourable and favourable parts of the permanent actions, respectively

- Q_{ik} refers to any variable action, one after the other
- A_d denotes the unique accidental action associated with the accidental situation, if this situation is due to this action. If it is due to another event or to a past action, A_d is substituted by 0.

In most cases some variable actions, which obviously are not the leading ones for a given verification, need not be considered as Q_{1k}.

The actions to be included in any combination are only those that are mutually compatible or are considered as such, as an acceptable approximation. Non-simultaneous actions should be considered in the same combination if their effects are simultaneous.

The cases of incompatibility or negligible compatibility are very numerous. They are given in the codes or standards on actions or identified by judgement (e.g. snow and maximum climatic temperature).

(b) Possible simplifications

Other simplifications may be envisaged and discussed, for example by giving directly design combinations for a given set of common variable actions, such as some imposed loads, wind, snow and temperature.

As an approximation to be recognized by judgement, it is frequently sufficient to limit the total number of variable actions to a maximum of three in any fundamental combination and to two in any accidental combination.

Judgement is necessary because the concept of one action is very blurred. For example the actions of wind, snow, water and imposed loads should be considered as different actions, but the imposed loads on different floors should be considered as one action.

Fundamental combinations that are obviously identified as non-critical may be omitted in the calculations.

In many cases Ψ_{oi} factors may be merged with γ_Q, and S_d may then be calculated, for persistent and transient situations, by

This simplification is mainly intended for common buildings.

$$S_d = S\left(\gamma_G G + \gamma_Q \sum_1^n Q_{ik}\right)$$

where

$\gamma_G = 1$ or 1.35 (take the more unfavourable)
$\gamma_Q = 1.5$ for $n = 1$, or 1.35 for $n \geqslant 2$ (take the more unfavourable).

In accidental combinations Ψ_{11} may often be substituted by Ψ_{21} for most or all variable actions, as a judged approximation or because the occurrence of a greater value during the accidental situation is judged to be very unlikely.

Attention is drawn to the risk that an accident results in consequences on variable actions; for example many persons may gather in some places in order to escape during or immediately after an accident.

(c) Possible refinements

In cases where the most likely consequences of a failure do not seem to be exceptionally severe, the following reductions of γ_F factors in fundamental combinations are possible

This may be the case, for example, if a failure should be limited to a small part of the structure.

- reduce $\gamma_{G\,sup}$ to 1.2 or, alternatively, Q_{1k} to $\Psi_{01} Q_{1k}$ or

This introduces one more combination. Attention is drawn to the necessity, in this case, to verify more completely and carefully than usual the serviceability limit states, which may be less covered than usually by ultimate limit state verifications.

In many cases this does not result in important changes of design.

- reduce to 1.2 the γ_Q value applicable to $\Psi_{oi} Q_{ik} (i > 1)$.

1.6.3. ULS of buckling

1.6.3.1. Definition: field of validity

The verifications treated in this section are stability verifications of slender structures made of columns, walls and beams (or of their elements), in which second order effects are important.

The cases in which torsion effects in columns or in beams cannot be neglected, are not completely covered here.

Local buckling of deep beams and shells is not covered in this Model Code.

Clause 1.6.3.2 is more generally valid.

1.6.3.2. Design principle

Stability should be verified by demonstrating that

- under the most unfavourable load cases and combinations of actions
- giving the materials and joints their design strengths and the associated deformability
- taking into account geometrical imperfections,

a field of stresses exists throughout the structure, which equilibrates the design action effects, second order action effects included.

Whether the positions and directions of the actions are fixed or are modified by the displacements and deformations of the structure shall be recognized and taken into account in this verification.

According to the case, the verification will be concluded by one of the two following possibilities

In many cases the verifications may be limited to substructures or isolated elements.

In the most general case (before simplifications) the design principle cannot be expressed by an explicit equation, but only by a set of second order differential equations resulting from moment–curvature relationships throughout the structure with the boundary conditions of the structure.

- either the design actions are smaller than a proportional loading which would result in instability, before the ultimate limit state of resistance is reached in any cross-section (instability is reached when a small increase of the loading results in very large deformations)
- or the design action effects, second order action effects included, are smaller than the action effects due to a proportional loading which would result in exceeding the limit state of resistance in a cross-section before reaching 'instability'.

For reinforced and prestressed concrete structures the first possibility is the more frequent if the structure is rather slender and none of its cross-sections is relatively weak by comparison with most cross-sections. If the structure is not very slender, then second order action effects are relatively small and the second possibility may be postulated a priori.

Within some limits relating to slenderness this can be directly presumed and the second-order action effects can be directly assessed (see section 6.6).

1.6.3.3. Models for the analysis of the structure and its cross-sections

The general type of both analyses is the non-linear type of analysis defined in clause 5.3.2.1.

The model to be used for calculating the design resistances of cross-sections, is the general model for columns defined in clause 6.3.3.4.

For the structural analysis and hence for studying the possibility of 'instability', models to be used generally are simplified models.

Simplifications consist of approximating the basic formulation to a greater or lesser degree, thus

(*a*) where action effects are not coplanar with a main plane of the structure, or buckling may occur out of this plane, by verifying successively the stability within two perpendicular planes, and then combining the results according to a conventional rule;

(*b*) for some kinds of structures, by verifying separately some parts of them, having calculated the design action effects imposed at their ends (without considering at this stage the second order effects) and possibly after having simplified these action effects (e.g. by substituting different bending moments by equal moments applied at both ends);

(*c*) for some structural elements, by substituting the real final deformed shape by an a priori chosen shape, restricting thus the limit state equation to the calculation of the magnitude of the deformation at one point;

(*d*) by assessing in a rough way the consequences of the creep due to permanent and long-term actions;

(*e*) for slightly slender columns, by using pragmatically calibrated formulae for assessing directly second order effects (as mentioned in clause 1.6.3.2).

The numerical basis for some simplified models may be obtained by a more simplified analysis.

Such an analysis may for example be elastic-type and based on rough approximate assessments of rigidities resulting in an assessment of 'buckling lengths' (i.e. distances between zero-curvature cross-sections) and 'slenderness ratios'. Such lengths and ratios may also be calculated a priori from pragmatically calibrated rules applicable to simple structural forms. Attention is drawn, however, to the fact that the results of such preliminary analyses and direct assessments often depend on the load case and possibly on the combination of actions.

The reason for this is the generally overproportional character of the second order action effects.

The 'linearisation procedure' (eq. (1.6–2)) may, however, be used in exceptional cases, for example for highly redundant and slightly slender structures. In this case γ_f should be given a relatively large value.

Depending on the models used in chapter 6, a γ_E factor on E_c may have also to be considered (see e.g. clause 6.6.2.3).

This reduction may be justified, in spite of the unfavourable character of low strengths, for two reasons:

- the conversion factor of the strength from a standardized specimen to a structural element of any shape, included in γ_c, is not applicable to deformability,
- a low mean strength for a whole structure or element is less likely than for one cross-section.

It should be considered whether such values include some real deformations due for example to shrinkage or temperature differences, and whether they may be modified according to tolerances and degree of control.

Fatigue damage consists of gradual crack propagation in structural parts.

Low cycle (oligocyclic) fatigue, due to less than 10^4 repetitions of actions, is not covered by fatigue limit states defined in this clause and in section 6.7. Although related to service conditions, fatigue limit states are limit states in their own right.

In cases where highly alternated tension and relatively high compression occur (e.g. some towers, marine structures or crane girders) fatigue verifications of concrete are necessary. Bond fatigue is also physically possible in some cases, but it is normally covered by the other verifications if the bond properties of the steels are normal.

Maintenance and redesign calculations should take into account the past and expected repetitions of loads.

1.6.3.4. Values of partial factors

(a) γ_F factors are given the same values as in clause 1.6.2.4(a).

The first formula (eq. (1.6–1)) given in clause 1.6.2.4(c) should normally be used.

(b) γ_s factors are given in the same values as in clause 1.6.2.4(b).

(c) In the calculation of resistances of critical regions, γ_c factors applicable to the compressive strength of concrete are given the same values as in clause 1.6.2.4(c).

In calculation of deformations, for the whole structure and for any structural element, γ_c may be decreased to

(i) 1.2 for fundamental combinations
(ii) 1 for accidental combinations.

1.6.3.5. Geometrical imperfections

The corresponding models and magnitudes to be used in the most common cases are defined in section 6.6.

1.6.4. ULS of fatigue

1.6.4.1. Definition

Fatigue damage occurs through repeated applications or variations of actions (mainly of loads).

Ultimate limit states of fatigue may be associated with the failure of reinforcing steel, of prestressing steel or of concrete.

1.6.4.2. Design principle

Fatigue design shall ensure that in any fatigue endangered cross-section the expected damage D will not exceed a limiting damage D_{lim}.

The verifications of this requirement can be performed according to four methods with increasing refinement.

Static actions not repeated more than 10^4 times or for which $\Psi_1 = 0$ are considered unable to produce fatigue. Examples of actions able to cause fatigue are loads due to vehicles, cranes, moving machinery, wind (gusts, turbulence, vortices, etc.) and wave action.

This is an indirect verification that the loss of strength will not be significant. The representative value of P should be chosen such that the unfavourable situation is covered.

In assessing the stress range, stress variations in opposite senses (due for example to successive arrangements of a moveable load) shall be, if relevant, taken into account.

Other design properties associated with the tensile stress of concrete (e.g. a formal shear stress) may also have to be considered.

This single value of Q may either correspond to identical magnitudes in all applications, or have fatigue effects equivalent to the effects of the action with its actual magnitudes.

The single value of Q may result into stress ranges in various cases, e.g.:
— the action is fixed but intermittent
— the action is fixed and alternate
— the action is moveable.

This value—as a fatigue equivalent one—should be taken as far as possible from structural codes or codes on actions on structures. In many cases the frequent value $\Psi_1 Q_k$ may be used as approximately equivalent or safe-sided.

For the assessment of the stresses and stress ranges, see (b); the frequent value of another variable action, e.g. temperature, is taken into account if relevant (i.e. if it increases the stress range in the case of non-linear behaviour).

For steel, for example, the limit state equation may be written

$$\gamma_{Sd} \max \Delta\sigma_{Ss}(G, P, Q, \Psi_1 T_k) \leqslant \Delta\sigma_{Rsk}(n)/\gamma_{s,fat}$$

The condition may also be presented as $n < N$ when N is the resisting number of applications of a stress range equal to $\gamma_{s,fat}\gamma_{Sd}\Delta\sigma_s$.

(a) First method
This is a qualitative verification that no variable action is able to produce fatigue. If the conclusion of this verification is not positive, a verification according to one of the following methods shall be made.

(b) Second method
This is a verification that

- for steel the maximum design stress range $\gamma_{Sd}\Delta\sigma_s(G, P, \Psi_1 Q_k)$
- for concrete in compression the maximum compressive stress $\gamma_{Sd}\sigma_{c,max}(G, P, \Psi_1 Q_k)$
- for plain concrete in tension the maximum design tensile stress $\gamma_{Sd}\sigma_{ct,max}(G, P, \Psi_1 Q_k)$

do not exceed the values given in subsection 6.7.3.

(c) Third method
This verification refers to a representation of the variable load dominant for fatigue by a single magnitude Q associated with a number of repetitions n during the required lifetime.

The stresses (or stress range) due to the application of Q (possibly due to applications in two senses or due to successive load arrangements) are multiplied by a factor γ_{Sd} given in 1.6.4.4. These design values shall be smaller than the resistances to fatigue for n cycles, as defined in sub-section 6.7.4, divided by a specific γ_M-factor ($\gamma_{s,fat}$ or $\gamma_{c,fat}$ depending on the material) also given in 1.6.4.4.

The limit state equations, depending on the material, are defined in subsection 6.7.4.

(d) Fourth method

This is a verification based on an assessment of the fatigue damage resulting from various magnitudes of loads. The load history during the required life should usually be represented by a spectrum in a discretized form. The accumulation of fatigue damage is calculated on the basis of the Palmgren–Miner summation.

1.6.4.3. Models for the analysis of the structure and its cross-sections

Linear elastic models generally may be used, and reinforced concrete in tension is considered to be cracked.

1.6.4.4. Values of partial factors

The γ-factors have the following numerical values

$$\gamma_{Sd} = 1.1$$
$$\gamma_{c,fat} = \gamma_c = 1.5$$
$$\gamma_{s,fat} = \gamma_s = 1.15$$

If the stress analysis is sufficiently accurate or conservative, and this fact is verified by in-situ observations, it may be possible to take $\gamma_{Sd} = 1.0$.

This includes, e.g. for bent bars, some conversion factors not included in the γ-factors.

The application of the partial safety factors is shown in section 6.7.

1.6.4.5. Effects of combined actions

In cases with superimposed loads due to different actions, e.g. winds, waves, vehicles, etc., it is necessary, if the verification is performed according to 1.6.4.2(d), to treat them according to whether they are correlated in time or not. If they are correlated, the corresponding stresses should be added; if not, the damages can be added separately.

Permanent actions including prestress should also be taken into account using their representative values.

1.6.5. ULS of static equilibrium and analogous limit states

1.6.5.1. Definition

These limit states are all limit states beyond which the structure, or a part of it, is overturned, slides or is lifted from its support.

With regard to these limit states, the structure, or the part of it under consideration, is considered to behave isostatically.

1.6.5.2. Design principle

In the simplest cases the condition may be written

$$S_{1d} \leqslant S_{2d}$$

where S_1 and S_2 are effects of destabilizing and stabilizing actions, respectively.

For frictional limit states it may be written

$$S_{1d} \leqslant \mu_d S_{2d}$$

where

μ_d is the design value of the coefficient of friction, and
S_1 and S_2 represent tangential and normal forces, respectively.

In other cases other fundamental variables (e.g. some geometrical variables) should be introduced in the limit state equations.

In some cases the overall stability cannot be maintained without a particular element, such as an anchoring device, of the structure. Then the condition should be written

$$S_{1d} \leqslant S_{2d} + R_d$$

where R_d is relatively small in comparison with S_{1d} and S_{2d}.

In these equations the deformations due to actions and possibly to foundation settlements (second order effects) should be taken into account.

Permanent actions, selfweight of the structure included, should be split into two parts: those not favourable to the stability (destabilizing actions) and those favourable to the stability (stabilizing actions). These two parts are used for the calculation of S_1 and S_2, respectively.

Variable and accidental actions generally are considered only when they are destabilizing (i.e. in S_1).

For variable actions specific models should sometimes be used (e.g. free vertical component of a wind pressure).

1.6.5.3. Values of the partial factors and combinations of actions

Design values of variable and accidental actions and combinations of their values are the same as defined in clauses 1.6.2.4 and 1.6.2.5. γ_Q-factors are introduced globally into the calculations.

Reference is made, if relevant, to the second paragraph of clause 1.6.2.4 al following Table 1.6.1 (i.e. taking into account the ξ factor).

For permanent actions $\gamma_{G\,sup}$ and $\gamma_{G\,inf}$ are respectively applied to the permanent actions included in S_1 and S_2, and should depend on

- the variability of these actions
- the possible correlation between them.

The γ_G-values given in this clause are associated with the representative values of permanent actions defined in clause 1.4.2.2.

In most cases, for cast-in-situ structures $\gamma_{G\,\mathrm{inf}}$ may be taken equal to 0.95 and $\gamma_{G\,\mathrm{sup}}$ to 1.05 if the selfweight of the structure is the major part of the permanent actions.

For prefabricated elements, values closer to 1 may be accepted. The same happens for accidental situations.

γ_G-values may also be taken closer to 1 for verifications during transient situations under control (e.g. the lower support reaction is measured before application of the whole destabilizing load). Such controls are always recommended when values closer to 1 than values given in the text are adopted.

If, as envisaged in clause 1.6.5.2, the stability involves a resistance R_d, the values 0.9 and 1.1 of γ_G-factors are substituted respectively by 1.1 and 1.35, and the values 0.95 and 1.05 by 1.2 and 1.35.

If the correlation between stabilizing and destabilizing actions is not relatively high, $\gamma_{G\,\mathrm{inf}}$ should be taken equal to 0.9 and $\gamma_{G\,\mathrm{sup}}$ equal to 1.1 for persistent and transient situations. For accidental situations they should be taken equal to 1.

They often represent the main cause of variability in such limit states. This is especially the case for friction coefficients and geometrical variables.

Design resistances R_d, if they intervene, as envisaged in clause 1.6.5.2, are given the same design values as for ultimate limit states of resistance.

Design values of the other design variables should be chosen carefully where they are judged to be treated as fundamental variables.

1.6.6. Serviceability limit states

1.6.6.1. Definition and classification

These limit states have been defined and classified in subsection 1.2.3. They are treated in detail in chapter 7.

For concrete structures the limit states of cracking defined in section 7.4 may have a paramount importance. Deformation limit states mainly depend on the type of building or civil engineering works and on its equipment and use.

1.6.6.2. Design principle

(a) Limit state of cracking and excessive compression

It should be verified that in any cross-section

$\sigma(F_d) < \alpha f_d$ for crack formation and excessive creep effects

$w(F_d, f) < w_{\mathrm{lim}}$ for maximum crack width

$\sigma(F_d) \leqslant 0$ for crack re-opening

where

σ is a defined stress
f_d is a tensile, shear or compressive design strength
w is a defined crack width.

As mentioned in subsections 7.4.4, 7.4.5 and 7.3.4 some of these rules may in some cases be substituted by stress limitations, detailing rules or other indirect verifications.

The α-factor (e.g. 0.6 for excessive compression) describes the limit state and is not a reliability factor.

In such equations f generally is not to be considered as a fundamental variable.

(b) Limit state of deformations

It should be verified that

$$a(F_d, f_d) \leqslant C_d$$

where a is a defined deformation (generally a deflection).

This rule may in some cases be substituted by a maximum slenderness ratio.

If not fixed by the Code, C_d should be fixed by the contract or chosen by the designer, possibly depending on non-structural parts.

(c) Limitation of vibrations

In the most common cases the limitation is ensured by indirect measures, such as limiting the deformations or the periods of vibration of the structure in order to avoid the risk of resonance. In the other cases a dynamic analysis is necessary.

See section 7.6.

1.6.6.3. Models for the analysis of the structure and its cross-sections

Elastic analysis is normally used. Non-linear analysis may be used.
The possibility of cracks should be considered.
Models for the analysis are defined in chapters 5 and 7.

Second order action effectss should be considered in very particular cases.

1.6.6.4. Values of partial factors

(a) γ_F-factors are taken equal to 1.
(b) γ_M-factors are taken equal to 1.

Pragmatic values smaller than 1 may be envisaged for indirect actions.

1.6.6.5. Combination of actions

(a) General rules

The combinations which should be considered depend on the particular limit state under consideration and are identified in the corresponding chapters.

They are defined as follows, in a symbolic presentation

rare: $G + P + Q_{1k} + \sum_{i>1} (\Psi_{oi} Q_{ik})$

frequent: $G + P + \Psi_{11} Q_{1k} + \sum_{i>1} (\Psi_{2i} Q_{ik})$

quasi-permanent: $G + P + \sum_{i \geqslant 1} (\Psi_{2i} Q_{ik})$

where G is taken according to clause 1.4.2.2 and Q_1 refers to any variable action, successively.

The appropriate representative value of P (i.e. P_m), where P_k should not be used, is specified in chapter 7.

This simplification is analogous to the simplification of the fundamental combinations defined in 1.6.2.5(b). If the most unfavourable variable action is easily identified, the number of these combinations is reduced to 2 for any number of variable actions.

Substituting frequent combinations by these combinations is another possible simplification which however may be excessively conservative, and not useful in many cases where the dominating frequent combination, among the n possible combinations, can be easily identified.

(b) Possible simplification

The first two paragraphs of clause 1.6.2.5(b) may be applied to combinations for serviceability limit states.

In common cases for reinforced concrete structures, the rare combinations may be simplified by avoiding reference to various Ψ_{oi} factors. They are substituted, in a symbolic presentation, by

$$G + Q_{1k}$$

or

$$G + 0.9 \sum_{1}^{n} Q_{ik} \text{ (take the more unfavourable)}$$

in which Q_{1k} is the most unfavourable variable action.

2. MATERIAL PROPERTIES

2.1. CONCRETE CLASSIFICATION AND CONSTITUTIVE RELATIONS

2.1.1. Definitions and classification

2.1.1.1. Range of applicability

The subsequent clauses apply to concrete with normal weight aggregates so composed and compacted as to retain no appreciable amount of entrapped air other than intentionally entrained air.

Though the relations given in the subsequent sections in principle also apply for heavyweight concrete, special consideration may be necessary for such concretes.

For technological aspects including the production of lightweight aggregate concrete refer to Appendix d. The constitutive relations given in these sections are applicable for the entire range of concrete grades dealt with in this Model Code. However, since the available information on the behaviour of concrete with a characteristic strength higher than 50 MPa is rather limited, the constitutive relations should be used with caution for $f_{ck} > 50$ MPa.

Note that throughout this section the following sign conventions are maintained which may differ from those used in other parts of this Model Code.

- Material properties are positive or to be used in absolute terms, e.g. compressive strength, $f_{cm} = |f_{cm}|$.
- Tensile stresses and tensile strains (elongations) are positive.
- Compressive stresses and compressive strains (contractions) are negative.
- Where multiaxial stress states are considered, $\sigma_1 > \sigma_2 > \sigma_3$.

2.1.1.2. Classification by strength — concrete grades

In this Model Code concrete is classified on the basis of its compressive strength. Design is based on a grade of concrete which corresponds to a specific value of its characteristic compressive strength f_{ck} as defined in clause 2.1.3.2.

Concrete grades for normal weight concrete can be selected from the following series:

C12 C16 C20 C25 C30 C35 C40 C45 C50 C55 C60 C65 C70 C75 C80

where the numbers denote the specified characteristic compressive strength f_{ck} in MPa. For production and quality control reasons concrete should be specified in steps of 10 MPa, and the values underline are recommended.

With regard to classification on the basis of density or durability refer to d.5.3 in Appendix d.

For reinforced concrete only grades C16 and above should be used. For prestressed concrete only grades C25 and above should be used.

2.1.2. Density

For normal weight concrete the following values of density may be used in design calculations

Where a higher accuracy is required concrete density may be determined experimentally e.g. according to ISO 6275.

$\rho = 2400\,kg/m^3$ for plain concrete
$\rho = 2500\,kg/m^3$ for reinforced and prestressed concrete.

2.1.3. Strength

2.1.3.1. Range of applicability

The information given in this section is valid for monotonically increasing compressive stresses or strains at a rate of $|\dot{\sigma}_c| \sim 1.0\,MPa/s$ or $|\dot{\varepsilon}_c| \sim 30 \times 10^{-6}\,s^{-1}$, respectively. For tensile stresses or strains it is valid for $\dot{\sigma}_{ct} \sim 0.1\,MPa/s$ or $\dot{\varepsilon}_{ct} \sim 3.3 \times 10^{-6}\,s^{-1}$, respectively.

2.1.3.2. Compressive strength

This Code is based on the uniaxial compressive strength f_c of cylinders, 150 mm in diameter and 300 mm in height stored in water at 20 ± 2°C, and tested at the age of 28 days in accordance with ISO 1920, ISO 2736/2 and ISO 4012.

For special requirements or in national codes test specimens other than cylinders 150/300 mm and stored in other environments may be used to specify concrete compressive strength. In such cases conversion factors should be determined by direct tests or as given in national codes for a given category of testing equipment.

The characteristic compressive strength f_{ck} (MPa) is defined as that strength below which 5% of all possible strength measurements for the specified concrete may be expected to fall.

For some verifications in design or for an estimate of other concrete properties it is necessary to refer to a mean value of compressive strength f_{cm} associated with a specific characteristic compressive strength f_{ck}. In this case f_{cm} may be estimated from eq. (2.1–1):

$$f_{cm} = f_{ck} + \Delta f \tag{2.1–1}$$

where $\Delta f = 8\,MPa$.

In practice, the concrete is regarded to comply with the grade specified for the design if the test results comply with the acceptance criteria given in Chapter 12.

Where concrete cubes 150/150/150 mm are used, the characteristic strength values given in Table 2.1.1 shall be obtained for the various concrete grades.

Table 2.1.1. Characteristic strength values (MPa)

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80
f_{ck}-cylinder	12	20	30	40	50	60	70	80
f_{ck}-cube	15	25	37	50	60	70	80	90
fck-cyl./fck-cu.	0,80	0,80	0,81	0,80	0,83	0,86	0,88	0,89

2.1.3.3. Tensile strength and fracture properties

2.1.3.3.1. Tensile strength

In this Code, unless stated otherwise, the term 'tensile strength' refers to the axial tensile strength f_{ct} determined in accordance with RILEM CPC 7.

The tensile strength of concrete is more variable than its compressive strength. It is influenced by the shape and the surface texture of the aggregates more than the compressive strength and may be reduced substantially by environmental effects. Therefore, the tensile strength of concrete should be taken into account in design with caution.

In absence of more accurate data for a particular concrete the lower and upper bound values of the characteristic tensile strength $f_{ctk,max}$ and $f_{ctk,min}$ may be estimated from the characteristic compressive strength using eqs (2.1–2) and (2.1–3)

$$f_{ctk,min} = f_{ctko,min}\left(\frac{f_{ck}}{f_{cko}}\right)^{2/3} \qquad (2.1\text{–}2)$$

$$f_{ctk,max} = f_{ctko,max}\left(\frac{f_{ck}}{f_{cko}}\right)^{2/3} \qquad (2.1\text{–}3)$$

where

f_{cko} = 10 MPa

$f_{ctko,min}$ = 0.95 MPa

$f_{ctko,max}$ = 1.85 MPa.

For some verifications in design or for an estimate of other concrete properties it is necessary to refer to a mean value of tensile strength f_{ctm} associated with a specified characteristic compressive strength f_{ck}. In this case f_{ctm} may be estimated from eq. (2.1–4)

$$f_{ctm} = f_{ctko,m}\left(\frac{f_{ck}}{f_{cko}}\right)^{2/3} \qquad (2.1\text{–}4)$$

where $f_{ctko,m}$ = 1.40 MPa.

The corresponding values for the characteristic tensile strength of different concrete grades are given in Table 2.1.2.

Table 2.1.2. Tensile strength for various concrete grades (MPa)

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80
f_{ck}	12	20	30	40	50	60	70	80
f_{ctm}	1.6	2.2	2.9	3.5	4.1	4.6	5.1	5.6
$f_{ctk,min}$	1.1	1.5	2.0	2.4	2.8	3.1	3.5	3.8
$f_{ctk,max}$	2.1	2.9	3.8	4.7	5.4	6.1	6.8	7.4

If the tensile strength is measured as splitting tensile strength $f_{ct,sp}$ or as flexural tensile strength $f_{ct,fl}$ conversion factors should be determined by means of direct tests.

If such conversion factors are not available the mean axial tensile strength f_{ctm} may be estimated from the mean splitting tensile strength $f_{ct,sp}$ according to eq. (2.1–5)

Though both f_{ctm} and $f_{ct,sp}$ depend on the size of the specimen, this size effect is in the range of the effect of specimen size on compressive strength. The effect of the depth of a beam on flexural tensile strength is more pronounced. It may be calculated on the basis of fracture characteristics, given in clauses 2.1.3.3.2 and 2.1.4.4.2 or estimated from eq. (2.1–6).

$$f_{ctm} = 0.9 f_{ct,sp} \qquad (2.1\text{–}5)$$

where

$f_{ct,sp}$ is the mean value of splitting tensile strength determined according to ISO 4108,

f_{ctm} is the mean value of axial tensile strength; it may be estimated from the mean flexural tensile strength according to eq. (2.1–6)

$$f_{ctm} = f_{ct,fl} \frac{1.5\,(h_b/h_o)^{0.7}}{1 + 1.5\,(h_b/h_o)^{0.7}} \qquad (2.1\text{–}6)$$

Eq. (2.1–6) is an approximation neglecting the effect of maximum aggregate size. It is valid for $h_b > 50$ mm.

where

$f_{ct,fl}$ is the mean value of flexural tensile strength determined according to ISO 4013

h_b is the depth of beam (mm)

h_o = 100 mm.

2.1.3.3.2. Fracture energy

The fracture energy of concrete G_F is the energy required to propagate a tensile crack of unit area.

In the absence of experimental data G_F may be estimated from eq. (2.1–7):

According to the RILEM Draft Recommendation TC50-FMC the fracture energy G_F is determined on notched specimens loaded in flexure. G_F corresponds to the area under the load-deflection relationship divided by the net cross-section of the specimen above the notch (see also clause 2.1.4.4.2).

With regard to the formulation of fracture properties refer to: H.K. Hilsdorf, W. Brameshuber, 'Code-Type Formulation of Fracture Mechanics Concepts for Concrete', International Journal of Fracture, Vol. 51, pp. 61–72, 1991.

$$G_F = G_{Fo} (f_{cm}/f_{cmo})^{0.7} \qquad (2.1\text{–}7)$$

where ↑ mean of cylinder compressive strength

f_{cmo} = 10 MPa.

G_{Fo} is the base value of fracture energy. It depends on the maximum [illegible]

Fracture energy G_F does, to some extent, depend on the size of the structural member as well as on other concrete properties not taken into account in eq. (2.1–7), resulting in deviations of G_F from the values according to eq. (2.1–7) of up to $\pm 30\%$.

Table 2.1.3. Base values of fracture energy G_{Fo} (Nmm/mm²)

d_{max} (mm)	G_{Fo} (Nmm/mm²)
8	0.025
16	0.030
32	0.058

Formel (2.1-7) liefert ca. 10x zu hohe Werte für Anwendung für Verbund-problem

The corresponding values for G_F for different concrete grades may also be taken from Table 2.1.4.

Note that in Table 2.1.4 G_F is given in (Nm/m²) whereas from eq. (2.1–7) and Table 2.1.3 values of G_F in (Nmm/mm²) are obtained.

Table 2.1.4. Fracture energy G_F (Nm/m²)

60 : fck

Max. aggregate size d_{max} (mm)	G_F (Nm/m²)							
	C12	C20	C30	C40	C50	C60	C70	C80
8	40	50	65	70	85	95	105	115
16	50	60	75	90	105	115	125	135
32	60	80	95	115	130	145	160	175

2.1.3.4. Strength under multiaxial states of stress

The strength of concrete under multiaxial states of stress may be estimated from the failure criterion given by eq. (2.1–8):

$$\alpha \frac{J_2}{f_{cm}^2} + \lambda \frac{\sqrt{J_2}}{f_{cm}} + \beta \frac{I_1}{f_{cm}} - 1 = 0 \qquad (2.1\text{–}8)$$

where

$$\lambda = c_1 \cos[1/3 \arccos(c_2 \cos 3\theta)] \quad \text{for } \cos 3\theta \geqslant 0 \qquad (2.1\text{–}9a)$$

$$\lambda = c_1 \cos[\pi/3 - 1/3 \arccos(-c_2 \cos 3\theta)] \quad \text{for } \cos 3\theta < 0 \qquad (2.1\text{–}9b)$$

$$\cos 3\theta = \frac{3\sqrt{3}}{2} \frac{J_3}{J_2^{3/2}} \qquad (2.1\text{–}10)$$

This failure criterion is one among several acceptable formulations. It has been chosen since it is not too difficult to use and agrees well with test data. For further details and the range of applicability of eq. (2.1–8) refer to 'Concrete under multiaxial states of stress—constitutive equations for practical design', CEB Bulletin 156, Lausanne, 1983 and to Ottosen, N., 'A Failure Criterion for Concrete', Journal Engineering Mechanics Division, ASCE, Vol. 103, EM4, August 1977.

The stress tensor (I_1) and the stress deviators (J_2 and J_3) used in eqs (2.1–8) to (2.1–10) may be calculated as follows

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3$$

$$J_2 = \tfrac{1}{6}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]$$

$$J_3 = (\sigma_1 - \sigma_m)(\sigma_2 - \sigma_m)(\sigma_3 - \sigma_m)$$

$$\sigma_m = (\sigma_1 + \sigma_2 + \sigma_3)/3$$

where σ_1, σ_2 and σ_3 are the principal stresses.

The strength ratio k may be estimated using eqs (2.1–1) and (2.1–4).

According to this failure criterion a given state of stress does not lead to failure if the left side of eq. (2.1–8) is negative.

The failure criterion is derived on the assumption that the biaxial compressive strength $f_{2cm} = 1.2 f_{cm}$.

For further information refer to: Kupfer, H.B., Gerstle, K.H., 'Behaviour of Concrete under Biaxial Stresses', Journal Engineering Mechanics Division, ASCE, Vol. 99, EM4, August 1973.

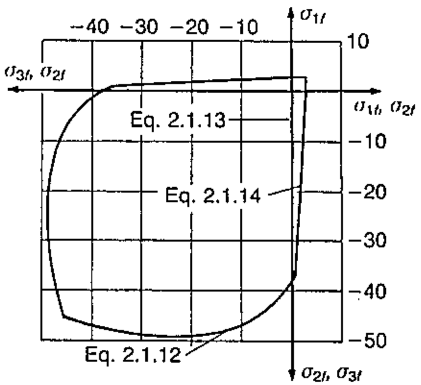


Fig. 2.1.1. Biaxial strength of concrete C30 according to eqs (2.1–12) to (2.1–14)

The parameters J_2, J_3 and I_1 in eqs (2.1–8) to (2.1–10) represent the invariants of the stress deviator and stress tensor, respectively, characterizing the state of stress considered.

The coefficients α, β, c_1 and c_2 are material parameters which depend on the strength ratio $k = f_{ctm}/f_{cm}$

$$\left.\begin{aligned} \alpha &= \frac{1}{9k^{1.4}} \qquad & \beta &= \frac{1}{3.7k^{1.1}} \\ c_1 &= \frac{1}{0.7k^{0.9}} \qquad & c_2 &= 1 - 6.8(k - 0.07)^2 \end{aligned}\right\} \qquad (2.1\text{–}11)$$

The strength of concrete under biaxial states of stress may be estimated from the simplified criteria, given in eqs (2.1–12) to (2.1–14). They are in acceptable agreement with the more realistic predictions from eqs (2.1–8) to (2.1–11).

Biaxial compression and tension-compression for $\sigma_{3f} < -0.96 f_{cm}$

$$\sigma_{3f} = -\frac{1 + 3.80\alpha}{(1 + \alpha)^2} f_{cm} \qquad (2.1\text{–}12)$$

where $\alpha = \sigma_{2f}/\sigma_{3f}$.

Biaxial tension

$$\sigma_{1f} = f_{ctm} = \text{const.} \qquad (2.1\text{–}13)$$

Biaxial tension-compression for $\sigma_{3f} > -0.96 f_{cm}$

$$\sigma_{1f} = \left(1 + 0.8\frac{\sigma_{3f}}{f_{cm}}\right) f_{ctm} \qquad (2.1\text{–}14)$$

where

σ_{1f} is the largest principal stress at failure
σ_{2f} is the intermediate principal stress at failure
σ_{3f} is the smallest principal stress at failure.

2.1.4. Stress and strain

2.1.4.1. Range of application

The information given in this section is valid for monotonically increasing compressive stresses or strains at a rate of $|\dot{\sigma}_c| \sim 30$ MPa/s or $|\dot{\varepsilon}_c| \sim 30 \times 10^{-6}\,s^{-1}$, respectively. For tensile stresses or strains it is valid for $\dot{\sigma}_{ct} \sim 0.03$ MPa/s or $\dot{\varepsilon}_{ct} \sim 3 \times 10^{-6}\,s^{-1}$, respectively.

2.1.4.2. Modulus of elasticity

Values of the modulus of elasticity for normal weight concrete can be estimated from the specified characteristic strength using eq. (2.1–15)

$$E_{ci} = E_{co}[(f_{ck} + \Delta f)/f_{cmo}]^{1/3} \qquad (2.1\text{–}15)$$

where

E_{ci} is the modulus of elasticity (MPa) at a concrete age of 28 days
f_{ck} is the characteristic strength (MPa) according to clause 2.1.3.2
$\Delta f = 8$ MPa
$f_{cmo} = 10$ MPa
$E_{co} = 2.15 \times 10^4$ MPa.

Where the actual compressive strength of concrete at an age of 28 days f_{cm} is known, E_{ci} may be estimated from eq. (2.1–16)

$$E_{ci} = E_{co}[f_{cm}/f_{cmo}]^{1/3} \qquad (2.1\text{–}16)$$

Where only an elastic analysis of a concrete structure is carried out, a reduced modulus of elasticity E_c according to eq. (2.1–17) should be used in order to account for the initial plastic strain

$$E_c = 0.85 E_{ci} \qquad (2.1\text{–}17)$$

Values of the tangent moduli E_{ci} and the reduced moduli E_c for different concrete grades are given in Table 2.1.6.

Table 2.1.6. Tangent moduli and reduced moduli of elasticity

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80
E_{ci} (10^3 MPa)	27	30	34	36	39	41	43	44
E_c (10^3 MPa)	23	26	29	31	33	35	36	38

The modulus of elasticity as obtained from eqs (2.1–15) and (2.1–16), respectively, is defined as the tangent modulus of elasticity at the origin of the stress–strain diagram. It is approximately equal to the slope of the secant of the unloading branch for rapid unloading and does not include initial plastic deformations. It has to be used for the description of stress–strain diagrams for uniaxial compression, uniaxial tension and multiaxial stress-states according to eqs (2.1–18) to (2.1–22), (2.1–23) and (2.1–29), respectively, as well as for an estimate of creep according to eqs (2.1–61) and (2.1–62). The reduced modulus of elasticity E_c according to eq. (2.1–17) includes some irreversible strains.

Even for a given strength the modulus of elasticity depends on the type of aggregates. Eq. (2.1–15) is valid for concretes made of quartzitic aggregates. For concrete made of basalt, dense limestone, limestone or sandstone the modulus of elasticity according to eq. (2.1–15) may be calculated by multiplying E_{ci} with the coefficients α_E from Table 2.1.5.

Table 2.1.5. Effect of type of aggregate on modulus of elasticity

Aggregate type	α_E
Basalt, dense limestone aggregates	1.2
Quartzitic aggregates	1.0
Limestone aggregates	0.9
Sandstone aggregates	0.7

To take full account of differences in aggregate stiffness or modulus, direct measurements of E_{ci} are necessary.

2.1.4.3. Poisson's ratio

For a range of stresses $-0.5 f_{ck} < \sigma_c < f_{ctk}$ Poisson's ratio of concrete, ν_c, is between 0.1 and 0.2.

2.1.4.4. Stress–strain relations for short-term loading

2.1.4.4.1. Compression

The stress–strain diagrams are generally of the form shown schematically in Fig. 2.1.2.

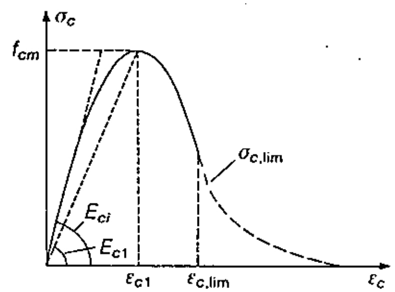


Fig. 2.1.2. Stress–strain diagram for uniaxial compression

The $\sigma_c - \varepsilon_c$-relationship may be approximated by the following function:

$$\sigma_c = -\frac{\dfrac{E_{ci}}{E_{c1}}\dfrac{\varepsilon_c}{\varepsilon_{c1}} - \left(\dfrac{\varepsilon_c}{\varepsilon_{c1}}\right)^2}{1 + \left(\dfrac{E_{ci}}{E_{c1}} - 2\right)\dfrac{\varepsilon_c}{\varepsilon_{c1}}} f_{cm} \quad \text{for } |\varepsilon_c| < |\varepsilon_{c,lim}| \qquad (2.1\text{–}18)$$

where

E_{ci} is the tangent modulus according to eq. (2.1–16)
σ_c is the compression stress (MPa)
ε_c is the compression strain
$\varepsilon_{c1} = -0.0022$
$E_{c1} = f_{cm}/0.0022$ = secant modulus from the origin to the peak compressive stress f_{cm}

The descending portion of the stress–strain relations should be considered as the envelope to all possible stress–strain relations of a concrete which tends to soften as a consequence of concrete micro-cracking.

Several relations exist to describe stress–strain relationships for concrete in compression. Among the suitable ones is the relation given by eq. (2.1–18). It can also be used as a basis to calculate stress–strain diagrams under multiaxial states of stress (clause 2.1.4.4.3).

Similar to tensile failure also compression failure of concrete is often a discrete phenomenon, i.e. there is a fracture region of limited width, in which compression strains are concentrated.

For practical reasons and due to lack of sufficient experimental data these strain concentrations generally are smeared as has been done in eqs (2.1–18) to (2.1–21). As a consequence, the descending branch of the stress–strain relation in compression is influenced by the length of the member subjected to compression, as can be seen from Fig. 2.1.3. Eqs (2.1–18) to (2.1–21) are reasonably accurate for a length of the member subjected to compression of approximately 200 mm.

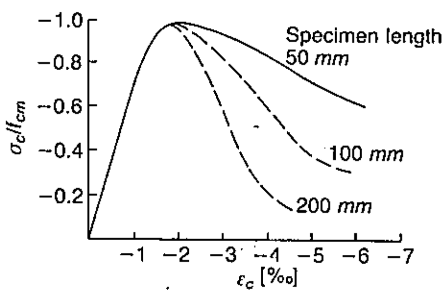


Fig. 2.1.3. Influence of specimen length on the uniaxial stress–strain curve for a constant cross-section 100 × 100 mm² (van Mier, J.G.M., Multiaxial strain-softening of concrete, Materials and Structures, No. 111, May/June 1986)

The size dependence of the stress–strain relation for concrete in compression is responsible for various size effects observed in the behaviour of reinforced concrete elements. Also refer to Hillerborg, A., 'The compression stress–strain curve for design of reinforced concrete beams', Fracture Mechanics: Application to Concrete, American Concrete Institute, ACI-SP-118, Detroit, Michigan, 1989.

For the descending part of the stress–strain diagram eq. (2.1–18) is valid only for values of $|\sigma_c|/f_{cm} \geqslant 0.5$.

The strain $\varepsilon_{c,\lim}$ at $\sigma_{c,\lim} = -0.5 f_{cm}$ may be calculated from eq. (2.1–19)

$$\frac{\varepsilon_{c,\lim}}{\varepsilon_{c1}} = \frac{1}{2}\left(\frac{1}{2}\frac{E_{ci}}{E_{c1}} + 1\right) + \left[\frac{1}{4}\left(\frac{1}{2}\frac{E_{ci}}{E_{c1}} + 1\right)^2 - \frac{1}{2}\right]^{1/2} \qquad (2.1\text{–}19)$$

Values for E_{ci}, E_{c1} and $\varepsilon_{c,\lim}$ for various concrete grades are given in Table 2.1.7.

Table 2.1.7. E_{ci}, E_{c1} and $\varepsilon_{c,\lim}$ for various concrete grades

Concrete grade	C12	C20	C30	C40	C50	C60	C70	C80
E_{ci} (10^3 MPa)	27	30.5	33.5	36.5	38.5	41	42.5	44.5
E_{c1} (10^3 MPa)	9	12.5	17.5	22	26.5	31	35.5	40
$\varepsilon_{c,\lim}$ (10^{-3})	−5.0	−4.2	−3.7	−3.3	−3.0	−2.8	−2.6	−2.4

The stress–strain relation according to eq. (2.1–18) is valid for monotonically increasing strains at a rate of approximately $-30 \times 10^{-6}\,\mathrm{s}^{-1}$. For substantially slower rates such as may occur during construction, stress–strain relations up to a stress $|\sigma_c| < 0.6 f_{ck}$ may be estimated from an incremental stress increases taking into account creep according to clause 2.1.6.4.3.

The strain $\varepsilon_{c,\lim}$ has no significance other than limiting the applicability of eq. (2.1–18).

For strains $|\varepsilon_c| > |\varepsilon_{c,\lim}|$ the descending branch of the $\sigma_c - \varepsilon_c$ diagram may be described using eqs (2.1–20) and (2.1–21):

$$\sigma_c = -\left[\left(\frac{1}{\varepsilon_{c,\lim}/\varepsilon_{c1}}\xi - \frac{2}{(\varepsilon_{c,\lim}/\varepsilon_{c1})^2}\right)\left(\frac{\varepsilon_c}{\varepsilon_{c1}}\right)^2 + \left(\frac{4}{\varepsilon_{c,\lim}/\varepsilon_{c1}} - \xi\right)\frac{\varepsilon_c}{\varepsilon_{c1}}\right]^{-1} f_{cm} \qquad (2.1\text{–}20)$$

with

$$\xi = \frac{4\left[\left(\frac{\varepsilon_{c,\lim}}{\varepsilon_{c1}}\right)^2\left(\frac{E_{ci}}{E_{c1}} - 2\right) + 2\frac{\varepsilon_{c,\lim}}{\varepsilon_{c1}} - \frac{E_{ci}}{E_{c1}}\right]}{\left[\frac{\varepsilon_{c,\lim}}{\varepsilon_{c1}}\left(\frac{E_{ci}}{E_{c1}} - 2\right) + 1\right]^2} \qquad (2.1\text{–}21)$$

The stress–strain relation for unloading of the uncracked concrete may be described by eq. (2.1–22)

$$\Delta\sigma_c = E_{ci}\Delta\varepsilon_c \qquad (2.1\text{–}22)$$

where

$\Delta\sigma_c$ is the stress reduction
$\Delta\varepsilon_c$ is the strain reduction.

As a simplifying alternative the descending branch of the $\sigma_c - \varepsilon_c$ diagram may also be approximated by a straight line according to eq. (2.1–38) and as shown in Fig. 2.1.8.

For cross-section design an idealized parabola-rectangle $\sigma_c - \varepsilon_c$-diagram may be used (see clause 6.2.2.2).

Figure 2.1.4 shows examples of stress–strain diagrams for concrete in uniaxial compression as derived from eqs (2.1–18) through (2.1–21).

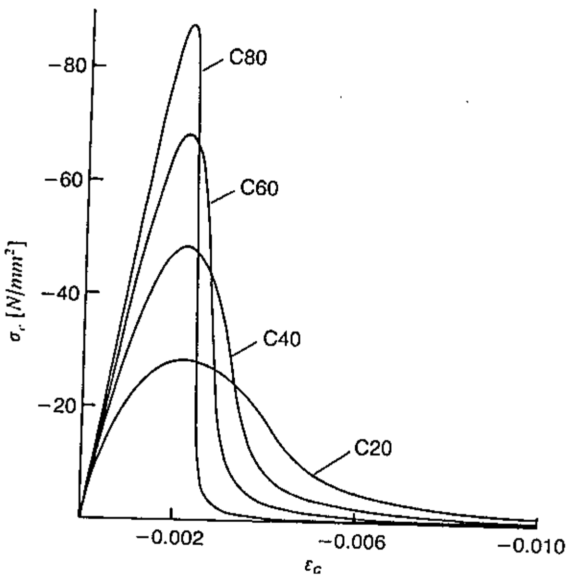


Fig. 2.1.4. Stress–strain diagrams for concrete in compression according to eqs 2.1–18 through 2.1–21

Tensile failure of concrete is always a discrete phenomenon. Therefore, to describe the tensile behaviour a stress–strain diagram should be used for the uncracked concrete, and a stress-crack opening diagram as shown in Fig. 2.1.5 should be used for the cracked section.

2.1.4.4.2. Tension

For uncracked concrete subjected to tension a bilinear stress–strain relation as given in eqs (2.1–23) and (2.1–24) may be used.

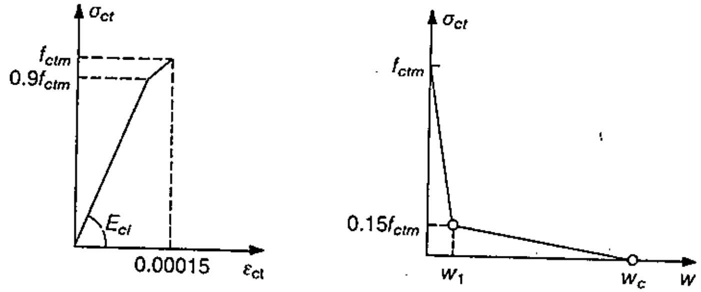


Fig. 2.1.5. Stress–strain and stress-crack opening diagram for uniaxial tension

For $\sigma_{ct} \leqslant 0.9 f_{ctm}$

$\sigma_{ct} = E_{ci}\varepsilon$ (2.

For $0.9 f_{ctm} < \sigma_{ct} \leqslant f_{ctm}$

$$\sigma_{ct} = f_{ctm} - \frac{0.1 f_{ctm}}{0.00015 - 0.9 f_{ctm}/E_{ci}} (0.00015 - \varepsilon_{ct}) \qquad (2.1\text{–}24)$$

where

E_{ci} is the tangent modulus of elasticity in (MPa) from eq. (2.1–15)
f_{ctm} is the tensile strength in (MPa) from eq. (2.1–4)
σ_{ct} is the tensile stress in (MPa)
ε_{ct} is the tensile strain.

For a cracked section a bilinear stress–crack opening relation as given in eqs (2.1–25) to (2.1–27) may be used (see Fig. 2.1.5):

$$\sigma_{ct} = f_{ctm}\left(1 - 0.85\frac{w}{w_1}\right) \quad \text{for } 0.15 f_{ctm} \leqslant \sigma_{ct} \leqslant f_{ctm} \qquad (2.1\text{–}25)$$

$$\sigma_{ct} = \frac{0.15 f_{ctm}}{w_c - w_1}(w_c - w) \quad \text{for } 0 \leqslant \sigma_{ct} < 0.15 f_{ctm} \qquad (2.1\text{–}26)$$

$$w_1 = 2\frac{G_F}{f_{ctm}} - 0.15 w_c \qquad (2.1\text{–}27a)$$

$$w_c = \alpha_F \frac{G_F}{f_{ctm}} \qquad (2.1\text{–}27b)$$

where

w is the crack opening (mm)
w_1 is the crack opening (mm) for $\sigma_{ck} = 0.15 f_{ctm}$
w_c is the crack opening (mm) for $\sigma_{ct} = 0$
G_F is the fracture energy (Nmm/mm^2) from eq. (2.1–7)
f_{ctm} is the tensile strength (MPa) from eq. (2.1–4)
α_F is the coefficient as given in Table 2.1.8.

The coefficient α_F depends on the maximum aggregate size d_{max} as given in Table 2.1.8.

Table 2.1.8. Coefficient α_F to estimate w_c

d_{max} (mm)	8	16	32
α_F[–]	8	7	5

This constitutive model is one among several acceptable formulations. It agrees well with test data.

The model is based on non-linear elasticity of the finite type, where the secant modulus of elasticity E_{csa} and Poisson's ratio ν_{csa} depend on the actual state and level of stress. The model describes path-independent, reversible, concrete behaviour. It is restricted to monotonically increasing proportional loading.

The post-failure stress–strain behaviour of concrete under multiaxial states of stress is not covered by the formulae presented here, because sufficient experimental data are not available. For further details refer to 'Concrete under multiaxial states of stress—constitutive equations for practical design', CEB Bulletin 156, Lausanne, 1983 and to Ottosen, N., 'A Failure-Criterion for Concrete', Journal Engineering Mechanics Division, ASCE, Vol. 103, EM4, August 1977.

The term β_{sa} is defined as the non-linearity index. It is a measure of the actual level of stress in relation to a failure state. At failure $\beta_{sa} = 1$. For definitions of β_{sa}, refer to Fig. 2.1.6.

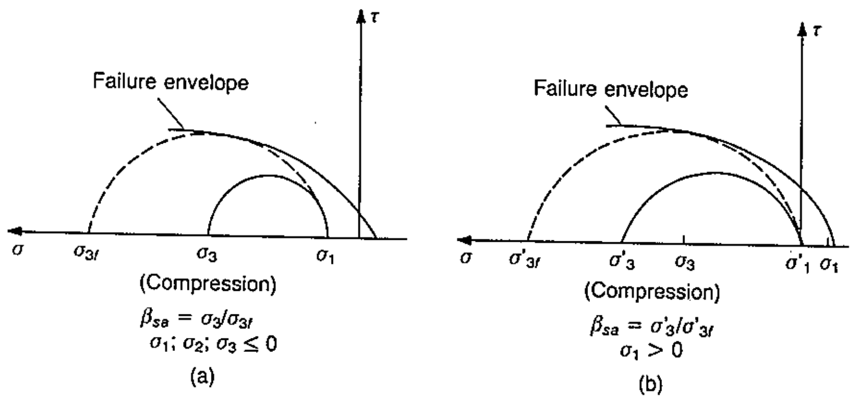


Fig. 2.1.6. Definition of β_{sa}: (a) triaxial compression; (b) one or more principal stresses ...

2.1.4.4.3. Multiaxial states of stress

The principal strains ε_1, ε_2 and ε_3 of concrete due to a multiaxial state of stress given by the principal stresses σ_1, σ_2 and σ_3 may be estimated from the following constitutive equations

$$\varepsilon_1 = \frac{1}{E_{csa}}[\sigma_1 - \nu_{csa}(\sigma_2 + \sigma_3)] \quad (2.1\text{–}28a)$$

$$\varepsilon_2 = \frac{1}{E_{csa}}[\sigma_2 - \nu_{csa}(\sigma_3 + \sigma_1)] \quad (2.1\text{–}28b)$$

$$\varepsilon_3 = \frac{1}{E_{csa}}[\sigma_3 - \nu_{csa}(\sigma_1 + \sigma_2)] \quad (2.1\text{–}28c)$$

where

E_{csa} is the actual secant modulus of elasticity for different stress levels due to the stress state according to eq. (2.1–29)

ν_{csa} is the actual Poisson's ratio for different stress levels according to eq. (2.1–33).

The actual secant modulus of elasticity may be estimated from eq. (2.1–29)

$$E_{csa} = \frac{E_{ci}}{2} - \beta_{sa}\left(\frac{E_{ci}}{2} - E_{cf}\right) + \left\{\left[\frac{E_{ci}}{2} - \beta_{sa}\left(\frac{E_{ci}}{2} - E_{cf}\right)\right]^2 - E_{cf}^2\beta_{sa}\right\}^{1/2} \quad (2.1\text{–}29)$$

where

$$\beta_{sa} = \frac{\sigma_3}{\sigma_{3f}} \quad \text{for } \sigma_1;\ \sigma_2;\ \sigma_3 \leqslant 0 \quad (2.1\text{–}30a)$$

$$\beta_{sa} = \frac{\sigma_3'}{\sigma_{3f}'} \quad \text{for } \sigma_1 > 0 \quad (2.1\text{–}30b)$$

$$E_{cf} = \frac{E_{c1}}{1 + 4[(E_{ci}/E_{c1}) - 1]\zeta} \quad \text{for } \zeta > 0 \quad (2.1\text{–}31a)$$

$$E_{cf} = E_{c1} \quad \text{for } \zeta \leqslant 0 \quad (2.1\text{–}31b)$$

$$\zeta = (\sqrt{J_{2f}}/f_{cm}) - (1/\sqrt{3}) \quad (2.1\text{–}32)$$

For triaxial compression σ_{3f} is obtained by increasing the actual stress σ_3 up to a failure state which may be determined from the failure criterion given in clause 2.1.3.4 (refer to Fig. 2.1.6(a)).

If at least one of the principal stresses is tension, i.e. $\sigma_1 > 0$, concrete behaviour becomes less non-linear. To account for this, the actual stress state is transformed by superimposing a state of hydrostatic compression $-\sigma_1$ resulting in a stress state σ_1'; σ_2'; σ_3' (refer to Fig. 2.1.6(b)).

The actual secant modulus of elasticity E_{csa} decreases as β_{sa} increases. At a failure state, i.e. $\beta_{sa} = 1$, $E_{csa} = E_{cf}$ according to eq. (2.1–31).

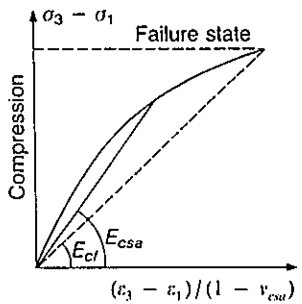


Fig. 2.1.7. Definition of E_{csa} and E_{cf}

where

E_c is the tangent modulus of elasticity for uniaxial compression according to eq. (2.1–15)

$E_{c1} = f_{cm}/|\varepsilon_{c1}|$ is the secant modulus of elasticity at failure for uniaxial compression (refer to Fig. 2.1.2)

E_{cf} is the secant modulus of elasticity at failure for a multiaxial state of stress according to Fig. 2.1.7

σ_3 is the largest principal compressive stress or smallest principal tensile stress

σ_{3f} is the principal stress σ_3 causing failure provided that σ_1 and σ_2 are unchanged (refer to Fig. 2.1.6(a))

$\sigma_3' = \sigma_3 - \sigma_1$

σ_{3f}' is the principal stress σ_3' causing failure provided that $\sigma_1' = 0$ and $\sigma_2' = \sigma_2 - \sigma_1$ are unchanged (refer to Fig. 2.1.6(b))

J_{2f} is the stress deviator J_2 for a failure stress state σ_1; σ_2 and σ_{3f} or $\sigma_1' = 0$; σ_2'; σ_{3f}' respectively.

For triaxial compression ζ in eq. (2.1–32) describes a decrease of E_{cf} with an increase of the deviatoric stress at failure. If at least one of the principal stresses is tension, ζ becomes negative and does not influence E_{cf} (refer to eq. (2.1–31b)).

According to eqs (2.1–33) the actual Poisson's ratio is constant up to $\beta_{sa} = 0.8$ and reaches a final value of 0.36 at failure.

The actual Poisson's ratio ν_{csa} may be estimated from eq. (2.1–33a) or (2.1–33b)

$$\nu_{csa} = \nu_c \text{ if } \beta_{sa} \leq 0.8 \qquad (2.1\text{–}33a)$$

$$\nu_{csa} = 0.36 - (0.36 - \nu_c)\sqrt{[1 - (5\beta_{sa} - 4)^2]} \text{ if } \beta_{sa} > 0.8 \qquad (2.1\text{–}33b)$$

where

ν_c is Poisson's ratio as given in clause 2.1.4.3

β_{sa} is the ratio to be taken from eq. (2.1–30).

This method is particularly suitable for numerical analysis. For further details refers to Darwin, D., Pecknold, D.A.W., 'Inelastic model for cyclic biaxial loading of reinforced concrete', Civil Engineering Studies, Structural Research Series, No. 409, University of Illinois, July 1974, and to Chen, W.F., Saleb, A.F., 'Constitutive equations for engineering materials', Vol. 1, p. 397 ff, John Wiley and Sons, 1982.

In FE analysis special attention has to be paid to the fact that element orientation and principal stress orientation do not necessarily coincide and may rotate with increasing stresses.

Stress–strain relations for biaxial states of stress may also be estimated from the following relations (2.1–34) to (2.1–42). They are based on the concept of equivalent uniaxial strain.

The incremental constitutive relations for a plane state of stress of an orthotropic material may be expressed by eq. (2.1–34):

$$\begin{Bmatrix} d\sigma_x \\ d\sigma_y \\ d\tau_{xy} \end{Bmatrix} = \frac{1}{1-\nu^2}\begin{bmatrix} E_x & \nu\sqrt{(E_x E_y)} & 0 \\ \nu\sqrt{(E_x E_y)} & E_y & 0 \\ 0 & 0 & (1-\nu^2)G \end{bmatrix}\begin{Bmatrix} d\varepsilon_x \\ d\varepsilon_y \\ d\gamma_{xy} \end{Bmatrix} \qquad (2.1\text{–}34)$$

where

$$\nu = \sqrt{(\nu_x \nu_y)} \qquad (2.1\text{–}35a)$$

and

$$(1-\nu^2)G = \tfrac{1}{4}[E_x + E_y - 2\nu\sqrt{(E_x E_y)}] \qquad (2.1\text{–}35b)$$

where

σ_x; σ_y: τ_{xy} are the normal stresses and shear stress, respectively, acting in the x-y plane
ε_x; ε_y; γ_{xy} are the normal strains and shear strain, respectively, acting in the x-y plane
E_x; E_y are the tangential moduli of elasticity
G is the shear modulus
ν_x; ν_y is Poisson's ratio for a stress increment $d\sigma_x$; $d\sigma_y$
ν is the equivalent Poisson's ratio.

Note that the shear modulus $G = E/2(1 + \nu)$ is assumed to be invariant with respect to the axis orientation.

For the direction of principal stresses 1; 2; eq. (2.1–34) simplifies to

$$\begin{Bmatrix} d\sigma_1 \\ d\sigma_2 \end{Bmatrix} = \frac{1}{1-\nu^2}\begin{bmatrix} E_1 & \nu\sqrt{(E_1 E_2)} \\ \nu\sqrt{(E_1 E_2)} & E_2 \end{bmatrix}\begin{Bmatrix} d\varepsilon_1 \\ d\varepsilon_2 \end{Bmatrix} \qquad (2.1\text{–}36a)$$

The variations of the tangent moduli of elasticity E_1; E_2 with stress are determined from the equivalent uniaxial strains ε_{iu}

$$d\varepsilon_{iu} = d\sigma_i/E_i \quad (i = 1; 2) \qquad (2.1\text{–}37a)$$

$$\varepsilon_{iu} = \sum_K d\varepsilon_{iu} \quad (K = 1 \ldots \text{stress increments}) \qquad (2.1\text{–}37b)$$

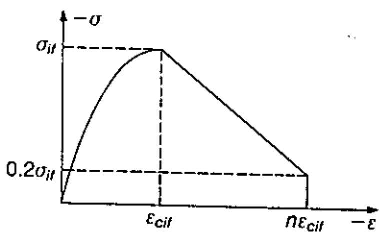


Fig. 2.1.8. Equivalent uniaxial stress–strain diagram

where

$d\varepsilon_{iu}$ is the equivalent uniaxial strain increment
E_i is the tangent modulus of elasticity at a stress σ_i.

If the strains ε_i in eq. (2.1–36a) are substituted by the equivalent uniaxial strains ε_{iu} the incremental constitutive relations may be expressed by eq. (2.1–36b)

$$\begin{Bmatrix} d\sigma_1 \\ d\sigma_2 \end{Bmatrix} = \begin{bmatrix} E_1 & 0 \\ 0 & E_2 \end{bmatrix} \begin{Bmatrix} d\varepsilon_{1u} \\ d\varepsilon_{2u} \end{Bmatrix} \qquad (2.1\text{–}36b)$$

The equivalent uniaxial strain for compression may be estimated from eq. (2.1–18) by replacing the peak stress $\sigma_c = -f_{cm}$ by the corresponding failure stress σ_{if}, the strain at peak stress ε_{c1} by an equivalent strain ε_{cif} and E_{c1} by $\sigma_{if}/\varepsilon_{cif}$. Relations for ε_{cif} are given in eqs (2.1–39) to (2.1–42).

Biaxial compression and biaxial tension-compression for $\sigma_{3u} < -0.96 f_{cm}$

$$\varepsilon_{c3f} = \varepsilon_{c1}\left(-3\frac{\sigma_{3f}}{f_{cm}} - 2\right) \qquad (2.1\text{–}39a)$$

$$\varepsilon_{c2f} = \varepsilon_{c1}\left(-3\frac{\sigma_{2f}}{f_{cm}} - 2\right) \qquad (2.1\text{–}39b)$$

$$\nu = \text{const.} = 0.2$$

where

ε_{c2f}; ε_{c3f} are the equivalent uniaxial strains at peak stress
$\varepsilon_{c1} = -0.0022$ which is the strain at peak stress $\sigma_c = -f_{cm}$ for uniaxial compression
σ_{3f}; σ_{2f} are the principal stresses at failure from eq. (2.1–12)
ν is the Poisson's ratio to be used in eq. (2.1–34).

Biaxial tension

$$\varepsilon_{cif} = \varepsilon_{ct1} = 0.00015 \qquad (2.1\text{–}40)$$

$$\nu = \text{const.} = 0.2$$

where

ε_{ct1} is the strain at peak stress f_{ctm} for uniaxial tension (Fig. 2.1.5).

Note that by introducing the equivalent uniaxial strains in eq. (2.1–36b) the effect of Poisson's ratio is eliminated and taken into account only in eq. (2.1–34).

For compression, the descending part of the stress-equivalent uniaxial strain relationship may be approximated by a straight line as expressed by eq. (2.1–38) and shown in Fig. 2.1.8.

$$\frac{\sigma_{ci}}{\sigma_{if}} = -\frac{\varepsilon_{ci}}{\varepsilon_{cif}}\frac{0.8}{n-1} + \frac{n-0.2}{n-1} \quad \text{for} \quad |\varepsilon_{cif}| < |\varepsilon_c| < |n\varepsilon_{cif}| \qquad (2.1\text{–}38)$$

The cut-off strain $n\varepsilon_{cif}$ of the descending part of the stress–strain relationship according to Fig. 2.1.8 depends on the strength grade as given in Table 2.1.9.

Table 2.1.9. Coefficient n to describe the cut-off strain

Concrete grade	C20	C40	C60	C80
n	3	2	1.5	1.2

Biaxial tension-compression for $\sigma_{3u} \geqslant -0.96 f_{cm}$

$$\varepsilon_{c3f} = \varepsilon_{c1} \left\{ -1.6 \left(\frac{-\sigma_{3f}}{f_{cm}} \right)^3 + 2.25 \left(\frac{-\sigma_{3f}}{f_{cm}} \right)^2 + 0.35 \left(\frac{-\sigma_{3f}}{f_{cm}} \right) \right\} \quad (2.1\text{–}41a)$$

$$\varepsilon_{cif} = \varepsilon_{ct1} \quad (2.1\text{–}41b)$$

$$\nu(\sigma) = 0.2 + 0.6 \left(\frac{-\sigma_3}{f_{cm}} \right)^4 + 0.4 \left(\frac{\sigma_1}{f_{ctm}} \right)^4 < 0.99 \quad (2.1\text{–}42)$$

Note that ν according to eq. (2.1–42) is valid for stress increments $d\sigma_1$; $d\sigma_3$. The 'secant' value of ν at maximum stress is approx. 0.35 to 0.40.

2.1.5. Stress and strain rate effects — impact

2.1.5.1. Range of applicability

The information given in subsection 2.1.5 is valid for monotonically increasing compressive stresses or strains at a constant range of approximately $1\,\text{MPa/s} < |\dot{\sigma}_c| < 10^7\,\text{MPa/s}$ and $30 \times 10^{-6}\,\text{s}^{-1} < |\dot{\varepsilon}_c| < 3 \times 10^2\,\text{s}^{-1}$, respectively.

For tensile stresses or strains it is valid for $0.1\,\text{MPa/s} < \dot{\sigma}_{ct} < 10^7\,\text{MPa/s}$ and $3 \times 10^{-6}\,\text{s}^{-1} < \dot{\varepsilon}_{ct} < 3 \times 10^2\,\text{s}^{-1}$, respectively.

For further details refer to 'Concrete Structures under Impact and Impulsive Loading' Synthesis Report, CEB Bulletin 187, Lausanne, 1988.

2.1.5.2. Compressive strength

For a given stress rate the compressive strength under high rates of loading may be estimated from eqs (2.1–43):

$$f_{c,imp}/f_{cm} = (\dot{\sigma}_c/\dot{\sigma}_{co})^{\alpha_s} \quad \text{for } |\dot{\sigma}_c| \leqslant 10^6\,\text{MPa/s} \quad (2.1\text{–}43a)$$

$$f_{c,imp}/f_{cm} = \beta_s(\dot{\sigma}_c/\dot{\sigma}_{co})^{1/3} \quad \text{for } |\dot{\sigma}_c| > 10^6\,\text{MPa/s} \quad (2.1\text{–}43b)$$

with

$$\alpha_s = \frac{1}{5 + 9 f_{cm}/f_{cmo}} \quad (2.1\text{–}44a)$$

and

$$\log \beta_s = 6\alpha_s - 2 \quad (2.1\text{–}44b)$$

where

$f_{c,imp}$ is the impact compressive strength,
$\dot{\sigma}_c$ is the stress rate (MPa/s)
f_{cm} is the concrete compressive strength from eq. (2.1–1)

$f_{cmo} = 10\,\text{MPa}$
$\dot{\sigma}_{co} = -1\,\text{MPa/s}$.

For a given strain rate the compressive strength may be estimated from eq. (2.1–45):

$$f_{c,imp}/f_{cm} = (\dot{\varepsilon}_c/\dot{\varepsilon}_{co})^{1.026\alpha_s} \quad \text{for } |\dot{\varepsilon}_c| \leqslant 30\,\text{s}^{-1} \qquad (2.1\text{–}45a)$$

$$f_{c,imp}/f_{cm} = \gamma_s(\dot{\varepsilon}_c/\dot{\varepsilon}_{co})^{1/3} \quad \text{for } |\dot{\varepsilon}_c| > 30\,\text{s}^{-1} \qquad (2.1\text{–}45b)$$

with

$$\log\gamma_s = 6.156\alpha_s - 2 \qquad (2.1\text{–}46)$$

where

$\dot{\varepsilon}_c$ is the strain rate (s^{-1})
$\dot{\varepsilon}_{co} = -30 \times 10^{-6}\,\text{s}^{-1}$
α_s is the coefficient from eq. (2.1–44a).

2.1.5.3. Tensile strength and fracture properties

2.1.5.3.1. Tensile strength

For a given stress rate the tensile strength under high rates of loading may be estimated from eqs (2.1–47):

$$f_{ct,imp}/f_{ctm} = (\dot{\sigma}_{ct}/\dot{\sigma}_{cto})^{\delta_s} \quad \text{for } \dot{\sigma}_{ct} \leqslant 10^6\,\text{MPa/s} \qquad (2.1\text{–}47a)$$

$$f_{ct,imp}/f_{ctm} = \lambda_s(\dot{\sigma}_{ct}/\dot{\sigma}_{cto})^{1/3} \quad \text{for } \dot{\sigma}_{ct} > 10^6\,\text{MPa/s} \qquad (2.1\text{–}47b)$$

with

$$\delta_s = \frac{1}{10 + 6f_{cm}/f_{cmo}} \qquad (2.1\text{–}48a)$$

$$\log\lambda_s = 7\delta_s - 7/3 \qquad (2.1\text{–}48b)$$

where

$f_{ct,imp}$ is the impact tensile strength
$\dot{\sigma}_{ct}$ is the stress rate (MPa/s)
f_{ctm} is the tensile strength from eq. (2.1–4)
$\dot{\sigma}_{cto} = 0.1\,\text{MPa/s}$
$f_{cmo} = 10\,\text{MPa}$.

For a given strain rate the tensile strength under high rates of loading may be estimated from eqs (2.1–49):

$$f_{ct,imp}/f_{ctm} = (\dot{\varepsilon}_{ct}/\dot{\varepsilon}_{cto})^{1.016\delta_s} \quad \text{for } \dot{\varepsilon}_{ct} \leqslant 30\,\text{s}^{-1} \qquad (2.1\text{–}49a)$$

$$f_{ct,imp}/f_{ctm} = \beta_s(\dot{\varepsilon}_{ct}/\dot{\varepsilon}_{cto})^{1/3} \quad \text{for } \dot{\varepsilon}_{ct} > 30\,\text{s}^{-1} \qquad (2.1\text{–}49b)$$

with

$$\log \beta_s = 7.112\delta_s - 2.33 \qquad (2.1\text{–}50)$$

where

$\dot{\varepsilon}_{ct}$ is the strain rate (s^{-1})
$\dot{\varepsilon}_{cto} = 3 \times 10^{-6}\,s^{-1}$.

2.1.5.3.2. Fracture energy
The information available regarding the effect of stress or strain rate on fracture energy is too incomplete to be included in this Model Code.

2.1.5.4. Modulus of elasticity

The effect of stress and strain rate on modulus of elasticity may be estimated from eqs (2.1–51)

$$E_{c,imp}/E_{ci} = (\dot{\sigma}_c/\dot{\sigma}_{co})^{0.025} \qquad (2.1\text{–}51a)$$

$$E_{c,imp}/E_{ci} = (\dot{\varepsilon}_c/\dot{\varepsilon}_{co})^{0.026} \qquad (2.1\text{–}51b)$$

where

$E_{c,imp}$ is the impact modulus of elasticity
E_{ci} is the modulus of elasticity of concrete from eqs (2.1–15) and (2.1–16)
$\dot{\sigma}_c$ is the stress rate (MPa/s)
$\dot{\varepsilon}_c$ is the strain rate (s^{-1})
$\dot{\sigma}_{co} = -1.0$ MPa/s and $\dot{\varepsilon}_{co} = -30 \times 10^{-6}\,s^{-1}$ for compression
$\dot{\sigma}_{cto} = 0.1$ MPa/s and $\dot{\varepsilon}_{cto} = 3 \times 10^{-6}\,s^{-1}$ for tension.

Eqs (2.1–51) are valid for all grades of concrete.

The effects of high stress and strain rates on the strains at maximum stress in tension and compression, respectively, may be estimated from eq. (2.1–52):

$$\varepsilon_{c1,imp}/\varepsilon_{c1} = (\dot{\sigma}_c/\dot{\sigma}_{co})^{0.02} = (\dot{\varepsilon}_c/\dot{\varepsilon}_{co})^{0.02} \qquad (2.1\text{–}52)$$

with

$\dot{\sigma}_{co} = -1$ MPa/s and $\dot{\varepsilon}_{co} = -30 \times 10^{-6}\,s^{-1}$ for compression
$\dot{\sigma}_{cto} = 0.1$ MPa/s and $\dot{\varepsilon}_{cto} = 3 \times 10^{-6}\,s^{-1}$ for tension

where

$\varepsilon_{c1,imp}$ is the impact strain at maximum load
ε_{c1} is the strain at maximum load for static loading from clauses 2.1.4.4.1 and 2.1.4.4.2 for compression and tension, respectively.

The development of tensile strength with time is strongly influenced by curing and drying conditions as well as by the dimensions of the structural members. As a first approximation it may be assumed that for a duration of moist curing $t_s \leqslant 7$ days and a concrete age $t > 28$ days the development of tensile strength is similar to that of compressive strength, i.e. eq. (2.1–4) is independent of concrete age for $t \geqslant 28$ days. For a concrete age $t < 28$ days residual stresses may cause a temporary decrease of the tensile strength.

In cases where the development of tensile strength with time is important it is recommended to carry out experiments taking into account exposure conditions and dimensions of the structural member.

2.1.5.5. Stress–strain diagrams

There is little information regarding the effect of high stress or strain rates on the shape of the stress–strain diagrams.

As an approximation, for monotonically increasing compressive stresses or strains up to the peak stress eq. (2.1–18) may be used together with eqs (2.1–43) and (2.1–45) for the peak stress $f_{c,imp}$, eq. (2.1–51) for the modulus of elasticity, $E_{c,imp}$, and eq. (2.1–52) for the strain at maximum stress, $\varepsilon_{c1,imp}$.

No information is available for the strain-softening region.

2.1.6. Time effects

2.1.6.1. Development of strength with time

The compressive strength of concrete at an age t depends on the type of cement, temperature and curing conditions. For a mean temperature of 20°C and curing in accordance with ISO 2736/2 the relative compressive strength of concrete at various ages $f_{cm}(t)$ may be estimated from eqs (2.1–53) and (2.1–54). To take into account the effect of temperature during curing the actual concrete age should be adjusted according to eq. (2.1–87).

$$f_{cm}(t) = \beta_{cc}(t) f_{cm} \qquad (2.1\text{–}53)$$

with

$$\beta_{cc}(t) = \exp\left\{ s\left[1 - \left(\frac{28}{t/t_1}\right)^{1/2}\right]\right\} \qquad (2.1\text{–}54)$$

where

$f_{cm}(t)$ is the mean concrete compressive strength at an age of t days
f_{cm} is the mean compressive strength after 28 days according to eq. (2.1–1)
$\beta_{cc}(t)$ is a coefficient which depends on the age of concrete t
t is the age of concrete (days) adjusted according to eq. (2.1–87)
$t_1 = 1$ day
s is a coefficient which depends on the type of cement (for cement classification, refer to Appendix d, clause d.4.2.1): $s = 0.20$ for rapid hardening high strength cements RS, 0.25 for normal and rapid hardening cements N and R, and 0.38 for slowly hardening cements SL.

Due to the counteracting effects of the parameters influencing the strength under sustained loads, $f_{cm,sus}(t, t_0)$ passes through a minimum. The duration of loading for which this minimum occurs depends on the age of loading and is referred to as the critical period $(t - t_0)_{crit}$. For an age at loading of 28 days, a concrete made of normal cement, type N, $(t - t_0)_{crit} = 2.8$ (days), $f_{c,sus\,min} = 0.78 f_{cm}$. It is generally referred to as sustained load strength of concrete.

There is insufficient experimental basis to give information on the tensile strength of concrete under high sustained tensile stresses.

2.1.6.2. Strength under sustained loads

When subjected to sustained high compressive stresses the compressive strength of concrete decreases with time under load. This strength reduction is counteracted by a strength increase due to continued hydration. The combined effect of sustained stresses and of continued hydration is given by eqs (2.1–55) and (2.1–56)

$$f_{cm,sus}(t, t_0) = f_{cm}\beta_{cc}(t)\beta_{c,sus}(t, t_0) \qquad (2.1\text{–}55)$$

with

$$\beta_{c,sus}(t, t_0) = 0.96 - 0.12\left\{\ln\left[72\left(\frac{t - t_0}{t_1}\right)\right]\right\}^{1/4} \qquad (2.1\text{–}56)$$

where

$f_{cm,sus}(t, t_0)$ is the mean compressive strength of concrete at time t when subjected to a high sustained compressive stress at an age at loading $t_0 < t$

$\beta_{cc}(t)$ is a coefficient according to eq. (2.1–54)

$\beta_{c,sus}(t, t_0)$ is a coefficient which depends on the time under high sustained loads $t - t_0$ (days). The coefficient describes the decrease of strength with time under load and is defined for $(t - t_0) > 0.015$ days ($=20$ min)

t_0 is the age of the concrete at loading

$t - t_0$ is the time under high sustained loads (days)

$t_1 = 1$ day.

2.1.6.3. Development of modulus of elasticity with time

The modulus of elasticity of concrete at an age $t \neq 28$ days may be estimated from eq. (2.1–57):

$$E_{ci}(t) = \beta_E(t)E_{ci} \qquad (2.1\text{–}57)$$

with

$$\beta_E(t) = [\beta_{cc}(t)]^{0.5} \qquad (2.1\text{–}58)$$

where

$E_{ci}(t)$ is the modulus of elasticity at an age of t days

E_{ci} is the modulus of elasticity at an age of 28 days, from eq. (2.1–16)

$\beta_E(t)$ is a coefficient which depends on the age of concrete, t (days)

2.1.6.4. Creep and shrinkage

2.1.6.4.1. Definitions

The total strain at time t, $\varepsilon_c(t)$, of a concrete member uniaxially loaded at time t_0 with a constant stress $\sigma_c(t_0)$ may be expressed as follows

$$\varepsilon_c(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t) + \varepsilon_{cs}(t) + \varepsilon_{cT}(t) \quad (2.1\text{–}59)$$

$$= \varepsilon_{c\sigma}(t) + \varepsilon_{cn}(t) \quad (2.1\text{–}60)$$

where

$\varepsilon_{ci}(t_0)$ is the initial strain at loading
$\varepsilon_{cc}(t)$ is the creep strain at time $t > t_0$
$\varepsilon_{cs}(t)$ is the shrinkage strain
$\varepsilon_{cT}(t)$ is the thermal strain
$\varepsilon_{c\sigma}(t)$ is the stress dependent strain: $\varepsilon_{c\sigma}(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t)$
$\varepsilon_{cn}(t)$ is the stress independent strain: $\varepsilon_{cn}(t) = \varepsilon_{cs}(t) + \varepsilon_{cT}(t)$.

The distinction between creep and shrinkage is conventional. Normally the delayed strains of loaded or unloaded concrete should be considered as two aspects of a single physical phenomenon.

Also, separation of initial strain and creep strain is a matter of convention. In structural analysis, the total load dependent strain as given by the creep function (refer to clause 2.1.6.4.3) is of importance. The initial and creep strain components are defined consistently, so that their sum results in the correct load dependent strain.

For the prediction of the creep function the initial strain $\varepsilon_{ci}(t)$ is based on the tangent modulus of elasticity as defined in eqs (2.1–15) and (2.1–57).

2.1.6.4.2. Range of applicability

The prediction model for creep and shrinkage given below predicts the mean behaviour of a concrete cross-section.

Unless special provisions are given the model is valid for ordinary structural concrete ($12\,\text{MPa} < f_{ck} \leqslant 80\,\text{MPa}$) subjected to a compressive stress $|\sigma_c| < 0.4 f_{cm}(t_0)$ at an age of loading t_0 and exposed to mean relative humidities in the range of 40 to 100% at mean temperatures from 5°C to 30°C.

It is accepted that the scope of the model also extends to concrete in tension, though the relations given in the following are directed towards the prediction of creep of concrete subjected to compressive stresses.

The model does not predict local rheological properties within the cross-section of a concrete member such as variations due to internal stresses, moisture states or the effects of local cracking.

The prediction model is not applicable to

- concrete subjected to extreme temperatures, high (e.g. nuclear reactors) or low (e.g. LNG-tanks)
- very dry climatic conditions (average relative humidity RH < 40%)
- structural lightweight aggregate concrete.

The effect of temperature variations during hardening can be taken into account in accordance with eq. (2.1–87). The effect of $0°\text{C} < T < 80°\text{C}$ is dealt with in subsection 2.1.8.

2.1.6.4.3. Creep

(a) Assumptions and related basic equations

Within the range of service stresses $|\sigma_c| < 0.4 f_{cm}(t_0)$, creep is assumed to be linearly related to stress.

For a constant stress applied at time t_0 this leads to

$$\varepsilon_{cc}(t, t_0) = \frac{\sigma_c(t_0)}{E_{ci}}\,\phi(t, t_0) \quad (2.1\text{–}61)$$

Here, concrete is considered as an ageing linear visco-elastic material. In reality, creep is a non-linear phenomenon. The non-linearity with respect to creep inducing stress may be observed in creep experiments at a constant stress, particularly if the stress exceeds $0.4 f_{cm}(t_o)$, as well as in experiments with a variable stress history even below stresses of $0.4 f_{cm}(t_o)$.

where

$\phi(t, t_0)$ is the creep coefficient
E_{ci} is the modulus of elasticity at the age of 28 days according to (eq. (2.1–15) or (2.1–16)).

The stress dependent strain $\varepsilon_{c\sigma}(t, t_0)$ may be expressed as

$$\varepsilon_{c\sigma}(t, t_0) = \sigma_c(t_0)\left[\frac{1}{E_c(t_0)} + \frac{\phi(t, t_0)}{E_{ci}}\right] = \sigma_c(t_0)J(t, t_0) \quad (2.1\text{–}62)$$

where

$J(t, t_0)$ is the creep function or creep compliance, representing the total stress dependent strain per unit stress
$E_c(t_0)$ is the modulus of elasticity at the time of loading t_0 according to eq. (2.1–57); hence $1/E_c(t_0)$ represents the initial strain per unit stress at loading.

In this section a so-called product formulation for the prediction of creep has been used, i.e. creep after a given duration of loading can be predicted from the product of a notional creep coefficient which depends on the age of concrete at loading and a function describing the development of creep with time. As an alternative, creep may also be described by a summation formulation as the sum of delayed elastic and of viscous strains. Advantages and disadvantages of both approaches as well as an alternative prediction model based on a summation formulation are given in: 'Evaluation of the time dependent behaviour of concrete', CEB Bulletin 199, Lausanne, 1990.

For variable stresses or strains, the principle of superposition is assumed to be valid.

On the basis of the assumptions and definitions given above, the constitutive equation for concrete may be written as

$$\varepsilon_c(t) = \sigma_c(t_0)J(t, t_0) + \int_{t_0}^{t} J(t, \tau)\frac{\partial\sigma_c(\tau)}{\partial\tau}\,d\tau + \varepsilon_{cn}(t) \quad (2.1\text{–}63)$$

The application of the principle of superposition is consistent with respect to the assumption of linearity. However, due to the actual non-linear behaviour of concrete some prediction errors are inevitable when linear superposition is applied to creep of concrete under variable stress, particularly for unloading or decreasing strains, respectively. For linear creep prediction models, the error depends on the type of model which is underlying the creep prediction (refer to CEB Bulletin 177).

The structural effects of time-dependent behaviour of concrete are dealt with in detail in section 5.8 of this Model Code and in CEB-Manual on 'Structural Effects of Time-dependent Behaviour of Concrete', CEB Bulletin 142/142 bis, Lausanne, 1984.

(b) Creep coefficient

The creep coefficient may be calculated from

$$\phi(t, t_0) = \phi_0\beta_c(t - t_0) \quad (2.1\text{–}64)$$

where

ϕ_0 is the notional creep coefficient eq. (2.1–65)
β_c is the coefficient to describe the development of creep with time after loading eq. (2.1–70)
t is the age of concrete (days) at the moment considered
t_0 is the age of concrete at loading (days), adjusted according to eqs (2.1–72) and (2.1–87).

The relations to calculate the creep coefficient are empirical. They were calibrated on the basis of laboratory tests (creep in compression) on structural concretes.

In this prediction model only those parameters are taken into account which normally are known to the designer, i.e. characteristic compressive strength, dimensions of the member, mean relative humidity to which the member is exposed, age at loading, duration of loading and type of cement. It should be pointed out, however, that creep of concrete does not depend on its compressive strength or age at loading per se, but rather on its composition and degree of hydration; creep of concrete decreases with decreasing water/cement ratio, decreasing cement content and increasing degree of hydration.

Due to the inherent scatter of creep and shrinkage deformations, the errors of the model and the general uncertainty caused by randomness of material properties and environment, a deformation prediction may result in a considerable prediction error. After short durations of loading or drying the prediction error is higher than after long durations of loading and drying. Based on a computerized data bank of laboratory test results a mean coefficient of variation for the predicted creep function $V_c = 20\%$ has been estimated. Assuming a normal distribution this corresponds to a 10 and 5 percent cut-off, respectively, on the lower and the upper side of the mean value of

$$\phi_{0.10} = 0.74\phi; \; \phi_{0.05} = 0.66\phi$$
$$\phi_{0.90} = 1.26\phi; \; \phi_{0.95} = 1.34\phi$$

The prediction error should be taken into account in a probabilistic approach where appropriate.

The notional creep coefficient may be estimated from

$$\phi_0 = \phi_{RH}\beta(f_{cm})\beta(t_0) \quad (2.1\text{–}65)$$

with

$$\phi_{RH} = 1 + \frac{1 - RH/RH_o}{0.46(h/h_o)^{1/3}} \quad (2.1\text{–}66)$$

$$\beta(f_{cm}) = \frac{5.3}{(f_{cm}/f_{cmo})^{0.5}} \quad (2.1\text{–}67)$$

$$\beta(t_0) = \frac{1}{0.1 + (t_0/t_1)^{0.2}} \quad (2.1\text{–}68)$$

where

$$h = 2A_c/u \quad (2.1\text{–}69)$$

f_{cm} is the mean compressive strength of concrete at the age of 28 days (MPa) according to eq. (2.1–1)
$f_{cmo} = 10$ MPa
RH is the relative humidity of the ambient environment (%)
$RH_o = 100\%$
h is the notational size of member (mm), where A_c is the cross-section and u is the perimeter of the member in contact with the atmosphere
$h_o = 100$ mm
$t_1 = 1$ day.

It is not known whether creep approaches a finite value. Nevertheless, the hyperbolic time function given in eq. (2.1–70) approaches an asymptotic value for $t \to \infty$. Evaluations on the basis of test results indicate that eq. (2.1–70) is a reasonably good approximation for the time development of creep up to 70 years of loading under the conditions indicated in Table 2.1.10. From experimental observations of creep up to 30 years one may conclude that the increase of creep from 70 years up to 150 years of duration of loading will not exceed 5% of the creep after 70 years.

The development of creep with time is given by

$$\beta_c(t - t_0) = \left[\frac{(t - t_0)/t_1}{\beta_H + (t - t_0)/t_1}\right]^{0.3} \quad (2.1\text{–}70)$$

with

$$\beta_H = 150\left\{1 + \left(1.2\frac{RH}{RH_o}\right)^{18}\right\}\frac{h}{h_o} + 250 \leqslant 1500 \quad (2.1\text{–}71)$$

where

$t_1 = 1$ day
$RH_o = 100\%$
$h_o = 100$ mm.

In cases where a lower level of accuracy is sufficient, the values given in Table 2.1.10 can be accepted as representative values for the creep coefficient after 70 years of loading of a normal weight ordinary structural concrete with a characteristic compressive strength between 20 and 50 MPa. These 70 year values may be taken as final creep coefficients.

Table 2.1.10. Creep coefficient ϕ (70y, t_0) of an ordinary structural concrete after 70 years of loading

Age at loading t_0 (days)	Dry atmospheric conditions (indoors) (RH = 50%)			Humid atmospheric conditions (out of doors) (RH = 80%)		
	Notional size $2A_c/u$ (mm)					
	50	150	600	50	150	600
1	5.8	4.8	3.9	3.8	3.4	3.0
7	4.1	3.3	2.7	2.7	2.4	2.1
28	3.1	2.6	2.1	2.0	1.8	1.6
90	2.5	2.1	1.7	1.6	1.5	1.3
365	1.9	1.6	1.3	1.2	1.1	1.0

The data given in Table 2.1.10 apply for a mean temperature of the concrete between 10°C and 20°C. Seasonal variations of temperature between −20°C and +40°C can be accepted. The same is true for variations in relative humidity around the mean values given in Table 2.1.10.

For classification of different types of cement refer to Appendix d, clause d.4.2.1.

Different types of cement result in different degrees of hydration. Creep of concrete depends on the degree of hydration reached at a given age rather than on the age of concrete. Therefore, the effect of type of cement is taken into account by modifying the age at loading such that for a given modified age the degree of hydration is approximately independent of the type of cement. The value for t_0 according to eq. (2.1–72) has to be used in eq. (2.1–68). The duration of loading $t - t_0$ used in eq. (2.1–70) is the actual time under load.

(c) Effect of type of cement and curing temperature

The effect of type of cement on the creep coefficient of concrete may be taken into account by modifying the age at loading t_0 according to eq. (2.1–72):

$$t_0 = t_{0,T}\left[\frac{9}{2 + (t_{0,T}/t_{1,T})^{1.2}} + 1\right]^{\alpha} \geqslant 0.5 \text{ days} \qquad (2.1\text{–}72)$$

where

$t_{0,T}$ is the age of concrete at loading (days) adjusted according to eq. (2.1–87)

$t_{1,T}$ = 1 day

α is the power which depends on the type of cement;

α = −1 for slowly hardening cements SL, 0 for normal or rapid hardening cements N and R, and 1 for rapid hardening high strength cements RS.

The creep behaviour of concrete with blended cements may as a first approximation be calculated with the formulae given here. However, larger prediction errors may be expected.

(d) Effect of high stresses

For stress levels in the range of $0.4 f_{cm}(t_0) < |\sigma_c| < 0.6 f_{cm}(t_0)$ the non-linearity of creep may be taken into account using eqs (2.1–73)

$$\phi_{0,k} = \phi_0 \exp[\alpha_\sigma(k_\sigma - 0.4)] \quad \text{for } 0.4 < k_\sigma \leqslant 0.6 \qquad (2.1\text{–}73a)$$

$$\phi_{0,k} = \phi_0 \quad \text{for } k_\sigma \leqslant 0.4 \qquad (2.1\text{–}73b)$$

where

$\phi_{0,k}$ is the non-linear notional creep coefficient, which replaces ϕ_0 in eq. (2.1–64)

$k_\sigma = |\sigma_c|/f_{cm}(t_0)$ which is the stress–strength ratio

α_σ = 1.5.

The main reasons for the non-linear behaviour are micro-cracking due to shrinkage or high loads and stress-induced ageing under load.

Eq. (2.1–73a) represents a simplification in so far as it does not take into account the observation that non-linearity decreases with increasing duration of loading and with decreasing change of moisture content during loading.

It should be noted that delayed elastic strains upon total unloading are linear functions of stress up to stress levels of $|\sigma_c| = 0.6 f_{cm}(t_0)$.

For mass concrete and at very high relative humidities, the coefficient α_σ may be as low as $\alpha_\sigma = 0.5$.

2.1.6.4.4. Shrinkage

The total shrinkage or swelling strains $\varepsilon_{cs}(t, t_s)$ may be calculated from

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cso}\beta_s(t - t_s) \qquad (2.1\text{–}74)$$

where

ε_{cso} is the notional shrinkage coefficient (eq. (2.1–75))

β_s is the coefficient to describe the development of shrinkage with time (eq. (2.1–79))

t is the age of concrete (days)

t_s is the age of concrete (days) at the beginning of shrinkage or swelling.

The notional shrinkage coefficient may be obtained from

$$\varepsilon_{cso} = \varepsilon_s(f_{cm})\beta_{RH} \qquad (2.1\text{–}75)$$

For curing periods of concrete members $t_s < 14$ days at normal ambient temperatures, the duration of moist curing does not significantly affect shrinkage. Hence, this parameter as well as the effect of curing temperature is not taken into account.

In eqs (2.1–74) and (2.1–79) the actual duration of drying $(t - t_s)$ has to be used. It is not affected by possible adjustments of t_0 or t_s according to eqs (2.1–72) and (2.1–87).

Similar to creep, shrinkage does not depend on concrete compressive strength per se. Shrinkage decreases with decreasing water/cement ratio and decreasing cement content.

A mean coefficient of variation of predicted shrinkage has been estimated on the basis of a computerized data bank, resulting in $V_s = 35\%$. The corresponding 10 and 5 percent cut-off values are

$$\varepsilon_{cs0.10} = 0.55\varepsilon_{cs};\ \varepsilon_{cs0.05} = 0.42\varepsilon_{cs}$$
$$\varepsilon_{cs0.90} = 1.45\varepsilon_{cs};\ \varepsilon_{cs0.95} = 1.58\varepsilon_{cs}$$

In cases where a lower level of accuracy is sufficient, the values given in Table 2.1.11 can be accepted as representative values for shrinkage of a normal weight ordinary structural concrete with a characteristic strength between 20 and 50 MPa after 70 years of drying. Usually these values may be taken as final shrinkage values.

Table 2.1.11. Shrinkage values $\varepsilon_{cs,70y} \times 10^3$ *for an ordinary structural concrete after a duration of drying of 70 years*

Dry atmospheric conditions (inside) ($RH = 50\%$)			Humid atmospheric conditions (outside) ($RH = 80\%$)		
Notional size $2A_c/u$ (mm)					
50	150	600	50	150	600
−0.57	−0.56	−0.47	−0.32	−0.31	−0.26

Though shrinkage reaches a final value, little information exists on the shrinkage strains of large members after long durations of drying. Therefore, the values calculated using eq. (2.1–79) for $2A_c/u = 500$ mm, and the values given in Table 2.1.11 for shrinkage of members with a notional size of $2A_c/u = 600$ mm, respectively, are uncertain and may overestimate the actual shrinkage strains after 70 years of drying.

with

$$\varepsilon_s(f_{cm}) = [160 + 10\beta_{sc}(9 - f_{cm}/f_{cmo})] \times 10^{-6} \qquad (2.1\text{–}76)$$

where

f_{cm} is the mean compressive strength of concrete at the age of 28 days (MPa)
$f_{cmo} = 10$ MPa
β_{sc} is a coefficient which depends on the type of cement: $\beta_{Sc} = 4$ for slowly hardening cements SL, $\beta_{Sc} = 5$ for normal or rapid hardening cements N and R, and $\beta_{Sc} = 8$ for rapid hardening high strength cements RS,

$$\beta_{RH} = -1.55\beta_{sRH} \text{ for } 40\% \leqslant RH < 99\%$$
$$\beta_{RH} = +0.25 \qquad \text{for } RH \geqslant 99\% \qquad (2.1\text{–}77)$$

where

$$\beta_{sRH} = 1 - \left(\frac{RH}{RH_o}\right)^3 \qquad (2.1\text{–}78)$$

with

RH is the relative humidity of the ambient atmosphere (%)
$RH_o = 100\%$.

The development of shrinkage with time is given by

$$\beta_s(t - t_s) = \left[\frac{(t - t_s)/t_1}{350(h/h_o)^2 + (t - t_s)/t_1}\right]^{0.5} \qquad (2.1\text{–}79)$$

where

h is defined in eq. (2.1–69)
$t_1 = 1$ day
$h_o = 100$ mm.

2.1.7. Fatigue

2.1.7.1. Fatigue strength

For a constant stress amplitude the number N of cycles causing fatigue failure of plain concrete may be estimated from eqs (2.1-80) to (2.1-85). They are valid for pure compression, compression-tension and pure tension, respectively.

Fatigue tests exhibit a large scatter in the number of cycles to failure. Therefore, often probabilistic procedures are applied in evaluating fatigue behaviour of concrete. For further details refer to 'Fatigue of concrete structures'. State-of-the-Art Report, CEB Bulletin 188, Lausanne, 1988.

Pure compression

For $S_{c,min} > 0.8$, the S-N relations for $S_{c,min} = 0.8$ are valid.

For $0 \leqslant S_{c,min} \leqslant 0.8$, eqs (2.1-80, 81 and 82) apply

$$\log N_1 = (12 + 16 S_{c,min} + 8 S_{c,min}^2)(1 - S_{c,max}) \quad (2.1\text{-}80)$$

$$\log N_2 = 0.2 \log N_1 (\log N_1 - 1) \quad (2.1\text{-}81)$$

$$\log N_3 = \log N_2 (0.3 - 0.375 \cdot S_{c,min}) / \Delta S_c \quad (2.1\text{-}82)$$

The relations (2.1-80) to (2.1-82) are valid for concrete tested under sealed conditions. They are also valid for large concrete sections of low permeability. Thin concrete sections which are allowed to dry may exhibit higher fatigue strengths, whereas permeable concrete immersed in water may have a lower fatigue strength than expressed by these relations.

(a) If $\log N_1 \leqslant 6$, then $\log N = \log N_1$.
(b) If $\log N_1 > 6$ and $\Delta S_c \geqslant 0.3 - 0.375 \cdot S_{c,min}$, then $\log N = \log N_2$.
(c) If $\log N_1 > 6$ and $\Delta S_c < 0.3 - 0.375 \cdot S_{c,min}$, then $\log N = \log N_3$.

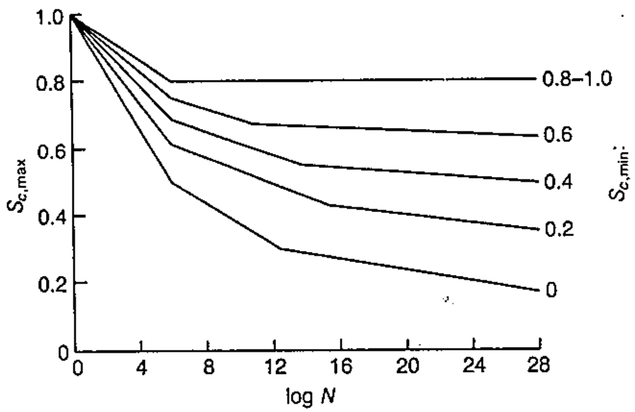


Fig. 2.1.9. S–N relations according to eqs (2.1–80) to (2.1–82)

with

$$S_{c,max} = |\sigma_{c,max}| / f_{ck,fat}$$

$$S_{c,min} = |\sigma_{c,min}| / f_{ck,fat}$$

$$\Delta S_c = |S_{c,max}| - |S_{c,min}|$$

The fatigue reference compressive strength $f_{ck,fat}$ has been introduced to take into account the increasing fatigue sensitivity of concrete with increasing compressive strength.

The fatigue reference compressive strength $f_{ck,fat}$ may be estimated from eq. (2.1–83)

$$f_{ck,fat} = \beta_{cc}(t)\beta_{c,sus}(t, t_0)f_{ck}(1 - f_{ck}/25f_{cko}) \quad (2.1\text{–}83)$$

If eq. (2.1–84) is applied it may be assumed that the concrete always fails in compression.

Compression–tension with $\sigma_{ct,max} \leqslant 0.026|\sigma_{c,max}|$

$$\log N = 9\,(1 - S_{c,max}) \quad (2.1\text{–}84)$$

Pure tension and tension-compression with $\sigma_{ct,max} > 0.026|\sigma_{c,max}|$

$$\log N = 12\,(1 - S_{ct,max}) \quad (2.1\text{–}85)$$

with

$$S_{ct,max} = \sigma_{ct,max}/f_{ctk,min}$$

where

N is the number of cycles to failure
$S_{c,max}$ is the maximum compressive stress level
$S_{c,min}$ is the minimum compressive stress level
$S_{ct,max}$ is the maximum tensile stress level
ΔS_c is the stress range
$\sigma_{c,max}$ is the maximum compressive stress
$\sigma_{c,min}$ is the minimum compressive stress
$\sigma_{ct,max}$ is the maximum tensile stress
f_{ck} is the characteristic compressive strength from Table 2.1.1
$f_{ck,fat}$ is the fatigue reference compressive strength from eq. (2.1–83)
f_{cko} = 10 MPa
$f_{ctk,min}$ is the minimum characteristic tensile strength from eq. (2.1–2)
$\beta_{cc}(t)$ is a coefficient which depends on the age of concrete at the beginning of fatigue loading, to be taken from clause 2.1.6.1, eq. (2.1–54)
$\beta_{c,sus}(t, t_0)$ is a coefficient which takes into account the effect of high mean stresses during loading (refer to clause 2.1.6.2, eq. (2.1–56): for fatigue loading it may be taken as $\beta_{c,sus}(t, t_0) = 0.85$.

The fatigue lives given by these equations correspond to a probability of failure $p = 5\%$ in a log-normal distribution for any given maximum stress. If limited data are available for an estimate of fatigue lives the evaluation of the 5% defective of fatigue life should be done at a confidence level of 75%.

Eqs (2.1–80) to (2.1–85) are applicable for stress levels $S_{c,max}$ and $S_{ct,max} < 0.9$ and for frequencies $f > 0.1$ cycle/min. For higher stress levels and lower frequencies, i.e. low cycle fatigue, lower values of $\log N$ than predicted by eqs (2.1–80) to (2.1–85) may be expected. For further details refer to CEB Bulletin 188.

A value of $\beta_{c,sus}(t, t_0) = 0.85$ has been chosen to take account of actual frequencies of loading which are in most practical cases significantly lower than those applied in experiments.

To estimate the fatigue life for a spectrum of load levels the Palmgren-Miner summation may be applied. Fatigue failure occurs if $D = 1$.

$$D = \sum_i \frac{n_{Si}}{n_{Ri}}$$

where

D is the fatigue damage

n_{Si} is the number of acting stress cycles at a given stress level and stress range
n_{Ri} is the number of cycles causing failure at the same stress level and stress range according to eqs (2.1–80) to (2.1–85).

2.1.7.2. Fatigue strains

For maximum compressive stresses $|\sigma_{c,max}| < 0.6 f_{ck}$ and a mean stress $(|\sigma_{c,max}| + |\sigma_{c,min}|)/2 < 0.5 f_{ck}$ the strain at maximum stress due to repeated loads of a given frequency f may be estimated from eq. (2.1–86):

$$\varepsilon_{cf}(n) = \frac{\sigma_{c,max}}{E_{ci}(t_0)} + \frac{\sigma_{c,max} + \sigma_{c,min}}{2E_{ci}} \phi(t, t_0) \qquad (2.1\text{–}86)$$

where

ε_{cf} is the strain at maximum stress due to repeated loads
$\sigma_{c,max}$ is the maximum compressive stress
$\sigma_{c,min}$ is the minimum compressive stress
E_{ci} is the modulus of elasticity of concrete at a concrete age of 28 days according to eq. (2.1–15)
$E_{ci}(t_0)$ is the modulus of elasticity of concrete at a concrete age t_0 according to (2.1–57)
$\phi(t, t_0)$ is the creep coefficient according to eq. (2.1–64)
t_0 is the age of concrete at the beginning of repeated loading (days)
t is the age of concrete at the moment considered (days).

In eq. (2.1–86) it is assumed that creep due to repeated loading is equal to creep under a constant stress $(\sigma_{c,max} + \sigma_{c,min})/2$ acting during a time $(t - t_0) = \frac{1}{1440} n/f$ = duration of repeated loading (days), where

n is the number of cycles applied at a frequency f
f is the frequency of repeated loading (min^{-1}).

Therefore, eq. (2.1–86) gives only a rough estimate of the creep strains due to repeated loads. It does not take into account variations of E_c due to repeated loads as well as of tertiary creep which develops prior to fatigue failure. For further details refer to CEB Bulletin 188.

2.1.8. Temperature effects

2.1.8.1. Range of application

The information given in the preceding sections is valid for a mean temperature taking into account seasonal variations, between approx. −20°C and +40°C. In the following section the effect of substantial deviations from a mean concrete temperature of 20°C for the range of approximately 0°C to +80°C is dealt with.

The data given in this section are limited to a maximum temperature +80°C because the information available on concrete properties for $T > 80°C$ is too complex for a Code type formulation, particularly with regard to the effects of type of aggregates and transient or steady moisture states. For such conditions experimental studies, using the particular concrete composition, are recommended.

2.1.8.2. Maturity

The effect of elevated or reduced temperatures on the maturity of concrete may be taken into account by adjusting the concrete age according to eq. (2.1–87):

The activation energy for concrete hydration is influenced by the type of cement and additions. Eq. (2.1–87) is valid for concretes made of Portland cements or cements containing only low amounts of components other than Portland cement clinker (CE I and CE II according to Appendix d, clause d.4.2.1).

$$t_T = \sum_{i=1}^{n} \Delta t_i \exp\left[13.65 - \frac{4000}{273 + T(\Delta t_i)/T_0}\right] \quad (2.1\text{–}87)$$

where

t_T is the temperature adjusted concrete age which replaces t in the corresponding equations
Δt_i is the number of days where a temperature T prevails
$T(\Delta t_i)$ is the temperature (°C) during the time period Δt_i
$T_0 = 1$°C.

The coefficient of thermal expansion depends on the type of aggregates and on the moisture state of the concrete. Thus it may vary between approx. $6 \times 10^{-6}\mathrm{K}^{-1}$ and $15 \times 10^{-6}\mathrm{K}^{-1}$. A value of $10 \times 10^{-6}\mathrm{K}^{-1}$ holds true, e.g. for concrete made of quartzitic aggregates.

2.1.8.3. Thermal expansion

Thermal expansion of concrete may be calculated from eq. (2.1–88)

$$\varepsilon_{cT} = \alpha_T \Delta T \quad (2.1\text{–}88)$$

where

ε_{cT} is the thermal strain
ΔT is the change of temperature (K)
α_T is the coefficient of thermal expansion (K^{-1}).

For the purpose of structural analysis the coefficient of thermal expansion may be taken as $\alpha_T = 10 \times 10^{-6}\mathrm{K}^{-1}$.

In cases where moisture exchange takes place, the effect of temperature on compressive strength depends on size and shape of the member. As a first approximation the effect of temperature in the range of 0°C < T < 80°C on compressive strength may be neglected since the reduction in strength due to a temperature increase is offset by an increase in strength due to drying.

2.1.8.4. Compressive strength

The effect of temperature at the time of testing in the range of 0°C < T < 80°C on the compressive strength of concrete without exchange of moisture (e.g. mass concrete) may be calculated from eq. (2.1–89)

$$f_{cm}(T) = f_{cm}(1.06 - 0.003T/T_0) \quad (2.1\text{–}89)$$

where

$f_{cm}(T)$ is the compressive strength at the temperature T
f_{cm} is the compressive strength at the temperature 20°C from eq. (2.1–1)
T is the temperature in (°C)
$T_0 = 1$°C.

2.1.8.5. Tensile strength and fracture properties

In the range of 0°C $< T <$ 80°C the uniaxial tensile strength f_{ct} and the tensile splitting strength $f_{ct,sp}$ are not significantly affected by temperature at the time of testing.

Eq. (2.1–90) may be used to estimate the effect of elevated or reduced temperatures at the time of testing on flexural strength $f_{ct,fl}$:

$$f_{ct,fl}(T) = f_{ct,fl}(1.1 - 0.005T/T_0) \quad (2.1\text{–}90)$$

where

$f_{ct,fl}(T)$ is the flexural strength at the temperature T
$f_{ct,fl}$ is the flexural strength at the temperature 20°C from eq. (2.1–6)
T is the temperature in (°C)
T_0 = 1°C.

The effect of temperature at testing on tensile strength of concrete strongly depends on the moisture state of the concrete and on temperature gradients. At elevated temperatures where moisture exchange takes place but no temperature gradient is expected, the effect of temperature may be neglected because a strength reduction due to increased temperature is offset by a strength increase due to drying. Only insufficient information is available on the influence of temperature gradients on the tensile strength.

Fracture energy G_F is strongly affected by temperature and moisture content at the time of testing. The effect of temperature on G_F may be estimated from eqs (2.1–91a) and (2.1–91b):

dry concrete: $$G_F(T) = G_F(1.06 - 0.003T/T_0) \quad (2.1\text{–}91a)$$

mass concrete: $$G_F(T) = G_F(1.12 - 0.006T/T_0) \quad (2.1\text{–}91b)$$

where

$G_F(T)$ is the fracture energy at a temperature T
G_F is the fracture energy at a temperature of 20°C from eq. (2.1–7)
T is the temperature (°C)
T_0 = 1°C.

2.1.8.6. Modulus of elasticity

The effect of elevated or reduced temperatures at the time of testing on the modulus of elasticity of concrete, at an age of 28 days without exchange of moisture, may be estimated from eq. (2.1–92)

$$E_{ci}(T) = E_{ci}(1.06 - 0.003T/T_0) \quad (2.1\text{–}92)$$

where

$E_{ci}(T)$ is the modulus of elasticity at the temperature T
E_{ci} is the modulus of elasticity at the temperature 20°C from eq. (2.1–15)
T is the temperature (°C)
T_0 = 1°C.

If moisture exchange takes place the effect of temperature on the modulus of elasticity is generally more pronounced than expressed by eq. (2.1–92).

2.1.8.7. Creep and shrinkage

2.1.8.7.1. Creep

The effect of temperature prior to loading may be taken into account using eq. (2.1–87).

Eqs (2.1–93) to (2.1–96) describe the effect of a constant temperature differing from 20°C while the concrete is under load.

The effect of temperature on the time development of creep is taken into account using $\beta_{H,T}$ from eq. 2.1–93

$$\beta_{H,T} = \beta_H \beta_T \quad (2.1\text{–}93)$$

with

$$\beta_T = \exp[1500/(273 + T/T_0) - 5.12] \quad (2.1\text{–}94)$$

where

$\beta_{H,T}$ is a temperature dependent coefficient replacing β_H in eq. (2.1–70)
β_H is a coefficient according to eq. (2.1–71)
$T_0 = 1°C$.

The effect of temperature on the creep coefficient is taken into account using eqs (2.1–95) and (2.1–96)

$$\phi_{RH,T} = \phi_T + (\phi_{RH} - 1)\phi_T^{1.2} \quad (2.1\text{–}95)$$

with

$$\phi_T = \exp[0.015(T/T_0 - 20)] \quad (2.1\text{–}96)$$

where

$\phi_{RH,T}$ is a temperature dependent coefficient which replaces ϕ_{RH} in eq. (2.1–65)
ϕ_{RH} is a coefficient according to eq. (2.1–66)
$T_0 = 1°C$.

For an increase of temperature while the structural member is under load, creep may be estimated from eq. (2.1–97)

$$\phi(t, t_0, T) = \phi_0 \beta_c(t - t_0) + \Delta\phi_{T,trans} \quad (2.1\text{–}97)$$

with

$$\Delta\phi_{T,trans} = 0.0004(T/T_0 - 20)^2 \quad (2.1\text{–}98)$$

The relations to predict the effect of temperature up to 80°C on creep given in this section are only rough estimates. For a more accurate prediction considerably more sophisticated models are required which take into account the moisture state of the concrete at the time of loading and distinguish between basic creep and drying creep. Neglecting these parameters the relations given in this section are generally more accurate for thick concrete members with little change in moisture content than for thin members where significant changes in moisture content occur, particularly at elevated temperatures.

where

ϕ_0 is the notional creep coefficient according to eq. (2.1–65) and temperature adjusted according to eq. (2.1–95)
$\beta_c(t - t_0)$ is a coefficient to describe the development of creep with time after loading according to eq. (2.1–70) and temperature adjusted according to eqs (2.1–93) and (2.1–94)
$\Delta\phi_{T,trans}$ is the transient thermal creep coefficient which occurs at the time of the temperature increase
$T_0 = 1°C$.

The effect of elevated temperatures on shrinkage is influenced considerably by the moisture content of the concrete prior to heating and the moisture loss after an increase of temperature. Eqs (2.1–99) to (2.1–101) represent shrinkage of a concrete after prolonged curing ($t_s > 14d$) or predrying.

2.1.8.7.2. Shrinkage

Eqs (2.1–99) to (2.1–101) describe the effect of a constant temperature differing from 20°C while the concrete is drying.

The effect of temperature on the time development of shrinkage is taken into account using $\alpha_{sT}(T)$ from eq. (2.1–99):

$$\alpha_{sT}(T) = 350\left(\frac{h}{h_o}\right)^2 \exp[-0.06(T/T_0 - 20)] \qquad (2.1\text{–}99)$$

where

$\alpha_{sT}(T)$ is a temperature dependent coefficient replacing the product $350(h/h_o)^2$ in eq. (2.1–79)
$h_o = 100$ mm
$T_0 = 1°C$.

The effect of temperature on the notional shrinkage coefficient is taken into account using eq. (2.1–100)

$$\beta_{RH,T} = \beta_{RH}\beta_{sT} \qquad (2.1\text{–}100)$$

with

$$\beta_{sT} = 1 + \left(\frac{8}{103 - 100RH/RH_o}\right)\left(\frac{T/T_0 - 20}{40}\right) \qquad (2.1\text{–}101)$$

where

$\beta_{RH,T}$ is a temperature dependent coefficient which replaces β_{RH} in eq. (2.1–75)
β_{RH} is a coefficient according to eq. (2.1–77)
$RH_o = 100\%$
$T_0 = 1°C$.

2.1.9. Transport of liquids and gases in hardened concrete

Liquids, gases or ions may be transported in hardened concrete by various transport mechanisms:

- permeation
- diffusion
- capillary suction
- mixed modes of transport mechanisms.

Transport characteristics are difficult to predict since they may vary by several orders of magnitude depending on concrete composition, type of materials, age, curing and moisture content of the concrete. Therefore, when a more accurate prediction of transport characteristics is required, they should be determined experimentally. For further details refer to RILEM-TC-116 PCD, State-of-the-Art Report: Performance Criteria for Concrete Durability. (To be published.)

2.1.9.1. Permeation

Permeation is the flow of liquids, e.g. water, or of gases, e.g. air, caused by a pressure head.

2.1.9.1.1. Water permeability

The transport of water is generally described by Darcy's law, eq. (2.1–102)

$$V = K_w \frac{A}{l} \Delta h_w t \qquad (2.1\text{–}102)$$

where

V is the volume of water (m^3) flowing during time t
Δh_w is the hydraulic head (m)
A is the penetrated area (m^2)
t is the time (s)
l is the thickness (m)
K_w is the coefficient of water permeability for water flow (m/s).

For mature concrete the coefficient of water permeability may be estimated roughly from the characteristic strength of concrete according to eq. (2.1–103):

$$\log(K_w/K_{wo}) = -0.7 f_{ck}/f_{cko} \qquad (2.1\text{–}103)$$

where

K_w is the coefficient of water permeability (m/s)
f_{ck} the characteristic strength (MPa)
$K_{wo} = 10^{-10}$ m/s
$f_{cko} = 10$ MPa.

In concrete the flow of water does not only occur in the capillary pores of the paste but also through internal microcracks as well as along the porous interfaces between the matrix and coarse aggregates. These effects compensate the low permeability of dense aggregates, so that in general the permeability of concrete is equal to or larger than the permeability of the hydrated cement paste matrix.

The flow of water in the hydrated cement paste depends on the presence of interconnected capillary pores which are mainly determined by the water/cement ratio of the mix and the degree of hydration of the cement. Despite a low water/cement ratio, insufficient curing, which may result in a low degree of hydration especially in the near surface region, may lead to a high permeability, whereas a high degree of hydration results in a low permeability even for higher water/cement ratios.

2.1.9.1.2. Gas permeability

For a stratified laminar flow the volume of gas flowing through a porous material is given by eq. (2.1–104):

$$V = K_g \frac{A}{l} \frac{p_1 - p_2}{\eta} p_m \frac{1}{p} t \qquad (2.1\text{–}104)$$

with

V is the volume of gas (m^3) flowing during time t
K_g is the coefficient of gas permeability (m^2)
A is the penetrated area (m^2)
l is the thickness (m) of the penetrated section
$p_1 - p_2$ is the pressure difference (N/m^2)
p_m is the mean pressure $= (p_1 + p_2)/2$ (N/m^2)
η is the viscosity of gas (Ns/m^2)
p is the local pressure, at which V is observed (Ns/m^2)
t is the time (s).

Similar to the flow of water, gases may pass through the pore system and microcracks of concrete under the influence of an external pressure. The coefficient of permeability K_g (m^2) in eq. (2.1–104) represents a constant material parameter. Therefore, the viscosity η of the gas flowing, as well as the pressure level p, have to be considered in the calculation of the volume of gas V.

If only one type of gas is considered η is normally taken as unity. Then K_g represents the specific permeability for the gas considered, and is given in (m/s).

If also the influence of the pressure level p_m is neglected, the volume of gas flowing can be calculated from

$$V = \bar{K}_g \frac{A}{l} \frac{p_1 - p_2}{p} t \qquad (2.1\text{–}105)$$

where

$\bar{K}_g$ is the coefficient of gas permeability (m^2/s).

As a rough estimate, K_g for air or oxygen may be determined from the characteristic compressive strength of concrete f_{ck} from eq. (2.1–106)

$$\log(K_g/K_{go}) = -0.5 f_{ck}/f_{cko} \qquad (2.1\text{–}106)$$

where

$K_{go} = 10^{-14}\,m^2$
$f_{cko} = 10\,MPa$
K_g is the coefficient of gas permeability (m^2).

Aside from the pore structure of the concrete, the moisture content exerts an essential influence on its gas permeability. Eq. (2.1–106) is valid for a relative pore humidity of the concrete of less than about 65%. With increasing relative humidity of the concrete K_g may be reduced by a factor up to 10^{-3}.

2.1.9.2. Diffusion

Gases, liquids and dissolved substances are transported due to a constant concentration gradient according to Fick's first law of diffusion

$$Q = D \frac{c_1 - c_2}{l} At \qquad (2.1\text{–}107)$$

where

Q is the amount of substance transported (g)

In most cases transient diffusion phenomena occur, i.e. the amount of substance diffusing varies with location x and time t. From Fick's first law of diffusion the balance for a volume element penetrated is derived as the second law of diffusion, which describes the change in concentration for an element with time according to eq. (2.1–108) which is valid for one-dimensional flow

$$\frac{\partial c}{\partial t} = D \frac{\partial^2 c}{\partial x^2} \qquad (2.1\text{–}108)$$

In cases where the diffusing substance becomes immobile, such as in the case of diffusion of chloride ions, eq. (2.1–108) has to be expanded

$$\frac{\partial c}{\partial t} = D\frac{\partial^2 c}{\partial x^2} + s \qquad (2.1\text{–}109a)$$

where s = sink, i.e. amount of transported substance which becomes immobile.

Frequently, the diffusion of ions is described by eq. (2.1–109b):

$$\frac{\partial c_{free}}{\partial t} = D_{eff}\frac{\partial^2 c_{free}}{\partial x^2} \qquad (2.1\text{–}109b)$$

where c_{free} = concentration of free ions, D_{eff} = effective diffusion coefficient. If some of the ions become immobile this is taken into account by an adjustment of the diffusion coefficient. Therefore, D_{eff} in eq. (2.1–109b) is not a constant but varies with time of exposure.

$c_1 - c_2$ is the difference in concentration (g/m^3)
l is the thickness of the penetrated section (m)
A is the penetrated area (m^2)
t is the time (s)
D is the diffusion coefficient (m^2/s).

The transport of water vapour in the pore system of concrete involves different transport mechanisms and driving forces, therefore $D \neq$ const. In most cases diffusion theory is applied to describe moisture migration. As driving force the local moisture concentration c (g/m^3) may be considered. However, a more convenient approach is the definition of a relative pore humidity $0 < H < 1$ which is correlated with the moisture concentration c by sorption isotherms.

For transient phenomena, such as drying of a concrete cross-section, the balance equation (2.1–108) is transformed to

$$\frac{\partial H}{\partial t} = \frac{\partial}{\partial x}\left(D(H)\frac{\partial H}{\partial x}\right) \qquad (2.1\text{–}112)$$

Eq. (2.1–111) is taken from Bazant, Z.P., Najjar, L.J., 'Drying of concrete as a non-linear diffusion problem', Cement and Concrete Research, Vol. 1, pp. 461–473, 1971.

2.1.9.2.1. Diffusion of water

The transport of water in the vapour phase can be described by Fick's first law of diffusion introducing a gradient of the relative pore humidity as the driving force. The diffusion coefficient D is a non-linear function of the local relative pore humidity H. The volume of water flowing is given by eq. (2.1–110)

$$V = D(H)\frac{dH}{dx}At \qquad (2.1\text{–}110)$$

where

V is the volume of transported water (m^3)
$D(H)$ is the diffusion coefficient (m^2/s) at relative pore humidity H
dH/dx is the gradient in relative pore humidity (m^{-1})
A is the penetrated area (m^2)
t is time (s).

For isothermal conditions the diffusion coefficient can be expressed as a function of the relative pore humidity $0 < H < 1$ according to eq. (2.1–111)

$$D(H) = D_1\left[\alpha + \frac{1-\alpha}{1+[(1-H)/(1-H_c)]^n}\right] \qquad (2.1\text{–}111)$$

where

D_1 is the maximum of $D(H)$ for $H = 1$ (m²/s)
D_0 is the minimum of $D(H)$ for $H = 0$ (m²/s)
$\alpha = D_0/D_1$
H_c is the relative pore humidity at $D(H) = 0.5D_1$
n is an exponent
H is the relative pore humidity.

The following approximate values may be assumed

$\alpha = 0.05$
$H_c = 0.80$
$n = 15.$

D_1 may be estimated from eq. (2.1–113)

$$D_1 = \frac{D_{1,o}}{f_{ck}/f_{cko}} \qquad (2.1\text{–}113)$$

with

$D_{1,o} = 1 \times 10^{-9}$ (m²/s)
$f_{cko} = 10\,\text{MPa}.$

2.1.9.2.2. Diffusion of gases

The diffusion of gases such as air, oxygen (O_2) or carbon dioxide (CO_2) is primarily controlled by the moisture content of the concrete. For intermediate moisture contents the diffusion coefficient for carbon dioxide or oxygen is in the range of

$$10^{-7} < D < 10^{-10}\,\text{m}^2/\text{s}$$

The diffusion coefficient for carbon dioxide D_{CO2} through carbonated concrete may be estimated from eq. (2.1–114)

$$\log(D_{CO2}/D_{CO2,O}) = -0.5 f_{ck}/f_{cko} \qquad (2.1\text{–}114)$$

with

$D_{CO2O} = 10^{-6.5}$ (m²/s)
$f_{cko} = 10\,\text{MPa}.$

Eq. (2.1–114) is valid for concrete stored in a constant environment of approximately 20°C, 65% relative humidity. For concrete exposed to a natural environment, particularly to rain, the diffusion coefficient is substantially lower than estimated from eq. (2.1–114).

Based on eqs (2.1–109) and (2.1–114) the progress of carbonation of a concrete under controlled conditions may be estimated from eq. (2.1–115):

$$d_c^2 = 2D_{CO2}\frac{C_a}{C_c}t \quad (2.1\text{–}115)$$

where

d_c is the depth of carbonation at time t (m)
D_{CO2} is the diffusion coefficient of CO_2 through carbonated concrete (m^2/s) (from eq. (2.1–114))
C_a is the concentration of CO_2 in the air (g/m^3)
C_c is the amount of CO_2 required for complete carbonation of a unit volume of concrete (g/m^3).

For normal weight concrete made of Portland cement and exposed to a standard environment, C_a/C_c may be taken as 8×10^{-6}.

It should be kept in mind, however, that in particular the relative humidity of the surrounding atmosphere as well as the properties of a particular concrete have a strong influence on D_{CO2} so that eq. (2.1–115) cannot give a reliable estimate of the progress of carbonation of a structure in service.

The prediction of the transport of chloride ions into concrete is very complex because chlorides penetrating into concrete may be transported not only by diffusion but also by capillary suction of a salt solution. In addition, the external chloride concentration is variable, and parts of the chloride ions intruded become immobile due to chemical reaction or time dependent physical adsorption. The amount of chlorides combined depends on the type of cement used and must be in equilibrium with the concentration of chlorides dissolved in the pore water. Only the dissolved chlorides take part in the diffusion process. In carbonated concrete all chlorides are dissolved in the pore water.

2.1.9.2.3. Diffusion of chloride ions

The diffusion coefficients of dissolved substances increase with increasing moisture content of the concrete. For chloride ions the effective diffusion coefficient as defined in eq. (2.1–109b) is in the range of

D_{Cl^-} = 1 to 10×10^{-12} m^2/s for concretes made of Portland cement
D_{Cl^-} = 0.3 to 5×10^{-12} m^2/s for concretes made of Portland blast furnace slag-cements.

2.1.9.3. Capillary suction

Liquids, particularly water, may be transported into concrete by capillary suction or absorption. Water absorption may be expressed by eq. (2.1-116)

$$w = w_1 (t/t_1)^n = M_w t^n \qquad (2.1\text{-}116)$$

where

w is the water absorbed per unit area at time t (m^3/m^2)
w_1 is the water absorbed at a given time t_1
t is the duration of water absorption (s)
$n = 0.5$
$M_w = W_1/t_1^n$ is the coefficient of water absorption ($m/s^{0.5}$).

Similar to water permeability, capillary suction is strongly influenced by the moisture content of the concrete. As the pore humidity of the concrete increases the rate of water absorption and thus M_w decrease.

For a uniform pore humidity and no substantial microstructural variations within a concrete section exposed to capillary suction, the exponent n in eq. (2.1-116) may be taken as $n = 0.5$. If the moisture distribution is non-uniform, $n < 0.5$.

For a rough estimate the logarithm of the coefficient of water absorption for a given concrete grade may be determined from eq. (2.1-117):

$$\log(M_w/M_{wo}) \; - \; 0.2 f_{ck}/f_{cko} \qquad (2.1\text{-}117)$$

where

$M_{wo} = 10^{-4}$ ($m/s^{0.5}$)
$f_{cko} = 10$ MPa.

Eq. (2.1-117) is valid for a uniform pore humidity of the concrete of approximately 65%. The coefficient of water absorption, depends not only on the moisture state of the concrete, but also on microstructural parameters, so that predictions solely based on a concrete grade are rather uncertain.

2.2. REINFORCING STEEL

2.2.1. General

Products used as reinforcing steel may be

- bars
- wires
- coiled rods
- welded fabric.

The methods of production, the methods of testing and of certification of conformity are as defined in relevant ISO or CEN Standards or RILEM Recommendations.

Reinforcing steel is characterized by

- geometry
  - size
  - surface characteristics
- mechanical properties
  - strength and yield stress
  - ductility
  - fatigue behaviour
  - behaviour at extreme temperature

- technological properties
  - — bond
  - — bendability
  - — weldability
  - — thermal expansion.

Types of reinforcement not covered by approval documents, can be used after it has been shown that they meet the specified requirements.

The mechanical and technological properties of reinforcement for structures are defined by Product Standards, and are generally secured by certification schemes and certificates of compliance.

Mechanical devices for splicing are dealt with in clause 9.1.2.4.

2.2.2. Classification

Reinforcing steels are normally classified on the basis of their

- size
- characteristic yield stress, which defines the grade
- ductility
- surface characteristics and bond properties
- weldability.

Each product should be clearly identifiable with respect to this classification.

The simultaneous use of steels of various types on the same site is allowed only on condition that no confusion between the types is possible during the construction.

It should be possible to distinguish clearly between

- plain bars of various grades or of various ductility classes
- high bond bars of various grades or of various ductility classes
- reinforcement that is weldable and that which is not.

2.2.3. Geometry

2.2.3.1. Size

For quality control purposes and design calculations, the mechanical properties of a product are referred to the nominal cross-sectional area.

The difference between actual and nominal cross-sectional area should not exceed the limiting values specified in relevant standards.

The actual section is determined by weighing a given length of bar, assuming a density of 7850 kg/m^3.

The nominal diameter is defined as the diameter of a plain circular cylinder of the same weight per unit length as the bar.

For welded fabric the following applies

- twin bars are allowed in one direction only
- adequate stiffness of the fabric should be ensured either by a limitation of the maximum spacing of the bars, or by introducing a minimum ratio between the diameter of the transverse bars and the diameter of the longitudinal bars.

2.2.3.2. Surface characteristics

Three shapes or surface characteristics are defined

- plain
- indented
- ribbed.

Ribbed bars or wires are considered as high bond if they satisfy the conditions and requirements imposed by the relevant standards or by the approval documents.

Plain bars do not satisfy these conditions.

For indented wires, reference should be made to relevant standards or technical documents.

Plain smooth wires (cold drawn wires) should not be used for reinforced concrete, except as non-structural reinforcement (spacers etc.) or in the form of welded fabric.

2.2.4. Mechanical properties

The mechanical properties are defined on the basis of standard tests.

The standard tests are defined in relevant ISO and CEN Standards and RILEM Recommendations.

2.2.4.1. Tensile properties

The characteristic values of

- the tensile strength (f_t)
- the yield stress (f_y)
- the total elongation at maximum load (ε_u)

respectively, are denoted f_{tk}, f_{yk} and ε_{uk}.

The requirements apply to the product in the condition in which it is delivered. In the case of coiled rods, the requirements apply to the material after straightening.

The value of f_{yk} should correspond to a 0.2% offset in the characteristic $\sigma - \varepsilon$ diagram.

For steels totally or partially cold-worked by means of axial tension, it will generally be the case that

$$f_{yc} \neq f_{yt}$$

where f_{yc} and f_{yt} are actual yield stresses for compression and tension respectively. The value of f_{yc} to be used in a calculation should therefore be stipulated in the approval documents.

2.2.4.2. Steel grades

The steel grade denotes the value of the specified characteristic yield stress in MPa. The Model Code contemplates reinforcing steel up to Grade 500.

Grades higher than 500 require further consideration concerning the validity of the given rules.

2.2.4.3. Stress–strain diagram

Indicative stress–strain diagrams of reinforcing steel in tension are represented in Fig. 2.2.1.

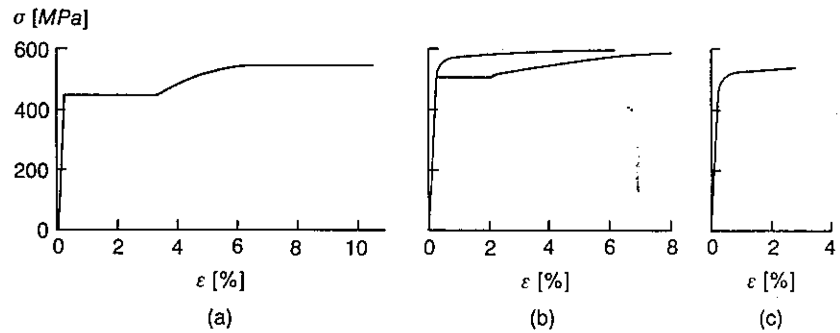


Fig. 2.2.1. Stress–strain relationships of reinforcing steel: (a) hot-rolled bars; heat-treated bars; micro-alloyed bars; (b) low-carbon, heat-treated bars; cold-worked bars; (c) cold-worked wires

Due to the diversity and evolution of the manufacturing processes for bars and wires, various stress–strain diagrams may be encountered.

As a simplification, actual stress–strain diagrams can in calculations be replaced by an idealized characteristic diagram according to Fig. 2.2.2, assuming a modulus of elasticity E_s equal to 200 GPa.

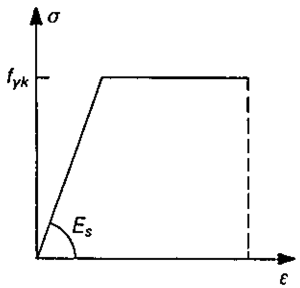


Fig. 2.2.2. Idealized stress–strain diagram

The actual diagram for a particular steel may be used if it is duly verified by the producer.

For high strength steels, the $\sigma - \varepsilon$ diagram is non-symmetrical in compression and in tension.

Some cold-worked steels have a lower modulus of elasticity in compression than in tension. The difference is not important in practice.

2.2.4.4. Ductility

Three ductility classes are defined for design purposes.

These classes are defined by minimum specified values for the characteristic value of the ratio f_t/f_y and the characteristic elongation at maximum load ε_{uk}, as follows.

Class A: $(f_t/f_y)_k \geqslant 1.08$ and $\varepsilon_{uk} \geqslant 5\%$
Class B: $(f_t/f_y)_k \geqslant 1.05$ and $\varepsilon_{uk} \geqslant 2.5\%$
Class S: $(f_t/f_y)_k \geqslant 1.15$ and $\varepsilon_{uk} \geqslant 6\%$.

Adequate ductility is necessary whether or not moment redistribution is taken into account in the calculations.

The characteristic value of the ratio f_t/f_y corresponds to the 5% fractile of the relation between actual tensile strength and actual yield stress.

Class S should be used where high ductility of the structure is required (e.g. in seismic regions).

In seismic design an additional requirement for Class S can be introduced.

Class S: f [illegible] $/f_{yk,nom} \leqslant 1.3$

2.2.4.5. Fatigue behaviour

The fatigue behaviour of reinforcing steel is described in Table 6.7.1.

Fatigue behaviour depends on factors such as bar size, rib geometry, bending of bars and welded connections, thus making it difficult to give generalized S-N curves. More information can be found in CEB Bulletin d'Information No. 188 'Fatigue of Concrete Structures'.

2.2.5. Technological properties

2.2.5.1. Bond properties

(a) Bars

For ribbed and for some indented products, the bond properties are quantified by means of the projected rib factors.

Ribbed products, having a projected rib factor satisfying the minimum requirements given by the relevant standards, may be assumed to be high bond bars.

Bars not satisfying these requirements should be treated as plain bars with respect to bond. For indented products, which cannot be considered as high bond bars, reference should be made to relevant standards or technical documents.

Poor straightening of ribbed bars and wires from coils can significantly reduce the projected rib factor and thus the bond properties of the straightened bars or wires.

(b) Welded fabric

Where welded joints are taken into account for calculation of the anchorage, each welded joint shall be capable of withstanding a shearing force not less than $0.3A_s f_{yk}$, where A_s denotes the nominal cross-sectional area of the anchored wire.

2.2.5.2. Bendability

The requirements concerning the bendability are specified in relevant standards.

Reinforcing bars should not be bent to a radius less than that used in the relevant rebend test specified in the standards.

2.2.5.3. Weldability

The requirements concerning the weldability are specified in relevant standards.

Depending on the type of reinforcement used, the methods for welding may be restricted.

2.2.5.4. Coefficient of thermal expansion

Within the temperature range from $-20°C$ to $180°C$ the coefficient of thermal expansion of steel may be taken as $10 \times 10^{-6}/°C$.

2.3. PRESTRESSING STEEL

2.3.1. General

Steels for prestressing are delivered as

- wires
- strands
- bars.

Prestressing steel is characterized by

- geometry
  - — size
  - — surface characteristics
- mechanical properties
  - — tensile strength and 0.1% proof-stress
  - — modulus of elasticity
  - — ductility
  - — relaxation
  - — fatigue behaviour
  - — behaviour at extreme temperatures
- technological properties
  - — surface conditions
  - — corrosion resistance
  - — thermal expansion.

The methods of production, the method of testing and of certification of conformity are defined in relevant ISO or CEN Standards or RILEM Recommendations.

The mechanical and technological properties of prestressing steels are defined by standards and are secured by certification schemes and certificates of conformity.

2.3.2. Classification

The classification of prestressing steel is based on values of

- the characteristic tensile strength, which defines the grade
- the characteristics 0.1% proof-stress
- the relaxation class.

Characteristic values are defined in clause 2.3.4.1.

Grade denotes the characteristic tensile strength in MPa, rounded off in tens.

Each product shall be clearly identifiable with respect to this classification.

Each coil of wire or strand or each quantity of bars shall carry a label giving at least

(a) the producer's name
(b) the product: wire, strand or bar
(c) the letters FeP followed by the grade
(d) the nominal dimensions (see 2.3.3.1)
(e) the surface characteristics (see 2.3.3.2)
(f) the relaxation class (see 2.3.4.5).

2.3.3. Geometry

2.3.3.1. Size

For quality control purposes and design calculations, the mechanical properties of a product are referred to the nominal cross-sectional area.

Details of dimensions and configuration along with the accepted tolerances, should be indicated in standards or technical approval documents.

For the test procedures, see document RILEM RPC 10.

2.3.3.2. Surface characteristics

Three shapes or surface characteristics are defined

- smooth
- indented
- ribbed.

2.3.4. Mechanical properties

The mechanical properties are defined on the basis of standard tests.

See document RILEM RPC 9.

The standard tests are defined in relevant ISO and CEN Standards and RILEM Recommendations.

2.3.4.1. Tensile properties

The characteristic values of

- the tensile strength (f_{pt})
- the 0.1% proof-stress ($f_{p0.1}$)
- the total elongation at maximum load (ε_{pu})

respectively designated f_{ptk}, $f_{p0.1k}$ and ε_{puk}, are specified corresponding to the 5% fractile.

The condition $f_{p0.1k} \geqslant 0.80 f_{ptk}$ should be fulfilled.

The characteristic tensile strength is derived from the characteristic failure load.

$0.9 f_{ptk}$ is generally assumed to be a good estimate for $f_{p0.2}$, for design purposes it may even be used for $f_{p0.1k}$, see Fig. 2.3.2.

For convenience in presentation $f_{p0.1k}$ or $f_{p0.2k}$ as relevant for the type of steel used is taken as f_{pyk}.

2.3.4.2. Modulus of elasticity

In case more precise information is not available, the modulus of elasticity of prestressing steel may be taken as

- 205 GPa for wires and bars
- 195 GPa for strands.

2.3.4.3. Force–strain diagrams

Indicative curves are given in Fig. 2.3.1(a) for wires and Fig. 2.3.1(b) for strands.

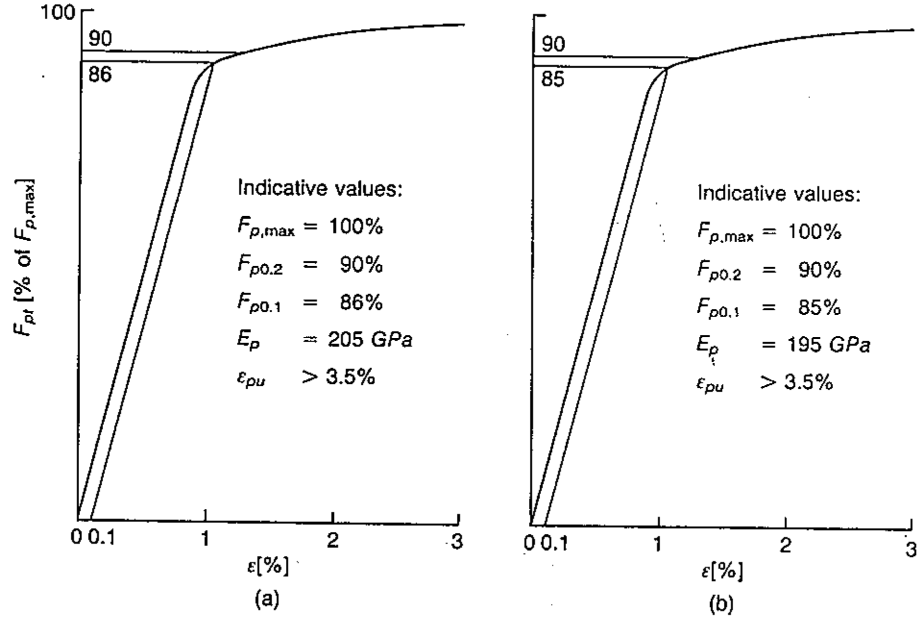


Fig. 2.3.1. Force–strain diagram: (a) for cold-drawn stress-relieved wires; (b) for stress-relieved strands

For calculation purposes the actual stress–strain diagram (corresponding to $f_{pt} = f_{ptk}$) can be replaced by a simplified schematic diagram.

However, it may be necessary in some design situations (see subsections 6.2.4 and 6.2.5) to use the actual stress–strain diagram (duly factored) instead of the idealized one.

Figure 2.3.2 shows a simplified bi-linear diagram. It is valid for temperatures from −20°C to 100°C.

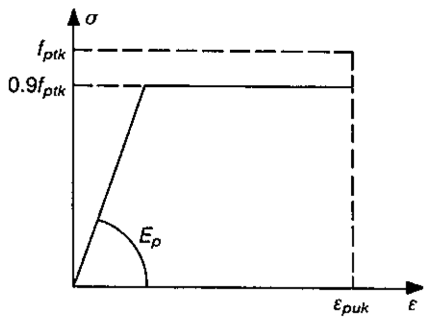


Fig. 2.3.2. Schematic stress–strain diagram for prestressing steel

2.3.4.4. Ductility

(*a*) *Reverse bending behaviour*
The minimum number of reverse bends should be

- for smooth wires: 4
- for indented wires: 3.

The minimum number of reverse bends is determined according to ISO 7801.

(*b*) *Behaviour under multiaxial stresses*
Prestressing steels should have an adequate behaviour under multiaxial stresses.

Tests on strands should be carried out according to a test method developed by FIP (see 'Deflected tensile test', FIP Notes 1987/1).
According to this test method, the behaviour may be assumed adequate if the value of 'D' (defined in the document mentioned above) does not exceed a maximum of 28.

(*c*) *Constriction*
The reduction in area after failure should be visible to the naked eye.

(*d*) *Unit elongation at maximum load*
The unit elongation at maximum load (ε_{uk}) shall be at least equal to 0.035.

2.3.4.5. Relaxation

Prestressing steels are divided into relaxation classes which refer to the relaxation at 1000 hours (ρ_{1000}) for initial stresses equal to 0.6, 0.7 and 0.8 times f_{ptk}.
Three relaxation classes are defined

- class 1: normal relaxation characteristics for wires and strands
- class 2: improved relaxation characteristics for wires and strands
- class 3: relaxation characteristic for bars.

For the relaxation test procedure, see document RILEM RPC 15.

For design purposes, the values according to Fig. 2.3.3 can be used.

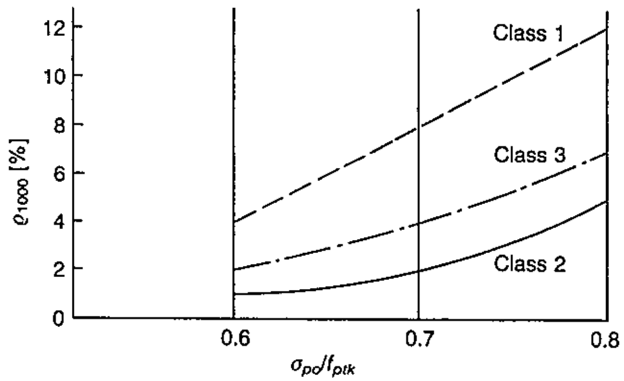


Fig. 2.3.3. Relaxation losses in % for different stress levels and relaxation classes

An indication of how relaxation varies with time up to 1000 hours is given in Table 2.3.1.

Table 2.3.1. Relationship between relaxation losses and time up to 1000 hours

Time in hours	1	5	20	100	200	500	1000
Relaxation losses as percentage of losses in 1000 hours	25	45	55	70	80	90	100

For an estimate of the relaxation up to 30 years the following formula may be applied

$$\rho_t = \rho_{1000} \left(\frac{t}{1000} \right)^k$$

where:

ρ_t is the relaxation after t hours
ρ_{100} is the relaxation after 100 hours
ρ_{1000} is the relaxation after 1000 hours

$k \approx \log(\rho_{1000}/\rho_{100})$

k to be 0.12 for relaxation class 1, and 0.19 for relaxation class 2.

Normally, the long-term values of the relaxation are taken from long-term tests. However, it may be assumed that the relaxation after 50 years and more is three times the relaxation after 1000 hours.

2.3.4.6. Fatigue behaviour

The characteristic fatigue strength (stress range) for 2×10^6 cycles, with a maximum stress being $0.7 f_{ptk}$, is given in Table 2.3.2

Table 2.3.2. Characteristic fatigue strength for 2×10^6 cycles

Basic material	$\Delta\sigma$ (N/mm²)
Wires (cold drawn or hot rolled)	
• smooth	200
• indented	180
Strands	
• from smooth wires	190
• from indented wires	170
Smooth bars	200
Ribbed bars	180

The values of Table 2.3.2 refer to the basic material as tested in the laboratory. For the performance of the embedded product used in the structure, refer to Table 6.7.2.

2.3.5. Technological properties

2.3.5.1. Surface conditions

Prestressing steel should be free from defects, which may have occurred at any stage from manufacture up to installation, to a degree which could impair its performance as a prestressing element.

The surface of steel should be free from corrosion defects.

However, a thin film of rust which can be removed with a dry cloth may be tolerated.

2.3.5.2. Corrosion resistance

The time of exposure in ammonium thiocyanate till failure should not be less than the values given in the approval documents.

For the test method, see 'The FIP stress-corrosion test with ammonium thiocyanate', FIP Special Report 1988/1. The test is carried out with a tension of $0.8F_{ptk}$.

The recommended lowest values of exposure are given in Table 2.3.3.

Table 2.3.3. Stress-corrosion test with ammonium thiocyanate: recommended lowest values of exposure time

Product	Lowest exposure time to failure (hours)	Exposure time to failure of 50% of test samples (hours)
Wire	1.5	4
Strand	1.5	4
Bar < 12 mm	20	50
Bar 12–25 mm	60	250
Bar 25–40 mm	100	400

2.3.5.3. Thermal expansion

Within the temperature range from −20°C to 100°C the coefficient of thermal expansion may be taken as 10×10^{-6}/°C.

This chapter will be further developed in the future.

3. GENERAL MODELS

This chapter contains engineering models describing the mechanical behaviour of reinforced concrete sub-elements. These models may serve either as a background for simpler models used in the operational parts of the Model Code, or as an input to more advanced design methods.

However, further assessment of their reliability is needed before these models can be used directly in practical design.

3.1. BOND STRESS–SLIP RELATIONSHIP

Under well defined conditions, it is possible to consider that there is an average 'local bond' versus 'local slip' relationship, statistically acceptable.

The bond stress–slip relationship depends on a considerable number of influencing factors like bar roughness (related rib area), concrete strength, position and orientation of the bar during casting, state of stress, boundary conditions and concrete cover.

Therefore the bond stress–slip curve, presented in Fig. 3.1.1 can be considered as a statistical mean curve, applicable as an average formulation for a broad range of cases. Further reliability handling would be needed to derive design bond stress–slip curves.

The first curved part refers to the stage in which the ribs penetrate into the mortar matrix, characterized by local crushing and micro-cracking. The horizontal level occurs only for confined concrete, referring to advanced crushing and shearing off of the concrete between the ribs. The decreasing branch refers to the reduction of bond resistance due to the occurrence of splitting cracks along the bars. The horizontal part represents a residual bond capacity, which is maintained by virtue of a minimum transverse reinforcement, keeping a certain degree of integrity intact.

With regard to the generation of bond stresses the following considerations apply.

Reinforcement and concrete have the same strain ($\varepsilon_s = \varepsilon_c$) in those areas of the structure which are under compression and in uncracked parts of the structure under tension.

In cracked cross-sections the tension forces in the crack are transferred by the reinforcing steel. In general, the absolute displacements of the steel u_s, and of the concrete u_c between two cracks or along the transmission length l_t are different.

Due to the relative displacement $s = u_s - u_c$ bond stresses are generated between the concrete and the reinforcing steel. The magnitude of these bond stresses depends predominantly on the surface of the reinforcing steel, the slip, the concrete strength f_c and the position of the reinforcing steel during concreting. Between cracks or along the transmission length l_t, a part of the tension force of the reinforcing steel, acting in the crack, is transferred into the concrete by bond (tension stiffening effect).

The local decrease of the relative displacement along the transmission length l_t is characterized by the strain difference

$$ds/dx = \varepsilon_s - \varepsilon_c$$

Depending on the selection of the coefficient α ($0 \leqslant \alpha \leqslant 1$) in eq. (3.1–1) all usual forms of a bond stress–slip relationship can be modelled, starting from a bond characteristic with a constant stress ($\alpha = 0$) up to a bond stress–slip relationship with linear increasing bond stress ($\alpha = 1$).

The parameters given in Table 3.1.1 are valid for ribbed reinforcing steel with a related rib area $A_{sr} \approx A_{sr,\min}$ according to relevant international standards, depending on the main influencing factors: confinement, bond condition and concrete strength.

Table 3.1.1. Parameters for defining the mean bond stress–slip relationship (according to eqs (3.1–1) to (3.1–4))

	Column 2	Column 3	Column 4	Column 5
	Unconfined concrete*		Confined concrete†	
	Good bond conditions	All other bond conditions	Good bond conditions	All other bond conditions
s_1	0.6 mm	0.6 mm	1.0 mm	1.0 mm
s_2	0.6 mm	0.6 mm	3.0 mm	3.0 mm
s_3	1.0 mm	2.5 mm	Clear rib spacing	Clear rib spacing
α	0.4	0.4	0.4	0.4
τ_{max}	$2.0\sqrt{f_{ck}}$	$1.0\sqrt{f_{ck}}$	$2.5\sqrt{f_{ck}}$	$1.25\sqrt{f_{ck}}$
τ_f	$0.15\tau_{max}$	$0.15\tau_{max}$	$0.40\tau_{max}$	$0.40\tau_{max}$

*Failure by splitting of the concrete.
†Failure by shearing of the concrete between the ribs.

3.1.1. Local bond stress–slip model

For monotonic loading the bond stresses between concrete and reinforcing bar can be calculated as a function of the relative displacement s according to eqs (3.1-1) to (3.1-4)

$$\tau = \tau_{max}(s/s_1)^{\alpha} \text{ for } 0 \leqslant s \leqslant s_1 \qquad (3.1\text{–}1)$$

$$\tau = \tau_{max} \text{ for } s_1 < s \leqslant s_2 \qquad (3.1\text{–}2)$$

$$\tau = \tau_{max} - (\tau_{max} - \tau_f)\left(\frac{s - s_2}{s_3 - s_2}\right) \text{ for } s_2 < s \leqslant s_3 \qquad (3.1\text{–}3)$$

$$\tau = \tau_f \text{ for } s_3 < s \qquad (3.1\text{–}4)$$

See also Fig. 3.1.1.

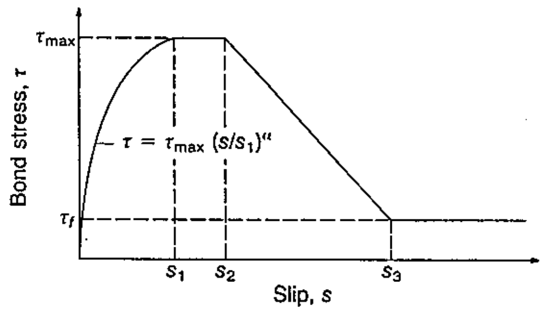


Fig. 3.1.1. Analytical bond stress–slip relationship (monotonic loading)

For bars with a related rib area $A_{sr} > A_{sr,\min}$ the bond strength τ_{max} increases and the characteristic slip value s_1 decreases. This influence is, however, neglected. Furthermore the stiffness of the increasing branch of the bond stress–slip relationship differs from confined concrete to unconfined concrete, but this influence is also neglected. Finally the dependence of bond properties upon concrete compaction and curing is neglected as well.

The values in the Tables 3.1.1 and 3.1.2 are applicable only in loading states for which the concrete is not subjected to lateral tension.

The values given in Table 3.1.1, columns 2 and 3, are valid for a concrete cover $c = 1\phi_s$, and a minimum transverse reinforcement equal to

$$A_{st,\min} = 0.25 n A_s$$

where

A_{st} area of stirrups (two legs) over a length equal to the anchorage length
n number of bars enclosed by stirrups
A_s area of one bar.

The values in columns 4 and 5 are valid for well confined concrete (concrete cover $c \geqslant 5\phi_s$, clear spacing $> 10\phi_s$ or closely spaced transverse (enclosing) reinforcement with an area $A_{st} > nA_s$) or high transverse pressure ($p \geqslant 7.5$ MPa as average transverse pressure under design load).

If the transverse reinforcement A_{st} is

$$A_{st,\min} < A_{st} < nA_s$$

or the transverse pressure p is

$$0 < p < 7.5\,\text{MPa}$$

the values for s_1, s_3, $\tau_{\max}$ and τ_f may be interpolated linearly between the values for unconfined and for confined concrete respectively. If a transverse reinforcement $A_{st} > A_{st,\min}$ is present simultaneously with a transverse pressure the effects may be added.

The values given in Table 3.1.1 are valid for those parts of the reinforcing bars which are a distance of $x > 5\phi_s$ from the crack.

For those parts of the reinforcing bar which are a distance $x \leqslant 5\phi_s$ from the next transverse crack, the bond stress τ and the slip s are to be reduced by the factor λ, where

$$\lambda = 0.2 \frac{x}{\phi_s} \leqslant 1$$

The parameters given in Table 3.1.2 are valid for smooth reinforcing steel, depending on the main influencing factors: roughness of the bar surface, bond conditions and concrete strength. They are valid for confined and unconfined concrete.

Table 3.1.2. Parameters for defining the bond stress–slip relationship of smooth bars (according to eqs (3.1.–1) to (3.1–4))

	Cold drawn wire		Hot rolled bars	
	Good bond conditions	All other bond conditions	Good bond conditions	All other bond conditions
$s_1 = s_2 = s_3$	0.01 mm	0.01 mm	0.1 mm	0.1 mm
α	0.5	0.5	0.5	0.5
$\tau_{max} = \tau_f$	$0.1\sqrt{f_{ck}}$	$0.05\sqrt{f_{ck}}$	$0.3\sqrt{f_{ck}}$	$0.15\sqrt{f_{ck}}$

The parameters given in Tables 3.1.1 and 3.1.2 are mean values.

It has to be kept in mind, that the scatter of different test series is considerable especially for small values of the slip. For a given value of the slip the coefficient of variation of the bond stresses may amount to approx. 30%. The scatter is due to the use of different test specimens and the hereby created different states of stress in the concrete surrounding the reinforcing bar, to the different measuring techniques, and to the different loading and deformation velocities. The heterogeneity of the concrete and the geometry of the reinforcing bars (related rib area, diameters) have also a significant influence on the $\tau - s$ relationship. The designer should take account of this scatter as far as possible, at least in the cases where a more accurate design is necessary.

The unloading branch of the bond stress–slip relationship is linear and valid for the increasing and horizontal part of the diagram. The slope S (see Fig. 3.1.2) is independent of the slip value s, and has an average value of $S = 200\,N/mm^3$.

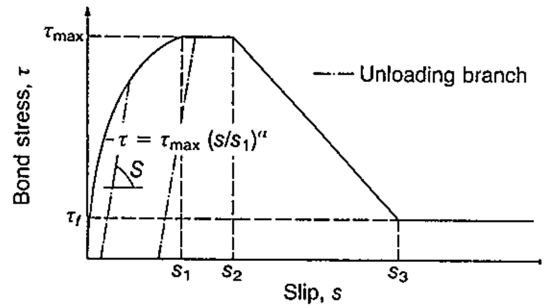


Fig. 3.1.2. Unloading branch of the $\tau - s$ relationship

3.1.2. Influence of creep

Creep influences will reduce the slope of the increasing part of the $\tau - s$ relationship.

The creep displacements can be described with isochrone curves (see Fig. 3.1.3). The slip $s_{n,t}$ due to a permanent load or a repeated loading can be calculated according to eq. (3.1–5)

$$s_{n,t} = s(1 + k_{n,t}) \qquad (3.1\text{–}5)$$

where the displacement factor k_t for a permanent load can be calculated according to eq. (3.1–6)

$$k_t = (1 + 10t)^{0.080} - 1 \qquad (3.1\text{–}6)$$

where t is the load duration (hours).

For a repeated loading the displacement factor k_n can be determined by eq. (3.1–7)

$$k_n = (1 + n)^{0.107} - 1 \qquad (3.1\text{–}7)$$

where n is the number of load cycles.

The validity of this relation is restricted to the ascending branch of the bond–slip relationship.

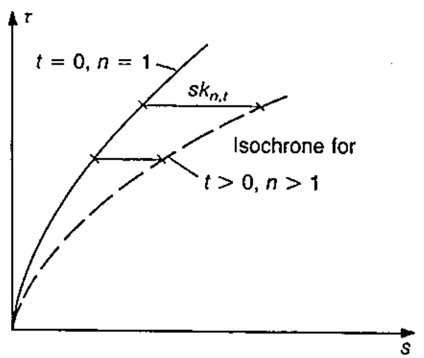


Fig. 3.1.3. Creep effects on the $\tau - s$ curve

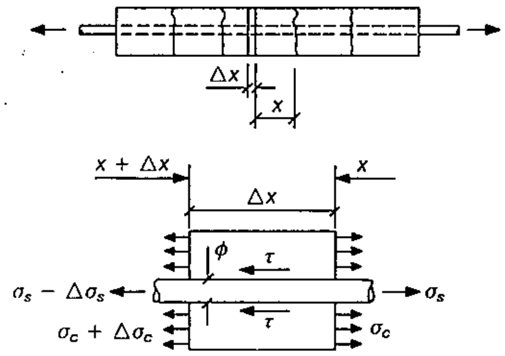


Fig. 3.1.4. Bond stress–slip model: $\varepsilon_{s,m}\Delta x = \varepsilon_{c,m}\Delta x + \Delta s$ *(slip)*

Section 3.2 gives simplified constitutive laws for reinforced or prestressed concrete in pure tension after cracking. From the first crack up to yielding distinction should be made between the crack formation phase, in which new cracks occur, and the stabilized cracking phase in which only crack widening is supposed to occur.

If the tension stiffening effect is neglected, the stiffness of a reinforced concrete bar or a structural member is underestimated.

3.1.3. Applications of the model

As possible applications of the bond stress–slip model, the following cases may be mentioned.

(*a*) *Reinforced concrete tie.* Taking into account the equilibrium and compatibility conditions in an elementary length of a reinforced concrete tie, as well as its boundary conditions, it is possible to use the model described in the previous clauses to predict crack formations and the elongation of the tie. Tension stiffening effects may thus be studied.

(*b*) *Anchorage of bars.* Similarly, and under different boundary conditions, the steel–stress versus the pull-out displacement of a bar may be studied.

3.2. TENSION STIFFENING EFFECTS

3.2.1. Definition

In a cracked cross-section all tensile forces are balanced by the steel only. However, between adjacent cracks, tensile forces are transmitted from the steel to the surrounding concrete by bond forces. The contribution of the concrete may be considered to increase the stiffness of the tensile reinforcement. Therefore this effect is called the 'Tension Stiffening Effect'.

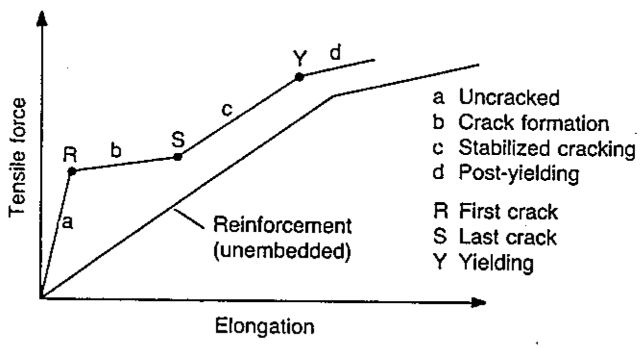


Fig. 3.2.1. Idealized behaviour of a reinforced concrete tie

The distinction between the stages uncracked concrete, crack formation phase, stabilized cracking and post-yielding is helpful in estimating deformation, crack width and damping.

More detailed information on the cracking process is given in clause 7.4.3.1.

For plain concrete under tension, see clause 2.1.4.4.2.

First crack

When the first crack occurs the distribution of steel and concrete strain within the transmission length l_t is given in Fig. 3.2.2.

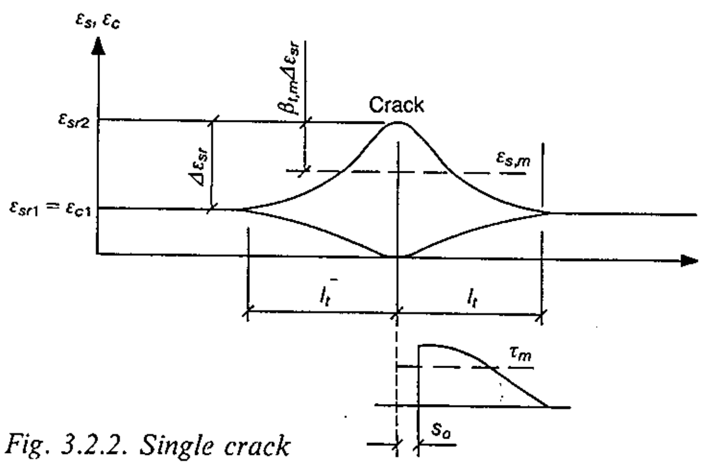


Fig. 3.2.2. Single crack

3.2.2. Crack pattern

During the state of crack formation one crack after the other occurs thus decreasing the stiffness of the member. When cracks appear, single cracks play an important role. In this state some part of the area between the cracks remains in state I ($\varepsilon_s = \varepsilon_c$). When the crack pattern has stabilized, the distance between the cracks s_r is equal to or less than twice the transmission length l_t (length over which slip between steel and concrete occurs).

The mean steel strain may be expressed as

$$\varepsilon_{s,m} = \varepsilon_{s2} - \beta_{t,m}\Delta\varepsilon_{sr} = \varepsilon_{s2} - \beta_{t,m}(\varepsilon_{sr2} - \varepsilon_{sr1}) \qquad (3.2\text{–}1)$$

where $\beta_{t,m}$ is considered to be an integration factor for the steel strain along the transmission length. (In clause 7.4.3.1 the abbreviation $\beta = \beta_{t,m}$ is used.)

$\beta_{t,m} = 0.60$ for pure tension

$\varepsilon_{s,m}$ is the mean steel strain
ε_{s1} is the strain of reinforcement in uncracked concrete
ε_{s2} is the strain of reinforcement in the crack
ε_{sr1} is the steel strain at the point of zero slip under cracking forces reaching $f_{ctm}(t)$
ε_{sr2} is the strain of reinforcement at the crack under cracking forces reaching $f_{ctm}(t)$; if the internal forces are lower than or equal to the cracking forces (e.g. in a working joint), then $\varepsilon_{sr2} = \varepsilon_{s2}$
$\Delta\varepsilon_{sr}$ is the increase of steel strain in the cracking state.

The bond force F_b transmitted along the transmission length can be described by

$$F_b = \phi\pi\tau_m l_t = A_s E_s \Delta\varepsilon_{sr} \qquad (3.2\text{–}2)$$

where

ϕ is the bar diameter
τ_m is the mean value of the bond strength (see also subsection 7.4.3).

After crack formation
After the crack formation has finished, the mean spacing between cracks $s_{r,m}$ can be taken as

$$s_{r,m} = \tfrac{2}{3}2l_t = \tfrac{4}{3}l_t \qquad (3.2\text{–}3)$$

Then the transferred bond force is reduced according to the reduced transmission length $l_{t,m} = \frac{2}{3}l_t$

$$F_{b,m} = \tfrac{2}{3}F_b = \tfrac{2}{3}A_s E_s \Delta\varepsilon_{sr} \qquad (3.2\text{–}4)$$

Accordingly the reduction of the steel strain from the crack to the point in the middle between the cracks is given by

$$\Delta\varepsilon_{s,m} = \tfrac{2}{3}\Delta\varepsilon_{sr} \qquad (3.2\text{–}5)$$

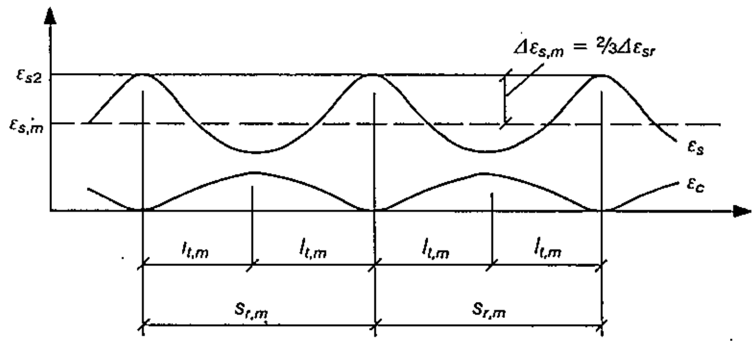


Fig. 3.2.3. Stabilized crack pattern with $s_{r,m} = \frac{4}{3} l_t$

The mean strain over the total member may be taken as

$$\varepsilon_{s,m} = \varepsilon_{s2} - \beta_{t,m}\Delta\varepsilon_{s,m} = \varepsilon_{s2} - \beta_{t,m}\tfrac{2}{3}\Delta\varepsilon_{sr} = \varepsilon_{s2} - \beta_t\Delta\varepsilon_{sr} \quad (3.2\text{–}6)$$

where

$\beta_t = \frac{2}{3}\beta_{t,m} = 0.40$ for instantaneous loading, and 0.25 for long-term and repeated loading.

For calculation of transmission length l_t, see clause 7.4.3.1.

3.2.3. Stress–strain relation of steel embedded in concrete

In a cracked zone the strain of the reinforcement varies along the bar (Fig. 3.2.2 and 3.2.3). The overall deformability of the reinforcement may be described by the mean value of the steel strain. When calculating the tension stiffening effect in this way, distinction should be made between the crack formation phase and the stabilized crack pattern.

For normal cases the steel stress at the last crack may be taken as

$$\sigma_{srn} = 1.3\sigma_{sr} \qquad (3.2\text{–}7)$$

with $\sigma_{sr} = \sigma_{sr1}$ the steel stress in the first crack.

In this case formula (3.2–9) reads

$$\varepsilon_{sm} = \varepsilon_{s2} - \frac{\beta_t(\sigma_s - \sigma_{sr}) + (1.3\sigma_{sr} - \sigma_s)}{0.3\sigma_{sr}}(\varepsilon_{sr2} - \varepsilon_{sr1}) \qquad (3.2\text{–}10)$$

For a member under pure tension σ_{sr} can be calculated as

$$\sigma_{sr} = N_r / A_s$$

For short-term loading

$$N_r = A_c(1 + \alpha\rho)f_{ct}$$

where

α is the modular ratio, and ρ is the geometrical ratio of reinforcement.

For long-term loading creep and shrinkage should be taken into consideration.

For different cases of application (minimum reinforcement, deflection, stability) different fractile values of the tensile strength f_{ct} should be applied.

It is proposed to take

- for deflections, the mean or the lower fractile value of f_{ct}
- for minimum reinforcement, an upper fractile
- for stability verifications, the mean value
- for crack width calculation, also the mean value.

It is assumed that in practice only deformed bars are used.

For practical application the tension stiffening effect may be taken into account by a modified stress–strain relation of the embedded reinforcement ($\sigma_s - \varepsilon_{s,m}$ relation) as follows:

(a) uncracked

$$\varepsilon_{s,m} = \varepsilon_{s1} \qquad 0 < \sigma_s \leqslant \sigma_{sr1} \qquad (3.2\text{–}8)$$

(b) crack formation phase

$$\varepsilon_{s,m} = \varepsilon_{s2} - \frac{\beta_t(\sigma_s - \sigma_{sr1}) + (\sigma_{srn} - \sigma_s)}{\sigma_{srn} - \sigma_{sr1}}(\varepsilon_{sr2} - \varepsilon_{sr1})$$

$$\sigma_{sr1} < \sigma_s \leqslant \sigma_{srn} \qquad (3.2\text{–}9)$$

(c) stabilized cracking

$$\varepsilon_{s,m} = \varepsilon_{s2} - \beta_t(\varepsilon_{sr2} - \varepsilon_{sr1}) \qquad \sigma_{srn} < \sigma_s \leqslant f_{yk} \qquad (3.2\text{–}11)$$

(d) post-yielding

$$\varepsilon_{s,m} = \varepsilon_{sy} - \beta_t(\varepsilon_{sr2} - \varepsilon_{sr1}) + \delta\left(1 - \frac{\sigma_{sr1}}{f_{yk}}\right)(\varepsilon_{s2} - \varepsilon_{sy})$$

$$f_{yk} < \sigma_s < f_{tk} \qquad (3.2\text{–}12)$$

where

ε_{sy} is the strain at yield strength
σ_s is the steel stress in the crack
σ_{sr1} is the steel stress in the crack, when first crack has formed
σ_{srn} is the steel stress in the crack, when stabilized crack pattern has formed (last crack)
β_t = 0.40 for short-term loading (pure tension)
β_t = 0.25 for long-term or repeated loading (pure tension)
δ = 0.8; coefficient to take into account the ratio f_{tk}/f_{yk} and the yield stress f_{yk}.

The value $\delta = 0.8$ is valid for ductile steel (type A) and $f_{yk} = 500$ MPa.

When calculating the effects of imposed deformations, the inclined line according to eq. (3.2–9) may be replaced by the dotted line shown in Fig. 3.2.4.

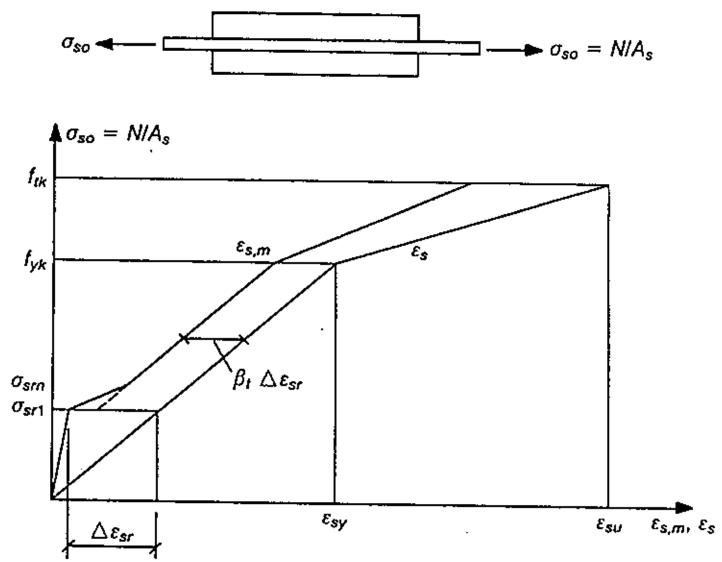


Fig. 3.2.4. Simplified stress–strain relationship of embedded reinforcing steel

3.3. LOCAL COMPRESSION

The compressive bearing capacity f_{cc}^* of locally loaded concrete is governed by the failure mechanisms described in the following. Corresponding approximate models and limiting f_{cc}^*-values presented in this chapter may be used when other more precise models are not available.

The lowest of the f_{cc}^*-values corresponding to the failure modes of subsections 3.3.1, 3.3.2 and 3.3.3, will be the f^*-value valid in each case

Dispersion of concentrated forces causes biaxial or triaxial compression immediately under the load, whereas it produces transverse tension further away.

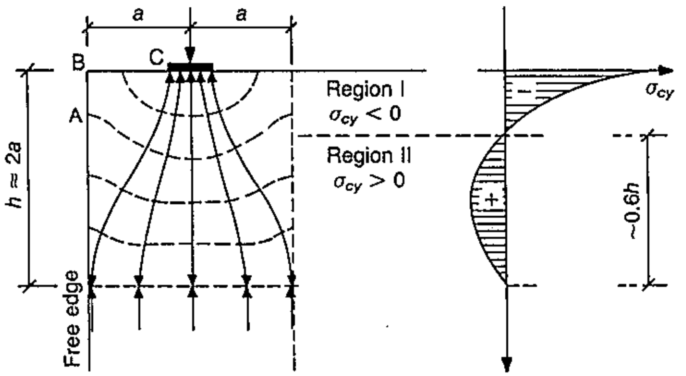


Fig. 3.3.1. Stress field under a concentrated load

Failure may be observed in the upper region (I) by transverse tension (splitting) or by compression (crushing). Similarly, a tension failure may occur in the lower region (II) by transverse tension (bursting).

Local spalling of corners (e.g. ABC in Fig. 3.3.1) is not considered here.

Reliability aspects, not included in this modelling, should appropriately be handled, taking into account the increased model uncertainties.

3.3.1. Spalling near the end face of a partially loaded surface

Near the end face of a partially loaded surface, lateral dilatancy of the locally compressed concrete is hindered by the surrounding mass of non-loaded concrete.

This surrounding concrete is therefore subjected to an expansion, possibly leading to transverse cracking. In order to avoid such a cracking, the local compression f_{cc}^* should be limited according to eq. (3.3–1):

$$f_{cc}^* = f_{cc}\sqrt{(A_2/A_1)} \not> 4f_{cc} \qquad (3.3\text{–}1)$$

where

f_{cc}^* is the bearing capacity of concrete under local compression
f_{cc} is the compressive strength of concrete under uniaxial stress (reductions of this strength in the sense of clause 6.2.2.2 are also applicable)
A_1 is the loaded area
A_2 is the cross-section of the surrounding concrete into which the stress field is developed (leading to a final uniform longitudinal stress distribution).

For helically reinforced concrete, or in presence of closed stirrups, the relevant provisions of confined concrete in section 3.5 have to be followed (leading to an additional increase of strength), under the condition that the transverse reinforcement is arranged in the first third of the stress field length close to the load.

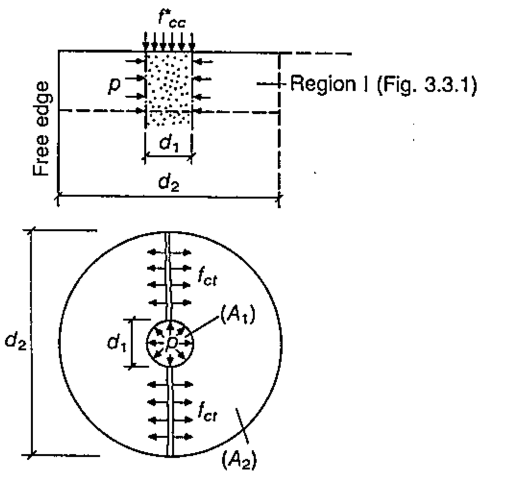


Fig. 3.3.2. Transverse expansion of concrete near a partially compressed end-face, including transverse cracking (spalling)

The following equations apply to Fig. 3.3.2:

equilibrium before spalling

$$pd_1 = f_{ct}(d_2 - d_1) \quad d_2 > d_1$$

$$p = \frac{f_{cc}}{10}\frac{d_2 - d_1}{d_1}$$

triaxial effect

$$f_{cc}^* = f_{cc} + 5p = f_{cc} + 0.5f_{cc}(d_2 - d_1)/d_1$$

with

$$d_2 \approx 2 \text{ to } 4d_1$$

$$f_{cc}^* \approx 0.7f_{cc}\sqrt{(A_2/A_1)}$$

Because of favourable size effects, however, the basic concrete strength may be taken as $1.3f_{cc}$. Thus

$$f_{cc}^* \approx f_{cc}\sqrt{(A_2/A_1)}$$

The neglect of an eventual frictional component at the edge of region I is compensated by an overestimation of the tensile resistance.

The explanation presented above is approximate. The overestimation of f_{ct}, acting over the full length of the crack, is compensated by the neglect of the shear stress on the lower surface of region I.

A refinement could be obtained by adding a compatibility equation.

As an 'effective' surrounding area A_2, the minimum area inscribed in the actual total one can be taken, geometrically similar to the loaded area A_1 and having the same centre.

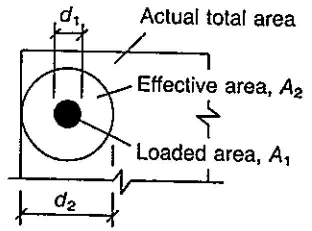


Fig. 3.3.3. Effective surrounding area A_2

For a rectangular loaded area bh (with $h < 2b$) an equivalent circular area could be used in the above modelling, with $d_{1,eq} = 1.125\sqrt{A_1}$.

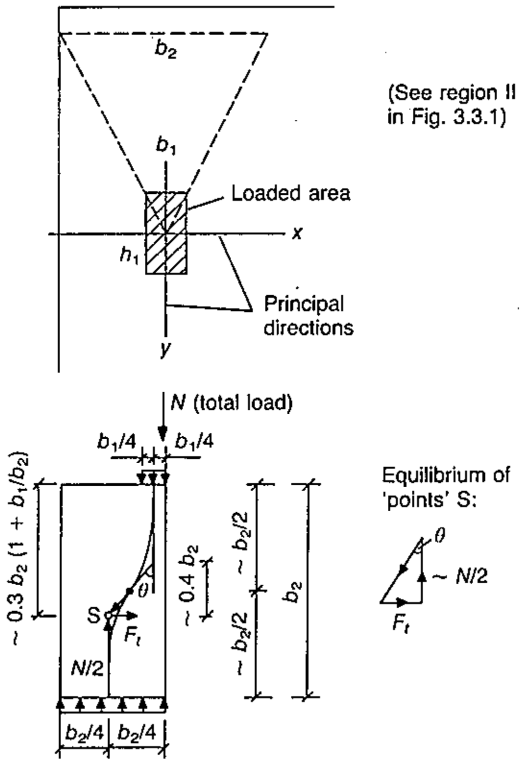


Fig. 3.3.4. Simplified model for the estimation of the bursting forces across each direction

In the simplified model shown in Fig. 3.3.4, the force trajectory is approximately considered as consisting of two circular arcs. Equilibrium of 'points' S gives

$$F_t = (N/2)\tan\theta = \frac{(N/2)(b_2/4 - b_1/4)}{0.4b_2}$$

$$F_t = 0.3N(1 - b_1/b_2)$$

3.3.2. Splitting in deeper zones

Transverse tensile bursting forces in the more remote region of the stress field should be estimated for both principal directions of the loaded area and should be resisted by the tensile strength of the concrete itself or by specially provided reinforcement (see also subsections 6.9.12 and 9.1.1) if the concrete is expected to crack longitudinally.

Substituting the stress field by two force-trajectories, within a length equal to the width of the effective surrounding concrete area, the bursting forces may be estimated as

$$F_{t,x} = 0.3N(1 - b_1/b_2) \qquad (3.3\text{–}2)$$

$$F_{t,y} = 0.3N(1 - h_1/h_2) \qquad (3.3\text{–}3)$$

where

$$N = b_1 h_1 f_{cc}^*$$

These forces are resisted as follows

$$F_{t,x} < f_{ct} 0.6 b_2 h_1 \quad \text{or} \quad A_{sx} f_{yk} \qquad (3.3\text{–}4)$$

$$F_{t,y} < f_{ct} 0.6 h_2 b_1 \quad \text{or} \quad A_{sy} f_{yk} \qquad (3.3\text{–}5)$$

where

$0.6b_2$ and $0.6h_2$ are approximate values of the respective heights of the tension region of the stress field

A_{sx} and A_{sy} are the corresponding cross-sections of well anchored reinforcement transversely arranged within the tension region.

This is a conservative approach considering the spreading of the force N separately in each direction x, y. In fact, for loaded areas approaching the square form, a biaxial spreading of the force N may mobilize substantially lower transverse tensile stresses, leading to considerably higher bearing capacities f_{cc}^*.

Care about longitudinal crack control, however, as well as uncertainties about unforeseen eccentricities, justify the use of the 'plane stress approach' in current design cases.

A slightly less conservative value may be found if the internal lever arm of bursting forces is taken equal to $0.5b_1$. In such a case, the numerical factor 0.30 in eq. (3.3–2) is reduced to 0.25 (see eq. (6.9–1)).

Equating of eqs (3.3–2) to (3.3–5) leads to an expression of the bearing capacity of locally loaded concrete, corresponding to the failure mode under consideration, see eq. (3.3–6)

$$0.3 f_{cc}^* b_1 h_1 \left(1 - \frac{b_1}{b_2}\right) = f_{ct} 0.6 b_2 h_1 \text{ or } A_{sx} f_{yk} \qquad (3.3\text{–}6)$$

Defining the mechanical volumetric percentage of bursting reinforcement in the direction x as

$$\omega_x = \frac{A_{sx}}{h_1 0.6 b_2} \frac{f_{yk}}{f_{cc}} \qquad (3.3\text{–}7)$$

it follows that

$$0.3 f_{cc}^* b_1 \left(1 - \frac{b_1}{b_2}\right) = 0.6 b_2 \left(\omega_x \text{ or } \frac{f_{ct}}{f_{cc}}\right) \text{ or}$$

$$\frac{f_{cc}^*}{f_{cc}} = 2 \frac{(b_2/b_1)^2}{(b_2/b_1) - 1} \left(\omega_x \text{ or } \frac{f_{ct}}{f_{cc}}\right) \qquad (3.3\text{–}8)$$

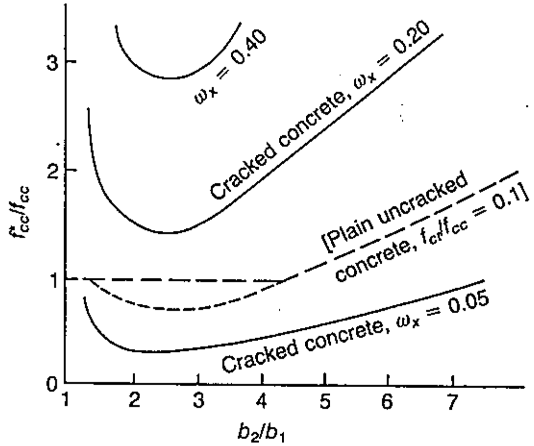


Fig. 3.3.5. Bursting resistance in relation to ω_x *and* b_2/b_1

The triaxial compressive stresses under the loaded area, in case of a relatively small loading area or significant confinement can be that large local pulverization of the concrete occurs. This pulverization occurs until no further volume reduction is possible. The pulverized material causes a quasi-hydrostatic pressure on the confining concrete, which may lead to local wedging off of a part of the surface area.

3.3.3. Surface crushing

When a relatively small area of a very large surface is compressed, or when significant confining capacity is available, a type of local bearing failure may occur, comparable to Prandtl's wedge.

If not more precisely calculated, the average bearing capacity can be calculated with the expression

$$f^*_{cc}/f_{cc} = 12.5\sqrt{(40/f_{cc})} \qquad (3.3\text{–}9)$$

(f_{cc} in MPa).

However, if limited penetration is to be considered, f^*_{cc}-values should not be taken higher than $4f_{cc}$.

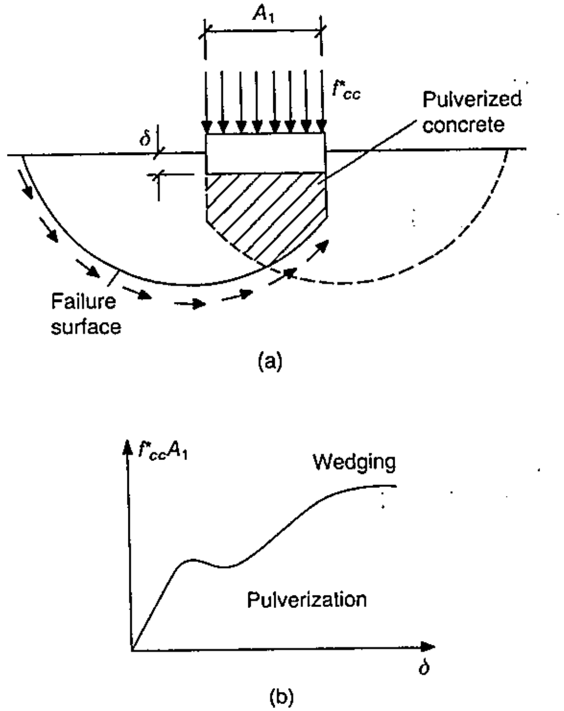


Fig. 3.3.6. Surface crushing (Prandtl's wedge)

In the compression–tension quadrant the biaxial failure envelope of plain concrete gives a compressive stress at failure, σ_{c2}, smaller than the uniaxial compressive strength f_{cm}. For the combination $\sigma_{c1} > 0$, $\sigma_{c2} < 0$, $\sigma_{c3} = 0$ the multiaxial failure criterion in clause 2.1.3.4 gives the following failure envelope (compare to eqs 2.1–8 through 2.1–11)

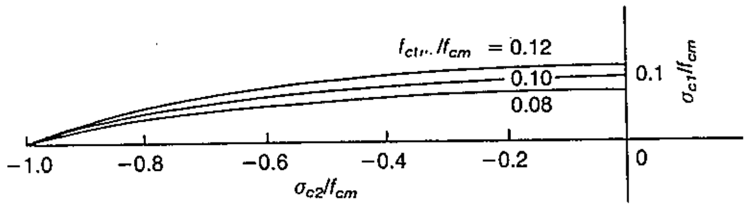


Fig. 3.4.1. Multiaxial failure envelope

3.4. BIAXIAL COMPRESSION AND TENSION

In reinforced concrete subjected to biaxial compression and tension, the concrete strength in the direction of the compressive stress, σ_{c2}, is reduced after cracking. This strength reduction is mainly because of the tensile stress, σ_{c1}, developed in the concrete between the cracks, due to tensile forces transferred by bond from the steel bars. Moreover, the concrete strips between the cracks are slender, and therefore less resistant to compression.

There is experimental evidence that the reduction in compressive strength increases as the crack spacing decreases. Therefore, any effect that decreases crack spacing (e.g. the use of smaller diameter bars) is expected to increase this strength reduction. For this reason, and also because of the higher tension forces they transfer to the concrete between the cracks, high bond deformed bars are expected to reduce the concrete strength in the direction of the compressive stresses more than plain bars.

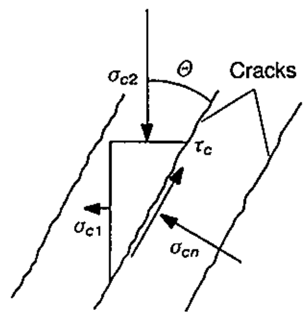


Fig. 3.4.2. Stresses acting in a crack

If the concrete cracks are at an angle θ to the direction of σ_{c2} (see Fig. 3.4.2), the normal and shear stress component on the plane of the crack are

$$\sigma_{cn} \approx |\sigma_{c2}| \sin^2 \theta \qquad (3.4\text{–}1)$$

$$\tau_c \approx \tfrac{1}{2}|\sigma_{c2}| \sin 2\theta \qquad (3.4\text{–}2)$$

The magnitude of the shear stress τ_c that can be developed in the crack is limited by the sum of

(*a*) the shear resistance of the interface due to concrete-to-concrete friction (given in clause 3.9.2.1 in terms of the normal stress component $\sigma_c = \sigma_{cn}$), and

(*b*) the maximum shear forces that can be transferred by dowel action by the reinforcing bars crossing a unit area of the crack (given in section 3.10 in terms of the axial stress σ_s in the steel bars, and other parameters, and becoming zero after yielding of bars).

If the cracks are not parallel to the direction of the compressive stresses, the latter will have to be transferred across the cracks by a combination of concrete-to-concrete friction (interface shear and compression normal to crack, section 3.9) and dowel action (section 3.10). This results in a further reduction of the concrete compressive strength, which is greatest when the cracks are at 45° to the direction of the applied compressive stresses, and smallest when they are parallel to it.

So, a limit is also implicitly imposed on the value of σ_{c2} that can be developed in the concrete. This limit value, which can be taken as a reduced compressive strength of concrete in the presence of cracks at an angle θ to the applied compression, is a maximum when the cracks are parallel to the applied compressive stress ($\theta = 0$), and a minimum when they are at an angle $\theta = 45°$ to it.

For cracking parallel to the direction of applied compression, the reduced design concrete strength due to transverse tension can be taken as

$$f_{cd}^* = f_{cd}/(1 + k\varepsilon_1/\varepsilon_{c0}) \qquad (3.4\text{–}3)$$

in which ε_1 is the average (smeared) tensile strain of cracked reinforced concrete normal to the direction of applied compression, and k a coefficient which depends on the surface roughness and the diameter of the bars. For medium diameter deformed bars, k can be taken equal to 0.1, whereas for small diameter smooth welded wire mesh, k is approximately equal to 0.2.

Confinement results in a modification of the constitutive law of the concrete; higher strength and higher critical strains are achieved.

Nevertheless, most of the other basic mechanical characteristics are practically unaffected, at least as far as design is concerned.

Therefore, due to lack of theoretical and experimental data, moduli of elasticity (E, G), Poisson's ratio and coefficient of thermal expansion of confined concrete should be considered equal to those for unconfined concrete.

Long-term behaviour characteristics of confined concrete (shrinkage and creep) should be taken as those of unconfined concrete.

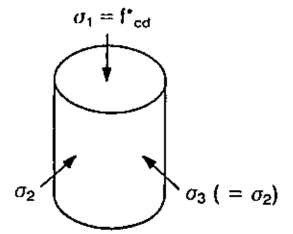


Fig. 3.5.1. Axially compressed concrete with lateral confinement

The reduction in concrete compressive strength due to simultaneously acting transverse tension depends on the magnitude of the average (smeared) tensile strain in the transverse direction, on the inclination of the cracks with respect to the direction of the compressive stress, on the surface roughness and diameter of the steel bars, etc.

3.5. DATA FOR CONFINED CONCRETE

3.5.1. General

General mechanical characteristics of concrete confined by means of adequate closed stirrups or cross-ties are taken as those of unconfined concrete, except where given below.

3.5.2. ULS under axial load-effects

3.5.2.1. Practical modelling

When a more precise study is not made, the following practical model may be used

(*a*) When axially compressed concrete reaches its plastic condition, confining-steel (closed stirrups or hoops) develops stresses close to its yield limit. Thus, the average confining stress laterally acting on concrete may be approximated by

$$\frac{\sigma_2}{f_{cd}} \approx \frac{\sigma_3}{f_{cd}} = \tfrac{1}{2}\omega_w \qquad (3.5\text{–}1)$$

where ω_w defines the volumetric mechanical ratio of confining steel.

(*b*) Taking into account the non-uniformity of distribution of these confining stresses, the 'effective lateral stress' may be approximated by the expression

$$\frac{\sigma_2}{f_{cd}} \approx \frac{\sigma_3}{f_{cd}} = \tfrac{1}{2}\alpha_n \alpha_s \omega_w \qquad (3.5\text{–}2)$$

where

α_n is a reduction factor expressing the effective concrete area in plan (depending on the hoop-pattern in the cross-section)

α_s is a reduction factor expressing the effective concrete area in elevation (depending on the spacing of hoops).

Examples

In Fig. 3.5.2(a):

$$\sigma = \frac{2A_s f_{yd}}{bs}$$

where s = spacing between hoops

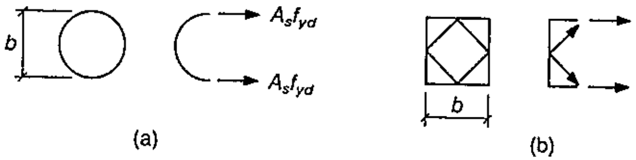


Fig. 3.5.2. Examples of confining reinforcement

$$\omega_w = \frac{4\pi b A_s}{\pi b^2}\frac{f_{yd}}{f_{cd}}$$

$$\frac{\sigma}{f_{cd}} = 0.5\omega_w$$

In Fig. 3.5.2(b):

$$\sigma = \frac{\left(2 + 2\frac{\sqrt{2}}{2}\right) A_s f_{yd}}{bs} = \frac{3.415 A_s f_{yd}}{bs}$$

$$\omega_w = \frac{\left(4 + 4\frac{\sqrt{2}}{2}\right) bA_s}{b^2 s}\frac{f_{yd}}{f_{cd}} = \frac{6.83 bA_s}{bs}\frac{f_{yd}}{f_{cd}}$$

$$\frac{\sigma}{f_{cd}} = 0.5\omega_w$$

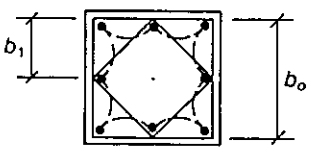


Fig. 3.5.3. Horizontal cross-section of column with non-uniform distribution of confining stresses

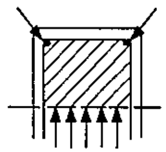


Fig. 3.5.4. Cross-section of beam with confining action of hoops

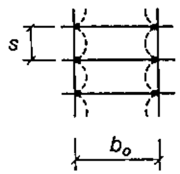


Fig. 3.5.5. Vertical cross-section of column with non-uniform distribution of confining stresses (in elevation)

Referring to Fig. 3.5.3, the coefficient α_n can be calculated as follows

$$\alpha_n \approx 1 - \frac{n(b_i^2/6)}{b_0^2} = 1 - \frac{8}{3}\frac{1}{n} \qquad (b_i < 200\,\text{mm})$$

where n is the total number of tied longitudinal bars. In the case of the compressive zone of beams, the neutral axis may be considered as a 'solid' border, hindering lateral expansion (Fig. 3.5.4).

The coefficient α_s follows from Fig. 3.5.5:

$$\alpha_s = \left(1 + \frac{1}{2}\frac{s}{b_0}\right) \qquad s < b_0/2$$

When a more precise analysis is not carried out, the values of the reduction factors α_n and α_s, may be taken from Fig. 3.5.6

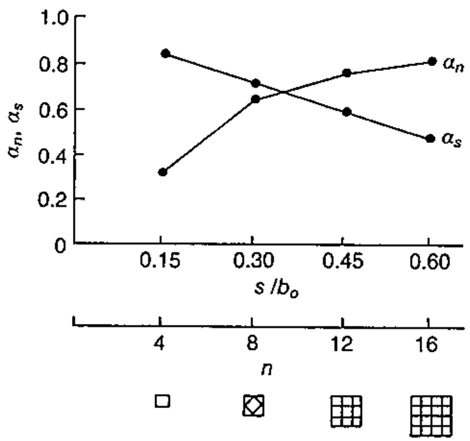


Fig. 3.5.6. Approximate values of α_n and α_s

Note. For circular columns and circular hoops

$$\alpha_n = 1 \qquad \alpha_s = \left(1 - \frac{1}{2}\frac{s}{b_0}\right)^2$$

For circular columns and spiral reinforcement

$$\alpha_n = 1 \qquad \alpha_s = \left(1 - \frac{1}{2}\frac{s}{b_0}\right)$$

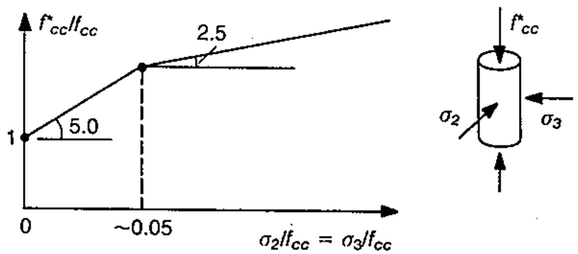


Fig. 3.5.7. Linearized approximation of compressive strength under triaxial axisymmetric loading

(*c*) When a more precise relationship between the triaxial compressive strength f^*_{cc} and the unconfined strength f_{cc} is not used, a linearized approximation may be adopted.

$$f^*_{cc} = f_{cc}(1.000 + 2.50\alpha\omega_w) \quad \text{for } \sigma_2/f_{cc} < 0.05 \qquad (3.5\text{–}3)$$

or

$$f^*_{cc} = f_{cc}(1.125 + 1.25\alpha\omega_w) \quad \text{for } \sigma_2/f_{cc} > 0.05 \qquad (3.5\text{–}4)$$

where $\alpha = \alpha_n \alpha_s$.

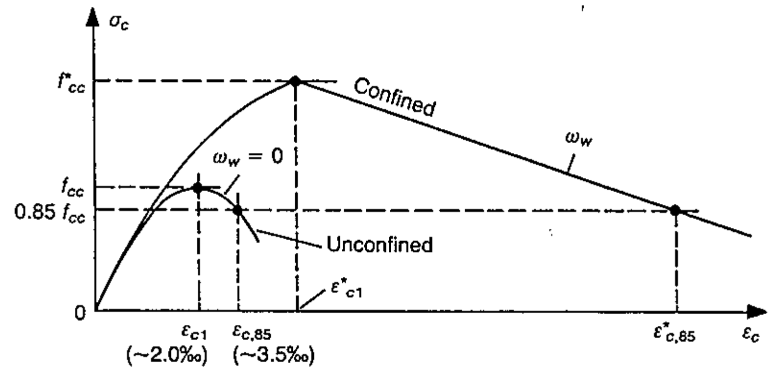


Fig. 3.5.8. Approximation for stress–strain relationship under triaxial axisymmetric conditions

(*d*) If a more precise model is not used, the following approximation may be applied in predicting the stress–strain relationship under triaxial axisymmetric conditions:

$$\varepsilon^*_{c1} \; = \; \varepsilon_{c1} + (f^*_{cc}/f_{cc})^2 \qquad (3.5\text{–}5)$$

$$\varepsilon^*_{c,85} \; = \; \varepsilon_{c,85} + 0.1\alpha\omega_w \qquad (3.5\text{–}6)$$

where $\alpha = \alpha_n \alpha_s$.

Increased characteristic strength and strains given in this section are rather conservative; therefore there is no need for a model uncertainty factor (γ_{Rd}) or for an increased partial safety factor for the material.

Thus, $\gamma_{c,cf} = \gamma_c = 1.50$.

The coefficient 0.85 on f_{cd} in Fig. 3.5.9 takes account of the unfavourable effect of long-term loads.

3.5.2.2. Idealized (σ — ε) diagram

(*a*) *Parabola-rectangle diagram*

For calculating the resistant load effects, if other more refined models are not available, a design diagram is used. See Fig. 3.5.9.

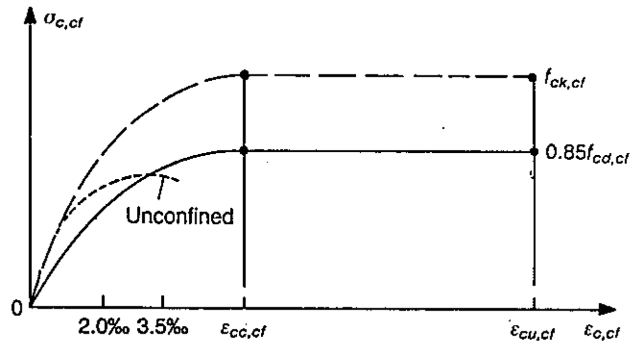


Fig. 3.5.9. Design (σ — ε) diagram for confined concrete

Appropriate triaxial models should be used. If more precise data are not available, the increased characteristic strength and strains may be estimated by the following equations

$$f_{ck,cf} = f_{ck}(1.000 + 5.0\sigma_2/f_{ck}) \quad \text{for } \sigma_2 < 0.05 f_{ck}$$
$$f_{ck,cf} = f_{ck}(1.125 + 2.50\sigma_2/f_{ck}) \quad \text{for } \sigma_2 > 0.05 f_{ck} \qquad (3.5\text{–}7)$$
$$\varepsilon_{cc,cf} = 2.0 \times 10^{-3}(f_{ck,cf}/f_{ck})^2 \qquad (3.5\text{–}8)$$
$$\varepsilon_{cu,cf} = 3.5 \times 10^{-3} + 0.2\sigma_2/f_{ck} \qquad (3.5\text{–}9)$$

where $\sigma_2(=\sigma_3)$ is the effective lateral compression stress at ULS due to confinement.

Simplified models for the evaluation of σ_2 may be used, i.e.

$$\sigma_2/f_{ck} = 0.5\alpha\omega_{wd}$$

where

α is the effectiveness of confinement $= \alpha_n \alpha_s$, see eq. (3.5–2); α_n depends on the arrangement of stirrups in the cross-section, and α_s depends on the spacing of the stirrups

ω_{wd} is the design mechanical volumetric ratio of confining reinforcement.

$$\omega_{wd} = \frac{W_{s,trans}}{W_{c,cf}} \frac{f_{yd,trans}}{f_{cd}}$$

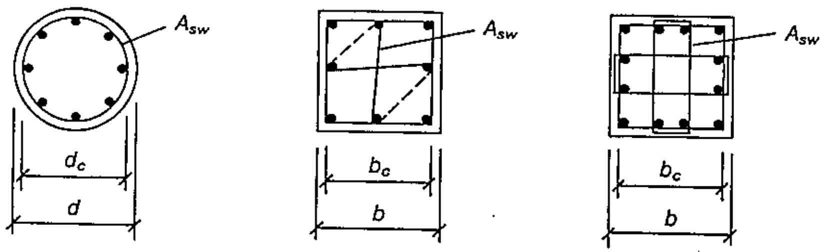
where

$W_{s,trans}$ is the volume of closed stirrups or cross-ties
$W_{c,cf}$ is the volume of confined concrete
$f_{yd,trans}$ is the design yield stress of transverse reinforcement
f_{cd} is the design strength of unconfined concrete.

(b) Rectangular diagram

If a confined concrete section is not entirely in compression, a simplified rectangular distribution of the compressive stresses can be taken as for unconfined concrete.

For the higher concrete grades, when a significant reduction of ε_{c1}- and $\varepsilon_{cu,85}$-values are applicable, correspondingly reduced values will be used in eqs (3.5–8) and (3.5–9).



$$\omega_{wd} = \frac{4A_{sw}}{d_c s} \cdot \frac{f_{yd}}{f_{cd}} \qquad \omega_{wd} = \frac{6A_{sw}}{b_c s} \cdot \frac{f_{yd}}{f_{cd}} \qquad \omega_{wd} = \frac{9A_{sw}}{b_c s} \cdot \frac{f_{yd}}{f_{cd}}$$

Fig. 3.5.10. Calculation of ω_{wd}

The mean steel strain $\varepsilon_{s,m}$ may be assessed on the basis of subsection 3.2.3 (tension stiffening effect).

In connection to this the following points should be observed

(*a*) An effective concrete area, subjected to tension, around the tensile reinforcement should be assessed. —> 7.4.3.1

(*b*) The mean concrete strain at the top of the cross-section $\varepsilon_{c,m}$ should be estimated taking into account the variation of the depth of the compression zone between adjacent cracks.

(*c*) An appropriate lever arm should be used.

The solid line in Figs 3.6.1(a) and (b) represents the case which is most representative for actual practice; it includes a general reduction factor β_b for the concrete tensile strength, such as caused by the effects of shrinkage, sustained loading etc. If the concrete is in the virgin state and the loading is of short-term character, the dotted line is nearer to reality.

3.6. MOMENT–CURVATURE RELATIONSHIP

The mean curvature at any section of an element is given by the relationship

$$\frac{1}{r} = \frac{\varepsilon_{s,m} - \varepsilon_{c,m}}{d} \quad (3.6\text{–}1)$$

It is also possible to use the diagrams in Figs 3.6.1(a) and 3.6.1(b)

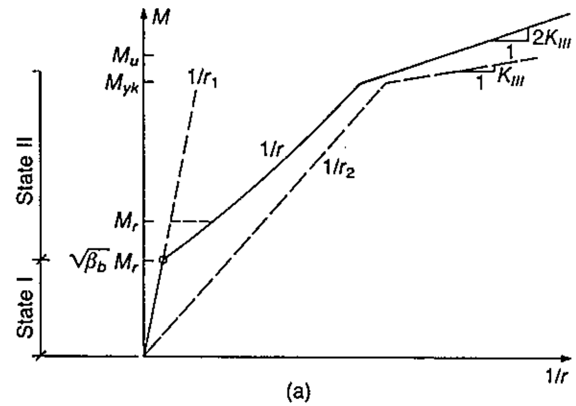


Fig. 3.6.1(a). Mean curvature—simple bending

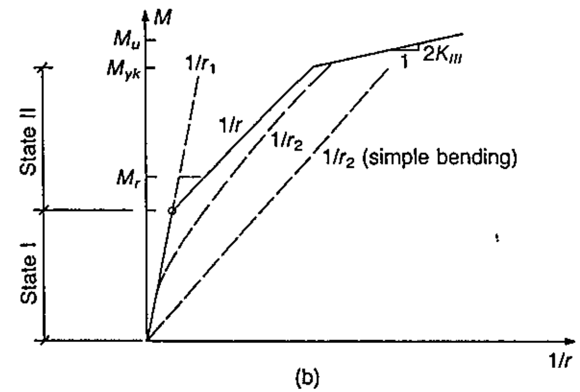


Fig. 3.6.1(b). Mean curvature—bending combined with compression (N = *const.*)

Eq. (3.6–2b) defines a hyperbolic law for the tensioning stiffening effect given by the expression

$$\frac{1}{r_{ts}} = \left(\frac{1}{r_{2r}} - \frac{1}{r_{1r}}\right)\beta_b\left(\frac{M_r}{M}\right)$$

In order to describe the mean behaviour of a structure taking into account previous actions due to shrinkage, temperature, previous loading–unloading of live loads, the mean curvature $1/r$ is assumed to be influenced by cracking already for a smaller value of M than M_r, i.e. for $\sqrt{(\beta_b)}M_r$ for simple bending. This latter value is obtained as the intersection between the descending branch of the curvature $1/r$ and the straight line $1/r_1$, eqs (3.6–2a) and (3.6–2b).

For the case of bending combined with compression, this intersection cannot be expressed by the same simple rule.

For practical applications, numerical values for $1/r_1$ and $1/r_2$ taking into account the reinforcement, creep and shrinkage can be found in CEB Bulletin 158 (Manual on Cracking and Deformations).

The mean curvature (instantaneous or long-term) in any section of an element may be determined as follows (see Figs 3.6.1(a) and (b)):

$$1/r = 1/r_1 \quad \text{for state I} \qquad (3.6\text{–}2a)$$

$$1/r = 1/r_2 - 1/r_{ts} = 1/r_2 - (1/r_{2r} - 1/r_{1r})\beta_b(M_r/M) \quad \text{for state II} \qquad (3.6\text{–}2b)$$

$$1/r = 1/r_y - (1/r_{2r} - 1/r_{1r})\beta_b(M_r/M_y) + (M - M_y)/2K_{III} \quad \text{for } M \geqslant M_y \qquad (3.6\text{–}2c)$$

where

$$K_{III} = \frac{M_u - M_y}{(1/r_u) - (1/r_y)}$$

M is the acting bending moment
M_y is the yielding moment
M_u is the ultimate moment
M_r is the cracking moment

$$M_r = W_1(f_{ct} - N/A_1) \qquad (3.6\text{–}3)$$

$1/r_y$ is the curvature corresponding to M_y, N
$1/r_u$ is the curvature corresponding to M_u, N

where

N is the applied normal force,
$f_{ct} = 0.7f_{ctm}$ if the local deformations are to be considered
$f_{ct} = f_{ctm}$ if the effects of an overall deflection are to be considered
W_1 is the section modulus in state I (including the reinforcement)
A_1 is the section area in state I (including the reinforcement)

$1/r_1$, $1/r_{1r}$ are curvatures in state I corresponding to the action M, N and M_r, N respectively
$1/r_2$, $1/r_{2r}$ are curvatures in state II-naked corresponding to the action M, N and M_r, N respectively

However, the possible interaction of shrinkage, creep and relaxation should be appropriately taken into account.

$1/r_{ts} = (1/r_{2r} - 1/r_{1r})\beta_b(M_r/M)$ is the tension stiffening

$$\beta_b = \beta_1\beta_2$$

β_1 is the coefficient characterizing the bond quality of the reinforcing bars; $\beta_1 = 1$ for high bond bars and 0.5 for smooth bars

β_2 is the coefficient representing the influence of the duration of application or of repetition of loading; $\beta_2 = 0.8$ at first loading and 0.5 for long-term loading or for a large number of load cycles.

At time t the mean curvature is the sum of the initial (instantaneous) curvature $1/r_0$ and the increment of the curvature $\Delta(1/r)$ due to the time dependent effects (creep and shrinkage of concrete, relaxation of prestressed steel):

$$1/r = (1/r_0) + \Delta(1/r) \qquad (3.6\text{–}4)$$

where each of the terms of the second member is evaluated by means of eq. (3.6–2).

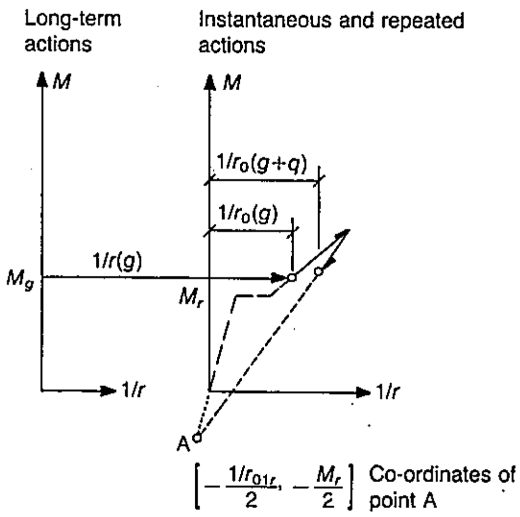


Fig. 3.6.2. Instantaneous mean curvature—simple bending: reloading/unloading

For instantaneous reloading/unloading, the graphical relation given in Fig. 3.6.2 may be taken.

Curvature at time t_2 due to permanent loads g (introduced at time $t_1 < t_2$) and instantaneous loading and unloading q (introduced at time t_2) is given by the following relation (see Figs 3.6.1, 3.6.2)

$$1/r(g + q) = 1/r(g) + 1/r_0(g + q) - 1/r_0(g) \qquad (3.6\text{–}5)$$

with

$1/r(g + q)$ is the curvature at time t_2 due to g and q
$1/r(g)$ is the curvature at time t_2 due to g
$1/r_0(g + q)$ is the instantaneous curvature at time t_2 due to g and q
$1/r_0(g)$ is the instantaneous curvature at time t_2 due to g.

3.7. ROTATION CAPACITY

The plastic rotation capacity Θ_{pl} of reinforced concrete flexural members can be derived from the distribution of the average steel strain along the member according to section 3.2. When calculating the distribution of the average steel strain, the shifting of the tensile force by the truss action (see subsection 6.3.3 and clause 9.2.2.5) may be taken into account whenever fully developed shear cracks appear.

$$\Theta_{pl} = \int_0^{l_{pl}} \frac{1}{d - x(a)} [\varepsilon_{sm}(a) - \varepsilon_{sm,y}] da \qquad (3.7\text{–}1)$$

where

l_{pl} is the length of the region, in which the tensile strain is larger than the yield strain
$x(a)$ is the depth of compression zone
$\varepsilon_{sm}(a)$ is the mean steel strain, calculated according to subsection 3.2.3
$\varepsilon_{sm,y}$ is the mean steel strain for $\sigma_s = f_{yk}$
a is the abscissa (see Fig. 3.7.1).

Assuming a bilinear stress–strain relationship for the reinforcement, the plastic rotation capacity Θ_{pl} may be estimated from eq. (3.7–2)

$$\Theta_{pl} = \int_0^{l_{pl}} \frac{\delta}{d - x} \left(1 - \frac{\sigma_{sr1}}{f_{yk}}\right) (\varepsilon_{s2} - \varepsilon_{sy}) da \qquad (3.7\text{–}2)$$

where

δ is the coefficient for taking into account the form of the stress–strain curve of the reinforcement in the inelastic range ($\delta \approx 0.8$)
x is the depth of compression zone
σ_{sr1} is the steel stress in the crack, when the 5%-fractile of the concrete tensile strength is reached
ε_{s2} is the strain of 'naked' bar in the crack
ε_{sy} is the strain at yield stress.

The possible plastic rotation capacity of flexural members may be calculated from the tensile force diagram, which may be determined from the moment distribution; a shifting of the tensile force due to a truss action may be taken into account, if the shear forces are large enough to cause fully developed shear cracks. In this connection an indicative value of the critical V may be taken twice as high as the value V_{Rd1} (see clause 6.4.2.3, eq. (6.4–8)). Using the tensile force–strain diagram of the tensile chord, one gets the strain along the axis of the member. The plastic curvature $(1/r)_{pl}$ integrated over the plastic length l_{pl} gives the plastic rotation capacity Θ_{pl} (see Fig. 3.7.1). $(1/r)_{pl}$ can be calculated from the plastic strain $\varepsilon_{m,pl}$ and the distance $(d - x)$ between the neutral axis and the reinforcing steel.

For the calculation of the steel strains entering eq. (3.7–1), the maximum concrete strain in the critical cross-section will be taken equal to the critical ε_{cu} from eq. (6.2–2) or, in case of adequate confinement, from eq. (3.5–6).

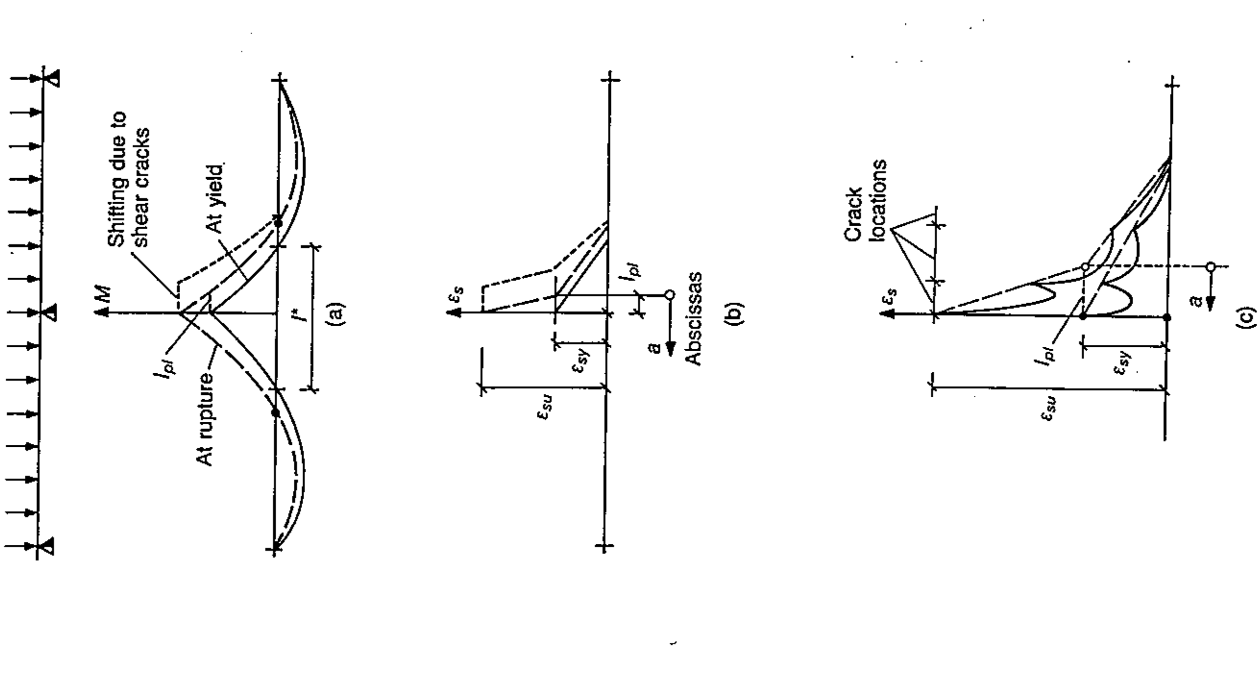


Fig. 3.7.1. Calculation of plastic rotation capacity Θ_{pl}: *(a) flexural moments,* M; *steel-bar force* F $\cong$ M/z; *(b) steel strain,* ε_s; *(c) detail of steel strain*

In general, in the region of a plastic hinge, a constant height x of the compression zone may be assumed.

In the absence of a more rigorous analysis, the plastic rotation capacity may be estimated by Fig. 3.7.2 instead of eq. (3.7–2).

In order to facilitate practical applications, the abscissas of Fig. 3.7.2 represent the design values of the normalized neutral axis depth (conventionally calculated for f_{cd}, f_{yd}, $\varepsilon_s \leqslant 0.010$, $\varepsilon_c \leqslant 0.0035$ in the critical section).

The maximum plastic rotation capacity occurs at a value x/d, when the concrete strain reaches the value ε_{cu} and the reinforcement is strained to the uniform elongation ε_u (see clause 2.2.4.1). For smaller values of x/d, failure is caused by rupture of the reinforcement; for higher values of x/d by crushing of the concrete in the compressive zone.

For high values of x/d, failure may occur before yielding of the reinforcement. Therefore, the influence of the type of reinforcement on the plastic rotation capacity decreases with increasing values of x/d.

Fig. 3.7.2 has been calculated applying the 5%-fractile of the material parameters for reinforcement and concrete; these values are sufficiently conservative.

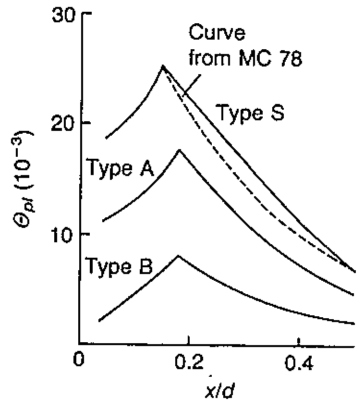


Fig. 3.7.2. Plastic rotation capacity as a functon of the relative depth of the compression zone for deformed non-prestressed bars

Fig. 3.7.2 neglects the favourable influence of transverse reinforcement and of longitudinal reinforcement in the compression zone. The favourable effect of well developed shear cracks is taken into account.

(Thus, it gives conservative values for plain bars.)

Fig. 3.7.2 is valid for a slenderness $l^*/d = 6$. For other values of l^*/d, the rotation capacity may be multiplied by the factor $\sqrt{(\frac{1}{6}l^*/d)}$ (l^* denotes the distance between two consecutive zero moment points on both sides of a support).

3.8. TORSIONAL STIFFNESS

For the calculation of the action effects, in the absence of more accurate methods, the eqs (3.8–1) to (3.8–3) should be used; they can be taken as constant for each span.

$$K_I = 0.03E_cC/(1 + 1.0\phi) \quad (3.8\text{–}1)$$

$$K_{IIm} = 0.10E_cC/(1 + 0.3\phi) \quad (3.8\text{–}2)$$

$$K_{IIt} = 0.05E_cC/(1 + 0.3\phi) \quad (3.8\text{–}3)$$

These values are valid for SLS as well as for ULS. They are not applicable in the case of hollow box section beams unless differently proved.

The torsional stiffness K for the rotation per unit length is defined as follows

$$d\theta/\mathrm{d}x = T/K$$

In the expression for K_I the factor 0.30 takes account of the non-linear behaviour of concrete before cracking. If necessary the calculation can be carried out for two extreme values.

where

K_I is the stiffness in state I, uncracked
K_{IIm} is the stiffness in state II, cracked
K_{IIt} is the stiffness in state II, torsional and shear cracks
E_c is the modulus of elasticity of concrete
C is the torsional moment of inertia in uncracked stage
ϕ is the creep coefficient to be used for long-term loading.

3.9. CONCRETE-TO-CONCRETE FRICTION

3.9.1. Definitions

The mechanism of shear transfer along a concrete-to-concrete interface which is simultaneously subject to shear and normal compression is called concrete-to-concrete friction.

The general term 'concrete-to-concrete friction' includes also the mechanism of friction along natural cracks which in literature is often referred to as 'aggregate interlock'.

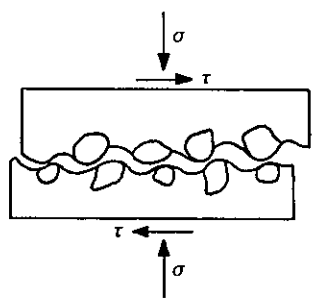


Fig. 3.9.1. Aggregate interlock

Concrete faces formed against metal or wooden moulds, as well as concrete surfaces either smoothened after concreting by means of a trowel or without any finishing, are considered as 'smooth'.

Crack interfaces, as well as concrete faces artificially roughened (e.g. washed fresh concrete faces, scrabbled or scraped interfaces) are considered as 'rough'.

The normal compression in the interfaces may be due to one or more of the following actions

- externally acting compressive force (e.g. normal axial load in a column due to vertical loads)
- prestressing force
- clamping effect of tensioned reinforcing bars crossing the interface.

Keyed interfaces are treated in sections 6.10 and 14.3.3.

3.9.2. Design shear stresses

Although the shear resistance of interfaces is due to stresses acting on the regions of contact between the two faces, the design shear stresses given in this section should be considered as average shear resistance of the total area the interface.

3.9.2.1. Smooth interfaces

The shear resistance of an interface due to concrete-to-concrete friction may be evaluated by means of the following expression

$$\tau_{fu,d} = 0.40\sigma_{cd} \qquad (3.9\text{–}1)$$

where σ_{cd} is the averaged normal compressive stress on the interface due to external actions and/or prestressing, and calculated taking account of the appropriate γ_F factors, corresponding to favourable effects of permanent and variable actions.

The shear slip needed for the mobilization of $\tau_{fu,d}$ may be calculated as follows

$$s_u = 0.15\sqrt{\sigma_{cd}} \qquad (3.9\text{–}2)$$

where s_u is in mm and σ_{cd} in MPa.

When the unfavourable effects of friction are accounted for, the following equation may be used

$$\tau_{fu,d} = 0.60\sigma_{cd}$$

where σ_{cd} is the averaged normal stress acting on the interface. In this case, σ_{cd} should take into account γ_F-values corresponding to the unfavourable effect of permanent and variable actions.

Eq. (3.9–1) denotes the friction between concrete surfaces, which are cast separately.

3.9.2.2 Rough interfaces

The shear resistance of an interface due to concrete-to-concrete friction may be evaluated by means of the equation

$$\tau_{fu,d} = 0.40 f_{cd}^{2/3}(\sigma_{cd} + \rho f_{yd})^{1/3} \qquad (3.9\text{–}3)$$

where

f_{cd} is the design value of the compressive strength of concrete

f_{yd} is the design yield stress of the reinforcement which perpendicularly intersects the interface

ρ is the reinforcement ratio.

The validity of this equation has been checked for concrete strengths up to 65 MPa, in which the major part of the particles did not fracture during the formation of the rough interface. However, $\tau_{fu,d}$-values cannot be higher than the one which in combination with high normal compressive stresses may lead to global damage of the concrete mass. Since equation (3.9–3) was derived on the basis of tests in relatively small areas of interfaces, it overestimates the shear resistance due to friction of large interfaces. Therefore, in case of large interfaces the $\tau_{fu,d}$ values calculated according to eq. (3.9–3) should be appropriately decreased.

When the unfavourable effects of friction are accounted for, the maximum mobilized friction stress may be calculated according to the following formula:

$$\tau_{fu,d} = 0.65 f_{cd}^{2/3}(\sigma_{cd} + \rho f_{yd})^{1/3}$$

In this case, σ_{cd} is determined taking into account γ_F-values corresponding to unfavourable effects of permanent and variable actions.

The shear stress values given in eq. (3.9–3) correspond to a shear slip value approximately equal to 2.0 mm.

Where less than s_u shear slip occurs along the interface, the mobilized shear stress corresponding to the actual shear slip value may be calculated as follows (see Fig. 3.9.2):

for $s < 0.10\,\text{mm}$

$$\tau_{fd} = 5\tau_{fu,d}\,s \qquad (3.9\text{–}4)$$

for $s \geqslant 0.10\,\text{mm}$

$$\left[\frac{\tau_{fd}}{\tau_{fu,d}}\right]^4 - 0.5\left[\frac{\tau_{fd}}{\tau_{fu,d}}\right]^3 = 0.3s - 0.03 \qquad (3.9\text{–}5)$$

with s in mm.

The shear slip along a rough interface is accompanied by a crack opening (dilatancy), which may be calculated as follows

$$w = 0.6s^{2/3} \qquad (3.9\text{–}6)$$

with w and s in mm.

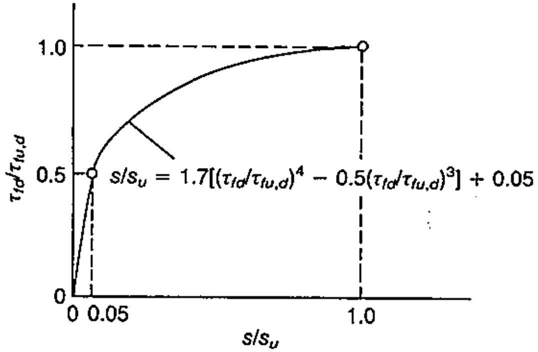


Fig. 3.9.2. Mobilized shear stress τ_{fd} as a function of shear slip s (s_u = 2.0 mm)

3.10. DOWEL ACTION

The design value of the maximum shear force which may be transferred by a reinforcing bar crossing a concrete interface (dowel action) may be calculated by means of eq. (3.10–1), provided that the geometrical conditions, listed below, are satisfied

This section applies only to dowels which are placed before casting of the concrete.

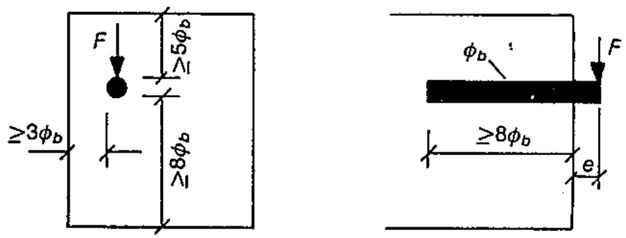


Fig. 3.10.1 Geometrical conditions

For smaller concrete covers than according to Fig. 3.10.1, the dowel strength may be neglected, since splitting of the concrete occurs at very small shear displacement values. However, in specific cases, data from available literature may be used for the evaluation of F_{ud}-values corresponding to smaller concrete covers under the condition that adequately small shear displacements are secured and consequences of brittle behaviour are appropriately faced.

$$F_{ud} = \frac{1.30}{\gamma_{Rd}} \phi_b^2 \{\sqrt{[1 + (1.3\varepsilon)^2]} - 1.3\varepsilon\} \sqrt{[f_{cd} f_{yd} (1 - \zeta^2)]} < A_s f_{yd}/\sqrt{3} \qquad (3.10\text{–}1)$$

with

$$\varepsilon = 3 \frac{e}{\phi_b} \sqrt{\frac{f_{cd}}{f_{yd}}}$$

where

ϕ_b denotes the diameter of the dowel
A_s denotes the cross-sectional area of the dowel
f_{cd} is the design value of the compressive strength of concrete
f_{yd} is the design value of the steel yield stress
e is the load eccentricity (see Fig. 3.10.1)
γ_{Rd} is the supplementary partial coefficient, may be taken equal to 1.3
$\zeta = \sigma_s/f_{yd}$ (where σ_s is the simultaneous axial stress on the bar).

The shear displacement along a concrete-to-concrete interface, which is needed for the mobilization of F_{ud} may be taken equal to $0.10\phi_b$.

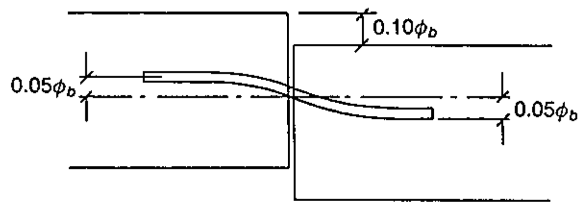


Fig. 3.10.2. Shear displacement needed for mobilization of F_{ud}

4. DATA FOR PRESTRESSING

4.1. TYPES OF PRESTRESSING

The prestress considered in this Model Code is exerted by tendons made of high-strength steel (wires, strands or bars).

Tendons may be used

(*a*) internal to the concrete, and

- (i) pretensioned, or
- (ii) post-tensioned; in this case they may be bonded by grouting, or provisionally or permanently unbonded

(*b*) external to the concrete; they may then be

- (i) totally within the external outline of the structure, or
- (ii) partially or totally outside (except in anchorage points) this outline.

The prestress may be

— non-detachable and non-adjustable (which is always the case for pretensioning and internal bonded tendons),
— non-detachable but adjustable,
— detachable and adjustable.

Anchorages may be active or passive or coupling.

An element of prestressed concrete is considered as fully prestressed if it is designed under restricted tensile stresses under service conditions. Otherwise it is considered as partially prestressed.

4.2. STRESSES AT TENSIONING, TIME OF TENSIONING

The maximum tensile force in the tendons at tensioning should generally not exceed the lower of the following values after transfer or prestressing to the concrete

$$\sigma_{po,max} = 0.75 f_{ptk} \qquad (4.2\text{–}1)$$

$$\sigma_{po,max} = 0.85 f_{p0.1k} \qquad (4.2\text{–}2)$$

For prestressed bars additional information is needed (e.g. refer to approval documents).

The minimum concrete strength required at the time when tensioning takes place is given in the approval documents for the prestressing system concerned.

For prestressing before placing the concrete (pretensioning) as well as in post-tensioning before transfer of prestressing to the concrete (in the jack) the following limiting values are recommended

$$\sigma_{po,max} = 0.80 f_{ptk} \qquad (4.2\text{–}3)$$

$$\sigma_{po,max} = 0.90 f_{p0.1k} \qquad (4.2\text{–}4)$$

Where couplers are used, relevant test data and technical approval documents should be consulted.

For prestressing after hardening of the concrete (post-tensioning), unforeseen deviation of frictional behaviour on the site can be important, as it may be impossible to obtain the needed prestressing force under the limitations of this clause. In such cases it is possible, if the available steels and prestressing technique allow it, to apply a higher stress at the end of the tendons. This stress should never exceed the value of $0.95 f_{p0.1k}$.

To comply with these limitations, special steps may have to be taken in the design of post-tensioned structures.

One solution consists of providing in the design the possibility to insert additional tendons (e.g. in supplementary ducts). Another solution consists of a limitation of the force (within the tendons at tensioning) as a function of the expected immediate losses due to friction according to Table 4.2.1.

Where particular rules are not given, the time when prestressing takes place should be fixed with due regard to

- deformation conditions of the component
- safety with respect to the compressive strength of the concrete
- safety with respect to local stresses
- early application of a part of the prestress, to reduce shrinkage effects.

Table 4.2.1. Limitation of force at tensioning

For tensioning at one end $\beta = \mu(\alpha + kx)$	$\beta \leqslant 0.28$	$0.28 < \beta \leqslant 0.40$	$0.40 < \beta \leqslant 0.60$
For tensioning at both ends $\beta = \mu(\alpha + kx)$	$\beta \leqslant 0.55$	$0.55 < \beta \leqslant 0.80$	$0.80 < \beta \leqslant 1.20$
$\sigma_{po.max}$	$0.9 f_{p0.1k}$	$0.8 f_{p0.1k}$	$0.75 f_{p0.1k}$

For $\mu(\alpha + kx) > 0.60$ (in the case of tensioning at one end) or 1.20 (in the case of tensioning at both ends) the possibility to insert additional tendons should always be provided for in the design.

For definitions of μ, α, k and x see clause 4.3.3.2.

Regarding the practical aspects of tensioning and grouting refer to sections 11.7 and 11.8.

4.3. INITIAL PRESTRESS

4.3.1. General

The initial prestress (at time $t = 0$) is calculated taking into account the prestressing force and the permanent actions present at tensioning.

The value of the initial prestressing force (at time $t = 0$) at a given section of abscissa x, is obtained by subtracting from the force at tensioning the different immediate losses described in this section.

4.3.2. Losses occurring before prestressing (pretensioning)

The following losses should be considered in design

(*a*) loss due to friction at the bends (in the case of curved wires or strands)
(*b*) losses due to drive-in of the anchoring devices (at the abutments) when anchoring on a prestressing bed

(*c*) loss due to relaxation of the pretensioned tendons during the period which elapses between the tensioning of the tendons and prestressing of the concrete.

4.3.3. Immediate losses generally present

4.3.3.1. Losses due to the instantaneous deformation of concrete

Account should be taken of the loss in tendon force corresponding to the deformation of concrete

- *in the case of post-tensioned tendons*, taking into account the order in which the tendons are stressed
- *in the case of pretensioned tendons*, as a result of their action when they are released from the anchorages.

4.3.3.2. Losses due to friction (post-tensioned tendons)

In a cross-section which is at a distance x from an operative anchorage device, the stress $\sigma_{po}(x)$ in the tendon being tensioned is lower than the stress at the anchorage $\sigma_{po,\max}$. The difference between these two stresses corresponds to losses due to friction:

$$\sigma_{po}(x) = \sigma_{po,\max} e^{-\mu(\alpha + kx)} \qquad (4.3\text{–}1)$$

where

μ denotes the coefficient of friction between the tendons and their sheathing

α denotes the sum of the angular displacements over a distance x (irrespective of direction or sign)

k denotes an unintentional angular displacement (per unit length) depending on the design layout (shape) of the tendon.

Values for μ and k are deduced from previous experience with the same type of materials and construction. When values of these coefficients are given in approval documents, they have to be taken as design values.

Before the above values are adopted, however, the diameters of the sheathing, the distance between supports and the quality of workmanship should be duly taken into consideration.

With external prestressing, the friction is concentrated at deviation devices.

All values given below should be considered as indicative mean values.

(*a*) *Internal post-tensioning (with grouting)*

The coefficient of friction μ is the product of the physical coefficient of friction μ_o and the squeezing factor χ, where χ is dependent on the degree of filling of the duct. Where more exact investigations are not available, this factor can be assumed to be 1.3 to 1.35 for tendons filling the duct between 50% and 60%. The physical coefficient of friction μ_o is influenced, interalia by the surfaces of prestressing steel and ducts (micro- and macro-structures), rust, pressure, elongation of the tendon, etc.

If more accurate values are not available and in the case of steel and duct being both without rust, the values given below can be assumed, for μ_o and for μ with a 50% filling. These values which are indicative mean values can be multiplied by 0.9 if slight lubrication is present, e.g. by means of soluble oil.

Cold drawn wire	$\mu_0 = 0.13$	$\mu = 0.17$
Strand	$\mu_0 = 0.15$	$\mu = 0.19$
Deformed bar	$\mu_0 = 0.50$	$\mu = 0.65$
Smooth and round bar	$\mu_0 = 0.25$	$\mu = 0.33$

Under site conditions variations of 50% are possible. In the case of rust, even higher variations are likely to occur.

The coefficient k takes account of unintentional angular deviations. Its value depends on the quality of workmanship and on the distance between supports of the tendon. Values for k are given in approval documents with $k = 0.005\text{–}0.01\ (m^{-1})$.

For the verification of the real values of prestressing losses at tensioning (see clause 11.7.3.2(d)) it is recommended to measure the transmission of prestressing force from one end of the tendon to the other.

(b) Friction losses in the case of unbonded internal tendons
Tests and practical experience have shown that the friction factors μ and k as listed below can be applied.

- For monostrands (individual strands in plastic; single or grouped) in slabs or reservoirs $\mu = 0.05\text{–}0.07$ $k = 0.006\text{–}0.01\ m^{-1}$
- For greased multistrand or multiwire tendons (e.g. in nuclear containments) $\mu = 0.13\text{–}0.15$ $k = 0.004\text{–}0.008\ m^{-1}$
- For dry multistrand or multiwire tendons (e.g. in nuclear containments with dry air as subsequent corrosion protection) factors as for ordinary post-tensioned tendons.

(c) In the case of external multistrand tendons

- For naked dry strands over steel saddle $\mu = 0.25\text{–}0.30$ $k = 0$
- For greased strands over steel saddle $\mu = 0.20\text{–}0.25$ $k = 0$
- For dry strands inside plastic pipe over saddle $\mu = 0.12\text{–}0.15$ $k = 0$
- For bundle of monostrands (individual sheets in plastic) over saddle $\mu = 0.05\text{–}0.07$ $k = 0$

These values correspond to a saddle radius of 2.5 to 4.0 m. For lower radii further test evidence is needed.