DRAFT FOR DEVELOPMENT

DD ENV 1992-1-3 : 1996

Eurocode 2: Design of concrete structures

Part 1.3 General rules — Precast concrete elements and structures

(together with United Kingdom National Application Document)

ICS 91.040; 91.080.40

Committees responsible for this Draft for Development

The preparation of this Draft for Development was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/2, Structural use of concrete, upon which the following bodies were represented:

Association of Consulting Engineers
British Cement Association
British Precast Concrete Federation Ltd.
Department of the Environment (Property and Buildings Directorate)
Department of Transport (Highways Agency)
Federation of Civil Engineering Contractors
Institution of Civil Engineers
Institution of Structural Engineers
Steel Reinforcement Commission

This Draft for Development, having been prepared under the direction of the Sector Board for Building and Civil Engineering, was published under the authority of the Standards Board and comes into effect on 15 September 1996

The following BSI reference relates to the work on this Draft for Development:
Committee reference B/525/2

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Contents

National foreword

This Draft for Development was prepared by Subcommittee B/525/2 and is the English language version of ENV 1992-1-3 : 1994 *Eurocode 2: Design of concrete structures Part 1.3: General rules — Precast concrete elements and structures*, as published by the European Committee for Standardization (CEN). This Draft for Development also includes the United Kingdom (UK) National Application Document (NAD) to be used with the ENV in the design of buldings to be constructed in the UK.

ENV 1992-1-3 results from a programme of work sponsored by the European Commission to make available a common set of rules for the structural and geotechnical design of building and civil engineering works.

This publication should not be regarded as a British Standard.

An ENV is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard. The publication of this ENV and its National Application Document should be considered to supersede any reference to a British Standard in previous DD ENV Eurocodes concerning the subject covered by these documents.

The values for certain parameters in the ENV Eurocodes may be set by individual CEN Members so as to meet the requirements of national regulations. These parameters are designated by |_| in the ENV.

During the ENV period of validity, reference should be made to the supporting documents listed in the National Application Document (NAD).

The purpose of the NAD is to provide essential information, particularly in relation to safety, to enable the ENV to be used for buildings constructed in the UK and the NAD takes precedence over corresponding provisions in the ENV.

The Building Regulations 1991, Approved Document A 1992, draws attention to the potential use of ENV Eurocodes as an alternative approach to Building Regulation compliance. ENV 1992-1-3 is considered to offer such an alternative approach, when used in conjunction with its NAD.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted to an EN.

Comments should be sent in writing to the Secretary of Subcommittee B/525/2, BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, a proposed revision, by 31 October 1996.

National Application Document

for use in the UK with ENV 1992-1-3 : 1994

Contents of National Application Document

National Application Document

National Application Document

Introduction

This National Application Document (NAD) has been prepared by Subcommittee B/525/2. It has been developed from the following.

a) A textual examination of ENV 1992-1-3;

b) A parametric calibration against BS 8110, supporting standards and test data;

c) Trial calculations.

1 Scope

This NAD provides information to enable ENV 1992-1-3 (hereafter referred to as EC2 : Part 1.3) to be used for the design of buildings to be constructed in the UK. It will be assumed that it will be used in conjunction with DD ENV 1992-1-1, the NAD of which refers to BSI publications for values of actions.

2 Partial factors, combination factors and other values

a) The values for combination coefficients (ψ) should be those given in table 1 of the NAD for EC2 : Part 1.1.

b) The values for partial factors for normal temperature design should be those given in EC2 : Part 1.1 except where modified by the NAD for that code.

c) Other values should be those given in EC2 : Part 1.1, except where modified by the NAD for that code, and EC2 : Part 1.3 except for those given in table 1 of this NAD.

Table 1. Values to be used in referenced clauses instead of boxed values

Reference in EC2 : Part 1.3	Definition	UK values
4.2.3.5.4 P(102) Eq (4.105)	Values of $\sigma_{o,max}$ related to f_{pk} and $f_{p0.1k}$	0.8 not 0.85, 0.9 not 0.95
4.2.3.5.4 P(103) Eq (4.106)	Values of A_p $\sigma_{pm,o}$ related to f_{pk} A_p and $f_{p0.1k}$ A_p	0.75 not 0.80, 0.85 not 0.90
4.4.1.1 (102)	Limiting stress for pretensioned elements	0.6 not 0.7
4.5.2 (103)	Limiting average bearing stress in dry joints	0.75 not 0.4
4.5.5.2 (101)	Value σ_{Rd} for dry concrete to concrete bearings	0.75 not 0.4. Also alter corresponding values in headings to tables 4.117 and 4.118.
5.2.3.2 (106)	Minimum value of joint dimension	Last sentence not applicable
5.4.10 (105)	Limiting value of l for pocket foundations with smooth joint surfaces	1.5 not 1.2, but not less than 200 mm

3 Reference standards

Supporting standards including materials specifications and standards for construction are listed in table 2 of this NAD.

Table 2. Reference in EC2 Part 1.3 to other codes and standards

Reference in EC2 Part 1.3	Document referred to	Document title or subject area	Status	UK document
Various	ENV 1992-1-1	Design of concrete strucures. General rules and rules for buildings	Published 1991	DD ENV 1992-1-1 : 1992
2.3.3.2 (104)	ENV 1991-1	Basis of design	Published 1994	DD ENV 1991-1 : 1996
6.2.1 (104)	Relevant CEN Product Standards	Tolerances	In preparation	BS 8110 : Part 1 : 1985
6.3.5 P(101)	Relevant CEN Product and other Standards	Construction and workmanship	In preparation	BS 8110 : Part 1 : 1985 and BS 5328 : 1990/91
A.105.2 (102)	ENV 1991-1 Basis of Design	Values of coefficients of variation	Published 1994	DD ENV 1991-1 : 1996

EUROPEAN PRESTANDARD
PRÉNORME EUROPÉENNE
EUROPÄISCHE VORNORM

ENV 1992-1-3 : 1994

October 1994

ICS 91.040.00; 91.080.40

Descriptors: Buildings, concrete structure, computation, building codes, rules of calculation

English version

Eurocode 2: Design of concrete structures — Part 1-3: General rules — Precast concrete elements and structures

Eurocode 2: Calcul des structures en béton — Partie 1-3: Règles générales — Eléments et structures en béton préfabriqués

Eurocode 2: Plannung von Stahlbeton- und Spannbetontragwerken — Teil 1-3: Allgemeine Regeln — Bauteile und Tragwerke aus Fertigteilen

This European Prestandard (ENV) was approved by CEN on 1993-06-25 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

CEN members are required to announce the existence of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Ref. No. ENV 1992-1-3 : 1994 E

Contents

Foreword

OBJECTIVES OF THE EUROCODES

(1) The "Structural Eurocodes" comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.

(2) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship needed to comply with the assumptions of the design rules.

(3) Until the necessary set of harmonized technical specifications for products and for the methods of testing their performance are available, some of the Structural Eurocodes cover some of these aspects in informative Appendices.

BACKGROUND OF THE EUROCODE PROGRAMME

(4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the "Structural Eurocodes".

(5) In 1990, after consulting their respective Member States, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.

(6) CEN Technical Committee CEN/TC250 is responsible for all Structural Eurocodes.

EUROCODE PROGRAMME

(7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991 Eurocode 1 Basis of design and actions on structures
EN 1992 Eurocode 2 Design of concrete structures
EN 1993 Eurocode 3 Design of steel structures
EN 1994 Eurocode 4
Design of composite steel and concrete structures
EN 1995 Eurocode 5 Design of timber structures
EN 1996 Eurocode 6 Design of masonry structures
EN 1997 Eurocode 7 Geotechnical design
EN 1998 Eurocode 8
Design provisions for earthquake resistance of structures
EN 1999 Eurocode 9 Design of aluminium alloy structures

(8) Separate sub-committees have been formed by CEN/TC250 for the various Eurocodes listed above.

(9) This Part 1-3 of Eurocode 2 is being published as a European Prestandard (ENV) with an initial life of three years.

(10) This Prestandard is intended for experimental application and for the submission of comments.

(11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.

(12) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of CEN/TC250/SC2 at the following address:

Deutsches Institute für Normung e.V. (DIN)
Burggrafenstrasse 6
D-10787 Berlin
phone:(+49) 30 - 26 01 - 25 01
fax:(+49) 30 - 26 01 - 12 31

or to your national standards organization.

NATIONAL APPLICATION DOCUMENTS (NAD's)

(13) In view of the responsibilities of authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by [ ] ("boxed values"). The authorities in each member country are expected to assign definitive values to these safety elements.

(14) Some of the supporting European or International standards may not be available by the time this Prestandard is issued. It is therefore, anticipated that a National Application Document (NAD) giving definitive values for safety elements, referencing compatible supporting standards and providing national guidance on the application of this Prestandard, will be issued by each member country or its Standards Organization.

(15) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works is located.

MATTERS SPECIFIC TO THIS PRESTANDARD

(16) The scope of Eurocode 2 is defined in 1.1.1 of ENV 1992-1-1 and the scope of this Part of Eurocode 2 is defined 1.1.2. Additional Parts of Eurocode 2 which are planned are indicated in 1.1.3 of ENV 1992-1-1; these will cover additional technologies or applications, and will complement and supplement this Part.

(17) In using this Prestandard in practice, particular regard should be paid to the underlying assumptions and conditions given in 1.3 of ENV 1992-1-1.

(18) The seven chapters of this Prestandard are complemented by

four Appendices which have the same normative status as the chapters to which they relate. These Appendices have been introduced by moving some of the more detailed Principles/Application Rules, which are needed in particular cases, out of the main part of the text to aid its clarity.

(19) As indicated in paragraph (14) of this Foreword, reference should be made to National Application Documents which will give details of compatible supporting standards to be used. For this Part of Eurocode 2, particular attention is drawn to the approved Prestandard ENV 206 (Concrete - performance, production, placing and compliance criteria), and the durability requirements given in 4.1 of this Prestandard.

(20) The provisions of this Prestandard are based substantially on the 1978 edition of the CEB Model Code and other more recent CEB and FIP documents.

(21) In developing this Prestandard, background documents have been prepared, which give commentaries on and justifications for some of the provisions in this Prestandard.

For ENV 1992-1-3, the following additional sub-clause apply:

(22) This Part 1-3 of Eurocode 2 complements ENV 1992-1-1 for the particular aspects of precast concrete elements and structures.

(23) The framework and structure of this Part 1-3 correspond to ENV 1992-1-1. However, Part 1-3 contains Principles and Application Rules which are specific to precast concrete elements and structures.

(24) This Part 1-3 of Eurocode 2 includes one Informative Appendix, Appendix 105.

(25) Where a particular sub-clause of ENV 1992-1-1 is not mentioned in this ENV 1992-1-3, that sub-clause of ENV 1992-1-1 applies as far as deemed appropriate in each case.

Some Principles and Application Rules of ENV 1992-1-1 are modified or replaced in this Part, in which case they are superseded.

Where a Principle or Application Rule in ENV 1992-1-1 is modified or replaced, the new number is identified by the addition of 100 to the original number. Where a new Principle or Application Rule is added, it is identified by a number which follows the last number of ENV 1992-1-1 with 100 added to it.

A subject not covered by ENV 1992-1-1 is introduced in this Part by a new sub-clause. The sub-clause number for this follows the most appropriate clause number in ENV 1992-1-1.

(26) The numbering of equations, figures, footnotes and tables in this Part follow the same principles as the clause numbering in (25) above.

(27) In this Part 1-3 of Eurocode 2, reference is also made to relevant CEN Standards for precast products.

1 INTRODUCTION

This clause of ENV 1992-1-1 applies except as follows:

1.1 SCOPE

1.1.2 SCOPE OF PART 1-3 OF EUROCODE 2

Addition after Principle P(5):

P(106) Part 1-3 gives a general basis for the design and detailing of concrete structures in buildings made partly or entirely of precast elements.

P(107) Precast structures are characterised by the presence of joints which provide connections between elements.

(108) This Part 1-3 provides Principles and Application Rules which complement those given in ENV 1992-1-1. Matters related to the production and assembly of structures are covered by other CEN standards.

(109) In precast concrete structures special consideration should be given to:

- detailing of bearings
- detailing of joints/connections
- structural safety and stability during multi stage construction
- pre-tensioning.

1.4 DEFINITIONS

1.4.2 SPECIAL TERMS USED IN PART 1-3 OF EUROCODE 2

Addition after Principle P(2):

P(103) The following terms are used in this Part 1-3 with the following definitions:

- A precast element is one that is manufactured in a factory or place other than the final position in the structure, protected from adverse weather conditions.

- A composite element comprises insitu and precast concrete with or without reinforcement connectors.

- Rib and block floors consist of precast ribs (or beams) with an infill between them made of blocks, hollow clay pots or other forms of permanent shuttering, with or without an insitu topping.

- A sandwich panel normally consists of two layers of concrete with a thermal insulation sandwiched between them.

- Diaphragms are plane members which are subjected to in-plane forces. A diaphragm may consist of several precast units connected together.

- Ties are tensile members, effectively continuous, in floors, walls or columns.

- Isolated precast members are those for which, in the case of failure, no secondary means of load transfer, e.g. due to redistribution of internal forces and moments, is available.

(104) The following types of reinforced or prestressed precast elements are commonly used:

- Linear elements (e.g. beams, joists, columns)
- Slab elements (e.g. solid slabs fully or partially precast, ribbed slabs, ribs and blocks, hollow core units)
- Wall elements (e.g. solid, ribbed or sandwich)
- Other elements (e.g. foundations, stairs)

(105) Transient situations: In precast concrete construction transient situations include the following:

- Demoulding
- Transport to the storage yard;
- Support and load conditions during storage;
- Transport to site;
- Erection (hoisting) and construction.

2 BASIS OF DESIGN

This clause of ENV 1992-1-1 applies except as follows:

2.1 FUNDAMENTAL REQUIREMENTS

Addition after Principle P(4):

P(105) The layout of the structure and the interaction between the structural members shall be such as to ensure a robust and stable behaviour.

(106) The necessary interaction between elements is obtained by tying the structure together using a) peripheral ties, b) internal ties, c) horizontal ties to columns and walls and d) where required, vertical ties.

Where a building is divided by joints into structurally independent parts, each part should have an appropriate tying system.

(107) Ease of assembly and maintenance should be considered in design.

(108) Where required appropriate mechanical devices should be detailed to allow ease of inspection and replacement.

2.2 DEFINITIONS AND CLASSIFICATIONS

2.2.3 MATERIAL PROPERTIES

2.2.3.1 Characteristic values

Addition after Application Rule (4):

(105) In precast construction it is necessary to check the concrete compressive strength, f_c, at a number of stages of construction (e.g. demoulding, transfer of prestress, see Clause 3.1.2.2 of ENV 1992-1-1).

2.3 DESIGN REQUIREMENTS

2.3.1 GENERAL

Addition after Priciple P(4):

(105) Where relevant, consideration should be given in design to the effects of dynamic actions (e.g. impulse) during transient situations. In the absence of a more rigorous analysis this may be allowed for by multiplying the relevant static effects by an appropriate factor.

2.3.3 **PARTIAL SAFETY FACTORS FOR ULTIMATE LIMIT STATES**

2.3.3.1 **Partial safety factors for actions on building structures**

Replacement of Application Rule (1) by:

(101) Partial safety factors for the persistent and transient design situations are those given in Table 2.2 of ENV 1992-1-1. However for precast elements, in transient situations, a lower factor may be acceptable provided:

a) the behaviour of the completed structure in the permanent condition is not adversely affected; and

b) it is permitted by relevant documents.

2.3.3.2 **Partial safety factors for materials**

Replacement of Application Rule (4) by:

(104) Higher or lower values of γ_c and γ_s may be used for precast elements if these are justified by adequate control procedures and relevant documents.[1]

2.5 **ANALYSIS**

2.5.1 **GENERAL PROVISIONS**

2.5.1.1 **General**

Addition after Appication Rule (6):

P(107) In addition to satisfying the requirements of ENV 1992-1-1 the analysis of precast concrete structures shall take into account the behaviour of the joints between elements.

An analysis shall be made for each relevant stage of construction using the appropriate geometry and properties for that stage.

(108) The analysis of precast concrete structures should account for:

- the behaviour of the structural units at all stages of construction and their interaction with other elements (e.g. composite actions with insitu concrete, other precast units);

- the behaviour of the structural system, with particular regard to actual deformations and strength of connections;

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[1] see ENV 1991-1 "Basis of Design" and Informative Appendix 105 to this Part 1-3 of ENV 1992

- the uncertainties influencing restraints and force transmission between elements arising from deviations in geometry and in the positioning of units and bearings.

(109) Horizontal restraint caused by friction due to the weight of any supported element may only be considered for non seismic zones (using $\gamma_{G,inf}$). In addition it may only be considered where

- the friction is not solely relied upon for overall stability of the structure; and

- the bearing arrangement precludes the possibility of accumulation of irreversible sliding of the elements, such as caused by uneven behaviour under alternate actions (cyclic thermal effects on the contact edges of simply supported elements).

(110) The effects of horizontal movements should be considered in design with respect to the resistance of the structure and the integrity of the joints. Proper bearing devices should be provided where necessary.

2.5.2 IDEALISATION OF THE STRUCTURE

2.5.2.1 Structural models for overall analysis

Addition after Application Rule (6):

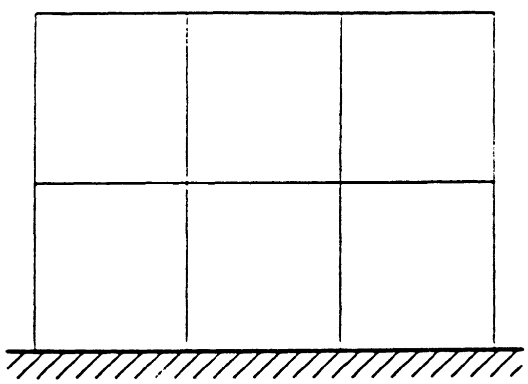
(107) In prefabrication the following structural systems are commonly used to ensure the overall stability. These and other systems may act alone or in combination:

a) Frame structures (Figure 2.106a), composed of linear precast elements (beams and columns). They may be designed either as cantilevering continuous columns (mainly for low rise buildings), or partly or wholly as a continuous framework.

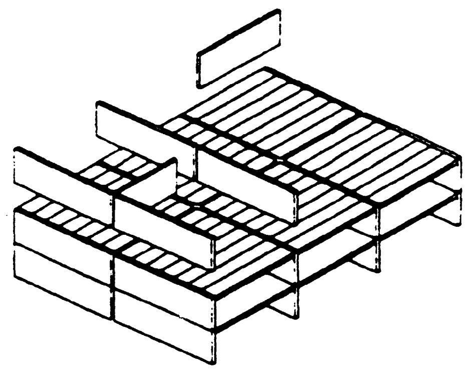
b) Cross wall structures or panel structures (Figure 2.106b) characterised by stiff in plane behaviour (shear walls) and hinged connections perpendicular to the plane direction (cross slabs). Longitudinal stability is obtained by walls or frames perpendicular to the cross walls.

c) Braced structures (Figure 2.106c) in which the beams and columns may have hinged joints. The horizontal stability is provided by bracing elements.

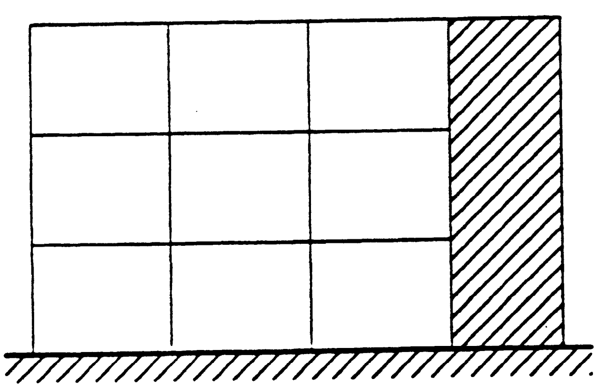
d) Floor or roof diaphragms (Figure 2.106d). Floors and roofs used to transfer horizontal forces to the bracing elements.



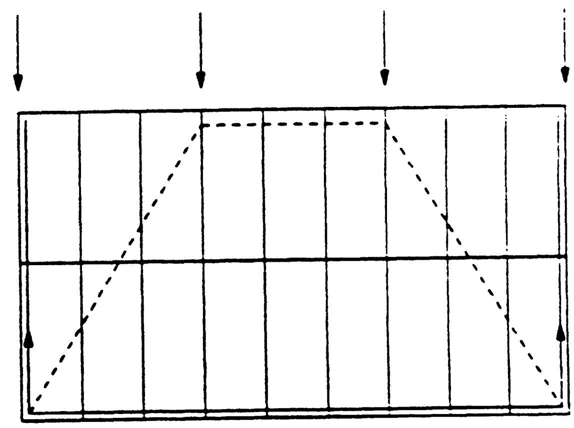
a) Frame Structure



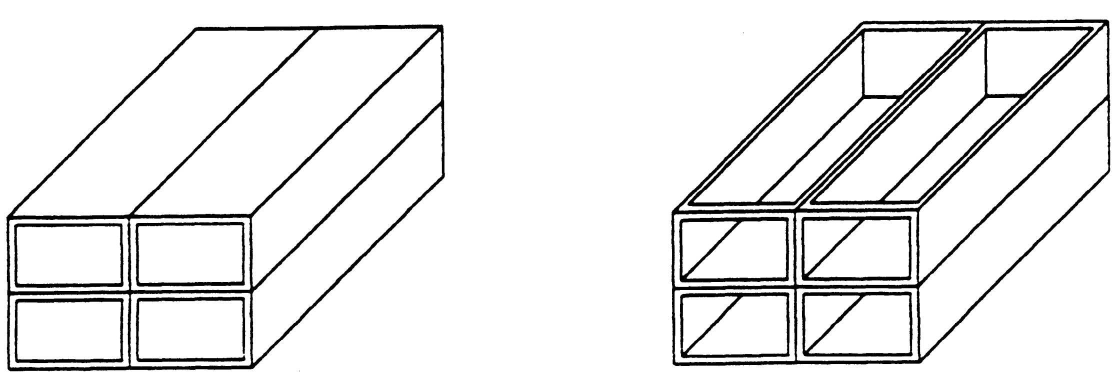
b) Cross Wall Structure (Panel Structures)



c) Braced Structure



d) Floor or Roof Diaphragm Plan



e) Cell Structure

Figure 2.106: Different Types of Structure

e) Cell structures (Figure 2.106e). Precast monolithic cell structures - e.g. room cells.

2.5.3 CALCULATION METHODS

2.5.3.1 Basic considerations

Replacement of Application Rule (5) by:

(105) The distance between movement joints for precast concrete structures may be greater than that for insitu structures since part of the creep and shrinkage takes place before erection.

Addition after Application Rule (5):

P(106) Where relative movements between an element and its support can occur the effects on the bearing and supported structure shall be considered.

(107) Continuity of joints may be assumed in design as for monolithic joints where:

- insitu concrete and conventional reinforcement detailing are used; or
- it is ensured by bolted or welded connections; or
- the connection is proved to be continuous by accurate testing, which should be carried out for unfavourable conditions of resistance and stiffness.

Connections should otherwise be considered as hinged.

2.5.3.5 Analysis of slabs

Additional clause after 2.5.3.5.7:

2.5.3.5.8 Design of precast floor systems

P(101) For the design of specific types of precast floor units, guidance given in the relevant CEN Product Standard shall also be followed.

P(102) Transverse load distribution between adjacent floor units shall be ensured by appropriate shear transfer connections.

(103) Shear transfer connections may be a) concreted or grouted joints, b) welded or bolted connections, or c) reinforced topping.

(104) The transverse distribution of point or line loads may be calculated by appropriate analysis or tests.

(105) Where precast floors are designed to act as diaphragms to transfer horizontal forces to bracing elements the following apply:

- the diaphragm should form part of a realistic structural model which takes account of the compatibility of deformations of the bracing elements;

- the effects of the resulting horizontal displacements on all parts of the structure should be considered;

- the diaphragm should be adequately reinforced to resist the tension forces in the assumed structural model;

- where stress concentrations exist in the diaphragm (e.g. around holes, connections to bracing elements) suitable detailing should be provided.

(106) Diaphragm action may be assumed where transverse reinforcement is provided. This may be concentrated in the supports provided the units are tied in in such a way that lateral force transfer is possible by arch, truss or vierendeel action. The transverse reinforcement may be placed in the topping where it exists.

07) Where simple supports have been assumed in design the effects of the actual restraints of the precast units shall be considered.

(108) Where adjacent isolated units (eg: double T-sections) are not provided with shear connectors then shear reinforcement should be provided in the ribs as for beams.

(109) Precast units provided with structural topping of at least |40| mm may be designed as composite sections provided the interface shear between topping and the precast units is verified in accordance with 4.5.3. The stresses in the precast unit should be checked for all stages of construction prior to and after composite action becomes effective.

(110) Transverse reinforcement may be entirely within the precast units or within the topping. Only that transverse reinforcement which is continuous may be used for two way spanning.

(111) Floors with ribs and blocks without topping may be treated as solid slabs for analysis provided there are transverse ribs at spacings, s_T, not exceeding those given in Table 2.104.

Table 2.104: Maximum spacing of transverse ribs, s_T, so that floors with ribs and blocks may be treated as solid slabs for analysis.

Type of building	$s_L \leq l_{eff}/\|\underline{8}\|$	$s_L > l_{eff}/\|\underline{8}\|$
dwellings	—	$\|\underline{12}\|\ d_0$
other buildings	$\|\underline{10}\|\ d_0$	$\|\underline{8}\|\ d_0$

where: s_L is the centre-to-centre spacing of longitudinal ribs
l_{eff} is the effective span of longitudinal ribs
d_0 is the thickness of ribbed floor

Additional clauses after 2.5.3.7:

2.5.3.8 Half joints

(101) Half joints may be designed using strut and tie models such as, for example, shown in Figures 2.107a and 2.107b. A combination of the two models shown may be used.

(102) All reinforcement should be suitably anchored.

2.5.4 DETERMINATION OF THE EFFECTS OF PRESTRESSING

2.5.4.2 Determination of prestressing force

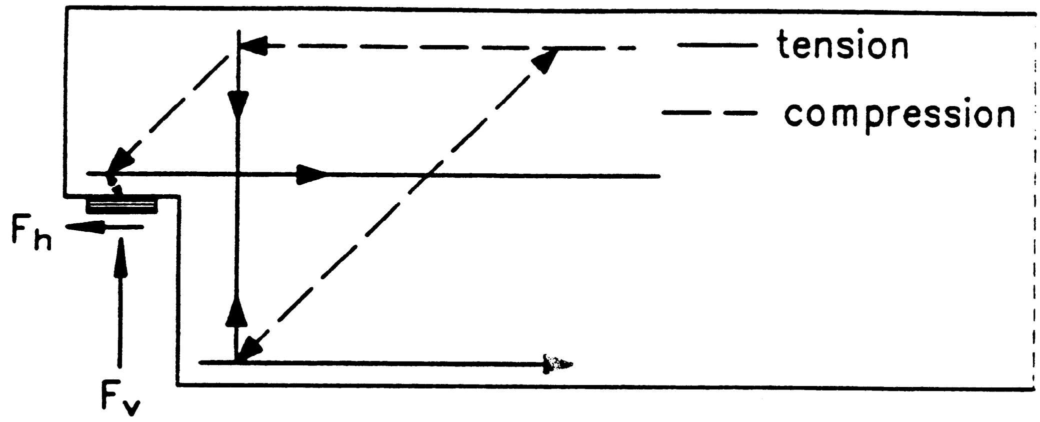
Replacement of Application Rule (4) by:

(104) The coefficients r_{sup} and r_{inf} may be taken as $|\underline{1.1}|$ and $|\underline{0.9}|$ respectively in absence of a more rigorous determination and provided that the sum of the losses due to friction and time dependent effects is ≤ 30% of the initial prestress.

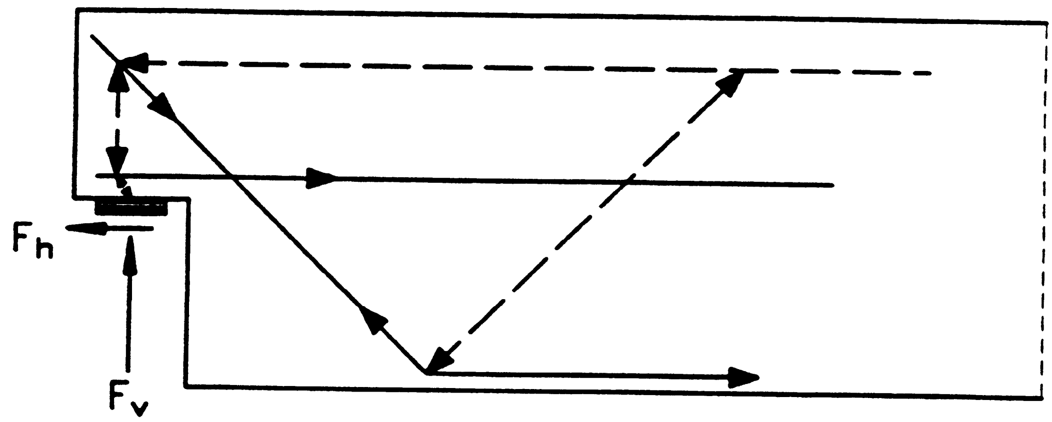
However, where there are adequate statistics on direct measurement of the prestressing force then:

$$r_{sup} = r_{inf} = |\underline{1.0}|$$

may be taken.



a: Vertical Suspension Reinforcement



b: Inclined Suspension Reinforcement

Figure 2.107: Examples of design models for half joints

Note: Figures do not show reinforcement details nor the complete equilibrium system (see also Figure 5.121)

3 MATERIAL PROPERTIES

This clause of ENV 1992-1-1 applies except as follows:

3.1 CONCRETE

3.1.2 NORMAL WEIGHT CONCRETE

3.1.2.3 Tensile strength

Addition after Application Rule (4):

(105) Equations (3.1-3.4) in ENV 1992-1-1 should be validated when applied to concrete strength classes greater than C50/60.

3.1.2.4 Strength classes of concrete

Replacement of Application Rule (3) by:

(103) In Table 3.101, the characteristic strength f_{ck} and the corresponding tensile strength are given for the different strength classes of concrete.

Addition after Application Rule (103):

(104) Intermediate strength classes between those mentioned in Table 3.101 may be used.

Table 3.101: Concrete strength classes, characteristic compressive strength f_{ck} (cylinders), mean tensile strength f_{ctm}, and characteristic tensile strength f_{ctk} of the concrete (in N/mm²). (The classification of concrete e.g. C20/25 refers to cylinder/cube strength as defined in Clause 7.3.1.1 of ENV 206).

Strength Class of Concrete	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C55/65	C60/70
f_{ck}	12	16	20	25	30	35	40	45	50	55	60
f_{ctm}	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1	4.4*	4.6*
f_{ctk} 0.05	1.1	1.3	1.5	1.8	2.0	2.2	2.5	2.7	2.9	3.1*	3.2*
f_{ctk} 0.95	2.0	2.5	2.9	3.3	3.8	4.2	4.6	4.9	5.3	5.7*	6.0*

* Values of f_{ct} for strength classes above C50/60 have been calculated in accordance with Equations (3.2)-(3.4) of ENV 1992-1-1 but should be validated.

3.1.2.5 Deformation properties

3.1.2.5.5 Creep and Shrinkage

Additional after Application Rule (5):

P(106) If the concrete compressive stress, as a result of pretensioning exceeds 0.45 f_{cmj} non-linearity of creep shall be taken into account.

(107) In such cases the notional creep coefficient should be modified according to

$$\phi_{0,k} = \phi_0 \exp(1.5(k_\sigma - 0.45)) \quad (3.106)$$

where

$\phi_{0,k}$ is the non-linear notional creep coefficient, which replaces ϕ_0

ϕ_0 creep coefficient according to 3.1.2.5.5 or Appendix 1, Clause A1.1.2, of ENV 1992-1-1

k_σ the stress-strength-ratio σ_c/f_{cmj} where σ_c is the concrete compressive stress under quasi-permanent loads and f_{cmj} is the mean value of concrete compressive strength at the time of loading. See Clause 4.2.1.3.3 (6) of ENV 1992-1-1.

(108) For creep at high temperatures see Clause A1.1.2(3) in Appendix 1 of ENV 1992-1-1.

Additional clauses after 3.4.2.1(5):

3.5 CONNECTION MATERIALS

3.5.1 GENERAL

P(101) Connection materials shall be stable and durable for the normal lifetime of the structure. Chemical and physical compatibility shall be checked. Materials shall be protected against adverse chemical and physical influences. They shall have the same fire rating as the structural elements.

3.5.2 SUPPORTING PADS

P(101) The strength and deformation characteristics of supporting pads shall be in accordance with the design criteria.

3.5.3 METAL FASTENINGS FOR CLADDING

P(101) This clause applies to permanent metal fastenings which are not fully protected against atmospheric influences.

P(102) Metals for fastenings shall be chosen from the following:

i) If not inspectable
- austenitic stainless steels;
- phosphor bronze;

ii) If inspectable
- hot-dip galvanised steels;
- copper and copper alloys;
- electrolytically galvanised or plated screw threaded parts protected with two layers of epoxy paint.

P(103) The suitability of the material shall be established before welding, annealing or cold forming is undertaken.

3.5.4 MORTAR

(101) The mean strength of mortar should not be less than |15| N/mm^2

3.6 LIFTING DEVICES

P(101) The material for lifting and handling devices shall not become significantly brittle with age or at low temperatures.

4 SECTION AND MEMBER DESIGN

This clause of ENV 1992-1-1 applies except as follows:

4.1 DURABILITY REQUIREMENTS

4.1.3 DESIGN

4.1.3.3 Concrete cover

Replacement of Note 3) under Table 4.2 by:

103) A reduction of 5 mm may be made where concrete of strength class C40/50 and above is used. In addition, for precast prestressed slab units which are produced under reliable quality control and which are in exposure class 1, another reduction of 5 mm may be made for all concrete strength classes. Where insitu concrete is placed against a precast element, the cover of the reinforcement to the interface may be reduced to:

|5 mm| in the precast member and
|10mm| in the insitu concrete.

Additional Note under Table 4.2:

(105) Where justified by adequate evidence the minimum concrete cover may be reduced for coated and corrosion resistant reinforcement. In these cases, reference should be made to relevant documents for mechanical properties including bond.

4.2 DESIGN DATA

4.2.3 PRESTRESSED CONCRETE

4.2.3.3.6 Multi-axial stresses

Replacement of Table 4.4 by:

Table 4.104 : Criteria for satisfying multi-axial conditions in tendons

Type of tendon	Minimum bending radius Ratio Nominal diameter
Single wire or strand, deflected after tensioning or tensioned while deflected	|15|
Single wire or strand, tensioned in smooth duct	|20|
Single wire or strand, tensioned in ribbed duct	|40|
Multi wire or strand tendon	Preceding values multiplied by n_1/n_2

in which: n_1 = total number of wires or strands in the tendon

n_2 = number of wires or strands transferring the radial force of all wires or strands in the tendon to the deviator. (See Figure 4.7 of ENV 1992-1-1).

4.2.3.4 Technological properties of prestressing steel

4.2.3.4.1 Relaxation

Addition after Application Rule (5):

P(106) **For pretensioned members relaxation losses caused by increased temperature in heat cured concrete shall be considered. The relaxation loss is accelerated during the application of thermal curing and the relaxation rate is reduced at the end of treatment.**

4.2.3.5 Design of members in prestressed concrete

4.2.3.5.4 Initial prestressing force

Replacement of Principles P(2) and P(3) by:

P(102) **The maximum force applied to a tendon P_o (i.e. the force at the active end, immediately after stressing, x = 0, see 2.5.4.2) shall not exceed $A_p\sigma_{o,max}$, where:**

A_p is the cross-sectional area of the tendon and $\sigma_{o,max}$ is the maximum stress applied to the tendon

$\sigma_{o,max}$ = |0.85| f_{pk} or |0.95| $f_{p0.1k}$ whichever is the lesser for pretensioned members in which damaged tendons can be replaced (4.105)

= |0.80| f_{pk} or |0.90| $f_{p0.1k}$ whichever is the lesser for all other situations (4.5)

P(103) The prestressing force applied to the concrete immediately after tensioning (post-tensioning) or after transfer (pre-tensioning), i.e. $P_{mo} = A_p\sigma_{pm,o}$, shall not exceed the lesser of the forces determined from:

$A_p \ \sigma_{pm,o}$ = |0.80| f_{pk} A_p, |0.90| $f_{p0.1k}$ A_p for pretensioned members complying with Equation (4.105) in which damaged tendons can be replaced (4.106)

$A_p \ \sigma_{pm,o}$ = |0.75| f_{pk} A_p, |0.85| $f_{p0.1k}$ A_p for all other situations (4.6)

where $\sigma_{pm,o}$ is the stress in the tendon immediately after tensioning or transfer.

4.2.3.5.6 Anchorage zones of pretensioned members

Addition after Application Rule (9):

(110) The anchorage of the tensile force, T_{dx}, due to applied loading in the ultimate limit state should be checked.

The following simplified expression may be used for the tensile force T_{dx} at a distance x from the support:

$$T_{dx} = M_{Sd}(x)/z + V_{Sd}(x) \cot\theta \qquad (4.184)$$

where

$M_{Sd}(x)$	=	applied bending moment in section x
z	=	lever arm
$V_{Sd}(x)$	=	applied shear force in section x
x	=	distance from centre of support
θ	=	angle of the concrete struts with the longitudinal axis of the member. For members without shear reinforcement, cotθ should be taken as |1.0|.

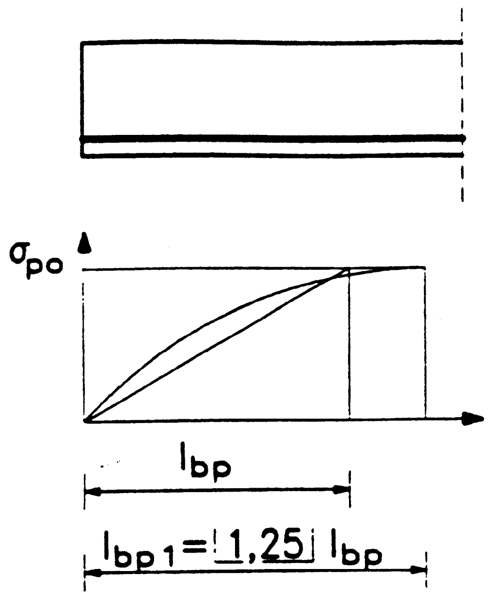
(111) The build-up of prestress may be assumed to be linear, with the transmission length l_{bp} based on Table 4.7 of ENV 1992-1-1.

Where a pretensioned member has been designed to Equation (4.106) then the values of β_b in Table 4.7 of ENV 1992-1-1, should be multiplied by $\sigma_{po}/0.8f_{pk}$; where σ_{po} is the actual stress of the prestressing steel.

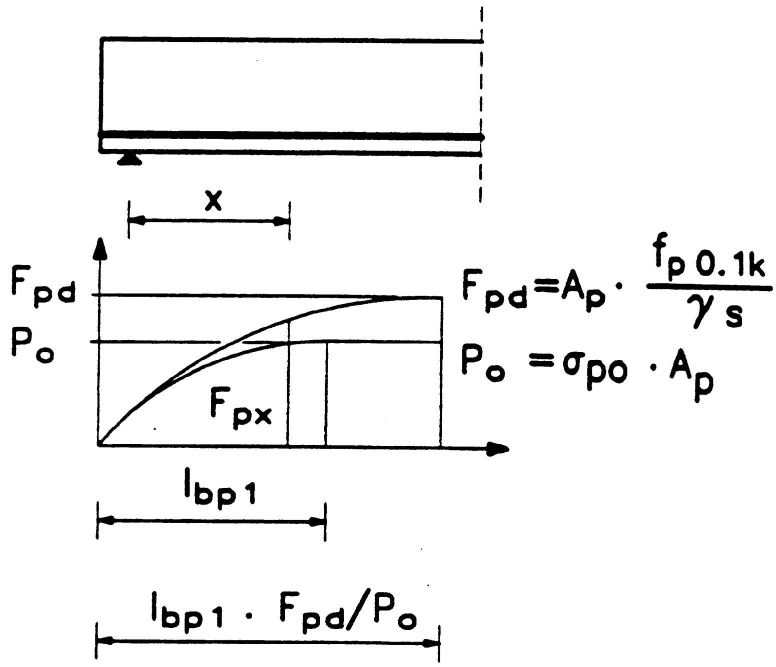
(112) For a linear build-up of prestress according to (111), the ultimate resisting force, F_{px}, at a distance x may be determined in accordance to 4.2.3.5.6 (9) of ENV 1992-1-1.

(113) For more accurate calculations, an alternative parabolic build-up of prestress may be assumed, see Figure 4.134a. The transmission length according to Equation (4.12) of ENV 1992-1-1 should then be increased by |25%|.

(114) For a parabolic build-up of prestress, the anchorage force, F_{px}, may be assumed to reach a maximum value of $F_{pd} = A_p f_{p0.1k}/\gamma_s$ as shown in Figure 4.134b.



a) Linear and Parabolic build-up of prestress



b) Limit envelope for anchorage

Figure 4.134: Build up of stress in the transmission zone.

(115) The transmission length l_{bp} of indented wires with diameter ≤ 9 mm and normal (not compacted) strands having cross section areas $A_p \leq 150\ mm^2$, all complying with surface characteristics specified in relevant standards and tensioned according to the values given in ENV 1992-1-1, 4.2.3.5.4 can be calculated according to ENV 1992-1-1, Equation (4.12) and Table 4.7.

4.3 ULTIMATE LIMIT STATES

4.3.2 SHEAR

4.3.2.3 Elements not requiring design shear reinforcement ($V_{Sd} \leq V_{Rd1}$)

Replacement of Application Rules (2) and (3) by:

(102) For simply supported pretensioned members without shear reinforcement, the shear capacity V_{Rd1} according to ENV 1992-1-1, Clause 4.3.2.3, Equation (4.18), should be checked in cracked regions. In uncracked regions of such members (i.e. where the concrete tensile stress is less than $f_{ctk\ 0.05}$), the shear capacity is limited by the principal tensile stress.

The shear capacity limited by the principal stress may be calculated according to Equation (4.185).

$$V'_{Rd1} = \frac{I.b_w}{S} \sqrt{(f^2_{ctd} + \alpha\sigma_{cpm}f_{ctd})} \qquad (4.185)$$

in which:

I = the second moment of area of the cross-section

S = the first moment of area of the cross-section

f_{ctd} = the design value of the tensile strength of the concrete $f_{ctk\ 0.05}/\gamma_c$ but not greater than 1.9 N/mm²

$\alpha = l_x/l_{bpd} \leq 1$

l_x = the distance from the end of the element to the considered section

l_{bpd} = the upper bound of the transmission length, $l_{bpd} = 1.2\ l_{bp}$

l_{bp} = the transmission length according to ENV 1992-1-1, Clause 4.2.3.5.6

σ_{cpm} = the average concrete compressive stress due to the fully developed effective prestressing force.

This calculation need not be carried out for sections closer to the support than the point of intersection of a 45° line from the face of the support and the centroidal axis of the member.

Addition after Application Rule (3):

(104) When concrete of strength class C55/65 or C60/70 is used, the coefficient ν according to Equation (4.20) of ENV 1992-1-1 should be taken as:

$$\nu = 0.7 - f_{ck}/200 \geq 0.4 \tag{4.120}$$

4.3.2.4 Elements requiring design shear reinforcement ($V_{Sd} > V_{Rd1}$)

4.3.2.4.2 Members with constant depth

Replacement of Application Rule (3) by:

(103) The concrete stress in the strut should be limited to $\sigma_c \leq \nu f_d$ where ν is the effectiveness factor given by:

$$\nu = 0.7 - f_{ck}/200 \begin{cases} \geq 0.5 & \text{for concrete strength class} \leq C50/60 \\ \geq 0.4 & \text{for concrete strength class} > C50/60 \end{cases} \tag{4.121}$$

4.3.2.4.4 Variable strut inclination method

Replacement of Application Rule (2) by:

(102) For elements with vertical shear reinforcement, the shear resistances are defined by:

$$V_{Rd2} = b_w z\ \nu\ f_{cd}/(\cot\theta + \tan\theta) \tag{4.126}$$

$$V_{Rd3} = \frac{A_{sw}}{s}\ z\ f_{ywd}\ \cot\theta; \tag{4.127}$$

however, with

$$\frac{A_{sw}\ f_{ywd}}{b_w s} \leq \frac{1}{2}\ \nu\ f_{cd}$$

The effectiveness factor ν is given by Equation (4.121). The lever arm, z, may normally be taken as 0.9 d.

4.3.3 **TORSION**

4.3.3.1 **Pure torsion**

Replacement of Equation (4.41) in Application Rule (6) by:

$$\nu = 0.7\,(0.7 - \frac{f_{ck}}{200}) \geq 0.35 \text{ for concrete strength class} \leq \text{C50/60} \qquad (4.141)$$

$$\geq 0.28 \text{ for concrete strength class} > \text{C50/60}$$

Theses values apply if there are stirrups only along the outer periphery of the member. If closed stirrups are provided in both sides of each wall of the equivalent hollow section or in each wall of a box section, ν can be assumed to be $0.7 - f_{ck}/200 \geq 0.5$, for concrete strength class ≤ C50/60 and ≥ 0.4 for concrete strength class > C50/60.

4.3.5 **ULTIMATE LIMIT STATES INDUCED BY STRUCTURAL DEFORMATION**

4.3.5.6 **Simplified design methods for isolated columns**

4.3.5.6.1 **General**

Replacement of Application Rule (1) by:

(101) For buildings, a design method may be used which assumes the compression members to be isolated and adopts a simplified shape for the deformed axis of the column. The additional eccentricity is then calculated as a function of the slenderness.

This method may also be used for walls.

4.3.5.7 **Lateral buckling of slender beams**

Addition after Application Rule (2):

P(103) Slender beams shall be checked to avoid lateral buckling during lifting, transport and erection. They shall be checked in the final position against lateral buckling from permanent loads. Slender beams and their supports must also accommodate unintentional out of balance loading at their supports.

(104) The supports should be designed to sustain at least a moment of

$$T_{Rd} = V_{Sd} \; l_{eff}/|\underline{300}| \qquad (4.187)$$

where l_{eff} is the effective span of the beam and V_{Sd} is the design vertical reaction (shear force, kN).

4.4 SERVICEABILITY LIMIT STATES

4.4.1 LIMITATION OF STRESS UNDER SERVICEABILITY CONDITIONS

4.4.1.1 Basic considerations

Replacement of Application Rule (2) by:

(102) Longitudinal cracks may occur if the stress level under the rare combination of loads exceeds a critical value. Such cracking may lead to a reduction in durability. In the absence of other measures, such as an increase in cover to reinforcement in the compressive zone or confinement by transverse reinforcement, it may be appropriate to consider limiting the compressive stress to $|\underline{0.6}|$ f_{ck} in areas exposed to environments of exposure Class 3 or 4 (see Table 4.1 of ENV 1992-1-1).

In pretensioned elements the limiting stress may be increased to $|\underline{0.7}|$ f_c provided it is justified by tests or experience. (See 2.2.3.1 (105)).

Addition after Application Rule (7):

(108) Harmful cracking which may impair the performance of the element in use should be avoided. In the absence of other measures this may be achieved by limiting the stresses in the concrete under the actions of self-weight and prestressing.

Additional clauses after 4.4.3.3.(3):

4.5 DESIGN OF CONNECTIONS

4.5.1 GENERAL

P(101) Connections shall be designed to resist all action effects implicit in the assumptions made in analysing the structure as a whole and in designing the individual members to be joined. The design shall ensure that the joint is able to accommodate the relative displacement needed to mobilise its resistance and to assure robust behaviour of the structure.

(102) The resistance and the stiffness of the joints may be based on analytical formulae, or on the results of laboratory tests. The influence of imperfections due to workmanship should be taken into account. Unfavourable deviations from testing conditions must be allowed for in deriving design values from test results.

P(103) Joints shall be designed to prevent premature splitting or spalling of concrete at the ends of the units and shall be dimensioned to take account of:

- tolerances
- assembly requirements
- ease of execution
- ease of inspection.

4.5.2 COMPRESSION JOINTS

P(101) Compression joints are those subjected to axial compression or compression with a small eccentricity.

(102) Bedded joints with mortar, concrete or hardening polymers as padding material may be used provided all necessary precautions are taken to prevent relative movement of the connected surfaces during hardening of the padding material.

(103) Dry joints should only be used where:

- the average bearing stress does not exceed |0.4| f_{cd}; and
- appropriate quality of workmanship is achieved in the production plant and on site.

(104) Compression joints can lead to significant tensile stresses in the adjacent elements. When the modulus of elasticity of the joint material is at least 70% of the modulus of the adjacent elements, (hard pack), bursting forces will occur in the adjacent elements (Figure 4.135a). When the modulus of elasticity of the pack material is significantly lower than that of the adjacent elements, (soft pack), splitting stresses will develop due to the transverse deformation of the pack material (Figure 4.135b).

(105) The transverse tensile stresses in joints with hard pack (Figure 4.136a) should be resisted by appropriate reinforcement in the adjacent elements.

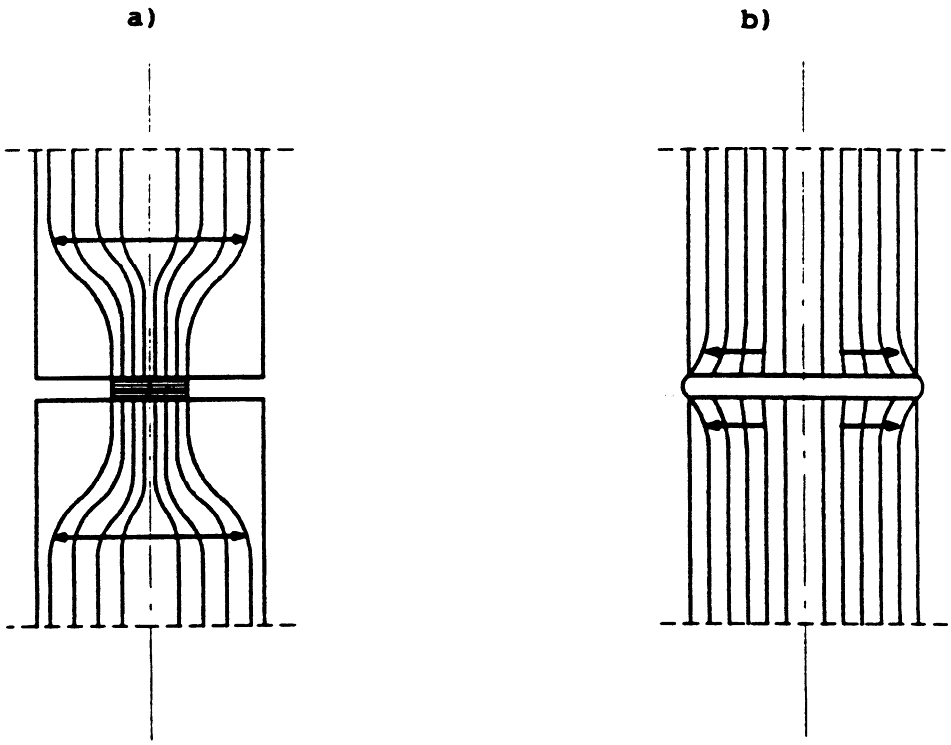


Figure 4.135: Transverse forces in joints transmitting compression

a) Bursting force from concentrated bearing

b) Splitting force from soft padding material

(106) The transverse tensile stresses in joints with soft pads (Figure 4.135b) should be resisted by appropriate reinforcement in the adjacent elements and, if required, in joints not fully contained.

(107) The calculation of the design bearing capacity of compression joints should be made according to accepted calculation models or tests.

(108) In case of combinations of axial load and shear in the joint, the latter may be disregarded if the shear force

$$V_{Sd} < \lvert \underline{0.1} \rvert \, N_{Sd} \qquad (4.188)$$

where N_{Sd} is the axial load.

4.5.3 SHEAR JOINTS

4.5.3.1 General

P(101) Shear joints transmit shear between adjacent precast units or between insitu concrete and a precast unit.

4.5.3.2 Basic Requirements

(101) A joint may be very smooth, smooth, rough or indented as defined below.

(102) Very smooth joint: A joint may be assumed to be very smooth when the surface is cast against steel or smooth wooden moulds.

(103) Smooth joint: A joint may be assumed to be smooth when it is slipformed or extruded or the concrete surface is smooth after vibration without further treatment.

(104) Rough joint: A joint may be assumed to be rough when:

- the surface is left rough after casting or by raking, resulting in a surface roughening of at least 3mm at a spacing of approximately 40mm; or

- the aggregate is exposed.

(105) Indented joint: A construction joint may be assumed to be indented where the geometry complies with Figure 4.136a.

(106) Good workmanship in concreting the joints is important. The surface should be free of cement laitance, sawdust, ice, snow etc. before insitu concrete is cast or the joint is filled.

4.5.3.3 Design shear resistance

(101) Transverse shear resistance of composite members should be checked in accordance with 4.3.2 of ENV 1992-1-1 and this Part 1-3 respectively. The effect of prestress on V_{Rd1} should be neglected if the insitu concrete constitutes the web (as in slabs with precast plates) or in regions of negative moment (as in slabs made continuous by insitu reinforcement).

(102) The longitudinal shear stress at the interface of insitu topping and precast element should be calculated with regard to the variation of the longitudinal force in the topping, which is a proportion of the total longitudinal force in the composite element. The shear stress in any section may be calculated according to Equation (4.189):

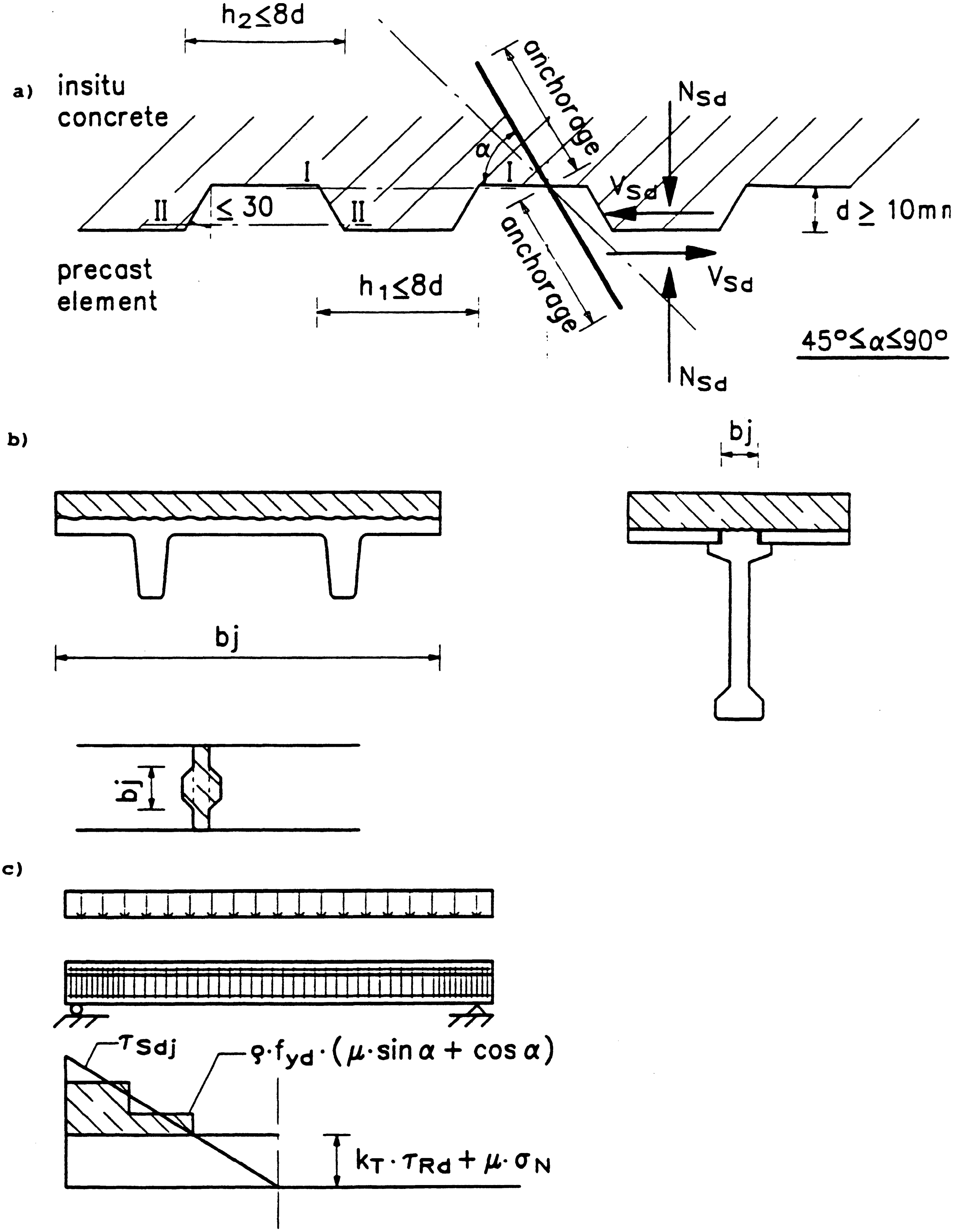


Figure 4.136: Construction joint

a) indented construction joint

b) examples for the definition of width of the joint

c) shear diagram representing the required joint reinforcement

$$\tau_{Sdj} = \beta\ V_{Sd}/(zb_j) \qquad (4.189)$$

where

β = ratio of the longitudinal force in the topping and the total longitudinal force M_{Sd}/z, both calculated for the considered section.

V_{Sd} = transverse shear force

z = lever arm

b_j = transverse length of interface (e.g. the width of a horizontal joint); see Figure 4.136b.

) The design shear resistance (for joints in composite members including joints between floor and wall elements) per unit area is:

$$\tau_{Rdj} = k_T\ \tau_{Rd} + \mu\ \sigma_N + \rho\ f_{yd}(\mu\ \sin\alpha + \cos\alpha) \leq 0.5\ \nu f_{cd} \qquad (4.190)$$

where

k_T = coefficient according to Table 4.115
$k_T = 0$ if the joint is subjected to tension

τ_{Rd} = basic design shear strength according to Table 4.116 for the concrete grade of the insitu concrete or the precast unit whichever is lower

μ = coefficient of shear friction, Table 4.115

σ_N = stress per unit area of external normal force across the joint, positive for compression and negative for tension, however $\sigma_N \leq 0.6\ f_{cd}$

ν = efficiency factor, see 4.3.2.3 above

ρ = A_s/A_j

A_s = area of reinforcement crossing the joint, including ordinary shear reinforcement (if any)

A_j = area of joint

α = defined in Figure 4.136a, but $45° \leq \alpha \leq 90°$.

(104) Shear reinforcement is required in the joint when:

$$\tau_{Sdj} > k_T \tau_{Rd} + \mu \sigma_N \quad (4.191)$$

The necessary amount of reinforcement should be determined using Equation (4.190). It should have a total area and a longitudinal distribution corresponding to:

$$\tau_{Sdj} - (k_T \tau_{Rd} + \mu \sigma_N) \quad (4.192)$$

A stepped distribution as illustrated in Figure 4.136c is acceptable.

(105) Shear reinforcement at the joint should be provided with adequate anchorage on both sides of the interface.

(106) The longitudinal shear resistance of grouted joints between slab or wall elements can be calculated according to (103). However, in cases where the joint may be cracked (e.g. in floors acting as diaphragms), k_T should be taken as 0 for smooth or rough joints and |0.5| for indented joints.

τ_{Rdj} for the average longitudinal shear between slab elements without indented joints should be limited to |0.1| N/mm².

Table 4.115: Values for the coefficients k_T and μ

Type of surface	k_T	μ
(monolithic *	2.5	1.0)
toothed (indented) surface	$2\eta_k$**	0.9
rough surface	1.8	0.7
smooth	1.4	0.6
very smooth	0	0.5

* See 4.3.2.5 of ENV 1992-1-1

** For members with uniform width of joint, $\eta_k = n\, h_2/l_j$

where l_j is the length of the joint
n is the number of keys in l_j, and
h_2 is the length of key (see Figure 4.136a)

Table 4.116: Value for τ_{Rd} (N/mm²) with γ_c = 1.5 for different concrete strengths.

f_{ck}	12	16	20	25	30	35	40	45	50	55	60
τ_{Rd}	0.18	0.22	0.26	0.30	0.34	0.37	0.41	0.44	0.48	0.48	0.48

4.5.4 FLEXURAL AND TENSILE JOINTS

P(101) Flexural joints are capable of transmitting bending moment (e.g. slab/wall joint, beam/column joint etc).

The bending moment acting in the joint depends on the stiffness of the elements and on the stiffness of the joint itself.

Particular attention shall be paid to reliable anchorage of the reinforcement bars in order to avoid bond failure.

P(102) Continuity of reinforcement across the joint shall be assured.

(103) Continuity within the joint may be obtained by:

- lapping of bars;
- welding of bars or steel plates;
- reinforcement grouted into apertures;
- overlapping reinforcement loops;
- threaded couplings;
- prestressing;
- threaded or filled sleeves.

Other types of connection may be used when properly justified.

4.5.5 BEARINGS

4.5.5.1 General Requirements

P(101) The integrity of bearings for precast members shall be ensured by:

a) effective reinforcement in the elements above and below the bearing,
b) preventing loss of bearing through movement and
c) suitable limitation of the bearing stress.

In the absence of a sliding bearing, horizontal forces at the bearing can reduce the load-carrying capacity of the supporting member considerably by causing premature splitting or shearing. The forces may be due to creep, shrinkage and temperature effects or may result from misalignment, lack of plumb or other causes. When likely to be significant, allowance should be made for these effects in designing and detailing the connection by the provision of:

a) suitable lateral reinforcement in the supporting and supported members; or
b) continuity reinforcement to tie together the ends of the supported members.

Where large rotations are likely to occur at the end supports of flexural members, suitable bearings capable of accommodating these rotations should be used. The rotations may also throw the line of action of loads on to the extreme edges of bearings; in such cases allowance should be made for consequent increase in bending moments or local bearing stresses.

Bearings shall be dimensioned and detailed in order to assure correct positioning, accounting for production and assembling tolerances.

(102) The design and dimensioning of the supporting and supported members at a bearing should take into account the anchorage requirements and the necessary dimensions of bends of the reinforcement in the members.

P(103) Possible local effects of prestressing anchorages and their recesses shall be taken into account.

4.5.5.2 <u>Bearings for non-isolated members</u>

(101) The nominal length of a simple bearing shown in Figure 4.137 may be calculated as:

$$a = a_1 + \sqrt{(a_2^2 + \Delta a_2^2 + a_3^2 + \Delta a_3^2)} \qquad (4.193)$$

where:

- a_1 is net bearing length and may be determined from $F_{Rd,sup} = a_1\, b_1\, \sigma_{Rd} \geq F_{Sd,sup}$, but should not be less than 40mm.
- $F_{Sd,sup}$ is design support reaction
- b_1 is bearing width of member (see (102) below)
- σ_{Rd} is the design resisting stress of the bedding material or the precast member. In the absence of other specifications, the following values should be used:
  $\sigma_{Rd} \leq \boxed{0.4}\, f_{cd}$ for dry concrete to concrete bearings (corresponding to Clause 4.5.2 (103))
  Otherwise:
  $\sigma_{Rd} \leq f_{cd,bearing} \leq \alpha\, f_{cd} = 0.85 f_{cd}$

For situations where the bearing width, b_1, is less than the member width, b, and subject to the requirements of Clause 5.4.8.1(1) and (2) of ENV 1992-1-1, then σ_{Rd} may be calculated from Equation (5.22) of ENV 1992-1-1.

$f_{cd,bearing}$ is the design strength of the bedding material or the precast element whichever is lower

a_2 is the distance assumed ineffective from the outer end of supporting member (seeFigure 4.137a and Table 4.117)

a_3 is the distance assumed ineffective from the outer end of supported member (seeFigure 4.137a and Table 4.118)

Δa_2 is an allowance for tolerances of a_2:
|15mm| for steel or precast concrete supports
|20mm| for masonry support
|20mm| for insitu concrete supports

Δa_3 is an allowance for tolerances of a_3:
$l_n/2500$, l_n is the clear distance between the faces of the supports in mm.

(102) Where bearing width b_n exceeds |600mm| the pressure distribution should be assessed. In the absence of more accurate information b_n may be limited to |600mm| in the calculation, assuming a uniform distribution of pressure.

4.5.5.3 Bearings for isolated members

(101) The nominal length of bearing for isolated members should be |20mm| greater than for non-isolated members.

P(102) Where a member is free to move relative to a support the net bearing length a_1 shall be increased to allow for the likely movement.

P(103) Where a member is tied to a support and the tie is provided away from the level of the support the net bearing length a_1 shall be increased to allow for effects due to rotation of the support about the tie.

Table 4.117: Distance a_2 assumed ineffective from outer end of supporting member

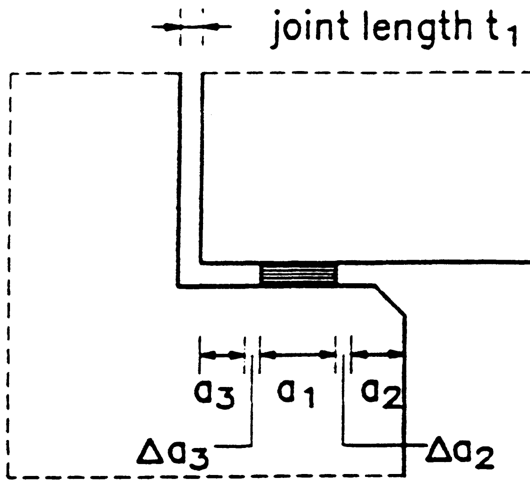
	Bearing stress $\sigma_{Sd} > 0.4\ f_{cd}$		Bearing stress $\sigma_{Sd} \leq 0.4\ f_{cd}$
Support material	Line supports (Floors)	Concentrated Supports (Beams)	
Steel	0	5mm	0
Concrete - non reinforced	25mm *	35mm	0
Brickwork	25m	35mm	25mm
Reinforced Concrete	Not less than nominal cover of reinforcement on outer face of support	35mm	0
Reinforced concrete vertical loop reinforcement exceeds 12mm diameter	35mm	Nominal cover plus inner radius of bend plus bar diameter	0

* Alternatively, a_2 for a line support may be calculated as: $100\ (\sigma_{Sd}/f_{cd} - 0.4)$. However $0 < a_2 \leq 25mm$

Table 4.118: Distance a_3 assumed ineffective from outer end of supported member

	Bearing stress $\sigma_{Sd} > 0.4\ f_{cd}$		Bearing stress $\sigma_{Sd} \leq 0.4\ f_{cd}$
Detailing Method	Line supports (Floors)	Concentrated Supports (Beams)	
Straight bars, horizontal loop or vertical loops not exceeding 12mm in diameter, close to end of member	10mm or end cover, whichever is the greater	15mm or end cover	0
Tendon or straight bars exposed at end of member	0	15mm	0
Vertical loop reinforcement bar size exceeding 12mm	15mm	End cover plus inner radius of bend of bars	0

a) Elevation



b) Plan

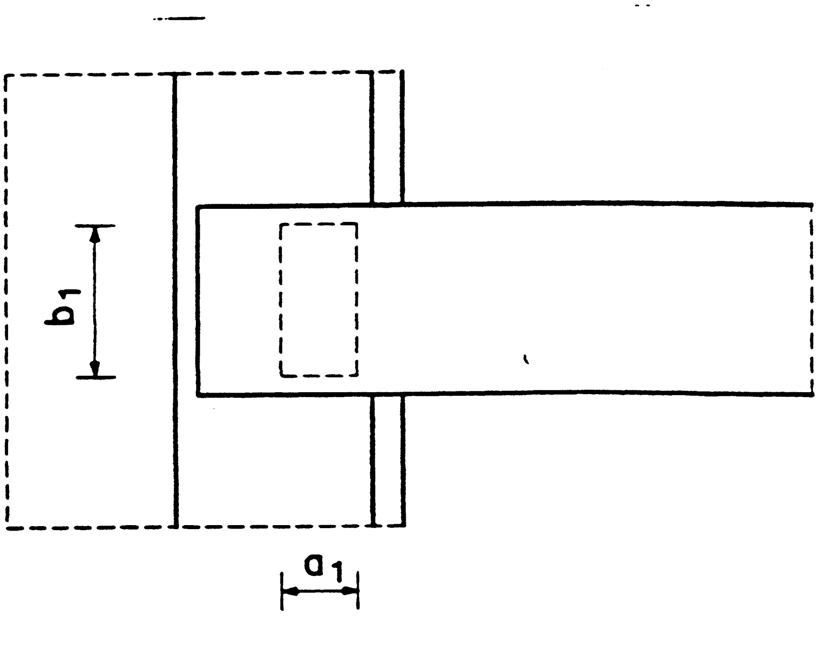


Figure 4.137: Bearing Layout

5 DETAILING PROVISIONS

This clause of ENV 1992-1-1 applies except as follows:

5.2 STEEL FOR REINFORCED CONCRETE

5.2.2 BOND

5.2.2.1 Bond conditions

Addition after Application Rule (3):

(104) Where it can be justified, the limit of |250mm| in Clause 5.2.2.1 (2)b of ENV 1992-1-1 may be increased.

5.2.2.2 Ultimate bond stress

Addition after Application Rule (3):

(104) The design values of the ultimate bond stress, f_{bd}, in ENV 1992-1-1, Clause 5.2.2.2., may be increased by |40|% provided one of the following conditions is met:

- cover to reinforcement at least |10| ϕ (ϕ: bar diameter) or
- significant transverse pressure exists or
- an appropriate confinement reinforcement is provided.

(105) Where concrete strength class greater than C50/60 is used, the value of f_{bd} should be limited to that which applies to strength class C50/60.

5.2.3 ANCHORAGE

5.2.3.2 Anchorage methods

Addition after Application Rule (4):

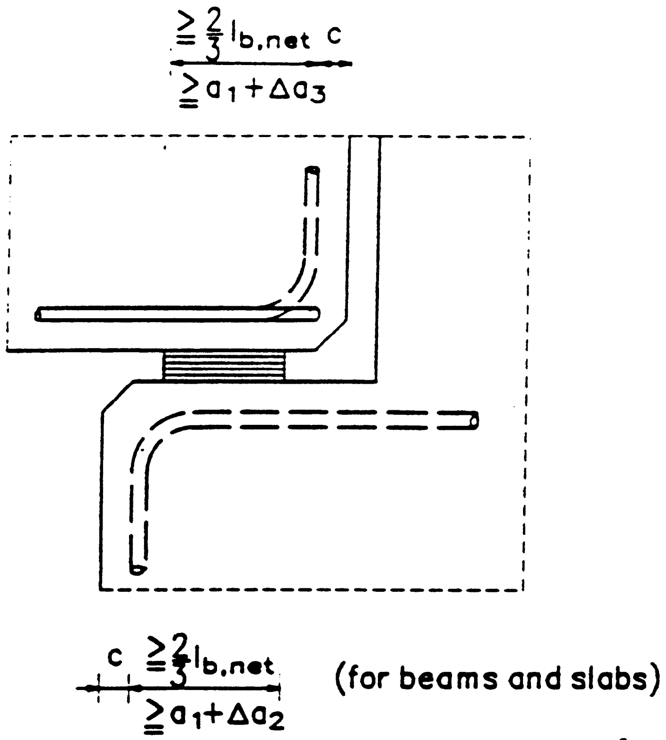
(105) The main flexural reinforcement in the supporting and supported members should be anchored effectively. An example is shown in Figure 5.121.

(106) Where insitu joints are incorporated in precast construction the size should be determined taking account of:

- the size of reinforcement,
- lap requirements,
- allowance for bends,
- cover,
- inaccuracies of placing precast units
- practicability of placing and compacting the concrete.

The joint dimensions should not normally be less than |10mm|.

(107) The bond stress applicable to the straight ends of threaded bars should be taken as for plain bars (see Table 5.3 of ENV 1992-1-1).



a_1, Δa_2 and Δa_3 are defined in Figure 4.137.

Figure 5.121: Example of the Detailing of Bearings

5.3 PRESTRESSING UNITS

5.3.3 HORIZONTAL AND VERTICAL SPACING

5.3.3.1 Pre-tensioning

Addition after Application Rule (1):

(102) **Bundling of tendons is permitted in areas outside the anchorage zones, provided that placing and compacting of the concrete can be carried out satisfactorily and that bond can be attained between the concrete and the tendons.**

For the detailing of the bundles of tendons, the rules in ENV 1992-1-1, Clause 5.2.7 for bundles of bars also apply.

5.4 **STRUCTURAL MEMBERS**

5.4.2 **BEAMS**

5.4.2.1 **Longitudinal reinforcement**

5.4.2.1.1 **Minimum and maximum reinforcement percentage**

Replacement of Application Rule (1) by:

(101) The effective cross-sectional area of the longitudinal tensile reinforcement should be not less than that required to control cracking (see Clause 4.4.2 of ENV 1992-1-1), nor less than:

$$|\underline{0.6}|\, b_t d/f_{yk} \nless |\underline{0.0015}|\, b_t d \quad \text{for concrete strength class} \leq \text{C50/60} \qquad (5.114)$$

$$\nless |\underline{0.0018}|\, b_t d \quad \text{for concrete strength class} > \text{C50/60}$$

(f_{yk} in N/mm²)

where b_t denotes the mean width of the tension zone; for a T-beam with the flanges in compression, only the width of the web is taken into account in calculating the value of b_t. Sections containing less reinforcement than that given by Equation (5.114) should be considered as unreinforced.

5.4.2.2 **Shear reinforcement**

Replacement of Table 5.5 by:

Table 5.105 - Minimum values for ρ_w

Concrete Classes*	Steel Classes		
	S220	S400	S500
C12/15 and C20/25	0.0016	0.0009	0.0007
C25/30 to C35/45	0.0024	0.0013	0.0011
C40/50 to C50/60	0.0030	0.0016	0.0013
>C50/60	0.0034	0.0018	0.0014
* as assumed in the design			

Replacement of Application Rule (7) by:

(107) The maximum longitudinal spacing s_{max} of successive series of links or shear assemblies is defined by the following conditions (with V_{Sd}, V_{Rd1} and V_{Rd2} as defined in 4.3.2 in ENV 1992-1-1):

- if $V_{Sd} \leq 1/5\ V_{Rd2}$ then $s_{max} = 0.8\ d \ngtr 300mm$ (see Note below)	(5.117)
- if $1/5\ V_{Rd2} < V_{Sd} \leq 2/3\ V_{Rd2}$ then $s_{max} = 0.6\ d \ngtr 300mm$ (see Note below)	(5.118)
- if $V_{Sd} \leq 2/3\ V_{Rd2}$: $s_{max} = 0.3\ d \ngtr 200mm$	(5.119)
(For V_{Rd2}, see Section 4.3.2.4 Equations (4.25) and (4.26))	

Note: The |300mm| limit applies to concrete strength classes up to C50/60. For concrete strength class greater than C50/60 this limit should be reduced to |200mm|.

Replacement of Application Rule (9) by:

(109) The maximum spacing of the legs in a series of shear links should not exceed :

- if $V_{Sd} \leq 1/5\ V_{Rd2}$ then s_{max} = d or |800mm|, whichever is less (5.120)

- if $V_{Sd} > 1/5\ V_{Rd2}$ then Equation (5.118) or (5.119) applies. (5.121)

The limit of |800 mm| in Equation (5.120) limit applies to concrete strength classes up to C50/60. For concrete strength class greater than C50/60 this limit should be reduced to |600 mm|.

Addition after Application Rule (10):

(111) Restraining moments may be resisted by top reinforcement placed in the topping or in the open cores of hollow core units. In the former case the horizontal shear in the joint should be checked according to 4.5.3. The length of the top reinforcement is governed by rules in ENV 1992-1-1, Clause 5.4.2.1.3.

(112) In the presence of a negative support moment, a reduced bearing length a_1 is permitted (see 4.5.5 above); temporary supports may be necessary during erection.

(113) Unintended restraining effects at the supports of simply supported floors should be considered in detailing.

5.4.7 REINFORCED CONCRETE WALLS

Additional clause after 5.4.7.4(1).

5.4.7.5 Definition of precast reinforced concrete walls

(101) In order to be considered as a reinforced concrete wall the reinforcement ratio should be $\rho_1 \geq |0.003|$. Otherwise the wall should be treated as a plain concrete wall and designed to ENV 1992-1-6.

5.4.7.6 Wall to floor junctions

(101) Where a wall is installed over a joint between two floor slabs or over a floor slab integrally connected with an end wall, and, in the absence of other appropriate measures or experimental justification, only |50%| of the loadbearing cross section of the wall may be considered effective for design purposes. The junction should be suitably detailed.

5.4.7.7 Sandwich panels

P(101) In the design of sandwich panels the effect of temperature, moisture, drying and shrinkage shall be considered. Reference shall also be made to the relevant CEN Product Standard.

(102) In composite sandwich panels only corrosion resistant materials should be used for connections between leaves.

(103) Fatigue should be considered where relevant.

(104) In structural parapets, the minimum reinforcement on each surface and in the horizontal and vertical directions should not be less than |1.3| cm^2/m. As a rule, edge reinforcement (see ENV 1992-1-1, Fig. 5.16) is not required.

In a non-structural leaf of a sandwich panel, the reinforcement may be placed in one layer.

Addition after Clause 5.4.8.3(2):

5.4.9 PRECAST SLABS USING CONCRETE STRENGTH CLASS GREATER THAN C50/60

(101) The minimum reinforcement should be in accordance with 5.4.2.1.1(101).

5.4.10 POCKET FOUNDATIONS

P(101) Concrete pockets shall be capable of transferring vertical actions, bending moments and horizontal shears from columns to the soil. The pocket shall be large enough to enable a good concrete filling below and around the column.

a) Pockets with keyed surfaces

(102) Pockets expressly wrought with indentations or keys may be considered monolithic foundations.

(103) Where vertical tension due to moment transfer occurs careful detailing of the overlap reinforcement of the column and the foundation is needed, allowing for the fact that the lapped bars are separated.

The lap length according to ENV 1992-1-1, Clause 5.2.4, should be increased by at least the horizontal distance between the column bar and the vertical lapping bar in the foundation (see Figure 5.122a). Adequate horizontal reinforcement for the lapped splice should be provided.

(104) The punching shear design should be as for monolithic column/foundation connection according to ENV 1992-1-1, Clause 4.3.4 as shown in Figure 5.122a, provided the shear transfer between the column and footing is assured. Otherwise the punching shear design should be as for pockets with smooth surfaces.

b) Pockets with smooth surfaces

(105) The forces and the moment may be assumed to be transferred from column to foundation by compressive forces F_1 - F_2 through the concrete filling and corresponding friction forces, as shown in Figure 5.122 b. This model requires that $l \geq |\underline{1.2}|\ h$.

(106) The coefficient of friction should not be taken greater than $\mu = |\underline{0.3}|$.

(107) This model requires that special attention is paid to:

- detailing of reinforcement for F_1 in top of pocket walls;

- transfer of F_1 along the lateral walls to the footing;

- anchorage of main reinforcement in the column and pocket walls;

- shear resistance of column ends;

- punching resistance of the footing slab under the column force, the calculation for which may take into account the insitu structural concrete placed under the precast element.

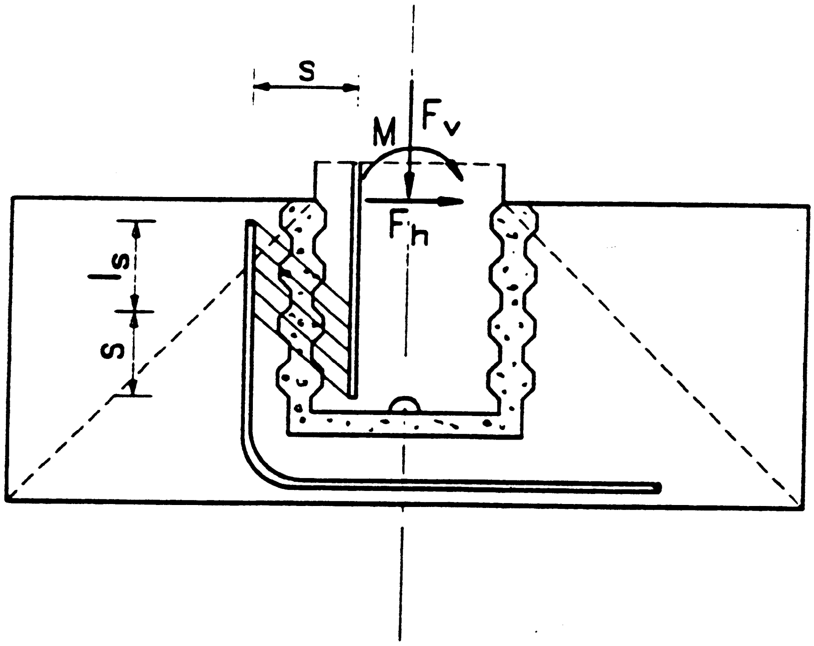


Figure 5.122 a: Pocket Foundation with Keyed Joint Surface

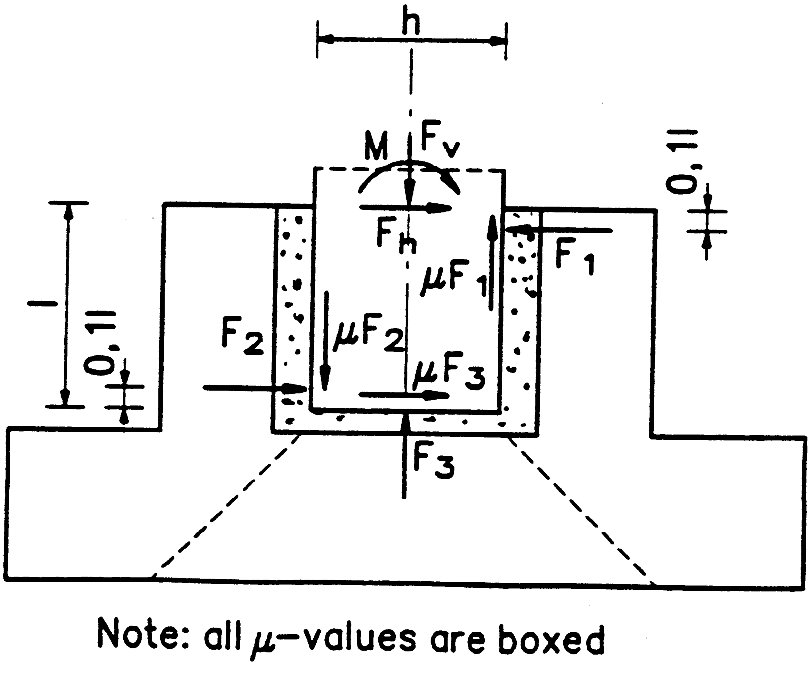


Figure 5.122 b: Pocket Foundation with Smooth Joint Surfaces

5.5 LIMITATION OF DAMAGE DUE TO ACCIDENTAL ACTIONS

Replacement of Clauses 5.5.1, 5.5.2 and 5.5.3 by:

5.5.1 TYING SYSTEM

P(101) Ties shall be provided:

a) to prevent local damage due to accidental actions such as impact or explosion.

b) to provide alternative load paths should local damage occur.

P(102) The following ties shall be provided:

a) Peripheral ties;
b) Internal ties;
c) Horizontal column or wall ties;
d) Where required, vertical ties, particularly in panel buildings.

(103) Where a building is divided by expansion joints into structurally independent sections, each section should have an independent tying system.

(104) In the design of the ties the reinforcement may be assumed to be acting at its characteristic strength and capable of carrying tensile forces defined in the following clauses. Reinforcement provided for other purposes may be regarded as part of or the whole of these ties.

5.5.2 PROPORTIONING OF TIES

P(101) Tie sections shall be the greater of either, that necessary to withstand loads generated by appropriate accidental actions as specified in Eurocode 1 or, that necessary to provide a specified load path around a damaged zone.

(102) For the purposes of tie design, forces other than those generated directly by accidental actions or consequent upon the occurrence of actual local damage may be ignored.

a) Peripheral ties

P(103) At each floor and roof level an effectively continuous peripheral tie within 1.2m from the edge shall be provided. The tie may include reinforcement used as part of the internal tie.

(104) The peripheral tie force should be capable of resisting a tensile force $F_{tie} = l_1 \times 10\text{kN/m} \leq |\underline{70}|$ kN (5.122)
(l_1 is length of end span [m]).

P(105) Structures with internal edges (e.g. atrium, courtyard etc.) shall have peripheral ties as for external edges which shall be fully anchored.

b) Internal ties.

P(106) These ties shall be at each floor and roof level in two directions approximately at right angles. They shall be effectively continuous throughout their length and should be anchored to the peripheral ties at each end (unless continuing as horizontal ties to columns or walls)

(107) The internal ties may, in whole or in part, be spread evenly in the slabs or may be grouped at or in beams, walls or other appropriate positions. In walls they should be within $|\underline{0.5\text{ m}}|$ of the top or bottom of floor slabs, see Fig. 5.123.

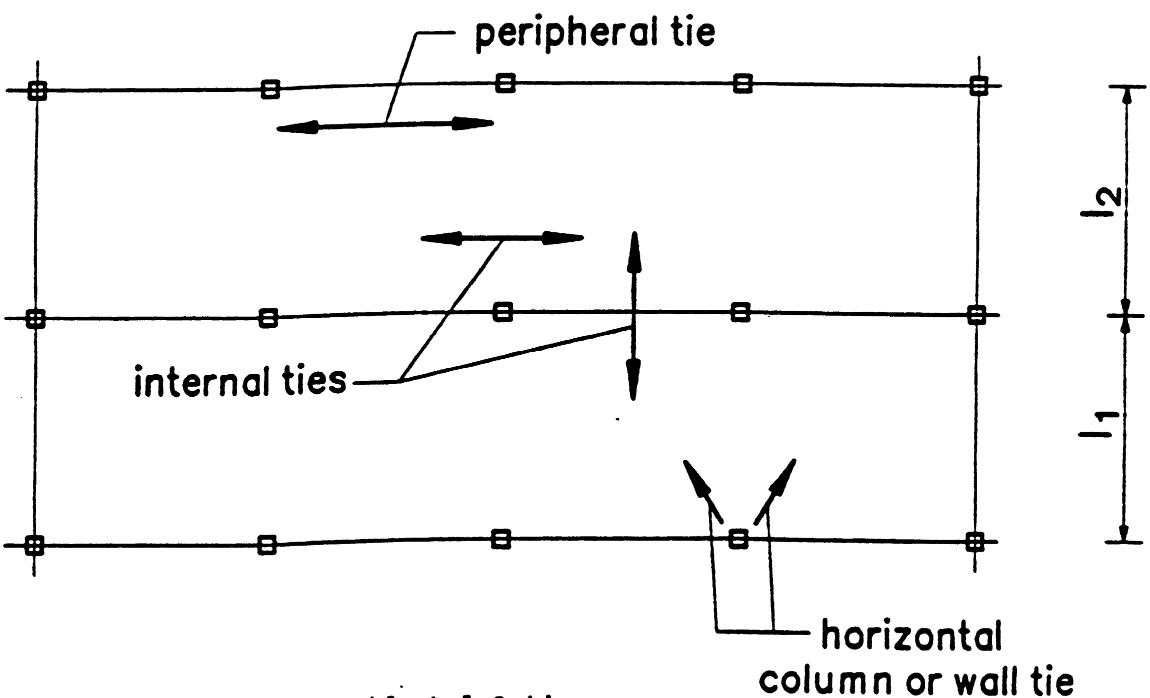


Figure 5.123: Ties for Accidental Actions

(108) In each direction, internal ties should be capable of resisting a tensile force F_{tie} (in kN per metre width) equal to:

$$F_{tie} = |\underline{20}| \text{ kN/m} \quad (5.123)$$

(109) In floors without screeds where ties cannot be distributed across the span direction, the transverse ties may be grouped along the beam lines. In this case the minimum force F_{tie} on an internal beam line is the lesser of:

(a) |70| kN

(b) $$\frac{(l_1 + l_2)}{2} \times |20| \text{ kN} \tag{5.124}$$

where l_1 and l_2 denote the span lengths (m) of the floor slabs on either side of the beam (see Fig. 5.123).

(110) Internal ties should be connected to peripheral ties such that the transfer of forces is ensured.

c) Horizontal ties to columns and walls

P(111) Facade columns and walls shall be tied horizontally into the structure at each floor and roof level.

(112) The ties should be capable of resisting a tensile force F_{tie} = |20| kN per metre of the facade. For columns the force need not exceed F_{tie} = |150| kN per column.

(113) Corner columns should be tied in two directions. Steel provided for the peripheral tie may be used as the horizontal tie in this case.

d) Vertical ties

P(114) In panel building of 5 storeys or more, vertical ties shall be provided in columns/walls to limit the damage of collapse of a floor in the event of accidental loss of the column/wall below. The ties shall form part of a bridging system to span over the damaged area.

(115) Where possible the ties should be continuous from the lowest to the highest level and be capable of carrying at least the design ultimate load applied by the floor immediately above that column/wall accidentally lost.

P(116) Where a column/wall is supported at its lowest level by an element other than a foundation (e.g. beam or slab) accidental loss of this element shall be considered in the design and a suitable alternative load path shall be provided.

5.5.3 CONTINUITY AND ANCHORAGE OF TIES

P(101) Ties in two horizontal directions shall be effectively continuous and anchored at the perimeter of the structure.

(102) Ties may be provided wholly within insitu concrete topping or at connections. Where the ties are not continuous in one plane, the bending effects resulting from the eccentricities should be considered.

(103) Ties may be post-tensioned.

(104) Ties should not normally be lapped in narrow joints between precast units. Positive mechanical anchorage should be used in these cases.

6 CONSTRUCTION AND WORKMANSHIP

This clause of ENV 1992-1-1 applies except as follows:

6.2 TOLERANCES

6.2.1 TOLERANCES - GENERAL

Addition after Principle P(3):

(104) Reference should also be made to relevant CEN Product Standards.

6.3 CONSTRUCTION RULES

Additional clause after 6.3.4.6.6.

6.3.5 PRECAST ELEMENTS AND STRUCTURES

P(101) The construction and workmanship shall comply with the relevant CEN Product and other standards.

7 QUALITY CONTROL

This clause of ENV 1992-1-1 applies except as follows:

7.4 CONTROL OF THE DIFFERENT STAGES OF THE BUILDING PROCESS

Addition after Application Rule (1):

(102) For precast elements, in addition to (1) in ENV 1992-1-1, the quality control also includes control during

- transportation
- erection
- execution of connections.

NORMATIVE APPENDICES

APPENDIX 1 Additional provisions for the determination of the effects of time-dependent deformation of concrete

Appendix 1 applies also to ENV 1992-1-3.

APPENDIX 2 Non-linear analysis

Appendix 2 applies to ENV 1992-1-3, provided that adequate ductility of the structure including joints can be ensured.

APPENDIX 3 Supplementary information on the ultimate limit states induced by structural deformations

Appendix 3 applies to ENV 1992-1-3.

APPENDIX 4 Checking deflections by calculation

Appendix 4 applies to ENV 1992-1-3.

INFORMATIVE APPENDIX

APPENDIX 105

GENERAL GUIDANCE FOR THE REDUCTION OF THE SAFETY COEFFICIENTS FOR MATERIALS, γ_M

A.105.1 INTRODUCTION

(101) The values of partial safety factors for materials in the 1992-1-1 are valid when the quality control procedures given in chapter 7 of the code are followed. Where strict and adequate quality control procedures operate, the partial safety factors may be reduced for the design of individual precast elements, on the basis of information derived from tests or measurements. Normally such reduction should be agreed on a job by job basis between the manufacturer and the client or the consulting engineer to the client or the Building Authority taking note of any contractual and regulatory requirements. It is not possible to set out the full details of the control procedures required in a code and this Appendix is intended to provide useful background information in arriving at the values for the reduced partial safety factors.

A.105.2 GENERAL PROCEDURE TO DETERMINE PARTIAL FACTORS

(101) For a simple model of resistance in the form R = m.G.f, the partial safety factor γ_m may be taken as

$$\gamma_m = (1 - 1.64\ V_f)/(1-3V_R) \qquad \text{(A5.101)}$$

Where m is the model uncertainty factor;
G is the geometrical factor;
f is the material strength;

$$V_R = \sqrt{(V^2_m + V^2_G + V^2_f)} \qquad \text{(A5.102)}$$

Where V_m is the coefficient of variation of m;
V_G is the coefficient of variation of G; and
V_f is the coefficient of variation of f.

The above formulae assume

a) log normal distribution of the three basic variables;

b) that the three variables are independent;

c) that the material strength refers to 5% fractile characteristic value; and

d) that the target reliability index $\beta = 3.8$ and weighted coefficient for resistance is 0.8

(102) The values of coefficients of variation should be obtained on the basis of tests and statistical evaluation, guidance for which may be found in ENV 1991-1 Part 1 - Basis of design.

A.105.3 GUIDANCE VALUES

(101) In the absence of other data the following may be considered appropriate subject to the conditions noted below.

a) $\gamma_c = 1.4$ and $\gamma_s = 1.10$ (fundamental combination) when the following conditions are met together:

- A strict regime of quality control operates under factory conditions in which the monitoring covers checks on geometry of the structure (including the geometry of the cross section), location of the reinforcement and properties of the materials including concrete, reinforcement and pre-stressing steels.

- Non complying units will be rejected on the basis of quality control method adopted.

- Quality assurance system is supervised by certified body; and

- 50% of tolerances defined in ENV 1992-1-1 are achieved

b) $\gamma_c = 1.3$ and $\gamma_s = 1.05$ (fundamental combination) when, in addition to the requirements under a), the following conditions are also met:

- statistical assessment is carried out of performance of the whole continuous production of identical elements;

- design is based on the actual values of the tolerances;

- the mean conversion factor η of the strength of drilled cores and standard specimen is greater than 0.9.

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