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Structural Concrete

Textbook on Behaviour, Design and Performance
Updated knowledge of the CEB/FIP Model Code 1990

Volume 3

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Updated knowledge of the CEB/FIP Model Code 1990

Volume 3

Durability – Design for fire resistance – Member design –
Assessment, maintenance and repair – Practical aspects

December 1999

Subject to priorities defined by the Steering Committee and the Praesidium, the results of *fib*'s work in Commissions and Task Groups are published in a continuously numbered series of technical publications called 'Bulletins'. The following categories are used:

category	minimum approval procedure required prior to publication
Technical Report	approved by a Task Group and the Chairpersons of the Commission
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Manual or Guide (to good practice)	approved by the Steering Committee of *fib* or its Publication Board
Recommendation	approved by the Council of *fib*
Model Code	approved by the General Assembly of *fib*

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This **Bulletin N° 3** has been approved as a *fib* Manual by the Steering Committee in April 1999.

Structural Concrete, the Textbook on Behaviour, Design and Performance, is published in 3 volumes (*fib* Bulletins 1, 2, 3):

Volume 1 Chapter 1: Introduction
Chapter 2: Design process
Chapter 3: Materials

Volume 2 Chapter 4: Basis of design

Volume 3 Chapter 5: Durability
Chapter 6: Design for fire resistance
Chapter 7: Members design
Chapter 8: Assessment, maintenance and repair
Chapter 9: Practical aspects

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First published by the International Federation for Structural Concrete (*fib*)
This volume printed 2000 as 2nd edition (minor editorial corrections).
Post address: Case Postale 88, CH-1015 Lausanne, Switzerland
Street address: Federal Institute of Technology Lausanne - EPFL, Département Génie Civil
Tel (+41.21) 693 2747, Fax (+41.21) 693 5884, E-mail fib@epfl.ch, Telex 454 478 EPFL CH

ISSN 1562-3610
ISBN 2-88394-043-6

Printed by Sprint-Druck Stuttgart

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5 Durability

By Steen Rostam

5.1 Introduction

5.1.1 Background

Concrete is the most versatile and robust construction material available and has therefore obtained a dominating position in construction.

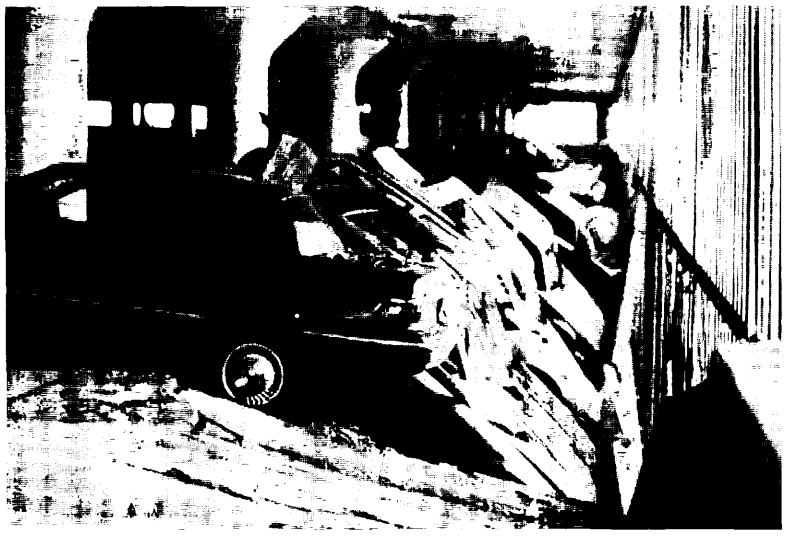


Figure 5.1-1: Example of the consequences of a non-durable concrete structure relative to its environment -the parking deck below was also fully occupied ..... (Photo: Courtesy Arminox)

Thus it becomes an economic disaster when a huge urban dwelling, a large bridge, or a major marine structure deteriorates just after a few years in service. With increasing frequency such examples have been reported from the 70´ies and on, as exemplified in Figure 5.1-1. The reasons are very complex but fortunately the main causes have been identified, and it is essential to have these causes highlighted with the aim of rectifying design methods, construction procedures, material compositions as well as maintenance and repair procedures, to ensure more reliable structures in the future.

Equally important is the need for developing assessment and rehabilitation procedures for the large stock of existing structures not living up to expectations.

5.1.2 Structural design versus durability design

The generally accepted aim of a design is "to achieve an acceptable probability that the structure being designed will perform satisfactorily during its intended life".

When designing a structure today the designer first defines the loads to be resisted. As these loads usually vary, he applies some safety factors to be on the safe side. These factored loads must then be resisted by the structure through selecting a combination of structural systems, element geometry, material types and material's strengths. As it is difficult to predict the precise properties of the materials, the geometry and the qualities, which are achieved in the structure, safety factors are applied to account for this by limiting the maximum allowable stresses. Mathematical equations are then used to verify that the probabilities of the loads, which exceed the resistances, are maintained at an acceptable low level.

When it comes to durability design to verify that the intended life can be achieved with an acceptable level of reliability, the situation is entirely different. It seems to be acceptable without question to use a grossly over-simplistic approach. The codes provide only qualitative definitions of exposure and they fail to define the design life in relation to durability, [Bamforth (1998)]. In particular, they fail to quantify the durability limit states that must be

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exceeded for the design life to be ended. Previous approaches fail to recognise that, in relation to durability, it is not the properties of the materials or components alone that define performance, but the condition of the structure in its environment as a whole, and its need for intervention. This performance can be defined by functional requirements such as fitness-for-purpose, which includes issues such as deflections, cracks and spalling, vibrations, aesthetics and structural integrity.

When it comes to residual service life assessment of existing structures the same situation of using over-simplistic procedures prevail as for service life design of new structures. This is in spite of the fact that once the structure is a reality a large part of the inherent uncertainties associated with new designs, can be eliminated or limited to required levels through appropriately planned inspections, measurements and testing. The longer the structure has served, and the longer it has interacted with the environment the more reliable could the residual service life forecast be. This is the condition, which shall be kept in mind when performing assessment and residual service life evaluation, as presented as part of Chapter 8.

5.1.3 Uniqueness of structural concrete

With respect to deterioration, concrete structures have some important characteristic properties, which differ fundamentally from structures made of other structural materials, [Rostam (1995)]. These properties are the following:

- The quality and the performance of the concrete assumed at the design stage of a specific structure are 'wishful thinking'. Nevertheless specific quality and performance requirements are specified in the design basis.
- The true quality and performance characteristics of the concrete are determined through the actual execution process during construction on site. Hence, this very short period of time constitutes the most important features in ensuring the required durability performance of the finished structure.
- If durability performance turns out to be sub-standard, this is most often not apparent nor detectable until some time has passed due to the nature of the deterioration of concrete structures exposed to an aggressive environment. The time passed before premature deterioration becomes apparent may often be longer than the contracted liability period of the contractor and the designer, but very much shorter than the service life expected by the owner.

5.1.4 Scope

This chapter on durability aims at providing the reader with a basic understanding of the complex set of multidisciplinary phenomena governing durability and long term performance of concrete structures as a basis for service life design. The focus is on the structure and its interaction with the environment. Valuable additional guidance may also be found in CEB (1992).

Concrete technology, material properties and deterioration mechanisms have been described in detail in Chapter 3.1. Cracking and crack control have been described in Chapter 4.3.2. These chapters include essential information related to durability and they are referred to for more detailed studies of background information related to durability. Assessment, maintenance and repair of existing concrete structures is presented in Chapter 8.

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5.2 Functional requirements

The concept of durability can be difficult to quantify and to use in practical design. According to Fagerlund (1979) the structure is considered to be durable in the actual environment as long as its function is acceptable. The concept of durability is therefore linked to a functional requirement which often can be expressed indirectly as a minimum or a maximum value of a certain single fundamental property or as a complex of fundamental properties.

Examples of functional requirements and corresponding fundamental properties could be:

- A *required* minimum load carrying capacity. The corresponding fundamental properties are: Concrete and steel strength, corrosion depth of the reinforcement, and depth of the spalls of the concrete surface.
- A *required* maximum acceptable deformation. The corresponding *fundamental properties* are: E-modulae, shrinkage, creep, thermal movement, and settlements.
- A *required* maximum penetrability for gaseous or liquid substance. The corresponding *fundamental properties* are: Concrete permeability, capillarity and diffusivity, and the size and location of cracks. These parameters are in turn functions of structural design, water-cement ratio, aggregate type, cement type, curing method, degree of hydration, etc.

All these fundamental properties are more or less time dependent. This leads to the obvious conclusion that the service life of a structure is the space of time during which it fulfils all the functional requirements. A **service life design** thus becomes the designer's choice of fundamental properties to fulfil the functional requirements during a predetermined space of time.

The service life concept is thus an operational way of quantifying durability, and as originally introduced in Fagerlund (1979), has led to service life design as the rational way to ensure a structure's sufficient resistance against deleterious actions from the environment. This also shows that service life is a function of the environment in which the structure is placed.

Therefore, designing new structures for a given service life, or determining the remaining service life of existing structures, requires the following:

- Identify and formulate the functional requirements to be fulfilled
- Determine the properties and type of aggressivity of the actual - or micro - environment of the structure
- Ensure rationale in the service life design by using available mathematical models describing the interrelations between environmental properties, material properties and deterioration mechanisms. Engineering judgement may be needed to compensate for insufficient or lack of models
- Consider the possibilities of updating the service life estimations of the structure in service by measuring the critical properties determining the deterioration processes through sensors or by non-destructive testing.

Each of these items are described and discussed in more or less detail in this Chapter 5 and in Chapter 8.

5.3 Definitions

The basic relationships between "durability", "performance" and "service life" are described below, [CEB (1992)].

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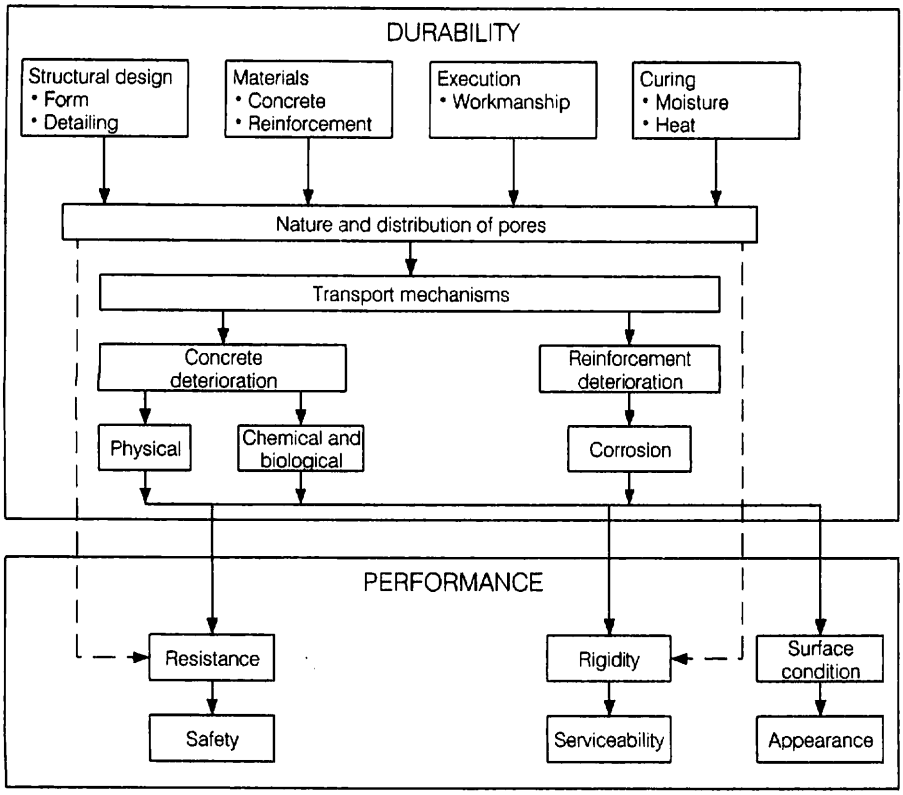


Figure 5.3-1: Relationship between durability and performance [CEB (1992)]

The **durability** of a structure means its resistance against the actions from the environment surrounding the structure. The elements of design, materials selection, execution and curing which determine the "quality" of the concrete is illustrated in Figure 5.3-1. In this respect the quality means the nature and distribution of pores in the concrete determining the possible transport mechanisms. The interaction between the concrete and the environment determines the possible deterioration mechanisms. Durability is a term which cannot be quantified and is therefore a term which is not operational in practice.

Performance of a structure means its combined short and long-term fulfilment of the functional requirements. These requirements relate to the safety, the serviceability and the appearance of the structure during its time in use.

The **service life** of a structure hence becomes the space of time during which the structure performs satisfactorily, i.e. the space of time during which it fulfils all the functional requirements, as stated in the previous section.

The development in time of nearly all types of deterioration mechanisms of concrete structures may be modelled by a two-phase curve illustrated on Figure 5.3-2 [Tuutti (1982)]. This is the basis for defining the service life of a structure.

The two phases of deterioration are the following:

- **The initiation phase**. During this phase no noticeable weakening of the material or the function of the structure occurs, but some protective barrier is broken down or overcome by the aggressive media. Carbonation, chloride penetration and sulphate accumulation - the latter two accelerated by cyclic wetting and drying - are examples of such mechanisms determining the duration of the initiation period.

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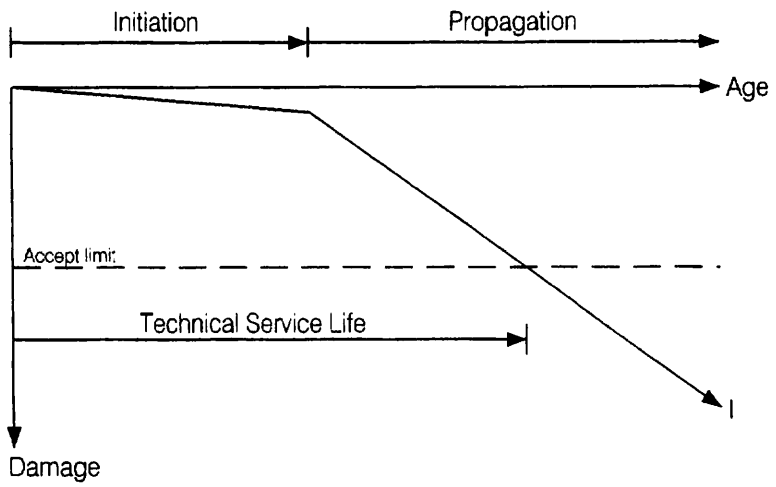


Figure 5.3-2: Service life of concrete structures. A two-phase modelling of deterioration [Tuutti (1982)].

- **The propagation phase**. During this phase an active deterioration develops and loss of function is observed. A number of deterioration mechanisms develop at an increasing rate with time. Reinforcement corrosion is one such important example of propagating deterioration.

In practice there are three different types of service life relevant for structures, depending on the type of performance being considered:

- **Technical service life**, the time in service until a defined unacceptable state of deterioration is reached.
- **Functional service life**, the time in service until the structure becomes obsolete from a functional performance point of view, due to changed requirements.
- **Economic service life**, the time in service until replacement of the structure is economically more attractive than keeping the structure in service.

The technical service life is related to the schematic, or mathematical, modelling of the deterioration in time as summarised above and illustrated in Figure 5.3-2.

The functional and economic service life reflect the more common reality that structures are replaced or upgraded when they are considered obsolete for other than purely technical reasons. Usually no prior engineering efforts can forecast such situations.

The process of creating a design basis for service life is similar to the process of creating a design basis for the design of new structures. When preparing a design basis for the assessment and re-design of existing structures, see Chapter 8, more factual information on the structure is available and a more reliable service life forecast can be made, which constitutes a **service life updating**.

5.4 Realism in service life design

5.4.1 Responsibilities

The service life of a structure represents its durability and is determined by specialised input from all parties involved:

- **The Owner** (client) by defining his demands and wishes
- **The Designer** by preparing the design, specifications and conditions
- **The Contractor**, who shall follow the requirements of the owner and designer in his construction work
- **The User** - this often, but not always, being the owner - who is responsible for timely maintaining the structure to ensure that the intended service life is obtained without unforeseen high costs.

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Any of these four parties may - by their actions or lack of attention - contribute to an unsatisfactory state of durability of the structure. Also, interactions between two parties, the so-called interface problems, may cause faults, which can have an adverse effect on durability. Thus, each of these parties have well defined responsibilities within the process of creating, operating and maintaining concrete structures, [Markeset and Rostam (1999)].

But, first and foremost the owner must ask for quality, he must check the quality received, he must maintain the structure, and he must pay the initial costs as well as the costs for maintaining the structure in a condition satisfactory to him, [Rostam and Schiessl (1994)].

5.4.2 Codes and standards as design basis

The minimum requirements to be fulfilled are stated in the national codes and standards. However, it is recognised that traditionally these documents do not yet include service life aspects in sufficient detail to provide what would be considered the most updated durability related quality of structures exposed to the aggressive environments. Historically and for reasons of tradition codes and standards differ considerably from country to country.

Only in very few cases can systematic differences in national requirements explain noticeable differences in the performance and durability of concrete structures in the different countries. Examples could be the acceptance or non-acceptance of using calcium chloride as an accelerator, or allowing high water-cement ratios, or adopting very small covers for exposed concrete structures, all of which would be very counterproductive to a long service life.

However, standards cannot by themselves ensure quality - not even good standards - as they only define the minimum requirements. Standards can only serve as helpful tools in support of more specific recommendations and project specifications developed to ensure fulfilment of the individual owner's requirements.

5.4.3 Deem-to-satisfy design

The deterministic design for durability will govern for some time still, but with regular updating of the characterisations of the environment and improvements of the modelling of transport and deterioration mechanisms. Hence, the currently used means of ensuring durability and service life in national codes as well as according to CEB-FIP Model Code 1990 (MC 90), [CEB-FIP (1992)], will to a large extent maintain the well-known deem-to-satisfy design approach.

The deem-to-satisfy approach means to specify requirements to parameters such as the cement type and quantity, maximum water/cement ratio, minimum air content, concrete cover, type of curing, control of early age cracking, limitation of crack widths, etc. The values chosen depend on the assumed aggressivity of the environment.

Main parts of the service life design procedures described in this chapter are based on the deem-to-satisfy approach but applied within a rationale where the time dependent features of the actions (environment) and resistances (layout, detailing and concrete quality) are taken into account in a deterministic way.

5.4.4 Probabilistic performance based service life design

Due to the nature of concrete and the very individual way concrete and concrete structures are made it is unavoidable that variations in properties occur. The variation in properties do not only occur between different structures, different types of concrete compositions, and different geographic locations, but also within different parts of the structure itself.

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In general it has been difficult to quantify these variations and to develop design procedures able to treat the variations in a rational and logic way aimed at service life designs similarly to the well-known structural designs. Therefore, more simplistic methods and experience related requirements have until now governed the design for durability and service life.

Service life design is currently in an interesting transition period. Modelling of environment and deterioration mechanisms is being developed on a probabilistic basis allowing reliability based service life designs. Service life design methods similar to the load-and-resistance-factor design procedure also used for structural design are also being perfected.

In a few years time such probabilistic performance based durability design of concrete structures is expected to be fully operational and may thus drastically change the design procedures for concrete structures, see Chapter 5.12.

5.5 CEB-FIP Model Code 1990

MC 90 is the first Code of Practice which states specific service life requirements to new designs, and indicates how this shall be achieved in the design process. The importance of the level of workmanship during execution, the integration of anticipated future maintenance procedures, and the need for quality assurance schemes throughout the processes of designing, constructing and operating structures are integral elements of a modern service life design.

MC 90 expresses the basic requirements for service life design as follows:

> *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.*

The Model Code introduces three new features in the design process, compared to current national codes of practice. These are:

- The influence of the period of use on the service life achieved
- The need to maintain acceptable appearance, meaning that structures should 'grow old gracefully', thus enhancing the reputation of concrete in the eyes of the public
- The costs for maintaining the structure in satisfactory use which lead to design accessible, inspectable, maintainable, repairable, and replaceable structures.

Measures necessary to ensure the required service life are chosen according to the environmental conditions and the significance of the structure. The service life depends equally on the behaviour of structural and non-structural elements. Both shall be considered during design, construction and use of the structure.

Non-structural elements such as drainage, joints, bearings, installations etc. may require specialist attention other than that of structural engineering. Particular structural components such as anchorages, couplers and deviators for prestressing tendons and their location in the structure may require particular attention.

The Model Code clauses on design, execution and maintenance will generally lead to a service life of the structures in excess of 50 years, based on the knowledge and experience behind the clauses. However, some structures will require a substantially longer service life, say 100 years or more, and other structures may only need a considerably shorter service life, say less than 25 years.

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5.6 Deterioration mechanisms

In order to understand, and cope with, the true and expected future durability performance of concrete structures, the deterioration mechanisms must be known. In fact, knowing and understanding the transport mechanisms of liquids and gaseous substance into and within concrete structures is the most important element in ensuring sufficiently durable concrete structures. This is also the fundamental basis for quantifying durability in the form of service life performance, [CEB (1992)].

The transport mechanisms being decisive for durability are described in detail in Chapter 3.1.7, to which reference is made. In the present chapter additional information focusing specifically on the design aspects of durable structures has been included.

Cracks may influence the type and rate of transport of substance into concrete. However, structural and load induced cracks are inevitable in concrete structures but do not necessarily indicate undue lack of serviceability or durability provided the crack type, extent, width and orientation are controlled through the design, execution and curing phases, see Chapter 5.7.6.

The different types and origins of cracks, and the measures to control cracking, have been described in detail in Chapter 4.3.2 to which reference is made.

The deleterious chemical and biological reactions have been presented in detail in Chapter 3.1.7, and a comprehensive treatment is found in Biczok (1972), to which reference is made.

The durability of a concrete structure will often be determined by the rate, at which the concrete is decomposed as a result of chemical reaction. For all these reactions, aggressive substances (ions and molecules) are being transported from somewhere, mainly from the environment, to- for this substance - a reactive substance in the concrete. However, even if the aggressive substance is already present in the concrete, it has to be transported in the direction of the reactive substance for the reaction to take place. *If no transport takes place, there will be no reaction*. The accessibility of the reactive substance in the concrete is therefore the rate-determining factor when an aggressive substance enters.

The rate-increasing effect of increasing temperature is mainly due to the effect on the transport rate (higher temperatures result in higher mobility of ions and molecules). Depending on the type of reaction, the accessibility will be determined by the permeability of still sound concrete or by the passivating layer of the reaction products.

The chemical reactions that may lead to a decrease in quality are well established. The most important are:

- The reaction of **acids**, **ammonium salts**, **magnesium salts** and **soft water** with the hardened cement
- The reaction of **sulphates** with the aluminates in the concrete
- The reaction of **alkalis** with reactive aggregates in the concrete, see Figure 5.6-1.

The mechanisms governing corrosion of steel in concrete have not been dealt with elsewhere in this document. Therefore, this Chapter provides a materials engineering description of the conditions of steel in concrete and the preconditions for reinforcement corrosion.

5.6.1 Water and moisture transport

The presence of water or moisture is the single most important factor controlling the various deterioration processes, apart from physical deterioration.

Water may find its way into concrete by:

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Figure 5.6-1: Cracking due to alkali-silica reactions in a column. The cracks form in the direction of least resistance thus indicating the path of the compression trajectories in this column

- *Water vapour diffusion* (vapour pressure gradient)
- *Capillary suction* (surface tension in small pores)
- *Hydraulic pressure* (water pressure gradient)

Relative and absolute influence of the three transport mechanisms on the water flow through a concrete structures, is based on:

- Net moisture movement over the year
- Average relative humidity and temperature inside and at the surface of the structure over the day, the month and the year
- Resistance of the structure to moisture movements, e.g. permeability and diffusivity determined by water/cement ratio, cracks, membranes, surface coatings, etc.
- temperature variation over the day, the month and the year
- Hydraulic pressure on a water retaining structure

This information enables the designer to estimate the position of the dew point and position of the evaporation front - information, which is needed to estimate the position and rate of deterioration of the active deterioration mechanisms.

The transport of water within the concrete is determined by the pore type, size and distribution and by cracks. Thus, controlling the nature and distribution of pores and cracks is essential when designing for durability, see Figure 5.3-1. In turn, the type and rate of deterioration processes for concrete and for normal reinforcement and prestressing reinforcement determine the resistance and the rigidity of the materials, the sections and the structural elements making up structure. The surface conditions of the structure are also determined in this way, and this is reflected in the safety, the serviceability and the appearance of a structure; i.e. these processes determine the performance of the structure, [Taheri (1998)].

Concrete exhibits comparatively high water content even when the humidity of the surrounding air is relatively low due to the high proportion and small radii of the gel pores. Increasing the humidity of the air will cause the larger pores to be filled with water, thus reducing the pore space available for the diffusion of gases. Consequently, the permeability of the concrete with regard to gases will decrease considerably with growing water content and, in the case of almost water-saturated concrete, the diffusion of gases (e.g. carbon dioxide and oxygen) becomes practically negligible.

Transport mechanisms are described in detail in chapter 3.1.7(1).

5.6.2 Passivation of steel in concrete

Steel in concrete is protected against corrosion by an electro-chemical passivation, which develops spontaneously when steel is cast into concrete. This passivation is due to the alkalinity of concrete: pH of the pore water runs up to values greater than 12.5 and for Portland cement concrete up to say 13.5. With such high pH-values, a microscopic oxide layer is formed on the steel surface changing the electrochemical potential of the steel towards a more noble condition. This oxide layer constitutes a "passive" film, which impedes the dissolution

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of iron. Corrosion of reinforcement is thus impossible, even if all other preconditions for corrosion (mainly the presence of moisture and oxygen) are fulfilled.

5.6.3 Depassivation of steel in concrete by carbonation or chlorides

A chemical reaction within the concrete increasing the risk of reinforcement corrosion is the reaction between calcium compounds, primarily $Ca(OH)_2$, and CO_2,. This leads to **carbonation** of the concrete, causing a decrease in alkalinity. As it is mainly the $Ca(OH)_2$ that causes the high pH of the concrete to develop, the pH will drop below 9 after the concrete has been totally carbonated. This will eliminate the ability to electro-chemically protect the reinforcement against corrosion.

Chloride ions (originating from seawater or de-icing salt) may penetrate through the pores to the interior of the concrete. Chloride intrusion is due to either diffusion, taking place in totally or partially water-filled pores, or capillary suction of chloride-containing water. Cement has a certain chemical and physical binding capacity for chloride ions (forming Fridell salt), depending on the chloride concentration in the pore water. However, not all the chlorides can be bound. There will always exist equilibrium between bound chlorides and free chloride ions in the pore water.

Only the free chloride ions are relevant to the corrosion of the reinforcement. It is important to note, therefore, that after carbonation of concrete bound chlorides are released again, so that the chloride content in the pore water, and consequently the risk of corrosion due to chlorides, will increase considerably. The critical chloride concentration at which corrosion will occur depends on many parameters and cannot be generally fixed.

5.6.4 Corrosion of steel in concrete

By the action of chloride ions or due to carbonation of the concrete, the passive film may be destroyed locally or over larger surface areas. A third de-passivating mechanism is a reduction of alkalinity due to the leaching out of alkalis by streaming water. In practice, this may happen in the region of weak points of the structure (e.g. leaky construction joints and wide cracks) in combination with bad concrete quality (honeycombing, high water/cement-ratio). If the pH of concrete drops below 9 at the reinforcement, or if the chloride content exceeds a critical value, the passive film and the corrosion protection will be lost.

Consequently, corrosion of reinforcement is possible when the passivation has been lost, if sufficient moisture and oxygen are available. This can be assumed to be the case for structures in the open air. The principles of passivation and depassivation hold true for reinforcing as well as for prestressing steels.

As a simplified model, the corrosion process can be separated into two single processes: the cathodic and the anodic process, as described in chapter 3.2.2-(4). Under practical conditions, rust products are more or less water-containing compounds and are thus usually expansive (up to 6-700% of dissolved iron) and able to exert very high expansion pressures.

This means that only oxygen is consumed to from rust products. This oxygen must normally diffuse through the concrete cover towards the reinforcement. Water is only necessary to enable the electrolytic process to take place, and combines with the rust depending on availability. A comprehensive treatment of reinforcement corrosion is given in Nürnberger (1996), and in Page et al (1990).

When corrosion develops in environments with low availability of oxygen, the volume of the rust products may only be 50-200% greater than the volume of the steel. Such corrosion processes proceed slowly, and in special cases the rust products may diffuse into the voids and pores of the porous concrete without causing cracking and spalling. In such rare cases se-

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rious corrosion may develop on the reinforcement without any visible warning and a sudden failure may occur.

From the above follows, that the preconditions for corrosion to occur are the following:

- Depassivation of the reinforcement must have taken place creating a location for an anodic process to develop
- The depassivated area must be in metallic contact with a neighbouring or nearby area of the reinforcement which is passivated, in order for a cathodic process to take place
- The cathodic area and the anodic area must be connected also electrolytically in order for ions to move between the two areas
- There must be sufficient moisture available in the concrete surrounding the reinforcement in order for the ions to migrate between the cathode and anode. This means that the electric resistivity of the concrete must be sufficiently low for this migration to take place at a rate which sustains corrosion as an active deterioration mechanism
- There must be oxygen available, primarily at the cathode in order to sustain the cathodic process creating hydroxyl ions, but also at the anode, where the availability of oxygen determines the type and volume of the rust products produced.

All the above conditions must be fulfilled in order for corrosion to take place. Of all the above mechanisms it is important to recognise that the rate at which iron is dissolved into rust at the anode is determined by the rate of the cathodic process. In fact, the cathodic process is in most ordinary cases the rate determining process. The cathodic process can be compared to the action of a motor, and the motor needs fuel, and in this case the fuel is the oxygen diffusing from the concrete surface through the concrete cover to the surface of the steel at the cathode.

Chloride induced corrosion is probably the most serious and widespread type of deterioration of concrete structures. Apart from chlorides entering into the concrete due to seawater spray or from de-icing salts, two other causes of having excessive amounts of chloride in the concrete shall be mentioned. Due to general chloride contamination of fine aggregates and water resources in several regions like the Gulf countries numerous concrete structures have been build - and, due to lack of other resources, are still being built - with contaminated sand and water in the initial mix. This has led to serious very early corrosion damage, Figure 5.6-2.

During the construction boom of the 60'ies and beginning of the 70'ies calcium chloride based accelerators was used in many industrialised countries later leading to serious corrosion damage, Figure 5.6-3.

Surprisingly enough calcium chloride based accelerators are still being marketed in some areas as accelerators for reinforced concrete. This in spite of the high risks of premature deterioration.

The interrelations described have the following consequences. Corrosion will not occur, neither in dry concrete (where the electrolytic process is impeded - or the electric resistivity is too high for the current to flow), nor in water-saturated concrete (where oxygen cannot penetrate), even if the passive layer at the surface of the reinforcement has been destroyed, see Figure 3.1-23. The highest corrosion rate will occur in concrete surface layers subjected to highly changing wetting (which ensures a low resistivity) and drying (allows diffusion of oxygen) conditions.

Both CO_2 and chlorides may penetrate to the steel surface through cracks some order of magnitudes faster than through uncracked concrete. This may influence the initiation period but with respect to carbonation it has been shown convincingly that with well distributed cracks and with relatively large cover a corrosion of the reinforcement at the crack usually will self heal. For chloride penetration at a crack the initiation period may be reduced but a large cover will considerably enhance the durability.

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Figure 5.6-2: Cracking of column due to chloride corrosion caused by contaminated aggregates and mixing water. Age of structure at first cracking: 3 years (Gulf climate)



Figure 5.6-3: Delamination of precast balcon partition wall due to chloride induced corrosion caused by calcium chloride added as accelerator during casing, allowing two casting per day per form. Age of structure: 15 years (North European Climate)

5.6.5 Early age Cracking

'Young' concrete is especially prone to cracking, see Chapter 4.3.2. During the transition phase leading from fresh ('green') to hardening ('young') concrete, a critical period with low tensile strength and a low deformability starts a few hours - at the earliest 2 h - after casting and lasts about 4-16 h.

There are two distinct types of cracking developing during the plastic phase of the hardening concrete: **plastic shrinkage** cracking, which most commonly occurs in slabs, and **plastic settlement** or **slump cracking**, which may occur in deep members. Both types are associated with bleeding of the concrete.

Cracks caused by plastic shrinkage are mostly surface cracks, but in a few cases they can penetrate a whole slab, the crack width decreasing considerably with increasing depth from the surface. Typical crack widths are of the order of 2-3 mm at the surface.

The **hydration heat** of cement, which is set free during the setting and hardening of concrete, creates a condition of self-equilibrating stresses with tensile stresses in the outer layers and compressive stresses in the core. If the tensile stresses exceed the still low tensile strength of hardening concrete, **thermo-cracks** are formed. The cracks are always surface cracks, mostly in the form of map cracking. They are normally a few millimetres or centimetres in

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depth and usually close up when temperature differences vanish. However, they become visible again when the surface is wetted (e.g. by rain) and then dries up again; the moisture sucked into the cracks reveals their permanent existence.

More serious are thermo-cracks developed due to tensile strains caused by temperature differences across construction joints between a previously cast and a new cast section. Such cracks can be 1-3 mm and will remain permanently open. Measures should be taken to avoid, or limit the occurrence and widths of these cracks. Rational methods and automated tools are available to plan a rational curing and temperature control of thermo-cracking in concrete. The corresponding discipline is curing technology.

The hydration of cement has another feature, which may increase the tendency of early age cracking, this is the **autogenous shrinkage**, also called **chemical shrinkage**. This shrinkage strain coincides with the thermal contraction when the concrete cools after the peak temperature from heat of hydration is passed. The consequence is that the available tensile strain capacity of the young concrete during cooling is reduced, thus reducing the thermal differences, which can be resisted by the concrete without cracking. The lower the water/cement ratio and the higher the absolute cement content, the stronger is the tendency of having micro cracking due to chemical shrinkage. When combined with cooling of the hydrating cement the strain capacity of the young concrete is reduced due to chemical shrinkage, and the risk of thermo-cracking similarly increased. This is a specific feature requiring special attention when using so-called high performance concrete, see Chapter 5.9.6.

5.6.6 Frost and de-icing agents

Transition from water to ice involves an increase in volume by 9%. In the case of completely water-filled pores, this will cause splitting of concrete. see chapter 3.1.7 (2.1 and 2.2).

Owing to the fact that the volume of water increases during freezing, and that diffusion also takes place during cooling, a sufficient quantity of pores not filled with water should be available to allow the water to expand, thus preventing damage by frost. The limit value of the water content causing damage to occur is defined by the critical degree of saturation.

As the diffusion processes during freezing of the water are to some extent irreversible, the filling up of the larger pores with water will increase as the number of freezing and thawing cycles increases. This means that in certain circumstances damage by frost will occur only after a series of freezing and thawing cycles, provided that there is no possibility of (at least partial) drying of the concrete between the individual cycles.

The application of de-icing agents to a concrete surface covered with ice will cause a substantial drop in temperature at the concrete surface (temperature shock) during thawing of the ice. The difference in temperature between the surface area and the interior of the concrete gives rise to a state of internal stresses likely to induce cracking in the region of the outer layer of the concrete.

For the reasons outlined above, any frost attack should be considered to be more severe in the presence of de-icing agents. Consequently, to ensure frost resistance under these circumstances a higher content of entrained fine air voids will be required. The corresponding loss in strength shall be accounted for in the mix design.

Aggregates, which are not frost-resistant will, as a rule, absorb water that expands during freezing and destroys the cement paste. Typical indications of such processes are local spallings above larger-sized aggregates (pop-outs).

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5.6.7 Salt crystallisation

In concrete the expansive forces due to salt crystallisation near the surface will usually only cause minor problems provided the concrete is not very weak and porous. Of more importance is the chemical effect of the increased concentration near the surface of aggressive substances.

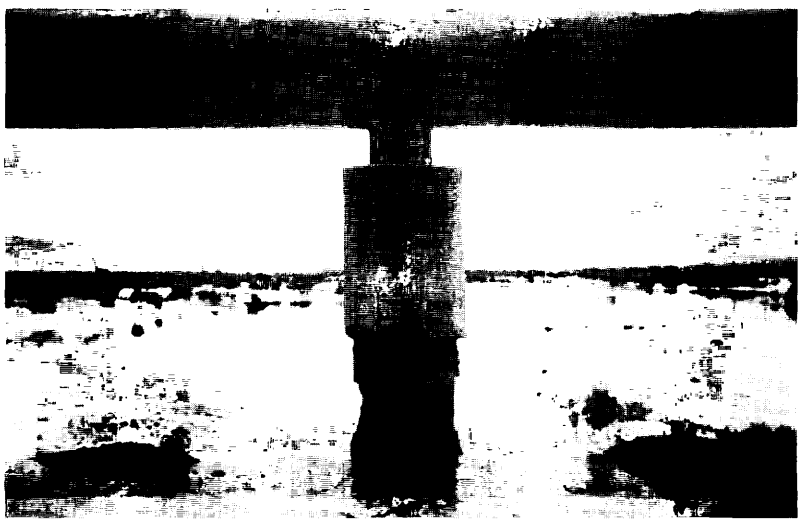


Figure 5.6-4: Salt scaling of concrete in a pile foundation carrying a pipeline. An in-situ cast top part of the pile is of better quality than the pile itself and is thus undamaged

However, in other porous materials such as sandstone, marble or masonry bursting and scaling due to salt crystallisation is a serious cause of deterioration and results in rapid deterioration of sculptures, monuments, etc. exposed to aggressive environments such as acid rain. Nevertheless, there are examples of severe scaling of concrete due to salt crystallisation, as shown on Figure 5.6-4.

5.7 Parameters influencing durability

The previous sections have illustrated how deterioration of concrete structures follows well identifiable deterioration mechanisms. These mechanisms represent the interaction between the actual environment and the structure, with its geometry and materials composition.

It has also become evident that all deterioration mechanisms depend on some aggressive substance penetration from the surrounding environment into the outer concrete layer of concrete (the cover), see Figure 5.7-1.

Mathematical modelling of deterioration mechanisms is the practical tool available for the engineer to calculate the rate and consequences of deterioration under different assumptions. These assumptions shall consider the environment on the one hand side, and the structural form, the concrete composition and the type and location of the reinforcement, together with the assumed type and frequency of maintenance, on the other hand side. Thus the expected service life of the structure can be determined under the prior assumed conditions. Later, when the structure is taken into service or has been in service for a prolonged period of time, the assumptions should be verified and an updating of the service life be made.

Although much is known today regarding scientifically sound and rational modelling of deterioration the nature of concrete and the diversities of the environment result in large variations in the behaviour of individual structures. Further research combined with factual data from the performance of real structures in service is much needed.

The main parameters governing deterioration relate to the following main elements of design:

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- The type and aggressivity of the (micro-) environment surrounding the exposed parts of the critical elements of the structure
- The transport mechanisms bringing the aggressive components of the environment to the exposed concrete surface and into the concrete
- The concrete composition in the final structure determining the type and rate of the detrimental reactions, which may occur when exposed to the environment in question
- The type and position of the reinforcement and its layout in the final structure
- The type and orientation of cracks.

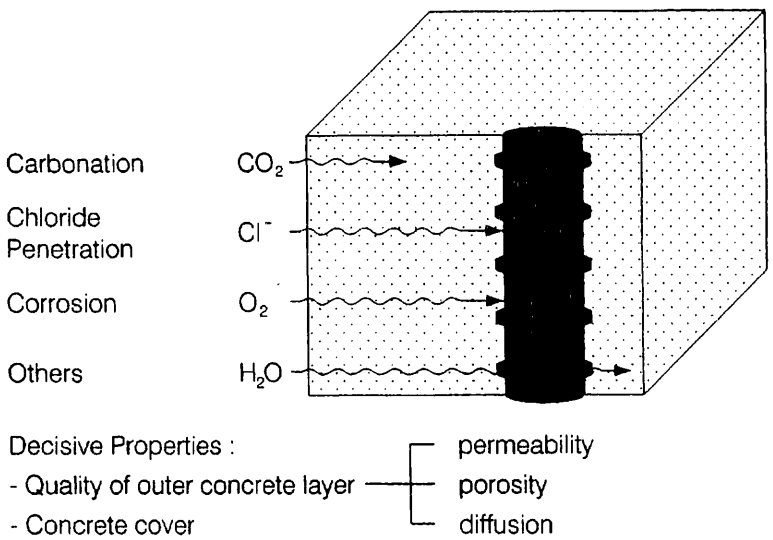


Figure 5.7-1: Importance of the outer concrete layer - or the skin of concrete - to protect the structure against deterioration

Through the service life design process the designer shall consider the above main elements of design by taking the following actions:

- Identify the specific physical actions and aggressive chemical substances, which may be expected from the environment in question during the foreseen service life
- Design the geometrical form of the exposed parts of the structure to be robust in resisting the ingress of the aggressive substance from the environment
- Select the cement type and concrete quality which would be able to resist the possible deleterious actions identified, provided the design, execution and assumed maintenance is adapted accordingly
- Select the type of reinforcement considered optimal in combination with the chosen concrete and the detailing of the structure and the reinforcement layout
- Design the concrete cover and consider crack widths and crack orientations relative to the reinforcement.

5.7.1 Identification of physical and chemical exposure

Depending on the available information on the type and use of the structure, the physical actions for which the structure shall be designed would usually be readily available, and the strength and abrasive resistance needed to resist these actions can be determined rather straight forward.

The environmental actions which constitute the true micro environment are much more difficult to determine. The reason being that the micro environment is the result of the local environment in direct interaction with, and influenced by the structure and its detailing, once the structure has been build.

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5.7.2 Geometrical form of exposed parts of the structure

An architectural design well thought of from the point of view of required long service life may lead to excellent performance of concrete structures with respect to durability and appearance. Complexity in structural form, as well as in execution and use, will usually increase the sensitivity of the structure to an aggressive environment, shorten the service life or require increased efforts in maintenance. Also ageing characteristics may deteriorate with increased complexity.

The robustness of an exposed structure or structural component is partly related to the ratio between the exposed surface area and the volume of the concrete member, or to the projected area of the facade. The larger this ration the greater is the risk of some deleterious substance penetrating locally into the concrete in sufficient quantity to initiate deterioration of the concrete or the reinforcement.

An architectural design based on precast concrete elements permanently exposed externally is illustrated in Figure 5.7-2. This solution creates large column free areas within the building but the result is leads to a large ratio between exposed surface and projected facade area. Furthermore, the large number of joints and connections collect dust and dirt which keeps water for long periods of time after rainfalls. This structure has thus a high durability risk factor.

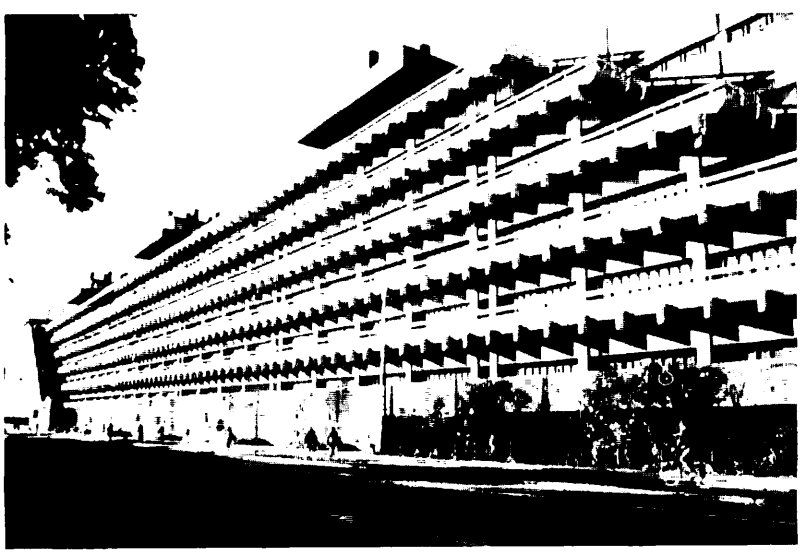


Figure 5.7-2: Architectural design of a building facade with high durability risk factor

Another architectural facade layout is pictured in Figure 5.7-3. A very smooth surface with no window sills creating shades for rain was of the facade which may cause uncontrolled run-off patterns. The drainage of rain from the window areas is very efficient, which allows quick drying out of the concrete after rainfalls. The durability risk factor of this facade is low judged from the geometric form alone.

Near outgoing edges and corners aggressive substance can penetrate into the concrete from more than one side thus leading to local concentrations. If the concrete or the reinforcement is prone to deterioration, this could lead to early development of damage at the outgoing corners and along the edges. A so-called "corner-effect" develops.

Cyclic wetting and drying effects will strongly accelerate the rate at which dissolved agents enter the concrete and concentrates near the surface of evaporation. In these cases the selection of rounded corners and edges will reduce the concentration effects and thus enhance the durability of the structure.

5.7.3 Cement type and concrete quality

The cement is the main component of concrete, which needs special attention when the concrete mix is composed.

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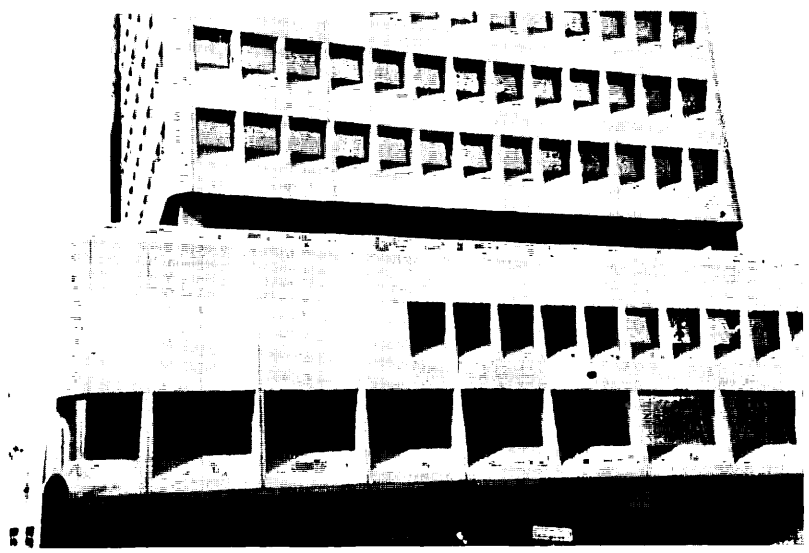


Figure 5.7-3: Architectural design of a building facade with low durability risk factor

The main differences to consider are whether a pure Portland cement based mix or a slag cement based mix shall be chosen, or whether special mineral admixtures such as microsilica and/or flyash should be introduced as blending agent to the Portland cement.

The water-binder-ratio is the most important single parameter to control in practice. It is the dominating factor controlling permeability and diffusivity of concrete.

The permeability and diffusivity of concrete gets lower, the lower the w/c-ratio, or water/binder ratio (w/b-ratio), and maximum values as low as 0.35 have been specified. Such low values also reduces or complicates the workability of the concrete, increases the risks of plastic shrinkage cracking as well as increases autogenous shrinkage (self-dissication) which increases the tendency of thermal cracking.

In highly aggressive environments the w/c-ratio (w/b-ratio) shall be low, preferably with a maximum value between 0.40 and 0.45, see Table 5.10-1.

In connection with requirements in codes and standards the values agreed upon are usually minimum values covering the large majority of cases encountered in practice. This does impose some individual evaluation to be made by the designer when faced with particularly aggressive environments, complex or sensitive structures or requirements for long service lives being beyond the service life assumed in the codes and standards.

In order to ensure a high quality concrete with good long term resistance against ingress of deleterious substance and with slow rates of degradation once the initiation period has passed, the following can be recommended regarding all other concrete components apart from the cement:

- The aggregates shall be non-reactive, when tested with approved methods, and in addition low alkali cements as well as blended cements further improve the resistance against AAR.
- Fine aggregates dredged from the sea shall be washed so chloride contamination of the concrete is avoided.
- No chloride containing additives (e.g. as accelerators) may be used.
- Seawater may never be used as mixing water.
- Under no circumstances may seawater be used as curing water on the young concrete.

5.7.4 Reinforcement

Normal reinforcement (black steel or carbon steel) as well as prestressing reinforcement will under all usual conditions be ensured sufficient protection against premature corrosion, provided the design and the execution fulfils the detailed requirements for durability.

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Nevertheless, practice has shown that apparently very well designed and executed structures sometimes locally exhibit serious premature corrosion damage. Investigations have shown that very local defects such as honeycombing, leaking through-cracks and local unintentionally small covers on single bars or links may lead to local deterioration in otherwise very well performing structures. This is part of the nature of concrete structures designed and constructed following current practice.

Possibilities to avoid or reduce the risks of such premature local corrosion defects would be to increased quality control of concrete cover, to ensure uniformly good compaction of concrete, e.g. using self compacting concrete, see Chapter 5.9.4, or use non-corroding reinforcement, see Chapter 5.10.8.

5.7.5 Concrete cover

The susceptibility of reinforcement to corrosion, together with the thickness of the concrete cover protecting the reinforcement and the quality (i.e. the permeability and alkalinity) of the cover, interact with the environment in a way which determines whether the environment is aggressive to the reinforcement or not. The values in the codes usually refer only to corrosion protection of reinforced and prestressed concrete structures. However, trading of cover (reducing cover) against reduced w/c ratio or intensified curing should be avoided or limited to avoid error-sensitive solutions. This is well illustrated in Figure 5.7-4, which shows the consequences for the service life if the concrete cover is not achieved locally, [Rostam (1996)].



Figure 5.7-4: Corrosion damage of mainly aesthetical nature due to inferior concrete cover

The modelling of the penetration of carbonation or chlorides into concrete, based on present day knowledge, can illustrate the considerable importance of the concrete cover and its quality, see Figure 5.7-5.

The surface layer of concrete, say the outer 5 to 10 mm, is especially susceptible to increased permeability caused by the increased cement paste content and locally increased water/cement ratio at surfaces cast against impermeable formwork. Similarly, trowelled or floated horizontal surfaces may be more porous than the bulk of the concrete due to the trowelling or the floating, and due to bleeding and settlement of the coarse aggregates. In such cases, any locally reduced concrete cover may reduce the durability of the structure considerably.

The permeability of concrete cover is influenced by:

- The water/cement ratio. The lower the water/cement-ratio the lower the permeability
- The type of cement and mineral admixtures. Blended cements may lead to concrete with much improved permeability compared to similar concretes based on Portland cement only
- The compaction. Honeycombing may increase the permeability of concrete to an extend, that protection of reinforcement is non-existing

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- The quality of curing. Curing determines the quality of hardening of the surface layer of concrete and controls the occurrence of thermal cracking and plastic shrinkage cracking.

The minimum distance between any concrete surface and the nearest reinforcement bar, or the nearest prestressing tendon, or the sheathing for such tendons, shall as a minimum comply with the relevant local code. However, experience has demonstrated that some individual judgement is needed when considering durability of structures in aggressive environments.

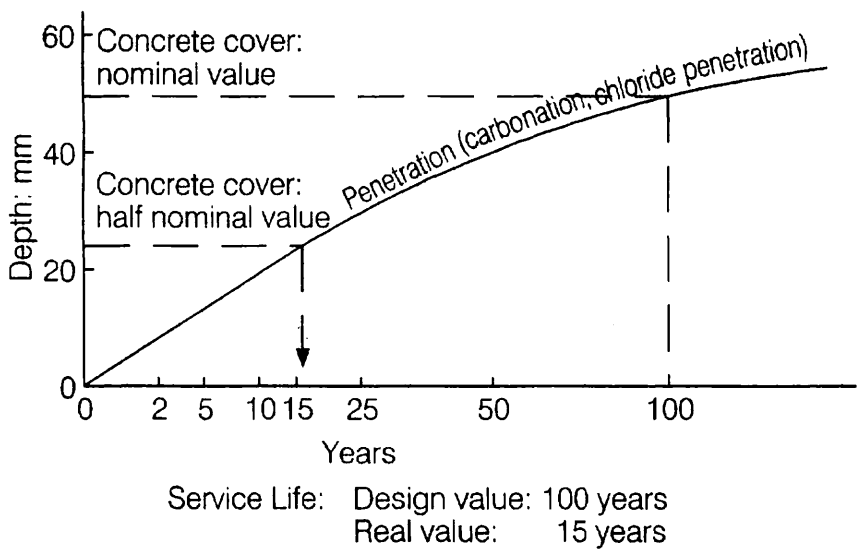


Figure 5.7-5: Time to corrosion initiation due to carbonation or chloride penetration depending on the real concrete cover as achieved in the finished structure. The example corresponds to high quality uncracked concrete

The values specified in codes are usually absolute minimum values with no downward tolerances allowed and no upward tolerance being specified. This is valid for MC 90 as follows:

The nominal values, c_{nom}, are equal to the minimum values plus tolerance according to the following rule:

$$c_{nom} = c_{min} + \text{tolerance}$$

Tolerance should be taken as 10 mm unless it in the individual case can be demonstrated that a lower value is reliably obtainable in practice. The tolerance should not be less than 5 mm. Requirements to spacers is treated in Chapter 5.9.5.

By selecting non-corrodable reinforcement the non-durability related factors listed below may be accepted to govern the selection of concrete cover. Other reasons than durability may warrant different - and sometimes maybe larger - covers such as:

- Ensuring bond strength
- Ensuring fire protection
- Using larger aggregate sizes.

5.7.6 Cracking

Cracking is inevitable in reinforced concrete structures subjected to bending, shear, torsion or tension; cracks do not indicate undue lack of serviceability or durability provided that the crack width is limited.

The fact that the calculated tensile stresses, or the calculated crack widths, do not reach the specified values does not mean that cracks will not be produced, or that those, which are produced will have widths smaller than the specified limits. The checking of the limit states of cracking should therefore be considered solely as a conventional means of obtaining a graduated system for measures of control against cracking, [CEB-FIP (1992)].

Three types of cracks may be considered:

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1 Load induced cracks, which are considered and controlled through the design process.

2 Early age cracking, i.e. thermal cracking, and plastic shrinkage cracking, which can be minimised or avoided through an appropriate pre-planned curing technology.

3 Micro-cracking, i.e. cracking occurring in the cement paste during hydration and continued hardening of the concrete. They do not necessarily reduce the durability, if small enough and randomly distributed. Microcracks are controlled through selecting an appropriate cement type and concrete mix.

In order to control load induced crack widths, and possibly to limit the number of joints in a structure, prestressing would be an option to be considered.

Normally the requirement "decompression" is sufficient to ensure that cracks due to e.g. temperature gradients within the sections not covered by direct calculations, will be limited to the surface areas of the concrete, and will not cross the prestressing steel. However, this may not be ensured in more complicated cross-sections like box girders. Temperature gradients may arise e.g. from sunlight.

Specifying small allowable crack widths can have very adverse effects on durability. The design formulae used to calculate crack widths during design will inadvertently lead the designer to think along the lines of:

- Using as small covers as allowed by the codes and specifications
- Using many small diameter bars with small bar spacing.

In addition, the designer would usually be forced to place excessive amount of reinforcement in sections with locally high bending moments, such as at the corners of frames and above intermediate supports.

All these measures are clearly counterproductive with respect to obtain long service life at low maintenance costs. Larger concrete cover of good quality concrete, and fewer but larger diameter bars - both leading to increased crack widths at the concrete surface - will result in structures being more resistant and robust against deterioration than limiting the crack widths through the traditionally used measures listed above.

Due to tensile cracks, the concrete structure will become water permeable up to a certain degree, depending on the crack width, the crack length, the hydraulic gradient, etc.

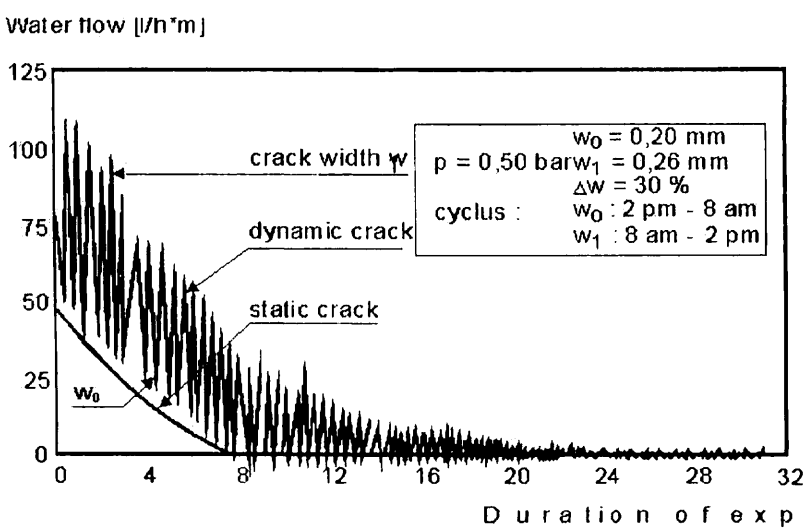


Figure 5.7-6: Selfhealing of active crack and static crack. Edvardsen (1996)

Practical experience has demonstrated that cracks have the ability to heal themselves, e.g. the water flow will be reduced with time. In extreme cases cracks can seal completely. The autogenous healing of cracks in concrete seems to be a complicated chemical/physical process. Although the term "autogenous healing of cracks" has become part of the civil engineering slang, the information about the process itself and the influencing parameters were limited. A quantification of the possible flow reduction has been lacking until recently, but for the first time

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the effect of crack healing was investigated on a large scale through both theoretical studies and verifying experiments, and reported in Edvardsen (1996), and summarised in Edvardsen (1999).

As an example taken from Edvardsen (1996), Figure 5.7-6 presents the water flow through an active crack (crack motion corresponding to a one-day cycle) compared with the water flow through a dormant crack with crack width equal to the minimum width of the active crack. The water pressure was the same in both cases. The figure shows that an autogenous healing also occurs in active cracks.

Early age cracking such as plastic shrinkage cracking and thermal cracking, and drying shrinkage cracking can be controlled, and often avoided, if using steel fibre reinforcement. However, other measures could be more cost effective, e.g. appropriate choice of cement, concrete mix, curing, cooling of the hardening concrete, membranes, and specific coatings.

Plastic shrinkage cracking should not be accepted as this reflects only insufficient curing which also would have other adverse effects. A small content of polypropylene fibres, say 0.5 kg/m^3 may reduce the risks of plastic shrinkage cracking.

The importance of cracking related to the durability of a structure may be critically influenced by whether or not cracks are longitudinal, i.e. follow the line of the reinforcing bars

This is especially important from the point of view of reinforcement corrosion, but in addition bond and shear strength could be seriously reduced by the development of longitudinal cracks.

The risks of obtaining cracking along the lines of some reinforcing bars are high; transverse reinforcement is particularly at risk, especially in cases where it has a lower concrete cover than the main bars, such as stirrups in beams.

5.8 Environmental aggressivity

With respect to durability one of the most important decision to be taken by the designer is the determination of the exposure conditions for which each member of a structure shall be designed. Due to the dominating influence of the micro-climate on the type and severity of the environment different parts of a structure may be categorised in different exposure classes. Obvious examples are the submerged, the tidal, the splash and the atmospheric zones of a marine structure, but also different geographic orientations (north/south/east/west, or seaward/landward orientation) may warrant different categorisations with respect to exposure classes. Even very local differences can be taken into account such as vertical faces, horizontal surfaces facing upward (risk of ponding) or downward (protected against wetting thus with risk of increased rate of carbonation).

The structure itself has decisive influence on the future micro-climate to be expected. This emphasises the need for a conscious evaluation by the designer of the structure/environment interaction depending on the chosen structural form and geometric detailing of the different exposed members.

Clearly, it is not easy to decide on the conditions of exposure of particular elements in particular environments. However, the factors listed below are known to have a dominating influence on the aggressivity of a particular environment:

- Water or moisture
  Aggressive substances, which may be present in moisture, are:
  (a) Carbon dioxide - necessary for carbonation
  (b) Oxygen - necessary for corrosion
  (c) Chlorides - promote corrosion

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(d) Acids - dissolve cement
(e) Sulphates - give expansive reaction with cement
(f) Alkalis - give expansive reaction with aggregate.

- Atmosphere
  Aggressive substances, which may be present in the atmosphere, are:
  (a) Carbon dioxide - necessary for carbonation
  (b) Sulphur - promote neutralisation of concrete

- Temperature
  Aggressive effects of temperature may be the following:
  (a) Without frost - a main deterioration rate determining factor
  (b) With frost - temperature variations determine type of frost damage

5.8.1 Water and moisture

As all deterioration processes, apart from most types of physical deterioration, require the presence of water, see Chapter 5.6.1, it is the moisture condition in the concrete rather than that of the surrounding atmosphere, which is decisive for the durability. Under steady conditions moisture will be constant but under varying external moisture conditions concrete takes water up from the environment more rapidly than it loses it and so the internal average humidity tends to be higher than the average ambient humidity.

This principle holds particularly true where members are subject to wetting and drying, e.g. frequent wetting, as in tidal regions can maintain concrete in a saturated condition.

Table 5.8-1 indicates the influence of the effective humidity on various processes related to durability. The table is only a very rough indication, but it highlights the fact that there are no chemical or corrosion problems in very dry concrete, and that there are no serious corrosion problems in water saturated concrete. The main corrosion problems occur at relative humidity inside the concrete within the range of 75 - 98 %. Chemical attack of the concrete and frost damage is more serious the more the concrete is wetted, [CEB (1992)].

Effective relative humidity	Process*				
	Carbonation	Corrosion of Steel		Frost Attack	Chemical Attack
		In carbonated concrete	In chloride contaminated concrete		
Very low (<45%)	1	0	0	0	0
Low (45-65%)	3	1	1	0	0
Medium (65-85%)	2	3	3	0	0
High (85-98%)	1	2	3	2	1
Saturated (>98%)	0	1	1	3	3

* 0 = insignificant risk; 1 = slight risk; 2 = medium risk; 3 = high risk.

Table 5.8-1: Rough indication of the influence of moisture state on durability processes [CEB (1992)]

5.8.2 Atmosphere

For a given atmospheric environment the water content in concrete will be higher when chlorides are present, due to their hygroscopic effect. This accounts, for example, for the heavy corrosion encountered with chloride-contaminated concrete placed indoors in perma-

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nently air-conditioned rooms where temperatures are average (20°C) and relative humidity is low (50-60%), such as in hot, humid and saline environments.

Acid rain has a negligible ability to dissolve concrete in spite of the, at times, very low pH values encountered in practice. The reason being the large buffering capacity of the alkaline concrete neutralising the acid rains. This also holds true for carbonated concrete. Only in cases with very porous or lean concrete the effects of acid rain may with time become visible just as an exposure of the aggregates.

5.8.3 Temperature

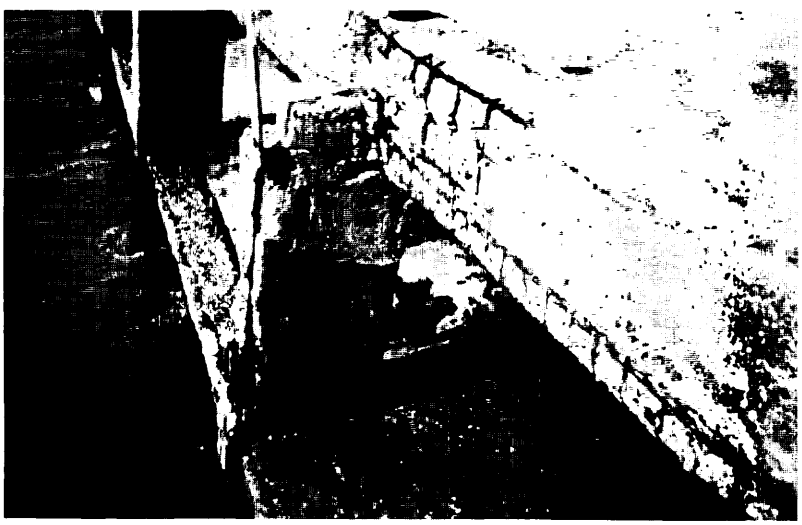


Figure 5.8-1: Reinforced concrete jetty in the Gulf exhibiting extensive chloride induced corrosion damage already after 2-3 years, and having reached a stage of failure and collapse after 7.5 years when this picture was taken

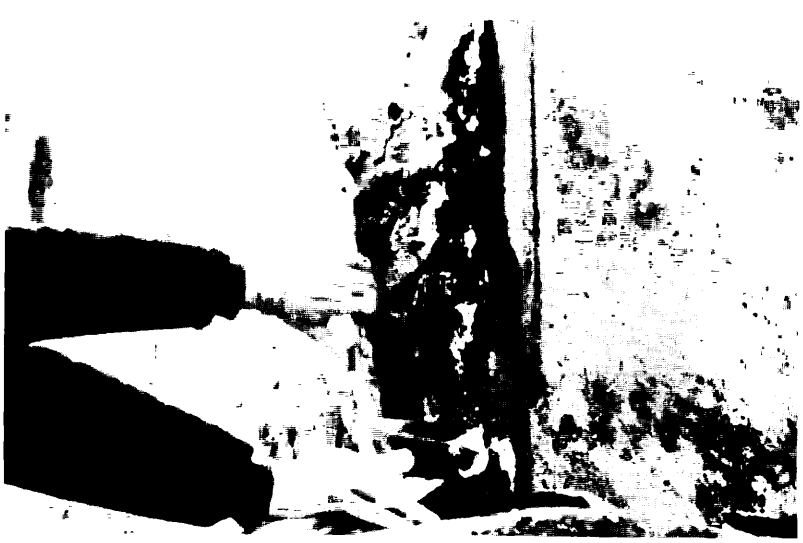


Figure 5.8-2: Bridge piers exposed for 18 years to a temperate marine environment. Extensive damage in the splash zone due to chloride induced reinforcement corrosion

For temperature levels above freezing the temperature is a decisive factor regarding the rate of deterioration. Chemical and electrochemical reactions are accelerated by increases in temperature. A simple rule-of-thumb is that an increase in temperature of 10°C causes a doubling of the rate of reaction. This factor alone makes hot and tropical environments considerably more aggressive than temperate climates. This effect of temperature can be clearly demonstrated by comparing the picture in Figure 5.8-1 with the picture in Figure 5.8-2. In the former case the average yearly temperature is approximately

30°C higher than in the latter case, which would lead to a 2x2x2=8 times faster deterioration in the Gulf compared to the rate of deterioration in the Nordic Countries. The pictures are clear documentation of this dependency of the temperature. This has important implications on design strategies and protective measures, which shall be applied under different climatic conditions, in addition to the effects of the type of aggressive medium represented in the environment

For temperature levels leading to freezing of concrete, see Chapter 5.6.6.

Air entrainment of the concrete can provide adequate resistance against frost damage. However, it shall be noted that dense and impermeable concrete such as high performance

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concrete may be difficult to have sufficiently air entrained, and particular care shall be taken to ensure and verify adequate frost resistance of such concrete in each individual case.

5.8.4 Classification of environmental exposure

One important observation is needed when defining exposure classes. That is to relate the exposure to the type and severity of deterioration that may result from the exposure. Hence, detailed knowledge of the governing deterioration mechanisms is needed in order to define exposure classes. In this respect a differentiation is needed between mechanisms deteriorating the concrete and mechanisms leading to reinforcement corrosion.

The designer may find valuable help in defining the design basis for the exposure of different elements of his structure from the classifications presented in different national or international codes or design guidelines.

One such guideline for exposure classes is the recent European Standard [CEN (1999)]. This is an updated version compared to the exposure classes presented in MC 90. This Standard specifically differentiates between exposures leading to reinforcement corrosion and exposures leading to deterioration of the concrete. This recent classification is presented in Table 5.8-2.

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: All exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: Very dry	Concrete inside buildings with very low air humidity.
2 Corrosion induced by carbonation		
Where concrete containing reinforcement or other embedded metal is exposed to air and moisture, the exposure shall be classified ad follows: NOTE: The moisture condition relates to that in the concrete cover to reinforcement or other embedded metal but, in many cases conditions in the concrete cover can be taken as reflecting that in the surrounding environment. In these cases classification of the surrounding environment may be adequate. This may not be the case of there is a barrier between the concrete and its environment.		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside building with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2

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Class designation	Description of the environment	Informative examples where exposure classes may occur
3 Corrosion induced by chlorides other than from see water		
Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from sea water, the exposure shall be classified as follows:		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water, the exposure shall be classified as follows:		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5 Freeze/thaw attack		
Where concrete is exposed to significant attack by freeze/thaw cycles whilst wet, the exposure shall be classified as follows:		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agent	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation, with de-icing agent or sea water	Road and bridge decks exposed
6 Chemical attack		
Where concrete is exposed to chemical attack, which occurs in natural soils and ground water as given in table 2, the exposure shall be classified as given below. The classification of seawater depends on the geographical location; the classification valid in the place of use of the concrete applies. Note: A special study may be needed to establish the relevant exposure condition where there is: • limits outside of table 2; • other aggressive chemical; • chemically polluted ground or water; • high water velocity in combination with the chemicals in table 2.		
XA1	Slightly aggressive chemical environment according to table 2	

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Class designation	Description of the environment	Informative examples where exposure classes may occur
XA2	Moderately aggressive chemical environment according to table 2 or exposure to sea water	
XA3	Highly aggressive chemical environment according to table 2	

Table 5.8-2: Exposure Classes for concrete structures according to prEN 206-1:1999-24

The aggressive chemical environments classified below are based on natural soil and ground water at water/soil temperatures between 5 C and 25 C and a water velocity sufficiently slow to approximate to static conditions.

The most onerous value for any single chemical characteristic determines the class.

When two or more aggressive characteristics lead to the same class, the environment shall be classified into the next higher class, unless a special study for this specific case proves that it is not necessary.

Chemical characteristic	Reference test method	XA1	XA2	XA3
Ground water				
SO_4^{2-} mg/l	EN 196-2	≥ 200 and ≤ 600	> 600 and ≤ 3000	> 3000 and ≤ 6000
pH	ISO 4316	≤ 6,5 and ≥ 5,5	< 5,5 and ≥ 4,5	< 4,5 and ≥ 4,0
CO_2 mg/l aggressive	prEn 13577	≥ 15 and ≤ 40	> 40 and ≤ 100	> 100 up to saturation
NH_4^+ mg/l	ISO 7150-1 or ISO 7150-2	≥ 15 and ≤ 30	> 30 and ≤ 60	> 60 and ≤ 100
Mg^{2+} mg/l	ISO 7980	≥ 300 and ≤ 1000	> 1000 and ≤ 3000	> 3000 up to saturation
Soil				
SO_4^{2-} mg/kg [1] total	EN 196-2 [2]	≥ 2000 and ≤ 3000 [3]	> 3000 [3] and ≤ 12000	< 12000 and ≤ 24000
Acidity ml/kg	DIN 4030-2	> 200 Baumann Gully	Not encountered in practice	

[1] Clay soils with a permeability below 10^{-5} m/s may be moved into a lower class

[2] The test method prescribes the extraction of SO_4^{2-} by hydrochloric acid; alternatively, water extraction may be used, if experience is available in the place of use of the concrete

[3] The 3000 mg/kg limit is reduced to 2000 mg/kg, where there is a risk of accumulation of sulphate ions in the concrete due to drying and wetting cycles or capillary suction.

Table 5.8-3: Limiting values for exposure classes for chemical attack from natural soil and ground water, as classified in Table 5.8-2, according to prEN 206-1: 1999-24

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5.9 Execution and workmanship

The execution stage constitutes the most vital step in achieving the required quality of concrete structures. From a durability point of view the essential part of the execution process is to ensure the correct concrete quality and thickness of the concrete cover - it is as simple as that!

The following two issues relate to the resistance of the concrete cover to ingress of aggressive substance from the environment:

- The compaction, in order to ensure uniform, dense and strong covers.
- The type and duration of curing, i.e. controlling moisture levels and temperature differences in order to limit or avoid early age cracking.

In addition, the following two recent developments are expected to obtain considerable influence on the quality and service life of future exposed structures:

- Permeability controlled formwork liners (PFL).
- Self compacting concrete (SCC)

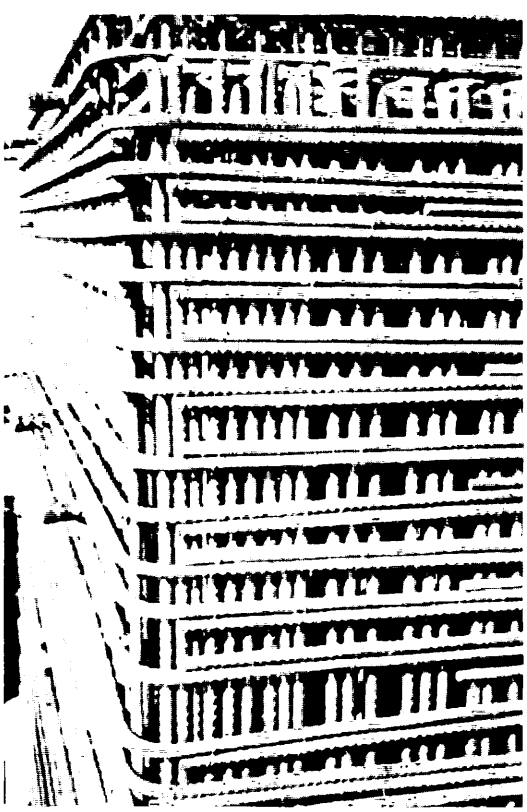


Figure 5.9-1: Example of reinforcement detailing which does not respect the need for reliable casting and compaction of concrete. Particularly the quality of the concrete "skin" is in danger

Means of providing the required thickness of the concrete cover with spacers is an integral, but often neglected part of ensuring durable structures

Because the execution phase itself has dominating influence on the final quality and performance of the structure the on site supervision and quality control is an essential part of the construction process.

Prestressing and prestressed concrete structures pose specific execution problems not dealt with in this chapter, but reference is made to the valuable references such as Concrete Society (1994).

5.9.1 Compaction of concrete

Adequate compaction of the concrete in the cover may be difficult to achieve due to the imited space and the need for the cover concrete to be moved through the outer layer of reinforcement. This movement of the concrete may cause "sieving" of the concrete if the spacing of the reinforcing bars is small or the concrete is stiff. When designing the reinforcement layout the realistic ability to compact the concrete on site shall be respected. Figure 5.9-1 illustrates a situation where this has not been achieved, [HETEK 1-2 (1997)].

The compaction by vibrators shall be achieved through vibrating the concrete inside the reinforcement cage. Experience has shown that vibrating directly in the cover concrete may lead to inferior compaction of the concrete, e.g. by leaving porous traces from the vibrator. The problem requires particular attention in the case of dense high performance concretes with mineral admixtures due to the tixotropy of such low water/cement ratio concretes with high amounts of plasticiser and superplasticiser.

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The use of external form vibrators to compact the concrete, and in particular the cover concrete, is also a questionable procedure. The reason being that the entrapped air voids in the concrete tend to move towards the location of the vibrator and accumulate to form porous concrete. With correct use of poker vibrators such air bubbles are extracted from the concrete when slowly lifting the vibrator head out of the concrete.

5.9.2 Curing

Curing of the concrete is part of the hardening process, which ensures an optimal development of the fresh, newly cast concrete into a strong, impermeable and durable hardened concrete in the cover zone free from plastic shrinkage and thermal cracks. During this initial stage of the life of the concrete, it is necessary to:

1. Use an appropriate hardening process. Casting must be planned such that the required strength at the time of form stripping is achieved.
2. Ensure against damage from drying. Premature drying-out of the concrete surface should be avoided, as this may lead to large plastic shrinkage cracks.
3. Ensure against damage through early freezing. The concrete must not freeze until a required minimum degree of hardening has been achieved.
4. Ensure against damage from thermal stresses. Differential movements due to thermal differences across the section or across a construction joint between hardened and newly cast concrete should not lead to cracks.

The increased sensitivity to too early drying out of some types of cement and concrete (composite and blended cements; chemical and mineral admixtures) has accentuated the need to develop simple and rational heat and moisture curing procedures. Several comprehensive tool are now available to design the appropriate types of moisture and heat curing (References!!!)

According to experience with Portland cement concrete, it is recommended to stay within the following limits for temperature stresses:

- a maximum of 20 °C temperature difference over the cross-section during cooling after stripping
- a maximum of 10 - 15 °C difference across construction joints and for structures with greatly varying cross-sectional dimensions.

Concretes with very low water/cement ratio, such as high performance concrete, may be particularly sensitive to heat and temperature control during the hardening process, and values for temperature differences should be lower - and at times considerably lower - than indicated for ordinary Portland cement concrete above.

The heat balance to be controlled is sensitive to changes in the selected level of insulation. In practice it is often necessary to decide at short notice whether to strip formwork or whether possible additional or reduced insulation of a hardening cross-section has to be made.

In short, good curing is needed to profit from a good concrete mix. Bad curing destroys an otherwise good concrete mix. And good curing cannot compensate for a bad concrete mix. All efforts to ensure an optimal heat and moisture curing may be in vain, if the initial quality of the concrete mix is inferior.

In practice temperature profiles can be calculated, and the whole curing process can be designed prior to casting the concrete, see among others [RILEM (1994)] and [HETEK 3-5 (1997)].

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5.9.3 Permeability controlled formwork liner (PFL)

Recognising the dominant influence of the quality of the outer concrete skin to protect the concrete as well as the reinforcement against penetration of aggressive substance has lead to the development of a special permeable formwork liner, which can be either flexible and tissue-like, or stiff like a plastic board. Such liners are able to improve the quality of the outer few cm or mm of the concrete cover by controlling the water/cement content and enhancing the curing in this thin outer zone.

Using PFL results in some of the excess water in the surface layer being temporarily absorbed by the liner thus reducing the water/cement ratio. When then hardening takes place the hydrating cement in this layer would need some additional water for optimised hardening, and this water is then readily available in the liner. Hence, an optimal moist or water curing takes place in this outer layer, and an improved quality of the cover is obtained.

The liner leaves a characteristic tissue like pattern on the surface of the concrete and requires therefore that care shall be taken when placing the liner on the formwork. Particular care shall be taken with the flexible tissue type liner as they tend to fold during casting and compaction if not correctly fixed to the form, and such folds will be permanently visible on the future concrete surface.

Numerous tests and practical experience have demonstrated the beneficial effects, and from a technical point of view this technology could be considered one of the most durability enhancing achievement within the past couple of decades. The main drawbacks of this system are the difficulties in fixing the lining to the formwork so attractive appearance of the concrete surface is ensured after stripping the form, and the relative high costs of the products.

PFL can improve the quality of the outer part of the concrete cover, but cannot compensate for local bad compaction and honeycombs. However, the new developments within self-compacting concrete could be a realistic solution to this problem, and thus a combination of PFL with SCC would have a valuable synergy effect in improving the durability of concrete structures.

5.9.4 Self-compacting concrete (SCC)

The need for durable concrete in an aggressive environment leads automatically to concrete with optimised mix composition and a low water/cement ratio. Such concrete is difficult to compact and the risk of honeycombs, particularly in the cover concrete, increases. In addition, the quality and efficiency of compaction is extremely dependent on the individual person handling the vibrator. Hence, the better the concrete mix is, from a durability point of view, the greater becomes the risk of having inferior or bad execution leading to reduced quality in the final structure. This inconsistency is due to the dominating influence of the execution process on the final quality and performance of the structure regardless of the good quality of the initial concrete mix.

The development of a concrete mix where the placing and compaction has minimal dependence on the available workmanship on site would therefore improve the true quality of the concrete in the final structure. This has been a main driving force in recent year's development of SCC. With the aid of a range of chemical admixtures and optimal grading of the aggregates concrete with low water/cement ratio can be made to flow through complicated form geometry and around complex reinforcement layout, without segregation. The form can be filled and a uniform compaction without honeycombs can be achieved, also in the cover zone of the concrete, with no or only minimal additional contribution to the compaction and levelling of the concrete from the workforce on site. The flowing concrete will exert an increased pressure on the form, which shall be taken carefully into account when designing the formwork.

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The use of SCC is also an environmentally friendly technology as the noise level from vibrators is nearly eliminated and the concrete workers need only minimal work with the vibrators, with all the adverse effects vibrating concrete has on the body.

The main current drawback with this technology is the sensitivity of such concrete to the precise dosing, mixing and transporting of the concrete and the dependence on the weather conditions while casting the concrete. Under adverse conditions the ability to flow may suddenly be lost. In addition, the increased cost of such concrete is noticeable, and the demand for expertise and experience is moved nearly fully to the mixing plant.

5.9.5 Spacers

The minimum concrete cover specified for a design is usually the value used to design the expected service life based on assumptions regarding the penetration of de-passivating and corrosive substance to the reinforcement. Therefore this minimum value shall be ensured in the final structure by taking the relevant tolerances into account in the selection of type, dimension and spacing of spacers.

Spacers shall be designed according to c_{nom} and not to comply only with c_{min} , see Chapter 5.7.5. This fact is often overlooked in practice!

The spacing of spacers shall be adequate to ensure the required cover considering the dimensions, i.e. stiffness, of the reinforcement and the fresh concrete pressure on the reinforcement, which can be expected during casting and compacting the concrete.

When controlling concrete cover after placing and hardening of concrete the measured values may not be less than c_{min}.

In aggressive environments spacer material should preferably have good adhesion to the concrete. The spacer geometry and fastening shall ensure good and stable positioning of the reinforcement.

The concrete cover in aggressive environments shall usually be minimum 50 mm according to several codes and standards. However, the special aggressive conditions like in tidal and splash zones would lead to larger covers in the above-water zone. Values of 75 mm have become normal for long life marine structures (100-120 years service life).



Figure 5.9-2: Reinforcement corrosion initiated by aggressive substance penetrated through the void along a plastic spacer

The spacer material shall have good bond to the concrete and shall have similar hygro-thermal deformation characteristics as concrete. In this respect Plastic spacers are not compatible with the surrounding concrete, in the sense that they have no direct adhesion and that they have different temperature coefficients (factor ten) than concrete, and furthermore age under exposure to air, sun and marine environment, see Figure 5.9-2. In aggressive environments high quality concrete spacers shall be the preferred option and it is important to ensure that the

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spacers are of the same high quality as required for the structural concrete itself. Hence, high quality concrete spacers shall be used in exposed areas.

5.9.6 High performance concrete (HPC)

The continuous demand for increased strength and improved durability and performance of concrete structures has led to the development of HPC. This development has had three main objectives in mind:

1. Protect the reinforcement against corrosion; in particular provide protection against ingress of chlorides by creating dense impermeable concrete in the cover zone with very low penetrability of aggressive ions such as chlorides, sulphates and CO_2.

2. Resist deterioration of the concrete itself when exposed to the aggressiveness of the environment such as sulphates, seawater and other chemical attacks, as well as resist freeze/thaw attack.

3. Provide adequately high strength to fit the structural requirements.

This development has been very successful in many respects. The advanced HPC products available have met very complex and demanding structural challenges today.

HPC for normal type structures will usually have a relatively high cementitious binder content (blended cements), a low water/cement ratio, say in the range of 0.35 - 0.40, and a high contents of plasticiser and superplasticiser. Such concrete can conveniently be used for bridges, marine works, offshore structures, high rise buildings etc. where the strength requirements usually remain within the range of say 50 - 80 MPa.

One drawback has been that these more refined concrete mixes become more sensitive towards the actual handling during execution. They set high demands on the competence, experience and workmanship of the workforce. To varying degrees these concretes differ from the long term known types of structural concrete through the following:

- The dosing and mixing becomes more complicated and sensitive, even with respect to timing and sequence of adding the different ingredients.
- The placing requires special methods and routines as such mixes are rather tixotropic and sticky.
- Compaction is more demanding as the vibration shall be more intensive and requires more vibration energy.
- The denseness of the concrete, when correctly placed and compacted eliminates bleeding, and the available water is so limited, that protection against evaporation shall be introduced promptly following levelling and trowelling of horizontal surfaces exposed to drying, in order to avoid plastic shrinkage cracking.
- The high content of cementitious material will most often generate more heat than normal concrete, however depending on the cementitious material used in the individual cases.
- The autogenous, or chemical shrinkage is more pronounced due to the low water/cement ratio, and this sets much stronger demands on controlling the temperature differences in hardening concrete if thermal cracking shall be avoided. The limiting temperature differences that are stated in Chapter 5.9.2 might have to be halved, and in extreme cases reduced to 1/3.
- HPC requires air entrainment to be frost resistant according to the generally adopted (rather severe) freeze-thaw tests. However, the mix with the high contents of superplasticiser is usually difficult to air entrain, and the air may easily disappear during the compaction due to the increased vibration energy needed. Hence HPC may be more sensitive to Freeze-thaw actions, i.e. less frost resistant than normal type structural concrete.

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As stated, the sensitive elements of HPC listed above are not necessarily valid for all types and uses of such concretes. It is however necessary to have these potential problems in mind when designing structures based on the application of HPC, taking the realistically available local workforce into account when making the selections.

One new issue has evolved during the last few years, which may influence the broad - but not always successful - application of HPC. This is related to the increased availability and competitiveness of non-corrodable reinforcement bars to avoid reinforcement corrosion in heavily chloride containing environments, see Chapter 5.10.8. The non-corrodable reinforcement bars are made from either glass, aramid or carbon fibres, or galvanised reinforcement, epoxy coated reinforcement or stainless steel reinforcement.

5.9.7 Supervision versus quality assurance and quality control

A quality assurance procedure (QA) shall be followed, in order to document the real quality obtained in the structure through the actual construction process. This shall also include a factual documentation of durability related parameters obtained, based on selected tests being made on the finished structure and included in the Birth Certificate (Chapter 5.9.8) part of the future Operation and Maintenance Manual to be handed over to the owner.

The main value of a quality assurance system enforced on the site operations is to motivate the Contractors workers to exert their true skills when doing their professional work. Bad quality is not produced wilfully but is the result of a lack of awareness of what is god and what is not god, and a lack of knowledge by the people on site. Hence, a formalised system provides one part of a motivating factor in improving quality and workmanship. On site guidance and supervision by competent personnel is another important part of quality improving measures to improve quality. Finally, quality control (QC) and the physical on site supervision is a further quality enhancing measure, which shall be adapted to the real nature and possibilities of the construction industry.

5.9.8 Handing-over situation: "Birth Certificate"

In order to document the fulfilment of the design specifications, and verify the subsequent performance, the Quality Assurance documentation for design and execution should be enlarged to include information gathered during the operation and use of the structure. Developing an **Operation and Maintenance Manual** specific for each structure can do this. This Manual should be prepared by the Designer and shall include all information from the structural design and the construction being relevant for the future inspections and maintenance. This Operation and Maintenance Manual shall also include recommendations regarding type and frequency of future inspections and should highlight possible sensitive or critical parts of the structure which are assumed beforehand to need particular attention during use.

When the structure is handed over to the Owner, the initial Operation and Maintenance Manual will constitute a "**Birth Certificate**" of the structure, [Rostam (1999)]. Information from future inspections and all other relevant events such as accidental impacts are then filled in as they occur. Depending on the nature and contents of such future information the type, frequency and selected special areas of concern shall be revised or updated by the Owner following his needs at that time.

5.10 Service life design

The requirements of national codes and standards will usually have been satisfied through the traditional structural design procedure. This shall satisfy the Ultimate Limit State requirements as well as the codified Serviceability Limit State requirements.

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If the codes or standards do not specify the target service life, this needs to be specified by the owner or fixed some way or other in order to form the basis of the service life design.

When the required target service life has been fixed, there are an additional number of special design issues, which should be considered also, such as:

- Structural layout and geometric form
- Structural system
- Location of expansion joints and construction joints
- Execution methods
- Environmental aggressivity
- Decisive deterioration mechanisms
- Selection of materials and concrete mix
- Concrete cover
- Maximum design crack widths
- Special protective measures
- Placing of installations and equipment

Detailing of several of these design issues lie outside the scope of the present chapter, but are equally important within the overall design and construction of durable concrete structures, see [CEB (1992)] and [Markeset and Rostam (1999)].

The service life design procedure will differ substantially whether a new structure is to be designed or whether the residual service life of an existing structure is to be determined and possibly prolonged through rehabilitation or upgrading, [Rostam (1991)].

In the following a separation is made between the target service life design of new structures, treated in this Chapter, and the residual service life assessment of an existing structure, treated in Chapter 8.

5.10.1 Target service life design of new structures

The end of the target service life for principal structures and their main structural components is usually defined as the end of the initiation period and onset of the identified critical deterioration mechanism(s). However, for some types of structures a part of the propagation period may be taken into account already at the design stage. Examples could be the robust parts of harbours and marine structures risking reinforcement corrosion when safety and performance is not jeopardised by some initial corrosion, cracking and maybe spalling.

For non-structural elements the end of their service life indicates only the time until renewal or replacement is necessary before their reduced performance or malfunction creates secondary damage to the principal structure. This refers to, among others, re-doing of coatings, replacement of drainage systems, bearings, joints or wearing courses.

Typical examples of expected - or specified - service lives for structures and main structural elements have been:

- 35 years: Off shore structures
- 50 years: Structures designed according to MC 90; Eurocode 2; National Codes.
- 100 years:Bridges, tunnels, harbours
- 200 years:Storm surge barriers

As stated in Chapter 5.5 the service life depends equally on the behaviour of structural and non-structural elements. Both shall be considered during design, construction and use of the structure.

Typical examples of accepted service lives of non-structural components are:

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- 10 years: Surface coatings
- 25 years: Drainage, bearings, expansion joints, wearing courses
- 50 years: Membranes, linings

When designing for durability, the concept of the service life is generally used, as defined in Chapter 5.3. Therefore, in order to formulate durability requirements it is necessary to identify the duration for which the structure is intended to perform satisfactorily without unforeseen high costs for inspection, maintenance and repair. If this duration - or target service life - is not fixed through existing codes or other requirements, it is an issue, which shall be settled in a dialogue between the owner and the designer - with the owner taking the final decision.

The economic consequences - in the form of a life cycle costing and its reliability - of selecting alternative solutions shall be one of the parameters governing this decision. First when this has been settled will the designer be able to select the structural layout, the material composition, the type and placing of the reinforcement, the execution methods, and an accepted frequency and intensity of inspections and foreseen maintenance, the latter to be agreed separately with the owner.

This process is essential in order to ensure the full awareness of the owner regarding the consequences of the different alternatives and the consequences of his final decisions.

5.10.2 Interaction between design and execution

Already at the design stage possible means of construction shall be considered and fixed, as this will influence the durability design.

It is important that this interaction between the foreseen execution and the provision to provide durable structures is identified at an early stage of the design in order to optimise the design and prepare the structure for easy inspection and maintenance.

In short, the structure shall be designed so it is:

- constructable (buildable)
- accessible
- inspectable
- maintainable
- repairable

In view of recent years intensified focus on all aspects of environmental impacts, the list could be completed by adding:

- replaceable
- demolitionable
- recycleable.

The latter aspects, though, being outside the scope of the present text.

5.10.3 Basic design strategies

The first and foremost task is to identify and quantify to the extent possible the type and aggressivity of the environment in which the structure is to perform during its service life. This will then clarify which type of deterioration that could be a threat to the future performance and service life of the structure. The design strategy will then be to create a structure, which has sufficient resistance towards the identified environment and its resulting threats of deterioration.

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In principle there are two basic design strategies which can be followed, [Rostam and Schiessl (1994)]:

A: Eliminate the risk of the degradation reactions considered.

B: Select optimal material compositions and structural detailing to resist the degradation reaction considered.

Strategy A can be subdivided into three alternative means of satisfying the requirement of eliminating the risk of deterioration due to the identified mechanisms:

A1: Change the environment on the exposed concrete surfaces, e.g. by tanking, membranes, coatings etc.

A2: Select non-reactive materials, e.g. stainless steel reinforcement, coated steel reinforcement, non-reactive aggregates, sulphate resistant cements, low alkali cements etc.

A3: Inhibit the reaction, e.g. cathodic protection or cathodic prevention. The avoidance of frost attack by air entrainment is also classified in this category.

Strategy B represents different types of design provisions. For example corrosion protection could be achieved by selecting an appropriate cover and a suitable dense concrete mix. In addition, the structure can be made more resistant against aggressive environments of different sorts by appropriate detailing such as minimising the exposed concrete surface, by rounded corners, and by adequate drainage.

Modelling of deterioration processes is only applicable for strategy B. An outline of a procedure for strategy B could be the following:

- The ideal procedure would start with the definition of the performance criteria related to the environmental conditions to be expected.
- The next important element is the realistic modelling of the actions (environment) and the material resistance against these actions.
- Based upon the performance criteria performance tests are indispensable for quality control purposes. The performance tests must be suitable both to check the potential quality of the material under laboratory conditions and, even more important, the in situ quality.
- Based upon all these elements the design procedure can be established.

5.10.4 Level of modelling for service life design

When designing the structure for its intended target service life the detailing of the modelling required in each case should be decided, as this will govern the whole design as well as the possibilities of later updating of the service life design, [CEB (1997)].

Three different levels of modelling the environmental actions and the transport and deterioration mechanisms can be identified. The level to be selected in the individual case depends on the type and detailing of information available and on the precision - or reliability - needed from the service life design. The three levels are:

- *Macro level design:* Design based on the simple "deem-to-satisfy" rules presented on a structural engineering level. This is the approach most generally used at present, where requirements have been presented as prescriptive values in the form of single values, tables or graphs. Examples are requirements to minimum cover, cement type, minimum cement content, maximum water/cement ratio, and curing time.

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Applying the macro level design for durability does not require explicit demands regarding service life. Nor does it provide any information on the expected service life, other than the general assumption related to the source of the requirement, e.g. 50 years for structures designed according to most national codes of practice.

- *Meso level design:* Design based on modelling of the micro environment and the deterioration mechanisms formulated on a "materials engineering level". This modelling takes into account variations in concrete quality locally in the structure.

  Applying the meso level design for durability requires information on the target service life. The calculations can be made on both a deterministic and a probabilistic basis.

- *Micro level design:* Probabilistic service life design based on detailed modelling on a "materials science level" of the micro environment and the identified deterioration mechanisms.

  Applying the micro level design for durability requires detailed information on the micro environment to be expected for the individual parts of the structure, and detailed information on the specific materials being used in the individual case. In addition, this approach is only warranted if followed up by an updating of the service life design calculations. This updating shall be using data from the structure and the concrete after the construction has been finished, e.g. as part of the Birth Certificate provided at handing over the structure from the contractor to the owner. This will also allow the owner to update the service life forecast at will.

As an example of deem-to-satisfy rules on macro level design, the European Code, [CEN (1999)], has recommended the limiting values for composition and properties of concrete as listed in Table 5.10-1.

The Eurocode 2 for concrete structures to which CEN (1999) belongs, has an á priori assumption of ensuring a 50 year service life of structures designed according to this code and exposed to environments which have been classified in accordance with the exposure classes identified in Table 5.8-2.

If a longer service life is required, or a more aggressive environment is to be considered, then a more in-depth service life design shall be made.

5.10.5 Multi-stage protection approach

A design strategy on meso level and following strategy B (Chapter 5.10.3) should consider possible measures to protect the structure against premature deterioration. A set of appropriate measures (one or more) can be combined to ensure that the required service life is obtained with a sufficiently high probability.

The protective measures available may influence either the duration of the initiation phase or the rate of propagation of active deterioration.

The strategy of the service life design is to select intelligently an appropriate number and types of co-operating measures to ensure the required service life, considering the environment in question.

This design strategy is considered a "*Multi-Stage Protection Strategy*" which leaves the selection of individual protective measures to the designer, [Rostam (1993)].

Protective measures may be established by, i.a.:

- the selected structural form,

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		Exposure Classes																
		No risk of corrosion or attack	Carbonation-induced corrosion				Chloride-induced corrosion						Freeze/thaw attack				Aggressive chemical environment		
							Sea water			Chloride other than sea water									
		X0	XC 1	XC 2	XC 3	XC 4	XS 1	XS 2	XS 3	XD 1	XD 2	XD 3	XF 1	XF 2	XF 3	XF 4	XA 1	XA 2	XA 3
	Maximum w/c	---	0,65	0,60	0,55	0,50	0,50	0,45	0,45	0,55	0,55	0,45	0,55	0,55	0,50	0,45	0,55	0,50	0,45
	Minimum strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
	Minimum cement content (kg/m^3)	---	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360
	Minimum air content (%)	---	---	---	---	---	---	---	---	---	---	---	---	4,0[1)]	4,0[1)]	4,0[1)]	---	---	---
	Other requirements												Freeze/thaw resisting aggregates in accordance with the recommendations in prEN 12620					Sulphate-resisting cement [2)]	

[1)] Where the concrete is not air entrained, the performance of concrete should be tested according to prEN FFF-1 in comparison with concrete for which freeze/thaw resistance for the relevant exposure class is proven.

[2)] When SO_4^{2-} leads to exposure classes XA2 and XA3, it is essential to use sulphate-resisting cement. Where cement is classified with respect to sulphate resistance, moderate or high sulphate-resisting cement should be used in exposure class XA2 (and in exposure class XA1 when applicable) and high sulphate-resisting cement should be used in exposure class XA3.

Table 5.10.1: Recommended limiting values for composition and properties of concrete [CEN (1999)]

- the concrete composition, including special additions or admixtures,
- the reinforcement detailing including concrete cover,
- a special skin concrete quality, including skin reinforcement,
- limiting or avoiding crack development and crack widths, e.g. by prestressing,
- additional protective measures such as tanking, membranes or coatings, including coating of reinforcement,
- specified inspection and maintenance procedures during in-service operation of the structure, including monitoring procedures,
- special active protective measures such as cathodic protection or monitoring by way of sensors.

A different level of reliability is associated with the protective ability of each type of measure. This level of reliability depends much on the quality assurance scheme associated with the establishing and possible maintenance of each protective measure.

The different measures may act simultaneously in contributing to the protection, or one measure may be substituted by the next, once the former has been overcome, eliminated or surpassed by the aggressive substance.

5.10.6 Robustness in design and construction

One of the main obligations of the conscious designer is to adapt the design to the conditions, under which the structure is to be constructed, operated and maintained. To avoid structures being sensitive to variations in the assumed qualities a degree of robustness in the design can be very advantageous with respect to the future performance and durability of the structure. The following sub-chapters indicate means of increasing the robustness of concrete structures.

(1) Adapt requirements to local conditions

The conditions under which structures shall be constructed and used differ from case to case. It is therefore essential to adapt the requirements for the concrete mix, casting, compaction and curing to what can realistically be achieved at the individual location. This shall consider the available concrete components, local workmanship and prevailing climatic conditions.

This is particularly important in remotely located structures and in geographic regions with little or no alternatives to the local cement, aggregates and water, and only local, maybe *unskilled*, labour.

One such example could occur in hot, arid and saline environments with unavoidable high chloride content in the available fine aggregate and in the mixing water. The difficult curing conditions and the unavoidable high initial chloride content in the concrete could then warrant the application of stainless steel reinforcement selectively in the most critically exposed parts of the structure. Such a solution would with "one stroke" eliminate the reinforcement corrosion problem at an initial cost increase, which usually would be marginal compared to the overall costs for construction and furnishing the whole building. In the long term this solution could become a remarkable cost saving decision taken up front, which could prolong the service life of the structure many fold times compared to current experience in such environments. The service life would in such cases be determined by the durability of the concrete itself, or by the structure becoming functionally obsolete some time in the future and needing replacement.

(2) Consequences of designing slender or minimal structures

The concept of minimal structures has developed into a challenging engineering and architectural discipline. By selecting combinations of structural form and materials, which minimise the total consumption of materials, very light structures can be achieved.

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This full utilisation of the material strength throughout the structure assumes á priori that no deterioration takes place while the structure performs its long-term load carrying duty under the influence of the prevailing environment. If local weakening or deterioration occur in a fully optimised structure there are no build-in alternative paths for the loads to follow while maintaining the overall safety level as required. Such structures, without reserves or redundancies, have no robustness. Therefore they impose very high demands - and costs - on the inspection and maintenance operations, [Rostam (1998)].

The engineering attractiveness of optimising concrete structures is gaining momentum, among others through the continuous development of still stronger high strength/high performance concretes. This development is valuable and attractive as long as the overall time dependent performance of the structure in its foreseen environment is taken fully into account up front.

However, the more we engineers learn to optimise the structural exploitation of the concrete and reinforcement strengths, the greater becomes the demand to also master the ageing effects and control the deterioration mechanisms threatening our structures.

(3) Adequate dimensions

The designer can ease the execution phase, and thus increase the probability of achieving good quality concrete in the final structure by using dimensions of the individual members, which enhances placing of reinforcement and placing and compacting concrete. Tight dimensions, small sizes members and difficult accessibility during casting often leads to insufficient quality of the concrete, in spite, maybe, of a good initial quality of the concrete being used.

Abrupt changes in geometry should be avoided if possible. The local concrete quality along edges and at corners usually becomes somewhat inferior compared to the bulk of the concrete and along smooth faces. In addition, the concrete along edges and at outgoing corners is exposed to the aggressive environment from two or three sides simultaneously. This is often reflected by the so-called corner effect, where cracks due to deterioration first develop along the exposed edges. Hence, edge zones and corners are more prone to premature deterioration than the remaining parts of the exposed faces, and rounded edges and corners, where possible, could improve the robustness of the structures.

(4) Congested reinforcement

The reinforcement, non-prestressed as well as prestressed, should to the extent possible be sufficiently distributed within the concrete zone foreseen for this purpose to ensure a good and reliable casting and compaction of the concrete, especially in the outer concrete layer constituting the skin of the concrete structure.

Congestion of reinforcement may lead to difficult casting conditions as highlighted in Chapter 0. Where possible the reinforcement layout should constitute a closed 3-dimensional reinforcement cage. When using links or stirrups, these should have a geometric form, which complies with the 3-dimensional closed cage action. In case of some unforeseen deterioration taking place, which would crack or split the concrete, e.g. bulk frost damage or alkali aggregate reactions, then the 3- dimensional reinforcement cages would tend to hold the pieces together. There are examples of concrete members cracked into pieces constituting a 3 dimensional "puzzle" by alkali aggregate reactions, but maintaining full load carrying capacity due to the valuable confining effect of the reinforcement cage.

(5) Fibre reinforcement

The beneficial effect of adding a small amount of polypropylene fibres to the concrete mix to reduce the risk of plastic shrinkage cracking has been mentioned. By adding steel fibres both the risk of plastic shrinkage cracking and risks of later drying shrinkage cracking may be

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reduced. By adding steel fibres to the concrete for floor relatively large areas can remain crack-free without expansion joints.

Whereas polypropylene fibres in concrete only influence the performance of the very young (green) concrete, due to their low strength, steel fibres and other high strength fibres, such as carbon fibres, can have a major structural effect in the finished structure. In case concrete without reinforcing bars is considered due to corrosion risks, e.g. for some foundations, retaining walls or seawalls, then steel fibres can be both a durability and a structural enhancement. Particularly it should be noted that steel fibres in exposed concrete may corrode, but practice has shown that only the outer exposed fibres corrode. Due to the size of the single fibre the expansive forces of the rust is insufficient to damage good quality concrete. The only disadvantage seems to be a rusty discolouring of the surface. Often this could be acceptable in e.g. marine structures.

5.10.7 Aesthetics

The visual appearance of a structure may deteriorate to a level where the structure becomes so unsightly, that the users loose confidence in the structure, or for other reasons deems the structure prematurely unfit for use.

The design should consider detailing which increases self-protection and robustness of the structure against aggressive environment.

This includes provisions to ensure satisfactory weathering and ageing of exposed surfaces thus allowing buildings to grow old gracefully without expensive maintenance. An appropriate selection of structural form should be ensured at an early, conceptual stage of the project.



Figure 5.10-1: The Sydney Opera House reflects the potentials of a close co-operation between architect and engineer

Unsightly or dirty structures reflects apparent bad quality, low maintenance, reduced safety and serviceability, and low social standard.

It is the responsibility of all involved: architects, engineers, contractors and owners, to contribute to improved reputation of concrete as our most important construction material. A pleasant visual appearance and attractive ageing properties may contribute substantially to an improved reputation of concrete structures, see Figure 5.10-1.

5.10.8 Durability enhancement measures

For the very large majority of ordinary structures to be placed in aggressive environments the design approach for durability described in the foregoing sections will ensure a satisfactory service life. A conscious choice of cement type, concrete mix (especially w/c-ratio), concrete cover, curing (moisture and temperature control) and geometry of the exposed parts of the structure, all adapted to the identified aggressivity of the environment, will normally result in a satisfactory service life of these structures.

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However, it must be recognised up front that in some selected or special cases the usual choice of design parameters for durability will not provide adequate service life, often just because a very small or local part of the structure is particularly prone to premature deterioration. This would typically be in the following situations:

- The environment is particularly harsh and aggressive. Typically this would be a hot, humid and saline environment, like splash zones of marine structures, or it would be structures exposed to extensive use of chloride based de-icing chemical. Further aggravation would be caused by cyclic wetting and drying or by other types of evaporative effects causing increased concentrations locally of chlorides, sulphates, magnesium salts, etc. Also the high temperature and wind may result in inferior quality of the hydration and curing of the concrete, leading to inferior in situ quality of the concrete.

- The available ingredients of the concrete are unavoidably polluted, particularly if this is with chlorides in the fine aggregates and in the mixing water. The advantages of concrete of being based mainly on local materials may in such special cases also be to its disadvantage.

- The available workforce does not have adequate knowledge or understanding of the major importance of the execution process in providing a long term durable structure. Realistically only mediocre or inferior workmanship can be expected to be represented on site. Hence, the more complex the structure is, and the more execution sensitive the mix is, the more unlikely are the chances of obtaining the required quality in the finished structure. Again, the advantage of concrete of being mainly based on local workforce may in such cases also be to its disadvantage.

- The required service life is very long compared to normal structures, say 100 or 200 years, or more.

The most serious conditions occur when the first two or three of the above situations coincide. Such a negative synergy effect is unfortunately the reality in large tropical and subtropical regions. In spite of this concrete still remains the dominating building and construction material in these regions, although short service lives are generally experienced leading to costly repairs and early reconstruction.

Also in tempered and even arctic climates can the environment be particularly aggressive, e.g. by de-icing salts or marine splash, and combined with difficult casting and curing conditions the concrete quality becomes insufficient to adequately resist the environment.

In all the above situations it should be recognised up front that maybe concrete alone should not be relied upon to provide the service life needed, and durability enhancement systems should be applied.

Therefore, the designer shall consider if the combined effects of the environment, the available cements, the local aggregates, the available workforce with its expected workmanship, and the foreseen curing possibilities will leads to a forecast of a final structure with shorter or less reliable service life than required. If this is the case, then special durability enhancement measures should be taken into consideration.

Such durability enhancement measures are of the following types [CEB-FIP (1992)]:

- Provide structural protection, by using:
  - smooth surfaces with rounded edges and minimise the area exposed to the environment
  - roof, eaves or similar to protect concrete surfaces against rain

- Provide special protection of the concrete, which usually also will protect the reinforcement, by using:

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- tanking of foundations (bituminous sheets), see Sub-chapter 5.10.8-(1)
- surface protection, surface coating and surface insulation
- lining of exposed surfaces (stainless steel lining), see Sub-chapter 5.10.8-(2)
- permeability controlled formwork liners, see Chapter 5.9.3
- self-compacting concrete, see Chapter 5.9.4
- fibre reinforcement, see Sub-chapter 5.10.6-(5)

- Provide special protection against corrosion of the reinforcement by using:

  - high performance concrete, see Chapter 5.9.6
  - increased concrete cover. Provide special skin reinforcement (e.g. stainless steel mesh) for very large covers, say > 100 mm, see Chapter 5.7.5
  - non-corrodable reinforcement, specifically stainless steel, see Sub-chapter 5.10.8-(4)
  - coating of the reinforcement, specifically epoxy coated reinforcement or hot dip galvanised reinforcement, see Sub-chapter 5.10.8-(5)
  - preventive cathodic protection, see Sub-chapter 5.10.8-(6)
  - corrosion inhibitors or pore blockers
  - prestressed reinforcement in protective sheathings with corrosion protective grout

- Provide intensified inspections for early warnings, and introduce regular maintenance routines, supported by using:

  - durability or performance monitoring through sensors
  - selective testing and service life updating

Some of the main measures mentioned above are commented in the following.

(1) Tanking

In aggressive soils the basic design strategy A1, as described in Chapter 5.10.3, would lead to a separation of the concrete from the environment by introducing a physical membrane. Usually a paint type coating will not be durable enough, and it is essential that the whole concrete surface be covered, meaning also the bottom of the foundation.

Bituminous sheets have proven very efficient in providing such protection by placing the sheets on a layer of blinding concrete and cast the foundation on top. After stripping the formwork the sheets are taken up the surface to cover all of the concrete to be below the surface of the soil. More than 25 years of experience with this solution in the Gulf countries have shown excellent performance.

(2) Lining

Also above the soil a separation between concrete and the environment may be advantageous. One possibility, which has a high degree of reliability, would be to use a thin stainless steel lining of the concrete surfaces most exposed to the aggressive environment.

Selectively used, this may prove a cost effective solution when all long term aspects are considered in the life cycle cost optimisation, see Figure 5.10-2.

(3) Increased concrete cover

The primary protection of the reinforcement is the quality and thickness of the concrete cover. The penetration of chlorides and other aggressive ions into concrete follows a diffusion- like rate, which can be seen from. Figure 5.7-5.

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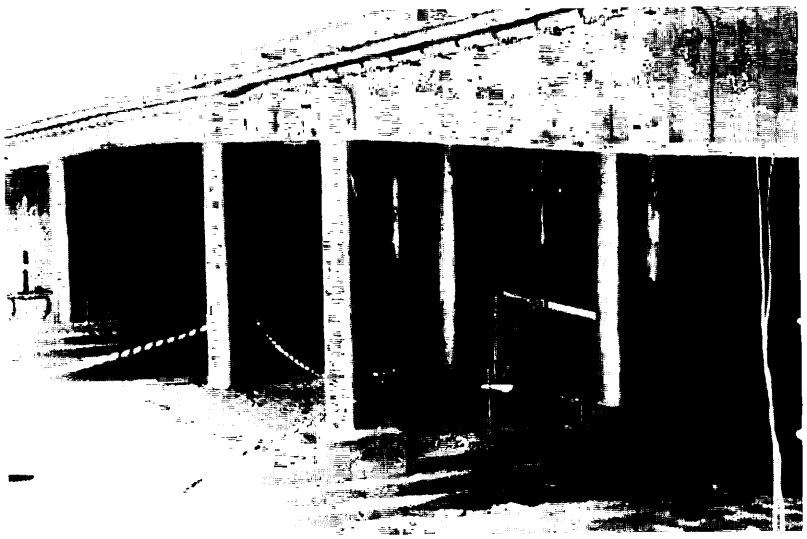


Figure 5.10-2: Permanent thin stainless steel lining used as lost formwork for concrete columns. The structure is nearly 70 years old and has been exposed to heavy de-icing salting each winter for 30 years without any trace of deterioration.

The simple rule of thumb, called the square root law, states that increasing the cover by a factor two increases the service life by a factor four. From Figure 5.7-5 it is seen that the real effect in uncracked concrete may be even more pronounced, in this case an improvement of factor seven.

This demonstrates most clearly the importance of ensuring the right concrete covers, as stated several times in this chapter.

However, the quality of the concrete shall also be ensured, and this is where high performance concrete comes into the game. The development of HPC is interesting in this respect, see Chapter 5.9.6.

(4) Non-corrodable reinforcement

Recent years development within the area of non-corrodable reinforcement for concrete structures is opening promising new doors in our fight against reinforcement corrosion. The following products are already available:

- Stainless steel reinforcement
- Non-metallic reinforcing bars such as reinforcing bars from glass fibres, aramid fibres or carbon fibres.

During the past few years stainless steel reinforcement has become commercially available in dimensions, strengths and alloy types which are fully compatible with normal structural requirements for reinforced concrete structures, and at competitive prices - relatively speaking. Such stainless steel reinforcements may be corrosion resistant even in highly chloride-contaminated environments. Used selectively in the most exposed zones of the structure the increased costs per kg steel compared to the costs of normal steel will have marginal or negligible effect on the overall initial construction costs. In addition the service life costs will be reduced due to savings in future repair and maintenance, [Concrete Society (1998)], [Abbott (1999)]. Reference is also made to chapter 3.2.5(3).

From a practical point of view this technology is particularly interesting because it "only" solves the corrosion problem. All other techniques and technologies within design, production and execution remain unchanged, a fact that is very attractive to the traditionally very conservative construction industry.

It is recognised that the most serious durability problem for concrete structures is the reinforcement corrosion problem. Therefore, ***the reliable and readily availability of stainless steel reinforcement at reasonable and foreseeable prices may change - or revolutionise - major parts of the building sector in aggressive environments, simply by solving the corrosion problem.***

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The main reason is also the added value which follows from the possibility of accepting the use of locally available materials, even with chloride contamination, and also accepting the qualifications of the local workforce as it is, and still produce highly durable long lasting reinforced concrete structures.



Figure 5.10-3: 60 year old stainless steel reinforced pier at Progreso in Mexico still fully intact without any maintenance whereas the remains in the foreground is what is left of an only about 30 year old pier reinforced with ordinary black steel[Knudsen et al (1999)]

Examples where this could be of particular value are the extremely aggressive environments in hot, arid and saline environments as found in e.g. the Middle East, particularly in the Gulf and Red Sea environments, and in similar regions of the Far East and Central America.

A very valuable and convincing documentation of the performance of stainless steel reinforcement in highly chloride contaminated concrete is presented by the 60 year old 2 km long concrete pier out into the Mexican Gulf at Progreso in Mexico reinforced with stainless steel, see Figure 5.10-3. No corrosion has taken place within the structure, despite the harsh environment and poor quality materials used in the construction. The chloride levels, at the surface of the reinforcement was between 10 and 20 times the traditionally assumed corrosion threshold level, [Knudsen et al (1999)].



Figure 5.10-4: Stainless steel reinforcement in concrete bridge deck in USA

Selective use can also be extremely beneficial in the industrialised regions suffering heavy winters thus using extensive quantities of de-icing salts, say Northern, Central and Eastern Europe, North America and Northern Japan etc, see Figure 5.10-4.

Stainless steel reinforcement can be combined with black steel cast into concrete without risks of galvanic corrosion due to bi-metal action. The reason is that the two types of steels reach nearly the same electro-chemical potentials when cast into concrete. This is an important observation, and is the precondition for general economical application of stainless steel

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reinforcement used selectively in the parts of the structure where this protection is needed, [Knudsen et al (1998)] and [Pedeferri et al (1998)].

Fibre reinforcement bars have also been developed and it is recognised that the corrosion resistance is equally achieved with such reinforcing bars. At present such non-metallic bars are based on glass, aramid or carbon fibres. However, the different mechanical characteristics between fibre reinforcement and steel reinforcement, together with the different conditions for practical use on site, seems to indicate only rather limited or special applications in concrete structures, e.g. the precast industry seems to present the largest potential use of fibre reinforcing bars. However, Figure 5.10-5 shows a recent application of carbon fibre reinforcement and stainless steel reinforcement used in a cable stayed concrete footbridge with 80 m central span, the largest bridge using carbon fibres as cables and main reinforcement in the deck, [Christoffersen et al (1999)].

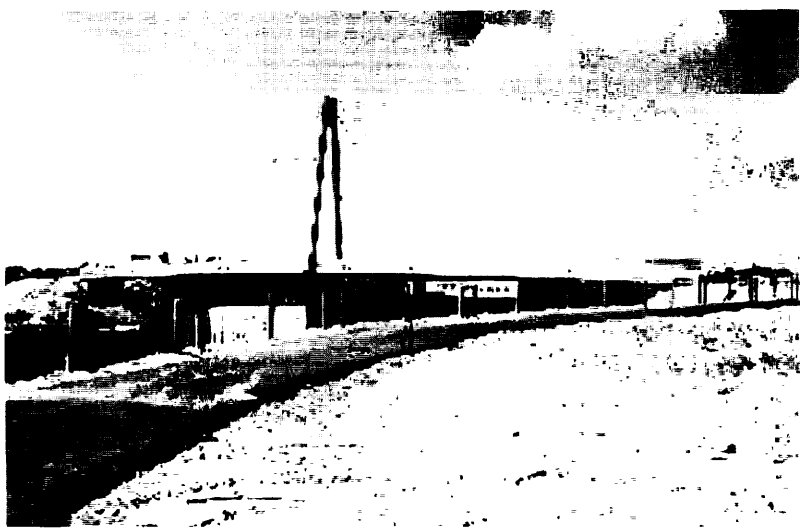


Figure 5.10-5:Reinforced concrete cable stayed foot bridge in Herning, Denmark. The main span is 80 m, being a record with carbon fibre cable stays, carbon fibre reinforcing bars in the deck and carbon fibre prestressing strands. The side spans are reinforced with stainless steel reinforcement. A full scale test bridge to verify performance and corrosion resistance

The special characteristics and applications of fibre reinforcement bars have not been dealt with in further detail in this text.

(5) Epoxy coating and hot dip galvanising of reinforcement

Numerous types of protective coatings to reinforcement have been tested. These are:

- Epoxy coated reinforcement, which has been a commercially available product for several decades, though only with limited application in Europe.
- Hot dip galvanised (HDG) steel reinforcement has been proposed and tested in several specific cases.

Epoxy coating of reinforcement has been used in North America since the mid 70'ies. This technology has for several years been under critical discussion.

The nearly unavoidable fine cracking occurring during bending, the pinholes occurring in the coating, and the inferior protective ability of the patch repaired zones have led epoxy coated reinforcement to be a less attractive solution. It also has the side effect that this technology will prevent the possible future use of cathodic protection, leaving no alternative to replacement of damaged members if corrosion develops. Currently, the coating industry is working on enhancing the technology. North American experience has thrown doubts on this approach, when following the traditional procedure of coating straight bars individually, then cutting them to length and bending them to the required shape.

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Figure 5.10-6: Fluidised bed dipping technique used to coat 62,000 fully welded 3-dimentional reinforcement cages for the precast tunnel segment for the East Tunnel of the Great Belt Link, Denmark

Using the fluidised bed dipping technique for 3D fully welded, grit blasted and pre-heated reinforcement cages will eliminate the need for cutting and bending coated bars, see Figure 5.10-6. In addition, the welding of the cages prior to coating will ensure the electric continuity, thus maintaining the possibility to install cathodic protection some time in the future, if need arises. This is a novel technique and damage to the coating is reduced to close to nil due to avoidance of the cutting and bending of coated bars. More durable coating formulations can also be used when there are no strict requirements for flexibility to allow for bending.

With respect to galvanised reinforcement not much practical experience is available regarding performance in very aggressive environment. However, it has been shown in tests that HDG steels may have a higher threshold value for chloride induced corrosion (initiation) compared to black steel. However, the protective ability depends on the reactions at the iron-zinc interface. The risk of pit corrosion is considered to increase for such HDG steel if exposed to chloride attack.

(6) Cathodic prevention

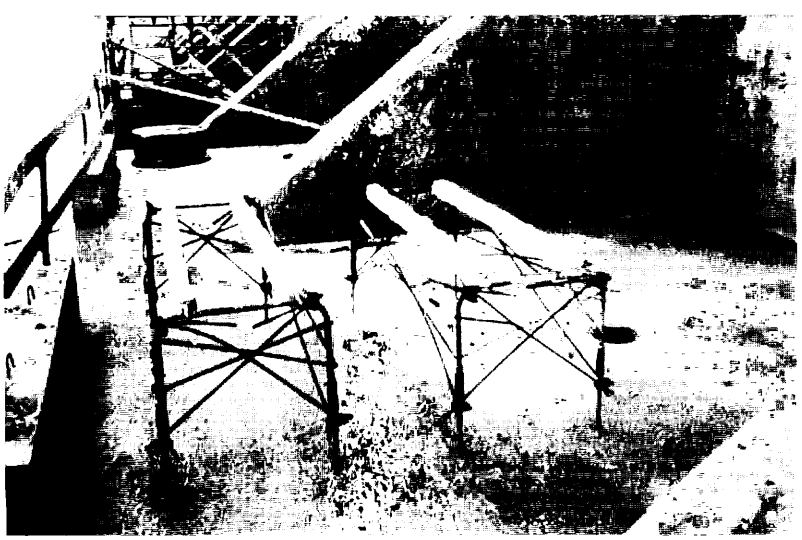


Figure 5.10-7: Sacrificial anodes placed on the bottom slab of prefabricated caissons to be placed permanently submerged in the sea. (The Great Belt Link, West Bridge

Cathodic prevention has become an interesting option to protect the parts of structures to be exposed to high chloride concentrations. In this way a cathodic protection system is installed already within the new structure ready to be used to protect the reinforcement against chloride corrosion.

When an impressed current system is foreseen, the system can either be energised from the start or be dormant until tests or corrosion sensors indicate that the initiation period is nearing its end and corrosion may begin.

In marine structures a sacrificial anode system may be installed simply by placing anodes in the water near or on the structure and link them up to the reinforcement. This can provide a

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very valuable initial protection of the reinforcement in the upper air containing part of the water, the tidal zone and part of the splash zone, see Figure 5.10-7, see [Storebælt (1998-1)].

5.10.9 Inserts and fixtures

Inserts and fixtures constitute elements being especially prone to early deterioration. Often deterioration starts from such elements corroding followed by cracking and local spalling, and developing further into more widespread damage to the structure. The interaction between inserts and the cast-in reinforcement can enhance rapid corrosion, and special precautions shall be taken to counteract such deleterious reactions.

When selecting the type of inserts and fixtures, the material composition and the way of embedding these elements in the concrete and e.g. fixing them to the reinforcement, the following shall be considered:

- Exposure conditions
  - chlorides - no chlorides
  - submerged
  - splash zone
  - atmospheric zone
- Contact to the reinforcement
  - electro-chemical potentials
- Material composition
  - black steel
  - galvanised steel
  - stainless steel

There is a major risk of macro cell corrosion inherent with inserts being in electric contact with the reinforcement and exposed to a corrosive environment at the concrete surface. In such cases a reliable long-term solution would be to use stainless steel for the cast-in part of the insert and then either hot dip galvanised steel for the member fixed to the insert, or also use stainless steel for this part, if no maintenance is foreseen.

5.10.10 Updating of service life

It has become clear from the previous descriptions that the use of specified concrete properties, or properties determined from separate test specimens, cannot be used for realistic service life predictions. Only testing of the finished structure and its materials will provide reliable data for such predictions due to the dominant influence of the execution process on the in situ quality of the concrete. This will determine if an acceptable service life can be realistically achieved, or whether additional durability enhancing measures will be needed in the future.

Based on current practice the requirements in codes to ensure sufficient durability are being given in a prescriptive form. Thus, requirements aimed at the concrete mix (cement type and content, w/c-ratio, etc.), at design parameters (cover, detailing etc.) and at execution (compaction, curing, etc.), while being essential to determine the accept/reject criteria for the product, cannot serve as basis for realistic service life calculations.

However, calculations predicting future performance will be very valid and useful when based on assessment of existing structures, provided adequate test methods exist to define the actual material properties in the structure. More work needs to be done on the development of in situ test methods, but it is already possible in many cases to make very useful and valid service life predictions.

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For example, if the carbonation depths are measured in a structure e.g. 5 years old, the time when carbonation will reach the reinforcement can be predicted with a high degree of reliability and any necessary protective procedures can be planned. Similarly, the time to initiation of chloride induced corrosion can be realistically estimated using the newest chloride transport models, if the calculation is based on real chloride profiles determined after the structure has been in service for some years.

As a result of these considerations, a design procedure according to strategy B, see Chapter 5.10.3 should contain the following:

a) Models describing the deterioration processes with quantification as far as possible.

b) Prescriptive requirements to ensure durability, such as maximum w/c-ratio, minimum concrete cover (including tolerance), curing procedures (moisture and temperature control), etc.

c) Quality control procedures covering all steps of the material production and the construction processes. Specifically important are interfaces where the responsibility for the quality of the product changes (e.g. handing over the ready mixed concrete at the site).

d) Definition of a "Reference Durability Period", e.g. 5 years. After that period relevant durability parameters such as porosity, permeability, diffusivity, carbonation, chloride penetration etc. can be determined to verify the achieved quality and resistance of the structure under the given environment.

   Already on completion of construction, surveys of some of the durability parameters such as cover, porosity, permeability and diffusivity could be carried out and entered into the "Birth Certificate", see Chapter 5.9.8. Procedures equivalent to this are already in normal use in some countries.

e) Using the models (item a above) and the durability parameters measured (item d above), the future performance of the structure can realistically be estimated and an updating of the achievable service life can be made.

Improved specifications, which include long-term qualities, are needed. However, they will not by themselves improve in situ quality without a changed attitude towards on site production and towards the interaction between design and production. The competence and experience of the work force is crucial. As stated in Chapter 5.4.1, the owner must ask for quality and must check the relevant quality of the structure he gets. If this would happen, quality will improve considerably just by the existence of the quality checks on durability made by the owner or his representatives.

5.11 Durability monitoring

Designing for long service life needs some form of verification during use of the structure. This becomes a natural part of the operation and maintenance activities, see Chapter 8. The service life design is based on some assumptions regarding deterioration mechanisms related to the aggressiveness of the environment. Due to the uncertainties between the prior assumptions made at the design stage and the reality of the service conditions of the finished structure, a need arises with time to verify - or adjust - the initial assumptions and revise the service life forecast accordingly.

This need for adjustment, and the benefits of obtaining more and more reliable forecasts of the service life to be expected, leads naturally to a need to monitor non destructively the durability performance of the structure in service, reference is made to Chapter 8.6.2.

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5.12 Recent advances in service life design

Recent years' extensive development in modelling deterioration mechanisms and in introducing reliability theories to handle the unavoidable uncertainties associated with the problems of durability have brought the possibilities of performing service life designs in practice close to daily application. It is foreseen that within five to ten years, reliability based overall service life designs may well have taken over today's general design procedures for concrete structures, [CEB (1997)] and [DuraCrete (1999)].

5.12.1 Probabilistic performance based service life design

The theories of probability and reliability in structural design have been developed and matured remarkably during the past five years. These theories have been transformed from the level of research and development to be directly applicable and operational in practical engineering design. The methodology has been internationally recognised and used for many decades as basis for the structural safety design through the well-known load-and-resistance-factor design. However, the factors governing the durability and performance of structures throughout their service life have only recently been developed in similar ways. [Siemes et al (1985)], [Siemes and Rostam (1996)], [DuraCrete (1999)], [Siemes at al (1998)] and [Høj and Rostam (1999)].

This has allowed the treatment of transport and deterioration mechanisms to be modelled on a probabilistic level and introduced in the general design of structures. Thus, design for safety and for durability can be performed using similar procedures. This opens the eyes of the owners now being able/forced to take decisions regarding the long-term performance and the consequences regarding maintenance and costs.

5.12.2 Modelling

Throughout the description in this Chapter the need for modelling the environmental aggressivity and the penetration of aggressive substance into concrete, and the corresponding deterioration mechanisms has been emphasised. Indication specific modelling within this text is beyond the scope of the text. Therefore some references of benefit to the reader are given in the following: [Helland et al (1995)], [HETEK 6-8(1997)], [Engelund (1998)], [CEB (1997)], [DuraCrete (1998)]

The importance of such modelling follows from the fact, that no prediction of future developments can be made without a model. For practical purposes such modelling should be rational mathematical modelling to be operational. In this respect it shall also be noted, that any such model, no matter how crude it is, is better than pure guesswork. Also educated guesswork in the form of experience based engineering judgement is valuable but should be presented in a mathematical format to be operational.

As soon as a model is available it opens up for the possibility to update or improve the model and its forecasting based on new information becoming available. Due to the well-known uncertainty associated with most parameters related to concrete and concrete structures and to their performance, a probabilistic treatment of the reliability of events occurring becomes natural. The result may then in turn be translated into a deterministic model or into deterministic requirements if, needed. This procedure corresponds fully to the well-known approach used for strength and safety of concrete structures. Following from this, normal safety levels, as well as service life design rules and acceptance criteria, can be chosen using well-known theories of reliability.

Such probabilistic reliability based treatment of the models and parameters governing the durability performance and service life of concrete structures exposed to different types of environmental aggressivity has been developed in a European research project, DuraCrete.

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The results are presented in a series of reports, with [DuraCrete (1999)] presenting the general guidelines for a probabilistic performance based durability design of concrete structures.

5.12.3 Deterministic versus Probabilistic Service Life Design

The merits of the probabilistic approach to durability design are illustrated by the following example of a marine structure. Two different environments are considered, representing yearly average temperatures of 10 °C (Northern Europe) and 30 °C (Middle East) respectively. The design requirement is 50 years service life. The service life is for illustrative purposes defined as the length of the initiation period, i.e. the time until corrosion initiation due to chloride ingress, see [Rostam (1999)] and [Engelund (1998)].

Figure 5.12-1 depicts the required concrete covers in each of the two environments, based on a traditional deterministic approach.

Figure 5.12-2 highlights the fact that the deterministic approach only provides a 50% probability of achieving the required 50 years corrosion free service life. This fact is often overlooked in usual design for durability. If, say only a 10% risk of having corrosion initiated before 50 years is considered acceptable, then much larger covers are required, as seen from Figure 5.12-2.

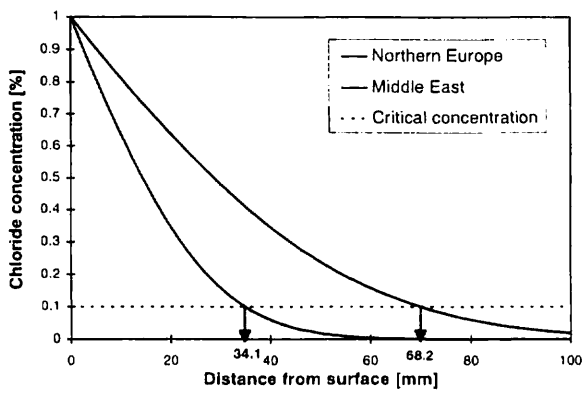


*Figure 5.12-1: **Deterministic approach.** Required concrete cover to ensure 50 years service life and assuming a chloride threshold value of 0.1% by weight of concrete.*

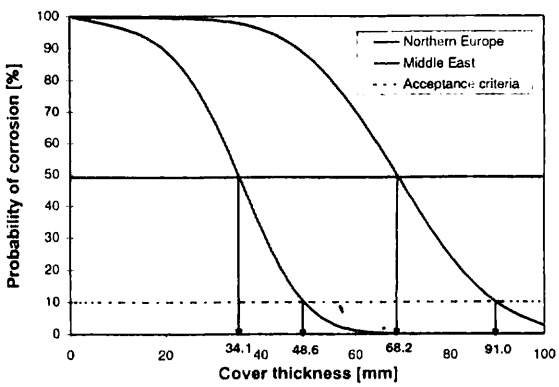


*Figure 5.12-2: **Probabilistic approach.** The deterministic approach provides only 50% probability of avoiding corrosion at the age of 50 years. Accepting 10% probability of having corrosion initiated after 50 years results in considerably larger covers*

The deterministic approach used here is based on mean values of the governing parameters. In the probabilistic approach the mean values and their known or assumed standard deviations are used together with the relevant distribution functions. This latter approach makes it not only possible to relate cover thickness to probability of corrosion but also to quantify the consequences of choosing different levels of risks of corrosion. These consequences relate not only to concrete quality and cover thickness, but more importantly also to the economic consequences. This can be illustrated by an example, see Chapter 5.12.4 below.

5.12.4 Life Cycle Cost Optimisation

As a simple demonstration the life cycle costing is illustrated through the following example. A 300 by 10 m concrete pier on concrete piles on the North Atlantic coast is analysed.

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The following four design strategies are considered:

1. A traditional design assuming ordinary concrete, usual construction methods and repairs only when extensive corrosion damage leading to cracking and spalling has been observed through regular inspections.

2. A design using very high performance concrete, large concrete covers, optimal execution and curing procedures, intensive quality assurance procedures and quality control on site. Regular inspections are made but only minor local repairs are assumed to be needed.

3. A traditional structural design and using ordinary concrete but providing preventive cathodic protection as sacrificial anodes in the water to protect the submerged and tidal zones, and prepared for CP by ensuring electric continuity of the reinforcement and providing cast-in anode bands in the parts above water. The external components of the impressed current system are only provided and energised when local corrosion initiation is observed.

4. A traditional design assuming ordinary concrete and usual construction methods. 50% of the reinforcement is replaced with stainless steel reinforcement. Regular inspections are performed.

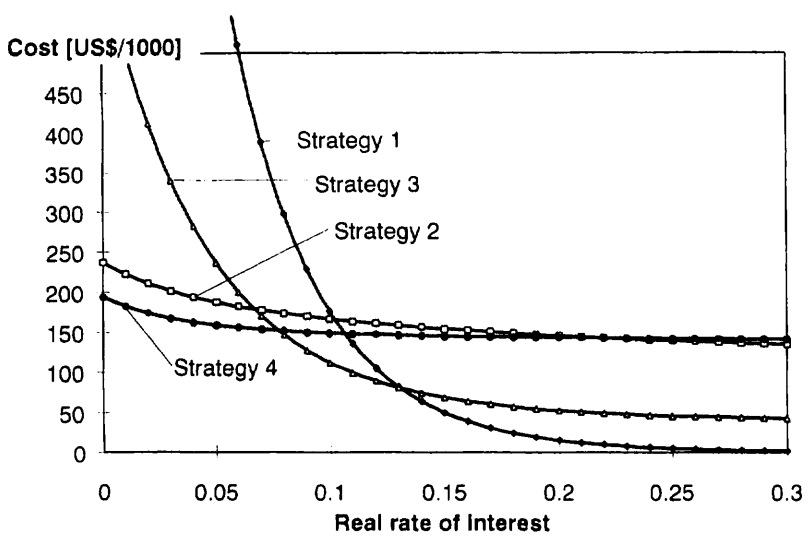


Figure 5.12-3: Life cycle costs for the four different design strategies

Realistic costs have been chosen for all elements of the above strategies. A probabilistic calculation of the service life costs has been made for each strategy, relating these costs to the real rate of interest. To illustrate the influence of the real rate of interest, values from 0 up to 30% have been analysed. The results are presented in Figure 5.12-3.

Figure 5.12-3 illustrates the two major consequences of the different design strategies. The most usual approach, strategy 1, together with the untraditional strategy of cathodic prevention, strategy 3, incur very high accumulated life cycle costs for relatively low rates of interest. But with high interest rates it becomes advantageous to postpone costs as long as possible.

The second observation is, that the life cycle costs are minimised in the usual cases of rather low interest rates, when either extreme measures regarding design, concrete quality and execution are taken (strategy 2), or when the critical corrosion problems are solved through non-corroding reinforcement (strategy 4). In this case the former solution has the uncertainty related to the availability of quality materials and the experience and workmanship of the local workforce. The latter solution is robust, from a durability point of view, with respect to the dependency of concrete materials, workmanship and quality of maintenance.

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5.12.5 Reliability updating

A very valuable consequence of adopting the probabilistic method in practice is that the uncertainties to be expected are taken into account at the initial design stage, where the concrete and the construction and maintenance conditions are not known. When these important, but initially assumed values are known, or can be determined through testing, the tools are already available to update the service life expectations, and in due time take necessary precautions. Hence, the service life and the economic consequences of any decisions taken (savings or additional costs) can be readily updated when inspections are made and additional information becomes available.

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6 Design of concrete buildings for fire resistance

by Karl R. Kordina

Preface:

Structural fire design and fire protection strategies need to be considered at a very early stage in the design of new buildings, and also in the renovation of existing buildings. Early planning can result in very significant cost savings. On the other hand, costs can become very high, if consideration is first given to fire safety when the building work is nearing completion.

Fire protection is achieved through a range of measures, including planning of evacuation routes and protection against smoke and toxic gases, as well as the fire design of loadbearing members.

Specific fire design provisions depend on the type and function of the building. They also vary according to local and national standards. Attention is therefore focused in this report only on general aspects of fire safety in concrete buildings and to give some additional information's to ENV 1992-1-2: Structural fire design.

6.1 Fire risks

6.1.1 Objectives and design provisions

Fires in buildings occur with unwelcome frequency also in industrialized countries but in spite of intensive efforts, it was not possible to reduce the fire frequency substantially. Designers have a very important role to play in providing adequate fire safety in buildings. Through the appropriate choice of design details and materials, they must protect lives, minimize damage and economic loss, and employ measures which restrict the spread of fire [Kordina/Kersken-Bradley (1996)].

National regulations for fire protection in buildings have a variety of aims, which can be summarized as follows:

- prevention of the outbreak of fire;
- avoid premature collapse of the structure;
- prevention of the spread of fire, smoke, hot and toxic gases beyond designated areas;
- safety of occupants in the fire zone, in other parts of the building and in adjacent buildings;
- evacuation of occupants from the immediate fire zone and other parts of the building;
- mitigation of damage to the building structure, the contents and to adjacent buildings;
- protection of life during firefighting and rescue operations.
- the mitigation of property damage is looked after by insurance activities on a voluntary rather than a mandatory basis.

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Government agencies are primarily interested in life safety, fire containmentation, and delaying the growth and spread of fire. They are also concerned with the possibility of general uncontrolled conflagration in built-up areas, and in ensuring that firefighting and rescue operations can proceed, even in buildings with difficult access such as large or high buildings. It is becoming apparent that some industrial installations also require national regulations because of special hazards which occur in the event of malfunction or accident. Examples are fires in nuclear power stations and in underwater tunnels.

Overall fire protection strategies involve both active and passive (or built-in) measures [National Building Regulations]. The passive built-in measures are preventive in character and remain permanently effective. Active measures, on the other hand, require the stimulus of the fire. They include detection, control by sprinklers or other installations, and firefighting.

Buildings are usually classified according to intend use, occupancy, size and location; nevertheless, the fire regulations are aimed primarily at the safety of human occupants. If special circumstances apply and the owner requires additional standards, such as preservations of the structure, then these will be over and above the minimum regulatory requirements. In this respect there is currently a need for a more rational approach to the specification of fire safety in buildings, which takes account of risk, and aims at the optimum economic level of protection.

Since 90 per cent of all fatalities in fires occur as the result of smoke and toxic gases, it is particularly important for evacuation routes to be provided, with floors and staircases remaining free of smoke and properly illuminated. Smoke and fire doors should shut off floors and protect staircases. The distance from any room to a staircase should not be greater than about 30 m. Staircases must be designed to withstand a fire for 90 minutes. In all cases two evacuation routes must be provided. Up to a height of about 20 m, fire brigade ladders may be considered to provide a second escape route.

6.1.2 Costs and Losses

Fortunately the risk of fire is generally low. While it remains so, people are reluctant to suffer too much inconvenience. Systems that are installed solely for the purpose of fire safety should therefore interfere with normal usage as little as possible, yet harmonize with public feelings on acceptable risks.

On the other hand, the risks of fire are not always obvious. As an example of unexpected risk, a person sleeping in a room adjacent to a fire is highly endangered because the concentrations of CO and CO_2 can rapidly reach fatal levels, well before the increase in temperature would wake the sleeper.

A major contribution to both fire prevention and fire protection is the development of an awareness of fire hazard in those who are in a position to do something to reduce it, be they the designer, the architect, legal authorities or members of the public. Building requirements concentrate on fire protection, on the assumption that fires will occur and that their consequences have to be mitigated.

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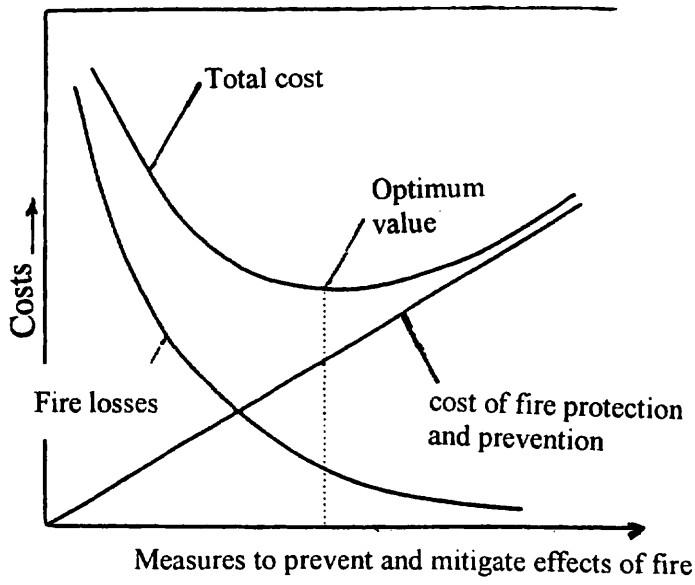


Fig. 6.1-1 Costs and losses

6.2 Structural fire design

6.2.1 Fire severity

The severity of a fire in a structure is dependent on many factors. However, building authorities assume for design purposes that a structure will be subjected to the worst-case fire, and indeed this approach is supported by the conventional wisdom of fire brigade experience, which is that any kind of combustible material will eventually burn.

The severity of a fire depends on three main factors:

- available fuel;
- ventilation, i.e. air supply available to promote its growth;
- the characteristics of the compartment where the fire commences.

For a given fire load, the severity of the fire can vary by a factor of two or three depending on the ventilation and the configuration of the area.

For practical purposes fire severity is expressed in the ISO 834 Specification [ISO 834] by means of a standardized temperature-time relationship which is also used in many national code regulations. Laboratory tests are conducted in furnaces using this relationship for the purpose of establishing the fire resistance of prototype structural elements [ENV (1992-1-2)]. This experimental approach has also been used to investigate the effects of various design variables, and much of the information available for design is derived from this type of study. Standard furnace tests are the main source of tabulated data in design codes on the fire resistance of different single elements. Typical results from fire tests in a building are shown in Fig. 6.2-1.

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Current building controls in most countries express the standard of safety for a building in terms of the fire requirements for individual elements. The intention is to ensure that the building structure does not collapse and that the separating elements are maintained during a fire. Fire resistance requirements are expressed for specified periods, ranging from 30 to 240 minutes.

According to the limit states philosophy a strength limit state is reached when a member can no longer carry its design load when combined with any additional thermally-induced loading. Other serviceability limit states may also have to be considered, such as those relating to the integrity of elements which provide horizontal or vertical barriers to the passage of fire. The latter may, for example, be required to prevent the progress of the fire through the gap between a floor slab and the suspended ceiling, or through holes provided in walls or floors for service pipes. A limit on deformation may also be used as a criterion for stability. In some special cases where re-use of the structure is important, additional criteria may be introduced which relate to reparability.

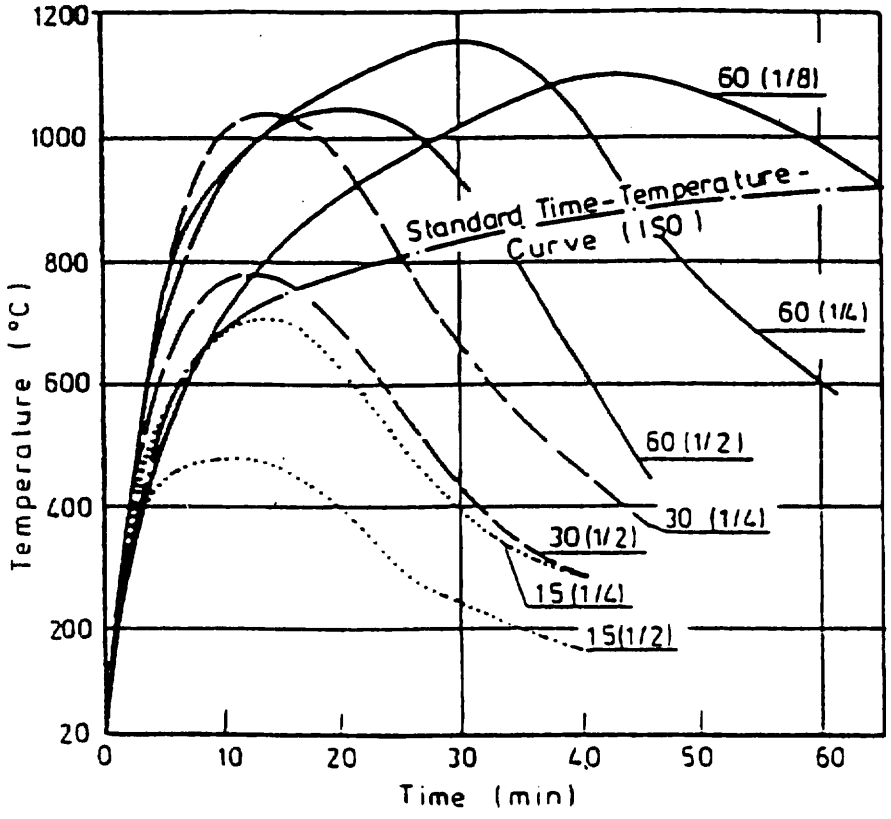


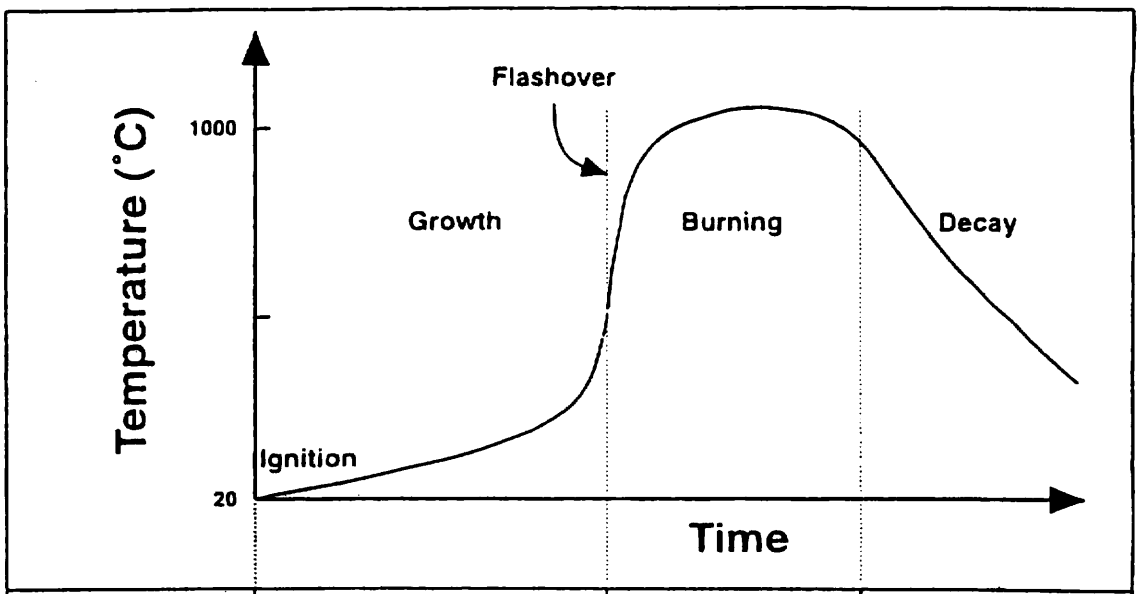
Fig. 6.2-1 Average temperature development with wood cribs in fire tests

6.2.2 Progress of fire in a building

Most standards describes the progress of fire in a building in terms of four main phases, which may be termed ignition, growth, the fully developed phase, and cooling. Ignition takes place if sufficient energy is supplied to combustible material in the presence of oxygen. Energy may be provided by a short circuit, a burning cigarette, sparks from a welding operation, or by machinery which is not operating correctly. It is also possible for energy to be

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supplied by the combustible material itself, i.e. for self ignition to occur. For the fire to develop into the growth and fully developed phases, air must be provided by some form of ventilation. In the growth phase, thermal energy is transferred by radiation and convection from the ignited material to other adjacent material, which becomes heated to the stage where it also ignites. Hot gases produced by the burning material can built up under the ceiling and cause a very rapid spread of fire. This process is called flameover. Smoke, toxic gases and heat may endanger life and property during the growth phase of the fire.



Stage	Growth	Burning	Decay
Fire behaviour	Fuel controlled	Ventilation-controlled	Fuel-controlled
Human behaviour	Escape	Death	
Detection	Smoke detectors Heat detectors	External smoke and flame	
Active control	Extinguish by hand, sprinklers or Fire Service. Smoke control	Control by Fire Service	
Passive control	Flammability, surface spread of flame	Fire resistance, containment, collapse	

Fig. 6.2-2 Development of fire in a building

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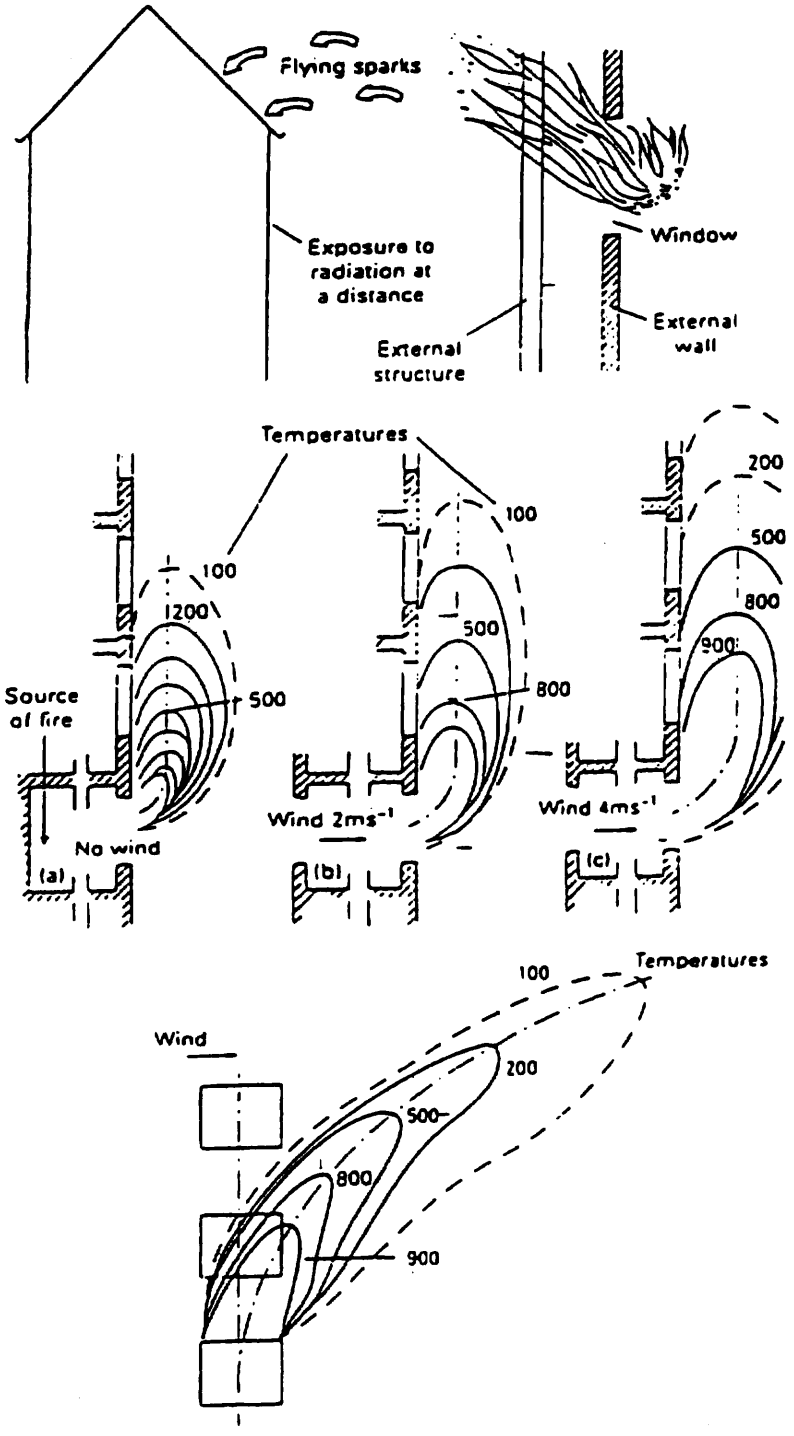


Fig. 6.2-3 Spread of fire through openings on outside of building

The ability of the load-bearing members to maintain structural adequacy in the presence of fire, and to resist the sustained loads acting, is of critical importance in the further sequence of events. Very high ambient temperatures occur during the fully developed phase of the fire and produce sharp temperature gradients and an attenuated rise in temperature within the structural members. The increasing internal temperature causes a fall-off in the elastic modulus and the yield strength of the steel reinforcement. The effects of temperature on the steel properties

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become noticeable at 200 °C; they are significant at 400 °C and very serious at 600 °C. The compressive strength and stiffness severe, the load capacity of individual members can fall to the level of the sustained loads which are acting, so that local collapse occurs. Large deflections occur prior to collapse, owing to the loss of elastic stiffness in both the steel and concrete. At an even earlier stage, structural damage will commence with spalling of cover concrete at edges and corners due to temperature gradients. As the protective concrete cover is lost by spalling, the temperature of the steel reinforcement reaches a critical level very quickly and a condition of collapse can develop. If the structure is prone to progressive collapse, the loss of one member due to the effects of fire can trigger a general collapse. Good compartmentation is therefore crucial in minimizing fire risk.

Of particular interest is the possibility of a fire spreading outside of the building and along the facade, and attacking external loadbearing columns. This is shown in Fig. 6.2-3.

6.2.3 Structural response

Building regulations in most countries provide for a minimum fire-protection strategy. Additional standards contain design aids for fire in the form of tables for minimum dimensions and minimum concrete cover (axis distances of the main reinforcement to the member surface) of beams, slabs, columns, tension members and the like [ENV 1992-1-2]. New standards such as the Eurocodes allow for additional computational methods to predict the development of fire in buildings and the response of the structure to the fire [FIP/CEB (1978), Pettersson (1978)].

A structure designed for a specified fire resistance time may collapse after exposure to fire over the specified time and may require demolition. If reparability is a design requirement, then limits on acceptable damage, residual deformation and degradation of materiel properties need to be specified.

In the design of a concrete structure for fire, an assessment of the loads and heat exposure is required, together with an analysis of structural response and an appropriate choice of structural system and components, including joints and supports.

Material limit states for concrete and steel can be expressed in terms of a "critical temperature", at which a specified deformation or rate of deformation is exceeded in a fire test specimen under constant stress. If the critical temperature is reached in a critical area of loadbearing structural element, failure is initiated by compression, tension or bending.

Other failure modes also have to be considered, such as shear and bond, lack of rotation capacity over the interior supports of flexural systems, and explosive spalling especially with humid concrete (Fig. 6.2-4).

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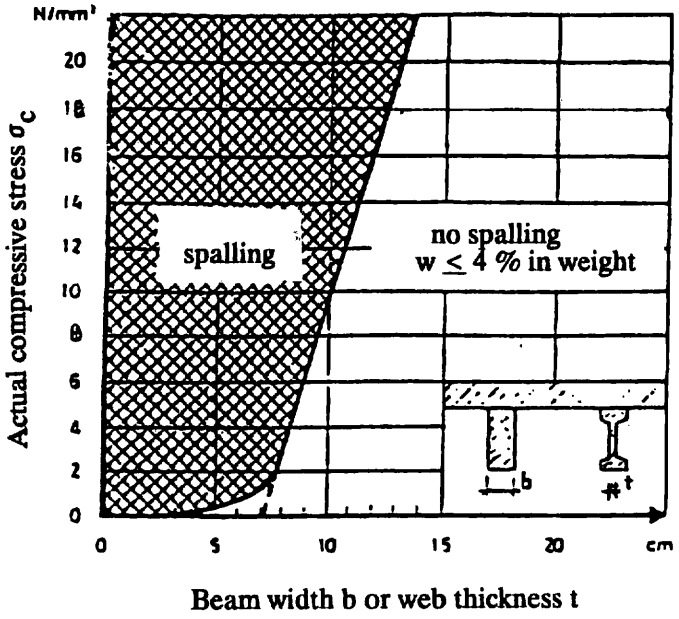


Fig. 6.2-4 Borderline between destructive and non-destructive spalling with plain and slightly reinforced concrete B 25 - B 55

(1) Spalling

Explosive spalling can occur during the first 10 - 20 minutes of severe fire exposure. This is governed by the water content of the concrete, its porosity and the level of loading in conjunction with the restrained thermal expansions. Spalling may be avoided by limiting stresses and keeping the water content of the concrete to less than 4 per cent of the weight. Fig. 6.2-4 gives additional information. High strength concrete (HSC) has a very small porosity and suffers by spalling more than normal concrete. Similar behaviour shows structural light weight concrete because of its relatively high water content stored by its porous aggregates which does not dry out under real conditions. Admixtures of plastic fibres can reduce spalling also in HSC concrete. Fibres of that kind are melting under fire attack opening exits for steam under high pressure - the most important cause of spalling.

(2) Separating functions

In treating the separating function of floors and walls, limit states are specified for thermal insulation and structural integrity against fire penetration. It needs to be emphasized that structural integrity cannot be checked by calculation, but only by test.

Integrity implies the absence of cracks, onfices or openings in separating elements through which flames and hot gases can pass, thus allowing the fire to break out on the other side. Retention of integrity is a design requirement for joints and junctions which should not open up as a result of excessive deformation of components. Occasionally, spalling of the concrete can lead to the formation of apertures in thin sections, with heat transfer through the construction. A fund of information on such phenomena is available in many countries and provides the basis for many national codes.

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(3) Fire resistance design

The current approach to fire-resistance design is dictated by the lack of simple and accurate methods for predicting structural behaviour in the presence of fire, and by the lack of adequate design data concerning both the variability in fire conditions and the effect of fire on material properties.

In a formal manner, three methods of assessment can be distinguished for use in structural fire design [ENV (1992), FIP/CEB (1978)].

- *Assessment Method I.* This is based on the standard heating conditions as formulated in ENV 1992 [ENV (1992)] or in ISO 834 [ISO 834].. The design criterion is that the fire resistance time of a building component is equal to or grater than that required by the relevant regulation. For design, the dimensions of structural elements are obtained from tabulated data, taking into account the loads and the required fire resistance time. Tables of that kind are published in ENV 1992-1-2.

- *Assessment Method II.* This is based on the concept of an equivalent fire exposure time which is used to relate the effects of a non-standard compartment fire to those of the ISO 834 standard fire and the related "standard heating conditions". However, the design criterion is that the fire resistance of a structural component, determined either by experiment of calculation, is equal to or grater than the equivalent time of fire exposure. This equivalent time relates the non-standard compartment fire exposure to the standard heating conditions in such a way that the effect on the component is the same. Figure 6.2-5 gives further details.

- *Assessment Method III.* This method uses an analysis of the structure when subjected to a non-standard compartment fire exposure. Assessment of the temperature is based on heat and mass balance equations, and account is taken of factors such as the fire load characteristics, ventilation conditions and the thermal properties of the structural components surrounding the fire compartment.

National fire standards usually give basic information on the above three approaches. At the present time, evaluation according to methods I and II does not usually differ significantly; however, method III needs expert help and approval. The use of most sophisticated computer programs is necessary, the results need very intensive critics with respect to their accord to reality. Assessment method III needs experience with real fires and laboratory test as well in order to avoid misleading results.

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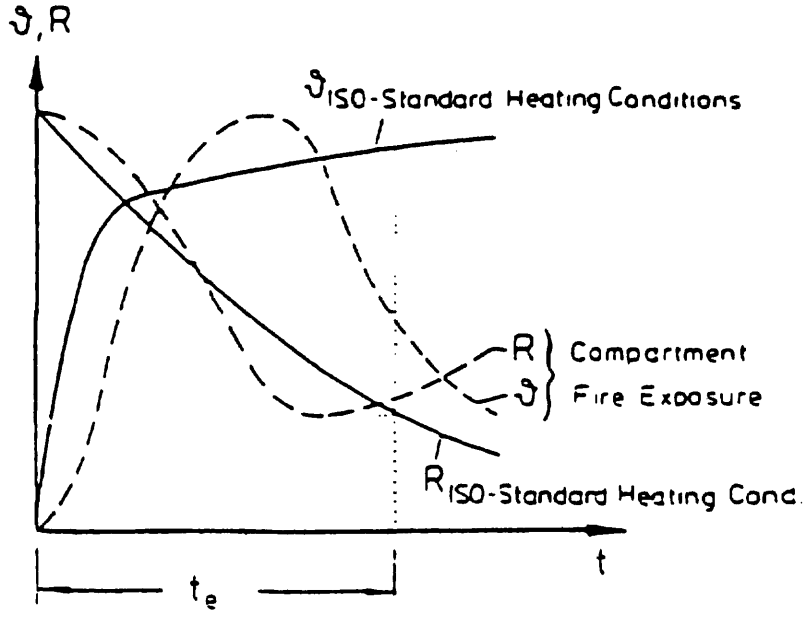


Fig. 6.2-5 Relation between "compartment fire exposure" and "standard heating conditions" (ISO 834), defining the "equivalent time of fire exposure"

(4) Tabulated data

As a result of the analysis of fire tests, carried out in accordance with ISO 834 on a large number of structural components, tabulated data have been made available which simplify the assessment of the fire resistance of commonly used structural elements [CEB (1991), Kordina/Meyer-Ottens (1999)]. As a further design aid a simplified analysis can be made of the moment capacity of reinforced concrete members exposed to fire.

6.3 Material properties

6.3.1 General aspects on the material properties

Values for the reduction of the characteristic compressive strength of concrete, and of the characteristic strength of reinforcing and prestressing steels are given in this section. They may be used with the simplified calculation methods. These values may also be used for the evaluation of the critical temperature of reinforcement when adapting tabulated data for critical temperatures other than 500 °C.

The material laws given below should only be applied for heating rates similar to those appearing under standard fire exposure until the time of the maximum temperature [ISO 834].

The standard fire conditions are defined between 20 °C and 1200 °C, the properties are also defined between the same limits.

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6.3.2 Concrete

The reduction of the characteristic compressive strength of concrete as a function of the temperature H is allowed for by the coefficient $\kappa_c(\theta)$ for which:

$$f_{ck}(\theta) = \kappa_c(\theta)\, f_{ck}(20°C) \qquad (6.3\text{-}1)$$

In the absence of more accurate information the following $\kappa_c(\theta)$ values, applicable to concretes with siliceous or calcareous aggregates, should be used (see Figure 6.3-1).

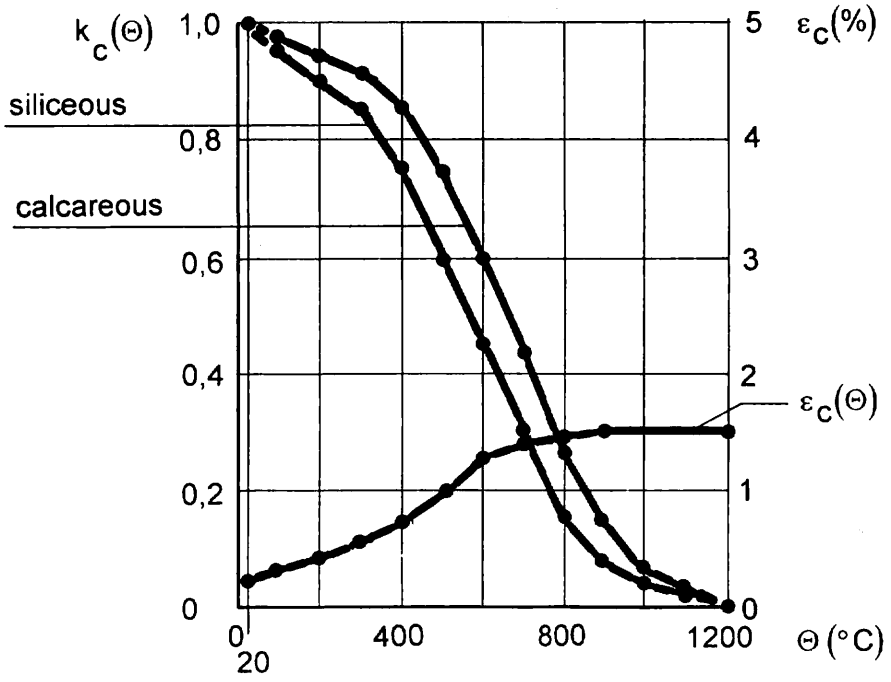


Fig. 6.3-1 Coefficient $\kappa_c(\theta)$ allowing for decrease of compressive strength (f_{ck}) of silicious or calcareous concrete at elevated temperature [ENV (1992)]

6.3.3 Steel

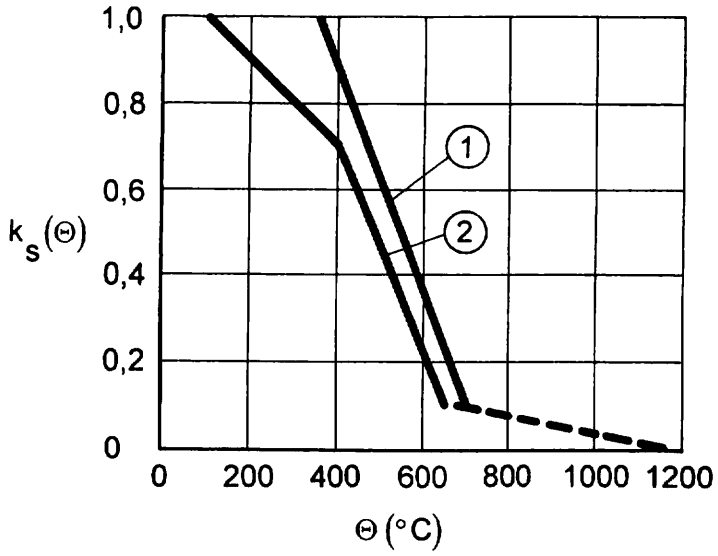
(1) The reduction of the characteristic strength of a reinforcing steel as a function of the temperature H is allowed for by the coefficient $\kappa_s(\theta)$ for which:

$$f_{sk}(\theta) = \kappa_s(\theta)\, f_{yk}(20°C) \qquad (6.3\text{-}2)$$

(2) The reduction of the characteristic strength of a prestressing steel as a function of the temperature θ is allowed for by the coefficient $\kappa_p(\theta)$ for which:

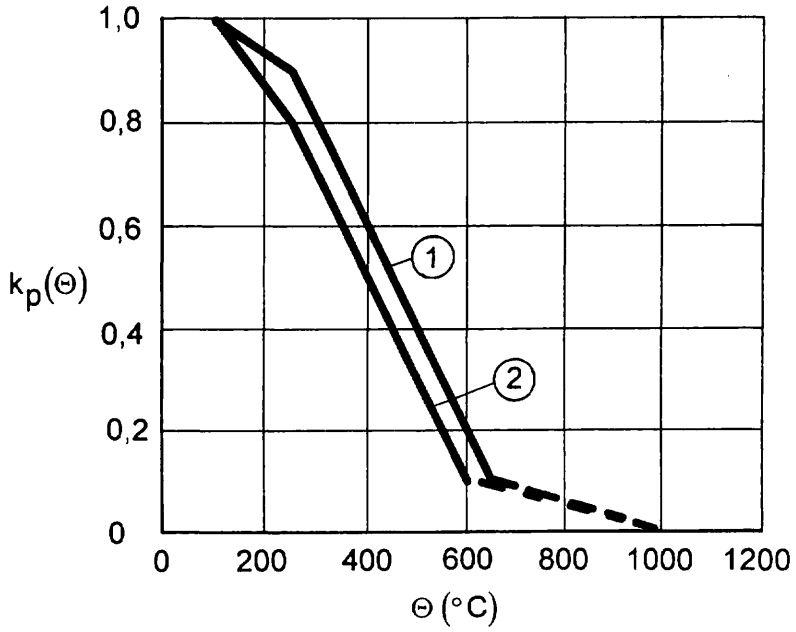
$$f_{pk}(\theta) = \kappa_p(\theta)\, f_{pk}(20°C) \qquad (6.3\text{-}3)$$

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Curve (1): $\kappa_s(\theta)$ applicable for strains, $\varepsilon_{s,fi} \geq 2{,}0$ %
Curve (2): $\kappa_s(\theta)$ applicable for strains, $\varepsilon_{s,fi} < 2{,}0$ %

Fig. 6.3-2 Coefficient $\kappa_s(\theta)$ allowing for decrease of characteristic strength (f_{yk}) of reinforcing steels at elevated temperature [ENV (1992)]



Curve (1): Bars
Curve (2): Wires and strands

Fig. 6.3-3 Coefficient $\kappa_p(\theta)$ allowing for decrease of characteristic strength (f_{pk}) of prestressing steels at elevated temperature [ENV (1992)]

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(3) Where $\kappa_s(\theta)$ and $\kappa_p(\theta)$ are taken from documented data they should be derived from tests performed under constant stress and variable temperature (transient tests). In the absence of more accurate information the following $\kappa_s(\theta)$ and $\kappa_p(\theta)$ values should be used for reinforcement (see Figure 6.3-2 and 6.3-3).

It should be noted that modern reinforcing steel recovers its original strength after a cooling down period nearly to 90 %. Tests are recommended, if the heating period differs significantly of a welding procedure of the steel members in question.

6.4 General design rules and tables

6.4.1 General

This section deals with detailing according to recognized design solutions as tabulated data (see 6.4.2).

Where necessary, explosive spalling shall be avoided by appropriate measures.

In the absence of more accurate data, the risk of explosive spalling can be assessed on the safe side by using Figure 6.2-4. For more accurate assessments, moisture content, type of aggregate, tightness of concrete and heating rate should be considered.

Where local experience indicates increased susceptibility of lightweight concrete to explosive spalling relevant documents should be used to determine member size or additional protective linings.

6.4.2 Tabulated data

(1) Scope

(1.1) In absence of more precise methods for structural fire design (i.e. general calculation method, simplified calculation method), reference should be made to the tabulated data given in national codes. Examples are given in the annex.

The following rules refer to single member analysis. The tables apply for the standard fire exposure.

(1.2) The tables have been developed on an empirical basis confirmed by experience and theoretical evaluation of tests.

(1.3) The values given in the tables apply to normal weight concrete made with siliceous aggregates (see ENV 1992-1-1, 3.1.2.1).

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If calcareous aggregates are used in beams and slabs either the minimum dimension of the cross-section or the minimum value of the axis distance, a, of reinforcement may be reduced by 10 %.

(1.4) The tabulated data takes into account requirements to prevent explosive spalling for all exposure classes and no further check is required. Examples of tables are given in the annex.

(2) General design rules

(2.1) Requirements for separating function may be considered satisfied where the minimum thickness of walls or slabs accord with standarized values.

(2.2) For loadbearing function (Criterion R), the minimum requirements concerning section sizes and heat protection (axis distance) of steel have been set up in the tables so that

$$E_{d,fi}/R_{d,fi} \leq 1,0 \qquad (6.4\text{-}1)$$

where:

$E_{d,fi}$ is the design effect of actions in the fire situation.
$R_{d,fi}$ is the design load-bearing capacity (resistance) in the fire situation.

(2.3) In order to ensure the necessary steel protection (cover, axis distance) in tensile zones of simple supported members, the tables are based on a critical steel temperature of θ_{cr} = 500 °C. θ_{cr} is the critical temperature of reinforcement at which yielding becomes imminent under the actual steel stress $\sigma_{s,fi}$. This assumption corresponds approximately to $E_{d,fi}$ = 0,7 E_d and γ_s = 1,15, where E_d denotes the design effect of actions according to ENV 1992-1-1.

(2.4) For prestressing tendons the critical temperature for bars is assumed to be 400 °C and for strands and wires to be 350 °C. If no special check is made for prestressed tensile members, beams and slabs the required axis distance, a, should be increased by

10 mm for prestressing bars, corresponding to θ_{cr} = 400 °C
15 mm for prestressing wires and strands, corresponding to θ_{cr} = 350 °C.

6.5 Overall design

6.5.1 Concept design and detailing

The normal steps in structural design, namely

- concept design,
- detailed design calculations and
- detailing,

also need to be applied in the case of structural fire design.

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The concept design and the detailing process together take care of those actions and situations which are unlikely to occur, and which are therefore not calculated in detail, but which require some consideration in design.

For this reason the author consider that concept design and detailing are as important in fire design as the detailed design calculations under normal temperature. In this respect, designing and detailing for fire refer to the following:

- making the structure robust in relation to fire;
- designing for thermal expansions in the building;
- detailing for thermal expansions;
- compartmentation.

6.5.2 Robust structures

Where fire resistance requirements do not strictly apply to the load bearing system, as in a one-storey building or a building with automatic sprinkler systems, the structure nevertheless should not collapse due to a small, local fire. This requirement would often be satisfied automatically because of the inherent fire resistance of normal concrete members which are not too slender. Without observing specific fire design requirements, concrete members are likely to have 50 to 60 minutes of fire resistance for the standard exposure situation.

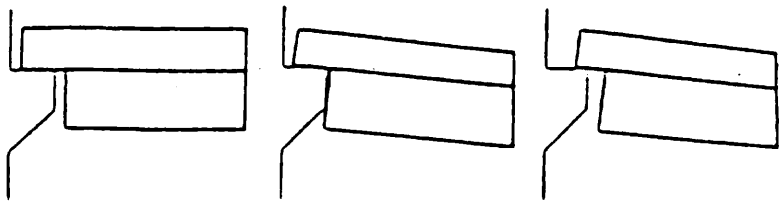


Fig. 6.5-1 Beam support area before, during and after fire exposure

Although a structure may not survive a fully developed fire, gradual collapse, or collapse preceded by visual and acoustic warnings, is preferred in view of the safety of firefighters. Stability failure may occur suddenly, and is undesirable in contrast with a strength failure preceded by large deformation.

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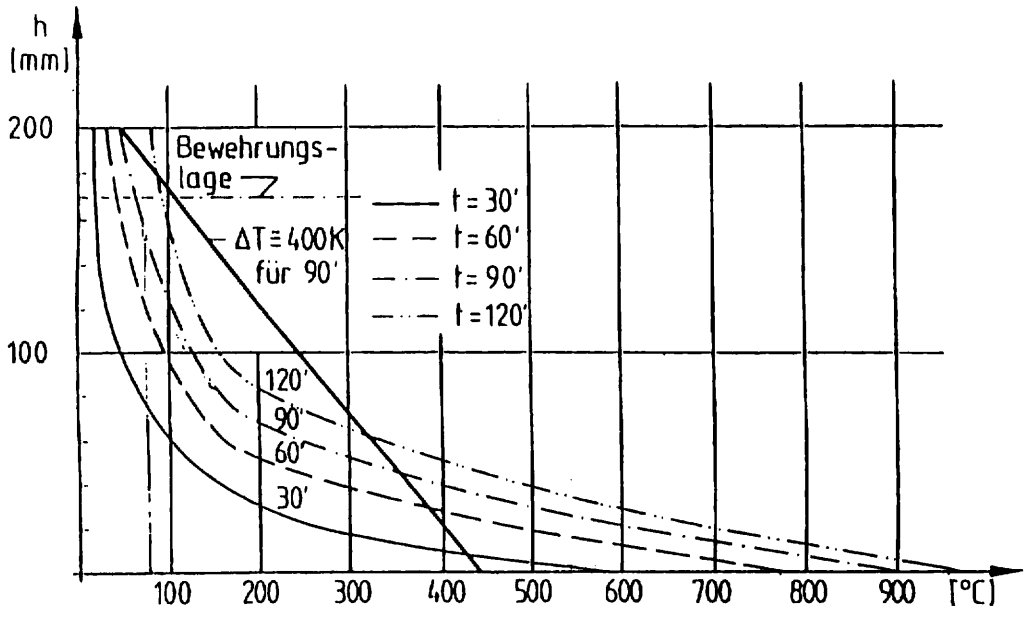


Fig. 6.5-2: Temperature gradient in concrete slabs after 30 to 120 minutes of standard fire exposure; upper reinforcement; assumed linear stress-strain relation

Supports and Joints:

Related to failure mode is the requirement that flexural failures should occur inspan regions and not at supports or joints. Thus, members should not slip off supports, and the joints of prefabricated composite structures should be overdesigned (Fig. 6.5-1).

6.5.3 Thermal expansion, restraint and behaviour of buildings in fire

In principle, design for thermal expansion only requires rough estimates of how the structure will expand and deform under fire, although it may be necessary to consider also the cooling down period.

Thermal expansion and deformation depend mainly on the temperature increase in members during fire exposure. Figure 6.5-2 gives thermal gradients in concrete slabs under standard fire exposure, which may also be considered as an equivalent time for fire exposure.

The thermal expansion of concrete can be expressed as a function of temperature. Siliceous concrete has values of about $10 \times 10^{-6}\ \theta_C$ for θ_C between 20 and 200 °C and of about $12 \times 10^{-6}\ \theta_C$ for θ_C over 200 °C. For steel average values are about $12 \times 10^{-6}\ \theta_S$.

For the simple assessment of normal reinforced concrete in fire, $\theta \simeq 600$ °C, we may take:

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$$\frac{\Delta \ell}{\ell} = 14 \times 10^{-6}\, \theta_c,$$

for concrete made with lightweight aggregates

$$\frac{\Delta \ell}{\ell} = 8 \times 10^{-6}\, \theta_c,$$

Fig. 6.5-3 gives more exact details:

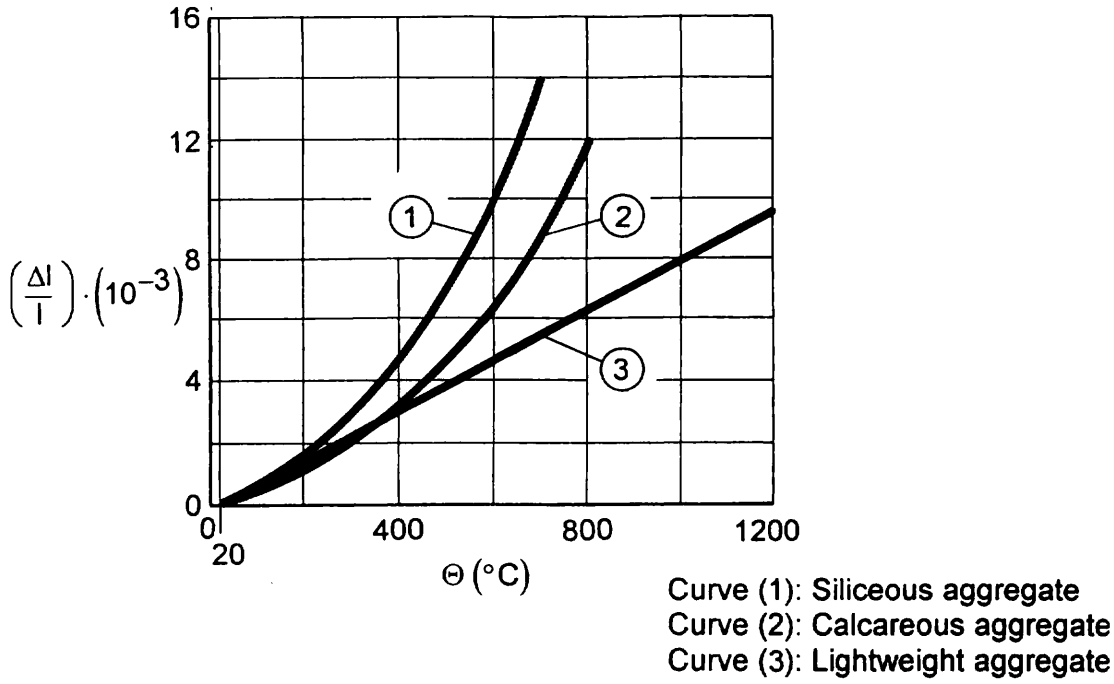


Fig. 6.5-3 Thermal elongation of concrete

Expansion Joints:

As a rough guide, the minimum width e of an expansion joint for fire should be e = 40 mm, at 30 m spacing.

The joint filling material should have sufficient durability and must be non-combustible; the sealing material may be combustible (Fig. 6.5-4).

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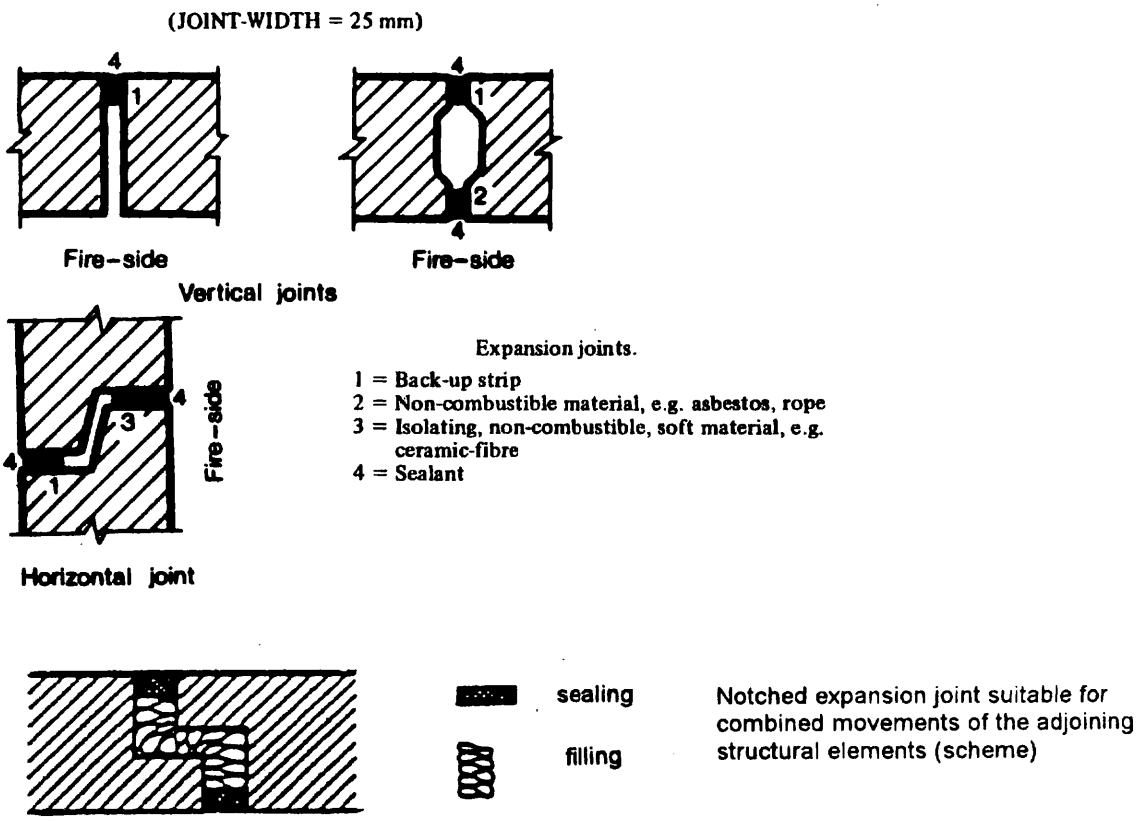


Fig. 6.5-4 a Examples of expansion joints [FIP/CEB, (1978)]

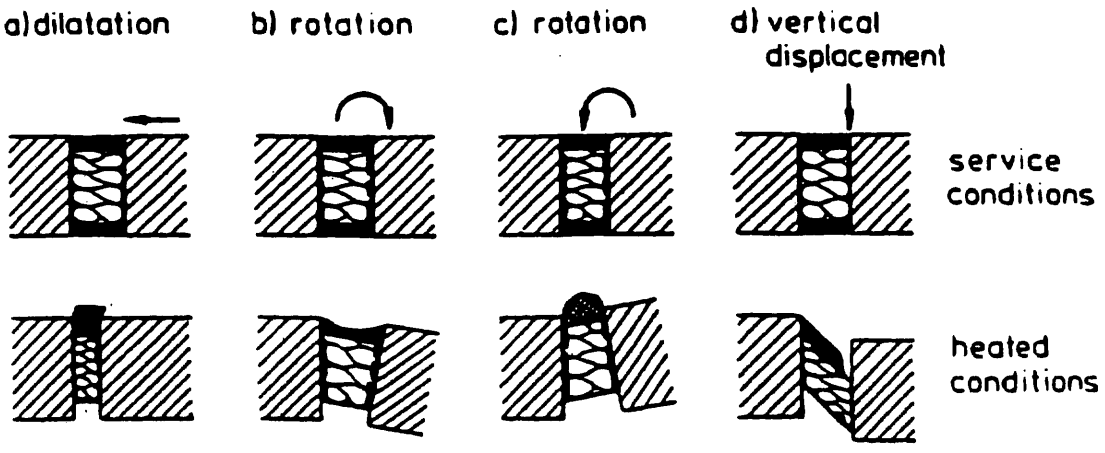


Fig. 6.5-4 b Expansion joints suitable for different movements of the adjoining structural elements [CEB (1991)]

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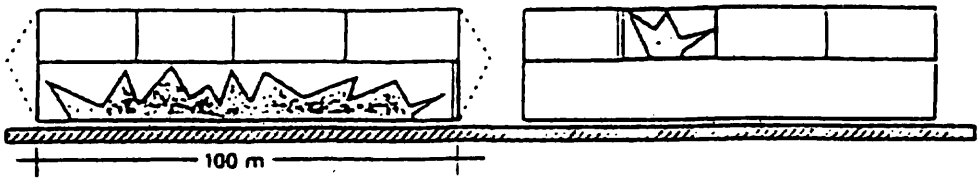


Fig. 6.5-5 Different situations concerning compartmentation [Kordina/Kersken-Bradley (1996)]

Further issues regarding expansion joints include the following. In Fig. 6.5-5 the top storey of the building is separated into relatively small fire compartments, while the lower storey has just one large compartment. It is interesting to compare the effects of a fire in the lower storey and in a compartment in the upper storey. If the average increase in temperature in the slab is 200 °C the potential expansion is in the order of 300 mm in the slab above the ground floor, but only about 80 mm for the roof slab. These figures ignore constraints and effects of curvature. Irrespective of the exact expansion which occurs, expansion joints are clearly more important in the slab above the ground floor than in the roof slab.

We compare now two different configurations for the horizontal constraints acting in the floor-slabs (Fig. 6.5-6). In one case there is a single stairwell in the middle of the building, in the other case there are two stairwells, one at either end of the building. The first arrangement clearly provides the minimum constraint to the slab and allows maximum expansion. The second case gives maximum constraint and minimum expansion. Disregarding the rest of the structure, no expansion joint would be required in the first case, whereas expansion joints would be needed in the second case to limit the compressive forces in the slab and the resultant forces on the stairwells. It should be noted that expansion joints in separating walls and slabs need special detailing.

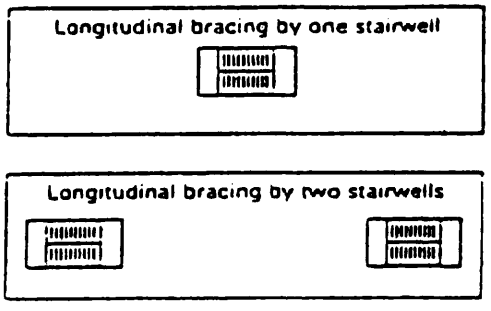


Fig. 6.5-6 Effect of horizontal restraint

Considering the columns supporting the slab in the case of the central constraint provided by the staircase, severe displacement at the column heads is likely to occur, as in Fig. 6.5-7, with horizontal forced acting on the columns. Expansion joints would be needed to protect the columns. On the other hand, in the design with stairwells at each end, the columns would be virtually unaffected, irrespective of the presence of expansion joints.

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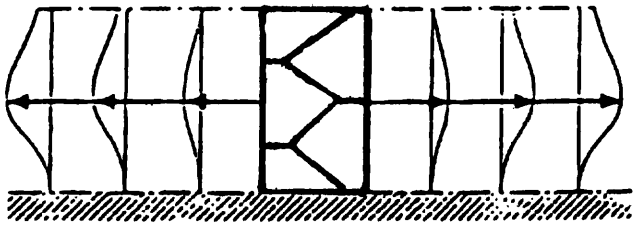


Fig. 6.5-7 Effect of central stairwell on surrounding columns (no expansion joint)

Tension members which are used for bracing or for supporting other members fail to fulfil their function when they are exposed to fire and expand. They need to become protected.

When exposed to fire a simply supported beam expands and also deflects because it is heated on three sides (Fig. 6.5-1). After cooling down, the expansion may disappear, but the deflection remains. The beam may slip off a support which is not large enough.

A severe fire can produce the temperature strains and gradients as shown in Fig. 6.5-2 in a slab and in Fig. 6.5-8 in a wall.

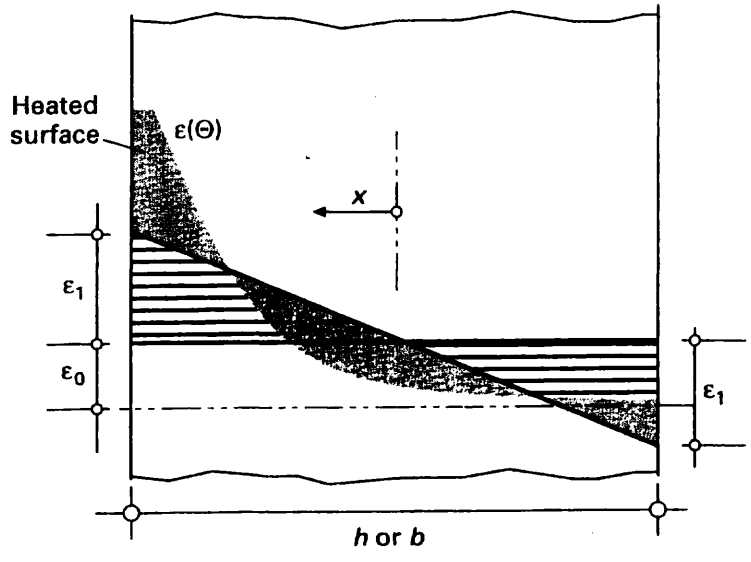


Fig. 6.5-8 Temperature-dependent strains [Kordina/Kersken-Bradley (1996)]

The temperature gradient produces internal stresses as well as curvature in the wall if exposed to a fire on one side. Strictly, expansion and curvature should be calculated on the basis of equilibrium of the internal stresses.

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If a linear stress-strain relation is assumed and the dependency of the modulus of elasticity on temperature is ignored, then the resulting stress estimates are on the safe side. The error is larger for curvature than for expansion. If a non-loadbearing wall is b = 250 mm thick equilibrium occurs with strains of around $\varepsilon_l \pm 0.004$. This results in a radius of curvature of only about 30 m when calculated from $l/R = 2\,\varepsilon_l/b$.

Nevertheless, the full benefit of a large b value (equal to the column width b_c) is only achieved if the columns are on the cooler side of the wall, and not exposed to fire on three sides. Without systematic investigation, only the following tentative guidelines can be given.

- If the wall separates-areas with high fire loads on one side and low fire load on the other, then the columns should be located in the low fire-load area.
- If this is not possible, protective layers can be applied to the column, so that full exposure only occurs on one side (this is not a typical "concrete" solution).
- The column should be designed and braced in such a way that plastification would occur at the column foot prior to failure of the bracing.

6.5.4 Compartmentation

Fire damage is mostly due to inadequate compartmentation, rather than to poor fire design of individual floors and walls, to achieve the separating function. Providing fire resistance for concrete members as such is usually not very expensive, but where compartmentation requires openings in walls and floors to be secured (doors, ducts, shutters) the cost may be considerable and the result may turn out to be useless because of elementary mistakes made on site and during use, as well as in design.

Frequent mistakes are geometrically incomplete compartments in areas where the geometry of the structure is not straightforward, and in particular where the possibility of fire propagation across facades and roofs has not been adequately considered (Fig. 6.5-9). Serious mistakes can be encountered in the design of air-conditioning systems and building services in general. This is mostly due to lack of communication between the structural designer and the designers of the building services.

The compartmentation as originally planned is often obstructed during use. The best known example is the fire door which is forced to remain open to provide easy access. In buildings where electric or electronic cables are newly or repeatedly installed, it is common to forget to close the seals in compartment walls and floors. Maintenance of devices such as fire shutters is often not performed. Finally, in the course of time, the original compartmentation concept is simply forgotten, with the result that new installations such as ventilation ducts and transport systems are introduced without provision for the automatic barriers needed for fire.

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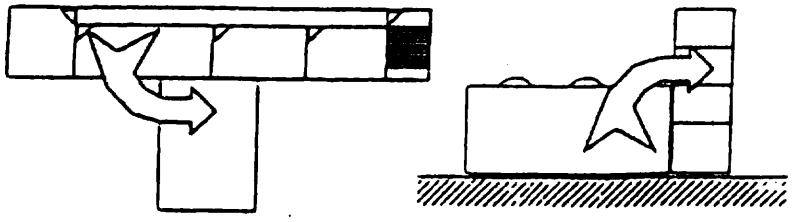


Fig. 6.5-9 Fire penetration from one part of the building to another part [Kordina/Kersken-Bradley (1996)]

Windows in facades are always a problem with regard to the horizontal and vertical compartmentation of a building. Breaching of fire compartments via windows is most effectively inhibited by fire-resistant parapets and balconies. Experience shows that traditional wall-window construction is also adequate, provided:

- fire-resistant parapets are used;
- the flashover distance is at least 1 m;
- no combustible materials are used in the construction of the facade.

Nevertheless, with increasing building height, fire brigades are less likely to control fire propagation along the facade, so that flashover distances need to be increased for high-rise structures.

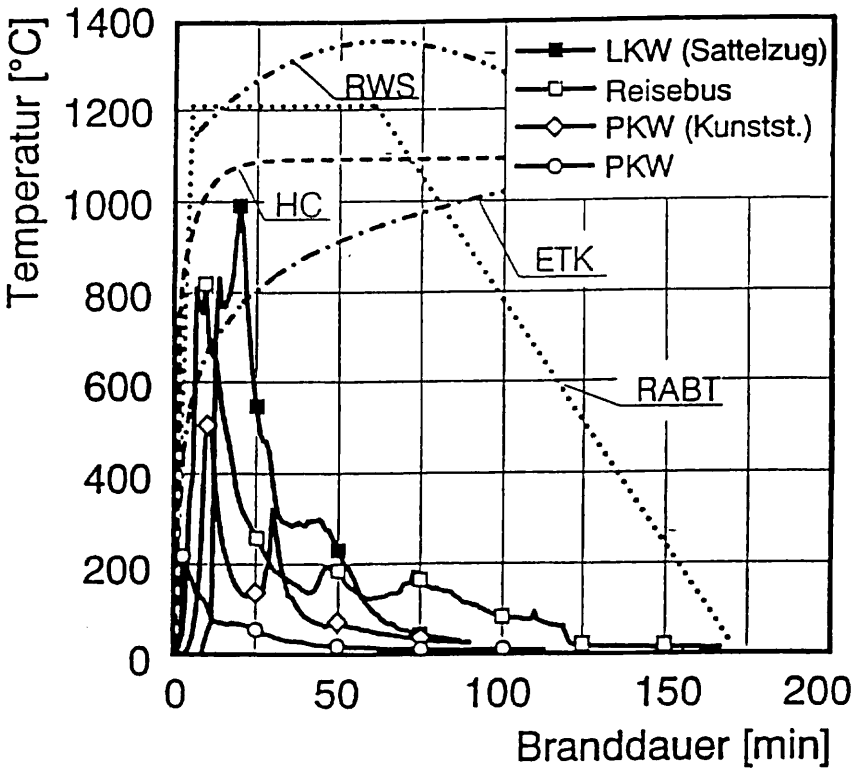
6.5.5 Fires in tunnels

Fires in a road or railway tunnel have two different dangerous aspects:

On the one hand may occur heavy failures in the concrete lining for instance by explosive spalling, on the other hand hot and toxic gases and dense smoke can not escape in the free atmosphere and can become in this way a heavy handicap to the tunnel users to find the escape routs. The gas-temperature during a tunnel fire goes up to 1000 - 1200 °C within about 5 min as shown in Fig. 6.5-10, which also shows the temperature-time function according to different standards for designing tunnel linings [EUREKA-Bericht (1996)].

The wholes in concrete linings by spalling may reach 40 cm and more so that the rocks behind the lining become visible and the water-tightness of the tunnel structure may be lost. Measures against spalling under ISO-834 fire have been found very recently by admixtures of plastic fibres of about 3 kg/m^3 [Diederichs/Jumppanen/Schneider (1995)]. Steeper increase of temperature as by ISO-834 should be investigated. Temperatures in the tunnel structure as well as restraining stresses in a fire can produce deformations and cracks for instance in a hollow bridge used for a subway or in the corners of a frame-shaped subway-tunnel [Haksever/Kordina (1981)].

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ETK	=	Einheitstemperaturzeitkurve DIN 4102 = ISO-834-Curve
HC	=	Hydrocarbon-Curve (DIN EN 1363 Teil 2)
RABT	=	Instruction on Equipment and use of Tunnel (Richtlinie für Ausstattung und Betrieb von Tunneln)
RWS	=	Rijks Waterstraat, NL

Fig. 6.5-10 Maximal temperatures in tunnels

Smoke and hot gases must be removed from the burning vehicles as quick as possible; longitudinal ventilation may be sufficient until tunnel length of about 1500 m; longer tunnels should get in addition a transverse ventilation for instance by an additional floor in the upper region of the tube with openings to the traffic way, where ventilation suck away the smoke. The openings should open above the burning vehicles automatically and very swift [Kordina (1998), Kordina (1997)].

6.6 Damage caused by fire exposure

Reparability means the ability of a structure or a part thereof, to be restored to an agreed level of safety, in practical use comparable to that before the fire occurred. Its judgement in most cases will be influenced by economic considerations, but under specific circumstances, high financial expenses for the repair may be unjustifiable [CEB (1991)]. The evaluation of a structure after a fire is very similar when plans and material data are missing. Drilling of concrete cores, testing of samples of the reinforcement are the most important measures. Concrete surfaces after spalling have to be controlled, if they are free of cracks; see also point 6.6.4.

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More detailed information about the reparability of damaged structures may be taken from the literature [CEB (1991), Schneider/Nägele (1989), Warner/Ranyan/Hall (1989)]. The following chapters only give an introduction into this problem.

6.6.1 Circumstances and indications at the place of the fire

Judgement of reparability should start with the attempt to obtain an overall image of the event, the fire.

A visit to the place of the fire should be undertaken by an expert in fire design as soon as it seems possible without endangering any persons.

There are a number of indicators pointing to the fire severity, such as:

- charring depth of wooden elements,
- burnt away PVC (polyvinyl chloride) cable insulation (> 300 °C),
- blue heat tinting of steel surface (< 400 °C),
- melted aluminium (> 650 °C),
- melted usual silicate glass (> 1000 °C)
- sintered brickwork (> 1200 °C),
- net of cracks in the concrete surface (depth ca. 2 cm at ca. 1000 °C),
- unburned fire load.

Not only the specific damages of the affected structural members, such as deformations, cracks, spalling and disintegration of concrete etc., but the overall condition of the structure and the building should be carefully recorded.

6.6.2 Material damage

Residual mechanical properties of the materials used in concrete structures, after cooling depend on a number of parameters, such as:

- kind of steel (reinforcing, prestressing), hot rolled, cold worked, tempered,
- maximum temperature attained,
- stress-level during the manufacturing process,

and with lower importance:

- rate of heating and cooling (if not chilled by a hose stream),
- duration of maximum temperature level.

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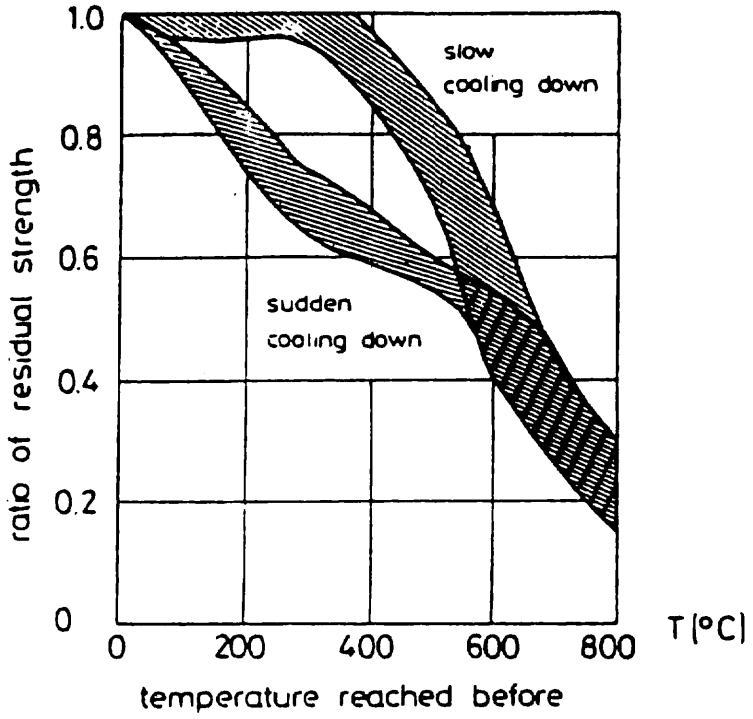


Fig. 6.6-1 Residual compressive strength of normal dense concrete as a function of the temperature reached and the cooling conditions [CEB (1991)]

Therefore, the following data and reference publications should only be regarded as basic information for a rough estimation of possible damages. They cannot compensate individual structural investigations, even more if the temperatures reached during the fire cannot be assessed with adequate reliability.

The residual compressive strength of concrete is normally not severely reduced if temperatures up to 300 °C have not been exceeded, and no sudden cooling down, e.g. by extinguishing water, has occurred (see Fig. 6.6-1). Concrete heated by more than 100 - 150 °C may loose its protective behaviour against corrosion. Application of additional layers could be necessary.

Hot rolled reinforcing steel recovers its original yield stress if the temperature does not exceed approx. 700 °C. With cold worked steel, the workhardening effect which increases the strength of such types of reinforcement under normal temperature, suffers regression if exposed to temperatures > 400 °C. The favourable influence of aftertreatments on the strength of prestressing steels decreases even if lower temperatures (ca. 300 °C) are reached. These reductions are not fully recovered (see Fig. 6.6-2 and 6.6-3).

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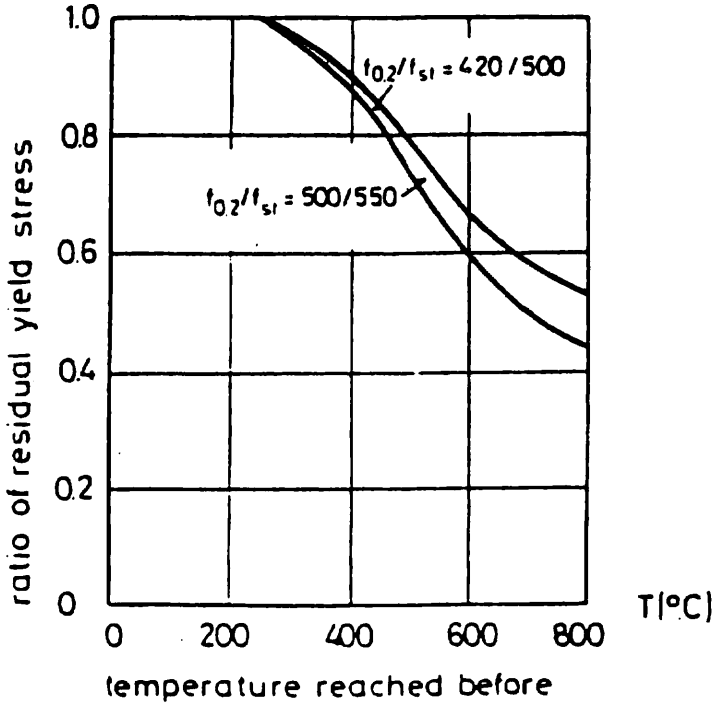


Fig. 6.6-2 *Residual yield strength after heating of cold-worked reinforcing steel; Exposure time to the maximum temperature 30 minutes [CEB (1991)]*

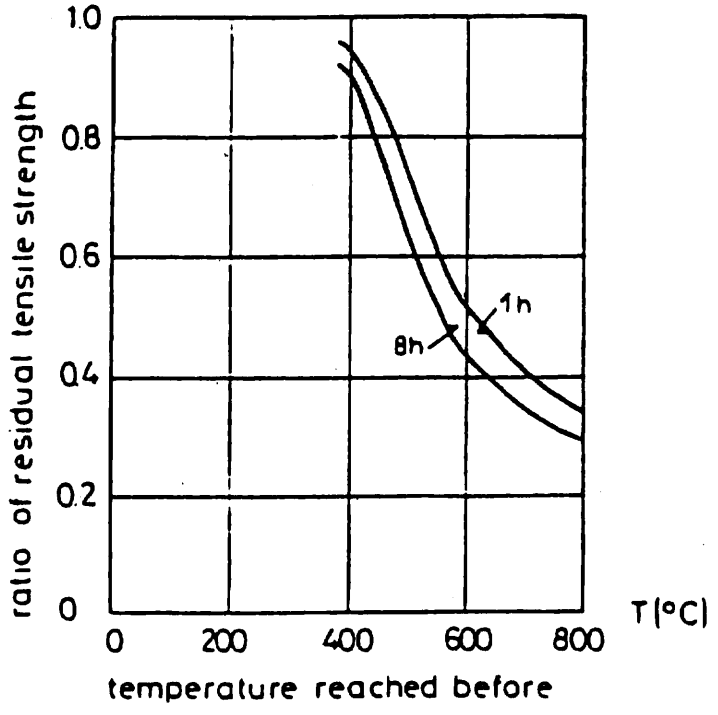


Fig. 6.6-3 *Residual tensile strength after heating of high strength prestressing steel (US-American); exposure time to the maximum temperature 1 or 8 hours [CEB (1991)]*

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6.6.3 Structural damage

The bond between concrete and reinforcement may be affected by the heating process. The residual bond strength does not only depend on the temperature reached and on the exposure time, but in addition on the condition of the steel surface.

The prestress which possibly decreases during a fire, can partly be regained during the cooling down period. Fig. 6.6-1 shows results derived from tests with high tensile prestressing wires of 9 mm diameter.

Cracks occur in any reinforced or prestressed concrete element when exposed to a heating, extinguishing and cooling down process. There are several reasons causing this phenomenon, such as:

- deformation due to mechanical influences and the thermal conditions,
- the temperature gradient in the cross section,
- the non-linearity of the temperature gradient,
- different temperature rise at the change of the size and shape of a cross section,
- different thermal elongation of the cement matrix and the aggregates,
- different thermal elongation of the concrete and the reinforcing steel,
- restraints of the re-contraction during the cooling-down process,
- sudden cooling-down by a fire extinguishing hose stream.

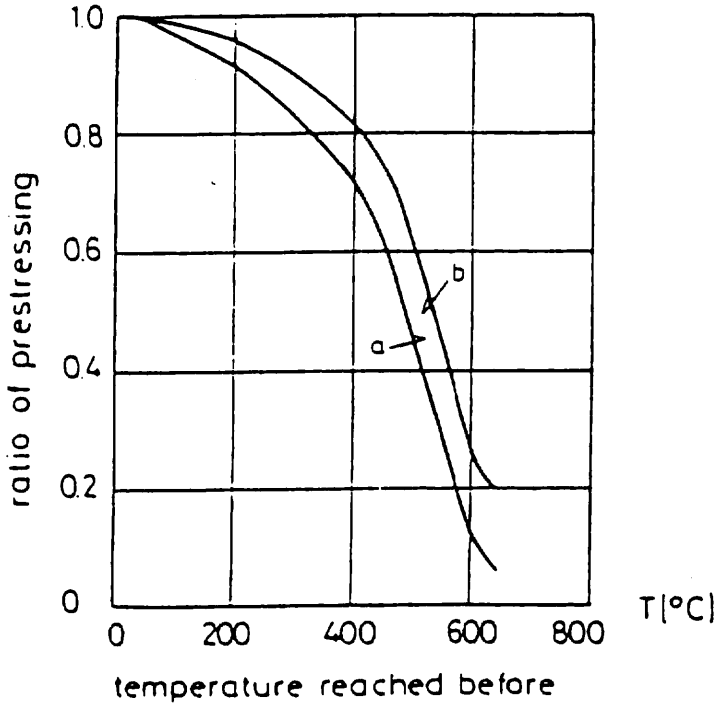


Fig. 6.6-4 Decrease of prestressing force under elevated temperature (a) and after cooling down (b)

The kinds of cracks described above only concern the directly affected concrete structure. But, if monolithic structures are concerned, thermal deformation of structural members exposed to a fire cause stresses in the surrounding assembly, which may not be directly

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effected by the fire. Therefore, cracks may occur in these unaffected regions. This fact has to be taken into account, mainly when the damage of the fire exposed part of the structure indicates a severe fire attack.

Explosive Spalling (see 6.2.3)

Residual deformations of the load-bearing concrete structure depend not only on the intensity of the fire attack but also on the static system involved. The decision to which degree these deformations may be tolerated must be regarded in connection with the system and in addition with other criteria such as the overall appearance etc.

In case of a fire, corrosive decomposition products may be released which can penetrate into concrete cross-sections and affect the reinforcement. Above all, this danger occurs if larger portions of polyvinylchloride have been involved in the fire. It should be taken into account that long term damages can be initiated if the transport of chloride into deeper areas of the concrete is not prevented by appropriate means as early as possible. The highest amount of chloride may be found in areas not severely affected by the fire.

6.6.4 Determination of the degree of deterioration, methods of repair

The identification of the severity and degree of possible deterioration of a concrete structure after a fire, requires the same test methods, instrumentation and techniques as used for damages caused by other influences. It should be considered however, that a careful examination of the residual properties of reinforcing or prestressing members may be necessary, even if they do not show an obvious damage [CEB (1991), Harmathy (1986), Schneider/Nägele (1989)].

Methods of repair after a fire are similar to those used for repair measures for other causes. In the case that the restored building has to fulfil fire resistance requirements, it has to be considered that the repair measures do not impair the fire resistance of the structure.

Recalculation of the load-bearing capacity of the repaired concrete structure may be necessary.

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Annex to chapter 6

(A1) Values given in the Tables provide minimum dimensions for fire resistance in addition to the detailing rules required by ENV 1992-1-1. It must be stated, that after the fire resistance time taken as a basis for design a failure of the member in question may occur. Some values of the axis distance of the steel, used in the tables are less than that required by ENV 1992-1-1 and should be considered for interpolation only.

Linear interpolation between the values given in the tables is allowed:

Symbols used in the tables are defined in Fig. 6-A 1.

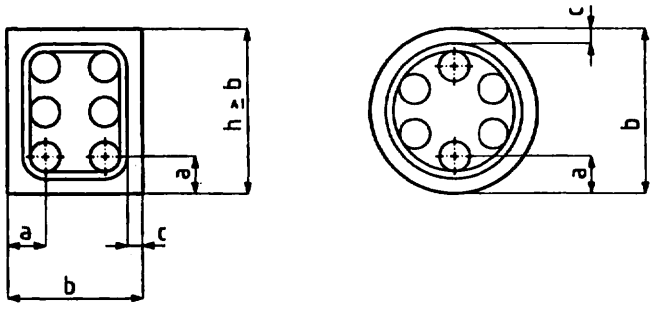


Fig. 6-A 1: Sections through structural members, showing nominal axis distance a, and nominal concrete cover c to reinforcement

The nominal values of axis distance a to a steel bar, wire or tendon, should not be less than the minimum values given in the Tables below.

The minimum axis distance for any individual bar should not be less than that required for R 30 and not less than half the average axis distance (see Equation (4.5)).

In tensile members, beams and slabs with concrete covers $c \geq \underline{50}$ mm to the main longitudinal reinforcement, surface reinforcement should be provided in order to prevent the fall off of the concrete unless it can be justified, generally by tests, that falling off does not occur within the period of fire resistance. Where necessary, for the cover so surface reinforcement reference should be made to ENV 1992-1-1, 4.1.3.3 (6) and (7).

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(A 2) Columns

Fire resistance of reinforced concrete columns may be satisfied by the use of Table 6-A 1 and the following rules.

In Table 6-A 1 a load level in the fire situation 1_{fi} has been introduced accounting for load combinations and the design column resistance to compression and, possibly, bending, including second order effekts. The effective length l_0 is assumed to be equal to the actual column length l_{col} (notation as in EVN 1992-1-1, 4.3.5).

Standard fire resistance	Minimum dimensions (mm) Column width b_{min}/axis distance a			
	Column exposed on more than one side			Exposed on one side
	μ_{fi} = 0,2	μ_{fi} = 0,5	μ_{fi} = 0,7	μ_{fi} = 0,7
1	2	3	4	5
R 30	|150/10|*	|150/10|*	|150/10|*	|100/10|*
R 60	|150/10|*	|180/10|*	|200/10|*	|120/10|*
R 90	|180/10|*	|210/10|*	|240/35|	|140/10|*
R 120	|200/40|	|250/40|	|280/40|	|160/45|
R 180	|240/50|	|320/50|	|360/50|	|200/60|
R 240	|300/50|	|400/50|	|450/50|	|300/60|
* Normally the cover required by ENV 1992-1-1 will control.				

Table 6-A 1: Minimum dimensions and axis distances for reinforced concrete columns; Rectangular and circular section

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Load-bearing solid walls

Adequate fire resistance of load-bearing reinforced concrete walls exposed on one side may be assumed if the following datas are applied.

REI 30 to 90: Cover as required by END 1992-1-1 thickness $\leq$ 120 mm.

(A 3) Beams

(1) General

Adequate fire resistance of reinforced and prestressed concrete beams may be assumed if the data given in tables 6-A 2 to 6-A 4 together with the following rules are used.

The Tables apply to beams which can be exposed to fire on three sides, i.e. the upper side is insulated by slabs or other elements which continue their insulating function during the whole fire resistance period.

For beams with varying width the minimum value b relates to the centroid of the tensile reinforcement.

Holes through the webs of beams do not affect the fireresistance provided that the remaining cross-sectional areal of the member in the tensile zone is not less than $A_c = 2b^2$.

(2) Simply supported beams

Table 6-A 2 provides minimum values of axis distance to the soffit and sides of simply supported beams together with minimum values of the width of beam, for standard fire resistances of R 30 to R 240.

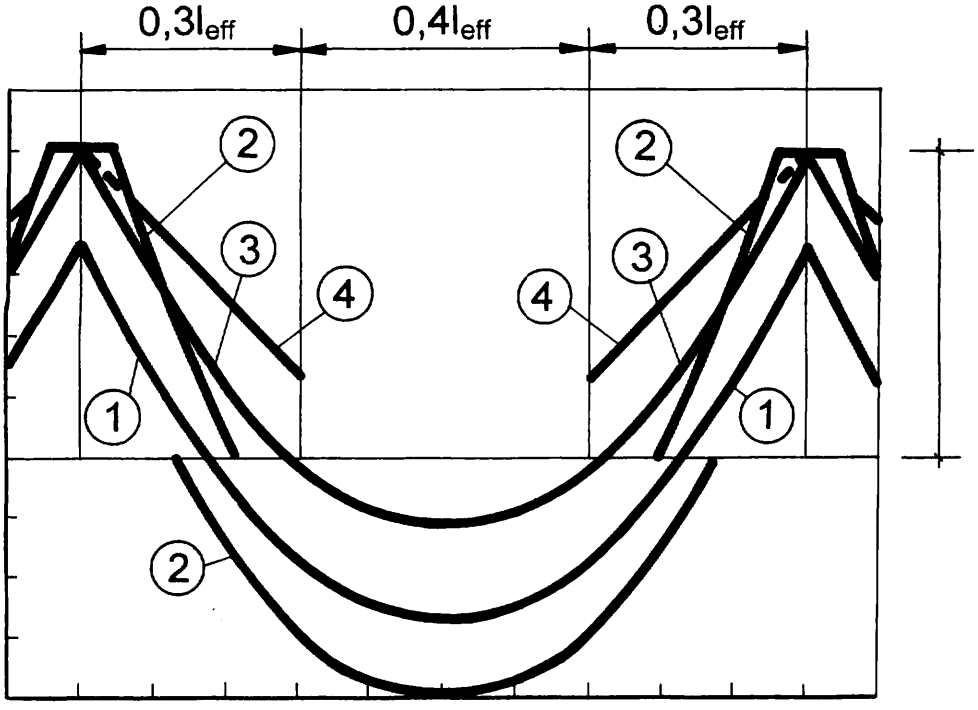
(3) Continuous beams

Table 6-A 3 provides minimum values of axis distance to the soffit and sides of continuous beams together with minimum values of the width of beam, for standard fire resistance of R 30 to R 240.

Table 6-A 3 and the following ruled apply for beams where the moment redistribution according to ENV 1992-1-1, 2.5.3.4.2 does not exceed 15 %. In the absence of a more rigorous calculation and where the redistribution exceeds 15 %, or the detailing rules of this Part 1-2 are not fololowed, each span of a continuous beam should be assessed using Table 6-A 2 for simple supported beams.

The area of top reinforcement over each intermediate support for standard fire resistance of R 90 and above, for up to a distance of 0,3 l_{eff} (as defined in ENV 1992-1-1, 2.5.2.2.2) from the centre line of support should not be less than (see Fig. 6-A 2):

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Explanation:

(1) Diagram of bending moments for the actions in a fire situation at t = 0
(2) Envelope line of acting bending moments to be resisted by tensile reinforcement according to ENV 1992-1-1
(3) Diagram of bending moments in fire conditions
(4) Envelope line of resisting bending moments

Fig. 6-A 2: Envelope of resisting bending moments over supports in fire conditions

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Standard fire resistance	Minimum dimensions (mm)				
	Possible combinations of a and b_{min} where a is the average axis distance and b_{min} is the width of beam				Web thickness b_w
1	2	3	4	5	6
R 30	b_{min} = | 80 | a = | 25 |	| 120 | | 15 | *	| 160 | | 10 | *	| 200 | | 10 | *	| 80 |
R 60	b_{min} = | 120 | a = | 40 |	| 160 | | 35 |	| 200 | | 30 |	| 300 | | 25 |	| 100 |
R 90	b_{min} = | 150 | a = | 55 |	| 200 | | 45 |	| 250 | | 40 |	| 400 | | 35 |	| 100 |
R 120	b_{min} = | 200 | a = | 65 |	| 240 | | 55 |	| 300 | | 50 |	| 500 | | 45 |	| 120 |
R 180	b_{min} = | 240 | a = | 80 |	| 300 | | 70 |	| 400 | | 65 |	| 600 | | 60 |	| 140 |
R 240	b_{min} = | 280 | a = | 90 |	| 350 | | 80 |	| 500 | | 75 |	| 500 | | 70 |	| 160 |
a_{sd} = a + 10mm (see note below)					

For prestressed beams the increase of axis distance according to 4.2.2(4) should be noted.

a_{sd} is the axis distance to the side of beam for the corner bars (tendon or wire) of beams with only one layer of reinforcement. For values of b_{min} greater than that given in Column 4 no increase of a is required.

* Normally the cover required by ENV 1992-1-1 will control.

Table 6-A 2: Minimum dimensions and axis distances for simply supported beams made with reinforced and prestressed concrete

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Standard fire resistance	Minimum dimensions (mm)			
	Possible combinations of a and b_{min} where a is the average axis distance and b_{min} is the width of beam			Web thickness b_w
1	2	3	4	5
R 30	b_{min} = | 80 | a = | 12 | *	| 160 | | 12 | *	| 200 | | 12 | *	| 80 |
R 60	b_{min} = | 120 | a = | 25 |	| 200 | | 12 | *	| 300 | | 12 | *	| 100 |
R 90	b_{min} = | 150 | a = | 35 |	| 250 | | 25 |	| 400 | | 25 |	| 100 |
R 120	b_{min} = | 220 | a = | 45 |	| 300 | | 35 |	| 500 | | 35 |	| 120 |
R 180	b_{min} = | 380 | a = | 60 |	| 400 | | 60 |	| 600 | | 50 |	| 140 |
R 240	b_{min} = | 480 | a = | 70 |	| 500 | | 70 |	| 700 | | 60 |	| 160 |
$a_{sd} = a$ + 10mm (see note below)				

For prestressed beams the increase of axis distance according to 4.2.2(4) should be noted.

a_{sd} is the axis distance to the side of beam for the corner bars (tendon or wire) of beams with only one layer of reinforcement. For values of b_{min} greater than that given in Column 3 no increase of a is required.

* Normally the cover required by ENV 1992-1-1 will control.

Table 6-A 3: Minimum dimensions and axis distances for continuous beams made with reinforced and prestressed concrete

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(A 4) Simply supported slabs

Table 6-A 4 provides minimum values of axis distance to the soffit of simply supported slabs for standard fire resistance of R 30 to R 240.

In two-way spanning slabs, a denotes the axis distance of the reinforcement in the lower layer.

Standard fire resistance	Minimum dimensions (mm)			
	slab thickness h_s (mm)	axis-distance *a*		
		one way	two way: $l_y/l_x \le 1,5$	two way: $1,5 < l_y/l_x \le 2$
1	2	3	4	5
REI 30	|60|	|10| *	|10| *	|10| *
REI 60	|80|	|20|	|10| *	|15| *
REI 90	|100|	|30|	|15| *	|20|
REI 120	|120|	|40|	|20|	|25|
REI 180	|150|	|55|	|30|	|40|
REI 240	|175|	|65|	|40|	|50|

l_x and l_y are the spans of a two-way slab (two directions at right angles) where l_y is the longer span.

For prestressed slabs the increase of axis distance according to 4.2.2(4) should be noted.

The axis distance *a* in Column 4 and 5 for two way slabs relate to slabs supported at all four edges. Otherwise, they should be treated as one-way spanning slab.

* Normally the cover required by ENV 1992-1-1 will control.

Table 6-A 4: Minimum dimensions and axis distances for reinforced and prestressed concrete simply supported one-way and two-way slabs

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The values given in Table 6-A 4 (Columns 2 and 4) also apply to one-way or two-way continuous slabs.

(A 5) Flat slabs

The following rules apply to flat slabs where the moment redistribution according to ENV 1992-1-1, 2.5.3.5.4 does not exceed 15 %. Otherwise axis distances should be taken as for one-way slab (Column 3 in Table 6-A 4) and the minimum thickness from Table 6-A 5.

For fire ratings of REI 90 and above, at least 20 % of the total top reinforcement in each direction over intermediate supports, required by END 1992-1-1, should be continuopus over the full span. This reinforcement should be placed in the column strip.

Minimum slab-thicknesses should not be reduced (e.g. by taking floor finishes into account).

The axis distance a denotes the axis distance of the reinforcement in the lower layer.

Standard fire resistance	Minimum dimensions (mm)	
	slab-thickness h_s	axis-distance a
1	2	3
REI 30	|150|	|10|*
REI 60	|200|	|15|*
REI 90	|200|	|25|
REI 120	|200|	|35|
REI 180	|200|	|45|
REI 240	|200|	|50|
* Normally the cover required by ENV 1992-1-1 will control.		

Table 6-A 5: Minimum dimensions and axis distances for reinforced and prestressed concrete flat slabs

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7 Members design

The members design is a synthesis process which requires the application of models and procedures presented within the previous chapters. Traditionally two different classes of structural members may be recognised: linear members and two-dimensional members; as a consequence, in the following, one analyses two different structures which may be considered as composed respectively by linear and two-dimensional members. Of course this distinction is not a sharp frontier, because in several cases two-dimensional structures may be designed as linear one and vice versa sometime a linear member may require a two dimensional approach.

7.1 Linear members

by G. Mancini

As linear member a reinforced concrete frame on elastic ground is analysed; the actual structure from which the frame is derived, is described in the following point.

7.1.1 Description of the structure

The analysed structure consists of a concrete box-culvert 22.10 m long, undercrossing with a right angle a railway line for high speed trains and built to grant the free flow of a stream. The railway consists of a double tracks spaced of 5.0 m over a bed composed by ballast, sleepers and rails having an overall thickness of 80 cm.

A plan view of the structure is pictured in Fig. 7.1 - 1, where the marked lines represent the culvert dimensions. Two retaining walls are present at the extremities of the culvert in order to contain the railway back-fill.

In Fig. 7.1 - 2 are reported the longitudinal section A-A and the transverse section B-B of the culvert. The structure has a longitudinal gradient to allow the stream flowing; furthermore, a poor concrete bed is provided under the foundation, with a thickness of 20 centimetres. It can be notice that the upper slab of culvert is 2.50 m deep under the ballast level, so that an average earth cover has been considered in the calculations.

The following material properties have been considered:

- Concrete Grade 30:
  compressive design strength: f_{cd} = 20.0 MPa;
  ompressive resistance for uncracked zones: f_{cd1} = 15.0 MPa;
  compressive resistance for cracked zones: f_{cd2} = 10.6 MPa;
  mean value of tensile strength: f_{ctm} = 2.9 MPa;
  modulus of elasticity: E_c = 29.0×10^3 MPa;
- Reinforcing steel, Grade 500:
  design strength: f_{yd} = 434.8 MPa;
  modulus of elasticity: E_s = 200.0×10^3 MPa;
- Soil: internal friction angle: ϕ = 30°;
  modulus of subgrade reaction (vertical): K_v = 20 N/cm^3;
  modulus of subgrade reaction (horizontal): K_h = 10 N/cm^3;
  unit weight: γ = 19 kN/m^3.

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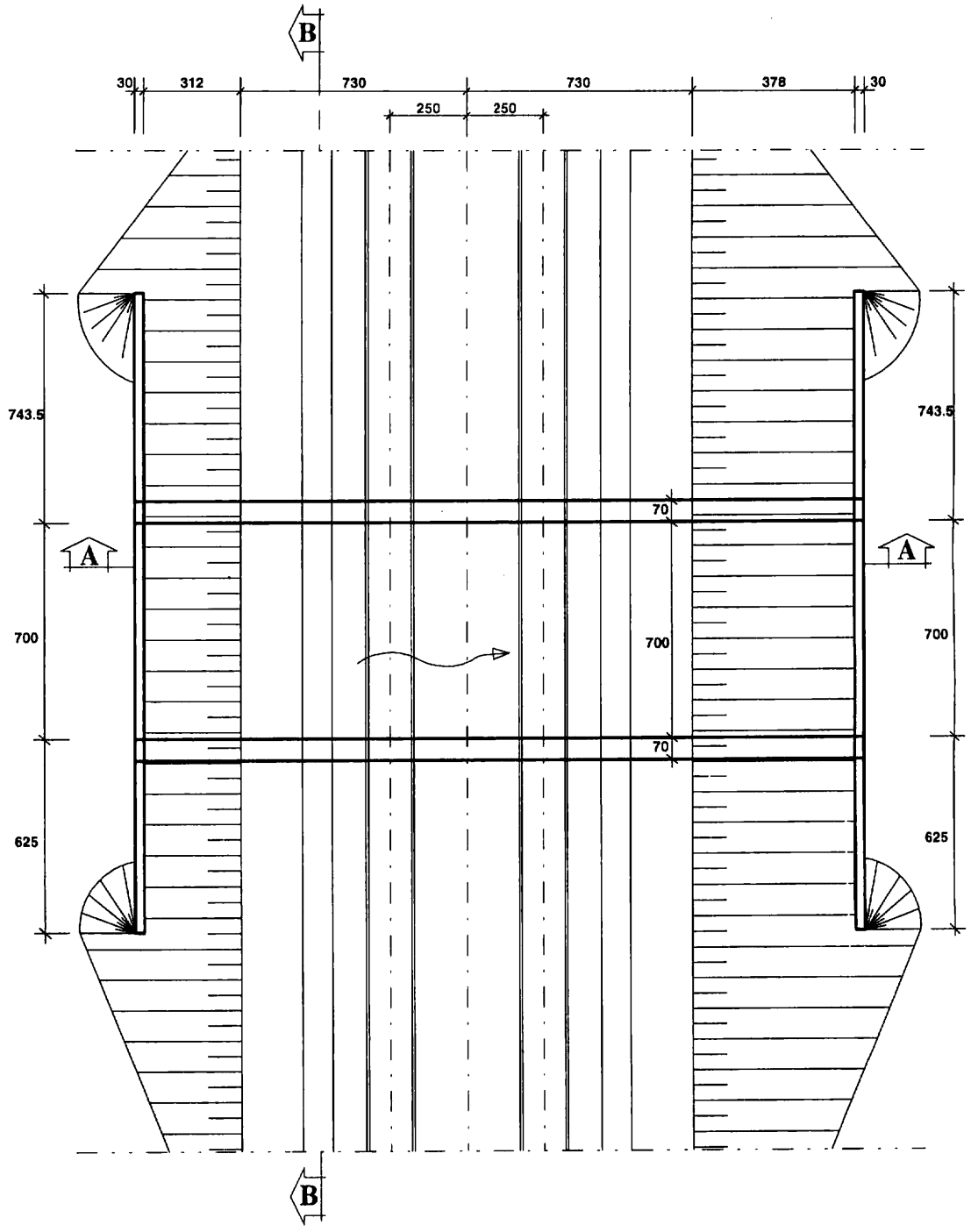


Fig. 7.1 - 1: Plan view of the structure

(1) Concrete cover

As environmental condition an Exposure Class 2 may be considered (Humid environment without frost: components in non-aggressive soil).

The minimum concrete cover for Class 2 is equal to 25 mm, which should be added to the tolerance value of 10 mm; as a consequence the nominal value for concrete cover results:

$c_{nom} = c_{min} + 10 = 25 + 10 = 35$ mm

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adopted in the calculations.

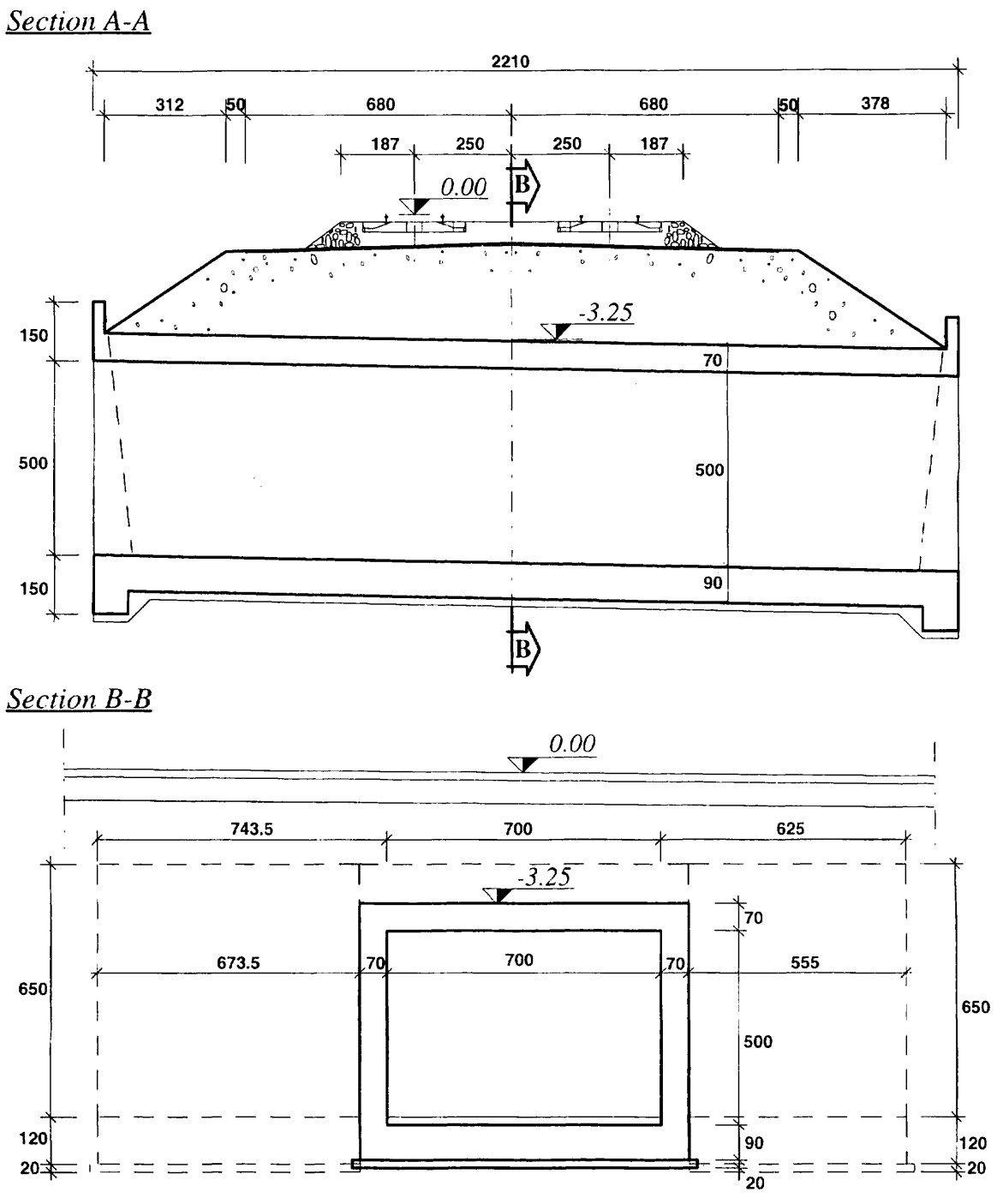


Fig. 7.1 - 2: Longitudinal and cross section of the culvert

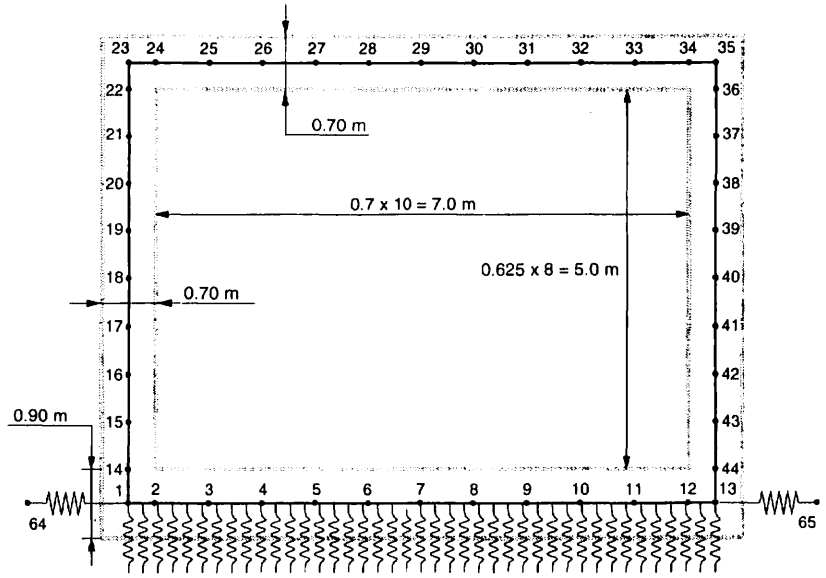
7.1.2 Structural model

A linear-elastic FEM analysis has been performed in order to evaluate the internal actions on the structure produced by the applied loads. Due to earth cover, the train load is spread into the soil in both longitudinal and transverse directions so that a distributed load is acting on the upper slab of the culvert; consequently, calculations may be referred to a transverse strip having 1.0 m of width, adopting a plane frame to represent the culvert; so the two-

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dimensional problem may be reduced to a linear one.

a) Nodes



b) Elements

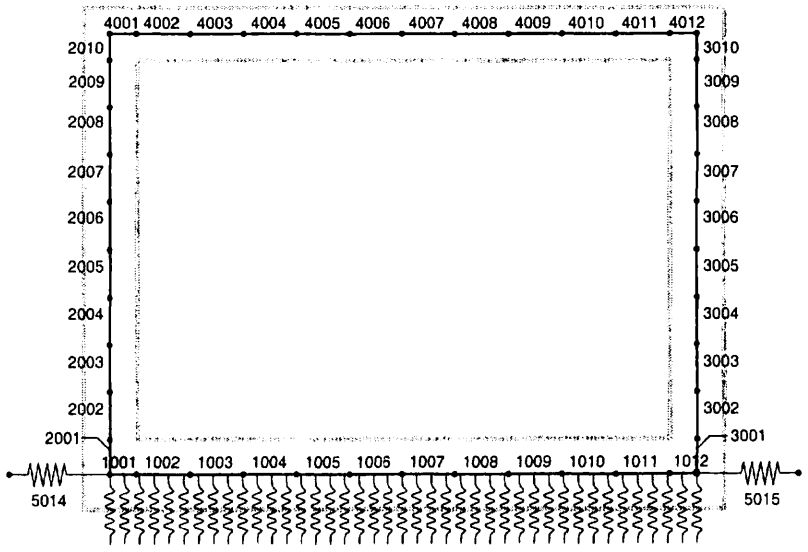


Fig. 7.1 - 3: FEM model

Fig. 7.1 - 3 represents the adopted mesh, with both nodes and elements. The slab, the uprights and the foundation are modelled by means of beam elements with area and inertia characteristics related to a rectangular section of unit breadth and height coincident with each respective concrete thickness. The previous quantities are resumed in Table 7.1 - 1.

The model may be restrained by means of spring elements representing the deforming ground, or rested on a continous elastic bed; in this second case, of course, a B.E.F. (Beam on Elastic Foundation) element should be used and its elastic parameter is evaluated as product of K_v with the foundation breadth. If the spring elements are adopted, their elastic constant is calculated by the moduli of subgrade reaction; for the vertical restraints two different collaborating widths are assumed: respectively 0.525 m and 0.70 m respectively for the

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external springs and for the internal ones.

Member	Area m^2	Inertia m^4
Upper Slab	0.700	0.02858
Uprights	0.700	0.02858
Foundation	0.900	0.06075

Table 7.1 - 1: Area and inertia of FEM beam elements

A spring element shall be used for the horizontal restraint; its elastic constant is related to the overall height of the culvert, and its value is obtained by multiplying the K_h value with an area of influence of 6.6×1.0 m. The elastic constants of each spring element (whether for vertical or horizontal restraints) are reported in Table 7.1 - 2., together with the subgrade constant of the B.E.F. element.

Element	Collaborating Area/Width [m^2 – m]	Elastic Constant [kN/m – kN/m^2]
Horizontal spring	6.600×1.0	6.60×10^3
Vertical spring (external)	0.525×1.0	10.50×10^3
Vertical spring (internal)	0.700×1.0	14.00×10^3
B.E.F.	1.000	20.00×10^3

Table 7.1 - 2: Spring constants

For calculations, the B.E.F. approach has been chosen.

7.1.3 Actions

In such a kind of structure several combinations of actions should be considered in agreement, for instance, with the requirements of Eurocode 1.3 Traffic Load on Bridges. For the sake of simplicity, in the following, one analyses just a combination of actions, considering only the dead weights, the live load due to trains, the lateral pressures exerted by earth and, of course, the subsequent subgrade reactions.

The external actions considered are loaded in the mathematical model in several different steps, as illustrated below:

- STEP 1: Self-weight: adopting a unit weight value of $\gamma = 24.5$ kN/m^3;
- STEP 2: Gravity load on upper slab due to earth cover and ballast:
  By considering the cover weight and the ballast presence with a nominal height of 0.8 m (unit weight: $\gamma_{ballast} = 18$ kN/m^3), the following uniformly distributed loads are obtained:
  $g_{soil} = 19 \cdot 2.5 \cdot 1.0 = 47.5$ kN/m
  $g_{ballast} = 18 \cdot 0.8 \cdot 1.0 = 14.4$ kN/m
- STEP 3: Lateral earth pressure at rest.
  This pressure is evaluated adopting a k_0 coefficient calculated as:
  $k_0 = 1 - \sin\phi = 0.5$

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A symmetric trapezoidal load diagram is obtained, acting on the two uprights with opposite versus. The top and bottom values are:

$p_{top} = 0.5 \cdot (47.5 + 14.4) = 30.95$ kN/m;

$p_{bottom} = 30.95 + 0.5 \cdot (19 \cdot 6.6 \cdot 1.0) = 93.65$ kN/m

- STEP 4: Train load.

As railway load on track, an uniformly distributed live load has been adopted deriving from Load Model 71 (LM71) of Eurocode 1.3 . This kind of load is composed by α×250 kN point loads spaced of 1.6 m each other; thus the corresponding uniform load value is obtained by (α = 1.1 for Italian National Code):

$q_{LM71} = 1.1 \cdot 250 / 1.6 = 171.88$ kN/m

A transverse spreading, with respect to the railway direction, may be evaluated to calculate the load acting on the upper slab of the culvert: starting from the sleeper width of 2.60 m, it may be considered a spread ratio of 4:1 within the ballast, of 2:1 within the earth cover and of 1:1 within the concrete up to the middle plane of the slab. So, the influence breadth results:

$b_{influence} = 2.6 + 2 \cdot (0.4/4 + 2.5/2 + 0.35/1) = 6.00$ m.

Having two trains on the rails and being spaced of 5.0 m, the uniform load on the slab turns out:

$q = 2 \cdot 171.88 / 6 = 57.3$ kN/m

This procedure may be certainly considered as conservative; in fact with a different approach the total live load may be distributed uniformly on a breadth of 3+5+3 = 11 m, taking into account the capability of the upper slab to distribute the load transversally.

For the dynamic allowance, according to the Code indication, a coefficient of 1.3 has been adopted. Finally, the value of train load results:

$q_{train} = 1.3 \cdot 57.3 = 74.50$ kN/m

- STEP 5: Lateral earth pressure from vertical live load.

The increase in the lateral earth pressure is conventionally evaluated considering an uniform surcharge of 40 kN/m^2 on one side of the earth adjacent to the culvert which results in a constant horizontal pressure acting on only one upright; the value of the lateral load is calculated as:

$q_{earth,variab} = k_0 \cdot 40 \cdot 1 = 0.5 \cdot 40 \cdot 1 = 20.0$ kN/m

The Fig. 7.1 - 4 resumes the applied loads on the structure.

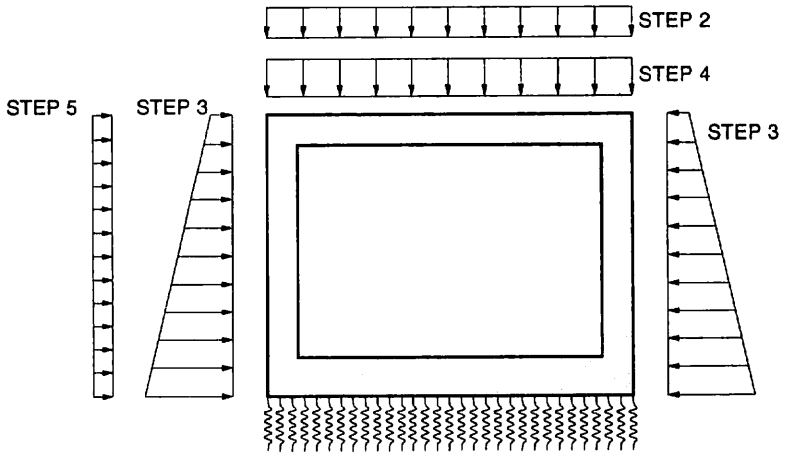


Fig. 7.1 - 4: Applied loads on structure

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7.1.4 Combination for ULS and consequent internal actions

The design is performed using the output of FEM analysis without any redistribution of internal actions (linear-elastic analysis). The following expression for combination of actions has been used:

$$S_d = S\left\{\gamma_G\, G_k + \gamma_Q\, Q_k\right\} \qquad (7.1 - 1)$$

where:

G_k = permanent loads: self-weight, soil and ballast weight, lateral earth pressure at rest;
Q_k = variable loads: train load, lateral earth pressure from train load;
$\gamma_G = 1.35$;
$\gamma_Q = 1.50$.

The results of FEM analysis, in terms of internal actions combined according to (7.1 - 1) are represented in Fig. 7.1 - 5.

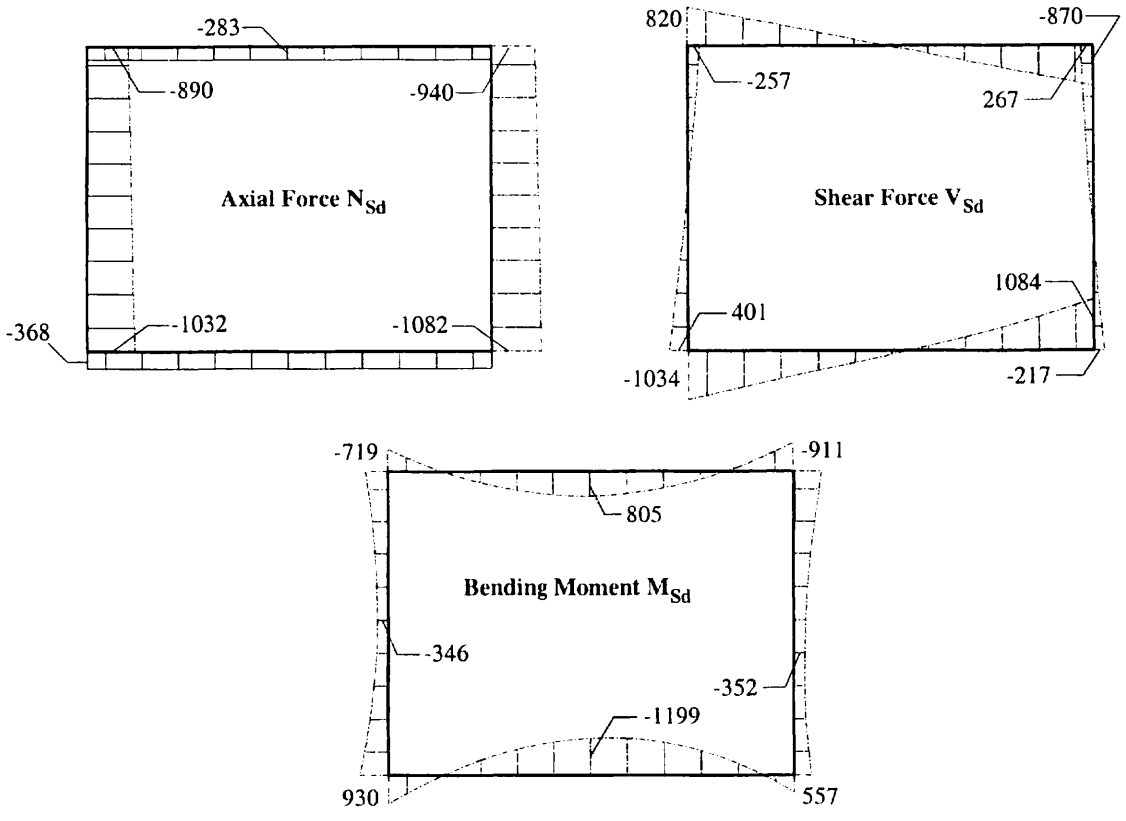


Fig. 7.1 - 5: Internal actions from FEM model at ULS

(1) Upper slab and foundation design

Referring to Fig. 7.1 - 5, three different sections are investigated for the upper slab in order to evaluate the amount of reinforcing steel: at midspan, where maximum positive bending moment is present, and at corners where maximum shear force and negative bending moment are reached. Similarly, the same sections have been chosen for the foundation. The internal actions at ULS are reported in Table 7.1 - 3.

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The following notation has been adopted:
- b = breadth of the equivalent beam;
- h = depth of the section;
- d = distance of reinforcement centroid from the extreme compressive fibre.

	Section	b [m]	h [m]	d [m]	N_{Sd} [kN]	V_{Sd} [kN]	M_{Sd} [kNm]
Upper Slab	Left Corner	1.0	0.7	0.655	-283	820	-719
	Midspan	1.0	0.7	0.655	-283	-25	805
	Right Corner	1.0	0.7	0.655	-283	-870	-911
Foundation	Left Corner	1.0	0.9	0.855	-368	-1034	930
	Midspan	1.0	0.9	0.855	-368	-84	-1199
	Right Corner	1.0	0.9	0.855	-368	1084	557

Table 7.1 - 3: Internal actions on upper slab and foundation

On the safety side, the axial forces have been disregarded because earth pressure may not have a favourable effect on N_{Sd} values.

	Section	μ [-]	ω [-]	ξ [-]	ζ [-]	A_s [cm^2]	x [m]	z [m]
Upper Slab	Left Corner	0.085	0.0905	0.1620	0.9395	27.1	0.106	0.613
	Midspan	0.095	0.1018	0.1745	0.9340	30.5	0.114	0.609
	Right Corner	0.107	0.1155	0.1901	0.9268	34.6	0.124	0.604
Foundation	Left Corner	0.064	0.0674	0.1362	0.9506	26.4	0.116	0.810
	Midspan	0.083	0.0883	0.1596	0.9405	34.6	0.136	0.801
	Right Corner	0.038	0.0394	0.1010	0.9642	15.4	0.086	0.821

Table 7.1 - 4: Tension reinforcement from General Table

The amount of reinforcement required for bending is calculated, for the sake of simplicity and on the safety side, without consideration for the compression reinforcement, using the General Table reported in CEB Bulletin 141. So, for each investigated section, one has to enter the General Table with the non-dimensional bending moment:

$$\mu = \frac{M_{Sd}}{b d^2 f_{cd}}$$

and to obtain the following quantities:

$$\omega = \frac{A_s}{b d} \frac{f_{yd}}{f_{cd}} \qquad \xi = \frac{x}{d} \qquad \zeta = \frac{z}{d}$$

being x the depth of neutral axis and z the internal lever arm; the obtained results are resumed in Table 7.1 - 4.

Considering shear effects, the amounts of A_s, previously calculated, should be increased to

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take into account the longitudinal component that shear force exerts on tension chords; the further reinforcement is evaluated by:

$$\Delta A_s = \frac{V_{Sd}(\cot\theta - \cot\alpha)}{2 f_{yd}} \qquad (7.1 - 2)$$

Assuming $\theta = 30°$ and $\alpha = 90°$, it can be found the total longitudinal reinforcement as reported in Table 7.1 - 5.

	Section	A_s [cm^2]	ΔA_s [cm^2]	$A_{s,TOT}$ [cm^2]
Upper Slab	Left Corner	27.1	16.3	43.4
	Midspan	30.5	0.5	31.0
	Right Corner	34.7	17.3	52.0
Foundation	Left Corner	26.4	20.6	47.0
	Midspan	34.6	1.7	36.3
	Right Corner	15.4	21.6	37.0

Table 7.1 - 5: Total longitudinal reinforcement

The compression chord results automatically verified since the term $V_{Sd}/2\ (\cot\theta - \cot\alpha)$ should be subtracted from the compression force, whose safety is controlled by using the General Table.

Regarding the transverse web reinforcement, the amount of tension steel is evaluated by imposing the equality between the acting force and the resisting one, according to:

$$\frac{V_{Sd}}{\sin\alpha} = \left(\frac{A_{sw}}{s}\right) f_{yd}\, z\, (\cot\theta + \cot\alpha) \qquad (7.1 - 3)$$

The values of A_{sw}/s for the investigated sections are resumed in Table 7.1 - 6.

	Section	V_{Sd} / sin α [kN]	A_{sw}/s [cm^2/m]
Upper Slab	Left Corner	820	17.8
	Midspan	25	0.5
	Right Corner	870	19.1
Foundation	Left Corner	1034	17.0
	Midspan	84	1.4
	Right Corner	1084	17.5

Table 7.1 - 6: Transverse web reinforcement

The compression of web concrete is checked calculating the acting force by the following:

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$$F_{Scw} = \frac{V_{Sd}}{\sin\theta}\left(\frac{\cot\theta}{\cot\theta + \cot\alpha}\right)$$

which should be lesser than the resisting force, according to:

$$F_{Scw} \leq F_{Rcw} = f_{cd2}\, b\, z\, \cos\theta$$

Table 7.1 - 7 reports the results for the investigated sections.

	Section	F_{Scw} [kN]	F_{Rcw} [kN]	$\Delta F(F_{Rcw} - F_{Scw})$ [kN]
Upper Slab	Left Corner	1639.3	5601.9	3962.6
	Midspan	50.0	5569.2	5519.1
	Right Corner	1739.4	5526.2	3786.9
Foundation	Left Corner	2068.3	7406.8	5338.5
	Midspan	167.2	7328.1	7160.9
	Right Corner	2168.4	7512.8	5344.4

Table 7.1 - 7: Check of web compression

(2) Uprights design

The verification of uprights should be performed considering the interaction of axial force and bending moment; but, for the case involved, the compression is less dominant on bending actions, so that the General Table approach is still valid. On the safety side, reinforcement design is performed considering only tension reinforcement, although a minimum amount of compressed reinforcement will be arranged.

To enter the Table, the axial force should be referred to the centroid of tension reinforcement, so that a modified value of acting moment is obtained:

$$M^*_{Sd} = M_{Sd} + N_{Sd} \cdot (d - h/2)$$

for the evaluation of the non-dimensional moment μ. The reinforcement area should be calculated through the expression:

$$A_s = \omega\, b\, d\, \frac{f_{yd}}{f_{cd}} - \frac{N_{Sd}}{f_{yd}}$$

Referring to the must loaded upright, Table 7.1 - 8 reports the internal actions while in the Table 7.1 - 9 reinforcement design for all the section coincident with FEM nodes is indicated.

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Section	Node	b [m]	h [m]	d [m]	N_{Sd} [kN]	V_{Sd} [kN]	M_{Sd} [kNm]	M^*_{Sd} [kNm]
2001	1	1.0	0.7	0.654	-1032	401	-930	1244
2002	14	1.0	0.7	0.654	-1017	335	-764	1073
2003	15	1.0	0.7	0.654	-1003	247	-583	888
2004	16	1.0	0.7	0.654	-988	164	-455	755
2005	17	1.0	0.7	0.654	-974	86	-377	673
2006	18	1.0	0.7	0.654	-959	13	-346	638
2007	19	1.0	0.7	0.654	-945	-55	-360	647
2008	20	1.0	0.7	0.654	-930	-118	-414	697
2009	21	1.0	0.7	0.654	-916	-176	-507	785
2010	22	1.0	0.7	0.654	-901	-229	-634	908
2010	23	1.0	0.7	0.654	-890	-257	-719	989

Table 7.1 - 8: Internal actions on left upright

Section	Node	μ [-]	ω [-]	ξ [-]	ζ [-]	A_s [cm^2]	x [m]	z [m]
2001	1	0.145	0.1610	0.240	0.902	24.7	0.157	0.590
2002	14	0.125	0.1370	0.214	0.916	17.8	0.140	0.599
2003	15	0.104	0.1120	0.186	0.929	10.6	0.122	0.607
2004	16	0.088	0.0939	0.166	0.938	5.5	0.108	0.613
2005	17	0.079	0.0838	0.155	0.943	2.8	0.101	0.616
2006	18	0.075	0.0794	0.150	0.945	1.8	0.098	0.618
2007	19	0.076	0.0805	0.151	0.944	2.5	0.099	0.617
2008	20	0.082	0.0871	0.158	0.941	4.8	0.104	0.615
2009	21	0.092	0.0984	0.171	0.936	8.5	0.112	0.612
2010	22	0.106	0.1144	0.189	0.927	13.7	0.123	0.607
2010	23	0.116	0.1262	0.202	0.921	17.5	0.132	0.603

Table 7.1 - 9: Vertical reinforcement design for normal actions

As for upper slab and foundation design, an additional amount of tension reinforcement arises from shear action, evaluated by the (7.1 - 2) for the same values of θ and α angles; Table 7.1 - 10 reports the total longitudinal reinforcement for the investigated upright.

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Section	Node	A_s [cm^2]	ΔA_s [cm^2]	$A_{s,TOT}$ [cm^2]
2001	1	24.7	8.0	32.7
2002	14	17.8	6.7	24.5
2003	15	10.7	4.9	15.6
2004	16	5.5	3.3	8.8
2005	17	2.8	1.7	4.5
2006	18	1.8	0.3	2.1
2007	19	2.5	1.1	3.6
2008	20	4.8	2.4	7.2
2009	21	8.5	3.5	12.0
2010	22	13.7	4.6	18.3
2010	23	17.5	5.1	22.6

Table 7.1 - 10: Total longitudinal reinforcement

Transverse reinforcement may be calculated according to expression (7.1 - 3), reporting the results in Table 7.1 - 11.

Section	Node	$V_{Sd} / \sin \alpha$ [kN]	A_{sw}/s [cm^2/m]
2001	1	401	9.0
2002	14	335	7.4
2003	15	247	5.4
2004	16	164	3.5
2005	17	86	1.8
2006	18	13	0.3
2007	19	55	1.2
2008	20	118	2.6
2009	21	176	3.8
2010	22	229	5.0
2010	23	257	5.7

Table 7.1 - 11: Transverse reinforcement design

The web compression may be checked in the same way as done for the upper slab.

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Section	Node	F_{Scw} [kN]	F_{Rcw} [kN]	$\Delta F(F_{Rcw} - F_{Scw})$ [kN]
2001	1	803	5395	4592
2002	14	670	5476	4806
2003	15	494	5554	5060
2004	16	328	5610	5283
2005	17	172	5637	5466
2006	18	25	5649	5624
2007	19	111	5646	5536
2008	20	237	5628	5392
2009	21	353	5597	5244
2010	22	458	5547	5088
2010	23	513	5511	4997

Table 7.1 - 12: Compression of web concrete verification

Due to the low values assumed by transverse shear within the uprights, the interaction M, N, V diagram does not works, then no reduction in the M, N interaction diagram should be considered.

7.1.5 Reinforcement layout

Some basic rules of detailing for structural members should be respected to design reinforcement layout, in addition to the static requests.

Firstly, minimum reinforcement area shall be provided, at least. For this example, the provisions relative to linear members may be applied as a consequence of the chosen calculation models; the upper slab and the foundation may be treated as beam elements, whereas the uprights as columns. The criteria described below have been respected to attain minima areas for longitudinal and transverse reinforcement.

- Beams (Upper slab and foundation)

Longitudinal reinf.:	$\geq 0.0015\ b\ d$	for steel Grade 500;
Transverse reinf.:	$\rho_w = \frac{A_{sw}}{s\ b_w\ \sin\alpha} \geq 0.12\%$	for steel S500 and concrete C30;
Spacing of shear reinf.:	$s_{max} \leq \{0.6\ d\ ;\ 30\ cm\}$	for the considered range of web concrete compression;

- Columns (Uprights)

Vertical reinf.:	$A_s \geq 0.15\ N_{Sd} / f_{yd}$	with a minimum bar diameter of 12 mm
Transverse bar diam.:	$\varnothing_t \geq \varnothing_l / 4$, 5mm	where $\varnothing_l$ is the diameter of vertical diameter
Spacing of shear reinf.:	$s_{max} \leq \{12\ \varnothing_l\ ;\ b\ ;\ 30\ cm\}$	where b is the smallest dimension of concrete section

As secondary step, one has to assess the design length to bond the bar in concrete, whether for anchorage or overlap. The design value of the bond stress is evaluated with the following expression:

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$$f_{bd} = \eta_1 \eta_2 \eta_3 f_{ctd} = \begin{cases} 1.87 \text{ MPa} & \text{good bond conditions} \\ 2.67 \text{ MPa} & \text{bad bond conditions} \end{cases}$$

where:

$f_{ctd} = 1.33$ MPa	design value of concrete tensile stress (for grade C30);
$\eta_1 = 1.4$	for intended bars;
$\eta_2 = 1.0/0.7$	good/bad bond conditions:
$\eta_3 = 1.0$	for bar diameter lesser of 32 mm

so that the basic length necessary for the transfer of the yield force of a bar of diameter $\varnothing$ results:

$$l_b = \frac{\varnothing}{4} \frac{f_{yd}}{f_{bd}} = \frac{434.8}{4 f_{bd}} \varnothing \cong \begin{cases} 58\varnothing & \text{good bond conditions} \\ 83\varnothing & \text{bad bond conditions} \end{cases}$$

The different values are relative to the position of each bar during concreting; good bond conditions may be referred to all the bars with a inclination of 45°–90° to the horizontal or with an inclination less than 45° to the horizontal, which are up to 250 mm from the bottom or at least 300 mm from the top of the concrete layer. So, with respect to the culvert, one should consider as in bad bond conditions the upper reinforcement of both the slab and the foundation, whereas in good conditions all the other bars.

The design anchorage length $l_{b,net}$ can be calculated from the following equation:

$$l_{b,net} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_b \frac{A_{s,cal}}{A_{s,ef}} \qquad (7.1 - 4)$$

with α_1 , α_2 , α_4 and α_5 assumed equals to 1.0 on the safety side, and α_3 evaluated considering the maximum adopted diameter (26 mm) and a straight bar, by means of:

$$\alpha_3 = 1 - 0.15 \frac{c_d - \varnothing}{\varnothing} = 0.948 \qquad (c_d = \text{concrete cover})$$

thus the following operative value may be adopted for $l_{b,net}$:

$$l_{b,net} = 0.948 \cdot \begin{Bmatrix} 58\varnothing \\ 83\varnothing \end{Bmatrix} \cdot \frac{A_{s,cal}}{A_{s,ef}} = \frac{A_{s,cal}}{A_{s,ef}} \cdot \begin{Bmatrix} 55\varnothing \\ 79\varnothing \end{Bmatrix} \qquad (7.1 - 5)$$

with respectively $A_{s,cal}$ and $A_{s,ef}$ area of reinforcement required by calculation and effectively provided. Two values are obtained with respect to the bond condition.

The design lap length is evaluated with the expression (7.1 - 6):

$$l_0 = \alpha_1 \alpha_3 \alpha_4 \alpha_5 \alpha_6 l_b \frac{A_{s,cal}}{A_{s,ef}} = \frac{A_{s,cal}}{A_{s,ef}} \cdot \begin{cases} 110\varnothing \\ 158\varnothing \end{cases} \qquad (7.1 - 6)$$

where:

α_1, α_3, α_4, α_5: calculated as above;

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$\alpha_6 = 2$: supposing that more than 50% of the total reinforcement is lapped.

It can be notice that a significant reduction of the design lengths $l_{b,net}$ and l_0 may be achieved if the amount of reinforcement provided is much greater than the requested one; so a minimum length shall be arranged, with respect to the following limits:

$l_{b,min} < \max\{0.3\ l_b\ ;\ 10\varnothing\ ;\ 10\ cm\}$ $\qquad$ $l_{0,min} < \max\{0.3\ \alpha_6\ l_b\ ;\ 15\varnothing\ ;\ 20\ cm\}$

In the design of secondary reinforcement one may provide a percentage of the primary one (20% for distributed loads) in order to cover the real two-dimensional behaviour, but, at least a minimum amount of reinforcement shall be arranged to avoid cracking by restrained shrinkage; the latter should be calculated by means of the following expression:

$$A_{s,min} = k_c\ k\ f_{ct,max}\ \frac{A_{ct}}{\sigma_{s2}} \qquad (7.1 - 7)$$

where, if pure tension is assumed to be active:

$f_{ct,max}$ = 2.39 MPa: upper fractile of concrete tensile strength for grade C30 at themoment when the first crack is expected to appear; because principal intrinsic imposed deformation is due to heat-curing, a time of 3 days is considered as reference to calculate concrete tensile strength;

σ_{s2} = 240 MPa: tension on steel limited to 240 MPa in order to satisfy the crack width limits if a bar diameter of 20 mm is adopted.

A_{ct} = 0.7 – 0.9 m^2: area of the concrete tension zone coincident with a rectangular section with a breadth 0f 1.0 m and a depth equal to the thickness of each structural member (0.7 for the upper slab and uprights, 0.9 for the foundation);

k = 0.56 – 0.5 : corrective factor equal to 0.56 for h = 0.7m and to 0.5 for h = 0.9 m, in case of intrinsic imposed deformation on rectangular section;

k_c = 1.0 : corrective factor that accounts for the scheme of tensile stress distribution, equal to 1.0 for pure tension.

So, for the structural member considered, the following quantities may be calculated:

- Upper slab and uprights: $A_{s,min} = 1 \times 0.56 \times 2.39 \times 0.7 / 240 \times 10^4 = 39.04\ cm^2/m$;
- Foundation: $A_{s,min} = 1 \times 0.5 \times 2.39 \times 0.9 / 240 \times 10^4 = 44.81\ cm^2/m$;

that shall be arranged in two layers. So for the upper slab and the uprights, a 20 mm bar has been disposed with a transverse spacing of 15 cm, whereas in the foundation a 22 mm bar diameter is present with the same spacing.

Particular attention must reserved to the corner zones, to complete primary reinforcement design; in fact, in case of closing moment, the main reinforcement exerts a splitting effect on the concrete which may lead to local crushing; these forces shall be equilibrated by an additional reinforcement put within the node. By reference to Fig. 7.1 - 6 it is possible to determine the minimum diameter of the mandrel to bent the bar in order to avoid a reduction of the internal lever arm z (equal to 0.8 d).

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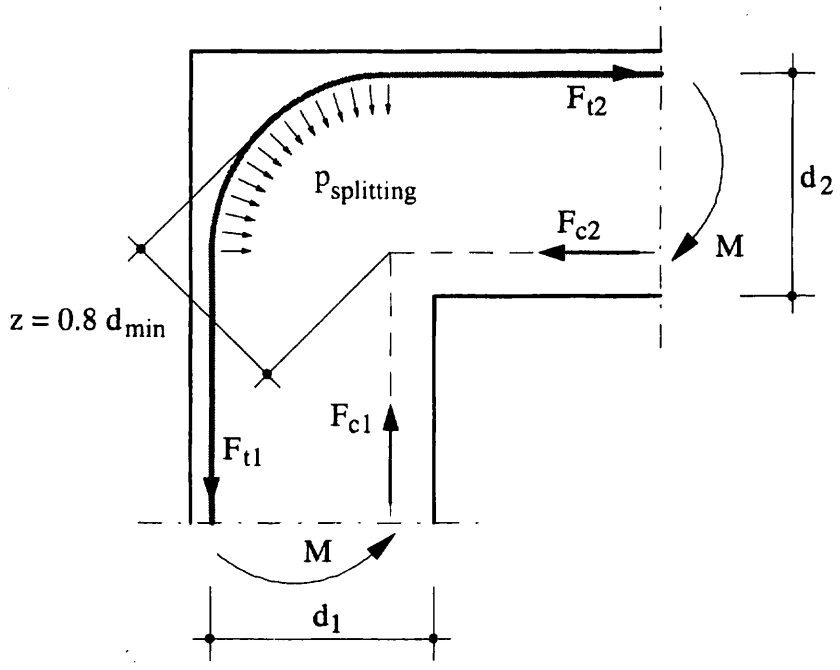


Fig. 7.1 - 6: Internal lever arm in the corner

The splitting force may be designed approximately for 25% of radial pressure exerted by the main reinforcement, i.e.:

$$p_{splitting} = 0.25 \frac{F_{t1} + F_{t2}}{2} \frac{1}{0.8 d_{min}} \qquad (7.1 - 8)$$

Considering the left corner between the upper slab and the upright, the tensile forces F_{t1} and F_{t2} are calculated multiplying the reinforcement areas, respectively from Table 7.1 - 5 and 7.1 - 10, with the design tensile strength f_{yd} . Being in this case $d_1 = d_2$, the total splitting force is evaluated as product of expression (7.1 - 8) for bent length:

$$F_{splitting} = p_{splitting} \frac{\pi}{2} 0.8 d_{min} = 0.25 \cdot \frac{22.6 + 43.4}{2} \cdot 434.78 \cdot \frac{\pi}{2} \cdot 10^{-1} = 563.43 \text{ kN}$$

from which the following steel area results:

$$A_{s,splitting} = \frac{F_{splitting}}{f_{yd}} = \frac{563.45}{434.78} \cdot 10 = 12.96 \text{ cm}^2$$

that has been arranged by means of 10 bar with a diameter of 14 mm, disposed as indicated in Fig. 7.1 - 7.

Similarly, for the corner between the foundation and the uprights, the necessary area for splitting action results:

$$A_{s,splitting} = 15.67 \text{ cm}^2$$

Fig. 7.1 - 8 represents the main reinforcement layout for the culvert in the upper slab, in the foundation and only in the left upright, being the right one equally reinforced.

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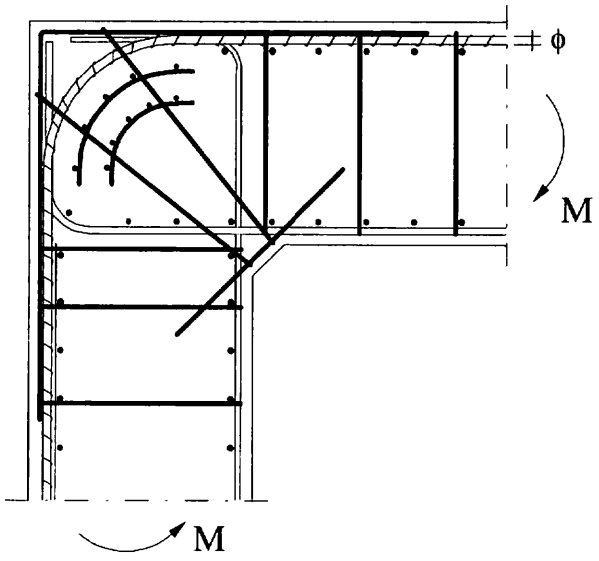


Fig. 7.1 - 7: Local reinforcement in the node to equilibrate the splitting action

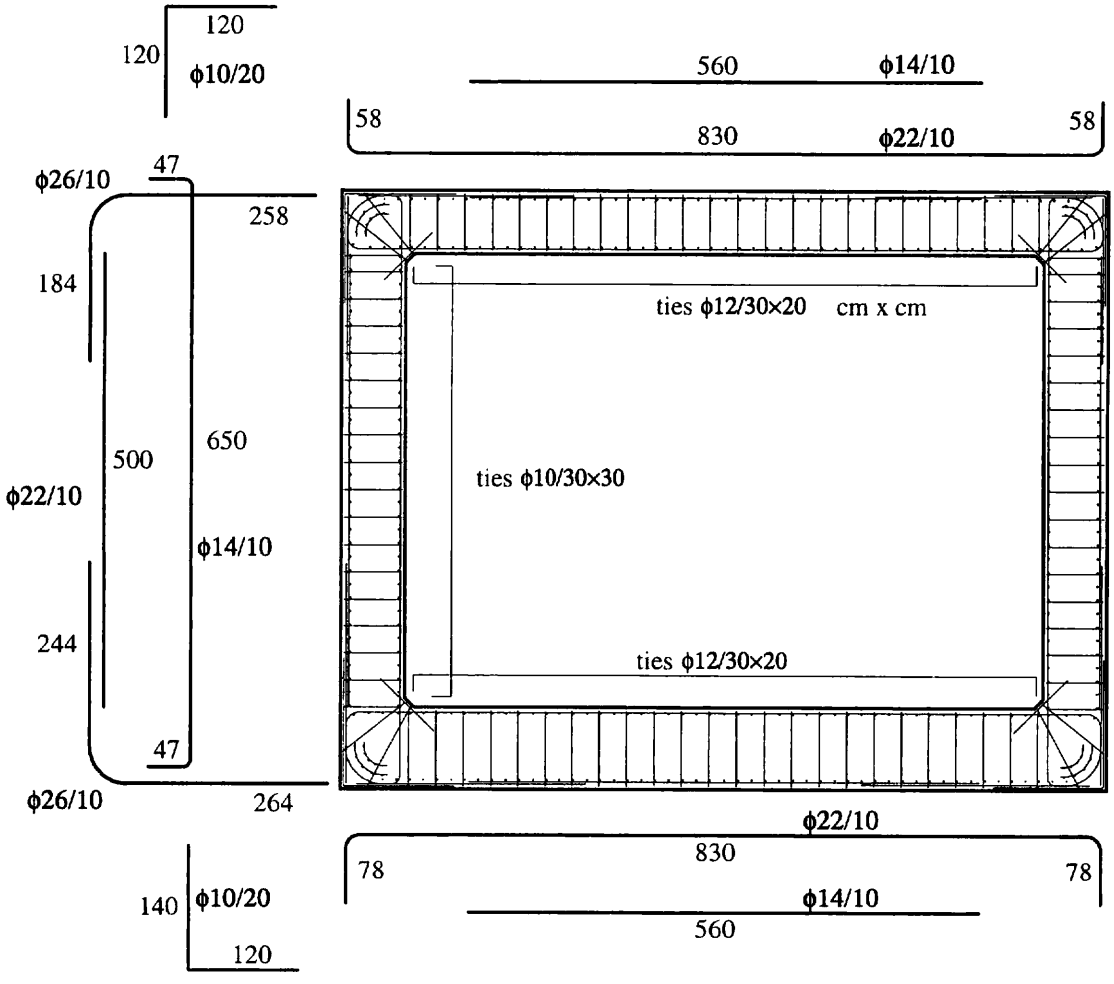


Fig. 7.1 - 8: Reinforcement layout

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7.1.6 Verification at serviceability limit state

The verification at serviceability limit state is relative to the following conditions:
- stress limitation;
- crack widths;
- deformation.

Different combination of actions should be considered, according to the provisions of the adopted Code; in this example, the adopted combinations are listed below together with the symbolic presentation:
- rare combination: $G_k + Q_k$
- quasi-permanent combination: $G_k + \Psi_2 Q_k$

(1) Stress limitation

One has to check that, under the rare combination, the maximum concrete compressive stress does not exceed $0.6\, f_{ck}$ = 18 MPa and that the tensile stress in ordinary reinforcement does not exceed $0.8\, f_{yk}$ = 400 MPa.

The diagrams of combined actions according to the rare combination are reported in Fig. 7.1 - 9, with respect to only bending moment and axial force.

The extreme stresses for the upper slab, the foundation and one of the two uprights are resumed in Table 7.1 - 13. The same sections investigated at ULS and the effective arranged reinforcement are considered.

These stresses, due to the SLS considered, have been calculated adopting the modular ratio method.

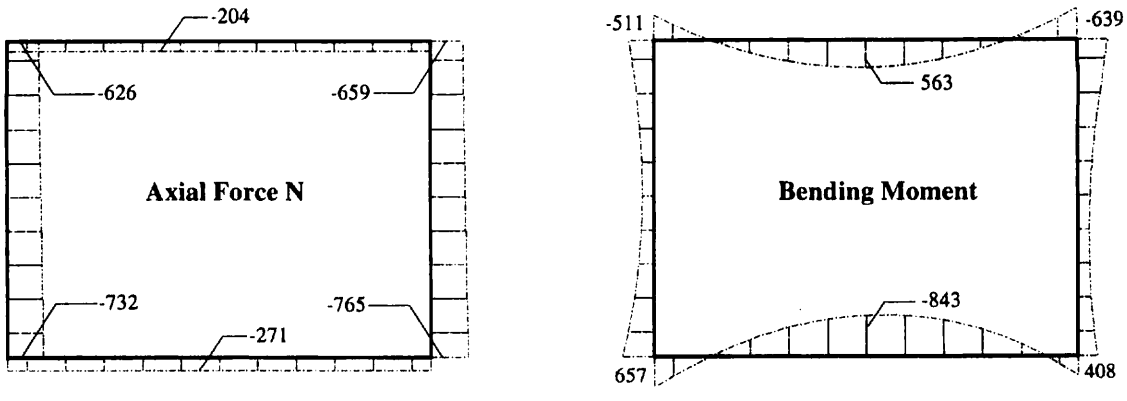


Fig. 7.1 - 9: Internal actions from FEM model at SLS in the rare combination

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Element	Node	Nk [kN]	Mk [kNm]	b [m]	h [m]	$A_{1,s}$ [cm^2]	$A_{2,s}$ [cm^2]	x [m]	σ_c [MPa]	σ_s [MPa]
4001	23	-204	-511	1.0	0.7	53.1	38.0	0.18	-9.1	142.7
4007	29	-204	563	1.0	0.7	38.0	15.4	0.16	-12.1	220.4
4012	35	-204	-639	1.0	0.7	53.1	38.0	0.17	-11.3	182.9
1001	1	-271	657	1.0	0.9	53.1	38.0	0.22	-7.7	133.0
1007	7	-271	-843	1.0	0.9	38.0	15.4	0.19	-11.9	245.4
1012	13	-271	408	1.0	0.9	53.1	38.0	0.24	-4.9	74.2
2001	1	-732	-657	1.0	0.7	38.0	15.4	0.19	-14.3	199.5
2002	14	-721	-538	1.0	0.7	38.0	15.4	0.21	-11.7	149.9
2003	15	-710	-407	1.0	0.7	38.0	15.4	0.23	-8.8	96.3
2004	16	-699	-316	1.0	0.7	38.0	15.4	0.26	-6.8	60.1
2005	17	-688	-260	1.0	0.7	38.0	15.4	0.29	-5.5	39.9
2006	18	-678	-239	1.0	0.7	38.0	15.4	0.31	-5.0	33.1
2007	19	-667	-250	1.0	0.7	38.0	15.4	0.29	-5.2	37.8
2008	20	-656	-290	1.0	0.7	38.0	15.4	0.26	-6.2	54.0
2009	21	-645	-357	1.0	0.7	38.0	15.4	0.23	-7.7	82.2
2010	22	-635	-449	1.0	0.7	38.0	15.4	0.21	-9.8	121.8
2010	23	-626	-511	1.0	0.7	38.0	15.4	0.20	-11.2	148.9

Table 7.1 - 13: Extreme stress on concrete and tension steel

where $A_{1,s}$ and $A_{2,s}$ are respectively the tension and compression reinforcement area. It can be notice that, in all the sections, verification is met.

(2) Crack width

The quasi-permanent combination of action is adopted to evaluate crack width, with a Ψ_2 coefficient equal to 0.6 as requested by the used Code. The nominal width value relative to exposure class 2 is of 0.30 mm for reinforced concrete member with respect to both appearance and durability.

In Fig. 7.1 - 10 are sketched the diagrams of axial force and bending moment for the quasi-permanent combination.

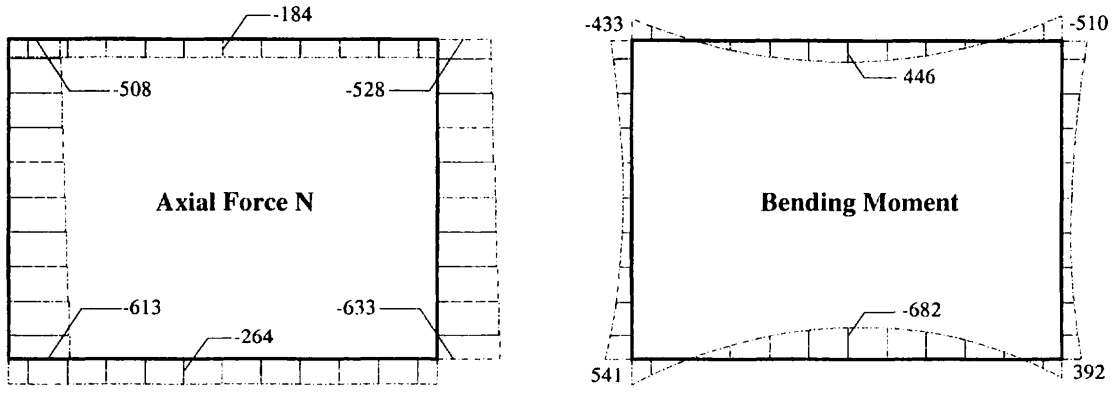


Fig. 7.1 - 10: Internal actions from FEM model at SLS in the quasi-permanent combination

Crack width is calculated by means of the following expression:

$$w_k = l_{s,max}\left(\varepsilon_{s2} - \beta\varepsilon_{sr2} - \varepsilon_{cs}\right) \qquad (7.1 - 9)$$

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where:

$l_{s,max} = \dfrac{\varnothing_s}{3.6\rho_{s,ef}}$ length over which slip between concrete and steel occurs;

$\varnothing_s$ diameter of the steel bar;

$\rho_{s,ef} = \dfrac{A_s}{A_{c,ef}}$ effective reinforcement ratio referred to concrete area in tension;

$\varepsilon_{s2} = \dfrac{\sigma_{s2}}{E_s}$ steel strain at the crack;

$\varepsilon_{sr2} = \dfrac{\sigma_{sr2}}{E_s}$ steel strain at the crack under forces causing f_{ctm} within $A_{c,ef}$;

$\beta = 0.38$ empirical factor to assess averaged strain within $l_{s,max}$;

$\varepsilon_{cs} = -0.385 \cdot 10^3$ concrete strain due to shrinkage;

Referring to the three sections of upper slab and foundation, and considering just two sections for the upright at nodal zone (higher bending moments), it is possible to obtain Table 7.1 - 14 where the principal results are reported.

Elem.	Node	Nk	Mk	M_{crack}	b	h	$A_{1,s}$	$A_{2,s}$	x	ϕ	$\rho_{s,ef}$	σ_{s2}	σ_{sr2}	$l_{s,max}$	$\varepsilon_{s2} - \beta\varepsilon_{sr2}$	w_k
		[kN]	[kNm]	[kNm]	[m]	[m]	[cm²]	[cm²]	[m]	[mm]	[-]	[MPa]	[MPa]	[mm]	[x 1000]	[mm]
4001	23	-184	-433	-259	1.0	0.7	53.1	38.0	0.18	26	0.044	119.6	65.4	163.2	0.47	0.14
4007	29	-184	446	259	1.0	0.7	38.0	15.4	0.16	22	0.033	171.0	90.3	184.9	0.68	0.20
4012	35	-184	-510	-259	1.0	0.7	53.1	38.0	0.18	26	0.044	143.6	65.4	163.2	0.59	0.16
1001	1	-264	541	433	1.0	0.9	53.1	38.0	0.22	26	0.044	105.9	80.3	163.2	0.38	0.12
1007	7	-264	-682	-433	1.0	0.9	38.0	15.4	0.19	22	0.033	192.5	110.9	184.9	0.75	0.21
1012	13	-264	392	433	1.0	0.9	53.1	38.0	*section uncracked*							
2001	1	-613	-541	-309	1.0	0.7	38.0	15.4	0.19	22	0.033	162.6	65.4	184.9	0.69	0.20
2010	23	-508	-433	-297	1.0	0.7	38.0	15.4	0.20	22	0.033	128.2	70.9	184.9	0.51	0.16

Table 7.1 - 14: Cracks opening evaluation

It can be notice that the limit of 0.3 mm has been respected.

(3) Deformation

Deformation limitation is carried out to control the maximum vertical deflection for passengers comfort. The limit values δ/L (deflection/span Length) are given by the Eurocode 1.3 as a function of the span length and the train speed. For the analysed structure, it may be considered that the earth cover behaves like a rigid body so that it undergoes the same deflection of upper slab. The limit value for maximum vertical deflection is calculated considering a span length of 7.7 m (uprights spacing) and a train speed over 280 km/h and, according to the provisions of the Code, it results:

$$\frac{\delta}{L} = \frac{1}{1500}$$

that should be multiplied by a factor 2 for structure with 1 span; finally, the following limit may be achieved:

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$$\frac{\delta_{lim}}{L} = \frac{2}{1500} = \frac{1}{750}$$

As a consequence of the transient nature of this event, the elastic deflection, calculated by the FEM model, is relative to the only live load; the check shall be performed loading only one track. Fig. 7.1 - 11 represents the applied load on FEM model (half of the train load plus the dynamic allowance together with the deformed shape of the structure).

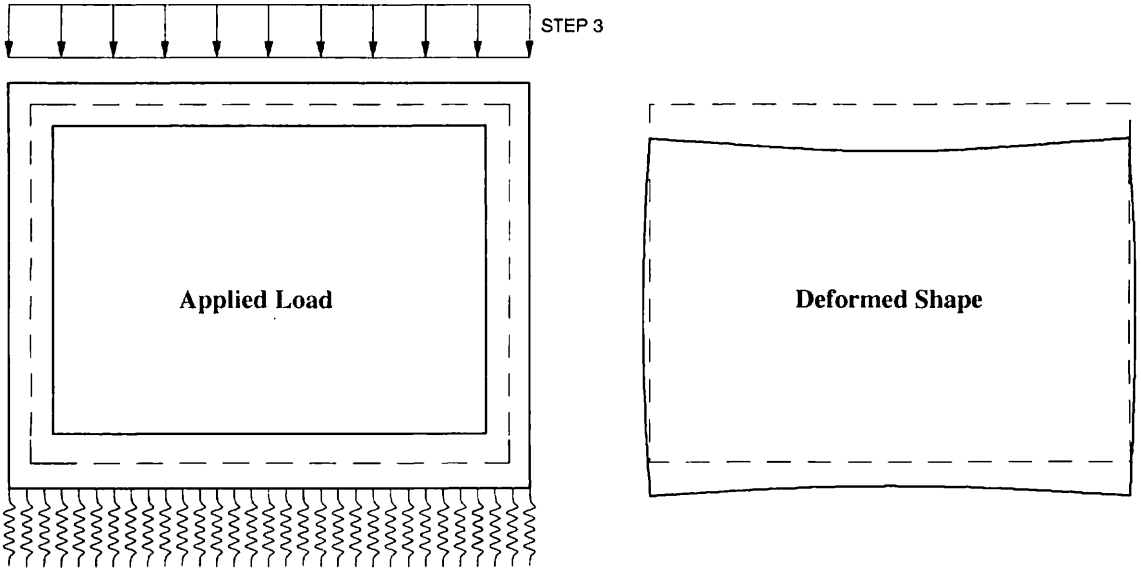


Fig. 7.1 - 11: Applied load and deformed shape for deflection check

According to the MC 90, the deformations should be calculated from the curvatures through the principle of virtual work. For the sake of simplicity, the simplified method may be followed; it is based on a bilinear relationship between load and deflection, by applying the following expression:

$$\delta = \left(\frac{h}{d}\right)^3 \eta\left(1 - 20\rho_{cm}\right)\delta_c \qquad (7.1 - 10)$$

where:

$\delta_c = 0.0035$ m	maximum elastic deflection of upper slab calculated with the FEM analysis;
$\eta = 3.64$	correction factor for cracking and creep effects;
$\rho_{cm} = 0.0029$	geometrical mean percentage of compressive reinforcement;
$h = 0.700$ m	depth of the upper slab;
$d = 0.655$ m	distance of centroid of reinforcement from the extreme concrete compressive fibre;

By applying the (7.1 - 10), for the node 29 of FEM model, the following value is obtained:

$$\delta = 0.0146 \text{ m} \qquad (7.1 - 11)$$

The previous value shall be corrected by subtracting the creep influence, included in the η coefficient; in fact it possible to estimate that the ordinary frequency of railway traffic

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corresponds, in the lifetime of the structure, to an equivalent application of the train as a sustained load for a period of two years. So, the amount of the deflection due to creep is valuable by means of the following expression:

$$\delta_{c,creep} = \phi\, \delta_c = 1.69 \cdot 0.0035 = 0.0059 \text{ m}$$

where $\phi = 1.69$ is the creep coefficient calculated for a period of two years.

Finally, the obtained value should be subtracted from the (7.1 - 11) so that the effective deflection results:

$$\delta_{effective} = \delta - \delta_{creep} = 0.0146 - 0.0059 = 0.0087$$

which leads to a deflection/span ratio of:

$$\frac{\delta_{effective}}{L} = \frac{0.0087}{7.70} = \frac{1}{885}$$

and it results lower than the corresponding limit.

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7.2 Slabs

by G. Mancini

As two dimensional member a prestressed concrete slab is analysed: the actual structure is described in the following point.

7.2.1 Description of the structure

The design example proposed in this section is related to a railway bridge deck made up by a continous slab on three spans with two orders of prestressing tendons (longitudinal and transverse prestressing). The slab is designed in category A (see Eurocode 2, Part 2, table 4.118) for fatigue reasons. The deck rests on abutments and circular piers and has a overall breadth of 13.60 m with two side-walks of 1.40 m width, two ballast retaining walls and, in the middle, two track spacing of 5.0 m. The slab presents a constant thickness of 1.50 m for a central zone 7.0 m width, whilst is tapered towards the extremity with a final height of 0.6 m. Fig.s 7.2 - 1 and 7.2 - 2 represent the principal geometric dimension of the slab bridge and supports' scheme.

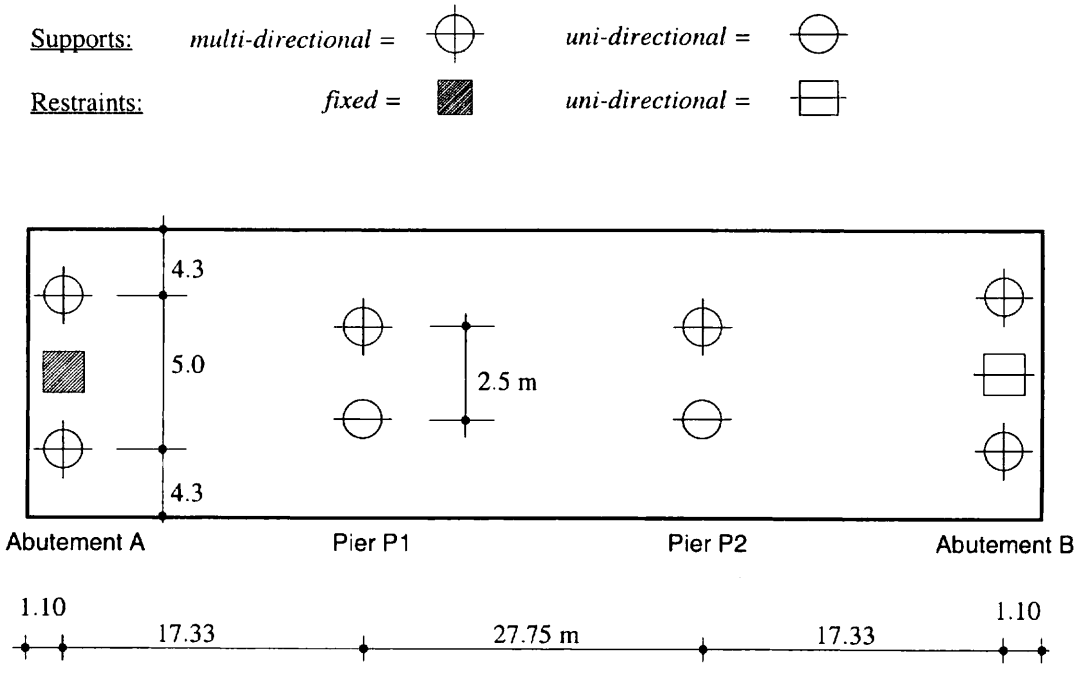


Fig. 7.2 - 1: Plan view of the structure and supports' scheme

(1) Material properties

The following materials properties have been considered:

- Concrete Grade 35: f_{ck} =
  compressive design strength:
  compressive resistance for uncracked zones:
  compressive resistance for cracked zones:
  mean value of tensile strength:

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modulus of elasticity:	E_c =	29.70 · 10^3 MPa;	
shear modulus:	G =	12.40 · 10^3 MPa	
Poisson ratio:	ν =	0.20	
– Prestressing steel, (strands ∅ 0.6"):	f_{ptk} =	1800	MPa;
0.1% proof–stress	$f_{p0.1k}$ =	1600	MPa
total elongation at maximum load:	ε_{pu} >	35	‰
modulus of elasticity:	E_p =	195.0 · 10^3 MPa;	
– Reinforcing steel, Grade 500:	f_{yk} =	500.0	MPa;
design strength:	f_{yd} =	434.8	MPa;
modulus of elasticity:	E_s =	200.0 · 10^3 MPa.	

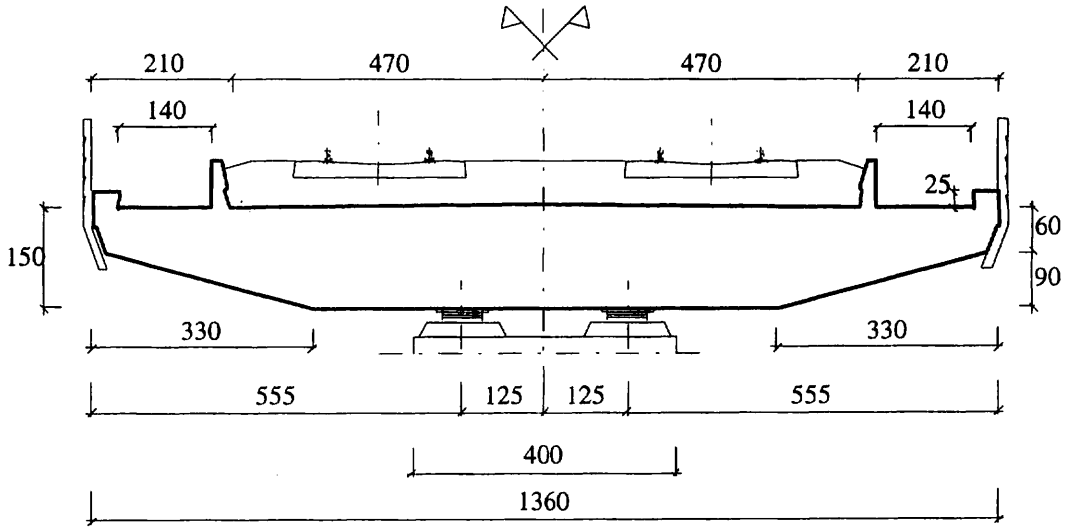


Fig. 7.2 - 2: Geometric dimensions of bridge cross section

(2) Concrete cover

As environmental condition an Exposure Class 2 may be considered (Humid environment with frost: exterior components exposed to frost).

The minimum concrete cover for Class 2 is equal to 25 mm, which should be added to the tolerance value of 10mm; as a consequence the nominal value for concrete cover results:

$$c_{nom} = c_{min} + 10 = 25 + 10 = 35 \text{ mm}$$

adopted in the calculations.

7.2.2 Structural model

To evaluate the internal actions on the structure a linear FEM analysis has been performed adopting shell elements to represent the reinforced slab; this kind of element takes account of all the slab and plate components as well as the out-of-plane shear forces. The thickness of shell elements has been assumed constant for the inner zone of the slab and stepped to fashion the tapered extremity. In Fig.s 7.2 - 3 and 7.2 - 4 the FEM model is sketched and the different thick of the element is reported too.

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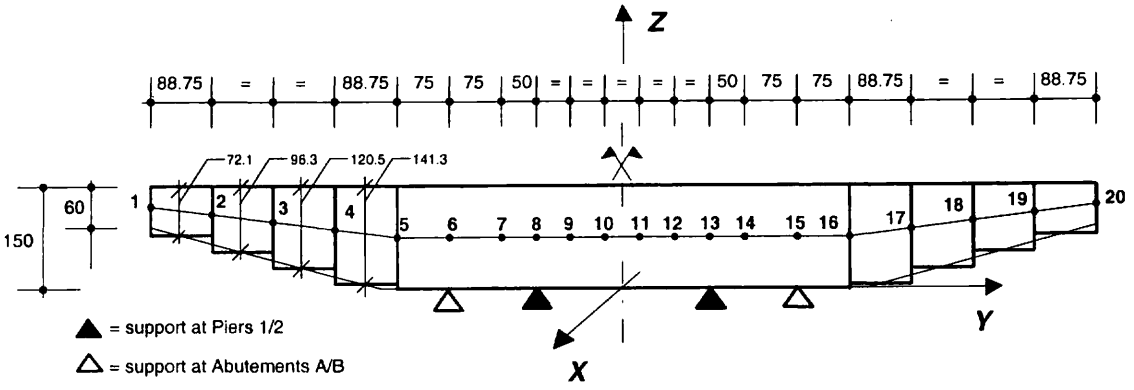


Fig. 7.2 - 3: Transverse view of FEM model

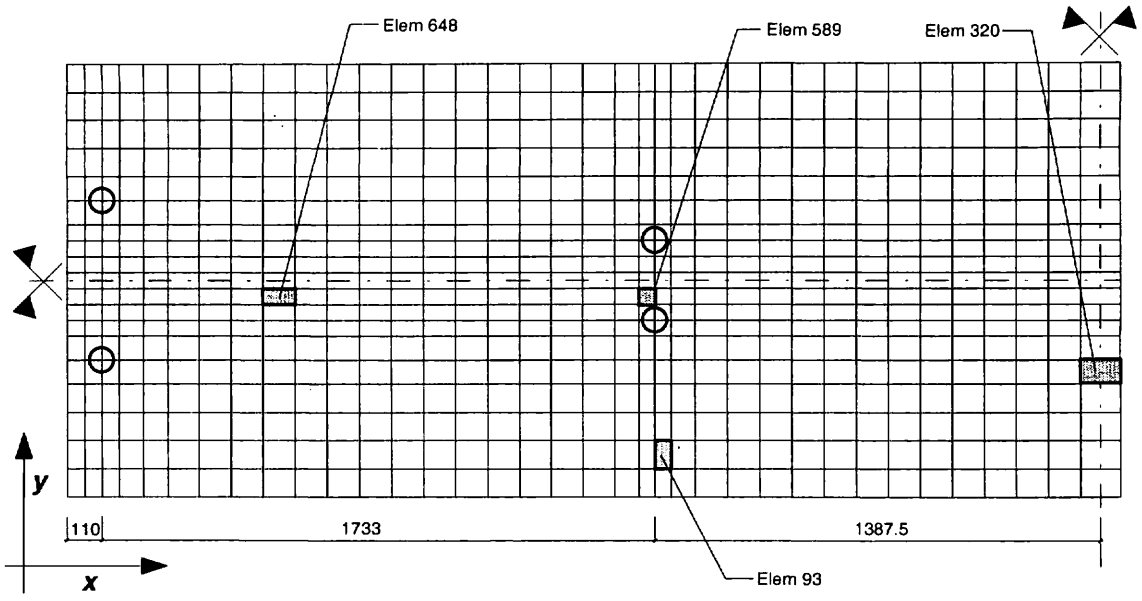


Fig. 7.2 - 4: Plan of FEM model and considered elements

The adopted shell elements are oriented with the following guidelines:

- local axis 2 is oriented as global axis Y of the deck;
- local axis 3 is oriented in the opposite direction of global axis X of the deck;
- local axis 1 is oriented as global axis Z of the deck.

Positive forces for FEM program output are reported in Fig. 7.2 - 5:

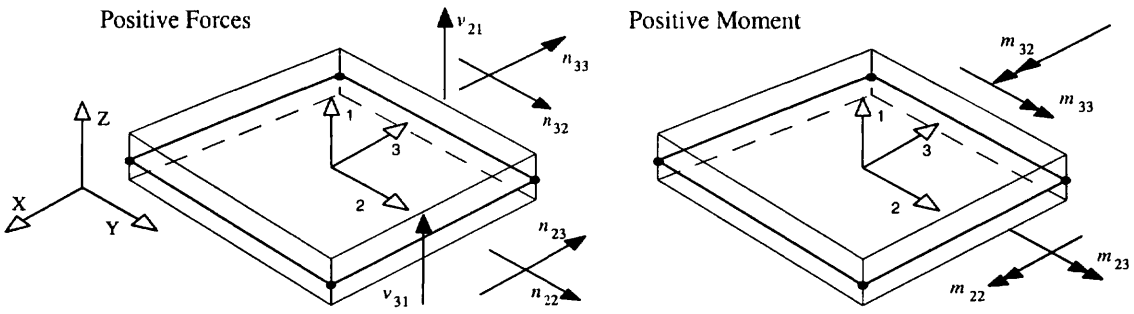


Fig. 7.2 - 5: Positive actions for FEM elements

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(1) Restraints

The external restraints have been introduced in the FEM model considering their real geometric dimensions; thus, few nodes have been restrained by means of spring elements in order to represent only an individual restraint or support. Fig. 7.2 - 6 shows a symbolic notation for the external restraints with the nodes involved.

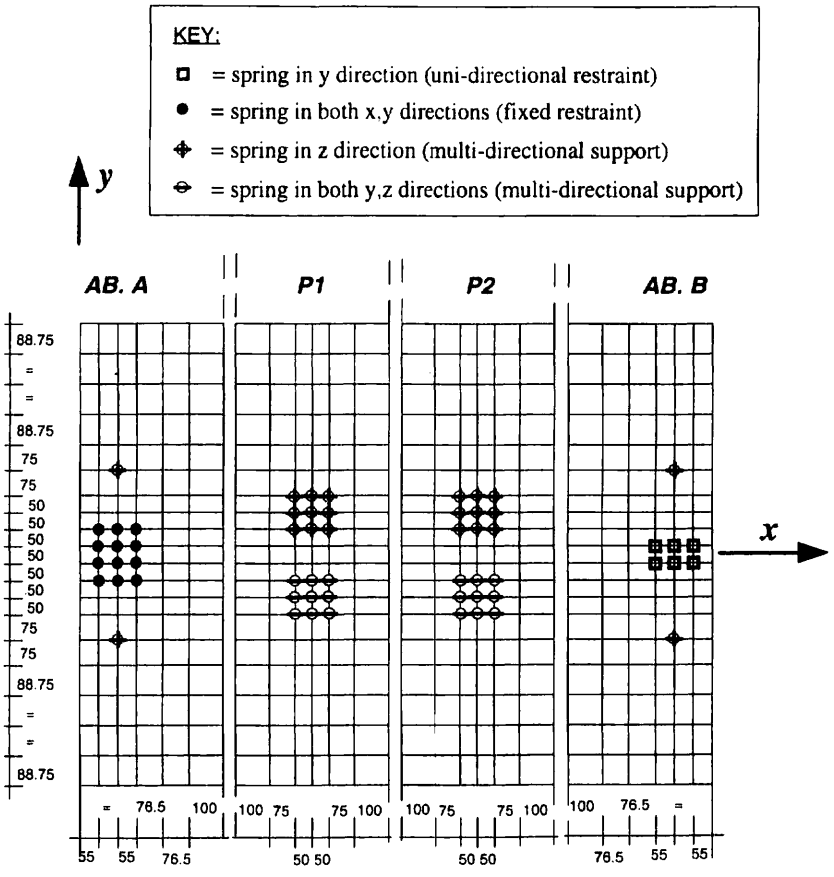


Fig. 7.2 - 6: External restraints on the FEM model

The elastic constant of the spring restraining elements is calculated to have the same stiffness of the substructure (abutments or piers) on which the slab is rested. For the x and y directions, it may be assumed that the pier, or the abutment front wall, behaves like a single column fixed at the base and free at his top, so that the relevant $K_{x/y}$ stiffness is valuable as:

$$K_{x/y} = \frac{3EI}{H^3}$$

where E is the Young modulus, I the inertia and H the height of the column. For the vertical direction, the intrinsic stiffness of pot-bearing is assumed, considering the substructure vertical behaviour as rigid.

For the sake of simplicity the calculus of the relevant stiffness is omitted and the final values of the spring constants are reported in table 7.2 - 1 .

It can be notice that the previous values are referred to the overall stiffness of the restraint, thus the elastic constant of any individual spring element may be obtained dividing the K

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values of table 7.2 -1 by the number of element representing the restraint or the supports.

Location	Kx,tot 10^6 kN/m	Ky,tot 10^6 kN/m	Kz,tot 10^6 kN/m
Abutment A	9.55	178.80	10.02
Pier P1	-	4.74	11.61
Pier P2	-	2.66	11.61
Abutment B	-	2.78	10.02

Table 7.2 - 1: Stiffness for restraining elements

(2) Prestressing forces

Two orders of prestressing tendons are arranged (in longitudinal and transverse directions) in order to avoid any tensile stress in concrete at service (required by railway code). The initial tensile stress of tendon is: $\sigma_{po,max} = 0.85\, f_{p\,0.1k} = 0.85 \cdot 1600 = 1360$ MPa. The number of tendons is 39 for the longitudinal direction and 64 for the transverse one. Each tendon is built up with 19 strands ∅ 0.6" having an area of 1.39 cm^2.

Fig. 7.2 - 7 reports tendon's layout for half deck, being symmetrically disposed.

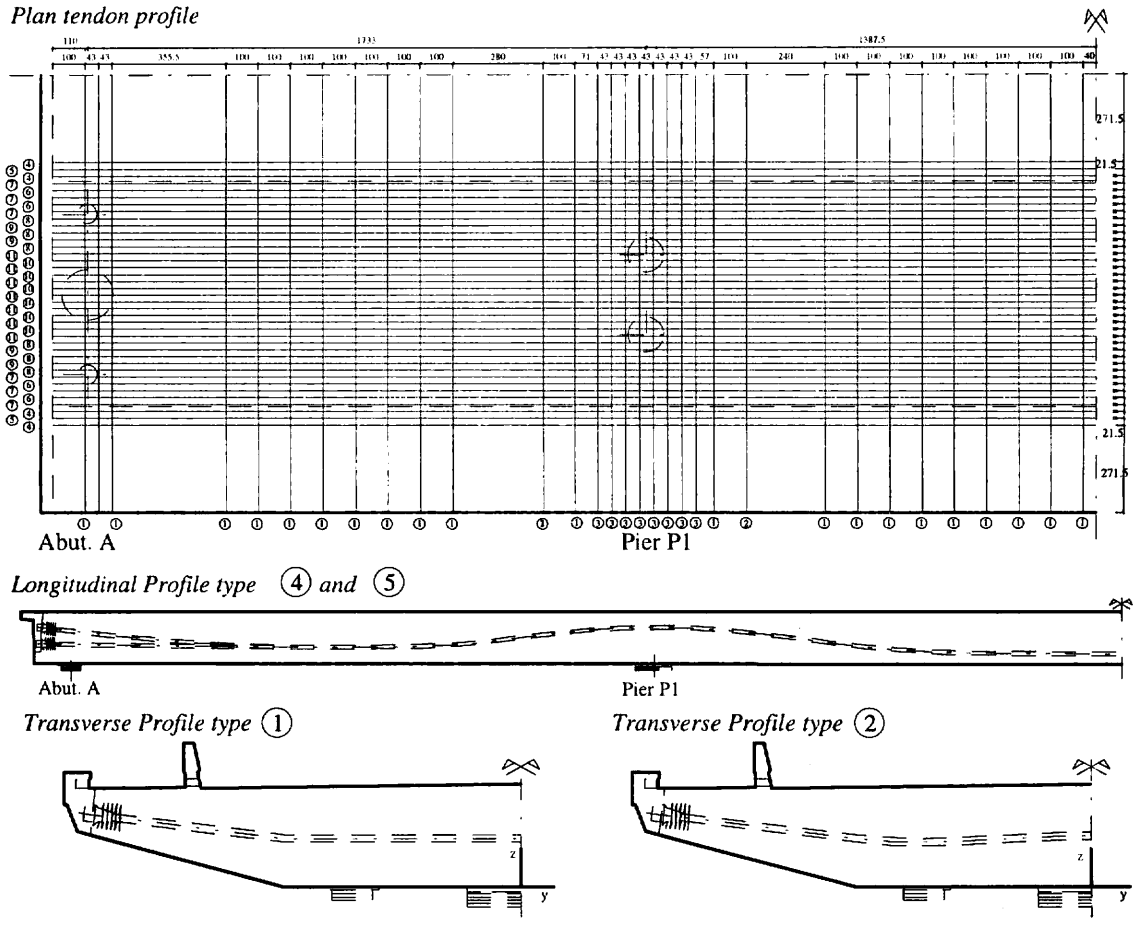


Fig. 7.2 - 7: Plan and principal section of tendon layout

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Immediate losses of prestressing due to friction have been evaluated by means of the following expression:

$$\sigma_{po}(x) = \sigma_{po,max} \cdot e^{-\mu(\alpha + k x)}$$

with:

$\mu = 0.19$	coefficient of friction between the tendons and their sheathing;
$k = 0.01$ rad/m	unintentional angular deviation.

Prestressing has to be introduced in the FEM model in order to calculated the hyperstatic actions that arise in the structural scheme. Considering prestressing as an external load, it is possible to introduce it by means of two inclined forces at anchorages (representing actions at the extremity) and of a system of equivalent loads along tendon's profile (representing tendon curvature and losses due to friction): these actions per tendon, should be applied consistently at the nodes of FEM model.

The equivalent loads may be calculated subdividing the tendon profile into elementary segments and evaluating the internal action able to equilibrate the external one due to end actions deriving by the prestressing.

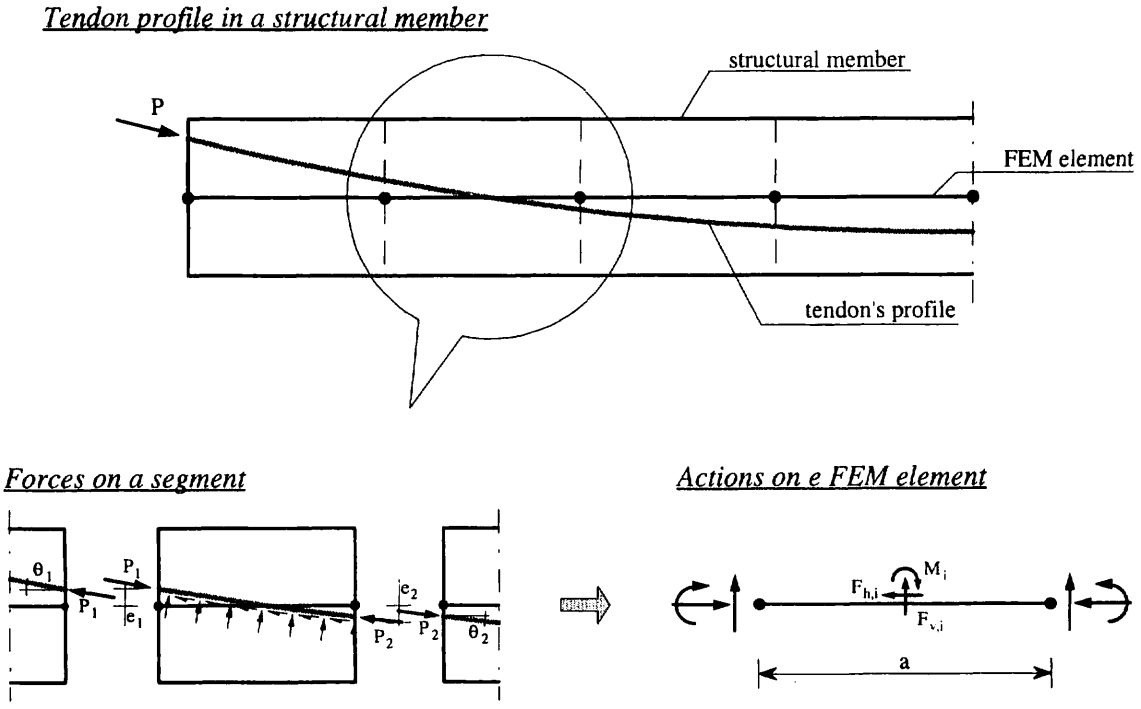


Fig. 7.2 - 8: Effect of prestressing on a segment and equivalent loads

Fig. 7.2 - 8 represents the forces acting on a segment of concrete due to a curved prestressing tendon; if the inclination of the cable changes from θ_1 to θ_2 while the prestress force changes from P_1 to P_2 due to friction, the equilibrating vertical and horizontal forces in the i-segment result:

$$F_{v,i} = P_2 \sin\theta_2 - P_1 \sin\theta_1 \qquad F_{h,i} = P_2 \cos\theta_2 - P_1 \cos\theta_1$$

while the balancing moment turns out:

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$$M_i = (P_2 \cos\theta_2 e_2 + P_1 \cos\theta_1 e_1) - (P_2 \sin\theta_2 + P_1 \sin\theta_1) \cdot a/2$$

The above procedure should be repeated for all the segments. It can be notice that the forces at the end of each segment extremity are the same with opposite versus if the right or the left segment is considered; these forces cancel out themselves with the exception at anchorages. Finally, for each tendon, the forces at the extremity of the cable plus the equilibrating system for each segment, shall be introduced in the FEM model.

The choice of the position of the elementary segments is relative to the kind of element adopted in the FEM model. If beam elements are used, it is possible to introduce a point load (or moment) whether along the element body or at nodes, consequently the segment extremities may placed indifferently at nodes or at the middle of the element. With shell elements, only nodal forces can be considered so that it is necessary to place segment extremities within two sequential nodes; furthermore, due to the two-dimensional scheme, one has to consider the transverse position of the tendon that, in general, do not coincide with a nodal alignment. As simple rule, the indications of Fig. 7.2 - 9 may be followed.

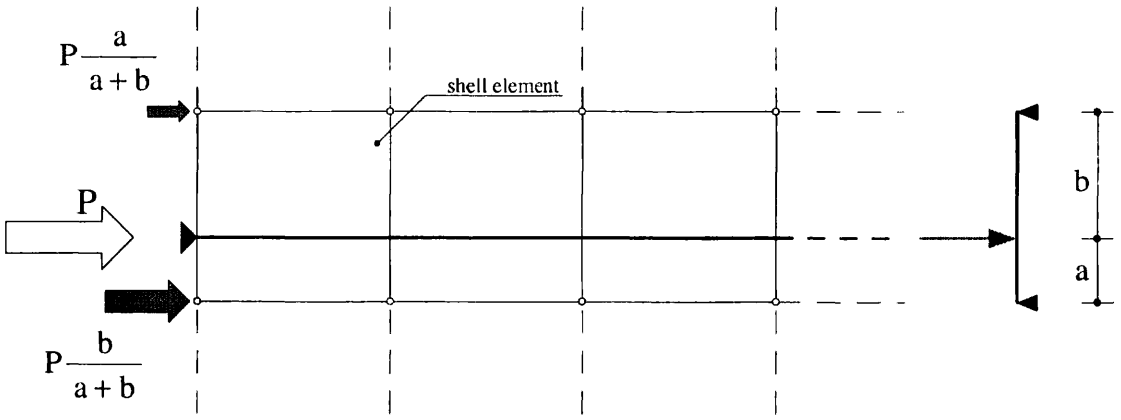


Fig. 7.2 - 9: Transverse distribution of prestressing

(3) Time-dependent prestressing losses

Time-dependent losses of prestress may be evaluated by means of the following equation:

$$\Delta\sigma_{p,c+s+r} = \frac{\varepsilon_{cs}(t_\infty, t_0)\, E_s + \Delta\sigma_{pr} + \alpha\, \phi(t_\infty, t_0)\,(\sigma_{cg} + \sigma_{cp0})}{1 + \alpha \dfrac{A_p}{A_c}\left[\left(1 + \dfrac{A_c}{I_c} z_{cp}^2\right)\left(1 + 0.8\,\phi(t_\infty, t_0)\right)\right]}$$

where:

$\Delta\sigma_{p,c+s+r}$: loss of initial tendon stress due to creep and shrinkage of concrete and relaxation of steel, between time t_0 and time t_∞;

$t_0 = 28$ days: age of concrete at prestressing time;

$t_\infty = 25550$ ds.: corresponding to a life-time of 70 years;

$\varepsilon_{cs}(t_\infty, t_0)$: shrinkage strain at time t_∞ calculated from:

$$\varepsilon_{cs}(t_\infty, t_0) = \varepsilon_{cs0} \cdot \beta_s (t_\infty - t_0) = -0.127 \cdot 10^{-3}$$

where: $\varepsilon_{cso} = \varepsilon_s (f_{cm}) \cdot \beta_{RH}$ with:

$$\varepsilon_s (f_{cm}) = [160 + 10\,\beta_{sc}\,(9 - f_{cm}/f_{cmo})] \cdot 10^{-6} = 0.000395$$

f_{cm} = mean compressive strenght of concrete at 28 days = f_{ck} + 8 MPa;

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f_{cmo} = 10 MPa;

β_{sc} = 5 for rapid hardening cements;

$$\beta_{RH} = -1.55\left[1-\left(\frac{RH}{100}\right)^3\right] = -1.018;$$

RH = 70 % relative humidity of the ambient atmosphere;

$$\beta_s(t_\infty - t_0) = \sqrt{\frac{t_\infty - t_0}{0.035\cdot h^2 + t_\infty - t_0}} = 0.574$$

$h = (2A_c / u) = 1217$ mm notional size of member;

$A_c = 17.43 \cdot 10^6$ mm^2 gross section of the beam;

u = 28640 mm perimeter of the member in contact with the atmosphere;

$\phi(t_\infty,t_0)$: creep coefficient at time t_∞ calculated from:

$\phi(t_\infty,t_0) = \phi_0 \cdot \beta_c(t_\infty - t_0) = 1.5708$ where:

$\phi_o = \phi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) = 1.598$ with

$$\phi_{RH} = 1 + \frac{1 - RH/100}{0.1\sqrt[3]{h}} = 1.281;$$

$$\beta(f_{cm}) = \frac{5.3}{\sqrt{f_{cm}/f_{cmo}}} = 2.556;$$

$$\beta(t_0) = \frac{1}{0.1 + t_0^{0.2}} = 0.488$$

$$\beta_c(t_\infty - t_0) = \left(\frac{t_\infty - t_0}{\beta_H + t_\infty - t_0}\right)^{0.3} = 0.983 \quad \text{with}$$

$$\beta_H = 1.5\left[1 + (0.012\,RH)^{18}\right]h + 250 = 2155 > 1500 \rightarrow 1500$$

If the improved prediction model of chapter 3 is used, the following values for $\varepsilon_{cs}(t_\infty, t_0)$ and for $\phi(t_\infty, t_0)$ may be evaluated:

$$\bar{\varepsilon}_{cs}(t_\infty, t_0) = 182.62 \cdot 10^{-6} \quad ; \quad \bar{\phi}(t_\infty, t_0) = 1.5754$$

in good aggrement with the previous one, at least for creep value.

$\Delta\sigma_{pr}$: loss of prestressing due to relaxation of steel calculated for a reduced initial tensile stress of $\sigma_p = \sigma_{pgo} - 0.3\,\Delta\sigma_{p,c+s+r}$ (where σ_{pgo} is the effective initial stress in tendons due to dead load and prestressing) and evaluated as percentage by the following formula:

$$\rho_t = \rho_{1000h}\left(\frac{t}{1000}\right)^{0.19} = \rho_{1000h} \cdot 3 \quad \text{where}$$

ρ_t = is the relaxation after t hours; for t > 50 years $\rho_{t.} = \rho_{1000h} \cdot 3$;

ρ_{1000h} = is the relaxation after 1000 hours evaluated from Fig. 7.2.10 ;

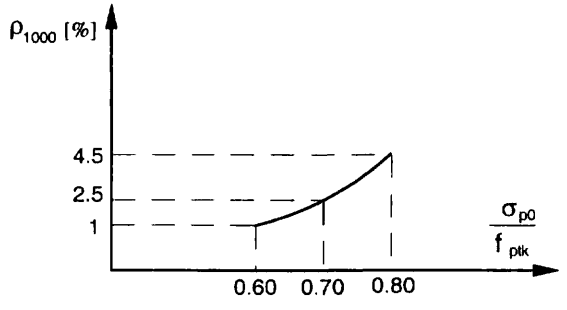


Fig. 7.2 - 10: Relaxation losses in % at 1000 hours for Class 2

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σ_c : stress on concrete at level of pretensioned steel due to self weight and permanent load;
σ_{cpo} : stress on concrete at level of pretensioned steel due to prestressing;
$\alpha = E_s/E_c$: modulus of elasticity ratio;
A_p : area of prestressing steel at the considered level;
A_c : area of concrete gross section;
I_c : inertia of concrete gross section;
z_{cp} : lever arm between centroid of concrete gross section and prestressing steel.

Time-dependent losses of prestressing should be calculated for each tendon along his profile so that a correct value may be used for each element. As a reference, the maximum value of prestressing losses, as percentage of initial steel tension, turn out:
longitudinal tendon: 19% at anchorage and 14% at pier axis;
transverse tendon: 18% at anchorage and 12% at midspan.

The effects of losses are taken into account with the same procedure used for the prestressing, but as actions of opposite sign.

7.2.3 Actions

The external loads applied on the structure should be evaluated according to the provisions of Eurocode 1.3 Traffic Load on Bridges. As vertical train load the load model LM71 plus the load models SW (SW/0 and SW/2 respectively) have been adopted with an α coefficient of 1.1. For the LM71, the 4 point loads have been reduced in an equivalent uniform load by smearing their characteristic value Q_{vk} along the influence length so that a $q_{vk,1}$ may be obtained:

$$Q_{vk} = 1.1 \cdot 250 \cdot \phi_{din} = 319.6 \text{ kN} \quad \rightarrow \quad q_{vk,1} = 319.6/1.6 = 199.75 \text{ kN/m}$$

where ϕ_{din} , being the dynamic factor equal to 1.162, is evaluated below.

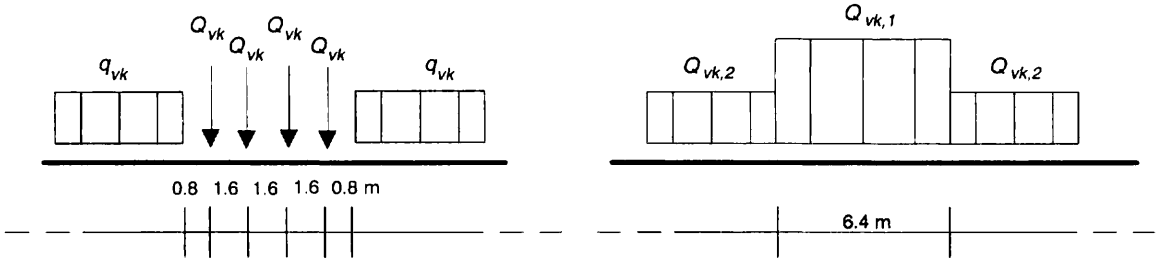


Fig. 7.2 - 11: Adopted load arrangement for LM71 load model

The uniformly distributed load q_{vk} according to Eurocode 1.3 is:

$$q_{vk} = 1.1 \cdot 80 \cdot \phi_{din} \quad \rightarrow \quad q_{vk,2} = 102.3 \text{ kN/m}$$

without any limitation in length. Fig. 7.2 - 11 shows the LM71 arrangement adopted in the calculations.

The load model SW/0 is represented in Fig. 7.2 - 12 and its characteristic value results:

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$$q_{vk} = 1.1 \cdot 133 \cdot \phi_{din} = 170.0 \text{ kN/m}$$

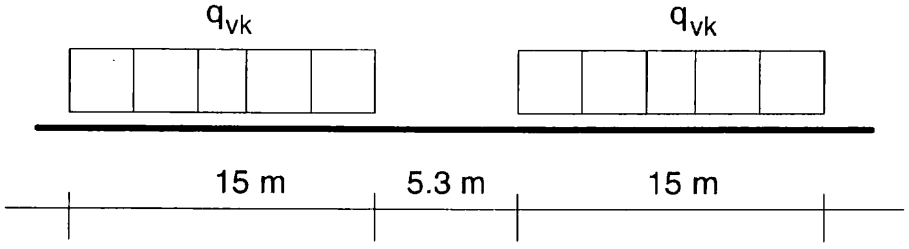


Fig. 7.2 - 12: Load model SW/0

The load model SW/2 is represented in Fig. 7.2 - 13 and its characteristic value results:

$$q_{vk} = 1.1 \cdot 150 \cdot \phi_{din} = 174.3 \text{ kN/m}$$

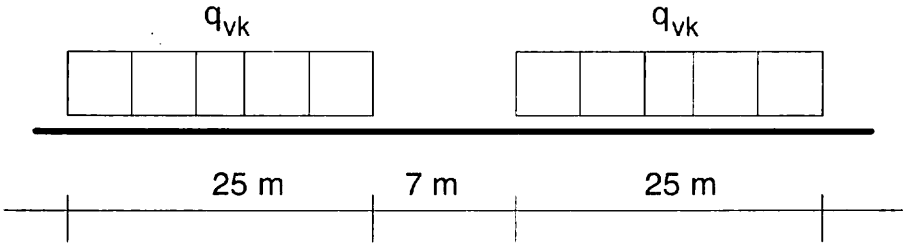


Fig. 7.2 - 13: Load model SW/2

The previous load model LM71, SW/0 and SW/2 have been introduced in the FEM analysis considering a spreading ratio of 4:1 in the ballast and of 1:1 in the concrete up to the middle plane of the slab. In the following as left track is denoted the track which has a positive value for the *y* co-ordinate, while right truck the other one. Fig. 7.2 - 14 shows which elements are involved by spreading effects, therefore subjected to variable load.

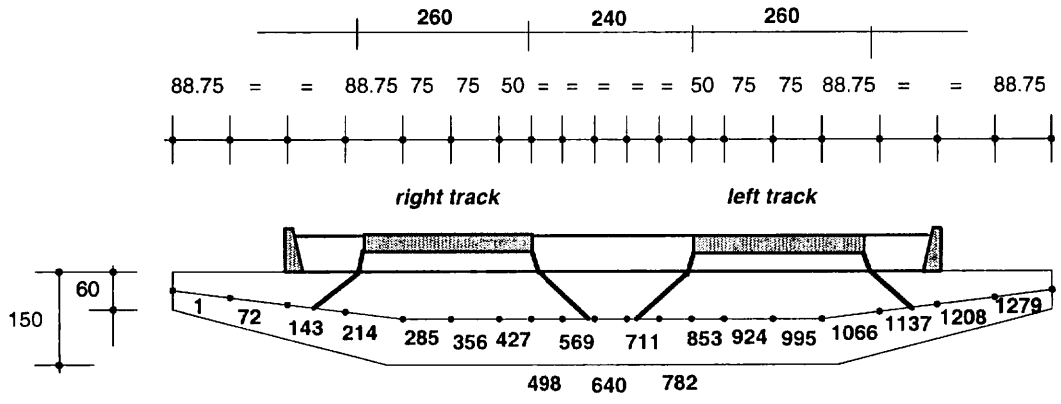


Fig. 7.2 - 14: Spreading effects on FEM model and loaded elements

The dynamic factor ϕ is calculated by means of the following expression (track with standard maintenance):

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$$\phi_3 = \frac{2.16}{\sqrt{L_\phi} - 0.2} + 0.73 = 1.162$$

where L_ϕ is the *determinant* length defined in the Eurocode 1.3 as:

$$L_\phi = 1.3\frac{L_1 + L_2 + L_3}{3} = 1.3\frac{17.33 + 27.75 + 17.33}{3} = 27.04\ \text{m}$$

Several other actions, arising from variable loads, should be considered in the analysis (as traction and braking, centrifugal forces, derailment, wind pressure, differential temperature variation etc.) but, for the sake of simplicity, in these calculations only the following actions have been considered (introduced in the mathematical model in different steps):

- STEP 1: Self-weight of the structure: adopting a unit weight value of γ = 25 kN/m3;
- STEP 2: Prestressing forces at time of tensioning;
- STEP 3: Prestressing forces after time-dependent losses:
  in the calculations, a limit value of tensile stress in tendon equal to $0.6 f_{ptk}$ after allowance for losses (t_∞), has been considered, according to the provisions of the applied Railway Code to avoid the risk of brittle failure due to stress corrosion.
- STEP 4: Track load comprehensive of;
  rails, sleepers and ballast (waterproofing included) evaluated as a cover with a nominal height of 0.8 m and a unit weight of γ = 18 kN/m^3), so that for a width of 9.5 m, an uniformly distributed load results:
  $g_{ballast} = 0.8 \cdot 1.8 \cdot 9.5 = 136.8$ kN/m;
- STEP 5: Others permanent loads composed by;
  transverse gradient for drain water, assumed as a load of 1.25 kN/m^2 it turns out:
  $g_{drain} = 1.25 \cdot 9.5 = 11.875$ kN/m;
  ballast retaining walls (with a cross section area of 0.25 m^2 and unit weight of 25 kN/m^3)
  $g_{walls} = 25 \cdot 0.25 = 6.25$ kN/m for each;
  ducts:
  $g_{ducts} = 3$ kN/m for each;
  border curbs (with a cross section area of 0.1 m^2 and unit weight of 25 kN/m^3):
  $g_{reinf\ beam} = 25 \cdot 0.25 = 6.25$ kN/m for each;
  noise barriers:
  $g_{barriers} = 8.00$ kN/m for each;
- STEP 6: Variable loads for maximum bending moment on first span (x = 6.18 m);
  the applied load is a LM71 model on the left track with the following longitudinal arrangement:

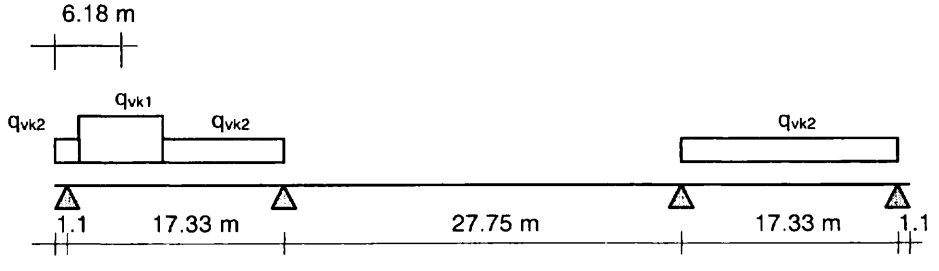


Fig. 7.2 - 15: LM71 arrangement for Load Step 5

plus a SW/2 train on the right track with the following longitudinal arrangement:

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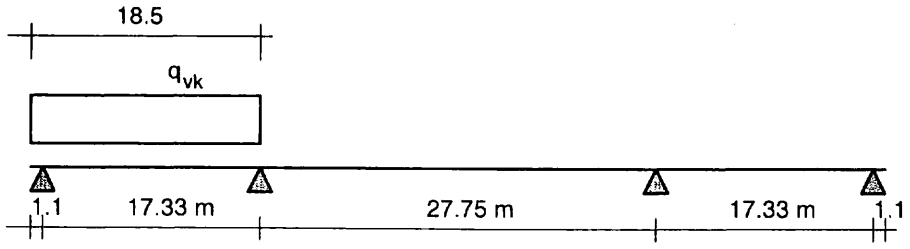


Fig. 7.2 - 16: SW/2 arrangement for Load Step 5

– STEP 7: Variable loads for minimum bending moment at pier P1 (x = 18.43 m); the applied load is a SW/0 model on the left track with the following longitudinal arrangement:

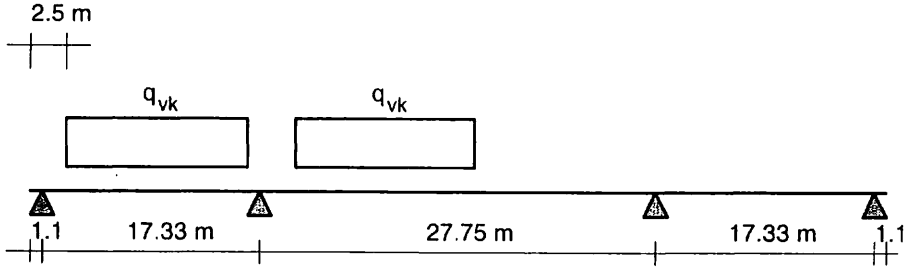


Fig. 7.2 - 17: SW/0 arrangement for Load Step 6

plus a SW/2 train on the right track with the following longitudinal arrangement:

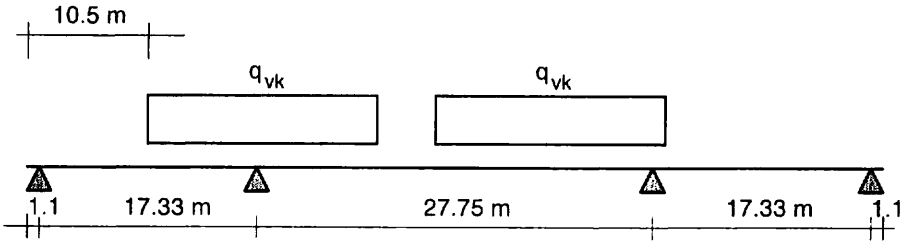


Fig. 7.2 - 18: SW/2 arrangement for Load Step 6

– STEP 8: Variable loads for max bending moment on second span (x = 32.305 m); the applied load is a LM71 model on the left track with the following longitudinal arrangement:

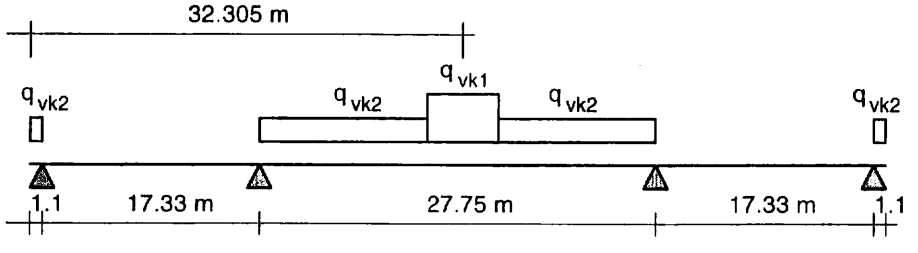


Fig. 7.2 - 19: LM71 arrangement for Load Step 7

plus a SW/2 train on the right track with the following longitudinal arrangement:

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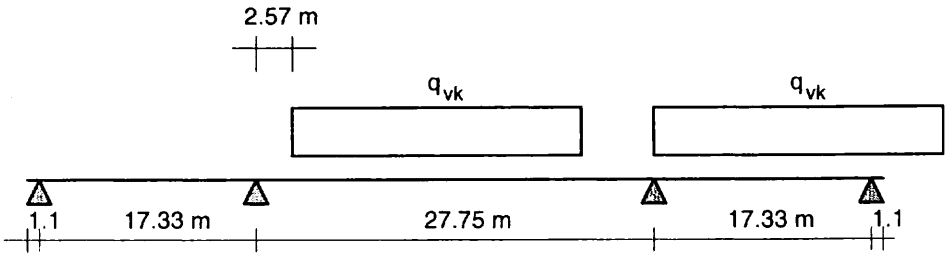


Fig. 7.2 - 20: SW/2 arrangement for Load Step 7

7.2.4 Combinations of actions

The design values for the external actions have been calculated adopting the combinations of loads specified in the applied Code as follow indicated in the symbolic presentation:

- Ultime Limit State

$$S_d = S\left\{\gamma_{G1}\, G_{1k} + \gamma_{G2}\, G_{2k} + \gamma_p\, P_k + \gamma_Q\left(Q_{1k} + \sum_{i>1} \Psi_{oi}\, Q_{ik}\right)\right\} \qquad (7.2 - 1)$$

- Serviceability Liit State: rare combination

$$S_d = S\left\{G_{1k} + G_{2k} + P_k + Q_{1k} + \sum_{i>1} \Psi_{oi}\, Q_{ik}\right\} \qquad (7.2 - 2)$$

- Serviceability Limit State: quasi-permanent combination

$$S_d = S\left\{G_{1k} + G_{2k} + P_k + \sum_{i>1} \Psi_{2i}\, Q_{ik}\right\} \qquad (7.2 - 3)$$

where:

G_{1k} = characteristic value of the action due to self-weight and permanent loads, ballast excluded;
G_{2k} = characteristic value of action due to ballast self-weight;
P_k = characteristic value of action due to prestress;
Q_{1k} = characteristic value of action due to the base variable action;
Q_{ik} = characteristic value action due to of the other independent variable loads;
γ_1 = partial factor of self-weight and permanent loads, ballast excluded, equal to 1.4 for unfavourable effect and 1.0 for favourable effect;
γ_2 = partial factor of ballast load equal to 1.8 for unfavourable effect and 1.0 for favourable effect;
γ_P = partial factor of prestress load equal to 1.2 for unfavourable effect and 0.9 for favourable effect;
γ_Q = partial factor of variable loads equal to 1.5 for unfavourable effect and 0.0 for favourable effect;
Ψ_{0i} = combination factor of variable loads equal to 0.8;
Ψ_{2i} = combination factor of variable loads for quasi-permanent combination at service, equal to 0.6 .

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7.2.5 Verification at serviceability limit state

The verification at serviceability limit state is relative to the following conditions:
- stress limitation at tensioning;
- stress limitation at service;
- crack widths;
- deformation.

(1) Verification at tensioning

At time of tensioning, no tensile stress should be present in the extreme fibres of the slab and the maximum compressive stress should not exceed the limit value of $0.6 f_{ck} = 21$ MPa. For the sake of simplicity, one reports the verification related to the four elements showed in fig 7.2 - 4, as subjected to the higher stress level.

The external actions are calculated adopting the rare combination (7.2 - 2) with only the load steps 1 and 2. From FEM analysis, the value of n_{22}, m_{22}, n_{33}, m_{33}, n_{23}, m_{23} are evaluated so that it results:

$$\sigma_{y,t} = \sigma_{22,t} = \frac{n_{22}}{h} - \frac{6\,m_{22}}{h^2} \quad ; \quad \sigma_{y,b} = \sigma_{22,b} = \frac{n_{22}}{h} + \frac{6\,m_{22}}{h^2}$$

$$\sigma_{x,t} = \sigma_{33,t} = \frac{n_{33}}{h} - \frac{6\,m_{33}}{h^2} \quad ; \quad \sigma_{x,b} = \sigma_{33,b} = \frac{n_{33}}{h} + \frac{6\,m_{33}}{h^2}$$

$$\sigma_{xy,t} = \sigma_{23,t} = \frac{n_{23}}{h} - \frac{6\,m_{23}}{h^2} \quad ; \quad \sigma_{xy,b} = \sigma_{23,b} = \frac{n_{23}}{h} + \frac{6\,m_{23}}{h^2}$$

where the subscripts t and b indicate respectively top and bottom fibre. The angles of principal directions (for which is $\sigma_{xy} = 0$) are:

$$\theta_1 = \frac{1}{2}\arctan\left(\frac{2\,\sigma_{23}}{\sigma_{22} - \sigma_{33}}\right) \quad ; \quad \theta_2 = \theta_1 + 90°$$

and the principal stresses result:

$$\sigma_{1,t/b} = \sigma_{22,t/b}\cos^2(\theta_1) + \sigma_{33,t/b}\sin^2(\theta_1) + \sigma_{23,t/b}\sin(2\theta_1)$$

$$\sigma_{2,t/b} = \sigma_{22,t/b}\cos^2(\theta_2) + \sigma_{33,t/b}\sin^2(\theta_2) + \sigma_{23,t/b}\sin(2\theta_2)$$

Referring to the elements marked in Fig. 7.2 - 4 one obtains:

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Elem.	h [m]	n_{22} [kN/m]	n_{33} [kN/m]	n_{23} [kN/m]	m_{22} [kNm/m]	m_{33} [kNm/m]	m_{23} [kNm/m]
648	1.5	-3091	-13159	6	-225	-2176	0
93	0.963	-7806	-8526	75	743	456	-51
320	1.5	-3516	-10418	1	-45	-812	0
589	1.5	-4280	-10007	-67	653	1945	20

$\sigma_{22,b}$ [Mpa]	$\sigma_{33,b}$ [Mpa]	$\sigma_{23,b}$ [Mpa]	$\sigma_{22,t}$ [Mpa]	$\sigma_{33,t}$ [Mpa]	$\sigma_{23,t}$ [Mpa]	$\theta_{1,b}$ [°]	$\theta_{2,b}$ [°]	$\theta_{1,t}$ [°]	$\theta_{2,t}$ [°]
-2.66	-14.58	0.00	-1.46	-2.97	0.00	0.02	89.98	0.15	89.85
-3.30	-5.90	-0.25	-12.91	-11.80	0.41	-5.48	95.48	-18.17	108.17
-2.46	-9.11	0.00	-2.22	-4.78	0.00	0.01	89.99	0.01	89.99
-1.11	-1.48	0.01	-4.59	-11.86	-0.10	1.29	88.71	-0.77	90.77

$\sigma_{1,b}$ [Mpa]	$\sigma_{2,b}$ [Mpa]	$\sigma_{1,t}$ [Mpa]	$\sigma_{2,t}$ [Mpa]
-2.66	-14.58	-1.46	-2.97
-3.27	-5.83	-13.05	-12.15
-2.46	-9.11	-2.22	-4.78
-1.11	-1.48	-4.59	-11.85

which not exceed the limit one.

(2) Verification of limit state of stress limitation in concrete

The serviceability limit states checked in this section are relative only to stress limitation, ensuring that, under service load conditions, concrete extreme stresses do not exceed the corresponding limit, for the quasi-permanent and the rare combinations. The limit stresses for concrete are:

- Quasi-permanent combination: Compressive stress = 0.4 f_{ck} = 14.00 MPa
- Rare combination: Compressive stress = 0.6 fck = 21.00 MPa

Applying to the structural FEM model the variable loads and combining them according to the railway code provisions, one obtain the maxima stress values at top and bottom fibres that have to be lower than the corresponding limit. One reports the results relative to the four elements indicated in Fig. 7.2 - 4.

Quasi-Permanent Combination

Elem.	h [m]	n_{22} [kN/m]	n_{33} [kN/m]	n_{23} [kN/m]	m_{22} [kNm/m]	m_{33} [kNm/m]	m_{23} [kNm/m]
648	1.5	-2420	-10152	4	-236	-1576	4
93	0.963	-6233	-6347	50	589	108	-37
320	1.5	-3539	-7855	2	81	233	4
589	1.5	-2736	-7900	-3	-151	-396	0

$\sigma_{1,b}$ [Mpa]	$\sigma_{2,b}$ [Mpa]	$\sigma_{1,t}$ [Mpa]	$\sigma_{2,t}$ [Mpa]
-2.24	-10.97	-0.98	-2.57
-2.65	-5.86	-10.31	-7.37
-2.14	-4.62	-2.58	-5.86
-2.23	-6.32	-1.42	-4.21

Rare Combination

Elem.	h [m]	n_{22} [kN/m]	n_{33} [kN/m]	n_{23} [kN/m]	m_{22} [kNm/m]	m_{33} [kNm/m]	m_{23} [kNm/m]
648	1.5	-2238	-10270	3	-226	-615	4
93	0.963	-6284	-6033	4	577	-133	-62
320	1.5	-2604	-7479	7	7	1279	-9
589	1.5	-3791	-8243	-55	-689	-1275	-26

$\sigma_{1,b}$ [Mpa]	$\sigma_{2,b}$ [Mpa]	$\sigma_{1,t}$ [Mpa]	$\sigma_{2,t}$ [Mpa]
-2.09	-8.49	-0.89	-5.21
-2.76	-7.02	-10.29	-5.51
-1.72	-1.58	-1.75	-8.40
-4.36	-8.89	-0.69	-2.09

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(3) Verification of serviceability limit state of cracking

The characteristic crack width should be calculated according to the provisions of Model Code 90. It has be notice, however, that from stress calculation neither for the quasi-permanent combination nor in the rare one, the maximum stress results tensile. Therefore, no specific reinforcement is required and it is sufficient to arrange the minimum amount of reinforcing steel, able to ensure a ductile behaviour in case of corrosion of prestressing steel.

(4) Deformation

Deformation limitation is carried out to control the maximum vertical deflection for passengers comfort. The limit values δ/L (deflection/span Length) are given by the Eurocode 1.3 as a function of the span length and the train speed. The limit value for maximum vertical deflection is calculated considering a span length of 27.75 m (central span) and a train speed *over 280* km/h; according to the provisions of the Code, it results:

$$\frac{\delta}{L} = \frac{1}{1600}$$

that should be multiplied by a factor 1.1 for continuous structures; finally, the following limit may be achieved:

$$\frac{\delta_{lim}}{L} = \frac{1.1}{1600} = \frac{1}{1455}$$

As a consequence of the transient nature of this event, the elastic deflection, calculated by the FEM model, is relative to the only live load; the check shall be performed loading only one track, reading the maximum deflection in correspondence of the track axis. Having loaded the right track with a LM71 load model plus dynamic allowance, placed in the middle of the of the central span, the obtained δ/L value is:

$$\frac{\delta_{effective}}{L} = \frac{0.0055}{27.75} = \frac{1}{5045}$$

and it results lower than the corresponding limit.

It can be notice that no further calculation is requested because, due to prestressing effect, the structure remains entirely compressed, so that the elastic value, calculated by the FEM analysis, has to be considered.

7.2.6 Verification of ultimate limit state

Verification at ULS should regard the structure as a whole and its component parts, analysing the resistance of the critical regions. In addition to the analysis of ULS of several shell element under the relevant combination of internal actions, in this example some case of *detailing* are investigated, i.e.:

- bursting force at anchorage of prestressing tendon;
- spalling force at anchorage of prestressing tendon;
- punching under support plate.

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(1) Slab ultimate limit state

Verification at ULS has been performed adopting the sandwich model for shell elements. The internal actions in a shell element at ULS are sketched in Fig. 7.2 - 21.

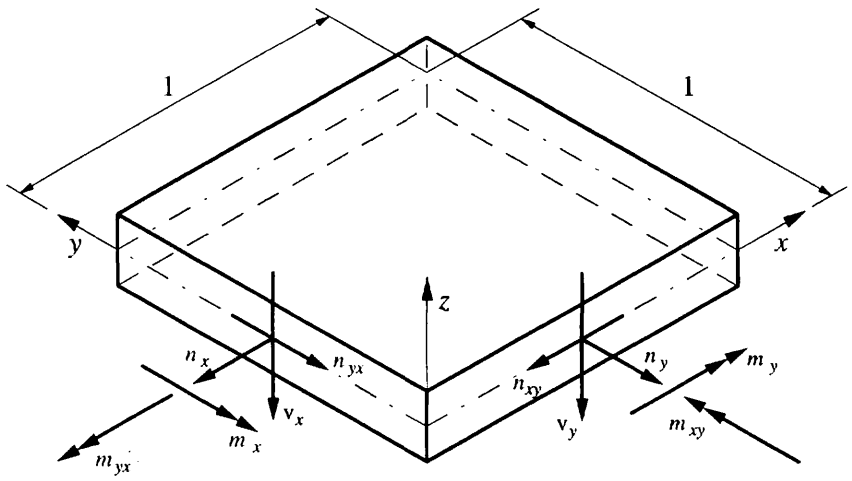


Fig. 7.2 - 21: Internal actions at ULS in a shell elements

Let us consider in this section only four elements on the whole (see Fig. 7.2 - 4). The external actions are derived from FEM model using the load step for trains which leads to the maximum values and combining the results according to the relevant combination formula. For the investigated elements, turns out (on brackets the notation of Fig. 7.2 - 5):

Elem.	h [m]	$n_{Sd,y}$ (n_{22}) [kN/m]	$n_{Sd,x}$ (n_{33}) [kN/m]	$n_{Sd,xy}$ (n_{23}) [kN/m]	$m_{Sd,y}$ (m_{22}) [kNm/m]	$m_{Sd,x}$ (m_{33}) [kNm/m]	$m_{Sd,xy}$ (m_{23}) [kNm/m]	$v_{Sd,y}$ (v_{13}) [kN/m]	$v_{Sd,x}$ (v_{12}) [kN/m]
648	1.5	-1779	-9096	5	-239	470	-14	-6	-5
93	0.963	-5746	-4610	-63	499	-671	-75	89	-150
320	1.5	-2130	-5922	10	38	3241	-13	20	47
589	1.5	-3865	-7748	-54	-1950	-4274	-41	-1124	-1095

As first step, one may design the inner layer checking if specific shear reinforcement is required or not. In fact, it is possible to calculate the principal shear $v_0^2 = v_x^2 + v_y^2$, on the principal shear direction φ_0 (such that $\tan\varphi_0 = v_y/v_x$), and to check that it turn out:

$$v_0 < v_{Rd1} = 0.12\,\xi\,(100\,\rho\,f_{ck})^{1/3}\ d \qquad (7.2 - 4)$$

where v_{Rd1} is specified in chapter 6.4.2.3 of MC 90 and $\rho = \rho_x \cos^2\varphi_0 + \rho_y \sin^2\varphi_0$. If the (7.2 - 4) is not satisfied, specific shear reinforcement shall be arranged (vertical stirrups) and diagonal compressive forces in concrete shall be checked. According to CEB Bulletin 223, and having set a minimum amount of longitudinal and transverse reinforcement in the bottom and top layer of:

$$A_{s,x} = A_{s,y} = 22.6\ cm^2/m$$

placed at 0.07 m from the external face, the following table may be calculated for the elements

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considered:

Elem.	d [m]	φ_o [°]	ρ_o [-]	v_o [kN/m]	v_{Rdl} [kN/m]	θ [°]	F_{Scw} [kN/m]	F_{Rcw} [kN/m]	A_s/s^2 [cm²/m²]	n_{xc} [kN/m]	n_{yc} [kN/m]	n_{xyc} [kN/m]
648	1.43	51.18	0.00158	7	417	26.56	-	-	-	0.0	0.0	0.0
93	0.893	-30.77	0.00253	174	327	26.56	-	-	-	0.0	0.0	0.0
320	1.43	23.14	0.00158	51	417	26.56	-	-	-	0.0	0.0	0.0
589	1.43	45.76	0.00158	1569	417	26.56	3509	13860	14.0	763.9	805.6	784.5

with variation of slab components due to v_x and v_y (i.e. n_{xc} , n_{yc} and n_{xyc}) only for element number 589.

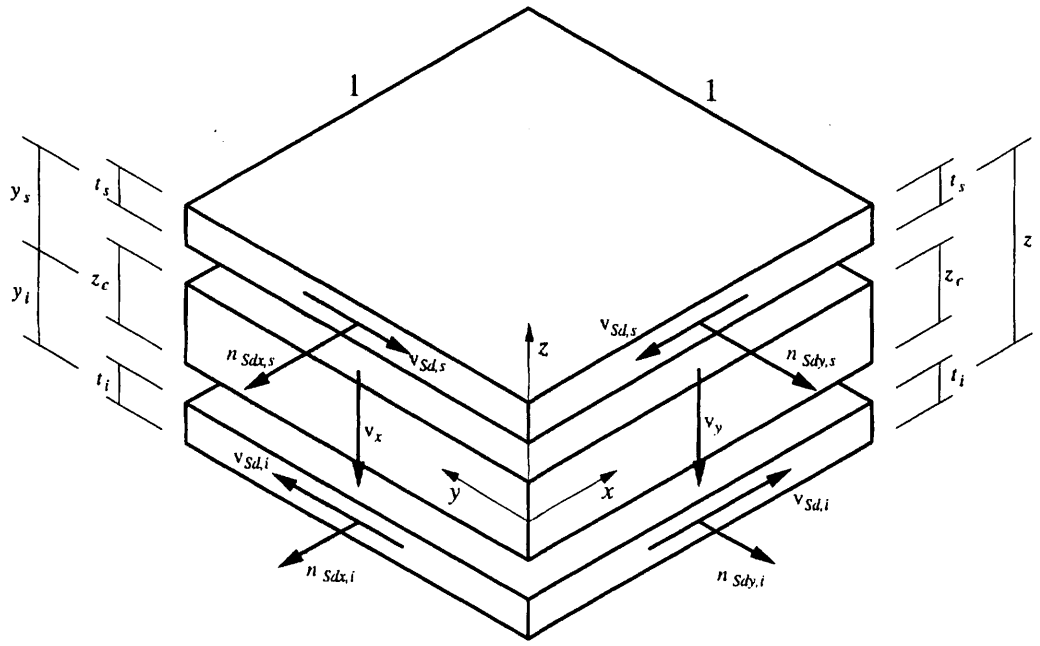


Fig. 7.2 - 22: Internal forces in the different layers

The outer layers should be designed supposing an initial thickness for both layers not lesser than twice the concrete cover evaluated at the centroid of reinforcement. One assumes:

$$t_s = t_i = 2 \cdot 0.07 = 0.14 \text{ m}$$

so that, internal lever arm z and in plane actions may be evaluated for the outer layers of each element referring to Fig. 7.2 - 22 and by means of the following equations:

$$n_{Sdx,s} = n_x \frac{z - y_s}{z} + \frac{m_x}{z} + \left(\frac{1}{2} \frac{v_x^2}{v_0} \cot\theta \right) \qquad n_{Sdx,i} = n_x \frac{z - y_i}{z} - \frac{m_x}{z} + \left(\frac{1}{2} \frac{v_x^2}{v_0} \cot\theta \right)$$

$$n_{Sdy,s} = n_y \frac{z - y_s}{z} + \frac{m_y}{z} + \left(\frac{1}{2} \frac{v_y^2}{v_0} \cot\theta \right) \qquad n_{Sdy,i} = n_y \frac{z - y_i}{z} - \frac{m_y}{z} + \left(\frac{1}{2} \frac{v_y^2}{v_0} \cot\theta \right)$$

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$$v_{Sd,s} = n_{xy}\frac{z-y_s}{z} - \frac{m_{xy}}{z} + \left(\frac{1}{2}\frac{v_x v_y}{v_0}\cot\theta\right) \qquad v_{Sd,i} = n_{xy}\frac{z-y_i}{z} + \frac{m_{xy}}{z} + \left(\frac{1}{2}\frac{v_x v_y}{v_0}\cot\theta\right)$$

where terms on brackets have be summed if shear reinforcement is required. In the design procedure is convenient to reach the minimum amount of reinforcement, so that a value of 45° for θ angle may be adopted.

For the chosen elements it turns out:

Elem.	h [m]	t_s [m]	t_i [m]	t_c [m]	y_s [m]	y_i [m]	z [m]	$n_{Sdy,s}$ [kN/m]	$n_{Sdx,s}$ [kN/m]	$v_{Sd,s}$ [kN/m]	$n_{Sdy,i}$ [kN/m]	$n_{Sdx,i}$ [kN/m]	$v_{Sd,i}$ [kN/m]
648	1.5	0.140	0.140	1.220	0.680	0.680	1.360	-713.5	-4893.3	13.0	-1065.1	-4202.2	-7.7
93	0.963	0.140	0.140	0.683	0.412	0.412	0.823	-3479.6	-1489.8	59.3	-2266.7	-3120.5	-122.3
320	1.5	0.140	0.140	1.220	0.680	0.680	1.360	-1093.1	-5344.2	14.8	-1037.3	-577.9	-4.4
589	1.5	0.140	0.140	1.220	0.680	0.680	1.360	307.0	32.7	787.8	-2560.7	-6252.6	726.9

At this stage each layer may be designed by applying the following equations (θ = 45°):

$$\sigma_c\, t = \frac{v_{Sd}}{\sin\theta\,\cos\theta} \le f_{cd2}\, t \qquad \text{safety verification on concrete side} \qquad (7.2 - 5)$$

$$n_{Rdx} = n_{Sdx} + v_{Sd}\cot\theta \qquad \text{required resistance along } x \text{ direction} \qquad (7.2 - 6)$$

$$n_{Rdy} = n_{Sdy} + \frac{v_{Sd}}{\cot\theta} \qquad \text{required resistance along } y \text{ direction} \qquad (7.2 - 7)$$

from which, if the (7.2 - 5) results satisfied, the reinforcement areas may be calculated as:

$$A_{sx} = \frac{n_{Rdx}}{f_{yd}} \quad ; \quad A_{sy} = \frac{n_{Rdy}}{f_{yd}}$$

If concrete strength requirement is not satisfied, an increase layer thickness shall be provided until verification is met; in this case new values for the layer action having changed z value.

It can be notice that if n_{Rdx} or n_{Rdy} value are negative, a compression force is present along that direction and no reinforcement is required; if both the n_{Rdx} and n_{Rdy} are negative it is possible to omit the reinforcement in both the directions but, in this case the verification is performed along the principal compression direction in the concrete subjected to biaxial compression and the checking equation is:

$$\sigma_c\, t = \frac{n_{Sdx} + n_{Sdy}}{2} + \sqrt{\frac{\left(n_{Sdx} - n_{Sdy}\right)^2}{4} + v_{Sd}^2} \le f_{cd1}\, t$$

For the considered elements, one obtains:

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	Top Layer Design				Bottom Layer Design			
Elem.	σ_c [MPa]	$f_{cd1/2}$ [kN/m]	A_{sy} [cm²/m]	A_{sx} [cm²/m]	σ_c [MPa]	$f_{cd1/2}$ [kN/m]	A_{sy} [cm²/m]	A_{sx} [cm²/m]
648	-35.0	-17.1	0.0	0.0	-30.0	-17.1	0.0	0.0
93	-24.9	-17.1	0.0	0.0	-22.4	-17.1	0.0	0.0
320	-38.2	-17.1	0.0	0.0	-7.4	-17.1	0.0	0.0
589	-11.3	-12.0	25.2	18.9	-45.6	-17.1	0.0	0.0

It can be notice that verification for concrete in compression is not satisfied for any layers except for element 589 top layer and element 320 bottom layer. Thus, an increasing of layer thickness is required and new values of plate actions are obtained:

Elem.	h [m]	t_s [m]	t_i [m]	t_c [m]	y_s [m]	y_i [m]	z [m]	$n_{Sdy,s}$ [kN/m]	$n_{Sdx,s}$ [kN/m]	$v_{Sd,s}$ [kN/m]	$n_{Sdy,i}$ [kN/m]	$n_{Sdx,i}$ [kN/m]	$v_{Sd,i}$ [kN/m]
648	1.5	0.300	0.240	0.960	0.600	0.630	1.230	-716.6	-5040.8	14.1	-1062.0	-4054.7	-8.8
93	0.963	0.220	0.190	0.553	0.372	0.387	0.758	-3588.5	-1465.5	66.5	-2157.8	-3144.8	-129.4
320	1.5	0.340	0.140	1.020	0.580	0.680	1.260	-1179.8	-5768.3	16.0	-950.6	-153.8	-5.5
589	1.5	0.140	0.430	0.930	0.680	0.535	1.215	708.7	870.1	794.7	-2962.5	-7090.0	720.0

which lead to the following values:

	Top Layer Design				Bottom Layer Design			
Elem.	σ_c [MPa]	$f_{cd1/2}$ [kN/m]	A_{sy} [cm²/m]	A_{sx} [cm²/m]	σ_c [MPa]	$f_{cd1/2}$ [kN/m]	A_{sy} [cm²/m]	A_{sx} [cm²/m]
648	-16.8	-17.1	0.0	0.0	-16.9	-17.1	0.0	0.0
93	-16.3	-17.1	0.0	0.0	-16.6	-17.1	0.0	0.0
320	-17.0	-17.1	0.0	0.0	-6.8	-17.1	0.0	0.0
589	-11.4	-12.0	34.6	38.3	-16.8	-17.1	0.0	0.0

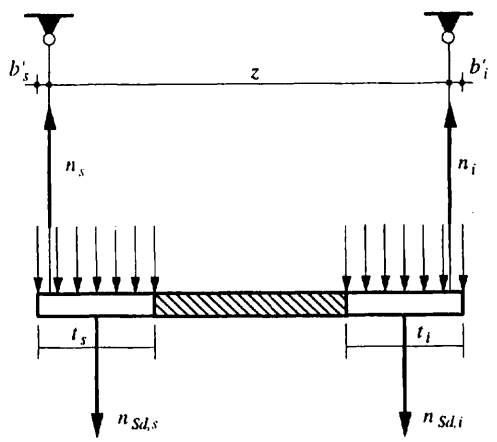


Fig. 7.2 - 23: Shell element equilibrium in one direction with two reinforcement layers only

Of course, minima values should be adopted for A_{sx} and A_{sy} if no reinforcement areas are required. For element 589, the A_{sx} and A_{sy} value are required at the centroid of the layer, whereas they are arranged at 0.07 m from the external surface of the slab in an eccentric

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position with respect to middle plane of the layer; so, the amount of reinforcement provided has to be changed to restore equilibrium conditions. This variation may be assessed with the aid of the mechanism pictured in Fig. 7.2 – 23.

The new forces acting on the reinforcements become:

$$n_s = \frac{n_{Sd,s}\left(h - \frac{t_s}{2} - b_i'\right) + n_{Sd,i}\left(\frac{t_i}{2} - b_i'\right)}{z} \qquad (7.2 - 8)$$

$$n_i = n_{Sd,s} + n_{Sd,i} - n_s \qquad (7.2 - 9)$$

For the investigated elements, the following areas have been detected.

	Forces referred to tension steel level				*Top layer reinf*		*Bottom layer reinf*	
Elem.	$n_{s,y}$ [kN/m]	$n_{i,y}$ [kN/m]	$n_{s,x}$ [kN/m]	$n_{i,x}$ [kN/m]	A_{sy} [cm²/m]	A_{sx} [cm²/m]	A_{sy} [cm²/m]	A_{sx} [cm²/m]
648	-702.5	-1070.8	-5026.6	-4063.6	0.0	0.0	0.0	0.0
93	-3522.0	-2287.2	-1399.0	-3274.3	0.0	0.0	0.0	0.0
320	-1163.8	-956.1	-5752.3	-159.3	0.0	0.0	0.0	0.0
589	1503.5	-2242.5	1664.8	-6370.0	34.6	38.3	0.0	0.0

The previous procedure should be repeated for all the elements of the structural model finding the amount of reinforcement to provide in the slab; it is useful, to control the structural behaviour and for a best fitted reinforcement layout, to summarise the results in a visual map.

(2) Verification to bursting force

For the calculation of the bursting force the symmetric prism analogy is used, evaluating the height of the prism so that his centroid results coincident with the centroid of prestressing tendons. For the sake of simplicity, only the longitudinal direction of prestressing tendon is considered with respect to the vertical plane, but transverse force due to bursting effect should be also calculated in the horizontal plane and for transverse prestressing too.

Checking situation is represented in Fig. 7.2 - 24, and the most unfavourable situation occurs when a single tendon is tensioned; considering the lower level of tendon (first tensioned) the height of the prism results:

$$h_{bs} = 2 \cdot 0.6 = 1.2 \text{ m}$$

and his length, for end anchored tendon, is:

$$l_{bs} = h_{bs} = 1.2 \text{ m}$$

while the width follows from the possible enlargement of the anchor plate that may be assumed equal to 0.43 m, corresponding to the transverse spacing of longitudinal lower tendons.

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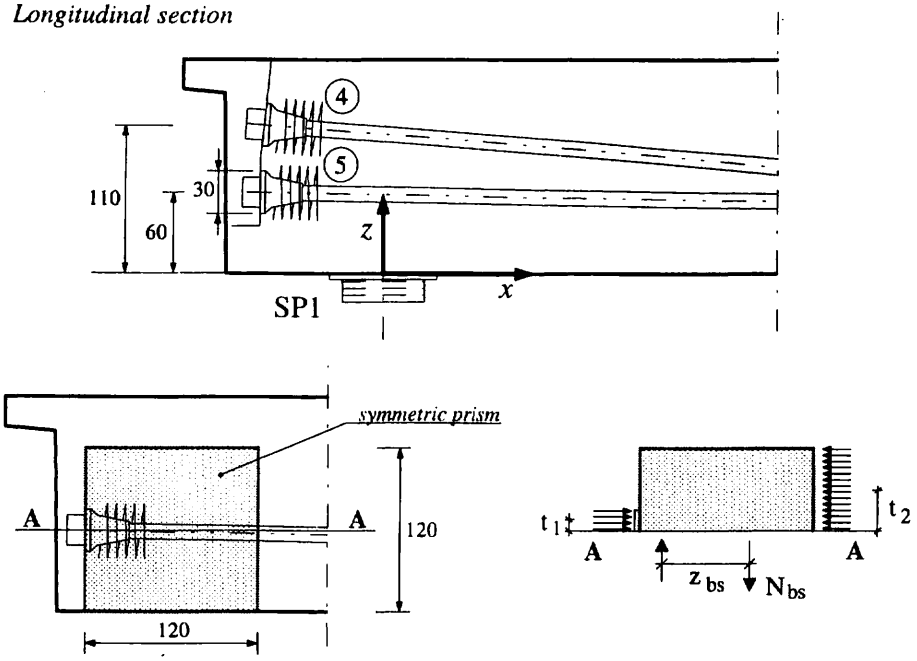


Fig. 7.2 - 24: Geometric dimensions for bursting calculation

The design force per tendon has been evaluated by means of the following expression:

$$F_{Sd} = \frac{f_{ptk}}{1.15} A_{sp} = \frac{1800}{1.15} (139 \cdot 19) \cdot 10^{-3} = 4134 \text{ kN} \qquad (7.2 - 10)$$

The bursting force follows from the moment equilibrium along section A-A:

$$N_{bs} = \frac{0.5 (n_1 + n_2) t_2 - n_1 t_1}{z_{bs}} \gamma_1 F_{Sd} = 852.6 \text{ kN} \qquad (7.2 - 11)$$

where:

$t_1 = 0.075$ m — distance between the centroid of tendons above section A-A to the centroid of the prism;

$t_2 = 0.300$ m — distance between the centroid of concrete stress block above section A-A to the centroid of the prism;

n_1 , n_2 — numbers of tendons above and below section A-A, respectively: considering the anchor plate as rigid a value of 0.5 may be assumed;

$\gamma_1 = 1.1$ — supplementary partial safety factor against overstressing by overpumping.

Bursting force shall be resisted by an area of reinforcement steel of:

$$A_{s,bs} = N_{bs} / f_{yd} = 19.61 \text{ cm}^2$$

distributed within $l_{bs}/3$ to l_{bs} , (i.e. from 0.40 m to 1.20 m) from the anchor plate. Thus the effective area on a meter length, may be found by the following:

$$\frac{A_{s,bs}}{s \cdot s} = \frac{A_{s,bs}}{b \cdot \frac{2}{3} l_{bs}} = \frac{19.61}{0.43 \cdot (1.20 - 0.4)} = 57.0 \text{ cm}^2/\text{m}^2$$

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that may be provided with ties having diameter of 22 mm and spacing both transversally and longitudinally of 250 mm (see Fig. 7.2 - 25); in fact ∅22/25×25 cm corresponds to 60.82 cm^2/m^2.

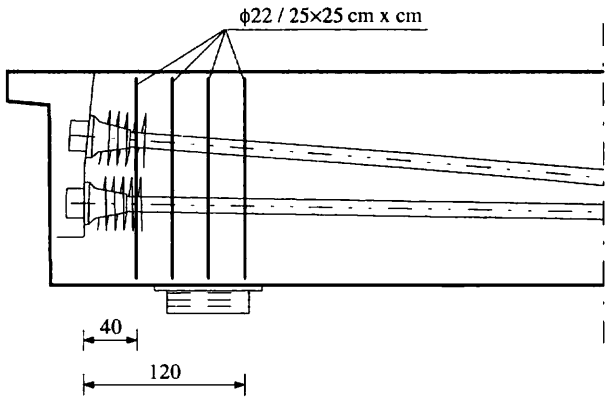


Fig. 7.2 - 25: Bursting reinforcement arrangement

(3) Verification to spalling force

The spalling force may be calculated with the equivalent prism analogy. As for bursting verification, only the longitudinal direction is considered; furthermore, spalling effects arise if upper tendon are tensioned firstly (the eccentricity leads to tension stresses). Thus, a section with a breadth of 0.43 m and a height of 1.50 m has to be verified for one tendon tensioning.

The length of the prism for end anchored tendon, is equal to the overall height of the section, i.e. l_{sl} = 1.50 m. Considering an eccentricity for upper prestressing tendon of 0.35 m, the extreme stresses at the end of prism length are calculated by means of the beam theory; for a prestressing force F_{Sd} = 4134.0 kN, evaluated with the (7.2 - 10), they result (negative if compressive):

$$\left.\begin{matrix}\sigma_{top}\\ \sigma_{bottom}\end{matrix}\right\} = F_{Sd}\left(-\frac{1}{0.43\cdot 1.50} \mp \frac{0.35\cdot 6}{0.43\cdot 1.50^2}\right) = \begin{cases}-15.38 \text{ MPa}\\ +2.56 \text{ MPa}\end{cases}$$

The section along which no shear force results, is placed at 0.428 m from slab bottom fibre (see Fig 7.2 - 26) and the moment for equilibrium turns out:

$$M_{sl} = \frac{2}{3}\sigma_{bottom}\cdot 0.214^2\cdot 0.43\cdot 10^3 = 33.61 \text{ kNm}$$

Assuming $z_{sl} = 0.5\ l_{sl}$ and b_{sl} = 0.43 m, the maximum spalling force turns out:

$N_{sl} = M_{sl} / z_{sl} = 44.81$ kN

Disregarding any concrete tensile resistance, the amount of reinforcement is:

$A_s = N_{sl} / f_{yd} = 1.031\ cm^2$

placed parallel to the end face in its close vicinity.

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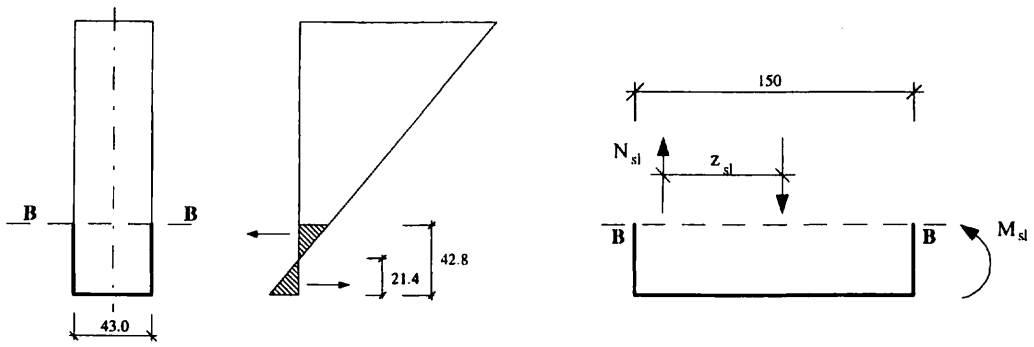


Fig. 7.2 - 26: Calculation scheme for spalling

(4) Verification to punching action

In the zone near supports high transverse stresses are reached in concrete due to the concentrated load on support plate, that may lead to local crushing. So the punching effect has to be investigated according to the provision of MC 90 regarding the slab with shear reinforcement. For the sake of simplicity prestressing effects are neglected and only ordinary reinforcement is taken into account; furthermore, in this example only one pier P1 support has been verified but all the others should be examined.

The external action results from load combination at ULS according to (7.2 - 4), and for pier P1 the higher value results: $F_{Sd} = 16029$ kN.

As first step one has to control if the slab is able to carry the punching load, calculating the maximum resistance that should be result higher than the applied load; the maximum resistance is evaluated as:

$$F_{Rd,max} = 0.5\, f_{cd2}\, u_0\, d = 0.5 \cdot 12.0 \cdot 3.77 \cdot 1.43 \cdot 10^3 = 32347 \text{ kN} > F_{Sd}$$

where u_0 is the length of the periphery of the circular support plate (see below).

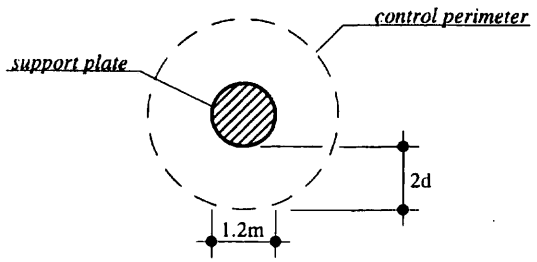


Fig. 7.2 - 27: Control perimeter for pier P1 supports

As second step, one has to evaluate if transverse reinforcement is requested; the shear stress due to loads at the control perimeter is:

$$\tau_{Sd} = \frac{F_{Sd}}{u_1\, d} = \frac{16029}{21.74 \cdot 1.43} 10^{-3} = 0.516 \text{ MPa} \qquad (7.2 - 12)$$

where:

d = 1.43 m effective depth of the slab taken as $(d_x + d_y)/2$;

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$u_1 = 21.74$ m length of the control perimeter calculated according to Fig. 7.2 - 27 for a support plate diameter of 1.2 m.

The corresponding stress resistance τ_{Rd} is given in MC 90 as:

$$\tau_{Rd} = 0.12\,\xi\,(100\,\rho\,f_{ck})^{1/3} = 0.44 \text{ MPa} \qquad (7.2 - 13)$$

with:

$\xi = 1 + \sqrt{200/d} = 1.374$ with d in mm;

$\rho = \sqrt{\rho_x \rho_y} = 0.0057$ having disposed an area of 67.1 cm^2/m in the x direction and of 98.8 cm^2/m in the y direction;

By comparing expression (7.2 - 12) with expression (7.2 - 13) it can be noticed that an amount of shear reinforcement have to be provided, so that the resistance to punching force result:

$$F_{Sd} \le 0.09\,\xi\,(100\,\rho\,f_{ck})^{1/3}\,u_1\,d + 1.5\,\frac{d}{s_r}\,A_{sw}\,f_{ywd}\,\sin\alpha \qquad (7.2 - 14)$$

where:

A_{sw} area of shear reinforcement in a layer around the plate;

s_r is the radial spacing of the layers of shear reinforcement

α is the angle between shear reinforcement and the plane of the slab, assumed equal to 90°.

The quantity A_{sw} with the (7.2 - 14) shall be calculated respecting two additional limits:

$$1.5\,\frac{d}{s_r}\,A_{sw}\,f_{ywd}\,\sin\alpha \;\ge\; 0.03\,(100\,\rho\,f_{ck})^{1/3}\,u_1\,d \qquad f_{ywd} \le 300 \text{ MPa}$$

Solving expression (7.2 - 14) with respect to A_{sw}/s_r and respecting the previous disequations, the following result is obtained:

$$\frac{A_{sw}}{s_r} = 89.7 \text{ cm}^2/\text{m}$$

that shall be arranged within the control perimeter in a circonferential layout as sketched in Fig. 7.2 - 28. If a radial spacing of 0.72 m is selected, three circumferential layers have to be arranged, and the total area of an individual layer results: $A_{sw} = 0.72 \cdot 89.7 = 64.58$ cm^2.

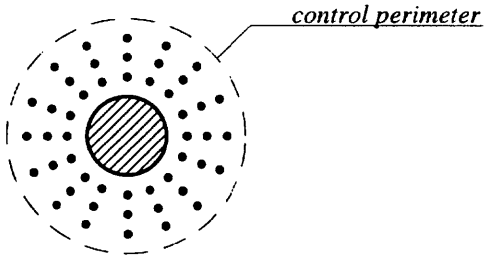


Fig. 7.2 - 28: Shear reinforcement layout for punching action

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7.3 Deep beams and discontinuity regions

by Kurt Schaefer

7.3.1 Principles and methods of design

(1) Introduction to the design of deep beams and discontinuity regions

(1.1) Definitions and general properties of D-regions

Bernoulli's assumption of linear strain distribution over the cross-section is a basis of the standard design methods for beams, columns and slabs (Chapter 4). However, Bernoulli's hypothesis is in general not valid for the design of deep beams. Fig. 7.3-1 presents some examples from a linear analysis. The span/depth ratio, for which the hypothesis is fairly well true depends on the geometric boundary conditions (cantilever, single span or continuous beam) and the type of loading (distributed or concentrated loads).

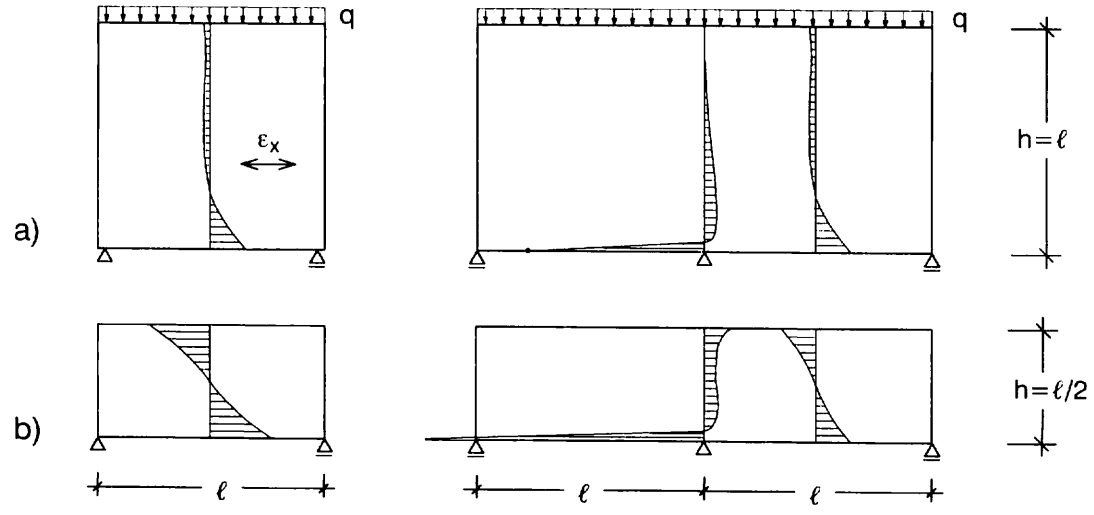


Fig. 7.3-1: Non-linear strain distribution in deep beams depending on l/h and boundary conditions

Unfortunately, the state of stress in such members cannot be derived section by section from the sectional effects M, N, V, as is normally the case for elements, whose depth is small compared with their span. The same applies also to those regions in beams and other members where the regular smooth stress distribution is disturbed due to discontinuities caused by either a member's geometry or concentrated loads. Geometrical discontinuities may be present in the shape of recesses, beam-column connections, openings, and all kinds of re-entrant corners or re-entrant edges between adjacent elements (Fig. 7.3-2 a and c). Statical discontinuities will develop for example from point loads, support reactions and prestressing anchor forces (Fig. 7.3-2 b and c).

For convenience and in recognition of the common mechanical behaviour, the label D-region will be used for both, Deep beams and Discontinuity-regions. The D fits also to Disturbed regions and Details, as discontinuity-regions are frequently termed in practise. On the other hand, B-regions (from Bernoulli, Beams, Bending) are those for which the bending theory and standard design methods apply.

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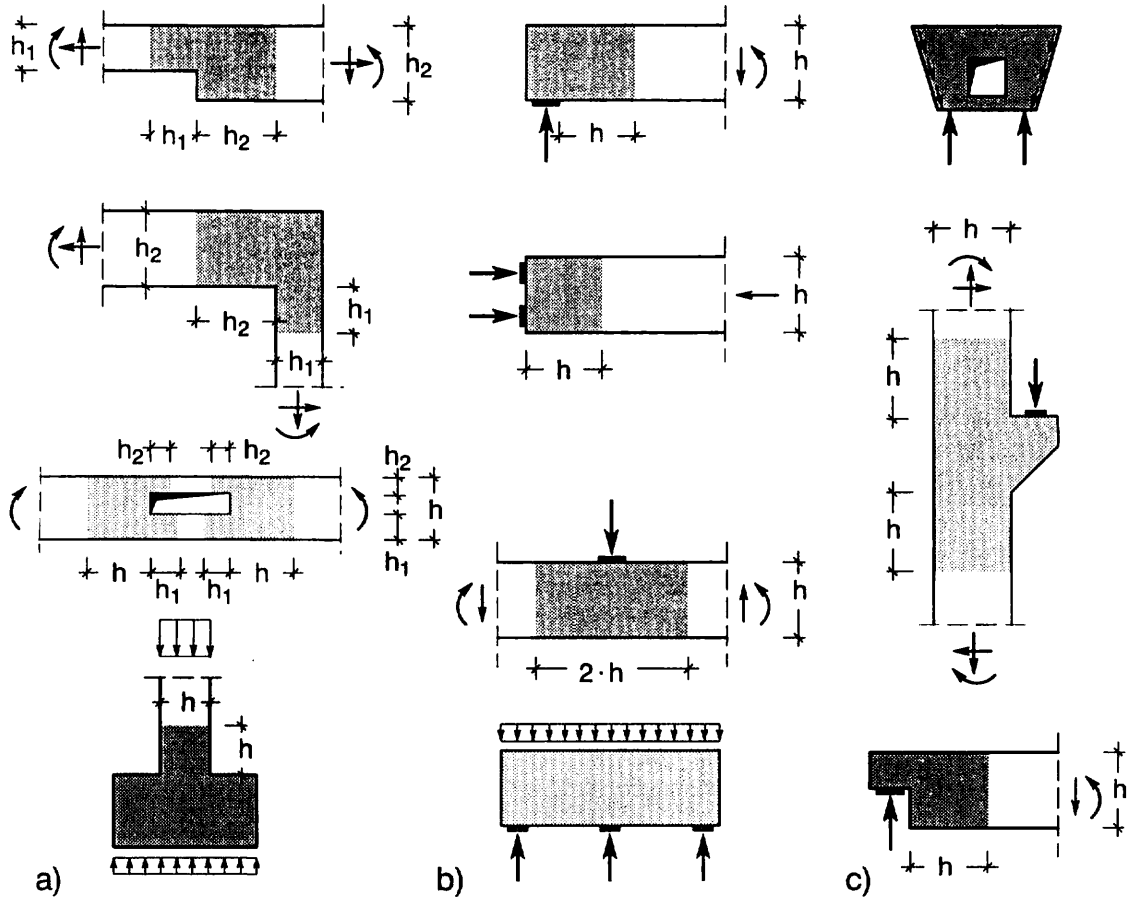


Fig. 7.3-2: D-regions (shaded areas) with nonlinear strain distribution due to:
a) Geometrical discontinuities. b) Statical discontinuities.
c) Statical and geometrical discontinuities

(1.2) The extent of D-regions

In B-regions, stresses and stress trajectories present a fairly smooth picture as against their turbulent pattern near discontinuities (Fig. 7.3-3). Stress intensities decrease rapidly with the distance from the origin of the stress concentration, e.g. an edge, opening or bearing. This behaviour allows an approximate delimitation of B- and D-regions in a structure, according to the following rule:

The D-region is confined to an area extending from the origin of disturbance approximately as far as the thickness of the cross-section (Fig. 7.3-2) [Schlaich/Weischede (1982), Schlaich/Schaefer/Jennewein (1987), Schlaich/Schaefer (1998)]. If the member thickness is different on both sides of a cross-section, then the length of the D-region to both sides is accordingly different.

The rule above is based on the Principle of Saint-Venant and therefore is not precise. No accuracy is necessary for it, since it serves only as a qualitative aid for identifying regions with disturbed stress flow and for developing the strut-and-tie models. Nevertheless, the subdivision of a structure into its B- and D-regions is of considerable value for the understanding of the internal forces in the structure. It also demonstrates that simple span/depth ratios are not satisfactory to classify beams, deep beams, short/long corbels, etc. A proper classification of members with respect to their mechanical behaviour requires that besides the geometry also the loading be considered.

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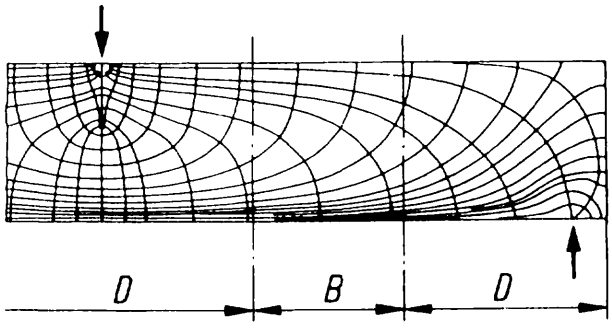


Fig. 7.3-3: Stress trajectories in a B-region and near discontinuities (D-regions)

(1.3) Overview over the design of D-regions

Normally the D-regions are investigated like in-plane loaded plates. If the flow of forces in the D-region is substantially three-dimensional, it is convenient to investigate the body in orthogonal sections as two-dimensional structures. However, the relationship between the models in different planes must be accounted for by means of the interacting forces or stresses, and the three-dimensional strength capacities for the concrete apply.

Occasionally, it may become necessary to have a closer look at certain areas of the structure by means of local models. To this end, refined models of the specific regions of interest can be used, with the boundary conditions derived from an overall model.

As long as the D-regions remain uncracked, they can be readily analysed with linear-elastic methods, using for example a finite element program. After the formation of cracks, the tensile forces are transferred to reinforcement bars which for practicality are placed parallel and orthogonal to the structure's edges. Frequently large tensile forces are concentrated in layers of reinforcement. Thereby the flow of internal forces changes considerably and linear elastic analyses are no longer realistic. If the nonlinear effects are to be considered realistically, the computer analyses turn non-economical and the results are considerably dependent on the skill of the user and his choice of adequate input parameters.

The conventional treatment of D-regions according to "past experience" or "good practise", quite often with boundary conditions that are substantially different from reality, cannot be generally satisfactory. It has, indeed, been one of the main reasons for poor performance and failures of reinforced concrete elements.

For a methodical design of D-regions in the cracked state, the internal forces (or stresses) due to loads, prestressing forces and imposed deformations (if applicable) have to be determined and checked against the respective bearing capacities of the concrete and reinforcements.

In practise, two methods are applied for the analysis of the internal forces: linear finite element-programs (FEM) or strut-and-tie models (STM). Both methods have advantages and disadvantages and are preferable in different practical cases. This will be discussed in more detail in Sections (2) and (3). Roughly simplifying, it can be summarised, that a design which is based on FEM programs is easier to apply for complicated deep beams, but has problems with concentrated forces, whilst STM are superior for standard cases of deep beams and for all D-regions in beams, columns and foundations. STM provide advise also on the anchorage of

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reinforcement. Fortunately, both methods compliment each other, and quite often it is reasonable to combine them for the design of a D-region.

(2) The design of D-regions with finite element computer programs (FEM)

The stresses in the FE mesh are automatically analysed on the basis of linear material behaviour, after the structure and its loads are specified. Some kind of input on the FE-mesh may also be required. Commercial computer programs further process the results to give the principal stresses and necessary reinforcements in orthogonal directions for a chosen grid.

If the results for two different mesh sizes differ considerably, the mesh should be refined. However, in the vicinity of singularities (point loads or re-entrant corners), convergence cannot be achieved, as the theoretical values of stresses approach infinity. Considering the plastic behaviour of concrete under high pressures and the reinforcement spacing, a refinement of FE-meshes below that limit would not really improve the design and is impracticable. An upper limit of the mesh size cannot be specified in general, as it depends on the actual structural problem, the type of FE elements used in the program, the reinforcement layout and the importance of stresses in the region considered.

Questions may also come up, on how to simplify the real boundary conditions (supports) for the FE-analysis. The results may differ considerably in the vicinity of a support area, if it is represented either by a point support, or a short line support, or several closely spaced point supports, or if a small support element is added to the structure in order to smoothen the results in the structure itself [Rueckert (1992)]. Again, it is one of the inherent shortcomings of FE-analysis, that singularities are difficult to handle with this method and FE results for such areas are not applicable for cracked reinforced concrete.

For the ultimate limit state the reinforcement is proportioned using its design strength f_{yd}. However, the corresponding steel strain is not compatible with the elastic tensile strains analysed for the structure. Furthermore, the concrete will crack in areas with high tensile stresses. Both effects lead to a (relatively modest) redistribution of internal forces in the cracked structure. Although internal forces are somewhat different now than the linear FE-results, a structure so designed is safe, as justified by the lower bound theorem of plasticity (static limit theorem see Chapter 4.1.3) and by ample experience.

A limited ductility is necessary for the redistribution of stresses. Therefore, ductility requirements are - besides crack distribution - another reason for providing a minimum reinforcement on all surfaces of concrete structures, in particular deep beams (Chapter 7.3.2).

The method of designing D-regions with FEM is so far very convenient, giving more or less automatically unique and safe results for the necessary reinforcement everywhere in the structure. The major problems of the method come up with the interpretation of the results and the development of a practical reinforcement layout.

If the structure is a large deep beam, the reinforcement can be equally spaced within strips, for which representative values of required reinforcement are taken from the FE-analysis. But in general it is better to concentrate the reinforcement near the edges of the structure, which need reinforcement anyhow for crack distribution, for folded plate action or for other effects not analysed. Such an arrangement of reinforcement is also statically more efficient (compare

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e.g. the lever arm z of the standard beam design with the lever arm *2/3 h* from an elastic analysis). However, large deviations of the reinforcement layout from the FE-results should be avoided, since the consequences are difficult to judge. The arguments in Section (3.2.2) for orienting the reinforcement layout at linear elastic behaviour apply here, too.

Finding a practical reinforcement layout which is consistent with the FE-results becomes more difficult with decreasing size of the D-region. In D-regions of linear members and slabs, e.g. beam-column-connections, it is normally not feasible and not necessary to arrange different strips of reinforcement over the depth of the element. In these cases the strut-and tie models (Section (3)) are, also for other reasons, clearly superior to FE-analyses.

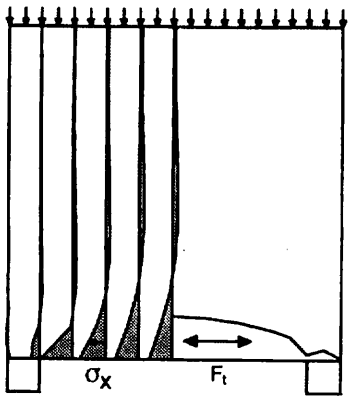


Fig. 7.3-4:
Distribution of chord forces in a deep beam from a linear elastic FE-analysis. Horizontal stresses σ_x in vertical sections and resultant tension chord force F_t [Rueckert (1992)]

Another problem is the anchorage of reinforcement, for which the (linear) FEM cannot give guidance, since the reinforcement is not modelled at all. It can be rather dangerous to derive anchorage forces and necessary reinforcements near supports or other discontinuities from linear FE-results. As an example, the chord forces in the span of a deep beam are in reality carried on by the reinforcement right into the node over the support, whilst the FEM tends to distribute them in a relatively wide tension field ahead of the node region (Fig. 7.3-4). It is therefore necessary, to extend the chord reinforcement determined for the span of a deep beam with its full cross-section into the support nodes and to anchor the reinforcement there for the full force, following the rules given in Chapters 4.4.4 and 4.5.

(3) Design of deep beams and discontinuity regions with strut-and-tie models

(3.1) The principles of the method

The stresses or inner forces of a structure can be plotted and visualised in the form of trajectories (Fig. 7.3-3). A similar, but not identical mapping of the "flow of forces" in a structure is possible, using a mechanical analogy between streamlines of fluids and force components in a plate (Fig. 7.3-5) [Fonseca (1995)]. The trajectory patterns or the streamlines of forces, flowing from the loaded edges through the structure into the supports, are indeed very helpful tools in order to correctly understand the load-bearing performance of a given structure, and they are an instrumental aid for the designer.

However, these patterns are, in general, fairly complicated and, at best, available only for linear-elastic material behaviour. To obtain a more practical model of the structure, the stress trajectories or streamlines of individual compression stress fields can be condensed and

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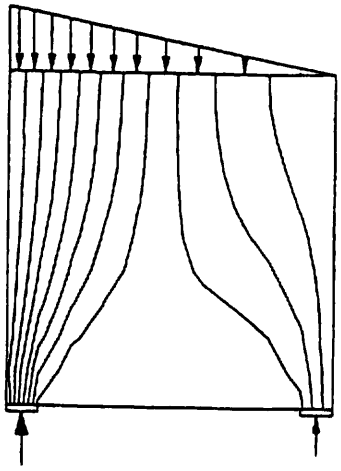


Fig. 7.3-5:
Load paths derived with the analogy of force flow and fluid flow

straightened to form the struts of a truss model. Accordingly, the forces in strips of parallel reinforcement bars can be idealised to form the ties of the truss model, which we prefer to call "strut-and-tie model" (STM), because the models sometimes consist of only very few members.

Truss or strut-and-tie models were successfully applied since the end of the 19th century for beams, later-on also for corbels and other special cases of reinforced structures. In particular, Swiss and Scandinavian engineers utilised parallel or fan-shaped stress fields to explain the real load bearing capacity of plane reinforced concrete structures by the theory of plasticity [Muttoni/Schwartz/Thuerlimann (1997)]. Stimulated by J. Schlaich, the strut-and tie method was generalised and improved to be applicable to all kinds of reinforced concrete members and to entire structures [Schlaich/Weischede (1982), Schlaich/Schaefer/Jennewein (1987), Schlaich/Schaefer (1991), Schlaich/Schaefer (1998)]. Many authors in Canada and USA used similar approaches to trace forces in a structure. The Strut-and-Tie Bibliography of the American Concrete Institute lists about 300 references together with abstracts [ACI (1997)]. Meanwhile such methods are included in many building codes worldwide.

After or parallel with developing a strut-and-tie model, the forces in the struts and ties are determined (Section (3)). Thereupon the struts, ties and nodes (or, to be more precise, the concrete stresses and the reinforcements) are checked or proportioned for the forces derived, with due consideration of reinforcement bends and anchorage of forces (Chapter. 7.3.2 and 4.4.4). Reinforcement for crack control or nominal reinforcement to assure a reasonable ductile behaviour of the structure complements the design.

The safety of structures designed with stress fields or strut-and-tie models is based on the lower bound theorem of the theory of plasticity. Normally, no analytical check of compatibility conditions is performed for strut-and-tie models.

In order to avoid gross incompatibilities and to reasonably adapt the strut-and tie model to the real behaviour of the structure, the struts and ties of the model should be placed and oriented at the elastic stress pattern (Section (3.2.2)). The elastic stress pattern for the orientation of the model can be derived from a simple linear FE-analysis with a relatively coarse mesh, since the structure's singularities will be treated separately, when dimensioning the nodes of the strut-and-tie model (Chapter.4.4.4).

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Before failure, the structure will adjust its internal forces to the model chosen for its design. However, this process is associated with plastic deformations which depend on the quality of modelling. Reinforcing bars can endure a great amount of plastic deformation without loss of strength, but the concrete's ductility is very limited. Also some types of mesh reinforcement exhibit relatively low ductility [CEB (1993)].

If the model is oriented reasonably at the elastic behaviour, the plastic deformations necessary for stress redistribution will be easily tolerated by the materials. In this case, the plastic deformations are of a similar magnitude as for a design with linear FEM. However, a design based on a poor model can lead to a premature failure of a part of the structure, before it has fully adapted its internal forces to the design model. Furthermore, models which are not oriented at the elastic behaviour are susceptible to wide cracks in the serviceability limit state. To avoid such mistakes, detailed rules for strut-and-tie modelling are given in Section (3.2) and applied thereafter for different types of structures in Chapters 7.3.2 - 7.3.5.

Since the design with stressfields is very similar to the strut-and-tie method, the following sections on STM apply in general also to the stress field method. Essential differences will be mentioned.

(3.2) Developing the strut-and-tie model

Developing a suitable model for a given D-region is very instructive but - like other engineering skills - requires some training and experience with the method. The following sections will provide guidance for the modelling process and will include also some warnings of frequent mistakes. Hastily drawn models rarely satisfy equilibrium, not to speak of compatibility. A systematic approach is therefore strongly recommended.

Three methods of modelling, which may also be combined, will be explained: The most instructive one is the loadpath method described in Section (3.2.3), which, however, is not feasible in all cases. It works particularly well, if the loads and support reactions are essentially vertical, as in deep beams. Always applicable and to some extent necessary is the method of developing the model from a linear elastic stress pattern (Sec. (3.2.4)). The fastest method is to adapt a known typical model to the special case, if the problems are related (Sec. (3.2.5)).

(3.2.1) Before modelling begins

As pointed out in Section (1.1), it is useful to identify the B- and D-regions of the structure and to choose division lines in-between them. If the structure contains B-regions, their sectional effects M, N, V are to be determined for a suitable statical system, e.g. a frame or continuous beam, in the usual way. Thereby the system lines continue straight through the D-regions, e.g. beam-column connections or regions with openings. The sectional effects are used to design the B-regions, following the standard code rules for linear members or slabs (Chapter 4.4.1).

The internal forces from the B-regions in the chosen borderlines are further needed as loading of the D-regions in those sections. This method allows to isolate a D-region from the structure and investigate it separately.

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In general, three forces may act in the section between a B- and D-region, as known from the truss model of the beam: Two chord forces, F_c and F_t, and a diagonal compression force F_θ.

$$F_c = M/z + 0.5\ V \cot \theta$$
$$F_t = M/z - 0.5\ V \cot \theta$$
$$F_\theta = V / \sin \theta.$$

The contribution of the shear force V to the chord forces is necessary for equilibrium. If it is omitted for the compression chord force applied to the D-region (as usually done for B-region design) then problems with the equilibrium of the model forces are to be expected. Another hint: Sometimes it is advantageous to represent the diagonal stressfield be two parallel forces with the same resultant.

After the forces from the B-regions and all the other forces acting directly at the D-region are defined (including support reactions), the equilibrium of the isolated D-region should be checked. Unknown or wrong support forces are the most frequent causes of problems with modelling. Distributed loads acting inside the D-region, like weight, may for simplification be replaced by line loads acting at the edges.

(3.2.2) General rules and advice for modelling

a) It is recommended to look for rather simple models with a small number of struts and ties. If necessary, the model can be improved or refined later-on.

b) Orienting the model at the elastic behaviour does not mean a perfect mapping of elastic behaviour, but rather adapting the main stream of forces to it. In particular the location and direction of struts with major forces should be oriented at the principal stress flow. Stress fields with low stress intensities may be represented more conveniently.

c) The model ties can be located to give a practical reinforcement layout with due regard to the ease of construction and an effective use of the reinforcement. Straight bars with few bends, arranged parallel and orthogonal to the edges of the member are to be preferred. The edges and surfaces of the structure should be fitted with near-surface reinforcement in order to control cracking. Several parallel bars in a tension stress field may be represented by a single model tie in their centre of gravity. The final reinforcement layout must conform with the model used for its design.

d) The angles θ between struts and ties in a node should be large, at least 45°, whenever possible (Fig. 7.3-6 a). Exceptions from this rule must be tolerated in the very frequent cases where a diagonal compression strut meets two ties which are orthogonal to each other (Fig. 7.3-6 b and c). In such cases a reduced compression strength applies, e.g. f_{cd2} for the node region (Chapter 4.4.4). Angles smaller than 30° are unrealistic and involve high incompatibility of strains.

e) Concentrated forces like point loads, support reactions or anchor forces, which are applied to the edge or corner of a member, tend to spread in the member and may be represented according to Fig. 7.3-7 a and b. For point loads located right at the corner or far from the corner the angle of deviation derived from theory of elasticity is:

$\delta = 32{,}5^0$, corresponding to a strut angle $\theta = 62{,}5^0$.

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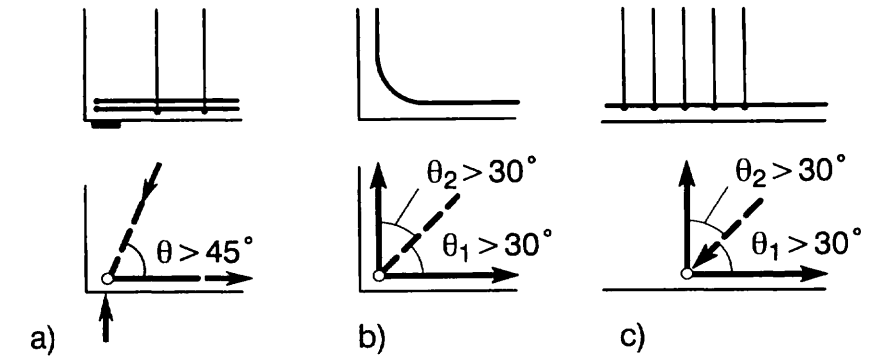


Fig. 7.3-6: Strut angles θ in compression-tension nodes and corresponding reinforcement (nominal reinforcement not shown).[1]
a) Tension in one direction. b) c) Tension in orthogonal directions

f) Depending on the shape and boundary conditions of the whole D-region the deviation angles δ of the stress resultants will increase or decrease with the distance from the origin of the concentrated force, and the model struts may be located conveniently. As an example, for the deep beam in Fig 7.3-7 c, a larger angle δ up to 45° may be chosen, corresponding to a strut angle θ > 45° at the support node. For larger span/depth ratios the model in Fig 7.3-7 d with an intermediate vertical tie provides a better resultant strut angle. Both models can also be combined as shown in Fig 7.3-7 e. This is a standard model for many applications, suitable e.g. for concentrated loads near the support of beams or for corbels and beam-column connections (Chapter 7.3.4). The statically indeterminate tie force F_w may be reasonably chosen, see (k). The given formula interpolates between $F_w = 0$ for $a < z/2$ and $F_w = F$ for $a > 2z$. The formula takes into account also the effect of a normal force N_{Sd}, (tension positive) which is not shown in the figure.

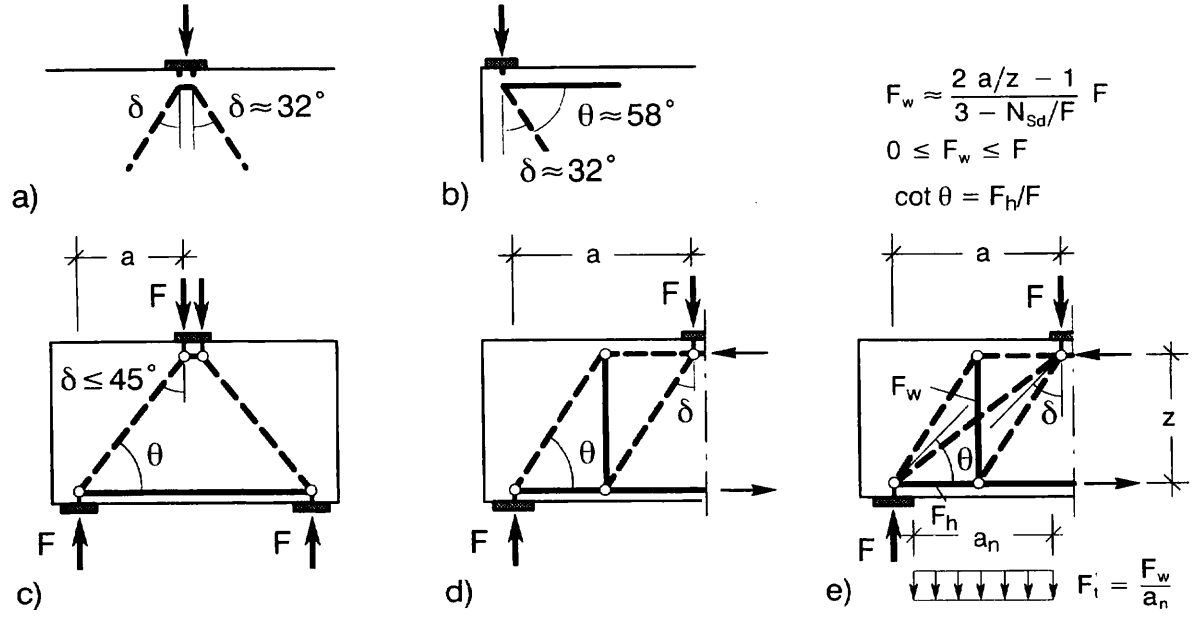


Fig 7.3-7: Change of direction of load paths.
a) Splitting of the concentrated force in order to represent the spreading of stresses.
b) Deviation of a concentrated force applied to a corner (compare Fig. 7.3-6 a).
c) Larger deviation angle δ taking into account the member's shape and its boundary conditions.
d) An intermediate model tie improves the angles between resultant struts and tie.
e) Combination of the two previous models

[1] In all figures with strut-and-tie models, the struts are drawn as thick dashed lines, the ties as thick solid lines

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g) Compression struts represent compression fields with a considerable width. Therefore an appropriate distance from edges must be kept. The same applies to model ties which represent distributed reinforcement. Furthermore, a node of the model (Chapter 4.4.4) may require a minimum distance of the model tie from the member's edges.

h) The strut-and-tie models are quite often kinematically underdeterminate (mechanisms). However, this does not mean that the structure lacks stability, because even the slightest movement of the quadrilateral truss panels will immediately evoke diagonal compressive forces in the concrete to stabilise the structure. The designer may add as many diagonal members to the model as required in order to make it statically determinate. These so-called "zero" members will not carry any significant forces and will not affect the flow of forces, provided that the kinematic model is adequate.

i) Kinematic models can be applied to one specific load case only. Therefore, the geometry of a kinematic model must be adapted to suit the specific load case. It is advisable to determine the forces in kinematic models already during the modelling process, because the equilibrium conditions for the nodes help to develop the geometry of the model after a few struts and ties have been chosen. The arrangement of reinforcement should ideally be designed to cover various load cases.

j) By contrast, a statically determinate strut-and-tie model will tolerate and cover a number of different load cases although it is self-evident that it cannot render an equally perfect representation of the various flows of forces.

k) For a better representation of the real stress flow it is sometimes appropriate to superimpose two simple models, whereby each model must in itself be able to establish equilibrium with part of the load applied (example in Chapter 7.3.5.). There is ample freedom for allocating the loads to the individual models, though some consideration must be given to stiffness conditions and crack width limitations. A stress distribution in a relevant section may provide guidance for the choice of statically indeterminate forces (Section (3.3.4)).

l) Complicated indeterminate models are better only, if they are optimised. Stiffnesses have to be allocated to the struts and ties for such an analysis and a special non-linear computer program is necessary [Rueckert (1992)].

(3.2.3) Modelling of D-Regions by means of the load path method

The load path method utilises the similarity between stress flow and fluid flow for modelling. The load path represents the line along which a force or force component[2] is carried through a structure from the point of loading to its support. In accordance with the law of continuity for fluids in between streamlines, but contrary to the forces in structures between stress trajectories, the force component associated with a load path remains constant on its way through the structure. The general procedure may best be explained by two examples. Further applications will be presented in Chapter 7.3.5.

The D-region shown in Fig. 7.3-8 a is loaded by asymmetric linear stresses q from the adjoining B-region. The stress diagram is subdivided in such a way that the associated resulting loads in the upper part of the structure find their equivalent counterpart on the

[2] More precisely: The component of a force in a chosen direction, e.g. the vertical component of a load.

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opposite side. The pattern that develops when drawing a line to link the opposite forces is the so-called load path (Fig. 7.3-8 b).

It is self-evident that the load paths will not cross each other. The load paths begin and end at the centre of gravity of the corresponding stress diagram, and have there the direction of the applied loads or reactions. They tend to take the shortest possible streamlined way in between.

Considering that concentrated forces tend to spread out as fast as possible in the structure, the resulting load paths will show a pattern which, originating from the supports, will propagate inward and show its maximum curvature near the supports.

Until now, equilibrium has been considered only in the direction of the applied loads. The curvature of the load paths, however, gives rise to deviation forces F_c and F_t which, for the sake of simplicity, will be plotted horizontally. Considering that the structure under review is not subject to horizontal loads, it follows that the deviation forces of the two load paths must balance each other.

Next, the horizontal deviation forces are replaced by a strut F_c and a tie F_t. Finally, the load paths are replaced by polygons, the break points of which intersect the resultants of the deviation forces (Fig. 7.3-8 c).

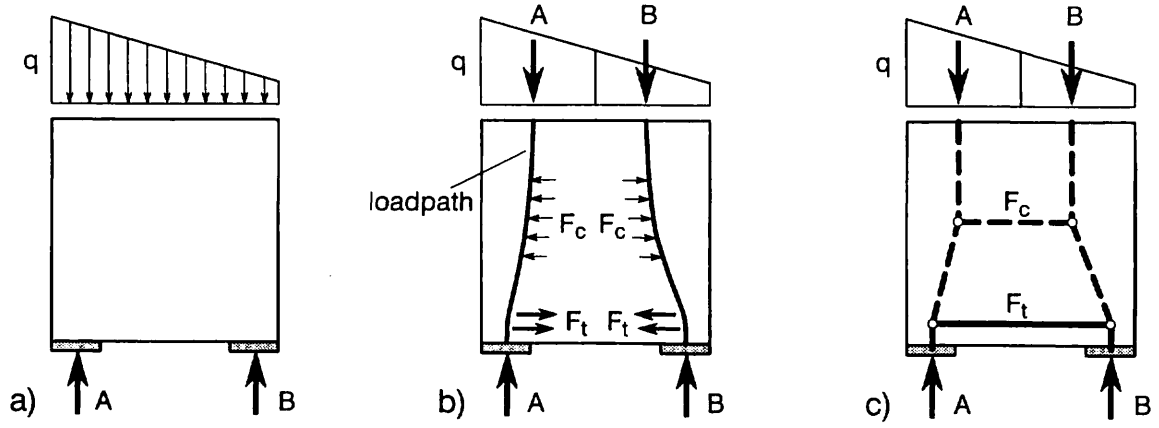


Fig. 7.3-8: Application of the load path method for modelling a typical D-region
a) Geometry and loading.
b) Load paths of the related loads; deviation forces F_c and F_t required for equilibrium.
c) Strut-and-tie model

The model so developed reflects the principal paths of forces and illustrates the load bearing behaviour of the structure. The tension and compression members are representative of curved two-dimensional or three-dimensional stress fields with the mainstream of stress orientated in the direction of their centrelines. The nodes of the members are, of course, no real hinges but rather entire regions where the internal forces (stresses) are diverted or introduced (anchored).

The second example shows a D-region which is supported on a B-region and loaded with a concentrated load F on one corner (Fig. 7.3-9 a). Proceeding as described above, the portion of stresses acting on the bottom part of the D-region is isolated in such a way that is in vertical equilibrium with the load F. Then a load path is drawn between F and the resultant of the isolated stresses (Fig. 7.3-9 b).

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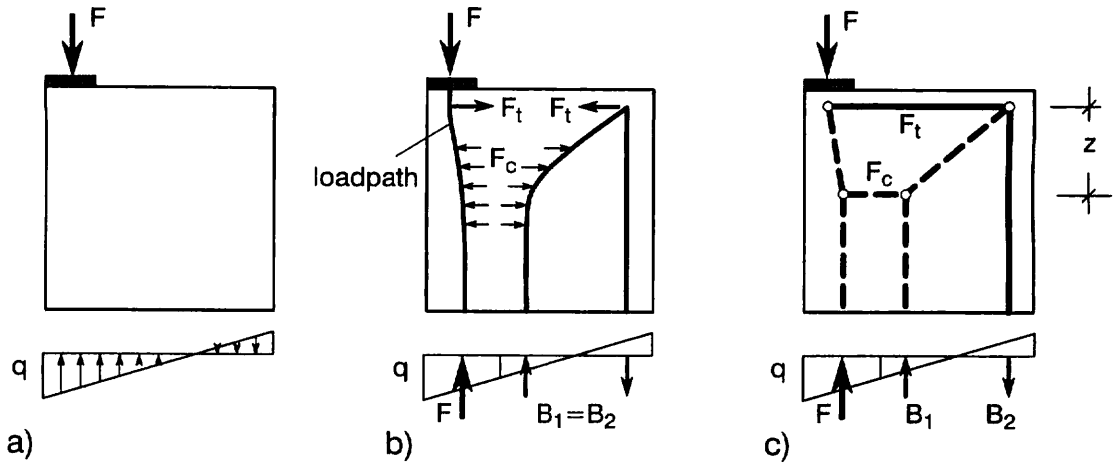


Fig. 7.3-9: Application of the load path method, including a U-turn
a) Geometry and loading.
b) Load paths and deviation forces.
c) Strut-and-tie model

However, there will remain two stress areas acting on the bottom part of the structure, with the resultants B1 and B2. These are equal in magnitude but of opposite directions. Their load path enters the D-region at B1, makes a U-turn and exits at B2. This U-turn of the load path and its associated deviation forces are necessary to equilibrate the deviation forces of the real load F as it travels through the D-region. By replacing the plotted load paths with polygons and linking them by means of the deviation forces, we again obtain a strut-and-tie model representing the flow of forces through the structure (Fig. 7.3-9 c).

The reader might notice, that the U-turn in the figure is shaped more like a V, with a distinct kink at its far end. Since the load paths are defined to carry longitudinal forces only in their direction (no shear or moment) they never can be directed at right angles to the vertical force, for which they are drawn here. The struts which will replace the load path in the model should not deviate more than 60^0, in accordance with Section (3.2.2d). It is further suggested to choose the tension tie of the U-turn straight, to avoid inclined and bent bars.

If also horizontal loads are applied to the D-region, the load paths for the horizontal components may be mapped accordingly and provide additional guidance for drawing the model.

(3.2.4) Developing the model from the elastic behaviour

In principle, a model can always be derived by combining and straightening bands of stress trajectories to form the struts and ties, but this method is suitable only for computer calculations [Rueckert (1992)]. Normally, a plot of principal stresses is sufficient to orient the major struts, and the model is then developed according to the principles given in the previous sections (Fig. 7.3-10).

Furthermore, stress distributions from individual sections allow to locate important compression struts like F_c in Figs. 7.3-8, 7.3-9 and 7.3-10 more precisely in the centre of gravity of the compressive stress field. Thereby a rough estimate of the centre of gravity is sufficient. Moreover, a modest increase of the internal lever arm z will render the model more realistic and more economic, as will be demonstrated by an example in Section (3.2.6).

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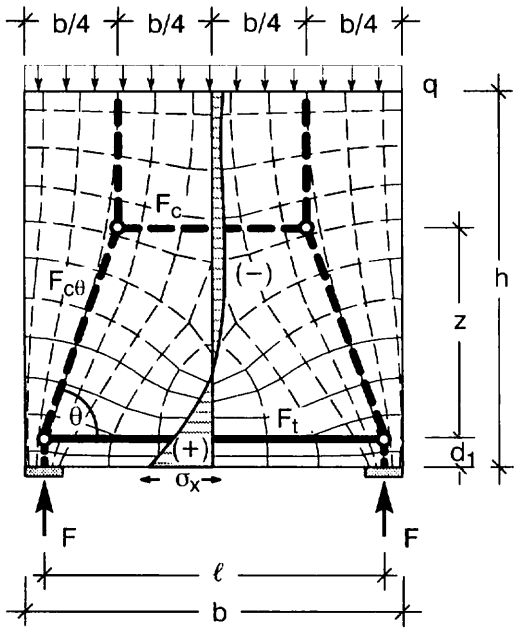


Fig. 7.3-10:
Orienting the strut-and-tie model of a deep beam at the theory of elasticity. Linear elastic stress trajectories, stress distribution in the middle section and related strut-and-tie model

(3.2.5) Typical Models

Anyone who has spent some time developing strut-and-tie models will observe that some typical models will appear over and over again in many different versions and combinations, even in apparently very different structures [Fig. 16 in Schlaich/Schaefer/Jennewein (1987)]. The reason for this phenomenon lies in the fact that there is only a very limited number of discontinuities with substantially different stress patterns. Furthermore, the corresponding models are related to each other and can be easily combined and adjusted to changing geometric proportions. This is of considerable informative and practical value for developing models, as will be shown with some examples.

Some typical models for concentrated loads applied to the corners of a square plate or deep beam were already developed above. If two regions D2 (Fig. 7.3-9) are put together bottom to bottom, the D-region in Fig. 7.3-11 c with forces in opposite directions is obtained. By stretching the length l of the region, the region D4 in Figs. 7.3-11 a and b appears.

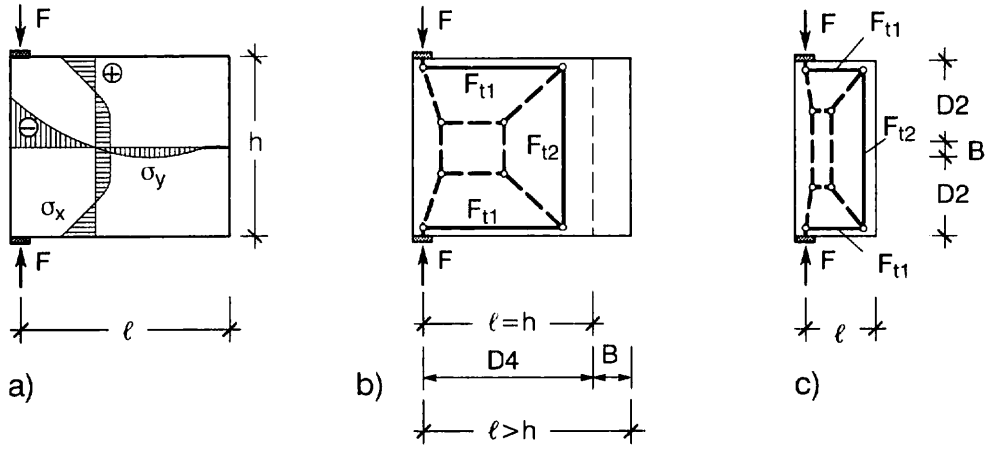


Fig. 7.3-11: Typical region D4
a) Stress diagrams (linear elastic).
b) c) Strut-and-tie models for different ratios d/l

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The same procedure, if applied to Fig. 7.3-10, gives the model for a rectangular plate (D8 in [Schlaich/Schaefer (1998)]) with point loads acting at all four corners. Try it!

For these two types of D-regions the stress field method would allow a straight (column-type) strut with constant width to carry the concentrated loads through the structure, with no need for horizontal ties. However, if the ties were really omitted, the structure may fail by shearing off a corner, as major disasters have taught.

Another typical stress situation is very frequently found under a concentrated load applied to a plate or column in its axis of symmetry (Fig. 7.3-12 a). The deviation of the two force components on either side of the axis creates transverse tension stresses, normally termed as splitting tension. A formula for the total tension force is easily derived from the simple model in Fig. 7.3-12 b, and compared with the elastic results in Fig 7.3-12 e. A similar formula given in the MC90, 3.3.2, with a factor 0.3 instead of 0.25 is (unnecessarily) conservative. Also a design based on the simple strut-and-tie model shown below will be safe if the reinforcement is arranged in accordance with the model (Fig 7.3-12 c).

The simple model for the D7-region in Fig. 7.3-13 is obvious. The refined model is drawn here only to show how the individual struts themselves could be modelled with STM in order to examine the transverse tension in the stressfield.

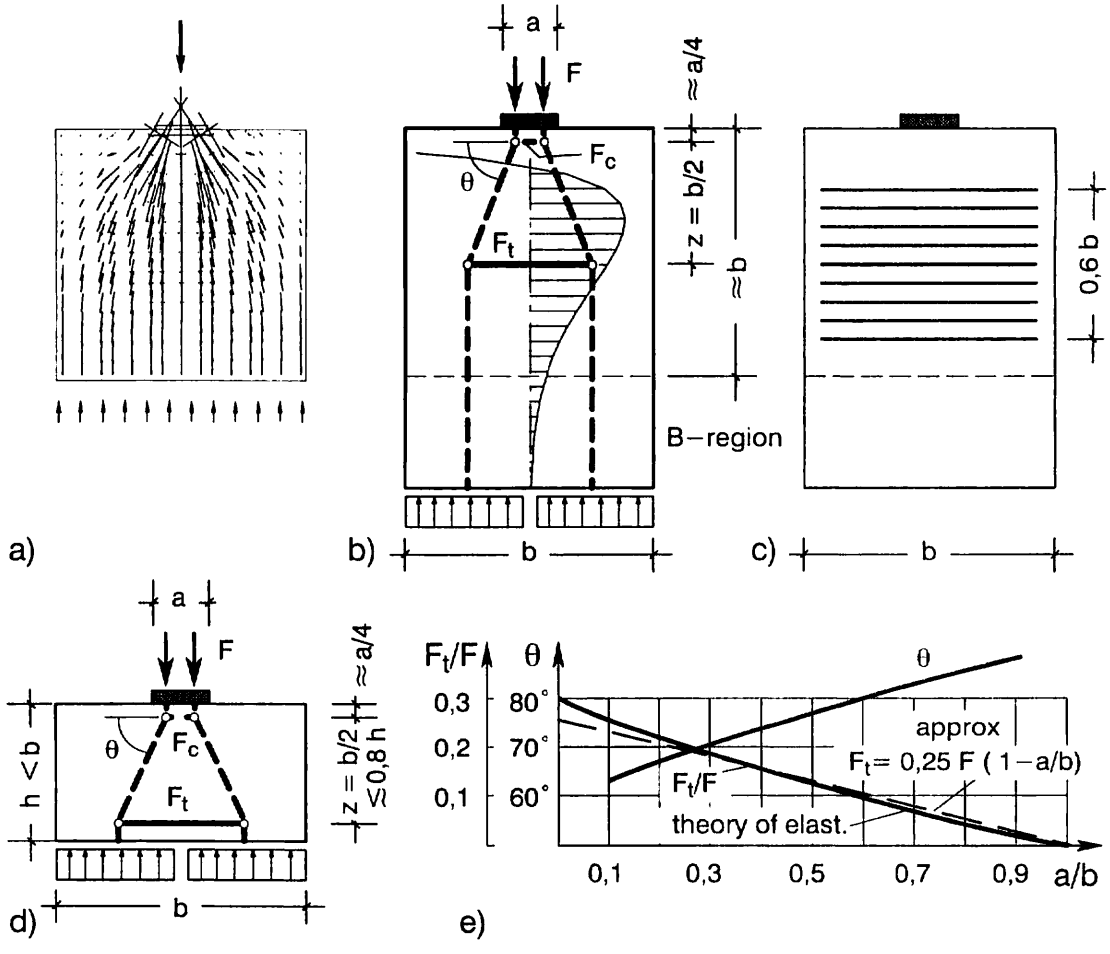


Fig. 7.3-12: Typical region D1. a) Pattern of principal stresses from a linear-elastic FE analysis. b) Strut-and-tie model with diagram of elastic stresses. c) Corresponding reinforcement for splitting tension (nominal reinforcement not shown). d) Strut-and-tie model for b>h. e) Internal force F_t and strut angle θ

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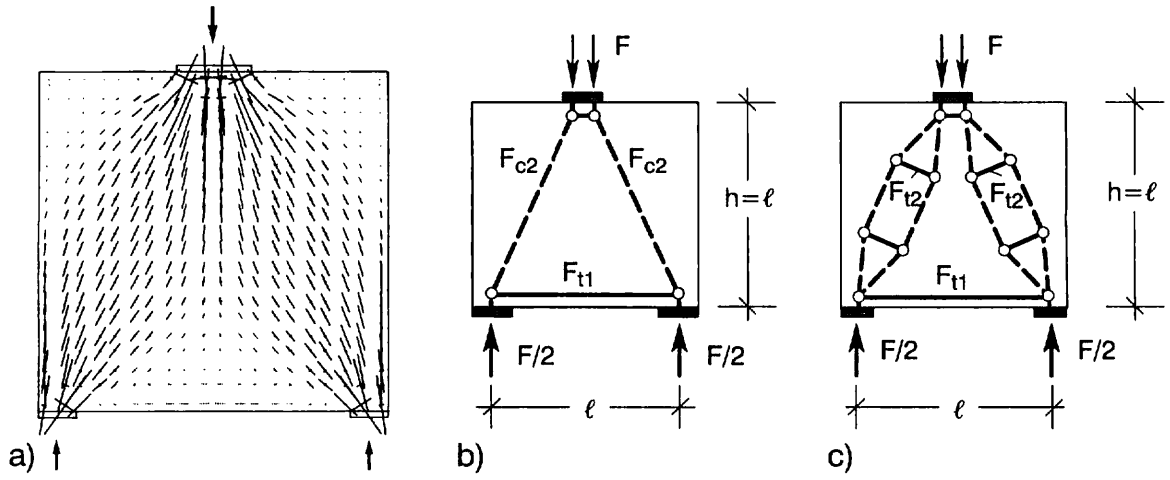


Fig. 7.3-13: Typical region D7. a) Principal stress pattern. b) Simple strut-and-tie model. c) Refined model showing the transverse tension in the struts

(3.2.6) Accuracy and Optimisation of the strut-and-tie model

The result of the modelling process is not a unique solution, a fact which is sometimes considered as a major drawback of the strut-and-tie method. The reasons for this non-uniqueness are found firstly in the inherent properties and characteristics of structural concrete itself, the loadbearing behaviour of which is to a large extent influenced by the chosen reinforcement layout. This opportunity should be positively exploited in order to adapt the structure to the specific requirements of a given case (e.g. absolute dimensions, fabrication, efficiency).

The second reason lies in the character of the method which is an approximate one. It is only reasonable to expect that a generalized procedure that would also be applicable to complex geometries cannot be as "accurate" as a method that has been tailored to suit one specific problem and is calibrated from ample experimental feedback, as is the design of beams and plates.

However, even a crude design method that will consistently cover the basic loadbearing behaviour will certainly be more useful and safer than a more or less arbitrary choice of reinforcement in the D-regions. Occasionally, the mere development of a strut-and-tie model will be enough to reveal the weak points in a structure and to substantially improve the detail design, by permitting for instance, to safely identify the need for reinforcement at a given point of the structure which otherwise would have remained hidden.

The strut-and-tie method is half graphical, with the advantage of being very obvious and intuitive. It requires much more involvement from the designer than computer analyses and it is very instructive. All this helps to avoid major mistakes and contributes essentially to the safety of the structure.

Anyhow, considering the complexity of structural concrete, the designer cannot produce the optimum solution for any given case but will in general have to be satisfied with a design that is just a sufficiently good one. This is a target an engineer with some experience in strut-and-tie modelling will easily reach without clear (numerical) criteria for the adequacy of a model.

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In special cases or if a computer program supports the modelling, the following criteria may be utilised for optimising a model.

The structure aims at carrying the loads as effectively as possible, with the least amount of internal forces and deformations. Since ties (reinforcement) deform much more than concrete compression fields, the model with the shortest ties and least tie forces is the most effective one. The product of tie length l_i and tie forces F_{ti} can be used as a simplified criterion:

$$\Sigma F_{ti} \cdot l_i = \text{minimum}$$

This criterion is also helpful in eliminating less desirable models (Fig. 7.3-14).

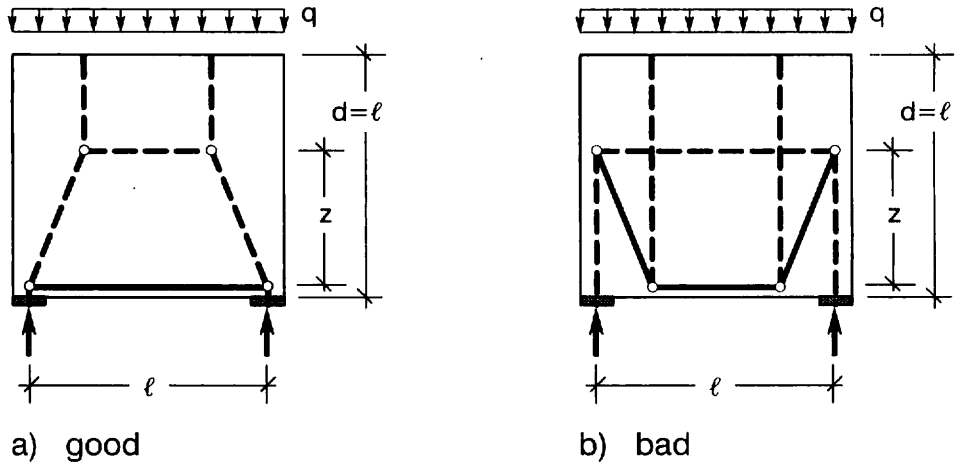


Fig. 7.3-14: Two different strut-and-tie models for the same structure. The good model (a) has shorter ties than the bad model (b)

For the exceptional case that the struts carry very high stresses over a substantial length (not only in the concentrated nodes) and are therefore exposed to high mean strains similar to the strain of the tie members, they should be introduced in the optimising criterion:

$$\Sigma F_i \cdot l_i \cdot \varepsilon_{mi} = \text{minimum}$$

where

F_i = force in the strut or tie member i
l_i = length of member i
ε_{mi} = mean strain of member i.

This approach would at the same time allow for the consideration of small strains of individual ties in rarely cracked or uncracked concrete. The above equation has some similarity with the principle of minimum strain energy for linear-elastic behaviour, but is modified for cracked reinforced concrete.

Strut-and-tie models allow a realistic assessment of the real ultimate capacity, if adapted to the crack pattern in the ULS. For example, the crack pattern of the test specimen in Fig. 7.3-15 a shows, that the compression chord has ultimately shifted to the upper end of the deep beam. Fig. 7.3-15 b shows a strut-and-tie model, which is adapted to the real behaviour and takes into account also the nominal mesh reinforcement. Concluding from this model, 94 % of the real ultimate capacity can be safely carried, while a prediction based on elastic behaviour

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can explain only 40% of the real capacity. Nevertheless, with respect to crack distribution and rotation capacity, it is recommended not to depart too far from elastic behaviour.

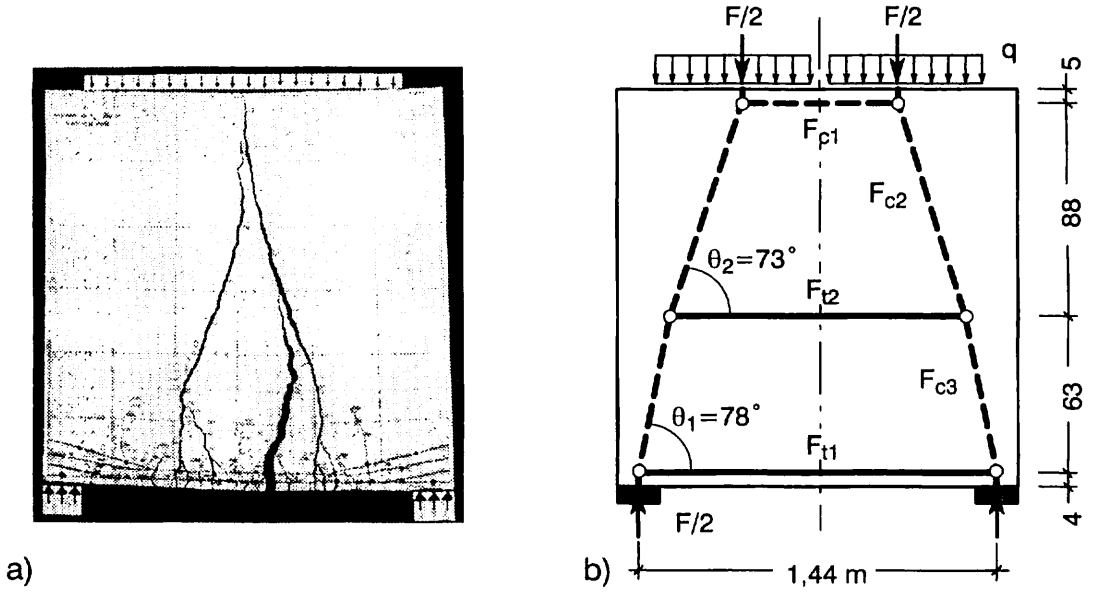


Fig. 7.3-15: Test of a deep beam.
a) Crack pattern before failure. b) Model explaining the ultimate capacity

(3.3) Checking the ties and struts

(3.3.1) Ties

As a general rule, tensile forces should be carried by reinforcing steel or prestressed steel. The necessary area of reinforcement for the ultimate limit state is derived from the design force F_{td} of the model tie and the design yield strength f_{yd} of the reinforcement. It is emphasised again, that the centroid and direction of the steel bars should coincide with the axis of the tie in the model.

For better crack distribution the reinforcement shall be distributed over the width of the tensile zone, which is represented by the model tie. If the model is reasonably oriented at elastic behaviour, crack limitations will in most cases be met by the calculated reinforcement and the minimum surface reinforcements prescribed for deep beams or other members in the code. When in doubt, a check of crack width can be performed for a critical strip of the structure, which is treated like an independent reinforced concrete tension member. For the serviceability limit state, sectional effects derived from a linear FEM analysis are normally more realistic and less sensitive to errors, as compared with STM-results.

The risk of wide cracks is particularly important when singular cracks starting from re-entrant corners or point loads intersect the reinforcement steel at acute angles. It is therefore recommended to either choose such a model where the reinforcement intersects the cracks at an angle as close to 90^0 as possible, or to add diagonal reinforcement, which is neglected in the model for simplification (Fig. 7.3-31). Otherwise, the critical reinforcement should be proportioned for lower stresses.

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(3.3.2) Concrete struts or compression stress fields

Typical for D-regions is the bottle-shaped stress field (Fig. 7.3-16 a) with its bottleneck proceeding into a concentrated node at a support or a re-entrant corner. This stressfield is associated with transverse tensile stresses, which can cause longitudinal cracks and initiate an early failure of the strut. It is therefore necessary to reinforce such stress fields in the transverse direction or to consider the transverse tension when determining the failure load of the strut. The transverse tension force can be determined from a strut-and-tie model of the stress field as shown in Fig. 7.3-12.

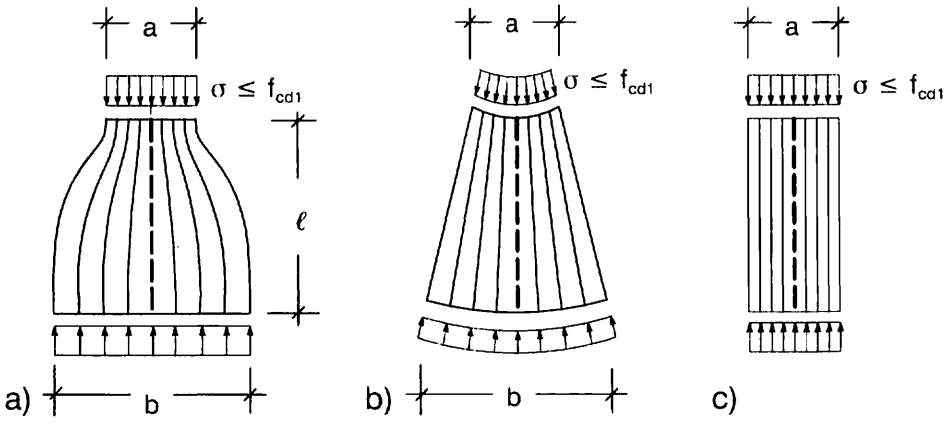


Fig. 7.3-16: The main compressive stress field configurations: a) the "bottle"; b) the "fan"; c) the "prism"

If the stressfield has bottle necks at both ends, as in Fig. 7.3-13, the transverse tension force occurs twice. However, such a stressfield can adapt its stress flow to a considerably reduced amount of transverse reinforcement by slimming its stress flow in between the two nodes. According to theory of plasticity, even the assumption of a parallel stress field in between the two nodes can be justified, in fact a very common simplification in applications of the stressfield method [Muttoni/Schwartz/Thuerlimann (1997)]. However, this is far from elastic behaviour and will provoke cracks, which may cause premature failure of the strut if not a minimal amount of transverse reinforcement is provided.

Fan-shaped stress fields (Fig. 7.3-16 b) allow a much better approximation of reality and are also frequently used for applications of theory of plasticity. A nominal transverse reinforcement is recommended also in this case, since stress trajectories tend to be curved in the real structure.

The prismatic stress field (Fig. 7.3-16 c) is typical for B-regions and shall be treated like columns or the bending compression zone of beams.

The strength f_{cd1} of the concrete in compression stress fields depends to a considerable extent on the multi-axial state of stress and on disturbances from cracks and reinforcement [Collins/Vecchio (1982); Baumann (1988)]. The MC90 allows in Clause 6.2.2.2 to determine the design resistance of a zone under essentially uniaxial compression to be calculated with a uniform stress diagram over the full area of this zone if appropriately selected. The average stress may be taken as

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$$f_{cd1} = 0.85\left[1-\frac{f_{ck}}{250}\right]f_{cd} \qquad \text{for uncracked zones}$$

$$f_{cd2} = 0.60\left[1-\frac{f_{ck}}{250}\right]f_{cd} \qquad \text{for cracked zones}$$

whereby the maximum extreme fibre strain is restricted to limits similar to those for the bending compression zone.

Based on many panel tests carried out by Collins and others, the Canadian Code [CSA (1994)] provides a more refined formula for the concrete strength which includes the transverse tensile strain as a main parameter. Implicating the code-formula for the transverse tensile strain in this formula will result in a strong dependence of the concrete resistance on the angle between struts and ties (compare the rules in Section (3.3.2)).

Except for prismatic stress-fields, the design strength of stress fields is, in fact, very rarely needed in practise. The reasons are firstly, that critical concrete stresses in D-regions occur in the regions of concentrated nodes. These are adjacent to the "bottle necks" of the stressfields and are checked with the node design (Chapter. 4.4.4). Furthermore, the parallel stressfields in B-regions are checked otherwise with standard methods for moment, shear and normal forces. Finally, these standard methods are in practise also frequently applicable to D-regions of beams, frames and other "linear" members.

7.3.2 Deep beams

(1) General

For many regular cases of deep beams, the typical strut-and tie models as illustrated in Chapter 7.3.1 (3.2) can be directly applied to determine a suitable layout and the necessary cross-sections of the reinforcement necessary for ultimate load conditions.

For proportioning the reinforcement, it is recommended to begin with the "additional reinforcement" according to Clause 9.5.1.1.2 of MC90, which is 0.2 % of the concrete cross-section in both orthogonal directions. These reinforcements, consisting of relatively thin bars, are arranged on both faces (0.1 % for each face), the horizontal reinforcement normally inside the vertical bars. Straight bars of these reinforcements should overlap near the edges with loop-shaped bars arranged there, or a stirrup type reinforcement should be chosen. The surface reinforcements can also be credited to ultimate load conditions and need to be supplemented only locally for major tie forces, in particular for chord forces along the edges. In areas with considerable tensile stresses, where the formation of cracks is to be expected, either from loads or restraints or both, the reinforcement should be designed for crack distribution according to MC90 Clause 7.4 and the rules in Chapters 7.3.1 (3.2.2) and (3.3.1) above.

At least two thicker corner bars should be placed along all edges of deep beams, for different reasons not accounted for in the analysis, such as folded plate action or restraints or differential settlement (see section (1.1))

Major chord reinforcement along edges should be placed in several layers, to increase the depth u of the node regions over supports and to ease the anchorage of reinforcement there

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(Chapter 4.4.4). The MC90, Clause 9.2.5.1.1, recommends a depth of 0.12 h or 0.12 ℓ (whichever is less), for the reinforcement in single span deep beams. For the same reasons, additional horizontal loops placed above the chord reinforcement may be necessary at end supports. The support regions of deep beams are normally the critical regions of the structure. They should be detailed and checked very carefully according to the rules outlined in Chapter 4.4.4.

Model ties at some distance from edges, as for example the chord reinforcement for negative moments in continuous deep beams, represent wide tensile stress fields and should therefore be distributed over a corresponding depth of e.g. 0.6 h, or 0.6 ℓ if ℓ < h. Stress-diagrams from a simple linear FE-analysis provide good guidance for the appropriate height and distribution of such reinforcements (see also Fig. 7.3-10, Fig. 7.3-12 and MC90 Clause 9.2.5.2).

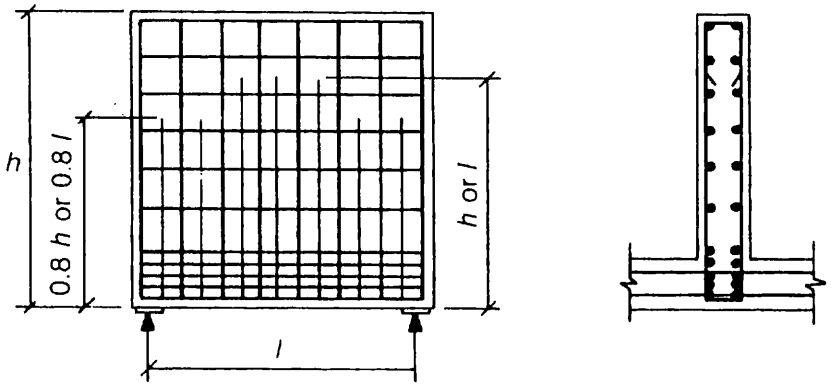


Fig. 7.3-17: Recommended reinforcement layout in a deep beam for loads from a lower floor (schematic) [MC90 Fig. 9.2.7]

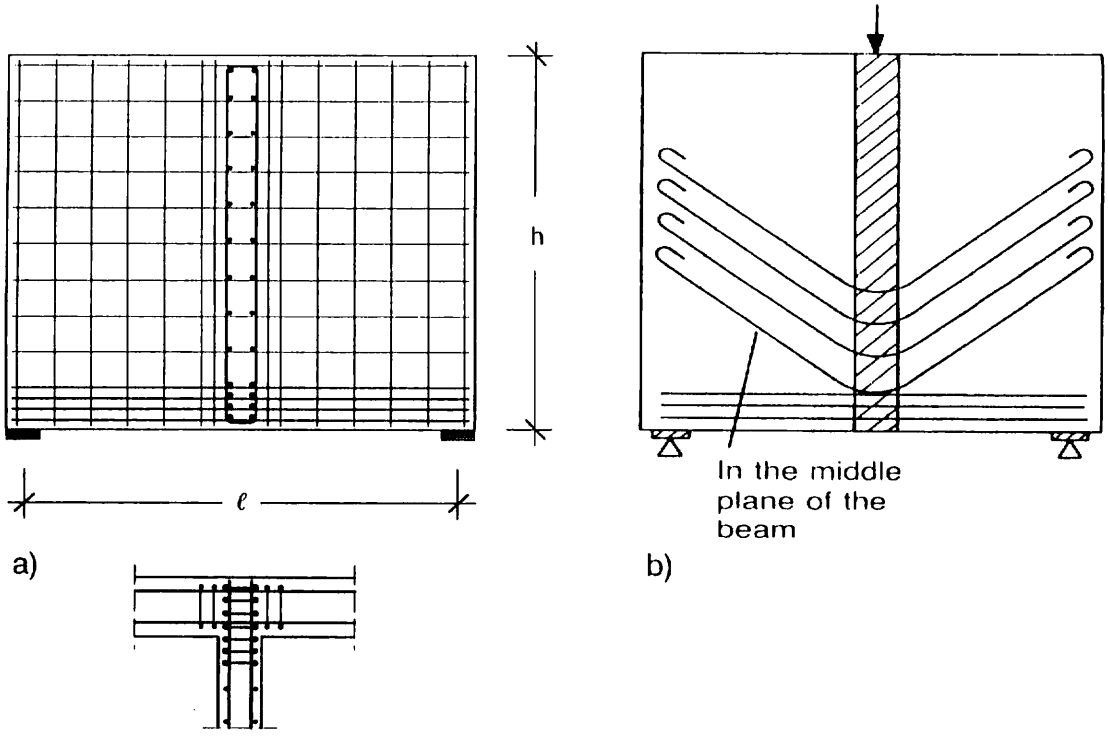


Fig. 7.3-18: Deep beams with hanging reinforcement for indirect support of another deep beam.
a) Vertical hanging reinforcement at the common edge
b) Diagonal hanging reinforcement for vertically distributed load [MC90 Fig. 9.2.8]

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Suspended loads, e.g. from a slab at the bottom of the deep beam, call for additional stirrup-type reinforcement in order to hang the load on the compression chord, comparable to the hangers of an arch bridge with suspended deck (Fig. 7.3-17). Concentrated suspended loads may be treated in the same way (Fig. 7.3-18 a, or, if the load is large, bent-up bars may be provided (Fig. 7.3-18 b). These help to avoid reinforcement congestion and reduce the tension chord force.

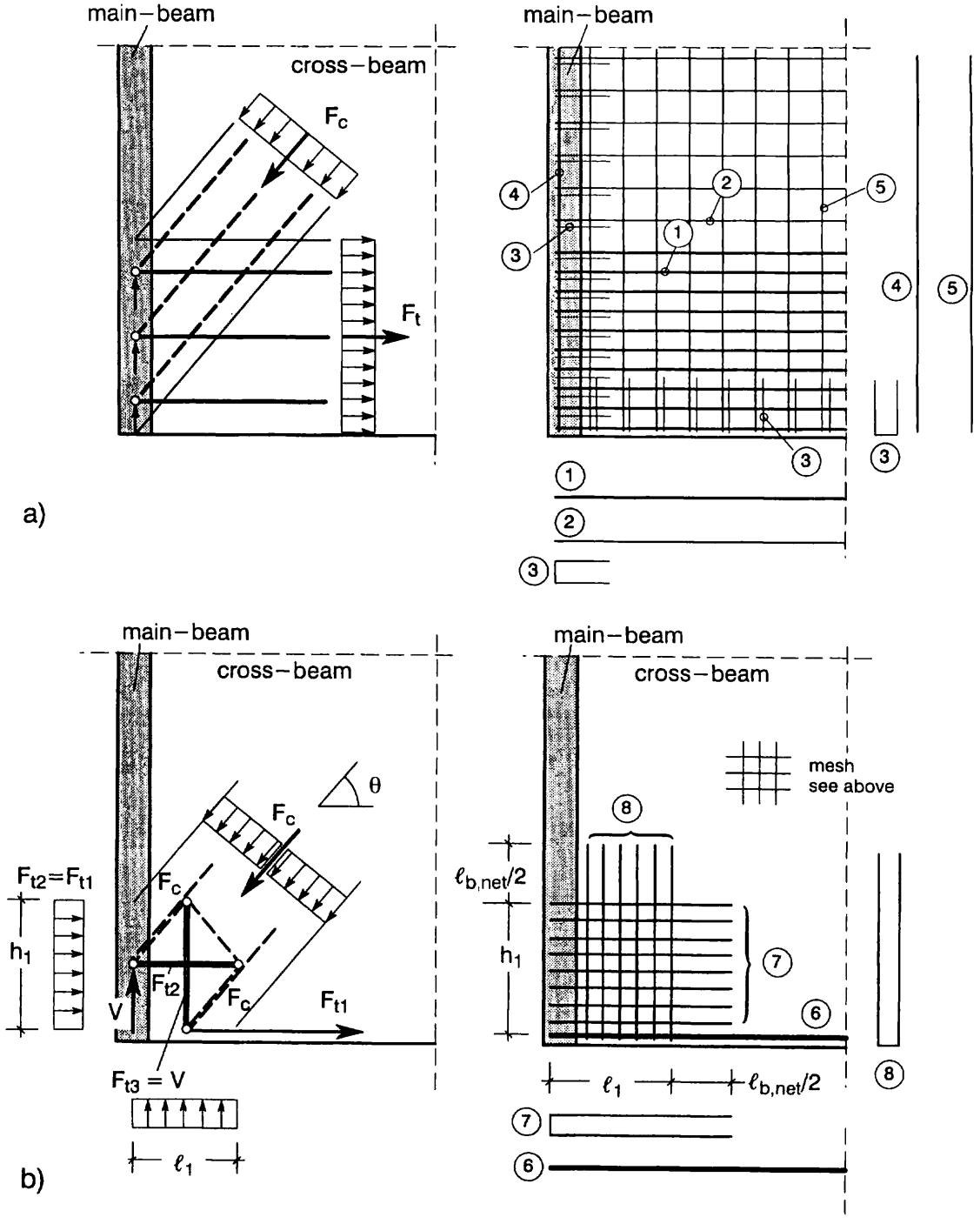


Fig. 7.3-19: Distributed transfer of shear forces between deep beams.
a) Model showing the horizontal reinforcement and the associated upward shift of the tension chord; b) Another model and corresponding reinforcement mesh for avoiding a concentrated node in the corner

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Concentrated suspended loads originate e.g. from cross-beams that are supported by main beams (indirect support). Three different possibilities of load transfer may be considered:

a) Normally, the cross-beam is assumed to transfer its load to the bottom corners, as if it were supported there by bearings. The support reaction from the cross-beam is then applied as a suspended load to the main beam. The vertical reinforcement for the suspended load should preferably be arranged within the common volume of the two intersecting beams, or very close to it (Fig. 7.3-18 a). Otherwise, considerable additional horizontal reinforcement may be necessary for equilibrium.

b) As an alternative, which avoids the concentrated node in the corner, the transfer of shear forces between the two beams can be assumed to be distributed over some depth of the beam. Since the vertical shear is actually transmitted by diagonal concrete compression stresses, horizontal equilibrium at any point of the edge requires horizontal (or diagonal) reinforcements (Fig. 7.3-19 a). These reinforcements need to be placed in accordance with the assumed distribution of shear forces over the depth of the beam, and they must be anchored within the common edge region with loops. They constitute part of the beam's tension chord which has shifted upward near the support as compared with the alternative (a).

   If Fig. 7.3-19 a is turned by 90^0, the similarity to the tension chord of a beam with diagonal compression and stirrups becomes obvious. This is a very typical situation for edges in all kinds of reinforced concrete structures.

c) Another alternative for better distribution of forces near the support is explained by Fig. 7.3-19 b. It has the advantage that the internal lever arm of the chord forces in the span is the same as for the standard case (a). Note, that the diagonal concrete stresses in the corner area amount to twice the strut force F_c divided by the (enlarged) cross-section of the strut! The tension forces F_{t2} and F_{t3}, for which additional reinforcements in orthogonal directions are to be provided, are equal to the main chord force F_{t1} and the support force V, respectively. Remember also, that the model node indicates the centre of the development length of reinforcement, and that reinforcements must extend beyond the node accordingly by at least half of the development length.

(1.1) Concrete stresses

Normally, concrete stresses need to be checked in the concentrated nodes only (according to Chapter 4.4.4), rarely anywhere else according to Chapter 7.3.1 (3.3.2). No shear design is necessary similar to that for beams.

Moments and support forces of continuous deep beams analysed for stiff supports may be completely offset by relatively low support settlements or deformation of the supporting columns or walls (example in the next section). The analytical results are therefore to be considered with much caution, and it is advisable to design brittle support members and support regions for two bounds of support force.

On the other hand, a deep beam structure has to undergo only very small deformations to adapt its inner forces and support reactions to those from an approximate analysis. A nominal surface reinforcement, as required by code rules, should provide the necessary ductility of the structure.

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There was much debate in recent years, whether class B steel is sufficient ductile for the application of plastic methods. The MC90, Clause 5.6.3, does not allow type B steel to be used in structures that are designed with statically admissible stress fields, which are the basis of the strut-and-tie method, too. However, the author's opinion on this matter is, that for deep beams also welded reinforcement meshes of class B steel can be used as nominal reinforcement, if the design model is reasonably oriented at the elastic behaviour. If there were any danger with respect to steel ductility in deep beams, class B steel ought to be excluded also for continuous deep beams that are designed with linear elasticity (see next section).

(2) Numerical example of a deep beam

(2.1) Structural analysis

The deep beam in Fig. 7.3-20 transfers distributed loads from the floors below and above to three supporting walls. The dead load of the beam is included in the load below, which is safe with respect to hanging reinforcement. Loads and sectional effects in the figure are already design values, multiplied by the relevant safety factors for actions.

The whole structure obviously consists of one D-region (Chapter 7.3.1 (1.1)) and will be designed using a strut-and-tie model. First of all, the support reactions have to be determined. They are given in Fig. 7.3-20 together with the corresponding moment diagram from a linear FEM-analysis for rigid supports (solid line), which is the basis of the following design. As expected, the negative moment of the deep beam is considerably less than the moment obtained from bending theory for slender beams (dotted line).

Much more surprising is a comparison with the moment diagram for a structure, the support of which is a 3 m high column instead of a wall (dashed line). If the elastic deformations of the supports are taken into account – normally they are not – the negative moments over the support disappear completely, and the maximum positive moment in the span increases by a factor of almost 2. These changes result from a differential settlement of less than 0.6 mm (!). The example shows, how sensitive continuous deep beams are with respect to their support conditions. Therefore, also structures designed with theory of elasticity need ductility in order to adapt their internal forces to simplified boundary conditions. Considering the large discrepancies of FEM-results arising from the assumption of support stiffness, the accuracy in determining the support forces and internal lever arms for the design of continuous deep beams becomes questionable and less relevant.

(2.2) Chord reinforcement

The required minimum reinforcement according to MC90 is $0.002 \cdot A_c = 600$ mm²/m. Reinforcement meshes ∅ 8 – 150 mm will be provided on both surfaces.

A strut-and-tie model is drawn in Fig. 7.3-20 b, based on the support reactions for rigid support. The model follows from the load path method (Chapter 7.3.1 (3.2.3)). It could also be composed of the two standard models in Fig. 7.3-10 for the two spans and Fig. 7.3.-12 for the interior support. For simplicity, the internal lever arm is chosen to be constant, about 40% of the average span l_m, an assumption which is somewhat conservative for the larger span and

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safe enough for the smaller span and the support chord (Fig. 7.3-15). Different lever arms could be considered in a refined model [Schlaich/Schaefer (1998)].

The horizontal strut and tie forces can be derived from the equilibrium of the model nodes or, if available, much faster from the moments (Fig. 7.3-12 a) divided by the lever arm.

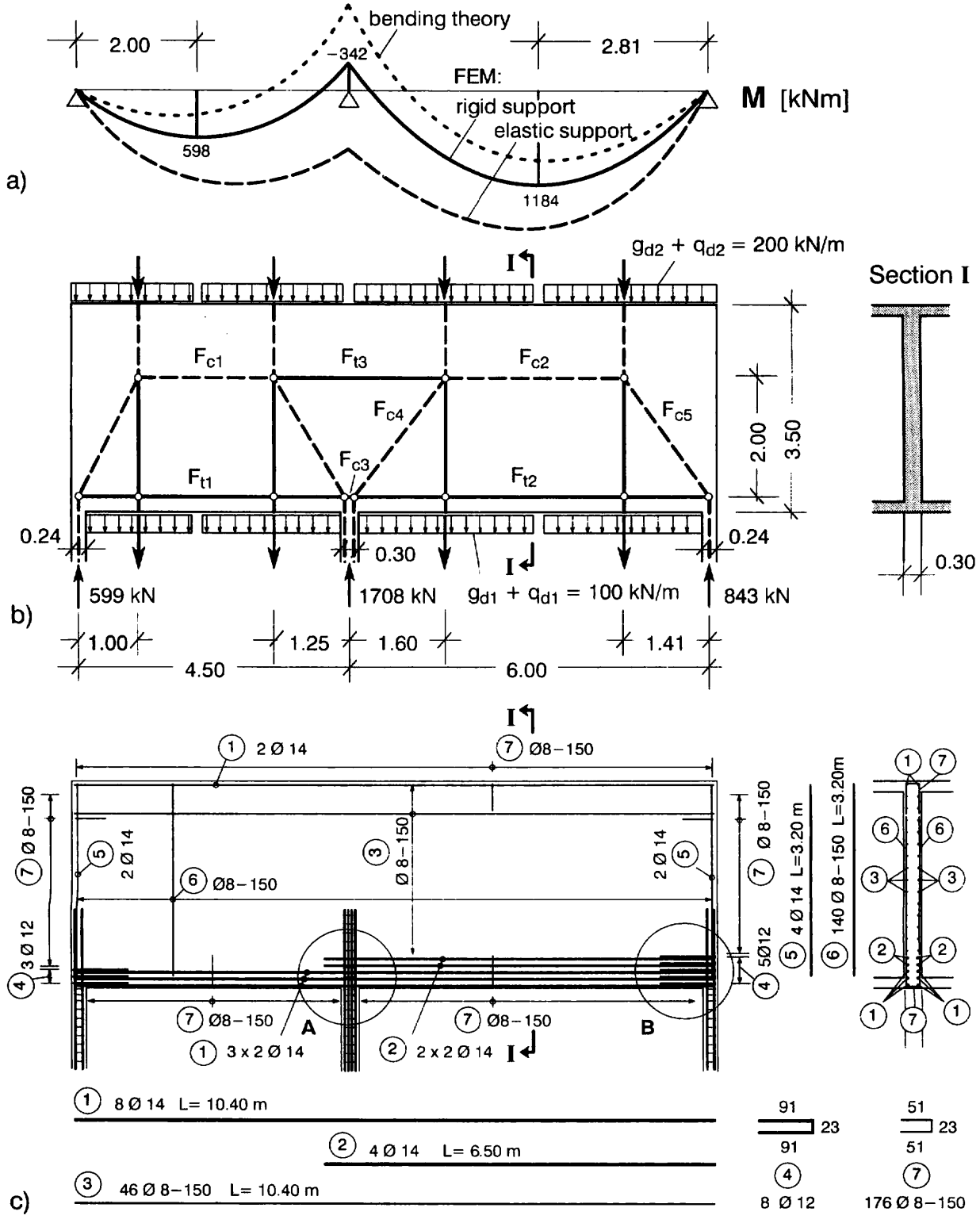


Fig. 7.3-20: Continuous deep beam. a) Sectional effects. b) strut-and-tie model. c) reinforcement

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Using f_{yd} = 500/1.15 = 435 MPa as design strength, the following chord forces and reinforcements are derived:

$F_{t1} = F_{c1}$	= 598/2.00 = 299 kN	$reqA_s$ = 687 mm²	→	6 ∅ 14 mm
$F_{t2} = F_{c2}$	= 1184/2.00 = 592 kN	$reqA_s$ = 1363 mm²	→	10 ∅ 14 mm
$F_{t3} = F_{c1}$	= 342/2.00 = 171 kN	$reqA_s$ = 393 mm²	→	2 ∅ 8 - 150 mm

Rules for placing the tensile reinforcement over the supports are given in MC90, Clause 9.2.5.2: The upper strip (Fig. 7.3-21) should be reinforced with a fraction of the required reinforcement that is interpolated between 0% for $l \leq h$ and 100% for $l \geq 3$ h, and some additional reinforcement should be placed near the upper face in cases where $l < h$. In the example, the required chord reinforcement for the negative moment is much less than the minimum mesh reinforcement within any reasonably chosen width for the reinforcement. Therefore, no extra reinforcement is necessary for F_{t3} (Fig. 7.3-20 b).

The nominal reinforcement is also sufficient to hang the lower slab into the compression chords of the span:

req. $a_s = (g_{d1} + q_{d1}) / f_{yd}$ = 100 000 / 435 = 230 mm²/m
< 670 mm²/m (2 ∅ 8 - 150 mm)

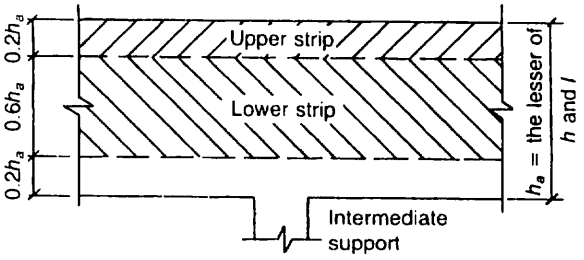


Fig. 7.3-21: Recommended distribution of reinforcement over an intermediate support [MC90 Fig. 9.2.3]

(2.3) Concrete stresses and node regions

Concrete stresses of the deep beam need to be checked only in the node regions with concentrated forces, i.e. the support nodes.

(2.3.1) End supports

Both end support regions are in principle standard compression-tension nodes according to Fig. 4.4-100. Only the end support of the large span will be considered here, as an example (Fig. 7.3-22). This node is particular insofar, as node stresses are quite high and compression reinforcement is necessary to resist the support reaction V_{Sd} = 843 kN.

The standard node region is determined by the width of the deep beam (b = 0.30 m), the length of the bearing (a_1 = 0.24 m), the depth of the node u, and the strut angle $\theta = 54.9^0$. This angle results from the chosen model or can be evaluated from tan θ = V/H, where H = F_{t2} = 592 kN denotes the tie force to be anchored in the node. The choice of u = 0.42 m corresponds with the recommended width of the tension chord of approximately 0.12 h (h < l) in MC90, Clause 9.2.5.1.1.

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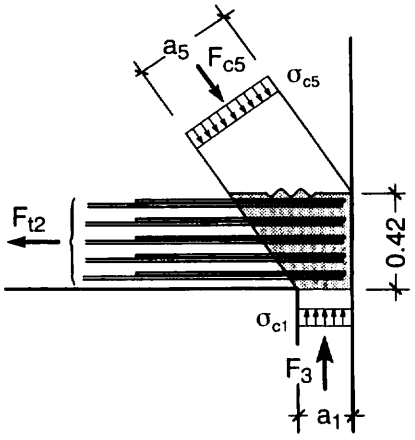


Fig. 7.3-22:
End-support-node regions of the deep beam in Fig. 7.3-20.

(a) Bearing stress

Considering the favourable strut angle and the well distributed chord reinforcement, the design strength f_{cd3} from Eq. (4.4-112) is applicable as concrete bearing stress σ_{c1} in the support area A_{c1}. The vertical steel bars, 4 ∅16, contribute with their design strength f_{yd} to the total bearing capacity V_{Rd}:

$$V_{Rd} = a_1 \, b \, f_{cd3} + A_s \, f_{yd}$$
$$= 0.24 \cdot 0.30 \cdot 8.6 + 804 \cdot 10^{-6} \cdot 435 = 0.62 + 0.35 = 0.97 \text{ MN} > V_{Sd} = 843 \text{ kN}$$

where

$f_{cd3} = 0.7 \, (1\text{-}f_{ck}/250) \, f_{cd} = 8.6$ MPa
$f_{cd} = f_{ck}/\gamma_c = 20/1.5 = 13.3$ MPa
$f_{yd} = f_{yk}/\gamma_s = 500/1.15 = 435$ Mpa

(b) Diagonal stress

Next, the average stresses from the diagonal strut force $F_{c5} = V_{Sd}/\sin\theta = 1030$ kN in the node face a_5 will be checked with Eq. 4.4-116. The strut width a_5 can be determined from the chosen geometry of the node region:

$$a_5 = a_1 \sin\theta + u \cos\theta = 0.24 \sin 54.9^0 + 0.42 \cos 54.9^0 = 0.44 \text{ m}$$

$$\sigma_{c2} = F_{c5} / a_5 \cdot b = 1.03 / 0.44 \cdot 0.30 = 7.8 \text{ MPa} < f_{cd3} = 8.6 \text{ Mpa}$$

(c) Anchorage of reinforcement

In the structure considered, the chord reinforcement cannot be extended beyond the short support length of 240 mm, in contrary, the development length over the support is reduced by 35 mm for the necessary concrete cover. Furthermore, 30 mm allowance will be considered for the length and inaccurate placement of the longitudinal bars. Therefore, only a fraction of the tension chord force $F_{t2} = 592$ kN can be anchored by the ∅ 14 mm bars, and 5 additional horizontal loops ∅12 mm will be supplemented in the node region.

The bond resistance $F_{bd,\,\varnothing 14}$ of the 10 ∅ 14 mm bars will be calculated first from the surface area of the bars and the design bond strength f_{bd}, following the rules in Chapter 3.2.4:

$\alpha_1 = 1$ straight bars
$\alpha_2 = 1$ no welded cross bars

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$\alpha_3 \cdot \alpha_4 \cdot \alpha_5 = 0.7$ minimum value, taking into account transverse pressure from support
$f_{bd} = 2.25$ MPa
$n = 10$ number of bars
ef $l_b = 0.24 - 0.065 = 0.175$ m anchorage length
$\varnothing = 14$ mm

$$F_{bd,\phi 14} = n \pi \varnothing \text{ ef } l_b f_{bd} / \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 = 10 \pi \cdot 0.014 \cdot 0.175 \cdot 2.25 / 0.7 = 0.247 \text{ MN}$$

Accordingly, the loop bars ($\alpha_1 = 0.7$) will contribute

$$F_{bd,\varnothing 12} = n \pi \varnothing \text{ ef } l_b f_{bd} / \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 = 10 \pi \cdot 0.012 \cdot 0.205 \cdot 2.25 / 0.7 \cdot 0.7 = 0.353 \text{ MN}$$

$$F_{bd} = 0.600 \text{ MN}$$

$$> 0.593 \text{ MN}$$

(d) Further consequences of compression reinforcement

The vertical bars from the supporting wall or column must be extended into the deep beam and anchored there with $l_{b,net} \cong 0.8$ m, which is more than $u = 0.42$ m, as assumed so far. If vertical forces are really transmitted over this length, the effective depth u of the node and the width of the tension chord should be adapted accordingly.

In reality, the transmission of forces from the compression reinforcement into the concrete will begin already below the support face a_1, if lateral confinement increases the resistance of the concrete, and it will predominantly occur in the lower part of the transmission length in the deep beam. As a consequence, the concrete stresses in the lower part of the node may increase whilst the steel stresses in the compression reinforcement decrease. It is wise to consider this for the choice of node depth and the check of nodes with compression reinforcement. The assumption of $u = 0.42$ m $< l_{b,net}$ and the evaluation of the diagonal stress based on that value were conservative in this respect. The same applies to the arrangement of horizontal loops within u for the anchorage of the full tension chord. Beyond u the nominal horizontal loop reinforcement Ø 8 – 150 mm is sufficient in the example considered. In other cases stronger horizontal loops may be necessary within the development length, as explained by Fig. 7.3-19 a.

(2.3.2) Inner support node

The node over the inner support of the deep beam is rather complicated (Fig. 7.3-23). It may be considered as a combination of two end-support nodes, one for each span, and a superimposed compression node. The diagonal struts are shared by both types of nodes. Accordingly, in the model of Fig. 7.3-20 the horizontal component of the diagonal strut F_{c4} is balanced by the tie force F_{t2}, which belongs to the end-support node, and by the compression force F_{c3}, which is part of a compression node (compare Fig. 4.4-98).

However, it is obvious, that the two tension chords will not anchor their forces completely within half the column width. Instead, a considerable part of the tension chord forces $F_{t1} = 299$ kN and $F_{t2} = 592$ kN is directly balanced through the node region by the continuous chord reinforcement. To preserve equilibrium, the horizontal compression in the node's concrete must increase accordingly. The real horizontal concrete compression force in the node therefore depends on the bond forces with parameters like length of the support, bar diameter and bond properties.

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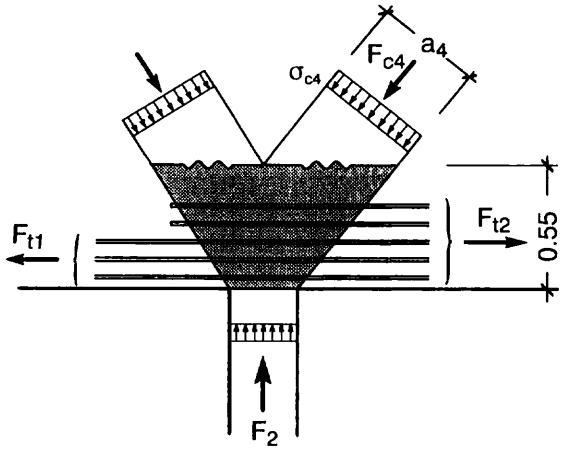


Fig. 7.3-23:
Interior support-node regions of the deep beam in Fig. 7.3-20.

In conclusion it can be stated, that the concrete in the node is under biaxial compression, but the horizontal compression is difficult to assess. On the other hand, tensile reinforcement penetrates the node region and is anchored there to some extent. Therefore, f_{cd3} will again be applied here as design node strength, though f_{cd1} according to Eq. 4.4-100 might eventually be considered.

(a) Bearing pressure

The concrete resistance is

$V_{Rcd} = a_1 \, b \, f_{cd3} = 0.30 \cdot 0.30 \cdot 8.6 = 0{,}77$ MN.

The remaining part of the support force, 0.94 MN, must be transmitted by compression reinforcement:

req. $A_s = 940\,000 / 435 = 2160$ mm^2 $\rightarrow$ $2 \cdot 4 = 8 \varnothing 20$ mm

With consideration of the large share of compression reinforcement and transverse tensile stresses due to the anchorage of thick bars, stirrups like those in the column are arranged within the development length.

(b) Diagonal pressure

The check of diagonal pressures at the node surface is analogous to that for the end-support node in Sec. (2.1.1). It will be briefly presented for the strut of the large span:

Strut force $F_{cd4} = 1.22$ MN
Strut angle $\theta = 51.3^0$

Horizontal support length for the diagonal strut of the large span: 0.17 m
Assumed depth of the node region from a pre-calculation: $u = 0.55$ m
Width of the diagonal strut at the node: $a_4 = 0.17 \sin\theta + 0.55 \cos\theta = 0.48$ m
Diagonal stress: $\sigma_{c4} = 1.22 / (0.30 \cdot 0.48) = 8.47$ MPa $< 8.6 = f_{cd3}$

Over the interior support it is not necessary to distribute the chord reinforcement over the total depth u of the node. Those parts of the diagonal strut forces, that belong to the

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compression node, equilibrate their horizontal components (e.g. F_{c3}) by direct compression, which works also above the reinforcement layers.

(c) Horizontal node pressure

An upper bound of the horizontal concrete compression can be obtained from the very conservative assumption that the horizontal component $H = F_{c4} \cos \theta$ of the diagonal strut is fully balanced by concrete compressive stresses. Considering the node geometry, the horizontal pressure is less than the diagonal pressure, and therefore needs no numerical check.

(2.4) Concluding remark

The example shows that deep beams, even those with a relatively low reinforcement percentage, may have problems with their support nodes; these are the real critical regions of the structure. Another lesson from the example is, that diagonal strut stresses may be more relevant for the design than bearing pressures and horizontal stresses.

7.3.3 Beam-column connections

(1) General behaviour

Beam-column connections are normally checked with standard methods for B-regions (Chapter 4.4.1 (5.2)) applied to the orthogonal sections through the inner corner. The balance of inner forces in between these sections must be taken care of by an appropriate arrangement of reinforcements.

The deflection of the chord forces of the horizontal member into the vertical direction of the columns is shown in the map of principal stresses for pure moment loading and elastic material in Fig.7.2-24 a. The deflection of the chord stresses generates radial stresses σ_2 in the common region of column and beam, here termed as corner region (Fig. 7.3-24 b and c):

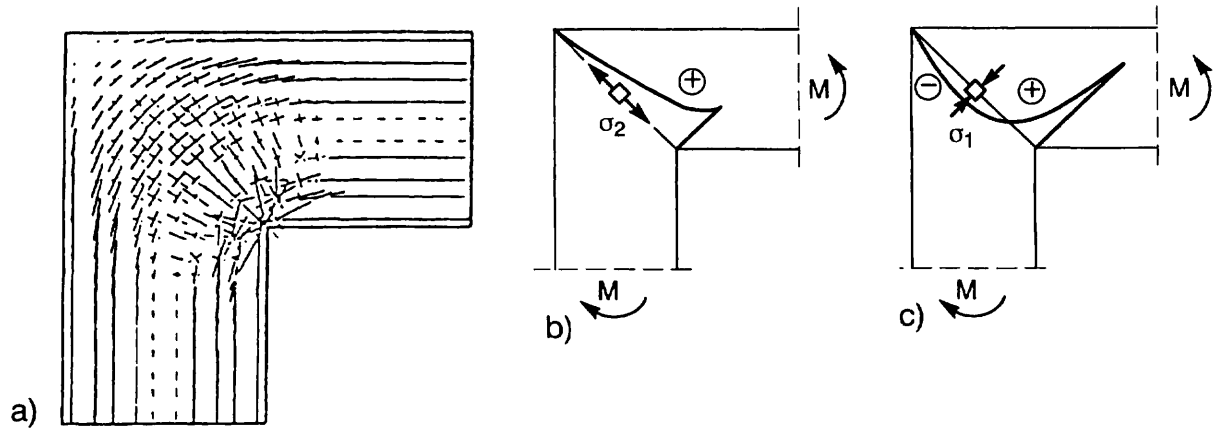


Fig. 7.3-24: Stresses in frame corners from a linear elastic FE-analysis:
a) Principal stresses.
b) and c) Distribution of principal stresses in the diagonal section

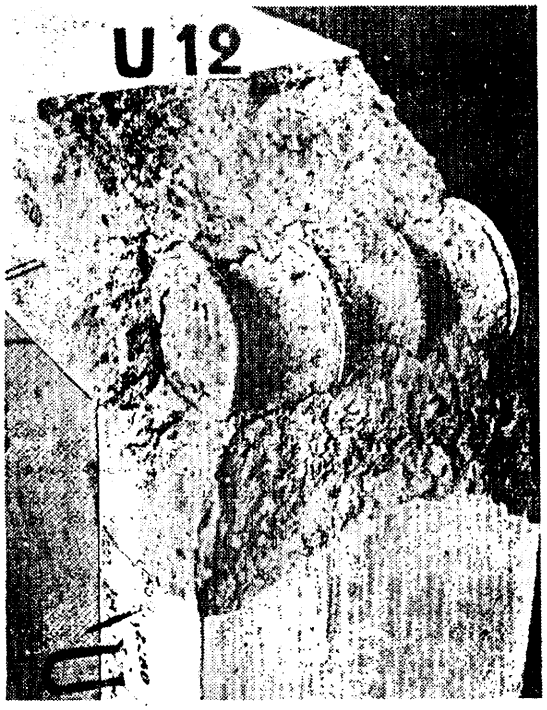
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The radial stresses are either compressive for negative (closing) moments, or tensile stresses for positive (opening) moments. These tensile stresses are the main cause for a completely different behaviour and accordingly different models for the two cases of moment direction:

Firstly, the concrete in the corner region tends to separate in the radial direction and must be tied back to the inner corner by stirrups. If stirrups are arranged in the radial direction of the principal stresses, the congestion of stirrups at the inner corner will complicate the construction. The concrete cover of the curved compression chord cannot be tied back. In beam-column-connections subject to high stresses, the cover will spall before failure (Fig 7.3-25). In such cases it is advisable to neglect the cover, when evaluating the moment resistance in the design sections mentioned above. Note that for the present purposes, concrete cover means all concrete areas which are not tied back to the interior of the corner region, i.e. the regular concrete cover of stirrups plus stirrup diameter plus approx. 1/3 of the distance between the loops or legs of radial ties.

Secondly, the radial tensile stresses weaken the concrete compressive strength of the compression chord. A close look at the map of principal stresses shows, that the deviation of the stress trajectories of the outer chords begins already before the intersection of beam and column. Therefore, the design sections are affected, too. Besides, the lever arm of internal forces decreases.

For these reasons, beam-column-connections with opening moments carry less moment than undisturbed B-regions with equivalent cross-section and reinforcement. In practical design, this can be taken into account by an effectiveness-factor < 1 for the resisting moment, or by a limitation of the reinforcement percentage. A rough reference for the admissible degree of utilization is the reinforcement percentage $\rho \leq 0{,}8$ % (for $f_y = 500$ N/mm^2, concrete C 20/25) in the sections of the frame corner. Sensibly interpreting MC90, the reduced concrete strength f_{cd2} (Eq. 4.4-6) should be applied for opening moments, whilst f_{cd1} (Eq. 4.4-4) is applicable for closing moments.



Another typical feature of the load-bearing behaviour are the peak stresses at the re-entrant corner (Fig. 7.3-24 c) where the trajectories experience a sharp bend. Assuming ideal elastic material behaviour, this would result in infinitely large stresses, but the high local strains will be tolerated by the plastic behaviour of the concrete in beam-column-connections with closing moments, and by the formation of cracks in those with opening moments. A considerable improvement is achieved by rounding the inner corner or by providing a haunch. This increases also the lever arm of internal forces.

Fig. 7.3-25:
Spalling of the compression zone under opening moment [Nilsson (1973)]

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(2) Beam-column-connections with negative (closing) moment

(2.1) Elementary models and associated reinforcement for pure moment loading

The simple model as depicted on Fig. 7.3-26 a is applicable only if the internal lever arms of the girder and the column, z_g and z_c, do not differ much in magnitude, and if the entire chord reinforcement of the horizontal and vertical member is extended around the corner.

If the lever arms of girder and column are very different, a truss-type model is appropriate for the corner region, in order to maintain reasonable angles between struts and ties (Fig. 7.3-26 b). The model shows, why additional horizontal reinforcement is necessary in the web of the beam. Omission of it has resulted in severe diagonal cracks extending from the outer column-face towards the inner corner. The horizontal reinforcement (loops or stirrups) is necessary for the anchorage of that part of the column chord force, which exceeds the girder chord force, due to the different lever arms. Assuming strut angles $\theta = 45^0$, the horizontal forces F_{t3} are in total equal to the difference of both chord forces, $F_{t2} - F_{t1}$. For other strut angles θ, the following formula applies:

$$F_{t3} = (F_{t2} - F_{t1}) \tan \theta$$

The model provides also information on the necessary anchorage length for the column bars and the large diagonal compression stresses in the corner region.

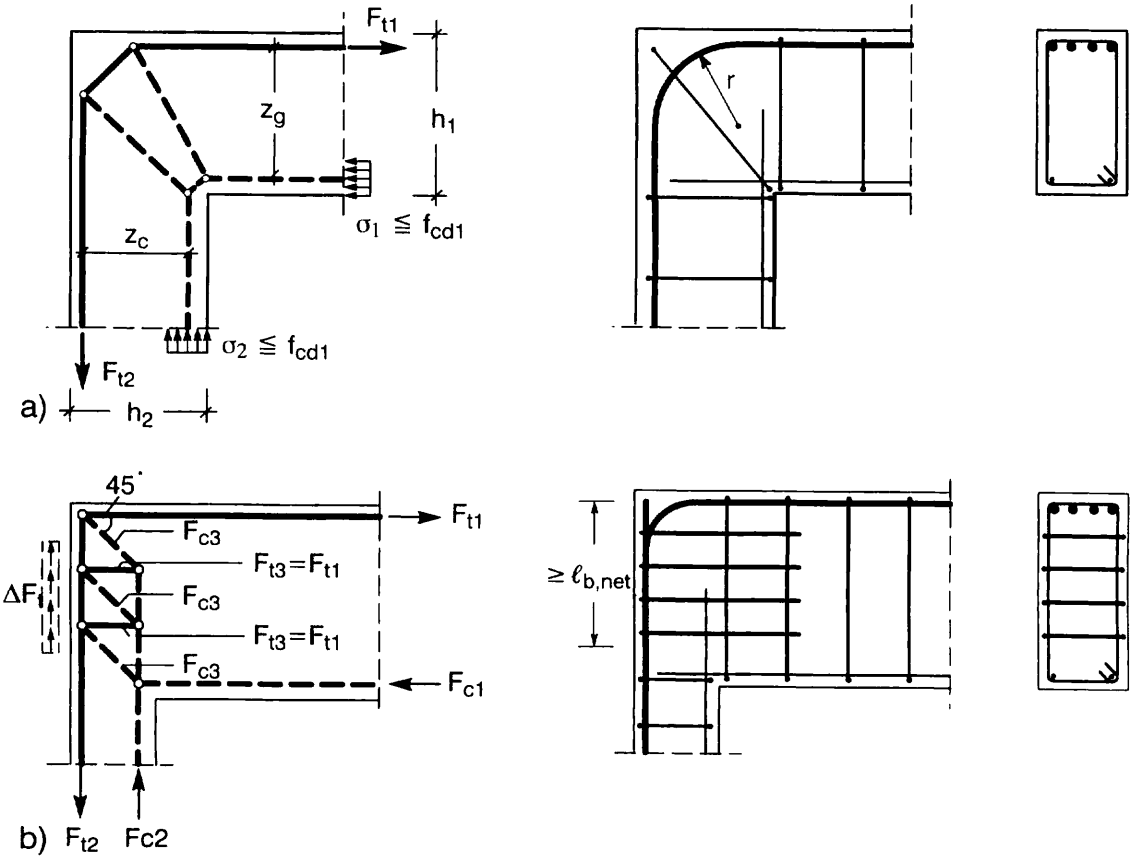


Fig. 7.3-26: Beam-column-connection with closing moment. Model and reinforcement.
a) Almost equal depth of beam and column.
b). Very different depth of beam and column

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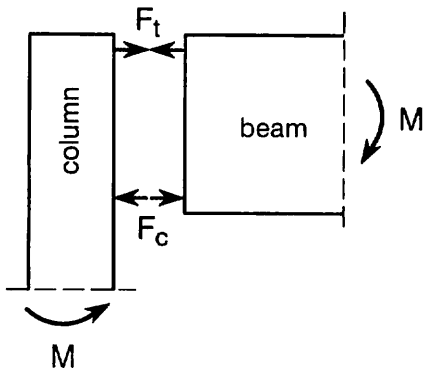


Fig. 7.3-27: Shear in the corner region of a beam-column-connection

Alternatively, the horizontal reinforcement and diagonal concrete stresses can be derived from a standard shear design of the cantilevering part of the column, according to Fig. 7.3-27. The beam's tension chord force F_{tl} acts on the column-end like a concentrated load and is transferred as a shear force through the cantilevering part of the column to the supporting compression chord of the beam. Since chord forces in beams are normally much larger than shear forces, the shear force in the cantilevering column is larger than in any other part of the beam and column! It is amazing that frame structures are usually carefully designed for the shear forces in B-region, whilst the more critical shear within the corner region is frequently overlooked, maybe, because it does not appear in the structural analysis of the frame system.

The concentrated compression node in the re-entrant corner needs no extra check, if the standard methods for B-regions were applied to the respective corner sections of the beam and column.

(2.2) Design Details

Splicing of chord bars in the corner region should be avoided or made with sockets. For moderate reinforcement percentages ($\rho < 0{,}4$ %, $f_y = 500$ MPa, C20), overlapping loops as illustrated in Fig. 7.3-28 may be used, provided that cross bars be laid out within the loops as a protection against splitting. This type of reinforcement is ideally suited for construction joints between walls and floors, and it is suitable also for opening moments (Sec. (3)).

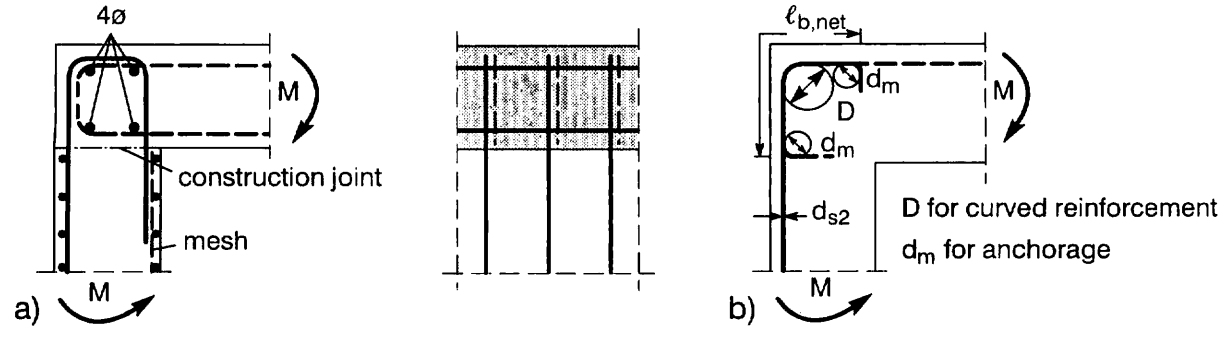


Fig. 7.3-28: Example of a wall-slab connection with loop reinforcement and hooked bars, respectively.

Of equal importance as the arrangement of the reinforcement in the frame plane is the distribution of the reinforcement in the cross section. The radial pressure from the curved chord bars generates tensile stresses transverse to the plane of the frame (Chapter 4.4.4 (3.2.3)). Normally, two loops or two-legged stirrups are insufficient to reasonably support the entire width of the curved compression chord. In connections with thick reinforcing bars, closely spaced stirrups or other transverse reinforcement it may be necessary to avoid splitting or spalling of the concrete (Fig. 7.3-29). In particular, the outermost chord bars are to be secured horizontally.

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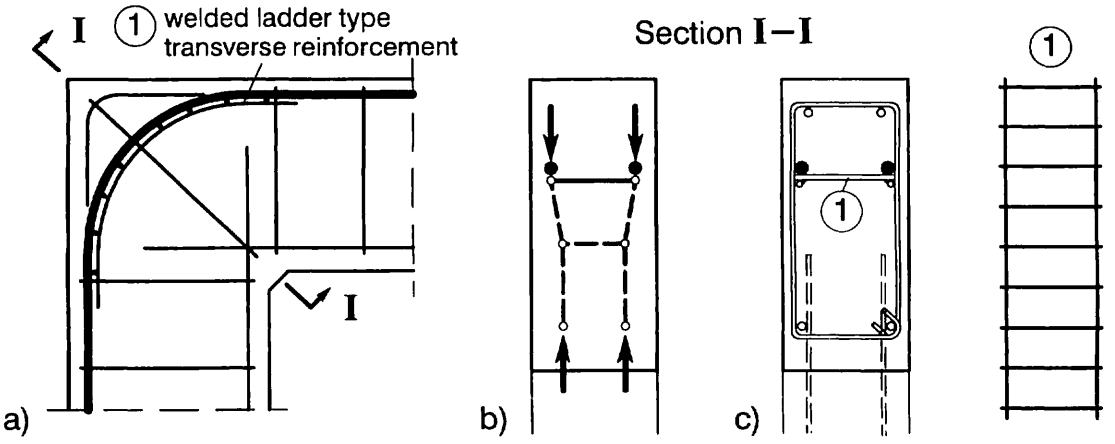


Fig. 7.3-29: Transverse reinforcement of a beam-column-connection with closing moment.
a) View. b) Diagonal section with model. c) Reinforcement

Considering the concrete stresses, the bending radius of the chord reinforcement should be chosen large, but on the other hand it should be small enough, to keep the internal lever arm in the diagonal above that of the orthogonal sections. It is recommended to choose the radius between 0,6 min d and 0,8 min d, where min d denotes the depth of either the girder or the column, whichever is less. The corner region outside the bent chord bars may be secured by a diagonal stirrup or a few thinner chord bars, bent with minimum radius (Fig. 7.3-26 a and 28).

(3) Beam-column-connections with positive (opening) moment

The model for negative moments (Fig. 7.3-25 a) is not suited (after sign reversal of the forces) for frame corners with positive moments, because reinforcement never shall be bent around a re-entrant corner. If the tension chord bars are continued straight beyond the corner, the problem remains, how to transfer the large diagonal tensile forces into the chord reinforcements near the inner corner. If any, a very congested reinforcement layout with radial stirrups that enclose the chord reinforcements would have to be used.

The simple model in Fig. 7.3-30 avoids the diagonal ties, but is not suited either, since the tensile chord reinforcements of the beam and column would have to be anchored within the depth of the compression chord near the outer face. In order to effectively divert the compression chord, the reinforcement should enclose it. This is possible to some extent by means of anchor plates or studs or thin loop-shaped bars. A check of the compression tension nodes 1 and 2 according to Chapter 4.4.4, Sec. (3.2.2) or (3.2.4), will reveal the difficulties.

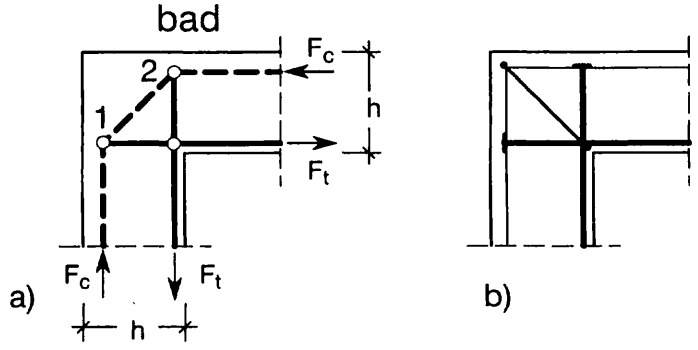


Fig. 7.3-30: Beam column connection with poor performance, feasible only for small positive moments
a) Simple model. b) Corresponding reinforcement

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For a much better behaviour, either the chord reinforcement must be extended as a loop around the corner region, or inclined stirrups must be adequately arranged (Figs. 7.3-31 b, c). The corresponding strut-and-tie model (Fig. 7.3-31 a) clearly reflects the path of the internal forces and indicates where the forces from reinforcements are anchored. A combination of both types of reinforcement is better than either of them alone.

Note, that the compression chord is located within the radial or loop reinforcement, which means that it has a smaller lever arm, particularly in those cases were loops with large concrete cover are used.

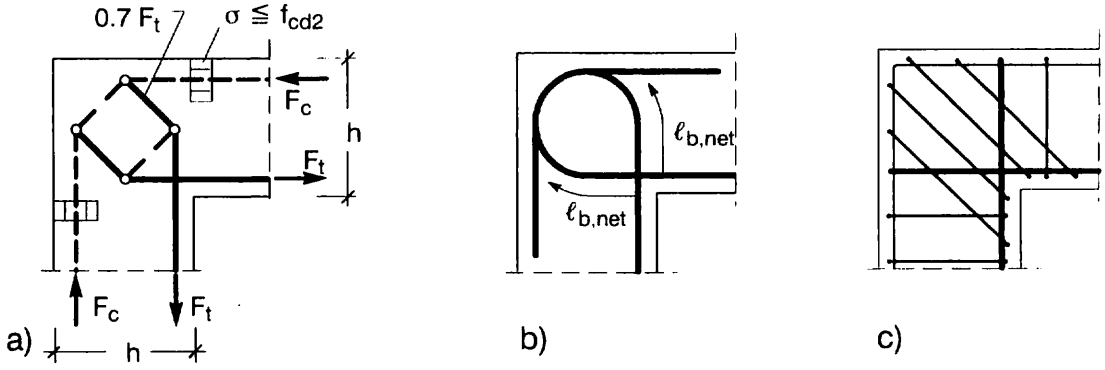


Fig. 7.3-31: Beam column connection for moderate positive moment.
a) Model. b) c) Two alternative reinforcement layouts

As can be seen from the strut-and-tie model in Fig. 7.3-32, the tensile force of the diagonal bars is extended via the stirrups to the outer compression chord. Similar to the elastic flow of forces (Fig. 7.3-23 a), the stirrups gradually deviate the compression chord also outside the corner section. The regions adjacent to the beam-column-joints shall therefore be adequately reinforced with stirrups, apart from the design for shear forces.

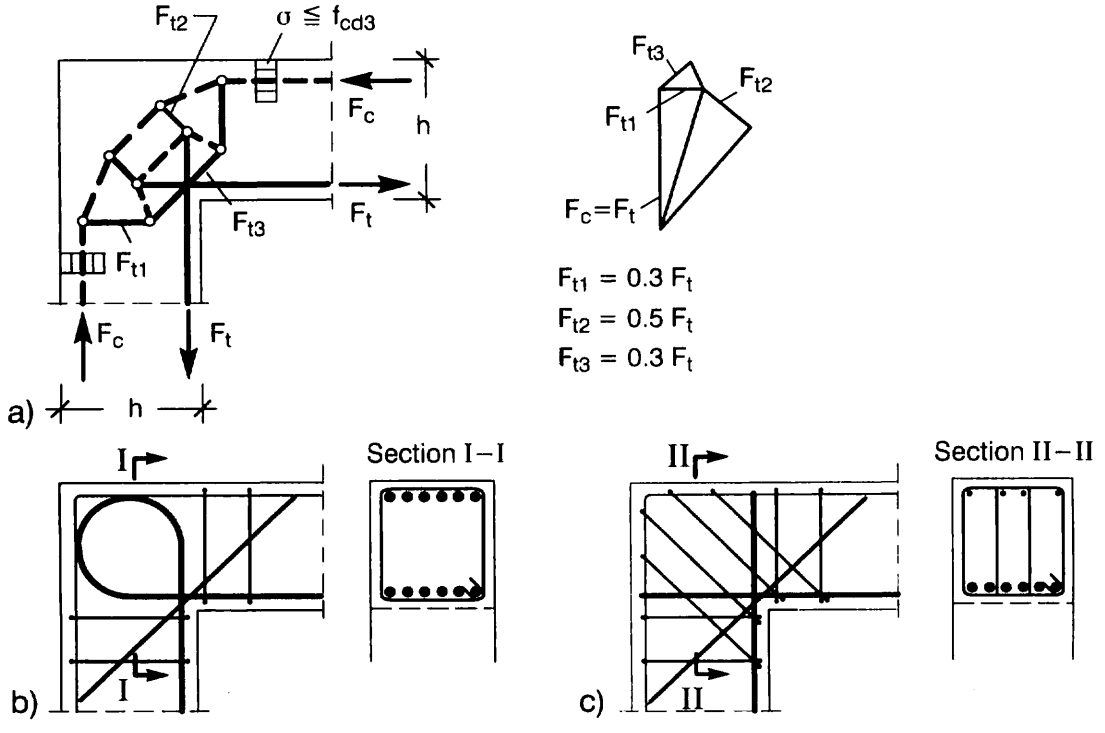


Fig. 7.3-32: Beam-column-connection for large opening moment.
a) Model with force polygon. b) c) Two alternative reinforcement layouts

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A major drawback of the reinforcement layouts in Fig. 7.3-31 b and c is the deviation (of about 45^0) of the reinforcement directions in the re-entrant corner from the direction of principal stresses σ_1 (Fig. 7.3-23 c). In members with relatively large moments, this may result in severe diagonal cracking which will also constrict the compression zone and entails a major reduction in efficiency of the section. Beam-column-joints with inclined bars at the inner corner behave much better, as has been demonstrated in tests (Fig. 7.3-32). The diagonal bars do not reduce the forces in the chord bars, they should be provided in addition to the reinforcement for the sectional effects. As a benefit for good detailing, the moment efficiency factor may be increased, or the intermediate effective strength of the concrete f_{cd3} from Eq. 4.4-112 utilised. Inclined bars are advisable for all kinds of re-entrant corners, also those in corbels, stepped beams, half-joints and at openings. They reduce the diagonal crack widths which tend to be particular wide in the corners, due to the discontinuity

(4) Rigid connection of column with continuous beam

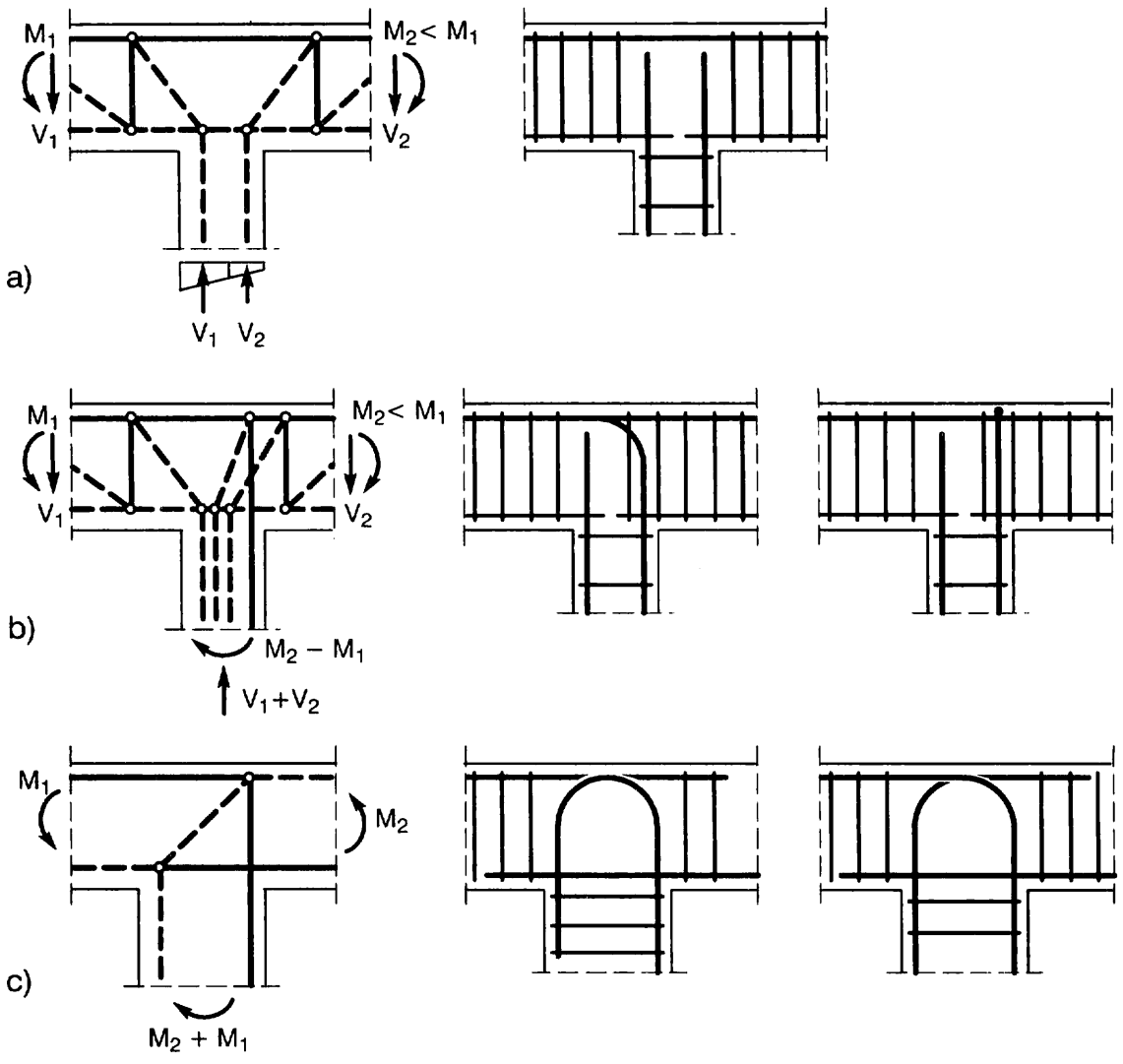


Fig. 7.3-33: Beam-column-joints. Models and corresponding reinforcement layout.
a) For small column moment, no tensile reinforcement in the column.
b) For column with tensile reinforcement.
c) Typical case of a sway frame joint

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For developing the reinforcement layout of joints of three or four members in a frame, it is recommended to split the sectional effects in two or three parts:

a) A moment that is transmitted in the continuous member from one span to the other, like the support moment in a beam over a hinged support. The reinforcement for this moment is arranged accordingly straight and continuous over the joint.

b) A closing moment that is transmitted from a column to one of the beams. The reinforcement for this moment shall be bent around the corner region as in the equivalent beam-column-joint, always "to the other side", e. g. reinforcement from the right hand column side into the beam on the left, and vice versa.

c) Only if necessary, an opening moment transmitted from a column to the other beam must be considered, too. Reinforcement for this moment is to be placed as for the opening beam-column-joint.

The Fig. 7.3-33 presents models for different cases together with the equivalent reinforcement layout. The model nodes indicate, how and where the anchorage of bars occurs. It is in general not correct, to assume the beginning of the development length at the interface of the members!

(5) Members with a kink and joints of profiled members

For joints of members with obtuse or acute angles, frequently occurring in stairs, the same principles shall apply and the same type of reinforcement is considered suitable as for beam-column-joints. Note, the smaller the change of direction of chord forces, the lower the radial forces and the better the stirrups absorb them (Fig. 7.3-34).

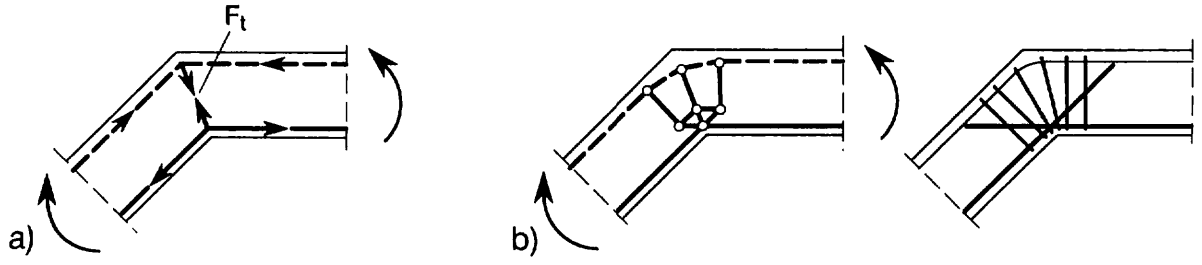


Fig. 7.3-34: Obtuse-angled corner of a beam or plate with positive moment.
a) Simple model. b) Refined model and corresponding reinforcement

In the tapered beam shown in Fig. 7.3-35 a, the compressive chords in the flanges produce an upward force component, F_t, at the kink. For equilibrium the node at the kink must be pulled down by stirrup reinforcement, which can be placed in the web only (Fig. 35 b). The web force is carried on by a truss model, as indicated in Fig. 35 a, until the upward component is compensated at some distance from the kink by the downward vertical loads acting on the beam.

Since chord- and stirrup-forces must join also in plan (Fig. 35 c), the compression chord in the flange is narrowed down at the kink, resulting in a concentration of compression stresses over the web. Moreover, the horizontal deviation of these compression stresses will be

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associated with a tensile force, F_h, which calls for horizontal reinforcement in the flange as shown in Fig. 35 d.

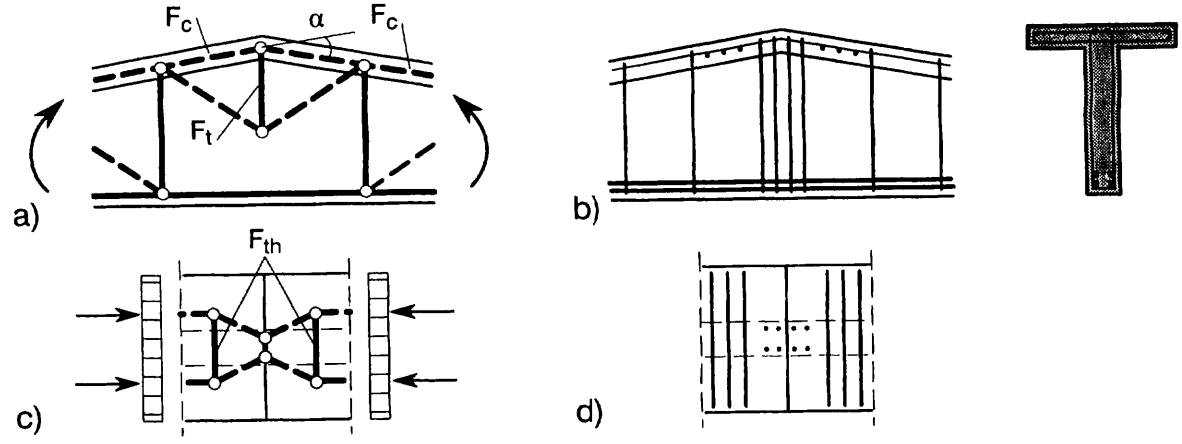


Fig. 7.3-35: Tapered beam with bent top flange:
a) b) Strut-and-tie model and corresponding reinforcement for the bent area in elevation.
c) d) Strut-and-tie model and corresponding reinforcement for the flange near the bent

The consequences of a kink in a chord, as explained above, are more pronounced if the angle between the members increases. Flanges will bend in the transverse or radial direction and thus evade the longitudinal stresses. As a consequence, all the reinforcement necessary for the frame moment must be placed in the web or very close to it. The reinforcement in the flanges (or slabs) contributes only towards crack distribution but not towards transmitting the moment.

For the same reasons, flanges in the compression zone of a profiled girder are ineffective near a connection or joint. Instead, the chord forces concentrate in and near the web, unless the radial forces are directly supported by a radial stiffener, as is standard practice in structural steel engineering (Fig. 7.3-36). The constriction of flange forces is frequently overlooked, when the cross sections are designed using standard methods for B-regions.

The transverse tensile forces, that are always associated with the constriction of chords at kinks or joints of profiled members, can be visualised and analysed with the use of simple strut-and-tie-models (see example in Schaefer/Schlaich/Jennewein (1991)).

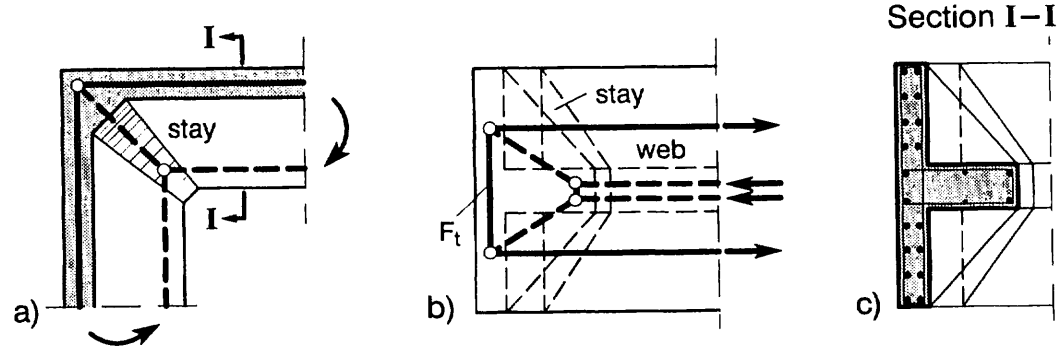


Fig. 7.3-36: Connection of T-beams.
a) Model in the frame plane; b) Top view of the model; c) Cross-section

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7.3.4 Corbels

(1) Load bearing behaviour and necessary checks

Similar to a beam-column-connection, the corbel transmits a closing moment to the column below and an opening moment to the column above it (provided the column above exists and can resist such a moment). As for frames, the column sections in the horizontal faces through the corners can be checked with standard methods for their sectional effects M_{Sd}, N_{Sd} and V_{Sd}, though, more rigorously, these faces are well within the D-region with disturbed stress flow.

One essential difference to the beam column-connection is the relatively large shear force in the corbel from its load F_{Sd}, which is transmitted by a diagonal strut from node 1 to a concentrated compression node 2 at the lower corner (Fig. 7.3-37 a). There, the vertical component F_{Sd} of the strut is supported by the compression chord of the column below and the tension chord F_{t2} in the column above. Usually the tension force is comparatively small.

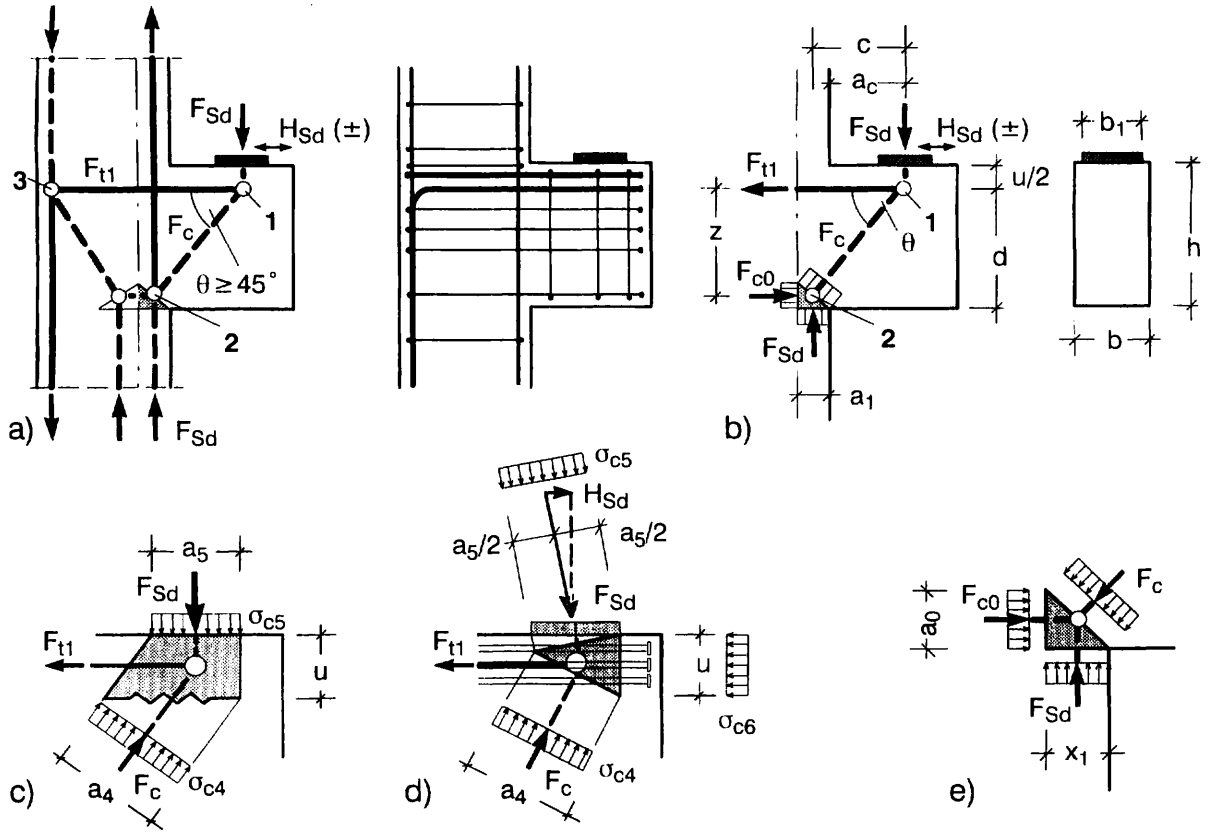


Fig. 7.3-37: Typical corbel.
a) Overall model and corresponding reinforcement.
b) Model of a typical cantilevering part.
c) Standard compression-tension node 1 for vertical load.
c) Idealised hydrostatic node 1 for inclined load and anchorage behind the node region.
e) Detail of the compression node 2 corresponding with b) above

The kinks in the described loadpath of the corbel load result in a horizontal tie F_{t1} in node 1 under the bearing and a horizontal compression force F_{c0} in the supporting node 2. These forces are balanced by a pattern of forces in the column which is almost a mirror image of that

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in the cantilevering part or the D-region. It can be concluded, that the column region adjacent to the corbel is subject to equivalent stresses, too. Since the overall models of corbel regions are different, depending on the particular loading and boundary conditions of the column above and below, a model of the whole D-region including the column part should be considered for the design [Schlaich/Schaefer (1998)]. It will, in fact, not suffice to simply anchor the main reinforcement behind the column face. With this in mind, the corbel part can be designed as explained in the numerical example in Sec. (2) below, because it is very frequently of the type shown in Fig. 7.3-37 b.

The example will show, that the design for horizontal compression F_{c0} in node 2 and for horizontal tension F_{t1} is identical with the design of a B-region for moment and normal force. It may therefore also be performed using tables or computer routines for beams. Note however, that the design moments are not referred to the column face, but to the relevant vertical section I-I through node 2. This section is determined by the requirement of zero shear force, in order to obtain the maximum chord forces F_{t1}.and F_{c0}. As a consequence, the lever arm c of the corbel load F_{Sd} with respect to its support is considerably larger than its distance a_c from the column face, a fact which is frequently overlooked in the literature on corbels and in practical design.

For the evaluation of forces and stresses, a horizontal force $H_{Sd} \geq 0.2\ F_{Sd}$ from restrained longitudinal deformations of the supported member should also be taken into account.

Nodes are critical points of the structure. Node 1 under the bearing corresponds mechanically to an end-support node turned upside down. For vertical load, it can be designed as a standard compression tension node as shown in Fig. 4.4-99.

To get an idea of an influence of the additional horizontal load component, the hydrostatic node region in Fig. 7.3-37 d will be discussed. It is assumed, that the horizontal tie is completely anchored behind the node, which is not far from reality if anchor plates or studs or loop reinforcement are used. Principal stresses in the node region and in the oncoming struts (including the diagonal strut) will not exceed the node strength if the inclined stresses σ_{c5} under the bearing and the (fictitious) horizontal compression stress σ_{c6} do not exceed it. These stresses should be checked against f_{cd1} in cases with anchorage by anchor plates or f_{cd3} where bond contributes essentially to the anchorage of reinforcing bars.

The relevant inclined constant stress σ_{c5} follows from equilibrium and the node's geometry.

$$\sigma_{c5} = \frac{V_{Sd}}{b\,a_5}\left(1+\left(\frac{H_{Sd}}{V_{Sd}}\right)^2\right)$$

In this formula the node stresses are assumed to be distributed over the whole width b of the corbel. This simplification leads to the same result as a check of the local pressure for the width b_1 of the bearing plate against an increased strength according to MC90, Clause 6.9.2. However, the code rule is restricted to bearing width $b_1 \geq b/4$.

The area of support should be smaller than the area enclosed by the chord reinforcement to preclude any damage to the edges of the corbel. This holds true also for such cases where a mortar bed is used. Uniform compression across the area of support is best achieved with

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elastomer supports which, to a certain extent, will also tolerate horizontal displacement and rotations.

The anchorage length of the tension chord in node 1 starts beneath the bearing plate and in the majority of cases is so short that horizontal loops, welded cross bars or even anchor plates are required. Double headed studs (head diameter 3 times bar diameter) proved very effective as chord reinforcement in corbel tests, that were recently carried out by Ghali and Birkle in Calgary (1999, not yet published). The use of bent-down chord bars should be avoided, unless the bearing plate is located well behind the bend.

Node 2 is essentially a compression node, since the vertical compression dominates the tie force from the upper column. All node stresses are safe, because the principal stresses are automatically checked with the standard analyses for B-regions suggested above.

Node 3 is an example for a combination of two types of compression-tension nodes. The horizontal tie force due to the closing frame moment is transmitted by bent bars according to Fig. 4.4-100. The other part of the horizontal tie force, due to the opening moment of the upper column, deviates the compression chord of the upper column towards node 2. The corresponding reinforcement should therefore enclose the compression chord of the column. Loops or stirrups are adequate for the anchorage in this case. If all the corbel bars are bent into the column, some additional stirrups must be provided immediately above and below the corbel reinforcement, to avoid spalling of the concrete near the bend.

Compression stresses in the diagonal struts do not exceed those in the adjacent nodes. However, the struts will be safe only, if the transverse tensile stresses of the bottle shaped stress-fields are endured by the concrete or if additional web reinforcement is provided (Chapter 7.3.1 (3.3.2)). For the ease of construction, these reinforcements are usually chosen at right angles. Horizontal stirrups are superior to the vertical type for very short corbels. The MC90, Clause 6.8.2.2.3, recommends closed horizontal stirrups if $a < z/2$ and closed vertical stirrups if $a > z/2$, or a combination of both in all cases $a < z$. These reinforcements for the bottle shaped stress fields need not be designed to cover fully the forces derived for linear elastic material behaviour, because the tensile forces will be greatly reduced after cracking when the stress trajectories straighten between the concentrated nodes. Different codes of practise therefore give very different rules for the required web reinforcement in corbels.

Under the aspects of crack control, it is prudent to provide an additional diagonal reinforcement near the top corner (compare Fig. 7.3-32b).

(2) Standard design for a corbel (numerical example)

An overall strut-and-tie-model of the D-region with the corbel is drawn in Fig. 7.-38 a. It is very similar to the model in Fig. 7.3-36 a, but without an essential moment in the upper column. The projecting part is typical (Fig. 7.3-37b) and can be designed as follows:

(2.1) Design of the tension chord and compression node 2

The chosen division line I-I for the projecting part is determined by the requirement of zero shear force in this section, in order to obtain the maximum chord force F_{tI}. The

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corresponding support width a_1 for F_{Sd} in node 2 will be derived from the concrete design strength f_{cd1} for compression nodes (Eq. 4.4-110):

$$f_{cd1} = 0.85\,(1 - f_{ck}/250)\,f_{cd} = 0.85 \cdot (1 - 30/250) \cdot 20.0 = 15.0 \text{ MPa}$$

$$a_1 = F_{Sd} / b\, f_{cd1} = 0.33 / 0.30 \cdot 15.0 = 0.073 \text{ m}$$

With a_1 the lever arm c of the corbel load F_{Sd} is determined:

$$c = a_c + a_1/2 = 0.150 + 0.073/2 = 0.187 \text{ m}$$

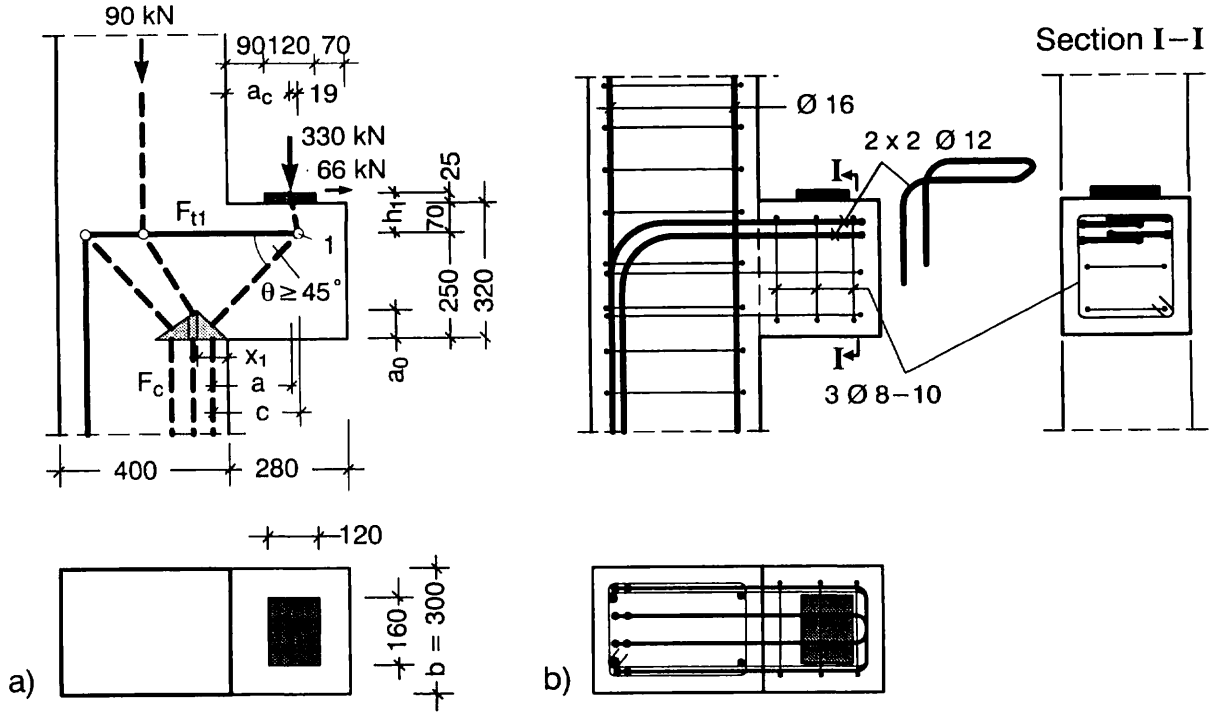


Fig 7.3-38: Corbel (numerical example).
a) Geometry and model
b) Reinforcement

The moment due to the corbel load, M_{Sd}, with respect to the chord reinforcement is counteracted in section I-I only by the horizontal compression force F_{c0} in node 2 with the lever arm z. If the necessary vertical depth a_0 = 0.08 m of the compression node is conservatively estimated or approximately known form a pre-calculation, then the horizontal compression stresses σ_{c0} due to F_{co} and the main tension tie F_{t1} can be checked simply by the following equations (notations as in Fig. 7.3-37 and 38):

$$z = d - a_0/2 = 0.25 - 0.08 / 2 = 0.21 \text{ m (internal lever arm)}$$

$$M_{Sds} = F_{Sd} \cdot a + H_{Sd} \cdot h_1 = 330 \cdot 0.187 + 66 \cdot 0.095 = 68.0 \text{ kNm}$$

$$F_{co} = \cdot M_{Sds} / z = 68.0 / 0.21 = 323 \text{ kN}$$

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$\sigma_{c0} = F_{co} / b \cdot a_0 \leq f_{cd1}$ $= 0.323 / 0.30 \cdot 0.08 = 13.5$ MPa $< f_{cd1}$.

$F_{t1} = F_{co} + H_{Sd}$ $= 323 + 66 = 389$ kN

req. $A_s = F_{t1} / f_{yd}$ $= 389\,000 / 435 = 894$ mm^2 8 Ø 12 mm Steel S 500

Alternatively, the horizontal compression stresses in the compression node 2 and the chord reinforcement may be checked for M_{Sds} and H_{Sd} by standard methods for a beam with the cross-section of the corbel.

(2.2) Design of node 1 under the bearing plate

The reinforcement in the example shall be anchored by loops (Fig. 7.3-38b). The node strength f_{cd3} from Eq. 4.4-112 is appropriate in this case of a tension-compression node.

$f_{cd3} = 0.7\,(1 - f_{ck}/ 250)\, f_{cd} = 0.7 \cdot (1 - 30 / 250) \cdot 20.0 = 12.3$ MPa

Applying the equation for σ_{c5} above, the node stresses under the chosen bearing plate are acceptable:

$\sigma_{c5} = 0.33 \cdot (1 + 0.20^2) / 0.3 \cdot 0.12 = 9.53$ MPa < 12.3 MPa

Looking more rigorously at the problem, the effective length of the bearing plate should be somewhat reduced to consider the eccentricity of the load in the support area.

The height u of node 1 is also large enough to keep the fictitious compression stress σ_{c6} below f_{cd3} (Fig. 7.3-37 d):

$\sigma_{c6} = F_{t1} / b \cdot u \cdot = 0.389 / 0.3 \cdot 0.14 = 9.26$ MPa

As a consequence, also the stresses σ_{c4} in the diagonal strut are acceptable – provided that web reinforcement prevents splitting of the diagonal strut.

Finally, the anchorage length of the reinforcement should be considered, following the code rules. However, it is the authors opinion that loops of thin bars are able to transfer their full yield force by radial pressure in the bend. Essential is that the loops enclose the node region. Crack widths are also not critical for the well distributed reinforcement bars with only 12 mm diameter.

(3) Corbels with suspended or indirect load

Two typical models for suspended loads, applied at the bottom of the corbel, are shown on Figs. 7.3-39 a and b. When combined, these models will render a reinforcement layout that provides a favourable crack distribution pattern and is suitable also for indirect load transfer, for instance from a crane girder (Fig. 7.3-39 c). Note, however, that the contributions of the two models should be selected as a function of the chord forces of the adjoining columns.

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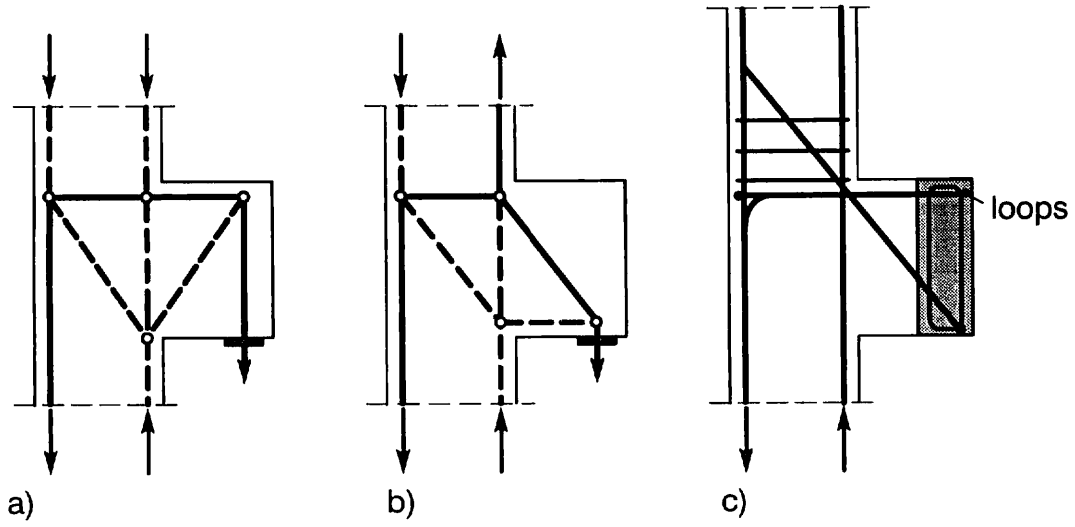


Fig. 7.3-39: Corbels with suspended load or indirectly applied load:
a) and b) Simple models showing possible load-bearing effects.
c) Reinforcement for the combined actions out of a) and b)

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7.4 Reinforcement Layout of Typical Elements

by G. Mancini

The following figures represent the reinforcement layout of some typical structural elements, in particular:

- Fig. 7.4 - 1 to Fig. 7.4 - 3: Pier of railway viaduct for high speed lines;
- Fig. 7.4 - 4 and Fig. 7.4 - 5: Abutment of one highway bridge;
- Fig. 7.4 - 6 to Fig. 7.4 - 9: Foundation on piles of the bridge pier pictured in Fig 7.4 - 1;
- Fig. 7.4 - 10 to Fig. 7.4 - 12: Prestressed beam for highway bridge;
- Fig. 7.4 - 13 and Fig. 7.4 - 14: Prestressed slab designed in chapter 7.2;
- Fig. 7.4 - 15 and Fig. 7.4 - 16: Prestressed slab for highway bridge;
- Fig. 7.4 - 17 and Fig. 7.4 - 18: Box culvert.

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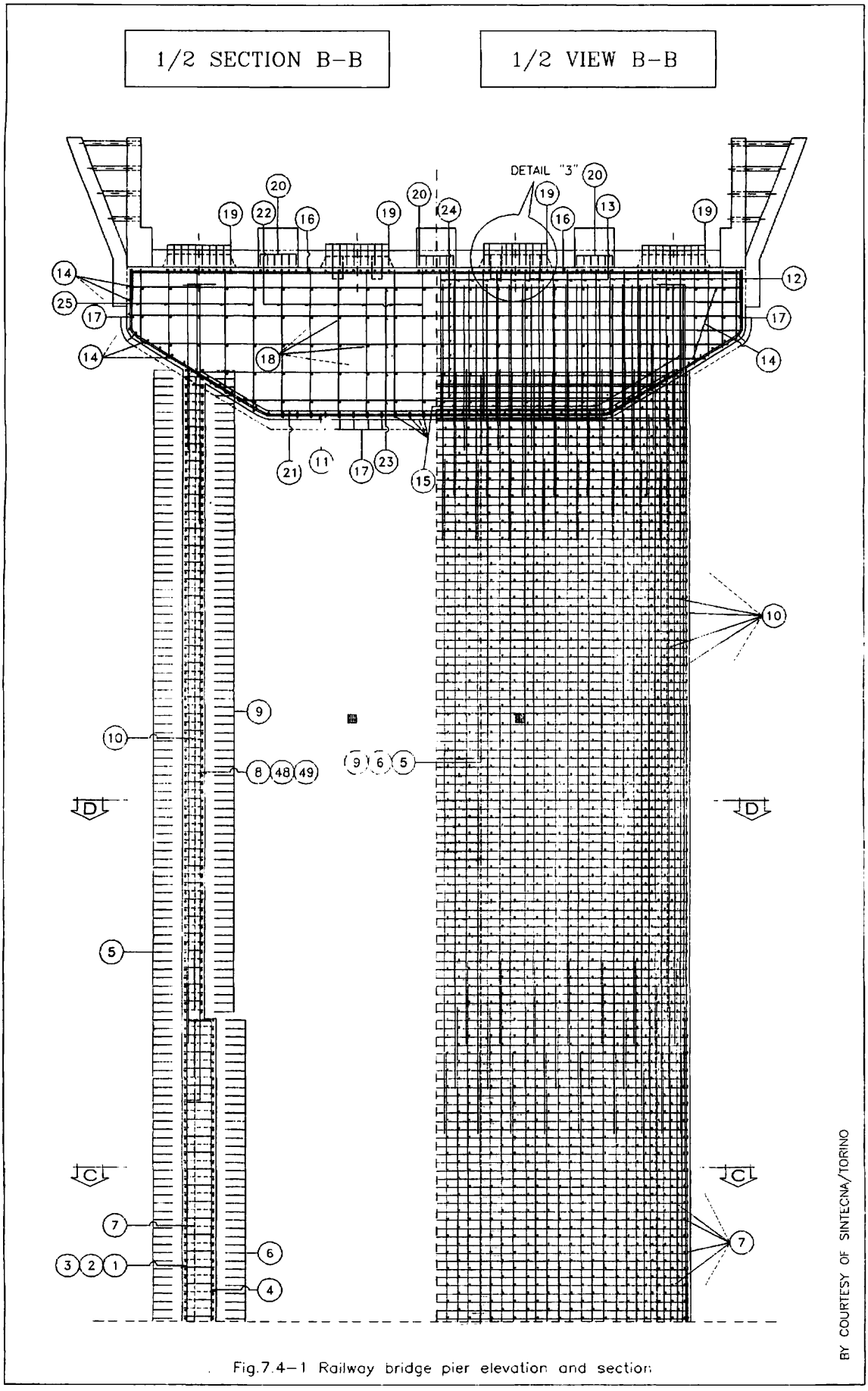


Fig.7.4–1 Railway bridge pier elevation and section

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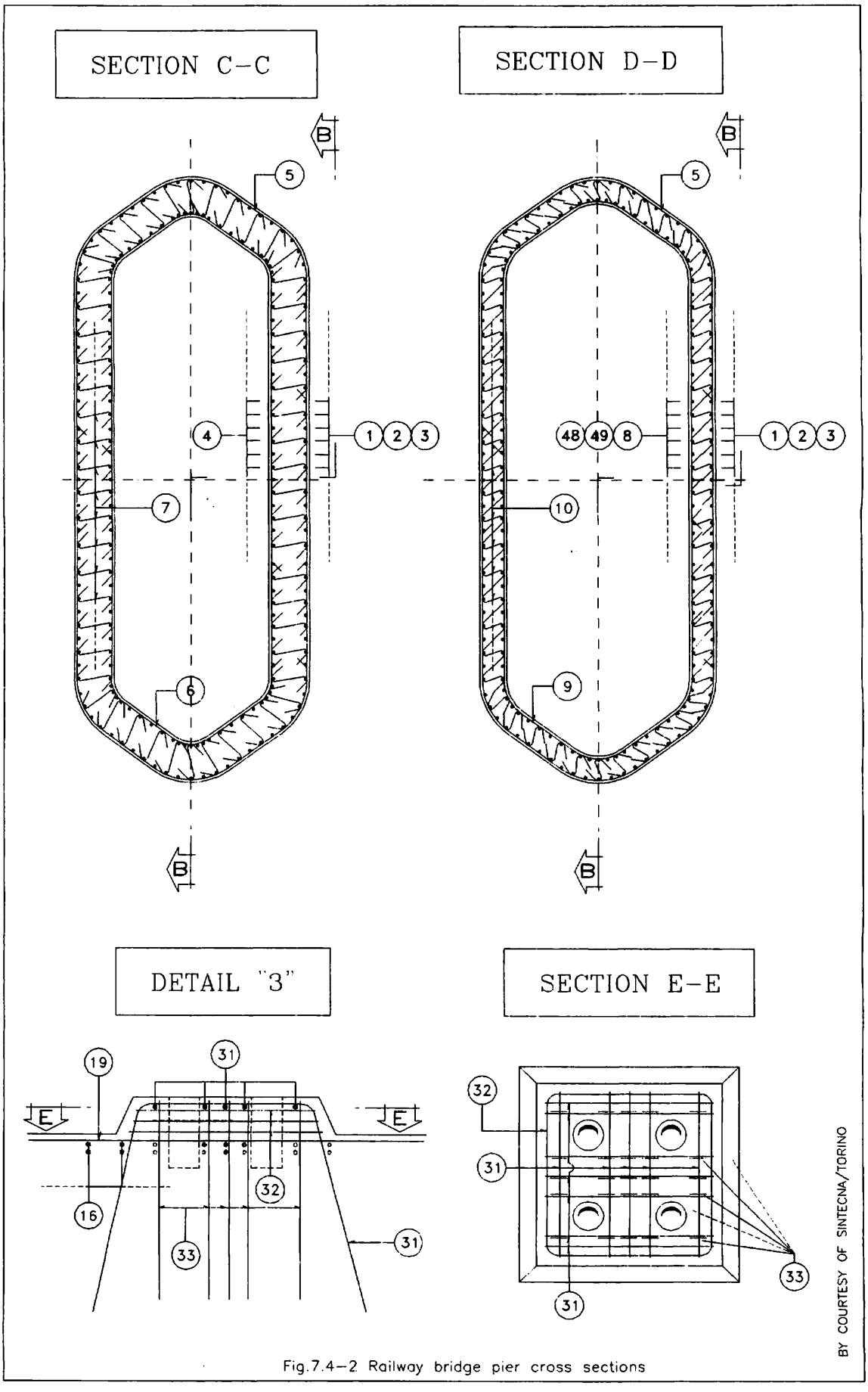


Fig.7.4–2 Railway bridge pier cross sections

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POS.	N.	Ø / DIST	LENG. (cm)	SHAPE	L.TOT. (cm)	WEIG. (Kg)
1	36	18 / 60	1200x2 + 346	296 50 1200 1150 50 alternate with pos. 2 e 3	98856	1975
2	36	18 / 60	1120+ 1200+ 426	176 50 1200 1070 50 alternate with pos. 1 e 3	98856	1975
3	36	18 / 60	1040+ 1200+ 506	456 50 1200 990 50 alternate with pos. 1 e 2	98856	1975
4	108	18 / 20	1176	50 1076 50	127008	2537
5	184	18 / 15	1080+ 1080+ 600	92 20 377 20 R=36 sv=91 92 R=96 sv=120 377 20 177 20 560 20	369840	7388
6	57	18 / 15	1200 + 1090	92 20 492 92 260 20 R=44 sv=42 R=44 sv=45 577 20 175 20	127110	2539
7	1566	18 / 30x40	104	75 54 25	162864	3253
8	36	18 / 60	1200 + 456	426 30 1170 30 alternate with pos. 45 e 49	59616	1191
9	77	18 / 15	1200 + 1857	92 20 493 92 260 20 R=44 sv=61 R=64 sv=80 514 20 238 20	181489	3625
10	1926	18 / 30x40	84	25 34 25	161784	3232
11	21	26 / 20	(435+435)x2 + (940+940)	100 100 R=44 sv=24 156 156 R=24 sv=25 256 30 30 256 R=44 sv=24	76020	1518
12	10 + 10	20 / 25	806 + 1058	122+248 402 80 80	18640	460
13	24	20 / 25	1090	262 402 82 82	26160	645
14	6 + 6	20 / 25	440	horizontal bend 79 402 79	5280	130
15	36	26 / 25	560	79 402 79	20160	840
16	21	26 / 20	1195 + 1170	1087 491 1081 149 44 145	49455	2061
17	49	26 / 25	490	50 390 50	24010	1001
18	94	20 / 50x50	440	22 396 22	41360	1020
19	10 < 4	26 / 125	620	396 112 112	24800	1034
20	6 < 3	26 / 125	520	396 62 62	9360	390
21	21	20 / 20	880	130 580 130 R=35 sv=20 R=35 sv=20	18480	456
22	120	18 / 50x40	300	262 19 19	36000	719
23	50	18 / 50x40	750 + 1190	710.1090 20 20	47000	939
24	9 + 9	20 / 25	750 + 1190	710.1090 20 20	16920	417
25	100	18 / 50x40	154 + 286	122.248 16 16	21700	439
31	8 < (5+7)	18	350	98 126 126	33600	671
32	8 < 3	18	478 + 538	106.121 106.121 271	12192	244
33	8 x 25	18	149	20 121	28000	559
48	36	18 / 60	1120 + 536	506 30 1090 30 alternate with pos. 8 e 49	59616	1191
49	36	18 / 60	1040 + 616	586 30 1010 30 alternate with pos. 8 e 48	59616	1191

Fig.7.4–3 Railway bridge pier reinforcement table

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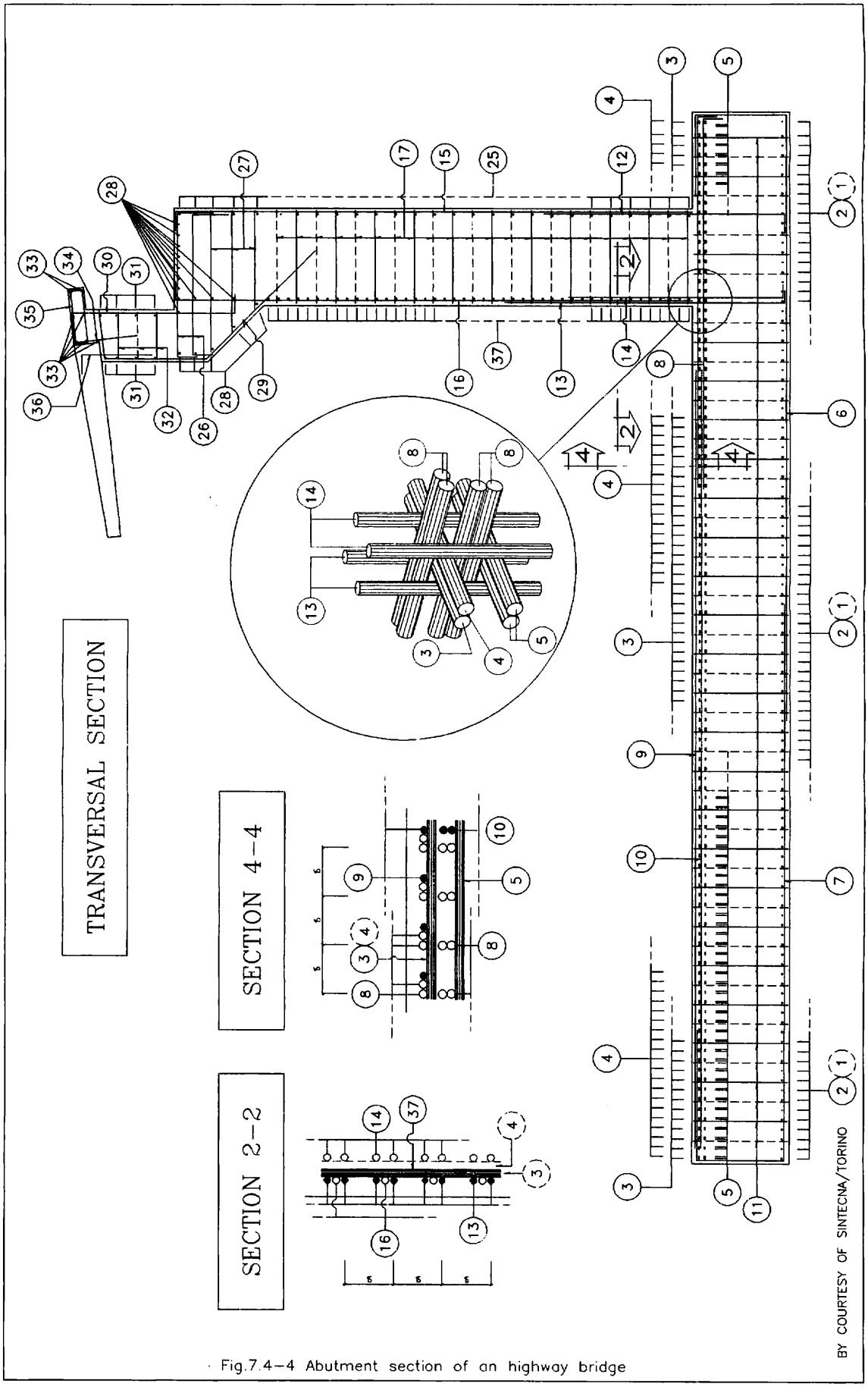


Fig.7.4–4 Abutment section of an highway bridge

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POS.	N.	Ø / DIST	LENG. (cm)	SHAPE	L.TOT. (cm)	WEIG. (Kg)
1	108 x 2	22 / 15	400	30 / 190 / 240	86400	2578.21
2	108	22 / 15	1200	1200	129600	3867.32
3	108	26 / 15	1200	1200	129600	5401.46
4	108 x 2	26 / 15	450	420 / 30	97200	4051.1
5	108 x 2	26 / 15	350 + 350	350 / 350	151200	6301.7
6	87	22 / 15	1200	126 / 142 / 932	104400	3115.34
7	87	22 / 15	1200	200 / 142 / 858	104400	3115.34
8	87	26 / 15	600 x 4	570 / 30	208800	8702.35
9	87	16 / 15	1200 + 1200	1200	208800	3296
10	4 + 4	20 / 90	1200 + 1200	1200	19200	474
11	1170	16 / 30x60	180	20 / 140 / 20	210600	3324
12	37	26 / 30	400	370 / 30	14800	616.83
13	60	22 / 15	450 + 450	430 / 20	54000	1611.38
14	60	22 / 15	350 + 350	330 / 20	42000	1253.3
15	37 + 6x2	26 / 30	820	30 / 790	40180	1674.62
16	60	22 / 15	820	30 / 790	49200	1468.15
17	352	14 / 30x60	180	140 / 20 / 20	63360	765.65
25	27	22 / 30	1200	60 / 1080 / 60	32400	966.83
26	37x2 + 3x5	22 / 30	300	227 / 25 / 48 / 3 additional bars in the bearing support	26700	796.74
27	31 x 3	22 / 30	280 - 210	220 - 150 / 30 / 130	22785	679.91
28	17	22	1200	1080 / 60 / 60	20400	608.74
29	31	22 / 30	450	70 / 159 / 221	13950	416.27
30	37	22 / 30	300	35 / 240 / 25	11100	331.23
31	4 + 4	16 / 30	1200	1080 / 60 / 60	9600	151.52
32	45	12 / 30x60	100	70 / 15 / 15	4500	39.95
33	7	12	1200	1080 / 60 / 60	8400	74.58
34	28	16 / 30	120	20 / 100	3360	53.03
35	28	12 / 30	220	16 / 13 / 76 / 16	6160	54.69
36	14	20 / 60	120	20 / 100	1680	41.43
37	44	22 / 15	1200	1080 / 60 / 60	52800	1575.57

Fig.7.4–5 Abutment reinforcement table

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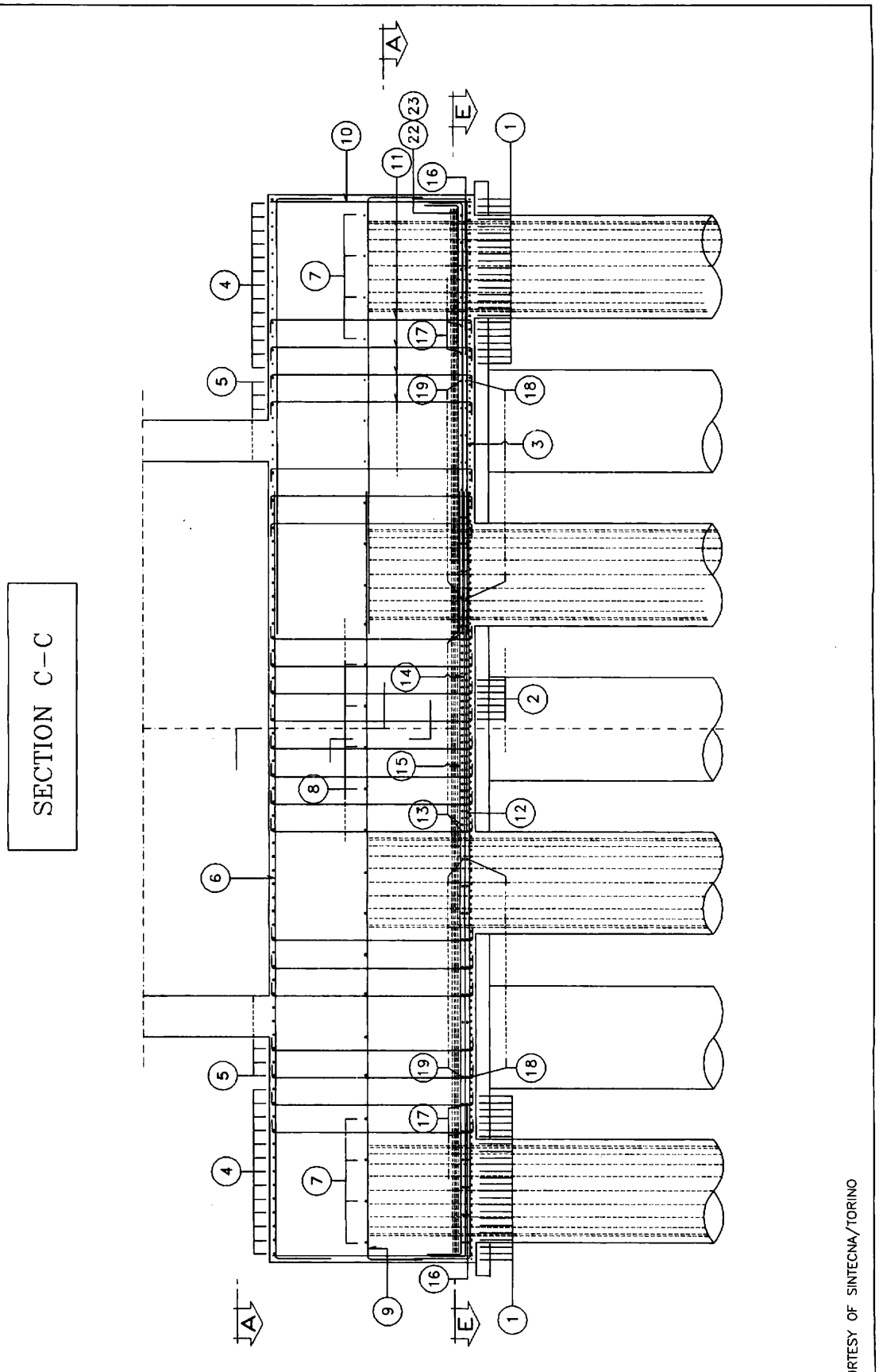


Fig.7.4–6 Pier foundation. Vertical section

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POS.	N.	ø / DIST	LENG. (cm)	SHAPE	L.TOT. (cm)	WEIG. (Kg)
1	25 + 25	26 / 10	259 ÷ 1116	80 [99÷956] 80	34375	1433
2	105	26 / 10	1142	80 [982] 80	119910	4998
3	99	26 / 10	(814÷1200) + (600÷720)	80 [734÷1120 200 520÷640] 80 alternate arrangement	230373	9601
4	18 + 18	26 / 20	259 ÷ 1116	99÷956 80 [ ] 80	17875	745
5	52	26 / 20	1142	982 80 [ ] 80	59384	2475
6	50	26 / 20	(814÷1200) + (600÷720)	734÷1120 520÷640 80 [200] 80 alternate arrangement	116350	4849
7	4 + 4	20 / 60	250 ÷ 893	170÷818 40 [ ] 40	4572	118
8	18	20 / 60	1062	982 40 [ ] 40	19116	471
9	17	20 / 60	(750÷1200) + (538÷600)	710÷1160 500÷560 40 [100] 40 alternate arrangement	35921	886
10	202	26 / 20	292	292	58984	2458
11	710	16 / 40x40	330	292 [19 19	234300	3698
12	31	26 / 10	1076	50 [976] 50	33356	1390
13	31	26 / 10	1070	50 [970] 50	33170	1382
14	31	26 / 10	1200 + (530÷655)	50 [1150 200 480÷605] 50 alternate arrangement	73935	3081
15	31	26 / 10	1200 + (524÷650)	50 [1150 200 474÷600] 50 alternate arrangement	73594	3067
16	6 + 6	26 / 40	123 + 836	123÷836	5754	240
17	6 + 6	26 / 40	117 + 830	117÷830	5682	237
18	9 + 9	26 / 40	976	976	17568	732
19	9 + 9	26 / 40	970	970	17460	728
20	8 + 8	26 / 40	1062 ÷ 1200	1062÷1200	18096	754
21	8 + 8	26 / 40	1056 ÷ 1200	1056÷1200	18048	752
22	(31+30) x 2	26 / 20	1040 + 1200	40 [side by side] 40 960÷1120	138880	5788
23	(31+31) x 2	26 / 20	1040 + 1200	43 [side by side] 43 954÷1114	138880	5788
24	(4x2) x 4	26 / 60	320	320	10240	427

Fig.7.4–7 Pier foundation. Reinforcement table

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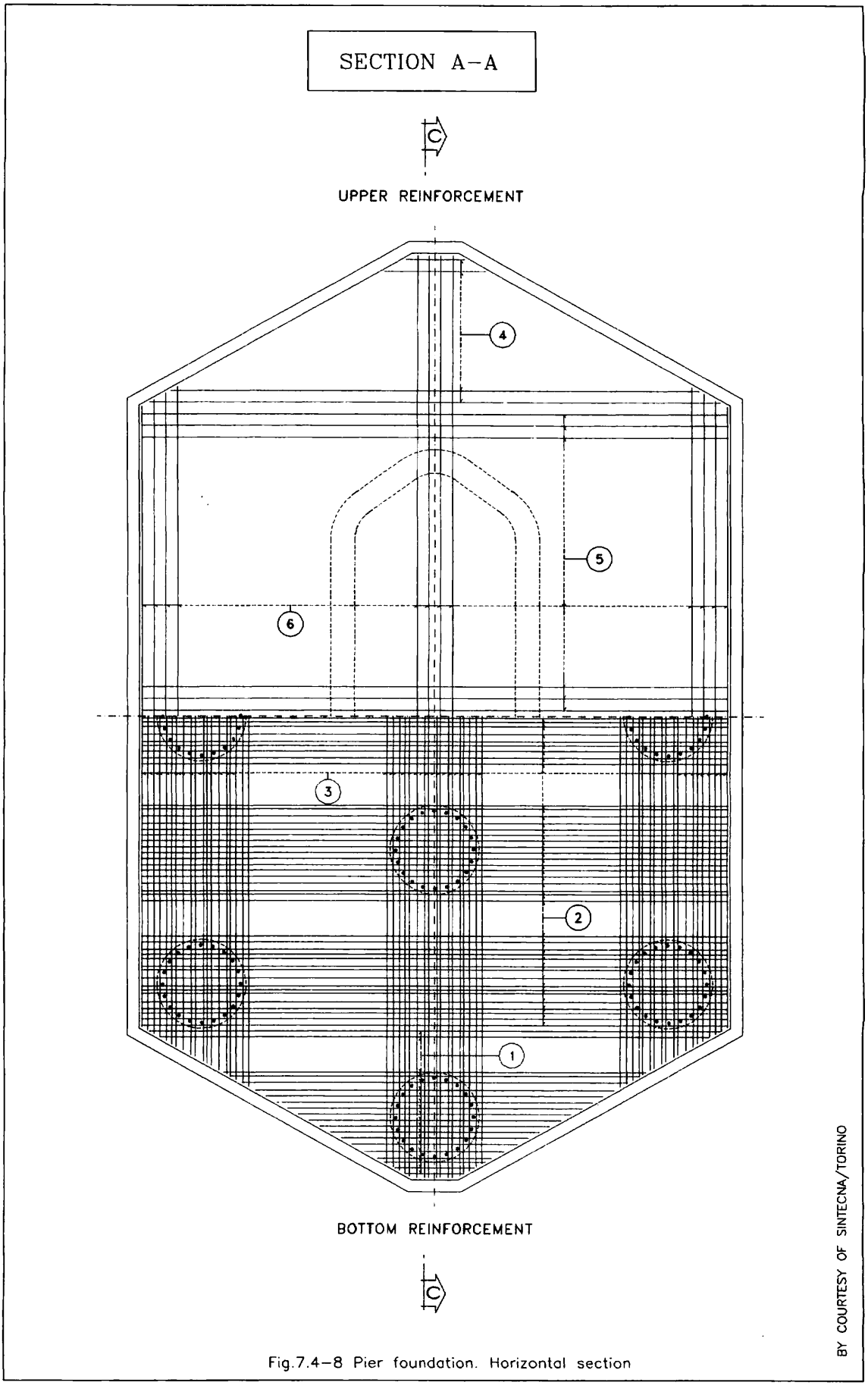


Fig.7.4–8 Pier foundation. Horizontal section

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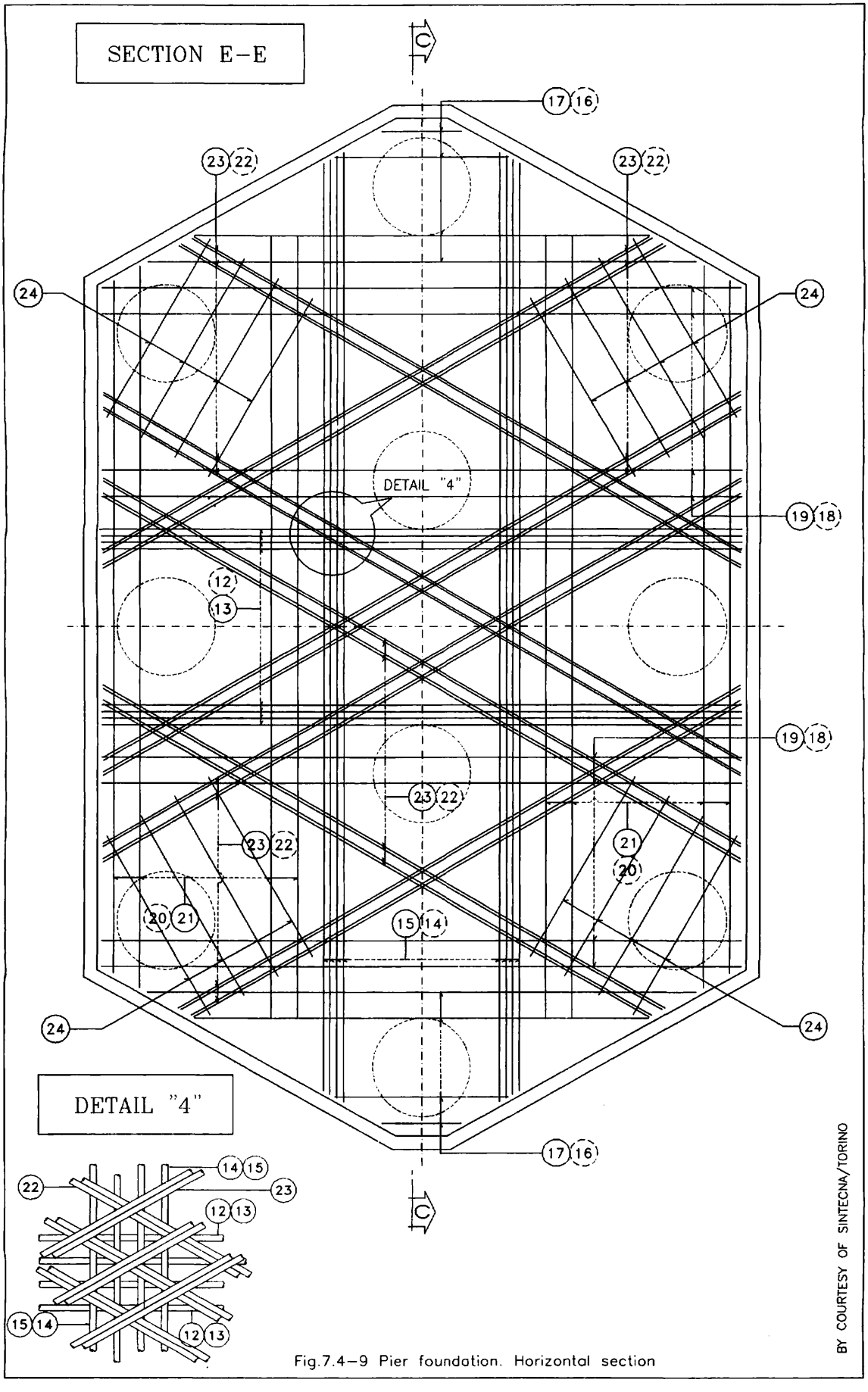


Fig.7.4–9 Pier foundation. Horizontal section

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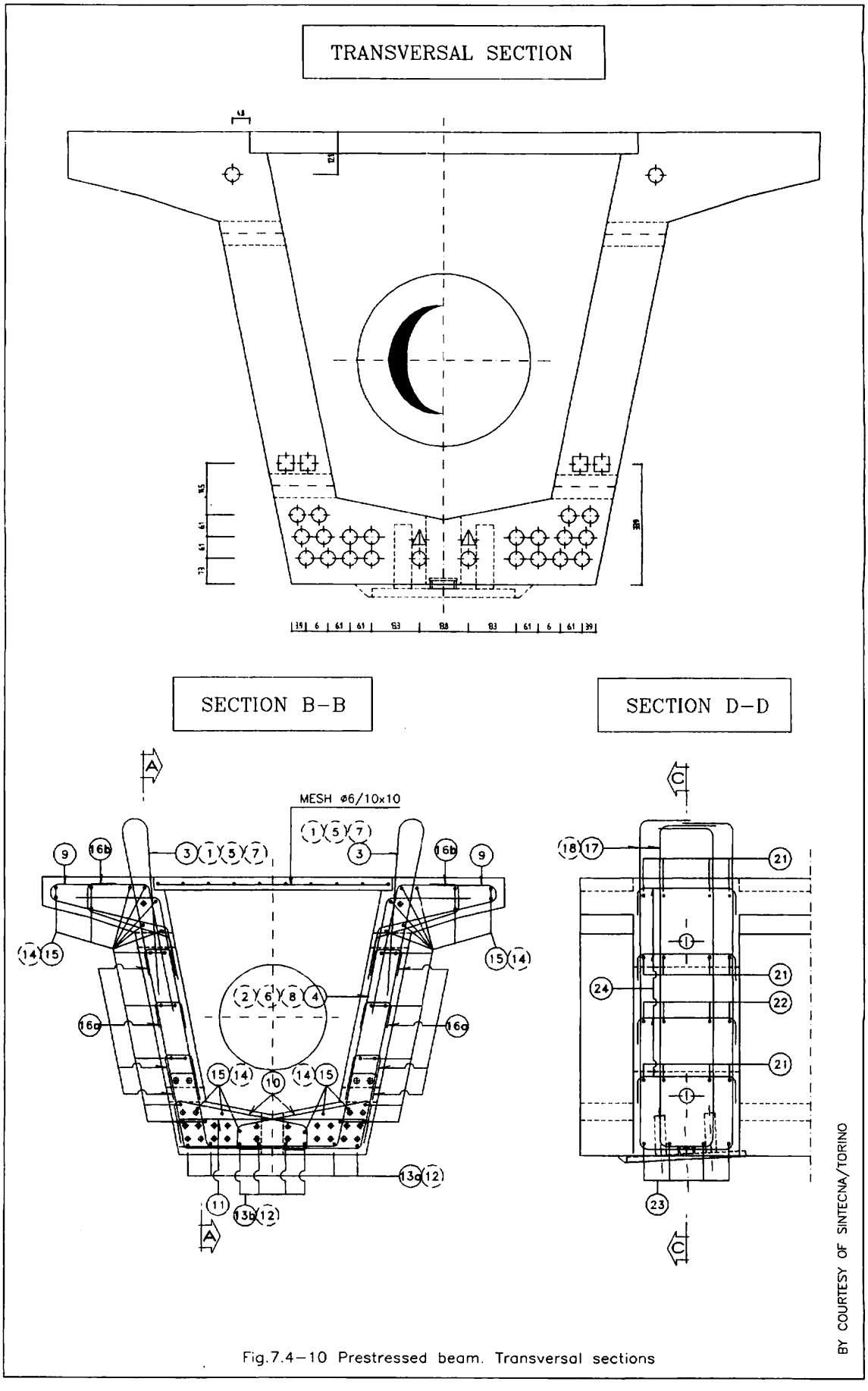
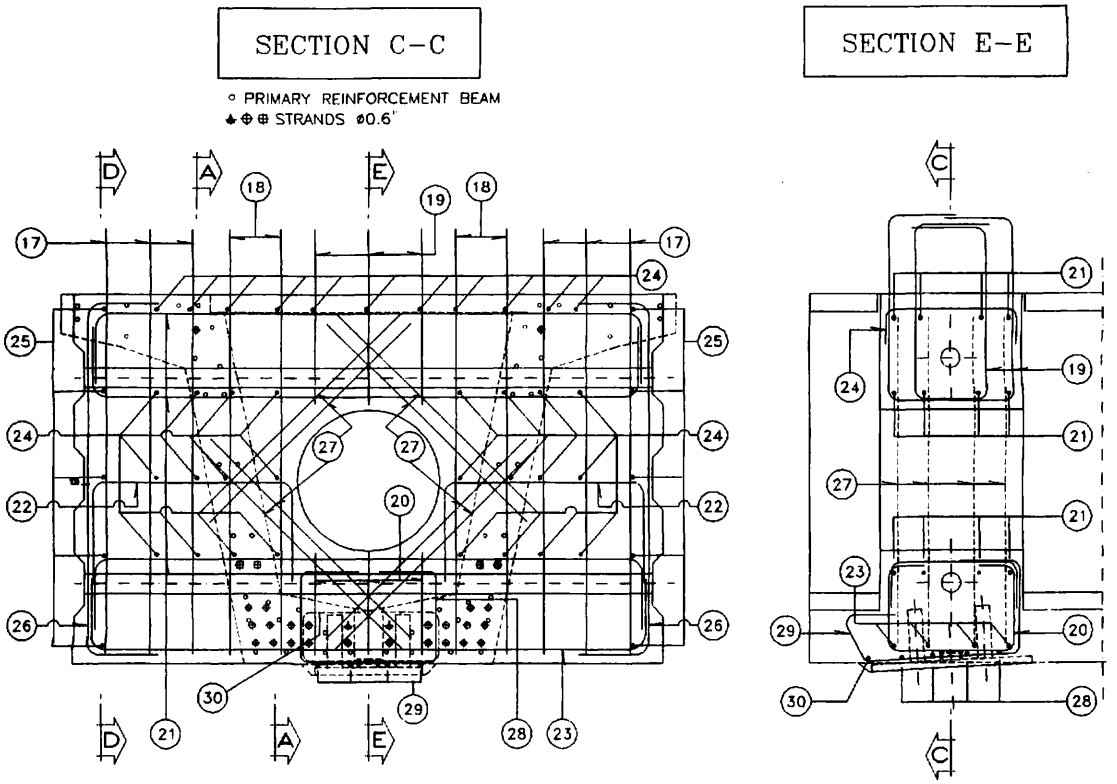


Fig.7.4–10 Prestressed beam. Transversal sections

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POS.	N.	ø / DIST	LENG. (cm)	SHAPE	L.TOT. (cm)	WEIG. (Kg)
1	18 × 2	16 / 10	460		19960	189
2	18 × 2	16 / 10	300		7800	123
3	21 × 2	14 / 20	460		19320	293
4	21 × 2	14 / 20	300		12600	152
5	25 × 2	12 / 20	460		23000	204
6	25 × 2	12 / 20	300		15000	133
7	25	10 / 20	460		11500	71
8	25	10 / 20	300		7500	46
9	121 × 2	8 / 20	150		36300	143
10	121 × 2	8 / 20	80		19360	76
11	121	10 / 20	90		10890	67
12	8	10	1200	1200	9600	59
13a	4 × 4	22	1180	222 / 98 / 780 bend within the web	8800	263
13b	4 × 4	14	830	41 / 9 / 780	6640	80
14	42	10	1200	1200	50400	311
15	42 × 42	12	740	10 / 730	62160	552

POS.	N.	ø / DIST	LENG. (cm)	SHAPE	L.TOT. (cm)	WEIG. (Kg)
16a	4x121 × 2	8 / 20	29		28072	111
16b	121 × 2	8 / 20	65		15730	62
17	13+3 × 2	14 / 12.5	320 ÷ 300		7440	90
18	12+2 × 2	18	320 ÷ 300		4960	99
19	3 × 2	14 / 15	180 ÷ 160		2040	25
20	3 × 2	14 / 15	121 ÷ 70		1140	14
21	(4+3)x2	20	195	195	4680	115
22	14x2)x2	16	94	54	1504	24
23	4 × 2	22	195	195	1560	47
24	35 × 2	10	60	34	4200	26
25	(5+5) × 2	10	100	33 / 34 / 33	2000	12
26	(4x2)x2	12	140	22	2240	20
27	(8+4)x2	10	80	80	5120	32
28	4 × 2	20	120		960	24
29	4 × 2	20	125		1000	25
30	1 × 2	14	90		180	2

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Fig.7.4–11 Prestressed beam. Transversal sections and reinforcement table

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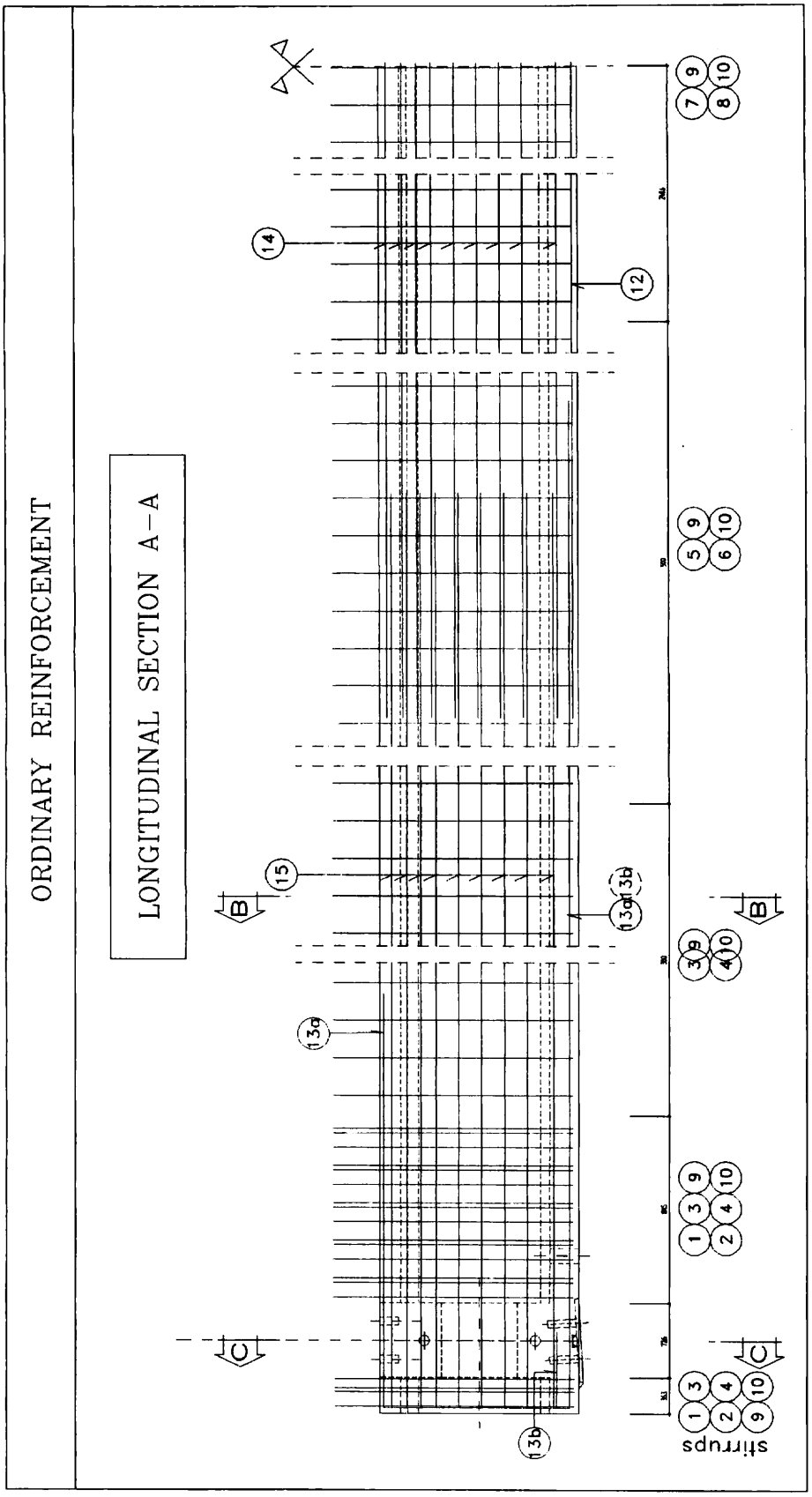


Fig.7.4–12 Prestressed beam. Longitudinal section

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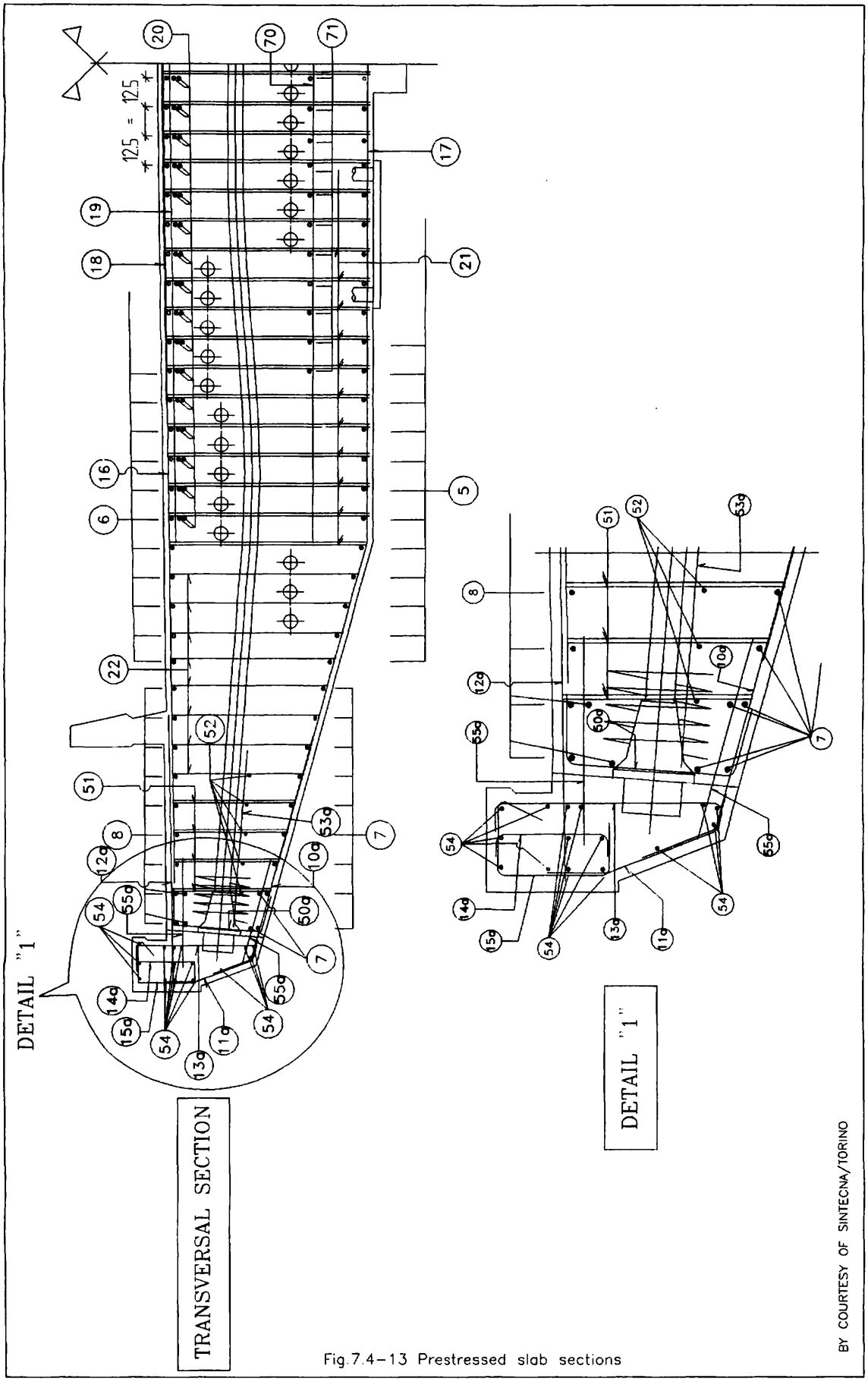


Fig.7.4–13 Prestressed slab sections

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POS.	N.	Ø / DIST	LENG. (cm)	SHAPE	L.TOT. (cm)	WEIG. (Kg)
5	42	22 / 215	700+ (1200x5)+ 700	25 | 1200x5 | 25 ; 675 (150) (150) 675	310800	9274.4
6	42	22 / 215	1200+ (1200x4)+ 1200	1175 (150) (150) 1175 ; 25 | 1200x4 | 25	302400	9023.74
7	11 x 2	18 / 215	600+ (1200x5)+ 600	25 | 1200x5 | 25 ; 575 (117) (117) 575	158400	3164.17
8	11 x 2	18 / 215	1140+ (1200x4)+ 1140	1117 (117) (117) 1117 ; 23 | 1200x4 | 23	155760	3111.43
10a	10x4 + 17x4	12 / 215	200 + 200	bend after tensioning 140g 170 ; 30	43200	383.54
11a	10x4 + 17x4	12 / 215	90	23 176g ; 52 140g 15	9720	86.3
12a	10x4 + 17x4	20 / 215	240	horizontal bend 197 ; 18 | 25	25920	639.23
13a	10x4 + 17x4	16 / 215	120	16 ; 150g 86 ; 18	12960	204.55
14a	15x4 + 17x4	16 / 43	70	18 ; 42 ; 15	8960	141.42
15a	13x4 + 17x4	16 / 215	115	33 ; 42 150g 20 ; 20	13800	217.81
16	17 x 2	26 / 215	1200	600 600 ; 198g	40800	1700.46
17	34 x 2	18 / 10.75	1200	250 700 250	81600	1630.03
18	17 x 2	26 / 215	500	250 250 ; 198g (alternate with pos.16)	17000	708.52
19	34 x 2	26 / 10.75	700	350 350 ; 198g	47600	1983.87
20	32 x 2	26 / 215	600 + 600	600	76800	3200.87
21	312 x 2	14 / 215x43	(185÷180) + (185÷180)	149÷144 [18 (disporre a quinconce) 18	227760	2752.28
22	120 x 2	16 / 43x43	135 + 175	97÷137 [19 (disporre a quinconce) 19	37200	587.14
50a	11x4 + 17x4	20 / 215	120	27 ; 67 109g ; 30	13440	331.45
51	201 x 4	18 / 215	105 ÷ 124	16 ; 150g 73÷92 ; 16	92058	1838.93
52	5 x 2	18 / 215	600+ (1200x5)+ 600	20 | 1200x5 | 20 ; 580 (120) (120) 580	72000	1438.26
53a	6x4 + 9x4	16 / 43	150	130 ; 20	9000	142.05
54	15 x 2	14	520+ (1200x5)+ 520	20 | 1200x5 | 20 ; 500 (2100) (2100) 500	211200	2552.77
55a	12x4 + 18x4	12 / 43	170	79 ; 12 ; 79	20400	181.11
70	11 x 2	22	600	600	13200	393.89
71	22 x 2	22	300	300	13200	393.89

Fig.7.4–14 Prestressed slab reinforcement table

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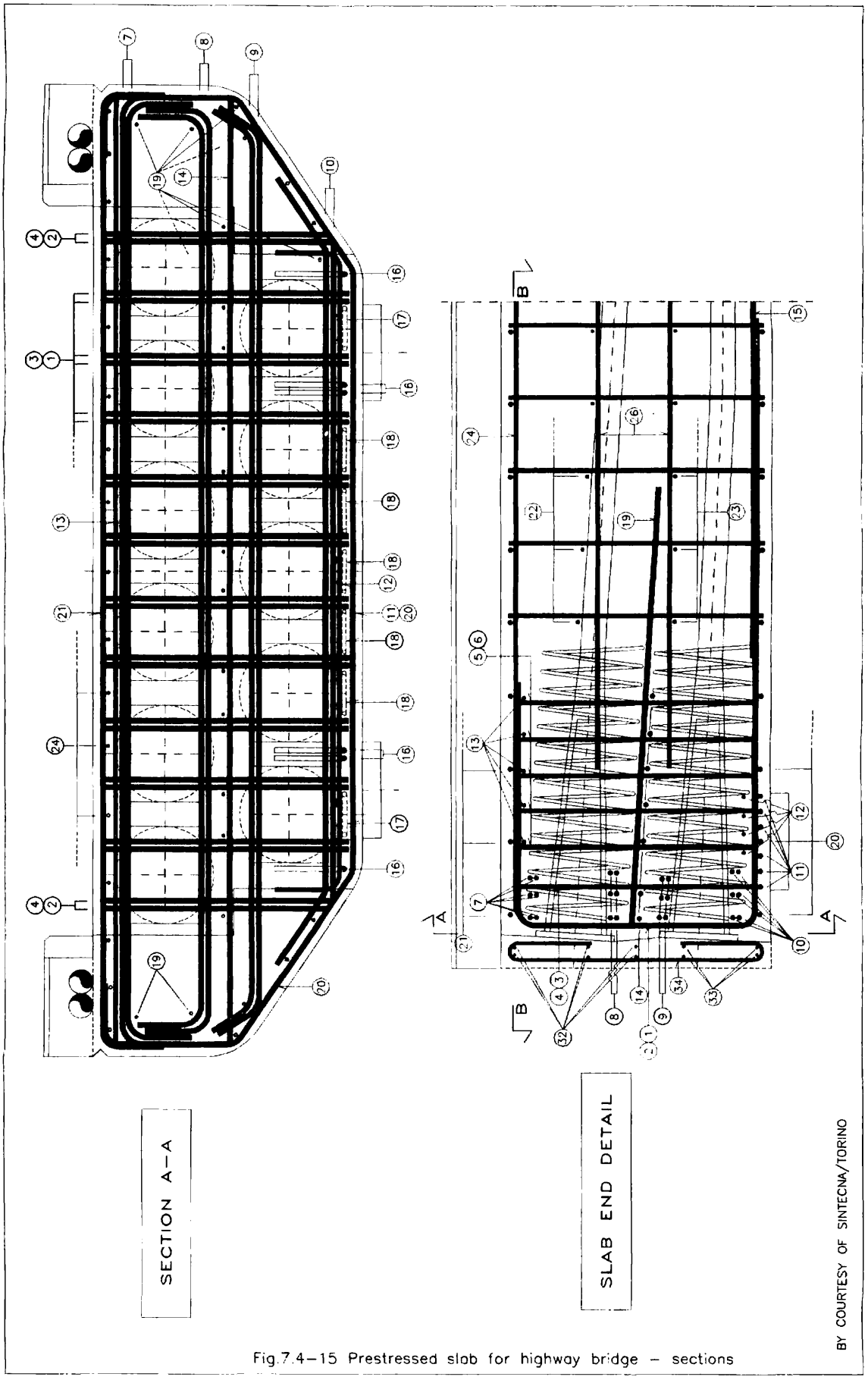
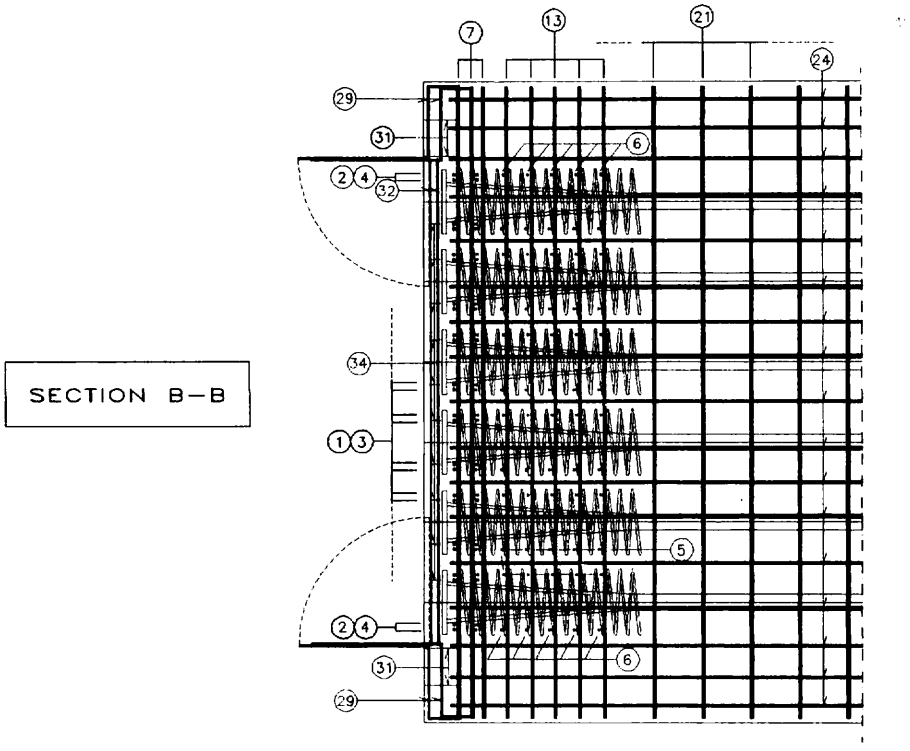


Fig.7.4–15 Prestressed slab for highway bridge – sections

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1	20	12.5 100 82 250 12.5	457	20 x 2	451
2	20	12.5 100 70 250 12.5	295	4 x 2	58
3	16	6 40 90 40 6	182	20 x 2	115
4	16	6 40 78 40 6	170	4 x 2	22
5	16 / 15	6 40 90 40 6	182	5x 10x 2	287
6	16	6 40 78 40 6	170	5x 2x 2	54
7	20	12.5 373 12.5 25 367 25 12.5 12.5 18 18	448 428	6 6	130
8	20	18 12.5 12.5 18 25 357 25 12.5 352 12.5	418 423	6 6	124
9	20	16 10 10 16 18 348 18 10 350 10	400 402	6 6	119
10	20	35 5 5 35 35 261 35 5 269 5	341 349	6 6	102
11	20	35 25 5 240 5	300	6 + 6	89
12	16	20 20 6 250 6	302	4 x 2	38
13	16	6 384 6 20 20	436	5 x 2	69
14	20	16 16 12.5 372 12.5	429	7 x 2	148
15	20	1200 600 1200 alternate arrangement	1200+ 1200+ 600	21	1554

POS.	ø	SHAPE	LENG. (cm)	N.	WEIG. (Kg)
16	20	30 12.5 250	292.5	6 x 2	87
17	20	25 250	525	2 x 2	52
18	16	12 250	512	5 x 2	81
19	16	20 180	200	26	82
20	20 / 30	12.5 25 25 12.5 47 47 6 80 80 6 5 242 5	593	97	1419
21	20 / 30	12.5 376 12.5 25 25	451	97	1079
22	16 / 30	6 384 6 14 14	424	89	596
23	16 / 30	14 14 E E 350	390	89	548
24	16	6 1180 1200 760 6 14 110 110 14 alternate arrangement	1200+ 1200+ 780	17	853
26	16	1200 1200 600 110 alternate arrangement	1200+ 1200+ 600	12	568
29	12	20 45 80	145	16 x 2	41
31	12	20 69 84 20	109 124	4 4	8
32	12	304	304	6 x 2	32
33	12	255	255	4 x 2	18
34	12	6 104 30 30	176	10 x 2	31

Fig.7.4–16 Prestressed slab for highway bridge – section and reinforcement table

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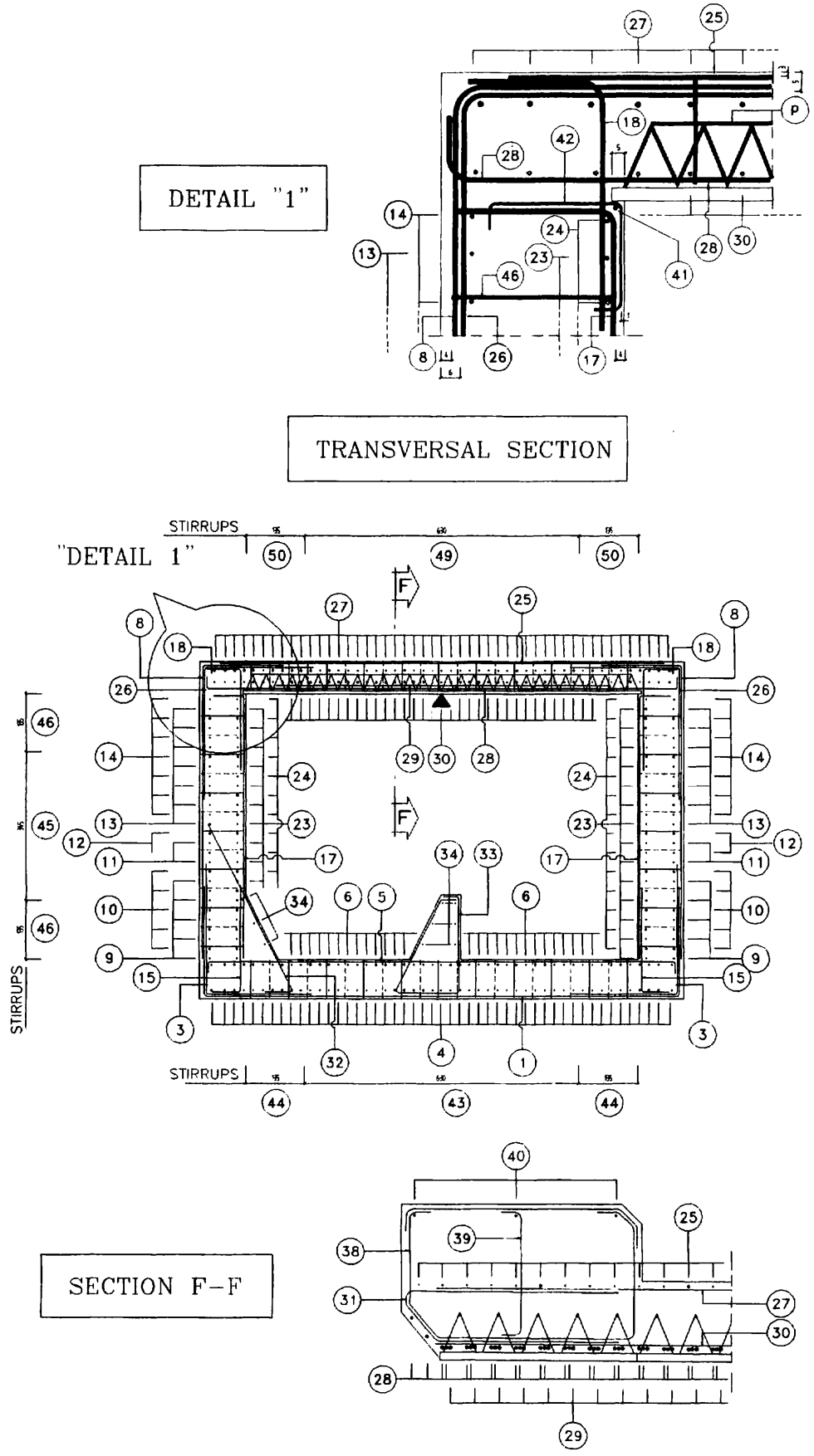
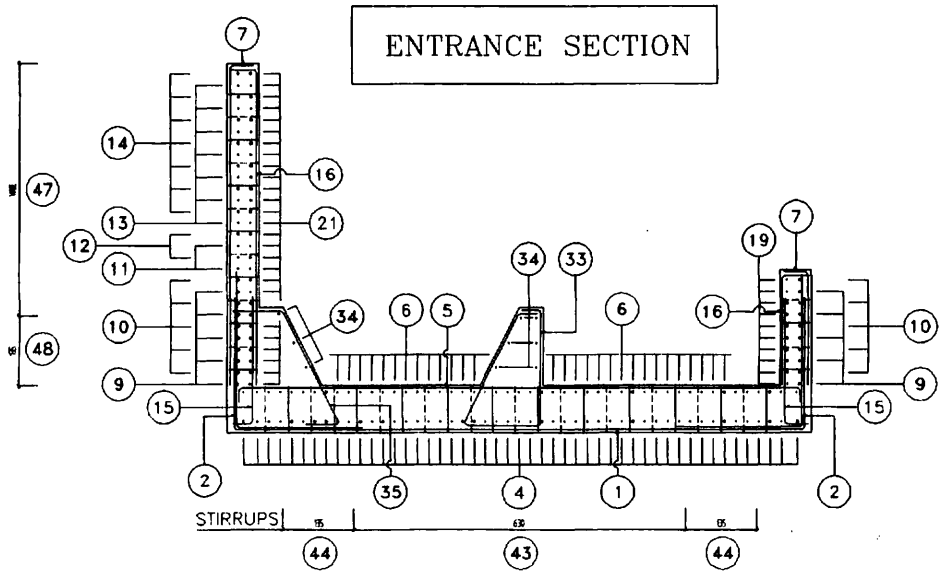


Fig. 7.4–17 Box culvert sections

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POS.	N.	Ø / DIST	LENG. (cm)	SHAPE	L.TOT. (cm)	WEIG. (Kg)
1	138	14 / 15	1200	168 | 864 | 168	165600	2001
2	35+2 + 33+2	20 / 15	400 + 400	200 | 200 | 200	108800	2683
3	70 + 2	20 / 15	400	200 | 200	56000	138104
4	49	14 / 15	1200 + 1200 + 400	50 alternate arrangement 50 | 1150 | 350 | 1200	137200	165794
5	129	18 / 15	900	864 | 18 | 18	116100	2319.19
6	49	14 / 15	1200 + 1200 + 400	1200 | 1150 | 350 | 50 alternate arrangement 50	137200	165794
7	135+39 x 2	14 / 17.7	150 + 530	30 | 500 | 150	48280	583
8	70 + 2	14 / 17.7	600	165 | 435	84000	1015
9	5 + 2	14 / 30	1200 + 1200 + 400	30 | 1170 | 370 | 30 | 135.6g | 1200 | 64.4g	28000	338.36
10	5 + 2	14 / 30	1200 + 1200 + 400	30 | 135.6g | 1200 | 64.4g | 30 | 370 | 1170	28000	338.36
11	2 + 2	14 / 30	1200 + 1200 + 185+330	30 | 135.6g | 1200 | 64.4g | 30 | 155-300 | 1170	10630	128.45
12	2 + 2	14 / 30	1200 + 1200 + 185+330	30 | 1170 | 155-300 | 30 | 135.6g | 1200 | 64.4g	10630	128.45
13	7 + 2	14 / 30	1200 + 565-1185	64.4g | 30 | 30 | 1170 | 135.6g | 535-1185	29050	351.04
14	7 + 2	14 / 30	1200 + 565-1185	135.6g | 30 | 1170 | 30 | 535-1185 | 64.4g	29050	351.04
15	138 x 2	14 / 17.7	200	30 | 170	55200	667
16	135+39 x 2	12 / 17.7	165 + 530	30 | 500 | 195	47260	420
17	70 + 2	14 / 17.7	450	44 | 406	63000	761
18	70 + 2	14 / 17.7	200	50 | 150	28000	338
19	28	12 / 15	235 + 690	205-660 | 30 | 64.4g	12950	114.97
21	28	12 / 15	205 + 690	135.6g | 30 | 205-660	12530	111.24
23	14 + 14	14 / 30	1200 + 240	190 | 50 | 64.4g | 1150 | 135.6g | 50	40320	487.23
24	14 + 14	14 / 30	1200 + 240	190 | 50 | 64.4g | 1150 | 135.6g | 50	40320	487.23
25	69	14 / 15	800	800	55200	667.04
26	69 x 2	26 / 15	400 + 400	200 | 200 | 200 | 200	110400	460.24
27	48	14 / 15	1200	1200	57600	696.05
28	69	20 / 15	900 + 900	18 | 18 | 864 | 18 | 18 | 864	124200	3062.96
29	69	22 / 15	600	600	41400	1235.39
30	48	14 / 15	1200	1200	57600	696.05
31	48 x 2	14 / 15	300	570 | 130 | 45 | 143g | 125	28800	348.02
32	35	14 / 30	300	250 | 50 | 66g	10500	126.88
33	69	14 / 30	500	176 | 26 | 66g | 120 | 152	34500	416.9
34	3 x 3	12	1200 + 1200 + 400	30 | 1170 | 370 | 30 | 135.6g | 1200 | 64.4g	25200	223.73
35	18 + 18	14 / 30	300	60 | 170 | 70 | 66g	10200	123.26
38	24 x 2	14 / 30	180 + 300	180 | 15 | 18 | 16.8 | 15.7 | 17 | 70 | 49-55 | 30 | 120 | 59-65 | 76	23040	278.42
39	24 x 2	14 / 30	100	15-12 | 70 | 15-12 | 76	4800	58
40	3 x 2	12	900	18 | 864 | 18	5400	47.94
41	2	16	1200	10 | 143g | 143g | 10 | 1180	2400	37.88
42	69 x 2	8 / 15	100	40 | 10 | 40 | 10	13800	54.45
43	756 + 181	12 / 30x45	80	14 | 52 | 14	74960	665.51
44	376	16 / 30x45	80	14 | 52 | 14	30080	474.76
45	118	12 / 30x45	100	62 | 19 | 19	11800	104.76
46	164	16 / 30x45	100	62 | 19 | 19	16400	258.95
47	119	12 / 30x45	60	32 | 14 | 14	7140	63.39
48	76	16 / 30x45	60	32 | 14 | 14	4560	71.97
49	293	12 / 30x45	70	15 | 40 | 15	20510	182.09
50	126	16 / 30x45	70	15 | 40 | 15	8820	139.21

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Fig.7.4-18 Box culvert section and reinforcement table

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8 Assessment, maintenance and repair

by Steen Rostam

8.1 Introduction

Ageing and degeneration is a natural and unavoidable process for all building materials. The main problem is not ***that*** they degenerate but ***how*** they degenerate and at what ***rate***.

The ideal service life situation occurs when ageing is so slow, that the structure maintains satisfactory serviceability for the anticipated service life without unexpected high costs for maintenance.

In Chapter 5 it has been highlighted that the quality and the performance of concrete is not known but assumed at the design stage and that the true quality and performance characteristics are determined through the actual execution process. This is a unique situation for concrete structures compared to structures made from other building materials and emphasises the special needs to verify the actual quality achieved in the finished structure. This information shall then be part of an Operation and Maintenance Manual, which will constitute a "Birth Certificate" of the structure, as described in Chapter 5.9.8.

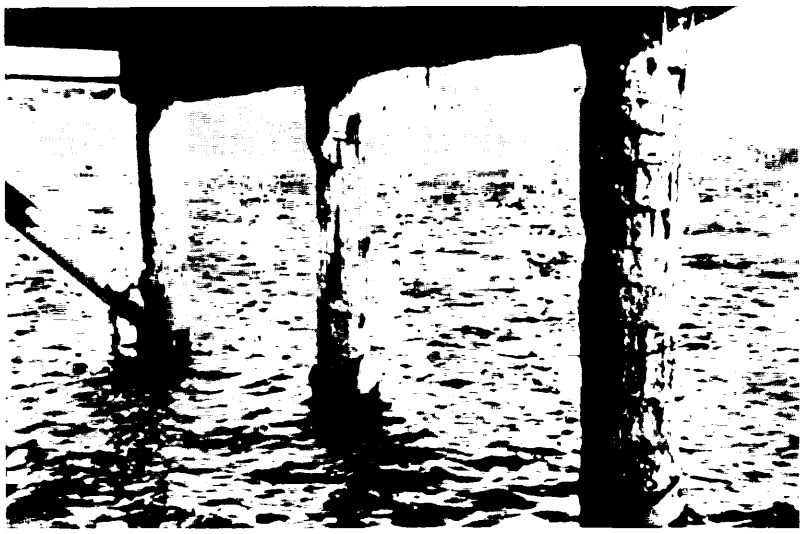


Figure 8.1-1: Marine structure. Synergy effect between chloride corrosion and freeze thaw action

Furthermore, it has been recognised that some environments are noticeably more aggressive than others, e.g. cyclic wetting and drying with chloride containing water at high temperatures being one of today's most severe environments for reinforced concrete. Similarly, marine structures exposed to freeze thaw action in addition to the saline environment are heavily exposed, see Figure 8.1-1. To cope with such conditions, care shall be taken during all phases of the design and execution of the structure to take advantage of available knowledge regarding exposure and deterioration of concrete structures. Already at these early stages the need for future inspection and maintenance shall be taken into account as a foreseen follow up on the information being part of the Birth Certificate. This means that structures shall be made accessible, inspectable and maintainable to the extent possible.

8.1.1 Scope

This Chapter on assessment, maintenance and repair aims at providing the reader with an understanding of the essential procedures of assessment of the safety, performance and remaining service life of existing structures.

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The main approaches to keep structures in satisfactory condition described in this Chapter require a basic understanding of the deterioration mechanisms. The measures needed and the costs depend on how the wear, ageing and deterioration processes have been foreseen by the owner. If a regular or irregular inspection is made based on a correct understanding of how concrete structures degrade, then foreseen and budgeted maintenance can keep the structure in satisfactory condition for very long, even in very aggressive environments. If ageing develops faster than anticipated the structure may need repair or upgrading at certain intervals to fulfil the service life expectations

Costs for such repairs - in some literature termed "corrective maintenance" - are usually unexpected and particularly inconvenient because they have not been foreseen in the planned operation and maintenance costs for the structure.

8.2 Definitions

The main tasks associated with the upkeep of structures are defined below, following the terminology presented in CEB Bulletin No. 243 (1998) and in part included in Figure 8.5-1:

- *Assessment:* The process of gathering and evaluating information about the form and current condition of a structure or its components, its service environment and general circumstances, whereby its adequacy for future service may be established against specified performance requirements, loadings, durability or other criteria.
- *Damage:* Physical disruption or change in the condition of a structures or its components, brought about by external actions and influences, such that some aspect of either the current or future functionality of the structures or its components are impaired.
- *Degradation:* A worsening of condition with time, usually resulting in damage.
- *Deterioration:* A worsening of condition with time, which may result in a progressive reduction in the ability of the structure or its components to perform some aspects of their intended function.
- *Evaluation:* As *assessment*, but may be applied more specifically in respect of suitability against a particular criterion such as specified loading. The term *assessment* is often used more commonly in connection with damaged or deteriorated structures.
- *Routine inspection (superficial inspection):* An examination, often at close range, of a structures or its components with the objective of gathering information about its form, current condition, service environment and general circumstances. It may include visual observations and simple testing.
- *In depth inspection (special investigations):* The process of inquiry into the cause and mechanism associated with some form of deterioration or degradation of a structures and the evaluation of its significance in term of its current and future functionality. The term may also be employed during the assessment of defects and deficiencies. The process of inquiry might employ sampling, testing and various other means of gathering information about the structure, as well as theoretical studies to evaluate the importance of the findings in terms of the functionality of the structure.
- *Maintenance:* A periodic or a-periodic activity intended to either prevent or correct the effects of minor deterioration, degradation or mechanical wear of the structure or its components in order to keep their future functionality at the level anticipated by the designer.
- *Repair:* Action taken to reinstate to an acceptable level the current functionality of a structure or its components which are either defective, deteriorated, degraded or damaged in some way and without restriction upon the materials or methods employed. The action

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may not be intended to bring the structure or its components so treated back to its original level of functionality or durability. The work may sometimes be intended simply to reduce the rate of deterioration or degradation, without significantly enhance the current level of functionality.

- *Rehabilitation:* Bringing the structure back to its original or higher level of function including durability and strength.
- *Retrofitting:* Action to modify the functionality or form of a structure or its components and to improve future performance. It relates particularly to the strengthening of structures against seismic loadings as a means of minimising damage during specified earthquake events or to increase load carrying capacity.

It follows from the definitions that **assessment** and ***maintenance*** are the actions known to be needed to keep a structure in satisfactory operating conditions within foreseen and budgeted expenses, usually based on some regular intervention. ***Repair*** and ***rehabilitation*** are the actions used if unforeseen and unbudgeted interventions are needed.

8.3 Requirements

A structure shall fulfil a number of requirements throughout its service life [Markeset and Rostam (1999)]. These requirements may differ with time, but their fulfilment determines the types of interventions needed. The main requirements are to satisfy the expectations from the following two main parties:

1. Requirements of the owner.
2. Requirements of society.

While maintaining the structure and its components, or when performing repairs these requirements shall be fulfilled. Therefore, these requirements shall be clarified prior to performing an assessment. In fact, the need for a sufficient clarification of the basic requirements prior to planning and performing an assessment is an often-overlooked or insufficiently fulfilled task. This may lead to insufficient, inadequate or even erroneous assessments, and to later conflicts between what has been achieved through the assessment and what was expected by the owner. Therefore, the following shall also be taken into account during the assessment process:

3. Requirements to the assessment, maintenance and repair process.
4. Responsibilities.

8.3.1 Requirements of the owner

Next to the concern of human life and welfare, the economic consequences of maintenance, repair or replacement is vital for the owners decision making. Thorough economic analyses must therefore be performed to evaluate the consequences of alternative decisions.

The economic evaluation should not be restricted to the damaged structure only. It should also cover a long-term plan for possibly replacing it with new facilities. In such cases, a close co-operation between the owner and the assessment engineer may allow the combined short and long term technical and economical consequences of different scenarios to be analysed in detail.

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In any case it should be realised that a problem of structural deterioration usually has several acceptable solutions. The owner shall decide the best-suited solution in each case. In his decision making, the owner has to take into account:

- Technical and functional aspects
- Economical consequences
- Social/political/legal requirements.

Such decision making depends on the overall long-term situation of the owner, and two different typical situations may prevail [Rostam (1991)].

One situation is that a structure is owned and operated by *one owner from cradle to grave*, i.e. the ***single owner*** situation. This owner may typically be the state, the community or public services such as power and water authorities, port authorities, telephone companies, etc.

In the case of a "single owner", designs as well as repairs may be governed by a rational long term service life approach in which the design, the construction and the maintenance may be considered in combination to ensure the required target service life of the structure. This allows a life cycle cost optimisation to govern the technical and economical decisions when choosing between alternative designs and repair schemes.

The other situation is that a structure is owned and operated by multiple successive private owners, each with a short economic horizon. This may be the case for ordinary dwellings, offices, and some factories, etc.

In the case of *multiple successive owners*, i.e. the ***multiple owner*** situation, long term maintenance cannot be relied upon. Robust initial designs is therefore the only way to ensure overall safety for an acceptable period of time, although this may be a more costly procedure. If codes or standards also covered maintenance and repair strategies, this would allow a more economic life cycle cost optimisation also in the multiple owner case.

8.3.2 Requirements of society

Society has a number of primary requirements, which must be fulfilled by its structures, regardless of whether they are publicly or privately owned. These requirements are the following:

- Maintain a minimum level of safety against failure of the structure or its components (global hazards), as well as maintaining minimum safety against hazards from falling debris (local hazards).
- Maintain uninterrupted the services to society provided by the structure. Taking infrastructures such as bridges, roads, tunnels, harbours etc. out of service temporarily due to needs for maintenance and repairs impose high costs on the users and thus on society.
- Ensure a minimum service life of privately constructed and owned structures. This is to avoid too short factual service lives where society then have to take over the responsibility of premature repair costs. This is reflected through codes and standards ensuring such minimum quality of the core structure. The aim is to avoid subjective short-sighted sub-optimisation by the private builders and owners.

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- Ensure limited health and environmental impacts of the structure both during construction and during subsequent use and maintenance of the structure.
- Satisfy minimum aesthetical demands of the structure and its impact on the environment.

8.3.3 Requirements to the assessment, maintenance and repair process

The final objective of the assessment and the subsequent maintenance or repair is to ensure that, at the end of the process, the structure performs satisfactorily and that its safety, functionality and foreseen remaining service life has been clarified sufficiently as expected by the owner.

In order to fulfil the requirements of the owner and society the following processes may be necessary, depending on the size and complexity of the problem:

1. Inspect, test and evaluate the causes of deterioration.
2. Assess the condition of the structure in relation to the requirements of the owner and society.
3. Evaluate alternative interventions to maintain or to ensure satisfactory short and long term safety and performance, and estimate the economic consequences of these alternatives.
4. Agree with the owner on the preferred intervention and develop the selected maintenance or repair project(s) and corresponding tender documents, taking adequate care of safety, health and environmental impacts.
5. Perform tender, tender evaluation and contract negotiations.
6. Execute the maintenance or repair project and perform adequate supervision
7. Make, as the individual case may warrant, a selective inspection and assessment of the condition of the maintained or repaired structure as an updating of the Operation and Maintenance Manual - or of the "Birth Certificate".
8. Have a quality assurance scheme adapted throughout to the assessment, maintenance and repair process, and apply relevant quality control of essential processes.

8.3.4 Responsibilities

The overall assessment, maintenance and repair process for concrete structures highlights the responsibilities held by the different parties involved:

- The owner must state his requirements to the structure after intervention, such as
  - the required remaining service life and the corresponding performance criteria
  - the need for upgrading of the structure if an improvement or a modernisation should be coupled to the intervention found necessary during the inspection and assessment process
  - the wish for an improved aesthetical appearance
- The engineer must assess the quality, the present condition and the estimated remaining service life of the structure if left without maintenance or repair and assuming unchanged or a defined future use of the structure
- The engineer must evaluate the ability of alternative interventions (maintenance procedures or repair methods) to provide the required short and long term performance. He must estimate the initial costs, the time dependent costs and the overall service life

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costs, based on the specific conditions of the owner (public or private), of the possible alternative solutions. He must describe the reliability and specific consequences of each method regarding future requirement for maintenance and re-repair.
- The owner must select his preferred solution.

These roles and the corresponding responsibilities of the different parties involved are essential. Although the engineer of course shall make his recommendations taking the overall scene into consideration to the extent he can, the owner shall take the final decisions. He shall require the quality he wants, he shall ensure that he gets what he has required, and he shall pay for costs of achieving this quality; see also Chapter 5.4.1.

8.4 Maintenance and repair categories

All experience show that a planned and budgeted regular upkeep of a structure leads to the lowest costs for operating a structure, provided the correct assessment and the correct interventions are made at the correct time. This latter condition is the most difficult part of the process.

Firstly, the natural degradation of structures will require an assessment of the structure performed at regular intervals if the structure shall remain undamaged. This assessment must be based on some type of regular inspection. It must reveal which mechanisms are threatening the structure and identify which parameters are governing the type and rate of deterioration.

Secondly, a long term budgeting of the costs for regular inspections and possible foreseen interventions must be made. This is the only means by which the funds needed for the inspections, assessments and interventions can be ensured on a regular basis.

As presented in Chapter 5 the life of a concrete structure is divided into two distinctly different periods:- The initiation period during which deterioration is not taking place, or only very local or superficial concrete deterioration has occurred. Regular, planned maintenance activities ought to take place during this period.

- The propagation period during which deterioration takes place. The structure will be in need of repair works which may be planned or unplanned. The owner may also choose to omit repair works and use the structure until surveillance shows non-fulfilment of structural, aesthetic or other requirements (i.e. until the accept limit is passed).

In order to keep an existing structure in satisfactory service condition with maintained local and global safety a number of interventions may become necessary as time passes. These interventions can be grouped into three different categories covering a mixture of both maintenance and repairs activities, as exemplified below:

- Non-repair
- Structural repairs
  - replace concrete
  - replace reinforcement and recast concrete
  - replace or strengthening elements
- Materials protection
  - surface protection
  - change environment

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- corrosion protection (coating of reinforcement, inhibitors, cathodic prevention)
- cathodic protection
- chloride extraction, re-alkalisation
- replace corroding reinforcement with non-corroding reinforcement

Maintenance and repair procedures shall be selected based on a clear understanding of how they will influence the ageing and the deterioration mechanisms favourably. This means that the intervention shall target the parameters identified during the assessment process to be governing these mechanisms, and modify them in a durability enhancing way.

Hence, a successful repair is much dependent on the technical competence and practical experience of the assessment engineer, see Chapter 8.5.4.

8.4.1 The "Law of fives"

Most of the problems related to maintenance and repair are problems of economy and user inconvenience.

An experienced contractor [de Sitter (1983)] has ventured into a rough estimate of what the true relative costs would be to ensure the required long service life of a concrete structure, depending on which approach was chosen to ensure this service life.

The simple curve in Figure 5.8-2 representing the two-phase service life modelling of concrete structures is valuable when illustrating these alternative approaches, see Figure 8.4-1.

The service life can be divided into four different phases according to de Sitter (1983):

- Phase A: Design-, construction- and curing period
- Phase B: Initiation processes under way, no propagation of damage has begun yet
- Phase C: Propagating deterioration has just begun
- Phase D: Advanced state of propagation with extended damage occurring

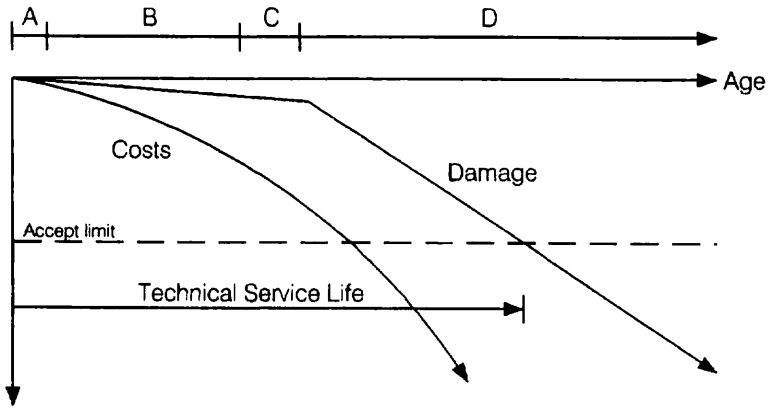


Figure 8.4-1: The "Law of Fives" representing the costs of alternative approaches to ensure the required service life of concrete structures

These Phases and the estimated relative costs are illustrated in Figure 8.4-1.

Based on yearlong experience the "Law of Fives" was introduced, stating that:

"One dollar spent in Phase A equals five dollars in Phase B equals twenty-five dollars in Phase C equals one-hundred-and twenty-five dollars in Phase D".

This means that the same lifetime may be achieved by a little extra attention to "good engineering practice" during Phase A, compared to the very costly approach of relying on future

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maintenance, repair or rehabilitation to provide the required service life. Much money may be saved in this way which can be used for better purposes.

From the above follows that interventions during the initiation period, so-called preventive maintenance, generally is much less costly than delaying the intervention until the deterioration has developed to a stage where major repair or rehabilitation becomes necessary. However, the timing of such interventions is also a factor strongly influenced by economy and non-technical issues such as reputation, aesthetics, expectations by the users and by society, and politics.

8.5 Assessment

The intention of the assessment shall be to evaluate the real condition of an existing structure, generally to ensure that there is an adequate factor of structural safety in respect of current or potential loadings and an adequate factor of user safety in respect of functionality and falling debris. Also, there is usually a requirement to make a prognosis about the future durability and performance of the structure, particularly where it is suspected or known to be experiencing structural deterioration.

Suitability of a structure for its current or proposed function can be assessed under various headings. In addition to the more commonly addressed issues of strength, overall stability and robustness; consideration might also be given to a broad range of additional factors including stiffness, fire resistance, dynamic behaviour, long term deformations, durability as well as appearance.

There are many reasons for undertaking an assessment of an existing structure. An owner or user will request a technical specialist to make an assessment for the for reasons such as:

a) If required for a specific structure or group of structures or their components,

- in case that the reliability of the structure is jeopardised because of concrete and reinforcement deterioration,
- when additional loads should be carried by the structure,
- to gather necessary data for the design of a repair or an upgrading.

b) On a regular basis, as generally carried out for larger stocks of structures of large numbers of components, e.g. for road and railway bridges,

- to ensure a decision basis allowing to preserve the safety and serviceability or functionality for normal traffic conditions or in case of special transports, of the whole bridge stock in an administrative region,
- to create a database of updated information about the condition of every bridge structure on the road network as an analytical tool for taking decisions about necessary maintenance measures,
- to establish priorities for the repair or replacement of heavily deteriorated structures.

The assessment is a complex interaction between:

- structural, environmental and service data,

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- data from visual inspection,
- test data from in situ and laboratory investigations.

From the above follows that an assessment shall provide the decision basis for selecting the appropriate maintenance, repair or rehabilitation needed to ensure the necessary safety and performance for the required pre-determined remaining service life.

However, it should also be recognised that unforeseen damage or weakening often is revealed during the inspections. Following a revised agreement with the owner a shift in scope may then become necessary. Therefore, a close contact with the owner is important throughout the whole assessment, maintenance and repair process.

8.5.1 Answers required

The need for an assessment of existing concrete structures appears when the structure in question:

- has deteriorated through the effects of the external environment and loads.
- has suffered damage from external environmental impacts such as seismic actions, floods, storms etc., or from specific incidents such as overloading, vehicle impact, fires or explosions, in short the effects of specific events for which the design has not catered or has not catered sufficiently.
- has suffered damage from internal deteriorating effects.
- is being used or is planned to be used for purposes significantly different from the original, such as the applications of greater live loads.
- is under consideration for redesign on structural alterations.
- has been exposed to a number of the above detrimental effects and is under consideration for major changes in purpose and/or structural alterations.

The owner of a deteriorating or damaged structure must consider several questions:

- Is it safe to use the structure, or what safety measures must be taken?
- How widespread is the problem and at what pace will it develop further?
- What are the economic consequences?
- What is the cause of the problem?
- What is the remaining service life?
- Which feasible repair strategies should be considered?
- What will be the influence of the repairs on the remaining service life of the structure?
- Can a change in operation and maintenance procedures improve the situation?
- May similar problems arise for other structures if not attended in time?
- Should general preventive measures be taken?

The first steps towards answering the above questions will be an inspection and a technical appraisal of the damaged or deteriorated structure.

The use of codes and standards aimed towards the design of new structures when assessing existing structures poses, however, a number of questions and, perhaps, objections:

- When updating the load carrying capacity of a structure the first question is: Should the analysis be based on present codes for new structures or should it be based on codes governing at the time of the original design of the structure?

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- Is a more extensive use of complex mathematical modelling of the structural interpretation allowable or are the otherwise hidden reserves of resistance in full or in part accounted for in the codes chosen?
- There is, for the structural engineer, a significant difference in the nature and extent of available knowledge about actual resistance parameters, loadings and the structural configuration when assessing or updating an existing structure compared to the similar situation during the design of a new structure.
- The engineer can in principle collect any amount of factual information he may consider important to his analysis, when he approaches an updating of an existing structure, whereas he only can make assumptions and specify, not verify, similar preconditions during the design phase of a new structure.

8.5.2 Decision making

A rational decision support diagram for the inspection and assessing engineer is presented in Figure 8.5-1 [CEB Bulletin 243 (1998)].

It is obvious that each case of diagnosis and assessment may be unique and that the exact formulation of answers required may be equally unique. Nevertheless, there is a stringent pattern for most cases, such as illustrated in Figure 8.5-1, which enables the competent structural engineer to formulate his requirements and decide on the assessment, maintenance and repair process, see Chapter 8.3.3.

The analysis of the structural capability of an existing concrete structure ideally requires not only the usual information pertaining to such analysis, but also comprehensive inspection data that account for the condition and preservation of each structural element. Deteriorated or otherwise damaged elements must be diagnosed and their residual resistances and structural characteristics evaluated before the structural analysis can be performed. Relevant load cases in the analysis may then describe the present structural performance capability. To verify the analysis (or vice versa) load tests are occasionally performed on the structure.

Having analysed the present capabilities of a concrete structure the next question will often be: for how long will the structure maintain its capabilities?

In order to obtain the right answers to the right questions at the right time the decision diagram presented in Figure 8.5-1 can provide valuable support to both the newcomers and the experienced personnel in inspection and assessment engineering.

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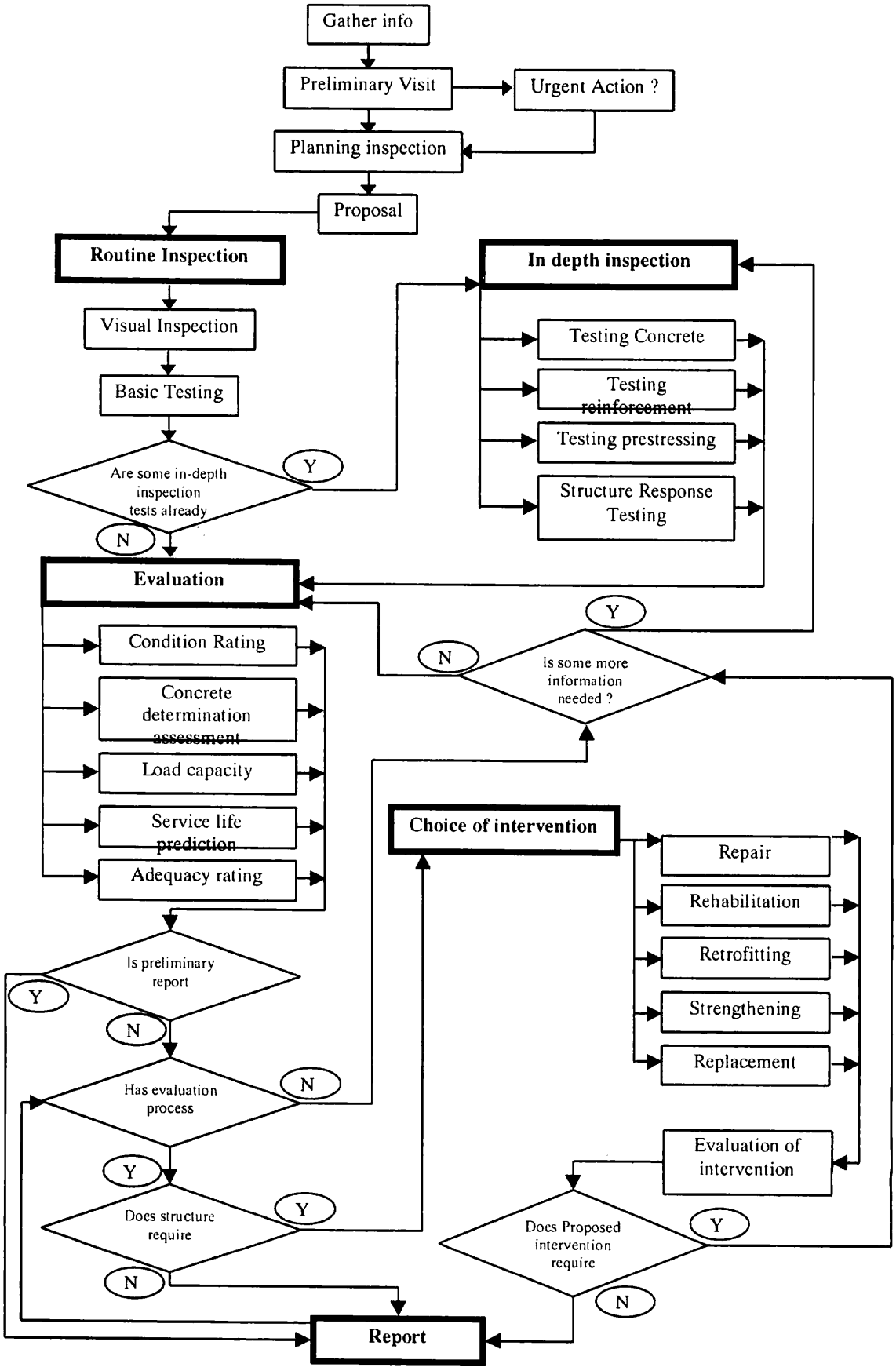


Figure 8.5-1: Decision diagram for a rational inspection and assessment strategy [CEB Bulletin 243 (1998)]

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8.5.3 Methods of assessment

Details of the procedures of investigation of the structure will depend on the specific requirements of the owner in addition to the technical needs for undertaking the work. However, whatever the detailed requirements the overall assessment procedure can follow the stages indicated in Figure 8.5-1.

Depending on the level at which inspections and assessment is being carried out, and on the property to be assessed, examples of the findings and how they may influence the performance characteristics of the structure are listed in Table 8.5-1.

Table 8.5-1: Example of findings and how they may influence the performance characteristics of the structure [CEB Bulletin 192 (1989)]

Level of Investigation	Characteristics		
	Load Capacity	Serviceability	Durability
Visual Inspection with Basic Testing	Cracks, deformations, buckling	Cracks, deflections, movements, movement ability	Cracks, honeycombing, damp patches, stalactite formation
	Strength, ductility, fatigue strength, corrosion	Abrasion, discoloration, static and sliding friction coefficients (e.g. for bearings)	Corrosion, carbonation, chloride content, frost resistance
In Depth Inspection	Failure of prestressing cables, large residual deformations	Deflection, vibrations	Forms of corrosion, vibration frequencies

(1) Visual inspection

The classical strategy of performing inspection and assessment has been based on the findings from visual inspections. With present day knowledge of the time dependency of the deterioration processes, the consequences of basing maintenance on the visual observations alone may be very costly for the owner.

Referring to Figure 5.3-2 it is obvious that during the initiation phase there will be neither visual damage nor any other signs of beginning deterioration of the structure. Only some way into the propagation phase will damage become visible and thus recorded through a visual inspection. This has been indicated on Figure 8.5-2, and the economic consequences of basing a maintenance strategy solely on such a visual inspection might be serious, as highlighted in Chapter 8.4.1 and illustrated in Figure 8.4-1.

By supplementing the visual inspection with selective testing the information on which decisions shall be taken become much more valuable as the technical and economic consequences of alternative solutions can be defined in detail also while the structure is still in the initiation phase. This allows the owner to select preventive maintenance at an early stage prior to the occurrence of active deterioration if he decides that this would be the overall optimal solution.

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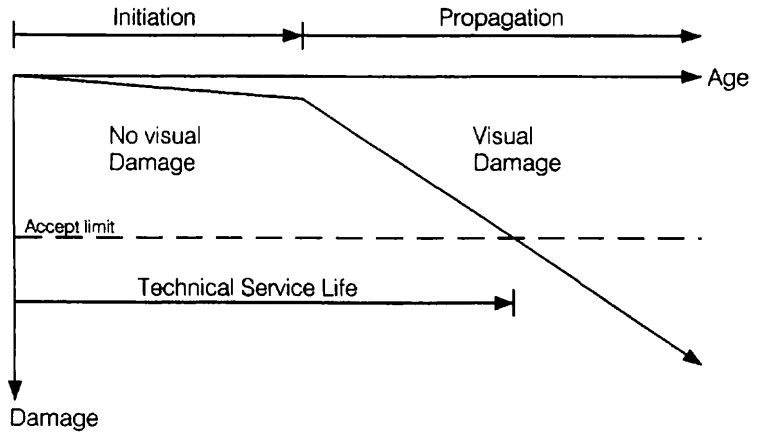


Figure 8.5-2: Visual inspection can only detect ongoing deterioration well into the propagation phase. The economic consequences could become serious, see Chapter 8.4.1.

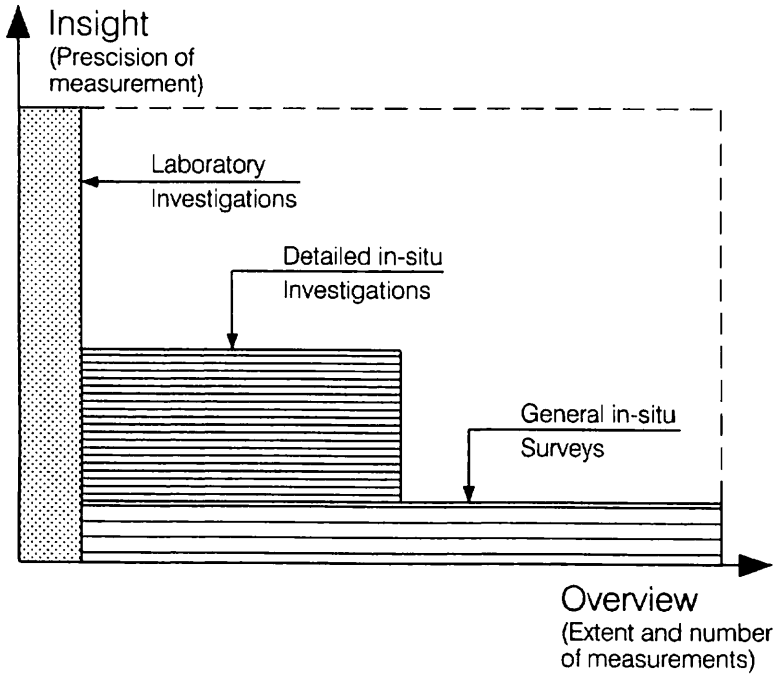


Figure 8.5-3: Type of information from different types of investigation and testing

(2) Selection of inspection and testing methods

The different inspection and test methods have each their characteristic merits. It is essential to know which information can be gathered from each of these methods. In this connection it can be valuable to relate different methods to the precision of the testing to give deep local insight level and the degree of overview of the condition of the structure which can be expected from each method. This has been illustrated in Figure 8.5-3.

The first stage will usually be a general in-situ survey aiming at a quick and rough assessment of the main problems, including an evaluation of possible imminent safety risks of the structure and hazards of the users and passers-by requiring immediate safety precautions. Visual inspection and simple in-situ testing constitute the main means of inspection at this stage.

This first stage survey constitutes also the basis for a rational and systematic planning of the scope and detailing needed for the subsequent inspections and testing. This follows the overall inspection and assessment strategy as presented in Chapter 8.5.2 and illustrated in Figure 8.5-1.

(3) Assessment based on random testing

One approach to inspection and testing is the random testing. By this method the types of relevant tests have been decided, partly through the first rough visual inspection combined with the experience of the assessment engineer, partly through some selected verifying local tests. The random test approach covering selected proportion of the structure or its compo-

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nents is then tested using the chosen test(s). The data is then treated statistically in order to determine the most probable cause of deterioration and - in particular - to determine the extent of damage. This allows then a determination of the size of the problem and thereby allowing the alternative repair methods to be budgeted.

The random approach to determine the magnitude of the maintenance or repair scheme is particularly valid for a large number of similar structures or structural components like exposed columns, parking decks, facade elements, balconies, piles or piers. This approach is not optimal for single or a few structures or structural components. In such cases the assessment based on the selective approach described below will be optimal.

(4) Assessment based on selective inspection and testing

The most direct approach to assessment is the selective approach [Rostam (1991)]. This requires a minimum level of knowledge and experience with structural and materials engineering, structural design, construction, environmental aggressivity, deterioration mechanisms and modelling, and repair materials and techniques. Altogether it requires a broad knowledge of all aspects of "durability technology". The approach is the following:

1) Based on an initial inspection and the personal experience of the assessment engineer the most probable cause(s) of deterioration is assumed.
2) Selective tests are chosen to verify the assumption(s) made or to modify the assumption(s) until sufficient certainty regarding the cause of deterioration and the deterioration mechanism(s) have been obtained.
3) In the case of multiple similarly affected structures or components a random testing of a chosen percentage of the members is made to estimate the extent of deterioration.
4) Alternative maintenance or repair methods are determined and their technical and short as well as long term consequences are evaluated in order for the owner to make his choice of method.

As can be seen, this approach aims directly at the goal up front. It combines the clearly insufficient knowledge, which can be gained through a first rough inspection based mainly - but not necessarily solely - on a visual inspection, with the knowledge and experience with durability technology of the individual assessment engineer.

The axiom of condition testing quoted above is adhered to, and only the types and number of tests directly needed to verify or to modify the initial assumption, as the case may be, are made. This is the quickest and usually the most cost optimal approach to the condition assessment of concrete structures.

However, this approach requires that the expert is the person who physically is on site and makes the initial inspection and determines which tests are needed and where to make the tests.

In some cases good results have been experienced with a competent and well-instructed engineer making the site investigation using a video camera. The video shall be used to give both clear overviews and very close-up live pictures of the observations and of the condition of the structure. Later, a - maybe remotely placed - expert evaluates the observations made through the video.

In all cases and a hammer is an indispensable tool when making inspections.

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(5) Testing methods

Testing methods and procedures for concrete structures have been listed and commented in CEB Bulletin 192 (1989) and CEB Bulletin 243 (1998). The different test methods have been grouped into the three groups below, and are listed in CEB Bulletin 192 (1989) to which reference is made:

- Structural tests, determining the strength of the structure or its components.
- Physical tests, determining non-destructively the different physical characteristics of the concrete and the possibility of having defects like voids or honeycombing in the concrete or insufficiently grouted cable ducts for prestressing tendons.
- Chemical and electro-chemical tests, determining the chemical composition and chemical changes of concrete, the water/cement ratio and the parameters governing the corrosion of reinforcement such as concrete resistivity and reinforcement potentials.

One specific situation where load testing of full-scale structures can have considerable benefits, both for the assessment of similar existing structures and for the design of new structures, is the load testing to destruction of structures which are to be demolished. This can yield extremely useful information about the true reserves of strength of complete full-scale structures. Such tests can also demonstrate the safety levels of structures deteriorated beyond what can be modelled, e.g. where reinforcement is suffering extensive corrosion in the anchorage zone or extensive delamination of concrete cover has occurred, [Rostam (1991)]. These tests provide information about the ways in which the strength is enhanced by alternative load-paths or complex load-carrying mechanisms, which only become significant when the structure is approaching the Ultimate Limit State, and can illustrate the type of failure mechanism, whether it is warned or unwarned.

However, one main objective always to be kept in mind when selecting each test to be performed is the following:

Why is the test to be performed, for what use are the results, and how shall the results enhance the decision process towards identifying the most optimal maintenance or repair method.

In other words, do not make tests or record observations if it is not known beforehand how this information shall be used in the assessment process. By adhering to this axiom a stringent, targeted and cost optimal assessment process may be ensured.

8.5.4 Requirements to the assessing engineer

Assessing engineers have a very important task in reassessing decaying or functionally obsolete structures and preventing such structures from falling into disuse by making ***realistic assessments*** of the structural capacity and by preparing the proper background for possible measures to adapt the structure to present day needs and requirements.

The assessing engineer faces a number of unique and difficult technical problems, such as:

- Making a realistic evaluation of the present loadings and their possible development in the future.
- Detecting the possible internal and external causes of any unusual behaviour of the structure, i.e. excessive deformations or settlements, decay or deterioration.

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- Investigating the importance of specific structural details maintaining the degree of reliability and durability of the structure, such as prestressing , anchors, bearings, joints, etc.
- Evaluating the influence of cracks and possible concrete and reinforcement deterioration on the overall safety and performance of the structure.

There are a considerable number of test procedures that can be applied during the investigation. It is important to realise that all, or practically all, tests that can be applied, are indirect tests that require a professional interpretation by the assessing engineer before it is possible to make use of the test results.

The assessing engineer must therefore have a thorough understanding of all his test procedures and the reliability of the results obtained.

In order to perform relevant inspections and assessment to provide the basis for proposing alternative types of maintenance or repairs the assessing engineers must know what they shall be looking for. Due to the safety consequences to be considered, the assessment should be performed by an engineer with broad knowledge and experience within materials technology, deterioration mechanisms, structural behaviour and construction procedures. For example, experience shows that fine cracking, local leaching of lime, slight discolouring or hollow sound may be the sole warning against severe safety hazards for users and passers-by. These few and simple symptoms, however, may cover a variety of serious problems.

It follows that to assess an existing structure requires the highest degree of professional integrity, expertise and knowledge from the assessing engineer if the results of his assessment are to be of any real use to the owner and society.

It is also important to realise that there are legal aspects in the risky business of assessment that should be dealt with openly and professionally. It is obvious that an assessment engineer must be able to insure against his liability for making incorrect assessments. On the other hand this requires that the assessing engineer can prove he is professionally and adequately trained to undertake such an important task. It is not likely that guidelines or codes in assessment procedures will be really helpful in this respect, as they cannot substitute for professional expertise and integrity. What is needed is an understanding between the client and the assessment engineer with respect to a clear division of risks and responsibilities.

8.6 Maintenance

According to the definition of maintenance, Chapter 8.2, this is an action to cope with the effects of minor deterioration, degradation or mechanical wear of the structure or its components.

However, in practice it often turns out that the owner considers the separation between maintenance and repair to depend on whether the intervention has been foreseen or not foreseen within the long-term operation and maintenance budget. The costs for maintenance have thus in due time been included in the long-term budget and the timing has been adjusted to fit into the long term activity plans of the involved personnel. Repair is when the intervention has become necessary following a normal inspection and possibly special testing revealing the need for interventions within a short period of time and requiring costs outside the normally allocated funds.

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The basis for identifying possible maintenance strategies is an understanding of the main deterioration mechanisms and the governing parameters. Particularly, the parameters describing "penetrability" of liquids, salts and gases into the pore system of concrete are of primary importance.

In addition, the owner's requirements to the structure including service life, serviceability, load-bearing capacity, aesthetics, financial options etc. has to be clarified. This is needed in order to provide a comprehensive overview for the owner to take the decision as to which maintenance strategy he considers optimal from the infinite number of combinations of maintenance and repair methods.

European Standards, EN 206 and EN 1504 - 1-10, are currently being developed for the protection and repair of damage to reinforced concrete structures. A comprehensive package of standards, ranging from test methods for specific properties, through specifications for the key repair materials, including coatings, mortars, bonding agents and injection materials, to a statement of general principles for repair work is being written, [CEN TC 104 (1996)]. The principles and methods cover methods relevant both for maintenance and for repair of concrete structures i.e. related to both defects in concrete and to reinforcement corrosion. Reference is made to Table 8.7-1 and Table 8.7-2.

8.6.1 Scope of maintenance

All concrete structures need maintenance either to ensure that the planned initiation period is in fact obtained or to extend the initiation period beyond what was originally expected - explicitly or implicitly.

The initiation period may be extended by reducing the aggressivity of the exposure or by increasing the resistance of the concrete structure against the environment.

Maintenance refers to three different main activities:

- Cleaning, repair or replacement of non-structural components such as joints, drainage installations, furnishings and components like wearing courses suffering foreseen wear. This constitutes the dominating and most usual part of maintenance of concrete structures.
- Repair or replacement of structural components with foreseen shorter service life than assumed for the main structure, such as bearings and structural joints.
- Patching or re-doing of special physical protective measures such as paint, surface coatings, and membranes, or maintenance of the installation of special preventive measures such as cathodic prevention to avoid premature reinforcement corrosion due to chloride ingress. The physical measures protect the structure against ingress of water, aggressive liquids, salts, frost or other aggressive matters.

8.6.2 Preventive maintenance

Maintenance of the concrete structure introduced during the initiation period with the specific aim of preventing a foreseen later occurrence of major deterioration developing during the initiation phase is considered preventive maintenance. In short, preventive maintenance

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intends to delay the transition from initiation to propagation. The possible economic consequences can be evaluated based on the "Law of Fives", see Chapter 8.4.1.

(1) Coatings and surface protection

The reasons for applying surface protection to new concrete are specific demands for:

- Resistance to aggressive chemicals (e.g. in chimney, sewage treatment works, chemical plants etc.)
- Resistance to abrasion (e.g. industrial floors etc.)
- Decreased contamination of the environment/contents (e.g. dust from walls, ceilings and floors, tanks etc.)
- Cleaning conditions (e.g. food factories, hospitals etc.), or appearance (e.g. roof tiles, crash barriers etc.).

Concrete structures designed and built in accordance with the current codes of practice with regard to materials characteristics, architectural and structural design, processes of execution and inspection and maintenance procedures, normally do not deteriorate prematurely due to alkali silica reactions, freeze/thaw action or reinforcement corrosion.

However, some older as well as some new concrete structures show or will show decreased durability due to reasons such as insufficient cover on reinforcement, porous concrete, reactive aggregates or faulty design. Surface treatment may successfully be used to diminish the rate of degradation of these structures, by reducing the rate of penetration of water and aggressive substance into the concrete.

The inspection and assessment of the structure in question shall provide information being sufficient to ensure that the appropriate materials and processes are being applied as surface protection.

Coating properties, which must be taken into account when specifying a surface protection, are:

- Resistance to carbon dioxide diffusion
- Resistance to water vapour diffusion
- Resistance to water ingress
- Flexibility (bridge building effect)
- Resistance to abrasion
- Resistance to UV light and temperature
- Resistance to chemicals
- Easiness of application
- Easiness of maintenance
- Adhesion
- Service life
- Aesthetics
- Toxicity

The priority of these properties, among other things, depends on the relevant mechanisms of deterioration, the degree of deterioration and the environmental conditions.

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Surface treatment of concrete must as the first step include a thorough assessment of aspects such as:

- The condition of the concrete structures from a concrete technological view point, including the governing mechanisms of deterioration, if any
- The risk of deterioration mechanisms acting after a possible surface treatment, primarily due to moisture accumulation
- The environmental exposure
- The objectives of surface treatment.

After these aspects have been analysed, the concept for the optimal surface treatment is selected; taking into account that the use of uncomplicated systems will minimise the risk of errors.

Unless otherwise stated by the producer there are a few basic rules that should be followed to ensure the best quality of the final product:

- The tensile strength of the substrate and the subsequent layers shall be as recommended.
- The chalking of the substrate and the subsequent layers shall be insignificant.
- The moisture content of the substrate shall be below 4-5% measured as absolute value.
- The relative humidity in the substrate shall as a general rule be below 85%.
- The surface temperature shall be more than 3 degrees Celsius above the viewpoint of the air.
- Newly painted surfaces do normally not withstand rain.
- Only few operations should be performed in direct sun, use of sunshades is recommended.

Surface coatings may be valuable protective measures for structures in very harsh environments as marine structures and bridge columns in rough saline waters.

To obtain the optimal surface coating, various minimum requirements to each single procedure, cleaning of substrate, filling (if any), sealing and/or coating, as well as to the environment must be complied with.

Depending on the type of paint used, special requirements for the temperature and relative humidity of the environment, as well as the temperature of the substrate during the application of the surface coating and sometimes during the curing of the paint, should be given. Generally, the temperature of the substrate must be more than 3℃ above the dew point of the air.

Performance testing of coatings is an essential part of the acceptance process. Pull-off, or bond, tests are typical types of tests but additional testing is generally needed, depending on the environment and the scope of the work.

8.6.3 Durability monitoring

Designing for long service life needs some form of verification during use of the structure. Such a need of verifying the performance of the structure is just as relevant after a repair.

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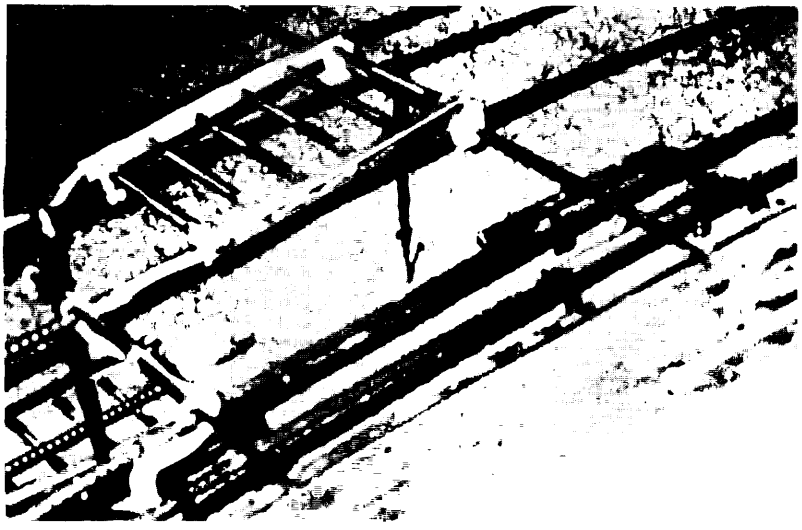


Figure 8.6-1: Corrosion sensor to be cast into selected spots of a new structure.



Figure 8.6-2: Corrosion sensor equivalent to the one shown in Figure 8.6-1 but designed to be inserted in a drilled hole in an existing structure.

This need for verification, and the benefits of obtaining more and more reliable forecasts of the service life to be expected, leads naturally to a need to monitor non destructively the durability performance of the structure in service.

Examples of corrosion sensors able to detect the depth of penetration of chlorides in quantities able to trigger corrosion are shown in Figure 8.6-1 and Figure 8.6-2. The sensors consist of six individual black steel bars or rings acting as anodes. For the cast-in sensors the anodes are fixed between two stainless steel bars forming a ladder. For the sensors to be fixed in drilled holes the anode rings are fixed in a stainless steel cylinder. Each anode will be electrically linked up to a separate highly active bar (platinised titanium) when current measurements are being taken and which acts as cathode when depassivation occurs on the anodes. Placing the sensors in the cover zone of the exposed part of the structure, and with the anodes at different depth from the surface, enables a detection of the progression of chlorides when the anodes one by one are depassivated and corrosion starts. A micro-amp meter inserted between the anodes and cathode will record a jump in corrosion current once depassivation on an anode has taken place. As corrosion signals are recorded with time a more and more precise prognosis can be made as to when the critical amounts of chlorides will have penetrated to the level of the reinforcement and reinforcement corrosion may start, i.e. the end of the initiation period may be forecast [Schiessl and Raupach (1992)].

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8.7 Repair

When damage has been observed in a structure, a large number of repair options are available. In recent years engineers have obtained a more clear understanding of the deterioration mechanisms governing the service life of structures. The only way of influencing the deterioration process is by influencing the parameters governing this mechanism.

A model relating aggressivity of exposure to concrete resistance is presented below [de Fontenay (1989)]. The model divides the problem into four fundamental groups describing this relationship, see Figure 8.7-1.

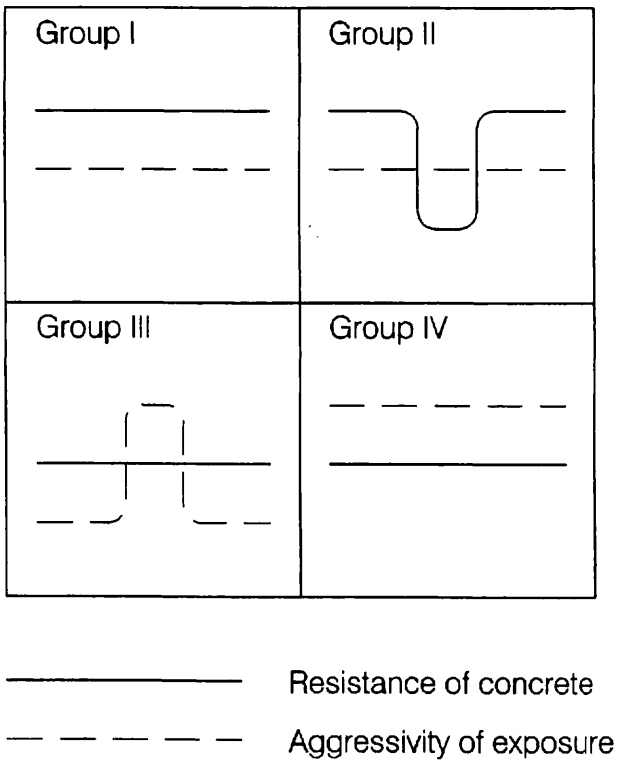


Figure 8.7-1: Four groups relating the relative resistance of the concrete along a structure to the aggressivity of environment [de Fontenay (1989)]

Group I: The substrate (existing concrete) is - as a whole - good enough. No deterioration is taking place.

Group II: The substrate is - as a whole - good enough, but deterioration is taking place at a local defect.

Group III: The substrate is - as a whole - good enough, but deterioration is taking place in a limited area caused by a local aggressive environment.

Group IV: The substrate is - as a whole - not good enough. Deterioration is probably taking place all over the structure.

During the initiation period a concrete structure may belong to group I. During the subsequent propagation period the structure gradually moves towards group IV.

8.7.1 Repair options

Depending on the performance and the quality the owner expects of his structure after repair, he can select between the following options:

- Do nothing for a certain time, but keep the structure, its safety and its performance under surveillance, e.g. by sensor monitoring.
- Re-analyse the structural capacity, possibly leading to a downgrading of the function of the concrete structure, e.g. by load restrictions.
- Prevent or reduce further deterioration, without improve the concrete structure, e.g. by inhibiting further reinforcement corrosion by cathodic protection.
- Improve, strengthen or refurbish all or part of the concrete structure, e.g. by prestressing or by wrapping of compressed members (columns).
- Reconstruct part or all of the concrete structure.
- Demolish all or part of the concrete structure.

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These options follow CEN TC 104 (1996), as does the factors to be considered below, and this is also in agreement with Rostam (1991). When choosing between these options there are a number of essential factors that should be considered in order to avoid conflicts with the owners direct and indirect requirements, with health and safety, and with structural or environmental consequences. These factors could be the following:

1. General

   a) The intended use, design life and service life of the structure
   b) The required performance characteristics, e.g. including fire resistance and water-tightness.
   c) The likely long-term performance of the protection or repair works.
   d) The availability of opportunities for additional protection, repair and monitoring
   e) The number and costs of repair cycles acceptable during the design life of the structure.
   f) The cost and funding of the alternative protection or repair options, including future maintenance and access costs.
   g) Properties and possible methods of preparation of the existing substrate.
   h) The appearance of the protected or repaired structure.

2. Health and safety

   a) The consequences of structural failure of the structure.
   b) Health and safety requirements
   c) The impact of the repair options on occupiers or users of the structure and on the public.

3. Structural

   a) The possible or necessary change of dynamic or other direct actions during or after the protection or repair.
   b) The means by which loads will be carried during and after the protection or repair.
   c) The need for future inspection and maintenance.

4. Environmental

   a) The future environment of the structure and whether it can be changed locally.
   b) The need or opportunity to protect part or all of the structure from weather, pollution, salt spray etc., including protection of the substrate during the repair work.

Structures that have passed the initiation period and the break point, see Figure 5.3-2, continue their life in the propagation period with deterioration taking place. The condition of these structures will worsen with time at a rate depending on type of deterioration and exposure conditions. More extensive and expensive measures are required to improve the condition from the actual level to a level prior to the break point. From there the condition will follow the nearly horizontal extended initiation curve until a new break point is reached.

Methods to improve the actual condition may be selected among the following two groups depending on the reason for needing an improvement of the condition:

- non-acceptable aesthetics or
- reduced safety and load-bearing capacity.

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An aesthetic improvement of the condition may be obtained by:

- patch repairs (not load-bearing)
- application or re-application of paint, surface coating or membrane

If an improvement of the load-bearing capacity is needed the solution should be chosen among the following methods:

- structural repairs (load-bearing)
- replacement of individual structural members or the whole structure
- strengthening of the structure by establishment of supporting elements or reinforcing elements

It should be emphasised that the execution of a structural repair is possible only under certain circumstances:

If only part of the cross-section of a compressed structural member, like a column, is removed during repair it is normally impossible to make the repaired part of the cross-section load-bearing mainly due to the fact that the repair concrete shrinks.

So-called shrinkage-compensating repair materials do not ensure the possibility of carrying out a load-bearing repair.

8.7.2 Decision of non-repair

In serious cases of deterioration a full replacement or extensive strengthening may become necessary. However, the timing of a replacement or strengthening of a weak structure will depend on the loading and the impact on structural safety as well as the level of functionality. If these essential requirements are acceptable for a period of time, then the replacement or strengthening may be postponed until the optimal economic condition regarding budgeting has been obtained.

The choice of action should in principle be based on a ***benefit/cost analysis*** taking all significant considerations into account. To be able to do this, it is indispensable to include realistic estimates of future deterioration and/or damage to the structure under the chosen conditions of use and environment, and such estimate must therefore be part of the overall assessment.

If a strategy of non-repair becomes a rational and feasible solution it implies that the structure is kept in service without repair until its safety falls short of an acceptable level. This safety level could be the codified level based on in situ measurements of strength data, or it could be a specific level approved for the particular structure and its special use. In many hot humid and saline environments, where certain generations of concrete structures, or individual structures, are suffering severe chloride corrosion, the non-repair option may be a realistic solution.

In the case of non-repair the safety of the structure must be handled on two different levels. Firstly, bits and pieces falling off the structure may create hazards to users and passers-by, but without threatening the overall safety and stability of the structure. Precautions must be taken against such hazards. Secondly, sensor monitoring, Chapter 8.6.3, and repetitive inspections and possible tests of vital parts of the structure must ensure that a warning is given

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before the overall safety and stability is reduced below the acceptable level. Such monitoring can today be followed by probabilistic treatment of data thus introducing a quantified risk analysis which can be a very powerful tool as decision support when keeping damaged structures in service. However, it must still be realised that unwarned failures may still occur due to the complex nature of the problem, and this is in fact what the risk analysis could quantify. Therefore, it is essential that the consequences of such a failure is carefully evaluated especially where human lives could be at risk.

The local variations in the depth of corrosion make the criteria of non-acceptable corrosion levels difficult to define precisely. In addition, the safety consequences of loss of reinforcement area will vary with different structural systems. Some systems, such as continuous or statically indeterminate structures, have large hidden reserves. Other systems are more vulnerable, e.g. some precast building systems. Designed according to conventional design codes slabs and beams often have larger build-in reserves than columns.

8.7.3 Repair principles and methods

Selection of repair materials, systems and products shall satisfy the preconditions following from the chosen repair options.

The most updated system for choosing maintenance and repair principles and methods is presented in CEN TC 104 (1996). The deterioration and the need for repair are referred to which material is the primary cause of deterioration. Therefore, the principles and methods are divided into two groups; the first deals with defects in concrete itself and the second with reinforcement corrosion. The principles and methods are presented in tabular form with the principle and its definition and some examples of methods, which comply with each principle, see Table 8.7-1 and Table 8.7-2.

Principles 1 to 6 in Table 8.7-1 cover defects to concrete structures caused by the following, singly or in combination:

a) Mechanical actions, e.g. impact, overloading, movement caused by settlement, and blast.
b) Chemical and biological actions from environments.
c) Physical actions, e.g. freeze-thaw action, thermal cracking, moisture movement, salt crystallisation and erosion.

Principles 7 to 11 in Table 8.7-2 cover reinforcement corrosion caused by:

a) Physical loss of the protective concrete cover.
b) Chemical loss of alkalinity in the protective concrete cover as a result of reaction with atmospheric carbon dioxide (carbonation).
c) Contamination of the protective concrete cover with corrosive agents (usually chloride ions) which were incorporated in the concrete when it was mixed or which have penetrated into the concrete from the environment.
d) Stray electrical currents conducted or induced in the reinforcement from neighbouring electric installations.

In addition to the above the concrete itself shall be repaired, where necessary, according to Principles 1 - 6.

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Principle No.	Principle and its definition	Methods based on the principle
Principle 1 [PI]	**Protection against ingress**	**1.1 Impregnation** Applying liquid products which penetrate the concrete and block the pore system **1.2 Surface coating with and without crack bridging ability** **1.3 Locally bandaged cracks** [1] **1.4 Filling Cracks** **1.5 Transferring cracks into joints** [1] **1.6 Erecting external panels** [1) 2)] **1.7 Applying membranes**
Principle 2 [MC]	**Moisture Control** Adjusting and maintaining the moisture content in the concrete within a specified range of values	**2.1 Hydrophobic impregnation** **2.2 Surface coating** **2.3 Sheltering or overcladding** [1) 2)] **2.4 Electrochemical treatment** [1) 2)] Applying a potential difference across parts of the concrete to assist or resist the passage of water through the concrete. (Not for reinforced concrete without assessment of the risk of inducing corrosion).
Principle 3 [CR]	**Concrete Restoration** Restoring the original concrete of an element of the structure to the originally specified shape and function. Restoring the concrete structure by replacing part of it.	**3.1 Applying mortar by hand** **3.2 Recasting with concrete** **3.3 Spraying concrete or mortar** **3.4 Replacing elements**
Principle 4 [SS]	**Structural Strengthening** Increasing or restoring the structural load bearing capacity of an element of the concrete structure.	**4.1 Adding or replacing embedded or external reinforcing steel bars** **4.2 Installing bonded rebars in preformed or drilled holes in the concrete** **4.3 Plate bonding** **4.4 Adding mortar or concrete** **4.5 Injecting cracks, voids or interstices** **4.6 Filling cracks, voids or interstices** **4.7 Prestressing - (post tensioning)** [1]
Principle 5 [PR]	**Physical resistance** Increasing resistance to physical or mechanical attack	**5.1 Overlays or coatings** **5.2 Impregnation**
Principle 6 [RC]	**Resistance to chemicals** Increasing resistance of the concrete surface to deterioration by chemical attack	**6.1 Overlays or coatings** **6.2 Impregnation**

1) These methods may make use of products and systems not covered by the EN 1504 series.

2) Inclusion of methods in this Prestandard does not imply their approval.

Table 8.7-1: Principles and Methods related to defects in concrete [CEN TC 104 (1996)]

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Principle No.	Principle and its definition	Methods based on the principle
Principle 7 [RP]	**Preserving or Restoring Passivity** Creating chemical conditions in which the surface of the reinforcement is maintained in or is returned to a passive condition.	**7.1 Increasing cover to reinforcement with additional cementitious mortar or concrete** **7.2 Replacing contaminated or carbonated concrete** **7.3 Electrochemical realkalisation of carbonated concrete** [1)] **7.4 Realkalisation of carbonated concrete by diffusion** **7.5 Electrochemical chloride extraction** [1)]
Principle 8 [IR]	**Increasing Resistivity** Increasing the electrical resistivity of the concrete.	**8.1 Limiting moisture content by surface treatments, coatings or sheltering**
Principle 9 [CC]	**Cathodic Control** Creating conditions in which potentially cathodic areas of reinforcement are unable to drive an anodic reaction.	**9.1 Limiting oxygen content (at the cathode) by saturation of surface coating** [2)]
Principle 10 [CP]	**Cathodic Protection**	**10.1 Applying electrical potential** [1)]
Principle 11 [CA]	**Control of Anodic areas** Creating conditions in which potentially anodic areas of reinforcement are unable to take part in the corrosion reaction	**11.1 Painting reinforcement with coatings containing active pigments** **11.2 Painting reinforcement with barrier coatings** **11.3 Applying inhibitors to the concrete** [1)2)]
1) These methods may make use of products and systems not covered by the EN 1504 series. 2) Inclusion of methods in the Pre-standard does not imply their approval.		

Table 8.7-2: Principles and Methods related to reinforcement corrosion [CEN TC 104 (1996)]

8.8 Examples from practice

In this Chapter a limited number of maintenance and repair cases have been summarised. The intention is to highlight some of the basic principles described in the foregoing text.

8.8.1 Preventive maintenance

Surface coating of a bridge pier exposed to very aggressive de-icing salts. After a couple of years in service the systematic inspections revealed that the quality of the concrete cover to resist the penetration of chlorides was below expectations. Testing the concrete cover as achieved in the final construction also revealed areas with cover less than required. Hence, it was decided to provide a surface protection being resistant to chloride ingress. In the present situation a polyurethane coat in three applications was provided, see Figure 8.8-1.

Preventive maintenance was foreseen when construction a pier located in extreme aggressive environment in the Middle East. At construction an anode mesh was placed in the cover zone of the part of the pier located in the splash zone. By carefully ensuring that no short circuit occurred between the anode and the reinforcement during casting and compaction, and ensuring electric continuity of the reinforcement, cathodic prevention can be easily energised

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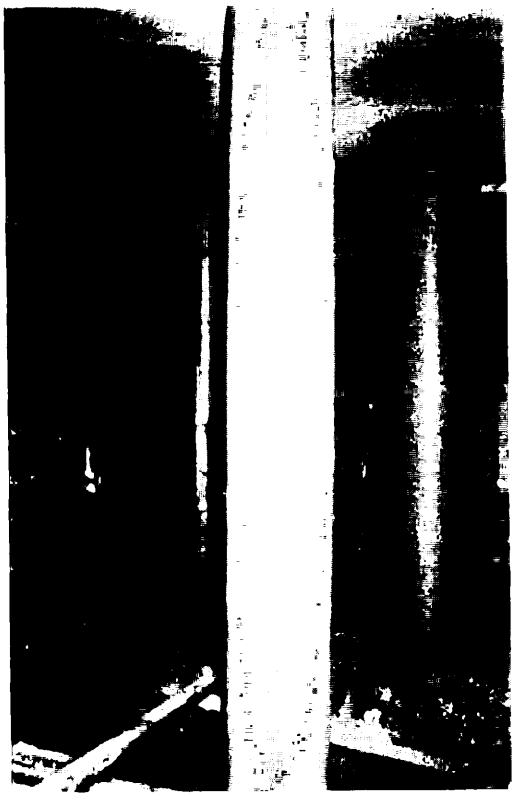


Figure 8.8-1: Preventive maintenance by providing a surface coating to a bridge pier exposed to a marine environment

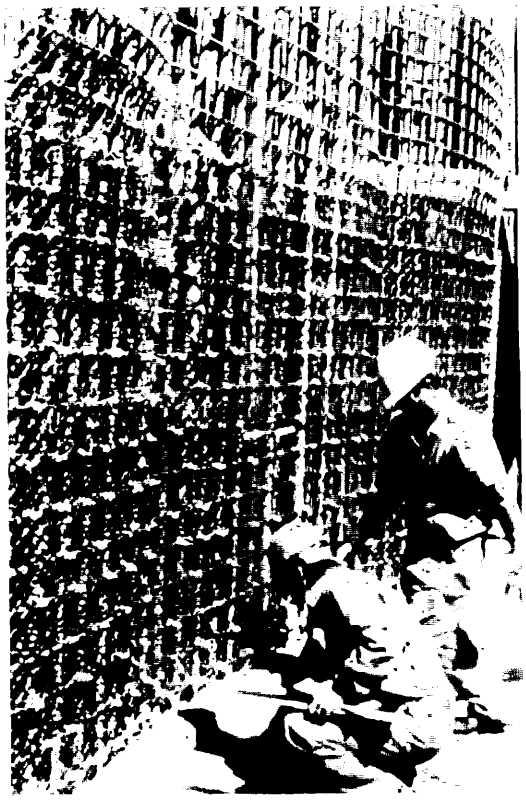


Figure 8.8-2: Anode mesh cast into the cover concrete in a new structure to allow early cathodic prevention.

when found necessary to maintain the structure in the initiation phase throughout the service life, see Figure 8.8-2.

Similarly, anode bands can be placed within the concrete, and beneath the reinforcement layer prior to casting the concrete.

The installation of cathodic prevention is much simpler and cheaper than installing it later when repair is needed. The requirement of anode quantity is much lower, and maintaining passivation is much easier than polarising and re-passivating corroding reinforcement.

8.8.2 Repair of corrosion damaged concrete columns

Corrosion of the reinforcement of concrete columns affects mainly the concrete cover, which may spall once the corrosion is extensive. In addition, the reinforcement is no longer able to provide the strength as foreseen in the design.

In case the geometry and size of the column is in excess of what is required from the structural calculation, a re-skinning of a column suffering extensive chloride corrosion may be an attractive solution. Re-skinning means cutting back all concrete with excessive chlorides and replacing corroded reinforcement followed by casting a new concrete cover - possibly with an additional thickness, if acceptable from a structural and aesthetical point of view, see Figure 8.8-3.

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Figure 8.8-3: Re-skinning of large concrete columns damaged by chloride corrosion

If the concrete dimensions are small and the column has not noticeable reserve capacity, a re-skinning is not feasible. If such a column is seriously corroded, a full replacement of the column or just of the chloride contaminated parts, if special external strengthening is not chosen, Figure 8.8-6.

8.8.3 Patch repairs

Local patch repairs constitute the most usual means of repairs of concrete structures. Few methods have been misused as seriously as this approach, in particular when the cause of deterioration is chloride induced reinforcement corrosion. The following description may illustrate how important it is to have a clear understanding of the causes of the deterioration that has lead to the local damage and which should be repaired through a patch repair.

Chloride corrosion of reinforcement cast into concrete has the effect that the local corrosion pit tends to depress the potentials in the near vicinity of the pit even if the chloride concentration in this neighbouring zone is above the threshold value for corrosion.

A locally spalled concrete section suffering chloride induced corrosion is illustrated in Figure 8.8-4. If repaired by traditional local patch repair only the anodic area is repaired. The chlorides in the neighbouring concrete may just shift the anode to these areas and corrosion continues, see Figure 8.8-5. In fact, if the repair itself is made very well, with a good cleaning of the corroding reinforcement then the repair has in fact created an enlarged cathode, which increases the rate of the continued corrosion. This effect is verified by Figure 8.8-7.

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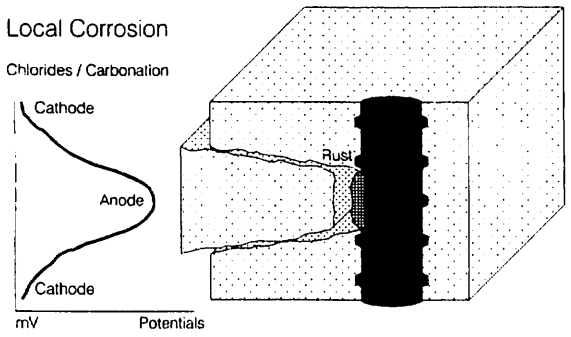


Figure 8.8-4: Damage due to local chloride induced corrosion. The electro-chemical potentials of the reinforcement are also indicated, with low potentials indicated location of corrosion. The spalling is due to the expansive effect of the rust products.

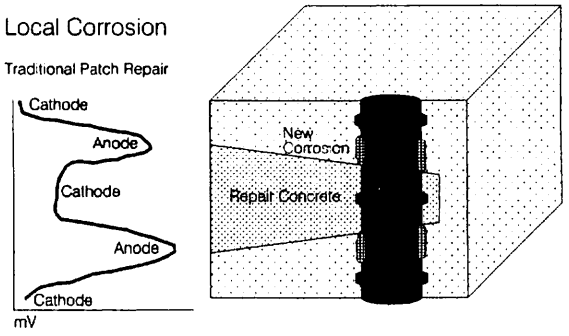


Figure 8.8-5: Traditional patch repair of the damage illustrated in Figure 8.8-4. Due to the corrosion in the neighbouring concrete the corrosion is just moved but continues.

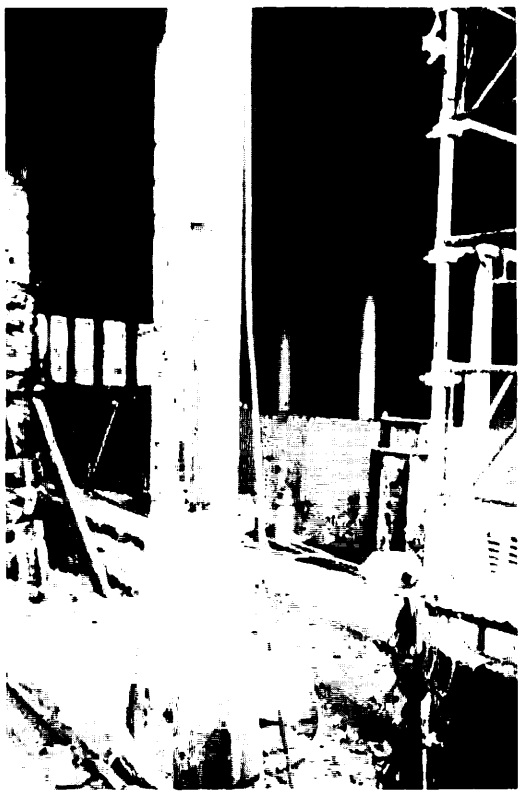


Figure 8.8-6: Replacing the chloride-contaminated part of a corrosion damaged concrete column



Figure 8.8-7: Cracks along a previous patch repair of chloride induced corrosion indicating renewed corrosion due to inadequate understanding of the chloride corrosion mechanism.

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One of the problems with chloride induced corrosion is that the chlorides are not consumed by the corrosion process, but are liberated again from the rust products. They act as catalyst for the corrosion, and therefore remain available for further corrosion. This means that preventing additional chlorides from entering into the concrete after corrosion has occurred does not by itself stop further chloride corrosion. This effect makes it difficult to perform reliable long lasting repairs of chloride corrosion damaged concrete structures.

The most reliable repairs of chloride corrosion damage - or means of preventing chloride corrosion to develop - is to use cathodic protection - or cathodic prevention.

8.8.4 Electro-chemical repairs

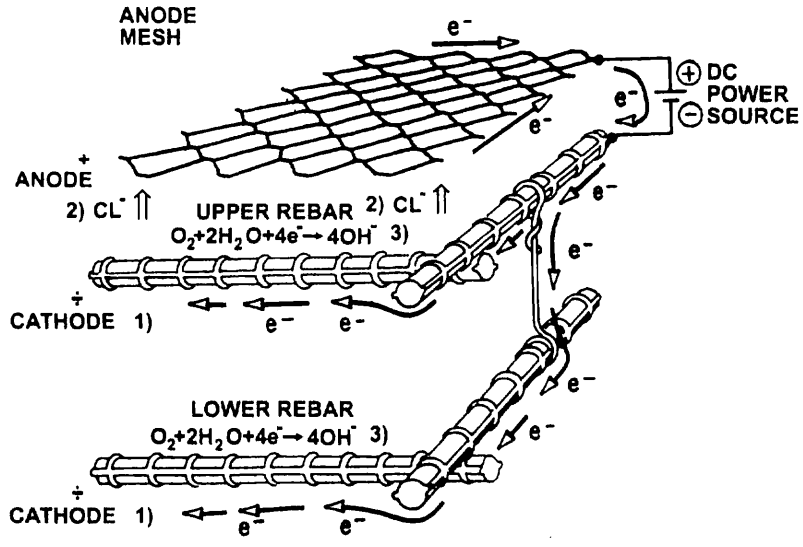


Figure 8.8-8: Electro-chemical processes of imposing an electric field between a cast-in reinforcement and an external electrode.

Electro-chemical repairs refer to three very closely related methods of solving the problems of reinforcement corrosion, either by preventing corrosion to develop or by stopping ongoing corrosion.

The methods rely on three different effects of imposing an electric field between the cast-in reinforcement and an external electrode, see Figure 8.8-8.

1. By connecting the reinforcement to the negative connection of a direct current electric source and the external electrode to the positive connection, the reinforcement is forced to remain negative i.e. to act as a cathode, and will thus not corrode: "**Cathodic Protection**".

2. The electric field will tend to lead negatively charged ions away from the reinforcement and towards the electrode, hence chloride ions - among others - will tend to move out of the concrete to the external electrode and by special means they can be remove, e.g. by washing: "**Chloride Extraction**".

3. The electrolysis taking place at the surface of the reinforcement will produce hydroxyl ions which create an alkaline environment. Properly controlled this can re-alkalise carbonated concrete and thus restore the corrosion protection of the reinforcement otherwise exposed to a carbonated concrete: "**Re-alkalisation**".

Two of the major differences between the practical application of these methods are the current density needed and the time needed for the process, as highlighted below.

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(1) Cathodic protection

Cathodic protection is a long term well known technique for protecting submerged or buried sheet piles, steel piles, pipe lines water mains, etc., against corrosion. Since the mid 70íes beginning 80íes this technique was developed for also protecting concrete structures in the atmosphere, particularly when threatened by chloride induced corrosion. For submerged or buried parts of such structures the classical methods can be used with either sacrificial anodes or with impressed current systems and inert anodes. But for parts of the structure above the water or ground the system is more complicated.

For structural parts in the atmosphere the problem is to achieve a sufficiently even distribution of the protective current to the reinforcement being covered by concrete with a rather high electric resistance.

The most reliable and robust system today seems to be the use of a surface mounted titanium mesh coated with a precious metal catalyst.

For new structures preventive cathodic protection in very aggressive environments may be required, and in such a case the anode mesh can be placed in the formwork prior to casting the formwork, but making sure that there are no short circuits between this mesh and the reinforcement. In addition the reinforcement must be electrically continuous in selected zones.

For an existing structure the most usual approach is to make necessary repairs of the damaged concrete and then fix the anode mesh on the surface and then cast a thin concrete overlay to ensure electrolytic contact between anode mesh and the reinforcing bars, see Figure 8.8-9.



Figure 8.8-9: Cathodic protection of 55 year old pier for a bascule bridge. Precious metal coated anode mesh has been fixed to the surface and a sprayed concrete overlay is being applied.

The anodes are then linked up to the positive connection of a transformer rectifier and the reinforcement linked up to the negative connection. This will force the reinforcement surface to remain electro-chemically negatively charged, and thus remain a cathode. With an inert anode the anode material is not consumed. With a sacrificial anode the material is consumed with time and shall be replaced at intervals.

Cathodic protection is generally to be a permanent installation needing regular inspection and maintenance. However, the current density needed to provide long term reliable protection, even in heavily chloride contaminated zones, is in the range of 5 - 20 mA/m^2 referring to the steel surface to be protected.

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(2) Chloride extraction

Chloride extraction is a temporary installation lasting one or two months and requiring a current density in the range of 500 - 2000 mA/m^2.

(3) Re-alkalisation

Re-alkalisation is also a temporary installation lasting a few weeks to a month and requiring a current density in the same range of 500 - 2000 mA/m^2 as chloride extraction.

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9 Practical aspects

by Konrad Zilch and Angelika Schießl

9.1 Introduction

This chapter deals with the transition from structural design to execution. During the design process, the verification of limit states according to chapters 4.3 and 4.4 is based on the strength or stiffness etc. of members. They depend on material properties and cross section dimensions. The nature of these quantities is not deterministic but stochastic due to the scatter in the raw materials used (aggregate, portland cement) and in the imperfections of the erection process itself (precision of formwork, water/cement-ratio). So a major point of interest are the deviations of actual properties from nominal values. From the point of structural safety, limiting values for these deviations (*tolerances*) must be defined that are in accordance with the basic assumptions of the safety concept. In order to assure the tolerances defined in design new strategies were developed to achieve a high level of quality in the execution. They are generally known as quality management and are also presented in this chapter. Finally some aspects in the erection of RC and PC structures are described followed by an introduction into prestressing and precast elements and structures.

9.2 Geometric tolerances

9.2.1 Definition and types

The term *tolerance* defines the maximum allowable deviation between an actual geometric quantity of a structural element and its nominal value specified by the designer[1] (Fig. 9.2-1). Under site conditions, certain deviations are inevitable; they may be due to the imperfection of geodetic measurement, the imperfection of formwork and its deformation during casting. They may also be caused by gross errors, but those cannot be treated within the context of tolerances in a reasonable manner. Therefore, they should be excluded by adequate measures of quality control.

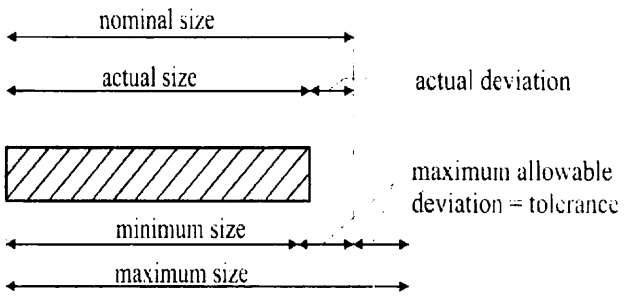


Fig. 9.2-1: Definition of nominal size, deviation and tolerance

[1] Some authors define tolerance as the difference between the maximum and minimum allowable size. Within this chapter, the definition given above will be used.

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Tolerances are criteria whether an imperfectly executed structure does comply with the requirements of safety, durability and serviceability. Their definition is part of the design concept as well as the definition of safety factors. (In this chapter, it will indeed be shown that there is a close connection between tolerances and safety factors.) On the other hand, tolerances are a tool to specify the quality of execution between the client and the executing company.

Geometric deviations (and the corresponding tolerances) may be subdivided into:

- deviations in cross section dimensions,
- deviations in the position of reinforcement,
- deviations in span,
- deviations from perfect shape (curvature) and direction of members,
- deviations in the location of members.

A typical example for an RC member is shown in Fig. 9.2-2. The influence of deviations on structural performance may differ depending on the type of structure, the erection procedure and the specific requirements of serviceability.

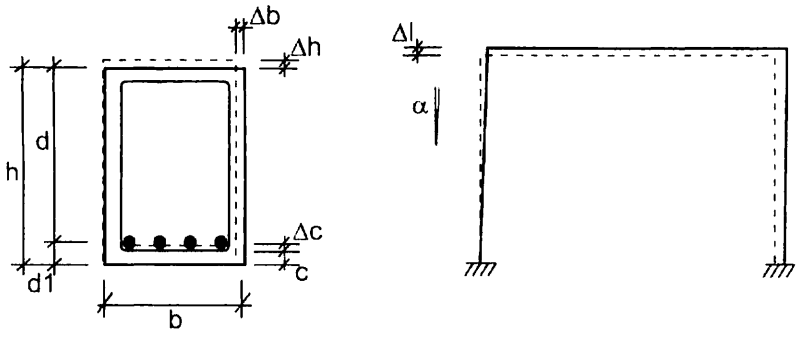


Fig. 9.2-2: Geometric dimensions and deviations of a reinforced concrete member

9.2.2 Effects of tolerances

(1) Effects on safety

Geometric deviations have an influence on structural safety (see chapter 4.2) with regard to actions as well as to resistance. A scatter in span may increase the internal forces of a member, but this scatter is usually small compared to the total length and can be neglected in design. In precast structures, however, the deviation in span can reduce the length of support (see 9.2.2 (4)) and must be taken into account.

Deviations in cross section dimensions and in the position of reinforcement can cause a considerable decrease of the inner lever arm. Observations on executed structures showed that these deviations do in general hardly depend on total member size [Maaß, (1980)]. This causes a size dependent coefficient of variation in the cross section dimensions.

Probabilistic calculations show that for a large range of specimen sizes the sensitivity α_i of β to geometric deviations is almost equal to zero. For d < 10 cm the sensitivity increases rapidly indicating a growing influence of geometric deviations on the reliability due to a stochastic size effect. In these cases b and d must be considered fundamental basic variables.

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From a theoretical point of view, the strong influence of geometry on the safety of small members would require the use of design values of the cross section dimensions in ULS design within the framework of partial safety factors. However, this is undesirable from the practical point of view, as the additional influence on dead load (see below) demands the consideration of upper and lower design values.

Therefore, in MC 90 it was decided to introduce the *nominal* values of cross section dimensions into design and cover the uncertainty by an increase of the material safety factors γ_s and γ_c. The calibration of these safety factors can only be performed for assumed reference member dimensions and deviations (see e.g. [Östlund (1991)]). As a practical consequence, the design procedure in MC 90 (and in almost any code) is applicable if

- the dimensions of the members are not below specified minimum dimensions and
- the deviations are limited to the specified tolerances by an adequate quality assurance.

The minimum dimensions used in MC 90 were set to b,h > 80 mm for linear members and h > 100 mm for slabs[2]; the required tolerances are given in 9.2.3. The verification of smaller members might be performed using design values of the dimensions. The design value of a length a can be derived from:

$$a_d = a_{nom} \pm \Delta a_d \text{ with } \Delta a_d = 1.2 \cdot \Delta a \qquad (9.2\text{-}1)$$

where Δa is the allowable tolerance.

On the other hand a reduction of safety factors would be justified if either the tolerances are reduced or the cross section dimensions considerably exceed the minimum values. MC 90 allows a reduction of γ_s and γ_c by 0.05...0.10 if the standard tolerances are reduced by 50 % and strictly controlled. This reduction might be applied to precast units that are produced under industrial conditions.

An alternative way to handle the size effect is a modification of action effects depending on the member size. German code DIN 1045 (7/88) requires (d in [cm])

$$M^* = M \cdot \frac{15}{d_{nom} + 8}$$
$$N^* = N \cdot \frac{15}{d_{nom} + 8} \qquad (9.2\text{-}2)$$

for $d_{nom} < 7$ cm.

Apart from the effects on the resistance geometric deviations will also have an influence on the dead load of the structure. This is of considerable importance for concrete structures where the dead load often dominates over the service load. The handling of this uncertainty is difficult due to the fact that the dead load may cause favourable and unfavourable effects. So, the definition and application of upper and lower fractile values of g_k and thus an enormous

[2] For concrete members, minimum dimensions must also be taken into account due to the fact that common theories regard the composite of aggregate and matrix as a homogeneous continuum. This simplification is only justified in cases where the dimensions of the concrete elements exceed the maximum aggregate size by a factor of at least five.

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increase in possible load combinations is required. Fortunately the scatter in density and cross section area is rather small. This allows an introduction of mean values as characteristic values $g_k = g_m$ and the assumption that g is fully correlated within the structure. For the determination of the minimum and maximum bending moments in a continuous beam it is sufficient to consider g_d to be constant in all spans with its upper or lower value. This simplification is generally on the unsafe side, but this might be accepted as there is still a sufficient safety margin on the resistance side.

In many cases the effect of geometric deviations on dead loads will be partially compensated by the effects on the bearing capacity. An unintentional increase in the thickness of a slab will not only increase the dead load, but also the inner lever arm (assuming that the concrete cover of the reinforcement remains unchanged) and vice versa. However, this is only true for the considered member. Supporting members, e.g. a beam supporting a slab, will have to carry the increased weight without participating in the increased resistance.

In some special cases where self weight or its scatter within the structure is the dominant action effect additional considerations are required. Fig. 9.2-3 shows a cantilever constructed bridge. The stochastic mean value of the residual moment M_g to be taken by the support is equal to zero. The limiting case of a vanishing correlation of g between the two cantilevers gives an upper bound for the standard deviation of M_g:

$$\begin{aligned} M_g &= M_l - M_r \\ \mu_{Mg} &= \mu_{Ml} - \mu_{Mr} = (\mu_{gl} - \mu_{gr}) \cdot \frac{l^2}{2} = 0 \\ \sigma_{Mg} &= \sqrt{\sigma_{Ml}^2 + \sigma_{Mr}^2} = \sqrt{\sigma_{gl}^2 + \sigma_{gl}^2} \cdot \frac{l^2}{2} = \sqrt{2} \cdot \sigma_{gl} \cdot \frac{l^2}{2} \end{aligned} \qquad (9.2\text{-}3)$$

It is assumed that the scatter in l can be neglected and that g is normally distributed. Introducing a coefficient of correlation ρ between the dead load on both sides the standard deviation reduces to:

$$\sigma_{Mg} = \sqrt{\sigma_{Ml}^2 + \sigma_{Mr}^2 - 2 \cdot \rho \cdot \sigma_{Mr} \cdot \sigma_{Ml}} = \sqrt{2} \cdot \sigma_{gl} \cdot \frac{l^2}{2} \cdot \sqrt{1 - \rho} \qquad (9.2\text{-}4)$$

In any practical case ρ will be between the limits of a full correlation (ρ=1) and no correlation (ρ=0). The estimation of ρ should take into account that the two cantilevers are cast with different formwork by different teams resulting in a rather weak correlation.

Alternatively, the scatter of dead load can be decomposed into a mean scatter of the whole structure g_0 (that could be found by comparison with other bridges) and a local fluctuation Δg within the structure. By definition, the first part is fully correlated within the structure and the second has a mean of zero. Using eq. (9.2-4) the residual moment is independent of the mean deviation g_0 and only a result of the local fluctuation.

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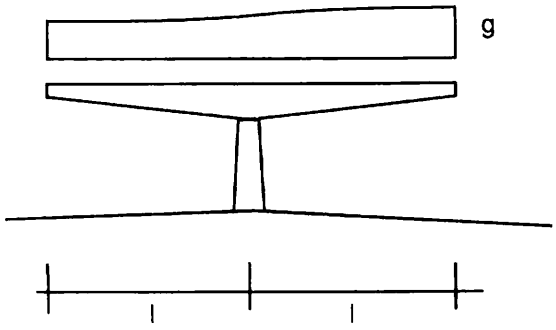


Fig. 9.2-3: Self weight of a cantilever construction

It is obvious that in practical design the required level of safety cannot be achieved by the usual assumption of a full correlation of g. For the verification of static equilibrium the use of upper and lower fractile values in the most unfavourable distribution is required.

A special case of geometric deviations causing action effects are unintentional inclination and curvatures of slender columns. Their influence on structural safety is often considerable. In the design procedure, these imperfections must be introduced as design values. The design value of an inclination or curvature cannot be derived from specified tolerances directly, as it is supposed to account for additional non-geometric effects like curvature due to thermal gradients in cases where these effects are not explicitly taken into account.

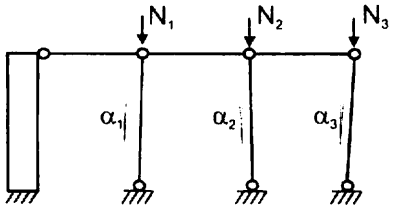


Fig. 9.2-4: Inclinations in a frame system

For single columns, the imperfection must be assumed to have the most unfavourable shape and direction. However, for frames and columns acting in parallel systems this is conservative from a probabilistic point of view. The destabilising force F_i of a column inclined with an angle α_i is:

$$F_i = N_i \cdot \tan\alpha_i \approx N_i \cdot \alpha_i \qquad (9.2\text{-}5)$$

If the inclination is a random variable with zero mean and N_i is deterministic, the standard deviation of F_i may be evaluated from

$$\sigma_{F_i} = N_i \cdot \sigma_{\alpha_i} . \qquad (9.2\text{-}6)$$

A full correlation of the inclinations gives a standard deviation for the total horizontal force of

$$\sigma_{F,tot} = n \cdot N_i \cdot \sigma_{\alpha_i} \qquad (9.2\text{-}7)$$

where n is the number of columns. The assumption that all columns show the design value of the inclination is very conservative.

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Assuming that the imperfections are mutually independent, the standard deviation is:

$$\sigma_{F,tot} = \sqrt{\sum N_i \cdot \sigma_\alpha^2} = \sqrt{n} \cdot N_i \cdot \sigma_\alpha \qquad (9.2\text{-}8)$$

In eq. (9.2-8) a complete lack of correlation is assumed as well as the equality of all acting vertical forces N_i. This is on the unsafe side when there is a systematic error (i.e. some degree of correlation) in the unintentional inclination. In this case, the total variation may be split into a systematic part α_s and a random part α_r. For the systematic and random part, eq. (9.2-7) and (9.2-8) apply, respectively. The combination of the two parts gives a total standard deviation of the acting horizontal force:

$$\sigma_{F,tot} = N_i \cdot \sqrt{n^2 \cdot \sigma_{\alpha s}^2 + n \cdot \sigma_{\alpha r}^2} = n \cdot N_i \cdot \sqrt{\sigma_{\alpha s}^2 + \frac{1}{n} \cdot \sigma_{\alpha r}^2} \qquad (9.2\text{-}9)$$

Due to the lack of data, the choice of the relation α_s/α_r is rather arbitrary. The formula given in MC 90 is based on the assumption that $\sigma_{\alpha_s} = \sigma_{\alpha_r}$:

$$\sigma_{F,tot} = n \cdot N_i \cdot \sigma_\alpha \cdot \sqrt{\frac{1}{2} \cdot \left(1 + \frac{1}{n}\right)} \qquad (9.2\text{-}10)$$

The application of the reduction factor in eq. (9.2-10) requires averaging and transfer of horizontal forces between columns and horizontal members. The connections must be designed for the maximum possible difference between F_i and the averaged force F_{tot}/n (also considering the case that F_i and F_{tot} have different signs). In structures made of precast elements this transfer may be difficult and may require expensive detailing. Neglecting the averaging effect might be more economic.

(2) Effects on durability

In RC and PC structures reinforcement corrosion is mainly prevented by a concrete cover of adequate thickness and quality (see chapter 5). Considering the fact that a large percentage of cases where structures do not fulfil the durability requirements can be attributed to an insufficient corrosion protection, the concrete cover c must be regarded as the most important geometric quantity.

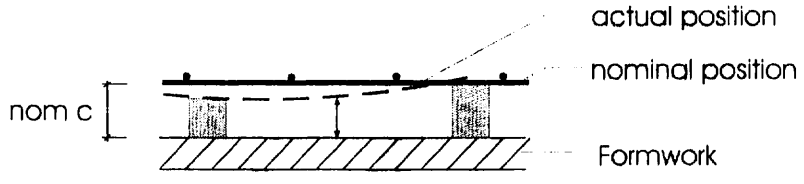


Fig. 9.2-5: Deviations in concrete cover

In structures cast in situ the variation in c (see Fig. 9.2-5) is rather large for various reasons. In beams, the longitudinal reinforcement is fixed to the stirrups. The concrete cover (especially for the top reinforcement) is basically the difference between member size l_c and stirrup size l_w. As stirrups are bent independently from the casting process the scatter in $l_c - l_w$ will generally be larger than the scatter in l_c. In slabs, a stepping on reinforcement during the casting process is inevitable. This reduces the concrete cover especially when the distance between spacers is large and the diameter and stiffness of bars is small.

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Many studies have been devoted to the determination of the deviations of concrete cover in precast members and structures cast in situ. Fig. 9.2-6 shows an example for the distribution of deviations measured on several office buildings in Munich in 1975/1976 according to Maaß and Rackwitz (1980). Surprisingly the mean value of the deviation Δc does not equal zero. The authors assumed that stirrups were bent with smaller dimensions than specified to avoid troubles during the installation of reinforcement that may not fit into the formwork.

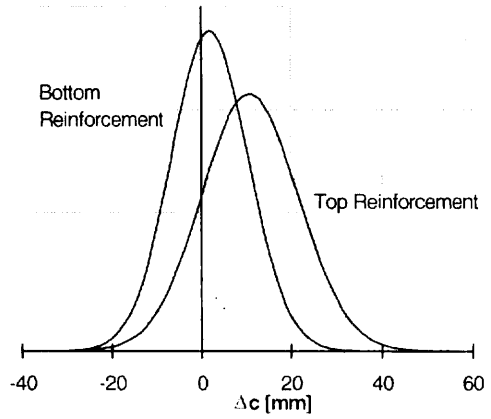


Fig. 9.2-6: Distribution of deviations in c according to Maaß and Rackwitz (1980)

To achieve an adequate level of reliability for design vs. durability, the required minimum concrete cover defined in chapter 5 must be compared with a fractile value of the actual cover. If the standard deviation is set to $\sigma \approx 8...10$ mm and $\mu = 0$ we get:

$$\Delta c_{5\%} = 1{,}64 \cdot \sigma \approx 15\text{mm}$$
$$\Delta c_{10\%} = 1{,}28 \cdot \sigma \approx 10\text{mm} \qquad (9.2\text{-}11)$$

These fractiles are used as additive safety elements for the determination of nominal concrete covers.

The choice of the fractile may be different for different environmental conditions. For interior members, a carbonisation of concrete will generally not cause severe corrosion damage due to the lack of water (see chapter 5). In this case, a rather high fractile (10%) may be adequate. A lower fractile should be chosen for members subjected to severe environmental conditions. In addition, it would be reasonable to choose Δc depending on the stiffness of reinforcement and on the number and distance between spacers. As this is not taken into account in code formulation, special attention should be paid to the type and number of spacers. Recommendations are given in Fig. 9.2-7.

Durability also requires an effective drainage of surfaces subjected to environmental influences like flat roofs. In any case where the inclination of the surface is small, possible deviations must be considered as well as the increase in deformations due to creep and cracking.

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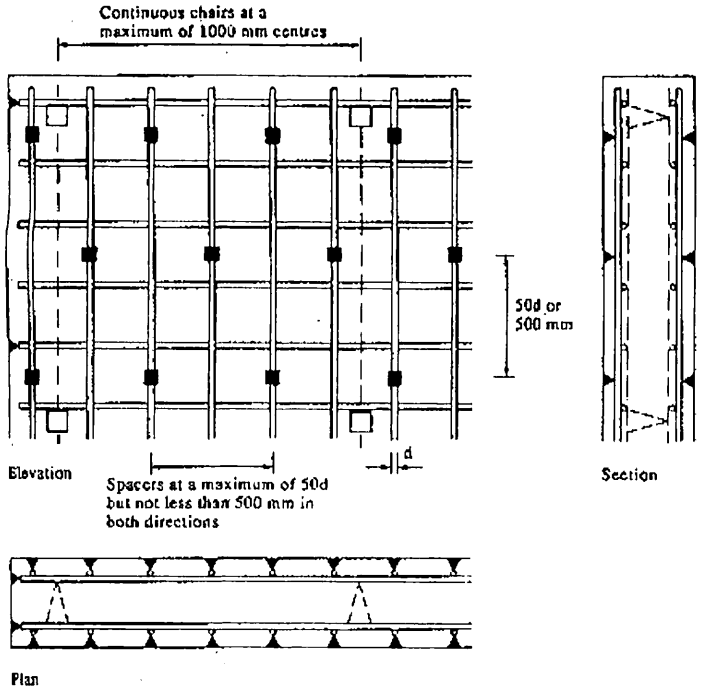


Fig. 9.2-7: Recommended spacing of supports and spacers (according to CEB 164)

(3) Effects on serviceability and structural appearance

Human vision is very sensitive to deviations from straight lines and surfaces. As this is a matter of individual perception objective limits and tolerances cannot be found, and the applicability of standards (e.g. German DIN 18202) to an individual case may be doubted. Deviations from smoothness are measured as the maximum distance of a point from a secant line with a certain base length (Fig. 9.2-8).

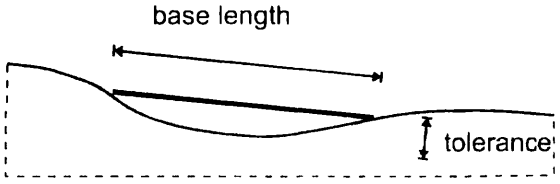


Fig. 9.2-8: Definition of evenness

Tolerances can have a significant effect on later operations in the construction process. As an example, if the surface of a concrete ceiling tile is not even, the thickness of the screed topping have to be increased that a constant minimum height is obtained. This may lead to a significant increase of costs. Considering the fact that the requirements on evenness have a considerable effect on costs (Fig. 9.2-9) it might often be reasonable to define the admissible surface tolerance between client and contractor.

The tolerances that can be achieved are mostly a function of the stiffness and the correct arrangement of the formwork (see 9.5.1). Generally a rough surface finish that can be achieved by the use of an unplaned wood planking will be less sensitive. If a sufficient stiffness of the formwork cannot be achieved, imperfections of the member shape can be reduced by a camber of the formwork. A camber might also be used to reduce the visible

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deformations of the structural member itself, but those member deformations are no matter of tolerances.

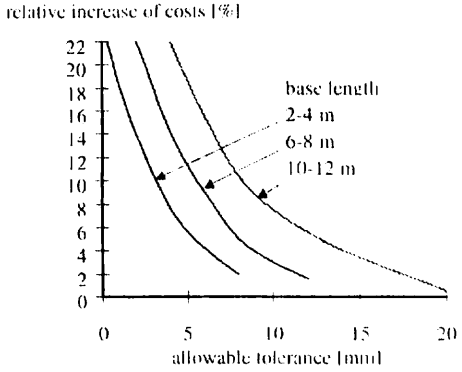


Fig. 9.2-9: Effects of evenness tolerances on structural costs according to Tiltmann (1977)

(4) Effects on erection of precast concrete structures

The use of precast elements is attractive in structures where single elements are used repeatedly or where a close schedule demands a quick progress of construction. Their application requires stricter tolerances than those specified for structures cast in situ. During the installation of prefabricated elements deviations cannot be adjusted by adapting the member size as in the case of conventional site concrete. In the worst case, an element may not fit into the allocated position. Specifications for tolerances are given in PCI Journal 1 (1985). In structures where different materials are combined attention must be paid to the fact that the tolerances specified in the relevant product standards may be different.

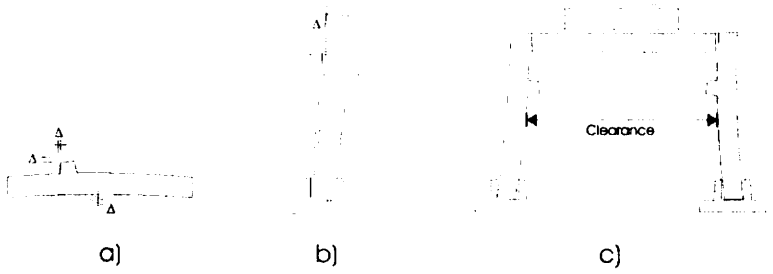


Fig. 9.2-10: (a) Product tolerances, (b) erection tolerances, and (c) effects of clearance

Distinction must be drawn between product tolerances and execution tolerances. Product tolerances are limits for the imperfection of the precast member itself, i.e. its size and shape due to the production in the concrete plant [see Figure 9.2-10 (a)]. Erection tolerances refer to deviations in the installation on site, i.e. the position and spatial orientation of the members in the structure [see Figure 9.2-10 (b)]. The determination of possible total deviations must take into account both types. On the other hand, deviations may reduce the support length of a member (Fig. 9.2-12). The minimum support length is

$$a_{min} = a_1 + a_2 + a_3 \qquad (9.2\text{-}12)$$

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where a_1 may be derived from the limitation of the contact pressure to the strength f_d of either the bearing or the concrete element:

$$F_{sup,d} = a_1 \cdot b \cdot f_d \tag{9.2-13}$$

The terms a_2 and a_3 depend on the concrete cover in the connected members according to Fig. 9.2-12. They are intended to prevent a spalling of concrete.

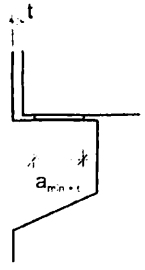


Fig. 9.2-12: Support length of a precast member

This minimum length must be increased to account for the erection tolerances t_e (i.e. the inclination of the column or its deviation from the nominal location in Fig. 9.2-12) and the product tolerances t_p (i.e. the length of the beam and the corbel). The total tolerance of the support length can be derived by the combination (in a stochastic sense) of t_e and t_p:

$$t = \sqrt{t_e^2 + t_p^2} \tag{9.2-14}$$

The nominal support length is $a_{nom} = a_{min} + t$.

(5) Admissible tolerances

Fig. 9.2-13 shows a comparison between the admissible tolerances according to several European standards and MC 90 for different geometric properties. However, one must keep in mind that there is an interaction between tolerances and safety factors or additive safety elements. For this reason tolerances can not be compared directly.

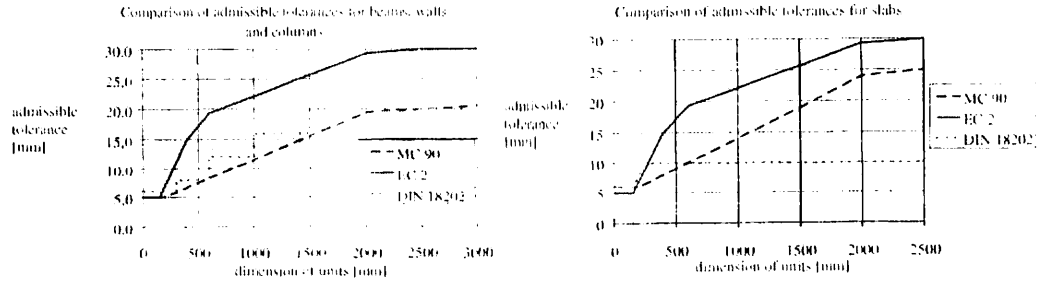


Fig. 9.2-13: Tolerances for the cross section of beams and slab thickness

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9.3 Quality requirements for material properties

9.3.1 Variations in material properties

Variations of material properties depend on the kind of product. Civil engineers deal with three kinds of products: natural products (e.g. wood), products made on the construction site (in-situ concrete) or industrially manufactured products (e.g. steel, glass). The quality of natural products can hardly be influenced by mankind and they depend on various conditions (e.g. climate for wood, geological origin for rocks etc.). However, new technologies have already been developed where the quality of natural products (e.g. boards) is estimated by machines and then an automatic graduation with respect to quality criteria can be made.

In concrete technology, engineers deal with industrially manufactured products (reinforcing and prestressing steel) and with products made on the construction site. Tolerances of these products depend on the properties of the ingredients and on the quality of fabrication. In the following statistical evaluations of variations in material properties (steel and concrete) are shortly presented. It can be seen that the deviations found in the case of industrially manufactured products are significantly lower than in the case of products made on the construction site

Russwurm (1988) gives an example for the coefficients of variation for steel products shown in Table 9.3-1. It can be seen that the coefficients of variation depend on different factors like the examined property, the manufacturer, the production year and also the kind of product. For example prestressing steel which needs to fulfil high requirements with respect to a good quality has the lowest average coefficient of variation and also the lowest yearly difference in variation. Regarding the entire statistical evaluation the coefficients are found in a range of 0,05-0,19.

Type of steel	manufacturer	property	coefficient of variation in different years of production		
Reinforcing bars	factory 1	R_e [1]	0.066	0.040	0.044
		A_{10} [2]	0.109	0.085	0.060
		ΔA_s [3]	0.016	0.016	0.017
	factory 2	R_e	0.037	0.035	0.043
		A_{10}	0.092	0.124	0.042
		ΔA_s	0.021	0.009	0.019
fabric	factory 1	R_e	0.063	0.064	0.070
		A_{10}	0.134	0.156	0.190
		ΔA_s	0.019	0.019	-
	factory 2	R_e	0.055	0.068	0.072
		A_{10}	0.101	0.163	0.134
		ΔA_s	0.020	0.015	0.019
Prestressing steel (St 1080/1230)		R_e	0.010	0.010	0.013
		A_{10}	0.12	0.07	0.1
		ΔA_s	0.004	0.005	0.005

[1]) R_e : yield strength
[2]) A_{10} : strain at rupture measured on a length of 10 d_s
[3]) ΔA_s deviation in cross section area

Table 9.3-1: Coefficients of Variation of Steel Products according to Russwurm (1988)

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Rackwitz et al. (1976) carried out an investigation on compression test results of concrete specimens manufactured in different areas in Germany. The objective was to find out if the deviation of actual from nominal strength decreases with increasing quality control. Furthermore different kinds of concrete production (ready mixed concrete or mixed in-situ) were studied. As a result of a statistical evaluation Rackwitz concluded that quality control more than the strength class has an effect on the standard deviation of the compression strength. Fig. 9.3-1 gives an idea of the existing scatter in concrete production.

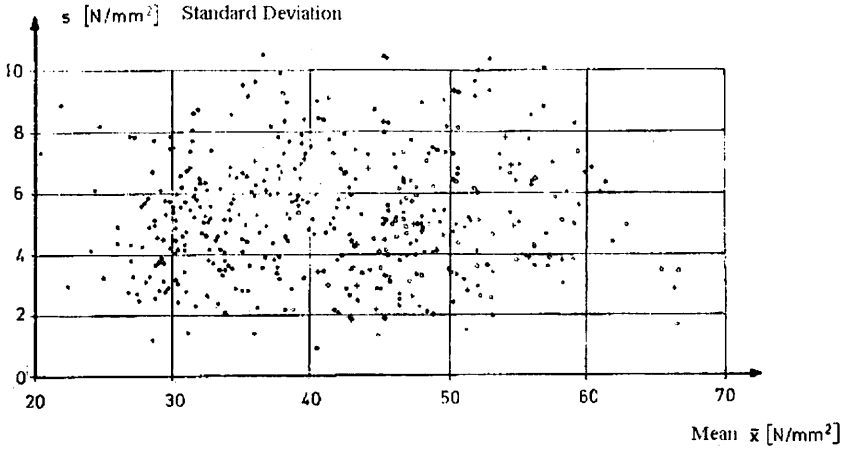


Fig. 9.3-1: Correlation between mean value and standard deviation of the compressive strength according to Rackwitz (1975)

But it is important to state that these deviations are not only caused by the material concrete itself but also by a great number of other conditions (place and time of testing, procedure of testing, quality of ingredients, variations in transporting, curing, compaction and temperature, etc.).

Since the study of Rackwitz was carried out, intensive research was devoted to the development of standardised requirements for concrete production in order to reduce the discrepancy between actual and nominal strength. This is important, because with the introduction of high performance concrete in the European standards, the quality requirements to a soundness production and the later use of a structure are even more pronounced than for normal strength concrete. According to the results of the latest statistical evaluations by e.g. Hosser and Gensel (1995), König et al. (1998), test methods and evaluation criteria were developed for the future European concrete standard prEN 206 in order to guarantee variations as small as possible.

9.3.2 Control methods of variations in material properties

Design is based on fractile values of material strength. As explained above, it cannot be expected that these characteristic values are reached in every batch of material. From the point of structural safety it is necessary to filter batches of bad quality by suitable test strategies (*tests of conformity*). For practical and economic reasons, tests can only be performed on a small number of specimens. Any conclusion based on the results has a stochastic character. Therefore, criteria for the judgement of concrete quality must be based on statistical methods. The basic hypothesis to be confirmed or rejected is: *The batch of concrete has a 5%-fractile value of compressive strength of x* or alternatively: *The percentage p of strength values below*

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the required characteristic value within the batch is smaller than 5%. In general, any test method and criteria will have a probability of acceptance A(p) for a batch with a certain value p.

When the conformity of concrete is tested with the nominal or characteristic values in design, there exists already some experience (*prior information*) with the material. The use of prior information is useful in the judgement of concrete quality as it reduces the uncertainties of the test strategy. For example, the standard deviation of the concrete produced in a specific plant may be known from previous tests.

The test procedure generally includes five steps. At first, the size of the batches has to be defined according to the relevant standards. According to prENV 206, the size of a batch is limited to 400 m^3 or the supply of one week or storey. From this volume of concrete, a certain number of specimens is taken (usually not less than 3). The choice of specimens is usually random, but it may be selective if there is reason to assume that certain sub-volumes (e.g. certain mixes) have a lower quality. Anyway it must be assured that the specimens are independent; for example, it does not make sense to take all specimens from one mix in 100 m^3. The third step is the testing itself and the fourth step is the statistical evaluation of the test results (see below). Based on this evaluation, finally a decision on acceptance has to be made. This may be the most difficult step, as in any case, results are not available during casting as the testing requires hardened specimens. The decision can only be made some weeks after the casting process. That means that in the worst case parts of the structure may have to be demolished and rebuilt.

Suppose that we have determined the value of a certain material property x (e.g. by a standard cube compression test) on n specimens and received n values x_i. The mean and standard deviation of the sample can be computed as[3]:

$$\bar{x} = \frac{1}{n} \cdot \sum x_i \tag{9.3-1}$$

$$s_n^2 = \frac{1}{n-1} \cdot \sum (\bar{x} - x_i)^2 \tag{9.3-2}$$

The conformity is usually checked by a double criterion of the form

$$\bar{x} \geq f_{ck} + k_1 = f_{ck} + \lambda \cdot s_n \tag{9.3-3}$$

$$x_{min} \geq f_{ck} - k_2 \tag{9.3-4}$$

where both criteria must be fulfilled. A graphical representation of this *operation characteristic* is given in Fig. 9.3-2 where the probability of acceptance is plotted over the percentage of strength values below f_{ck}. For the parameters λ and k_2 MC 90 only gives the general remark that they should be determined by statistical studies. The calibration of the parameters λ and k_2 can be done by numerical simulations and curve fitting. Generally the choice of parameters includes a contradiction between the economic interest of the producer

[3] The denominator n-1 in (9.3-2) instead of n (as one might expect) must be attributed to the fact that $\bar{x}$ is only an estimate for the mean value increasing the uncertainty in the test results.

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and the safety requirements of the client. Whereas a high probability of acceptance is economically advantageous for the producer the safety requirements of the client demand conservative values. Generally, λ and k_2 will depend on the amount of available information, i.e. the number of specimens.

An upper limit for the acceptance probability A(p) is given by

$$A(p) \cdot p \leq 0{,}05. \tag{9.3-5}$$

The application of this relation guaranties that the average fractile in batches that have passed the test is above the required value. Another limit is a *total* test performed on the *complete* batch that would give a sharp or deterministic decision. In Fig. 9.3-2 it is shown that this test criterion has the shape of the unit step function. As most tests are destructive the test criterion is hypothetical since it would destroy the complete production.

Any practical test strategy will be less sharp and have a probability of rejection unequal zero for $p < 0$ and of acceptance for $p > 0$. The shaded areas in Fig. 9.3-2 identify the remaining (economic) risk for the producer and the safety risk for the client. With an increasing number of specimens n the operation characteristic will asymptotically approach the unit step. An optimum number of n can be found by minimising the total costs of a possible rejection and tests.

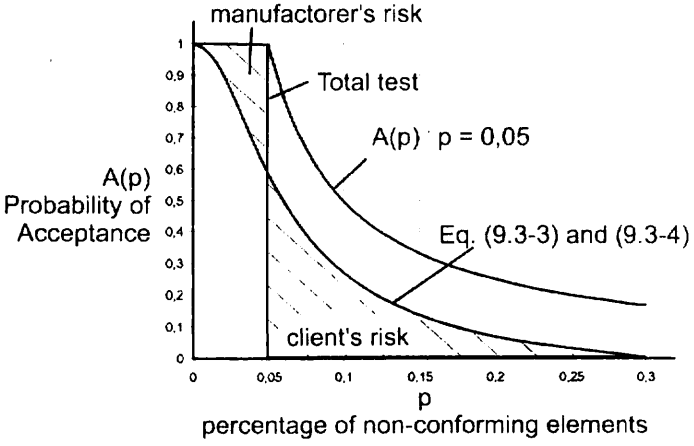


Fig. 9.3-2: Operation characteristic for tests of conformity

In some cases the assessment of the concrete quality in existing structures is required. Testing can be performed by destructive and non destructive methods. The first testing method is based on compressive tests on cores taken from the structure by drilling. The results obtained are representative for the actual behaviour as the concrete tested has been cast and hardened under conditions identical to those of the structure. In many cases, the damage caused by drilling will influence the performance and appearance of members, especially for linear members like beams or columns where the cross section would be severely reduced by the extraction of a core.

Alternatively, concrete strength can be estimated by non-destructive methods like the rebound hammer test. Compared to destructive tests, the reliability of these methods is much smaller as the relevant material property is not measured directly but via related properties like the modulus of elasticity and damping properties. A conversion formula must be based on

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a regression analysis [Rackwitz, 1976]. In addition, the results of rebound tests depend strongly on the experience and skills of the performing person. Generally, the correlation between the results of non-destructive tests and the actual concrete strength are rather weak.

9.3.3 Influences of variation in material properties

(1) Effects on safety

Variations in material properties can lead to an overestimated classification in the wrong strength class which may have an influence on the safety of the structure. The most important example to be named is the influence of scatter in strength on the safety of the structure. Especially those structural elements, of which the failure depends merely on the compression strength (columns, walls, support constructions, compressive struts in shear zones of beams, etc.) are concerned. For this reason, regular internal and external quality control is carried out. However, a certain degree of uncertainty in material properties cannot be avoided.

(2) Effects on economy

If deviations result in significant reductions of the required material properties it may come- in the worst case- to demolition and reconstruction of the structural element. If it is proved that the results of the conformity test (see 9.3.2) cannot fulfil the required design strength, the concrete manufacturer is liable for any damage and any conflict will be solved on his expenses. Therefore the manufacturer will attempt to produce concretes with a certain tolerance upwards to account not only for the uncertainties in the production but also in the test method.

Contrary to this costs and materials may be reduced if excellent quality can be assured and proved. An example is the construction of a bridge across the river Main at Veitshöchheim in 1985 (Fig. 9.3-3) [Naumann et al. (1987, 1988)]. The bridge has a total length of 1280 m with spans of 40 - 61.7 m and an arch of 162 m. The provided concrete strength class was B 45 (corresponding to a C35/45). According to German DIN 1045 (12.78), the 5% fractile value β_{WN} referring to the cube strength is multiplied with a factor 0,6 to take into account the strength difference between the test specimen and the actual structure and the effect of sustained loading ($\beta_R = 0{,}6\,\beta_{WN}$). Due to special measures in concrete technology (e.g. cooling of fresh concrete) the mean compressive strength after 90 days was 78 N/mm^2 with a standard deviation of only 2,9 N/mm^2 (5%-fractile: 73 N/mm^2). As a consequence of the good quality of the concrete it was agreed by the construction supervision and other experts, that for this case the regulations in DIN 1045 were conservative. It was decided to take this high quality into account by an increase of β_R to $0{,}7\,\beta_{WN}$ under consideration of the safety factors according to DIN 1045. This change of β_R due to negligible tolerances in material properties resulted in a significant saving of reinforcement and costs. However, such reductions must be made in a close co-operation between experienced specialists of concrete technology and construction

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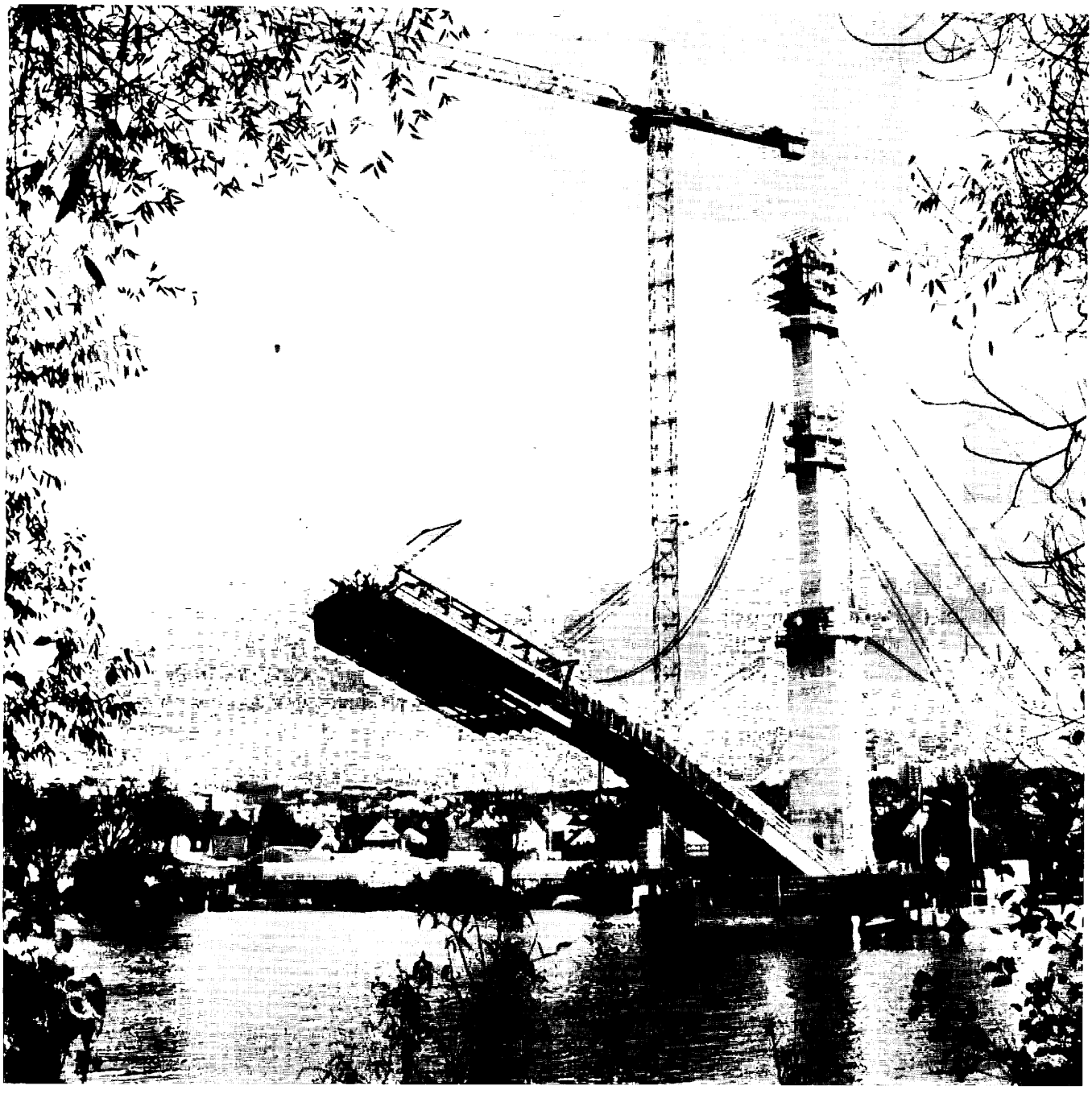


Fig. 9.3-3: Main bridge Veitshöchheim

(3) Effects on serviceability and structural appearance

The aggregates in concrete consist of natural or artificial, mineral substances agglutinated together by the cement matrix. Aggregates take a mean volume in the concrete of about 70% which means that the aggregates are quantitatively the principal components of concrete. Consequently, the elastic modulus of concrete depends to a high degree on the elastic modulus of the aggregates. In DIN 52100, properties of rocks for normal aggregates for concrete production are given. The elastic moduli of these rocks fluctuate between 2.000 N/mm^2 (siliceous sands) and 100.000 N/mm^2 (diorite and gabbro). These differences also exist

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in the field. Adjacent gravel pits may provide aggregates of different geological origin resulting in different rock properties (e.g. elastic modulus). In many codes (e.g. EC 2) the secant modulus of elasticity is a function of strength regardless of the type of aggregates used. In the MC 90 (see also chapter 3) Table 2.1.5 takes account of the effect of aggregate on modulus of elasticity.

As an example the elastic modulus according to EC2 for a C50/60 independent of the type of aggregate is 37 kN/mm^2. According to MC90 the elastic modulus for a concrete made with basalt aggregate is 46,4 kN/mm^2 and for a concrete made with sandstone aggregates is 27,0 kN/mm^2. So, when using different kinds of aggregates for the same structure, the difference in elastic modulus may result in higher deflections (up to 40%). Therefore, if concrete structures are sensitive to deflections (e.g. incremental launching, cantilevered construction bridge), it is substantial to consider an aptitude test before execution starts.

9.4 Quality management

9.4.1 General

It has been shown in the previous sections, that adequate quality in planning and execution is essential to avoid variations in material properties and geometry. Poor quality may result in damage and an increase of risk and costs. Therefore, all parties included in a building project (building owner, building contractor and staff) shall be involved in the assurance of quality. This idea is described in detail by various organisations and authors, like e.g. by CEB 234 (1990), DBV (1985), Jungwirth (1984): The building owner is obliged to specify the requirements to be fulfilled by the structure in detail. Quality means, that the building contractor tries to satisfy these requirements. Quality assurance is realised by systematic operations and control, which are summarised in a quality assurance system (quality management system, respectively). The objective is a feedback between all parties involved in the construction progress and a well functioning communication.

Technical circumstances are in a steady progress and specialisation increases proportionally. So, one process (e.g. placing, tensioning and grouting of a tendon) may require a number of different, qualified specialists. Therefore, the objective of a quality management system is to record and to co-ordinate the different processes. Quality assurance shall not only be gained by control but also by increasing the individual and personal responsibility of all parties involved in the building process. As a consequence a quality management system can attribute to organisational and qualitative efficiency and economy. As a long time effect other companies and financing institutions, respectively, are attracted to participate and invest in new projects.

9.4.2 Concept of quality management

Each project in the construction industry is different in many aspects: the location, the logistics, the erection process (complexity, subcontractors, etc.) and the work schedule (time to be taken for construction) have to be planed and organised individually. Therefore quality management has to be adjusted for each particular project. Because of this specific peculiarity of the construction industry quality management systems consists of two parts:

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- *Project quality documentations* can be regarded as a result of project quality planing. The concept and the content has to be developed for each singular project or case including specific procedures or instructions. The length and level of details depends on the size of the project and of the company, respectively. If more participants are involved (owner, designer, material supplier, etc.) a quality agreement should be made to provide a co-ordination of each project quality planning and documentation. Special attention should be drawn to the interfaces between the participants. However, adequate project quality planning does not mean unnecessary formalism but flexibility and communication of all parties.

- *Company quality systems* can be considered as a permanent organisational framework for practical work performance - independent of any particular project. These company quality systems are developed by each company under consideration of ISO 9000 providing general rules and generic concepts. However, it is not possible to standardise company quality systems since they have to be adjusted to the specific objectives and functions of the company. The concept of the company quality system should be summarised in a quality manual which is available for external bodies (partners in joint ventures, approved certifiers, etc.). In addition general procedures and work instructions should also be contained in a company quality system describing organisational or technical processes and particular work items, respectively. The structure of a quality management system can be seen in Fig. 9.4-1.

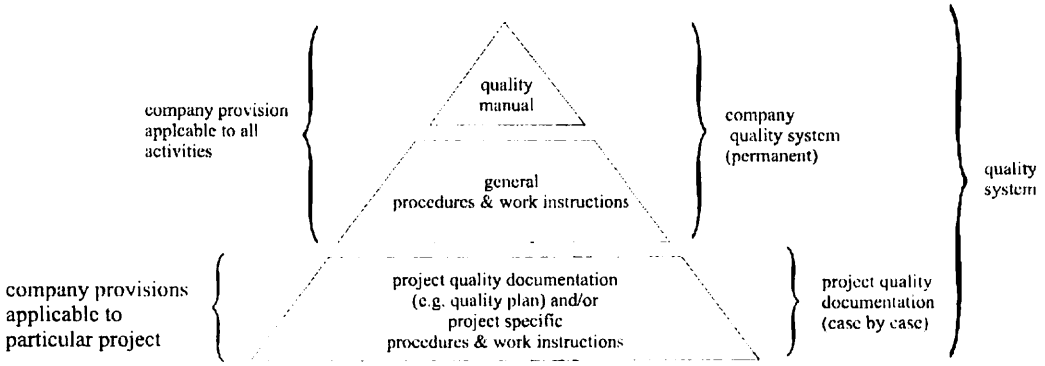


Fig. 9.4-1: Structure of quality systems in the construction industry according to Jungwirth (1984)

Since many middle class construction companies are replaced by big companies surveying of the entire company system becomes more and more complicated. Therefore it is important to implement quality management systems in the company structure by experienced and qualified personal. The quality management system must be flexible and has to be adjusted to the particular structure of the company. Transparency should be provided and everybody should have good access if information is required. The management of the company must introduce and demonstrate the system. All employees should be involved in the quality system and the responsibility of each should be increased with respect to his function. The theoretic aspects of the quality management system should be explained in training programs. Quality management is a continuous learning process, because it develops with respect to the progress of the company.

9.4.3 Quality assurance plan

The content of a quality assurance plan depends on the extend of the individual quality system. With regard to MC 90 important topics are:

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- Organisation: Design, construction and maintenance has to be co-ordinated in advance. The participants of one building project have to be determined and the functions, authority and responsibilities of each have to be defined and specified (e.g. by names or qualification). The interrelation of all participants who manage, perform and control work affecting quality have to be assured.

- Planning: The building process should be planned and described to achieve a systematic and faultless performance.

- Design control: Procedures should be established and maintained to identify *input* requirements, to perform and verify design, and to record the design changes.

- Document control: Important documents which are related to the quality of the project should be reviewed and approved regularly. Also, it has to be assured that documents are available at the right places at the right time.

- Procurement: Measures should be taken, that the services procured and the products delivered are in accordance with the specified requirements.

- Production and construction: Regular control is performed to guarantee good quality in production and construction (materials, equipment, processes and procedures, personnel, suppliers, utilities, etc.)

- Records: Performance quality records which prove that execution is performed according to the agreements and requirements, respectively. These records should be available for all parties for an agreed period after completion of the project.

These elements should be contained in a quality assurance plan. It has to be decided by the executives which elements correspond to the particular construction project and if complements have to be added.

In addition, a very important tool to assure quality are checklists - to check individual and common processes. They should be established prior to the construction in collaboration with all participants and accompany the entire building process. The checklists should be distributed to all parties contributing to a special process and during the building progress, parties should compare the checklists. This measure promotes communication and an overview of the construction process.

9.4.4 Quality control

(1) General

Since quality management depends to a high degree on the company and the particular project, quality control has to be defined according to the company quality system and to the project quality documentation. When a quality control system is established different parameters should be considered: First of all, the degree of control depends, of course, on the technical complexity and on the extend of the structure to be built. The client and the building contractor (or the building owner) should then define the intended degree of control. However it is important to adjust the degree of control to all participants involved in the project. Ideally, the quality control should base on experience and correspond to the company quality system. If the degree of control is defined, the building contractor has to inform the

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responsible personnel and discuss how to successfully implement control into the building process. A more detailed description of quality control is given in MC90.

(2) Phases of control

The definition of quality control should contain the phases where it is important to control the building process. The first process to be controlled is the planning and the design phase. In this phase, calculation models, drawings, descriptions and necessary requirements should be controlled and discussed between architect, design engineer, building owner and building contractor. If communication exists already in the planning phase, misunderstandings between the theoretical and the practical side can be avoided and defaults might be discovered in advance.

When the construction process starts, the quality and the completeness of the supplied materials and components, respectively, should be controlled- before the construction process starts as well as during construction. In the execution phase it is important to limit deviations to the specified tolerances. Therefore the quality of workmanship should be controlled and documented regularly. Also, it is important to control the interfaces of different construction processes executed at the same time and to assure good communication between all participants.

When the structure is completed the final acceptance of the structure has to be controlled and documented. In addition, a control system for the future should be determined and organised.

(3) Types of control

In quality control there are two defined types of control: production control and compliance control. However, both types of control are somehow interconnected: *Production control* relates to production and execution processes and it applies to raw materials, components of production and execution. Control can be performed by testing (also preliminary testing) but also by inspecting. Production control has the objective to discover and avoid defaults in advance - before a bad quality product is accomplished. But especially in a complex construction process, it is important not only to control during the process but also after the construction process: Therefore *compliance control* relates to the results of the production and execution processes. Compliance control is applicable to all stages of the building processes since it is important to include results of singular steps as much as possible in the quality control system. So, it is proposed to control the results of promotion, design, production and execution.

(4) Control levels

To assure independent and neutral control there have to be two control levels. Internal control has to be defined by the building contractor and to be performed by the producer and supplier, respectively. Internal control should be integrated in the production process (production control) but also include regular documentation of results (compliance control). However, internal control is only performed during the building process and not when it comes to final acceptance of the product. The controlling body has to be trained and a high degree of independence has to be guaranteed.

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External control is made by the client or by another body acting for the client. External control is important to guarantee a constant good quality of single results during the building process (compliance control). As a consequence the final acceptance is only the conclusion of a steady control process. In cases of reliable certified production, external control may be reduced to inspections, identification of the product and verification of the certificates.

9.5 Aspects in erection of RC and PC structures

9.5.1 General

The erection of in situ RC and PC structures can be subdivided into certain procedures shown in Figure 9.5.1. Especially for costly projects it is important to determine a time schedule and a construction progress chart and to co-ordinate the individual building procedures already in the planing phase.

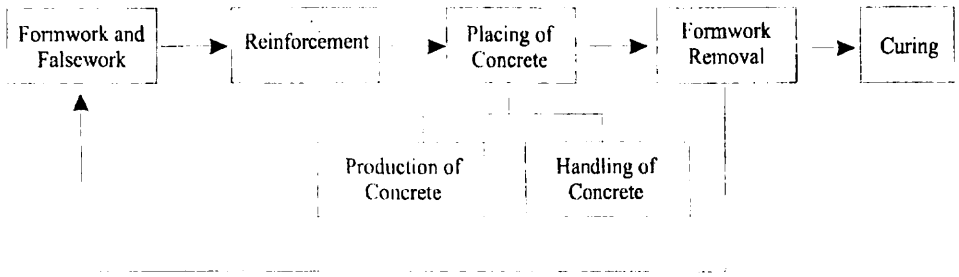


Fig. 9.5-1: Concreting procedures

Figure 9.5-2 shows the co-ordination of the individual processes at the example of a bridge. It can be seen that the columns are built first followed by the construction of the superstructure: After the arrangement of the scaffold and the formwork, the bridge is reinforced, casted, cured and prestressed. When an adequate strength is attained the bridge is striked and the scaffold and the formwork is moved forwards. The formwork consists of elements which can be adapted to the length of the different spans and consequently be reused.

9.5.2 Formwork and falsework

A major advantage of concrete compared to other materials is the possibility to form monolithic structures of almost any size and shape. On the other hand, this requires a support of concrete by formwork and falsework during its hardening process. For standard situations like slabs and columns in office or residential buildings there is a huge variety of standardised formwork systems. However, the design and construction of formwork and scaffolds for engineering structures like bridges, towers and high-rise building is a task comparable to the design of the structure itself.

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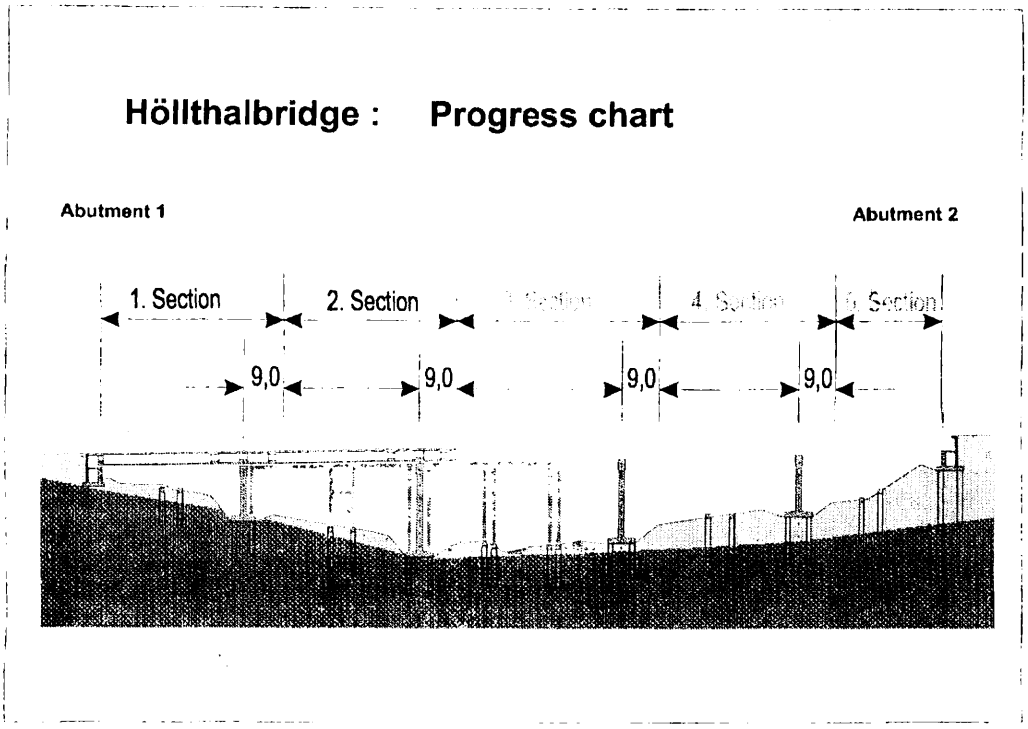


Fig. 9.5-2: Progress chart of the Höllthalbridge

(1) Formwork

In concrete structures, formwork plays an important role in terms of economy. Berner (1999) estimates that formwork and falsework takes 50% of all wages and 30% of the total costs for the preliminary building works. So, an optimal use and reuse of formwork material may result in a noticeable saving of total costs. For this reason the formwork industry is very innovative and more and more prefabricated formwork elements and systems are used. For a maximum reuse of the formwork it is advised to choose the same dimensions of footings, columns, and beams. In addition columns and the corresponding supporting elements should be adjusted to each other and be chosen that cut-outs are possibly avoided.

Formwork must assure the proper shape of the concrete structure within the specified tolerances. This requires a strict limitation of deformations and as a consequence careful design and consideration of the applying loads. Before hardening concrete causes a horizontal pressure on the formwork that depends on the rate of placement, consistency of concrete and the kind of compaction. Fig. 9.5-3 shows the distribution of pressure. In the upper parts pressure can be assumed to act as hydrostatic

$$p_{hyd} = \gamma_c \cdot x \tag{9.5-1}$$

where γ_c is the unit weight of fresh concrete. With the beginning of the solidification of concrete the increase of p with x will be slowed and finally stopped, and after hardening concrete will be able to carry its own weight without lateral pressure (though the pressure will still act in a stiff formwork, as the system of concrete and formwork is fixed in a compressed state). For practical design, simplified distributions are used (Fig. 9.5-2). The values h_s and h_h are

$$h_s = v \cdot t_s \qquad h_h = v \cdot t_h \tag{9.5-2}$$

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where v is the rate of casting in [m/h], t_s/t_h the beginning of solidification/hardening in [h]. For h_s and h_h see the relevant codes [German DIN 18218 or ACI – Guide to formwork for Concrete (1983)]. For special casting techniques, e.g. where concrete is pumped from the base, p can exceed the hydrostatic pressure.

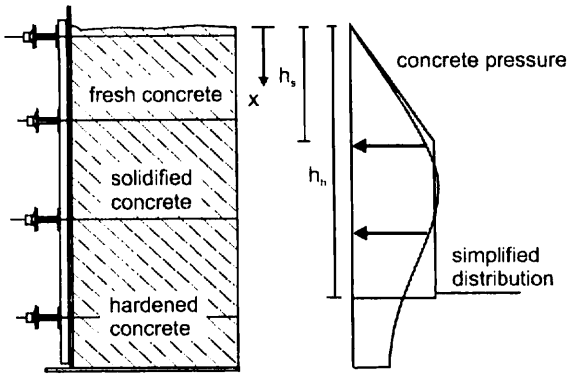


Fig. 9.5-3: Horizontal pressure of concrete

In the planing process the architect has to define the finish of the concrete structure. This choice depends on the kind of structure and on the visibility of the structural element. The parameters of exposed concrete to be varied are the colour (almost white to a dark grey-depending mainly on the cement and the water/cement ratio) and the structure of the concrete surface. According to the demands of the architect or other requirements (low production time, high degree of reuse, technical demands) the kind of formwork is chosen:

- Planed and unplaned planking is the traditional formwork material. The advantage of this type of formwork is, that the desired surface finish can be influenced by quality of the surface of the planking. As a consequence the degree of tolerances and the structure of the future concrete surface can be determined and the planking can be treated accordingly. Another advantage is that wood is very well suited for absorbing the bleeding water at the concrete surface. As a consequence the concrete shows a good quality with regard to durability. In terms of economy planking has the disadvantage that it normally can be used only once. In the case of unplaned planking, special attention has to be drawn to the fact to use a good quality form oil, otherwise singular wood fibres may adhere to the finish which should possibly be avoided in case of architectural concrete.

- Plywood sheets can be manufactured by building workers on site or be ordered by special formwork suppliers. Formwork suppliers offer ready made formwork systems but they also design the formwork system for individual projects. Sheet materials are available in all different kinds: plywood sheets from low cost untreated materials to high cost resin film coated boards. Therefore, the advantages and disadvantages depend on the material used. However, when considering large-purpose panels for repeated use, high costs may be justified, because good quality of the panels results in an increase of using times. A disadvantage of many plywood sheets is that the absorbing properties are poor. This may result in a bad concrete surface especially, when a concrete with high w/c-ratio is placed.

- Metal panels are normally used in bigger projects because otherwise they are too expensive in purchase. On the other side they can be reused for a very long time. So

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often, they are used for permanent construction, e.g. for precast concrete elements. When metal panels are used on site cranes are needed for their transportation and placing. Mostly, they are used in special constructions, like for example as slide formwork.

(2) Falsework

From the point of structural safety, scaffolds are very sensitive to errors in design and execution as may be seen from the rather large number of failures during construction (compared to the number of failure during service). This may be attributed to several facts:

The dominant action effect on falsework is the dead weight of concrete. As it was explained in 9.2.1, for the dead weight the characteristic value is generally set equal to the mean value. Therefore, the characteristic value of loads will be reached or even exceeded with a very high probability by definition. In structures under service conditions a considerable part of the (nominal) action is usually the live load with a low probability of being exceeded during lifetime. So the design loads on the structure may be considered more or less fictitious whereas the design loads during construction are realistic.

For economic and practical reasons the connections between the elements of the scaffold are often hinges resulting in statically determined systems and loaded close to their limit. Unlike most other structures, the nominal capacity is very close to the actual capacity whereas RC structures usually have some hidden reserves not taken into account in design like monolithic connections considered as hinges, additional reinforcement etc.

Finally it must be said that due to the fact that they are only temporary constructions falsework quite often is not designed and executed with the required precaution.

The vertical actions of fresh concrete can easily be calculated from its density and the member size. However, when concrete is dropped from a certain height it might be necessary to consider the dynamic effects of falling concrete. For the design of the scaffold, transitory states must be taken into account as the loading of single fields in a continuous beam. If the scaffold is sensitive to this (e.g. scaffolds for arches) or if it would be uneconomic to design for this most unfavourable combination, it is possible to apply the concrete load in an order that avoids this combination. However, in this case the casting sequel must be prescribed precisely by the designer.

In many cases the relevant criterion for the design of scaffolds is not the strength, but the stiffness. The estimation of stiffness must take into account shear deformations, deformations in connections of the scaffold and a settlement of the foundation. Wooden scaffolds may also show significant deformations due to drying shrinkage or swelling. To adjust unavoidable deformations the formwork may be banked. The required camber must be determined by an adequate calculation of formwork deformation and not be chosen to large, as a negative sag is often less desirable than a positive one.

The amount of required scaffold material also depends on the period of striking. In general, this will depend on the development of concrete strength. As loading of concrete at early ages may lead to extended micro cracking and increased creep, generally having a strong influence on the serviceability and durability, criteria for striking can not be derived from ULS considerations alone.

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9.5.3 Curing

The last important step in the concrete production is curing. In practice curing is often underestimated since the benefits with regard to short-time-effects are barely obvious. But with regard to long time effects (especially to guarantee an adequate durability of a structure), adequate curing should be performed after placing of the concrete.

According to Spear (1983) curing can be defined as a procedure for insuring the hydration of cement in outer concrete layers (concrete skin). Powers found out in (1947) that the process of cement hydration stops below a relative humidity of 80 %. Consequently if fresh concrete is exposed to a relative humidity below 80% the concrete strives for humidity equilibrium with the environment. In other words, the water evaporates since the internal humidity is adapted to the environmental conditions. Direct exposure to sun or wind accelerates this process -especially close to the surface. As a result, the hydration of concrete in the surface layers slows down or even ceases. Although this reduction of hydration rate does not mean a significant loss in the overall strength of the structure, it may result in a high porosity of the surface layer. Consequently, the concrete is more susceptible to abrasion and, when exposed to unfavourable environmental conditions, also to carbonisation. For the sake of avoidance it is important to provide adequate curing of the concrete.

Curing means to keep the concrete moist or avoid drying. For the practical engineer it is important to know, how long the concrete has to been cured. The duration depends on several factors:

- Curing sensitivity of the concrete as influenced by its composition
- Concrete temperature
- Ambient conditions during and after curing
- Exposure conditions of the finished structure

The curing concept given in the Model Code 90 is related to the rate of development as a function of w/c ratio and class of cement (MC 90, Table d.11)) and the environmental conditions (MC 90 Table d.10). The minimum curing duration is determined as a function of the rate of development of impermeability and the environmental conditions. The minimum duration time is then described in days.

In the new European standard generation there has been developed a new curing concept. According to Grübl (1996), concrete properties with regard to durability are influenced by three material parameters:

- water-cement ratio
- binder
- hydration degree.

In prEN 206, first an environmental class has to be determined to which the concrete structure will be exposed. These classes describe the environment as a function of carbonation corrosion, chloride corrosion, freeze/thaw cycle, chemical attack and abrasion. According to the environmental classes, limiting values for composition and properties of concrete are provided (Maximum w/c ratio, minimum strength class of concrete, minimum cement content and minimum air pores content). By using this concept which considers the material parameters w/c ratio and binder, a good quality concrete mix with regard to durability (carbonisation, strength and compaction) can be achieved.

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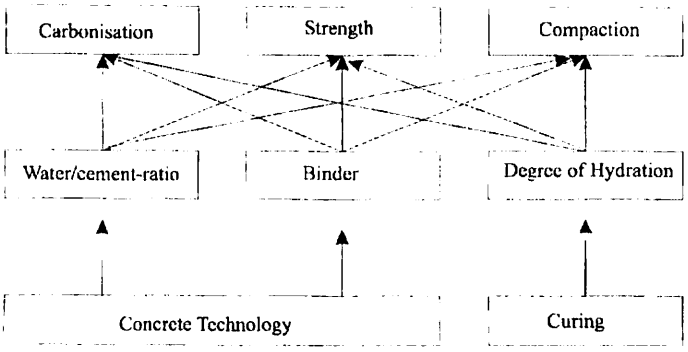


Fig. 9.5-5: Dependency of the properties which materially determine the durability of concrete on the technological parameters according to Grübl (1996)

Fig. 9.5-5 shows that w/c ratio and binder are covered by the concrete technology. The idea of the new curing concept is now, to determine curing as a function of degree of hydration. As a consequence it is possible to assure that the concrete on the construction site - especially in the surface layers- corresponds to the composition specified by the concrete technology. As criterion to judge the degree of hydration the strength has been chosen, because compared with carbonisation and compaction, more experience and testing methods are available.

In the curing concept of the future European standard the duration of curing is defined in days. The concrete should have a certain strength related to the 28-day-strength when the curing duration is finished. This strength ratio is a function of climate conditions (humid, moderate, dry and very dry) defined by the average relative humidity of a certain region. This has the background that hydration does not cease at the end of curing if there is a certain water supply given by a humid environment. Therefore, the duration of curing at a high outside relative humidity can be shorter compared to dry environments. As a consequence, the environmental conditions as used in the old codes (e.g. MC 90) are not required any more. When the strength ratio is determined the time of curing can be chosen as function of strength ratio as presented in Fig. 9.5-6.

The next question was how this concept can be realised in design and how the strength of the concrete at a certain time can be tested on site, if great accuracy is required. The easiest case for the designer is to determine the duration of curing according to a table which directly gives the days of duration as a function of climate and strength development of the concrete. The values given in this table are determined with respect to the concept described above. Of course, these values are on the safe side. Duration time can be saved, if a more accurate method is applied. For this method the exact duration, at which the concrete achieves the demanded strength ratio at a temperature of 20°C, needs to be known (e.g. to be determined by aptitude tests). In cases of deviation, the influence of temperature on strength development has to be considered. This is possible by applying so-called *maturity functions*, which describe the relation between strength development and temperature.

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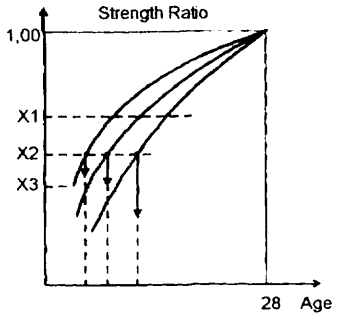


Fig. 9.6-2 Strength developments of European concretes with a W/C ratio of 0,6 according to Grübl (1996)

There are two exceptions deviating from the concept that concrete has to be cured until a certain strength ratio has to be developed. The first exception is made for structures not exposed to any moisture (e.g. interior members). In these cases, curing may be reduced to 12 maturity hours. The other exception concerns surfaces which need exceptionally high resistance against abrasion. In this case the respective strength ratio is increased by a factor 0.2.

9.6 Prestressing

Prestressing leads to an improved behaviour in the serviceability limit state. Stiffness will be increased and cracking can be avoided under severe environmental conditions. So prestressing has become an important construction method even though planing and execution of prestressed structures requires a higher technical input and experience than reinforced concrete structures. This is manifested by the fact that the quality standards for the manufacture of prestressing devices but also for the execution of prestressed structures are very high. However the widely spread use and the gained experience has lead to a huge variety of different prestressing devices and prestressing methods (see Chapter 3.2), as well as to sophisticated design models. In the following some important aspects are described when planing and executing prestressed structures.

9.6.1 Time of prestressing

In many cases the use of prestressed structures requires further considerations during construction. It should be kept in mind that under construction prestress may become the dominant action effect before the full dead load and service load is applied. So, the transient states may be decisive in design or require a certain prestressing procedure.

Post tensioned structures can either be prestressed in one or several steps to avoid increased unfavourable combinations of prestress and external loads. For prestressing in one step it is important that the concrete should have hardened to a certain degree to avoid increased creep by early mikrocracking and subsequent losses of prestress. The age of concrete when prestress may be applied depends on the strength development and therefore on the type of cement and the temperature and curing conditions. However the strength should be sufficient to support the applied prestress (proved by hardening tests). In general, these strength values are given in approval documents. The required strength for full application of prestress is between 80% and 90%.

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In the case of greater compact cross sections it is advised to consider several steps of prestressing. Considerable internal stresses may occur because of the non-uniform distribution of hydration heat and shrinkage strains causing cracking of the concrete that has not yet its full strength. To avoid this a low percentage of prestress can already be applied in the first days. It is important though not to exceed a certain percentage of the low early strength of concrete (as already mentioned before up to 20%). The remaining prestress can then be fully applied after an adequate hardening time. In many cases it may be useful to apply prestress step by step. The degree of prestress that is applied in the first step can be chosen to equilibrate the already installed dead load and then be increased according to the advance of the construction process.

9.6.2 Effects of prestressing during construction

Structures are often prestressed to such degree that dead loads are taken by the generated prestressing force. Before the removal of the scaffold, at least parts of the prestress must be applied to take the dead loads and as a consequence to avoid the appearance of too high stresses resulting in cracking or even failure. In general the prestressing forces will cause deformations of the concrete member and thus a considerable redistribution of the forces within the scaffold. Depending on the stiffness ratio of the concrete member and the scaffold[4] it may come to unfavourable effects.

In general distinction can be made between a very stiff scaffold construction or a rather elastic one. In Figure 9.6-1, three examples are shown were problems may occur. In Figure 9.6-1 a, the scaffold is very stiff. So the dead weight of the concrete member applied during casting does not result in a deformation of the scaffold. So if a prestressing force exceeding the dead load is applied the member will be lifted off the scaffold. Thus, the whole dead load will be activated to reduce the unfavourable effects of the prestress.

However, in cases where the scaffold is rather elastic (Figure 9.6-1 b) the deformations due to prestressing will hardly reduce the supporting forces from the scaffold (spring back effect). In the limiting case $EI_{conc}/Ei_{scaf} \rightarrow \infty$, no dead weight will act on the member as it is still carried by the scaffold girder. The member is therefore subjected to prestressing effects alone. This leads to moments equal to the ones caused by dead load but of opposite sign and requires adequate consideration in design.

In the last example (Figure 9.6-1 c) the scaffold girder is supported by three columns taking the dead weight of the concrete member. If the deformations of the scaffold remain small, the same phenomena as shown in the first example may happen. The concrete member rises in such way that the whole weight has to be taken by the outer columns. So, if the columns are only dimensioned for 1/4 F it may come to a collapse of the scaffold system.

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[4] The stiffness of the scaffold depends on many parameters (e.g. the number of flying shores, the stiffness of the flying shores, settlements, the stiffness of the scaffold girder) and is therefore difficult to determine.

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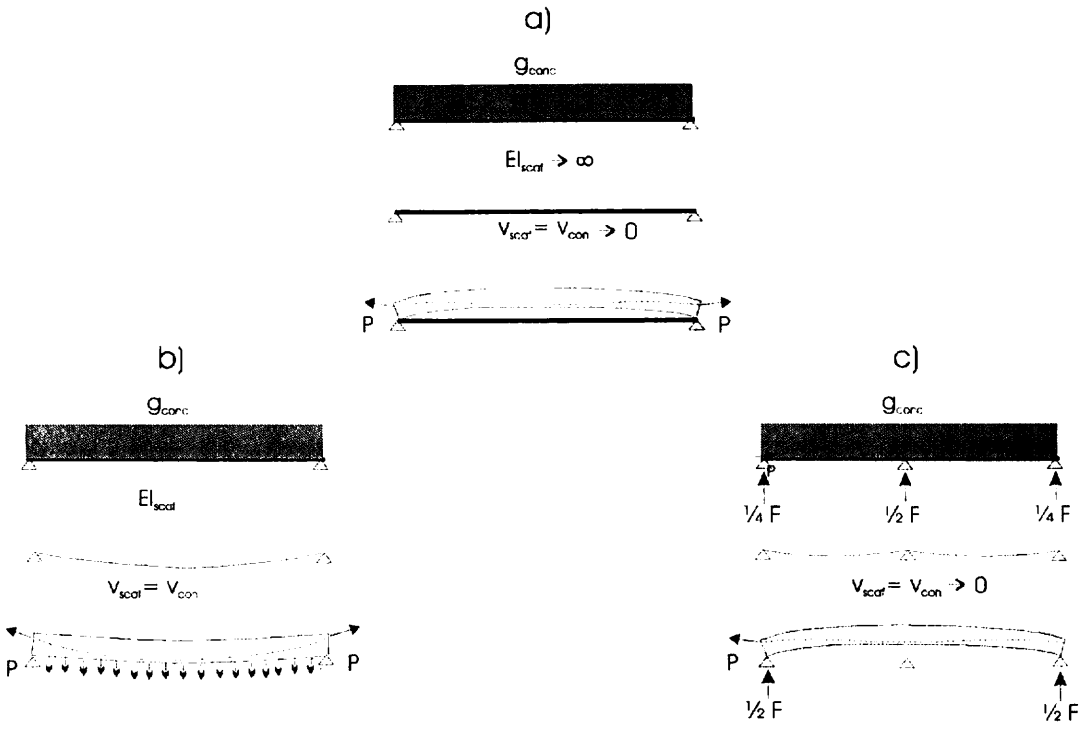


Figure 9.6-1: Prestressed beam on a) stiff scaffold, b) elastic scaffold and c) scaffold supported by columns

It can be seen that prestressing is a susceptible construction method. In many cases the design engineer has to calculate and design not only for the final condition of the structure but also for transient situations. Especially in bridge design noticeable inner redistributions of forces and stresses occur and lead to different statical systems. The engineer should be aware of this and consider all transient conditions during construction.

9.7 Precast elements and structures

9.7.1 General

MC 90 defines that *precasting of structural elements derives from the philosophy of industrialisation of the production processes, aimed at improving economy and quality of buildings.* In precast plants a higher quality level is possible because of the constant production conditions (e.g. weather conditions, temperature, aimed concrete strength), lower production costs (reduction of costs for formwork and falsework) and lower production time (parallel production of wall- and floor systems, production during winter).

When using precast elements it is important to choose an adequate modular co-ordination. The choice depends on the function of the building – the module of a parking garage is different than that of a hospital. According to the modular co-ordination the dimensions of the precast elements (cross-section, length) can then be determined. In this respect it is also important to determine the allowable tolerances (see also section 9.2.2(4)). In the dimensioning process transport and erection play also an important role. The size of members are limited by the size of transport vehicles but should be as large as possible for financial reasons.

After transport of the precast elements to the construction site they are connected. Special attention should be paid to an adequate bracing of the structure. Columns (for 1 or 2 floors) or wall panels, frames, bracings or cores (for more than 2 floors) can be used for horizontal

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bracing possibly jointed to stiff ceiling panels – the arrangement of bracing elements should be planed and executed according to the accepted rules.

Generally analysis and design of precast structures should be carried out with respect to the ultimate and serviceability limit state requirements of in-situ structures. Modifications or supplementary requirements are given in MC 90 or other Codes (e.g. ENV 1992-1-3). In analysis special attention should be paid to the following aspects:

- The behaviour of structural units as such (this may be important in transitory situations).
- The behaviour of structural assemblies, in particular with regard to actual deformability, strength and fatigue resistance.
- The uncertainties influencing restraints and force transmission between elements, that may depend on errors in geometry and in the positioning of units and bearings.
- All external and internal actions intervening in all phases (e.g. possible changes of the statical system); some non-conventional conditions may be envisaged (e.g. for dimensioning against overall instability, explosions, impacts, progressive collapse).

In design the limit states have to be verified but it is also important to consider an easy and reliable assembly and maintenance of the precast elements and the connections. In addition, the designer should account for all static and dynamic transitory situations occurring for example during transport (verification for combinations of actions such as wind, dead weight and possible superimposed loads during assembly). Since the quality of precast elements is usually very high, MC 90 allows a reduction of the partial safety factors γ_c (only for precast elements, not for joints) and γ_s provided that there is a complete quality assurance system supervised by an independent control body. Reduction values are given in MC 90.

Nowadays the precast industry offers a great number of different elements as well as whole system units (floor systems, wall systems, beam and column systems and segmental systems). For these systems, design criteria and detailing rules have been developed which are generally approved by the building authorities and which are specified in standards for precast elements (e.g. ENV 1992-1-3). In this chapter these precast systems will not be further treated.

9.7.2 Joints

Cast-in-situ structures are manufactured by a whole casting process. Precast structures are manufactured by single elements which are connected later to an entire structure. The connecting devices are ususally determined and designed during the planing process and then somehow installed during the manufacturing process (e.g. fixed at the formwork). For smaller loads they can also be applied afterwards. According to MC 90 the design of joints should be based on the following assumptions:

- The joint is able to accommodate the relative displacements needed to mobilise the resistance of the joint.
- The joint is able to resist all action effects resulting from the analysis of the structure as a whole as well as those resulting from the analysis of the individual members.
- The strength and deformability of the joints secure a robust and stable behaviour of a structure as a whole.
- Account is taken of the tolerances anticipated in manufacture and erection.

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The resistance and stiffnes of these joints is often generally approved or can be determined by analytical models or experimental tests. With regard to the different action effects to be transmitted the following joints are distinguished:

- Compression joints (dry joints, bedded joints)
- Shear joints (plane joints, key joints)
- Flexural and tensile joints (reinforced joints, welded joints)

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NOTATION LIST

Roman lower case letters:

$1/r$ ..... curvature of a section of an element
$1/r_{(g)}$ ..... curvature due to g
$1/r_{(g+q)}$ ..... curvature due to g and q
$1/r_{0(g+q)}$ ..... instantaneous (initial) curvature due to g and q
$1/r_{I}$ ..... curvature of an uncracked concrete section (state I)
$1/r_{Ir}$ ..... curvature in state I under cracking moment
$1/r_{2}$ ..... curvature of a cracked concrete section (state II)
$1/r_{2r}$ ..... curvature in state II under cracking moment
$1/r_{ts}$ ..... tension stiffening correction for curvature
a ..... deflection
a_c ..... elastic deflection (calculated with rigidity $E_c I_c$)
b ..... breadth of compression zone or flange
b_{red} ..... reduced breadth of web
b_x ..... smaller side dimension of a rectangular section
b_y ..... greater side dimension of a rectangular section
b_w ..... breadth of web
c ..... concrete cover, concentration of a substance in a volume element
c_1 ..... column dimension parallel to the eccentricity of the load
c_2 ..... column dimension perpendicular to the eccentricity of the load
c_{min} ..... minimum concrete cover
c_{nom} ..... nominal value of concrete cover (= c_{min} + tolerance)
d ..... effective depth to main tension reinforcement
d' ..... effective depth to compression reinforcement
d_{max} ..... maximum aggregate size
e ..... load eccentricity
e_0 ..... first order eccentricity (= M_{Sd} / N_{Sd})
e_{01} ..... smaller value of the first order eccentricity at one end of the considered element
e_{02} ..... greater value of the first order eccentricity at one end of the considered element
e_{tot} ..... total eccentricity
f_{bd} ..... design value of bond stress
f_c ..... cylinder compressive strength of concrete
f_c^* ..... cylinder compressive strength of concrete under triaxial loading (confined strength), reduced concrete strength due to transverse tension
f_{cc} ..... cylinder compressive strength of concrete under uniaxial stress
f_{cd}^* ..... design compressive strength of concrete under triaxial loading (confined strength), reduced design concrete strength due to transverse tension
f_{cd} ..... design value of f_c
f_{cd1} ..... average design strength value in an uncracked compression zone
f_{cd2} ..... average design strength value in a cracked compression zone
$f_{cd,fat}$ ..... design fatigue reference strength of concrete under compression
f_{ck} ..... characteristic value of f_c
$f_{ck,cf}$ ..... value of f_{ck} of confined concrete
$f_{ck,cube}$ ..... characteristic value of cube compressive strength of concrete
$f_{ck,fat}$ ..... fatigue reference compressive strength
f_{cm} ..... mean value of compressive strength f_c at an age of 28 days

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f_{ct} ..... axial tensile strength of concrete (determined according to RILEM CPC 7)
f_{ctd} ..... design value of f_{ct}
f_{ctk} ..... characteristic value of f_{ct}
f_{ctm} ..... mean axial tensile strength
$f_{ct,fl}$ ..... mean flexural tensile strength (at T = 20°C)
$f_{ct,sp}$ ..... mean splitting tensile strength
f_d ..... design value of strength
$f_{p0,1}$ ..... 0,1% proof stress of prestressing reinforcement
$f_{p0,2}$ ..... 0,2% proof stress of prestressing reinforcement
$f_{p0,1k}$ ..... characteristic 0,1% proof stress
$f_{p0,2k}$ ..... characteristic 0,2% proof stress
f_{pt} ..... tensile strength of prestressing reinforcement
f_{ptd} ..... design tensile strength of prestressing reinforcement
f_{ptk} ..... characteristic tensile strength of prestressing reinforcement
f_{py} ..... tension yield stress of prestressing reinforcement
f_{pyd} ..... design value of tension yield stress of prestressing reinforcement
f_{pyk} ..... characteristic value of tension yield stress of prestressing reinforcement
f_R ..... relative (or projected) rib area
f_t ..... tensile strength of non- prestressing reinforcement
f_{tk} ..... characteristic value of tensile strength of non- prestressing reinforcement
f_y ..... tension yield stress of non- prestressing reinforcement
f_{yc} ..... strength of steel in compression
f_{ycd} ..... design strength of steel in compression
f_{yd} ..... design value of tension yield stress of non- prestressing reinforcement
f_{yk} ..... characteristic value of tension yield stress of non- prestressing reinforcement
g_d ..... design value of distributed permanent load
h ..... overall depth of member, total height; notional size of a member (2 A_c/u; u: perimeter in contact with the atmosphere)
h_b ..... depth of beam
h_f ..... depth of flange
Δh_w ..... height of water column
i ..... radius of gyration
l ..... design span, effective span, length of an element, thickness of a penetrated section
Δl ..... measured elongation between two measuring points
l_0 ..... design lap length, effective length (of columns); distance between measuring points
l_b ..... basic anchorage length
l_{bp} ..... basic anchorage length of pretensioned reinforcement
l_{bpd} ..... design anchorage length of pretensioned reinforcement
l_{bpt} ..... transmission length of pretensioned reinforcement
$l_{b,min}$ ..... minimum anchorage length
$l_{b,net}$ ..... design anchorage length
l_{ch} ..... characteristic length (fracture parameter)
l_p ..... development length for prestressing reinforcement
l_{pl} ..... plastic length (region in which tensile strain is larger than yield strain)
Δl_{pl} ..... residual elongation after unloading
$l_{p,max}$ ..... length over which the slip between prestressing steel and concrete occurs
$l_{s,max}$ ..... length over which the slip between steel and concrete occurs
l_t ..... transmission length
m ..... moment per unit width (out-of-plane loading); mass of substance flowing; degree of hydration

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n ..... number of bars, number of load cycles; force per unit width (in-plane-loading)
n_{Ri} ..... number of cycles leading to failure at stress levels $S_{i,min}$ and $S_{i,max}$, respectively
n_{Si} ..... number of cycles applied at constant minimum and maximum stress levels $S_{i,min}$ and $S_{i,max}$, respectively
p ..... local gas pressure
q ..... distributed variable load
q_d ..... design value of distributed variable load
r ..... radius
s ..... slip (relative displacement of steel and concrete cross-sections), shear slip (at interfaces); spacing of bars
s_{max} ..... maximum bar spacing
s_r ..... distance between cracks; radial spacing of layers of shear reinforcement
$s_{r,m}$ ..... mean spacing between cracks
t ..... time, age, duration; thickness of thin elements
t_0 ..... age at loading
t_s ..... concrete age at the beginning of shrinkage or swelling
t_T ..... effective concrete age
u ..... length of a perimeter; component of displacement of a point
u_0 ..... length of the periphery of the column or load
u_l ..... length of the control perimeter for punching
u_{ef} ..... length of the perimeter of A_{ef}
u_n ..... length of the control perimeter for punching outside a slab zone with shear reinforcement
v ..... shear force per unit width (out-of-plane loading), component of displacement of a point
w ..... crack width; component of displacement of a point
w_c ..... crack width for $\sigma_{ct} = 0$
w_k ..... calculated characteristic crack width
w_{lim} ..... nominal limit value of crack width
x ..... depth of compression zone, distance
z ..... internal lever arm

Greek lower case letters:

α ..... coefficient, reduction factor
α_e ..... modular ratio (E_s / E_c)
$\alpha_{e,p}$ ..... modular ratio (E_p / E_c)
$\alpha_{e,sec}$ ..... secant modular ratio ($E_{s,sec}$ / $E_{c,sec}$)
α_{sT} ..... coefficient of thermal expansion for steel
α_T ..... coefficient of thermal expansion in general
β_1 ..... coefficient characterizing the bond quality of reinforcing bars
$\beta_c(t,t_0)$ ..... coefficient to describe the development of creep with time after loading
γ ..... safety factor
γ_c ..... partial safety factor for concrete material properties
$\gamma_{c,fat}$ ..... partial safety factor for concrete material properties under fatigue loading
γ_F ..... partial safety factor for actions
γ_G ..... partial safety factor for permanent actions
γ_Q ..... partial safety factor for variable actions
γ_s ..... partial safety factor for the material properties of reinforcement and prestressing steel

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$\gamma_{s,fat}$ ..... partial safety factor for the material properties of reinforcement and prestressing steel under fatigue loading
δ_{ii} ..... node displacement
ε ..... strain
ε_c ..... concrete compression strain
ε_c* ..... concrete compression strain under triaxial stress
ε_{cm} ..... average concrete strain within $l_{s,max}$
ε_{c0} ..... concrete strain at peak stress in compression
$\varepsilon_{cc}(t)$ ..... concrete creep strain at concrete age $t > t_0$
$\varepsilon_{ci}(t_0)$ ..... stress dependent initial strain at the time of stress application
$\varepsilon_{cn}(t)$ ..... total stress independent strain at a concrete age t ($=\varepsilon_{cs}(t) + \varepsilon_{cT}(t,T)$)
$\varepsilon_{cs}(t,t_s)$ ..... total shrinkage or swelling strain at concrete age t (t in days)
$\varepsilon_{c\sigma}(t)$ ..... total stress dependent strain at a concrete age t ($= \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t)$)
ε_{ct} ..... concrete tensile strain
$\varepsilon_{cT}(t,T)$ ..... thermal strain at a concrete age t
ε_{cu} ..... ultimate strain of concrete in compression
ε_{d0} ..... strain of prestressed reinforcement corresponding to P_{d0}
ε_{pu} ..... total elongation of prestressing reinforcement at maximum load
ε_r ..... strain at the onset of cracking
ε_s ..... steel strain
ε_{s1} ..... steel strain in uncracked concrete
ε_{s2} ..... steel strain in the crack
ε_{sm} ..... mean steel strain
$\Delta\varepsilon_{sr}$ ..... increase of steel strain in cracking state
ε_{sr1} ..... steel strain at the point of zero slip under cracking forces
ε_{sr2} ..... steel strain in the crack under cracking forces (σ_{ct} reaching f_{ctm})
ε_{sT} ..... thermal strain of steel
ε_{su} ..... strain of non- prestressing reinforcement at maximum load
$\Delta\varepsilon_{ts}$ ..... increase of strain by the effect of tension stiffening
ε_u ..... total elongation of reinforcing steel at maximum load
ε_{uk} ..... characteristic total elongation of reinforcing steel at maximum load
ε_{yd} ..... design yield strain of non - prestressing reinforcement ($= f_{yd} / E_s$)
ε_v ..... transverse contraction
ζ ..... ratio of bond strength of prestressing steel and high-bond reinforcing steel
η ..... viscosity of gas
θ ..... angle between web compression and the axis of a member; rotation
θ_f ..... angle between inclined compression in a flange and the axis of the member
λ ..... slenderness ratio ($= l_0 / i$)
μ ..... coefficient of friction, relative bending moment
ν ..... relative axial force
ν_c ..... Poisson's ratio of concrete
ν_s ..... Poisson's ratio of steel
ν_{Sd} ..... relative design axial force ($= N_{Sd} / A_c f_{cd}$)
ρ ..... ratio of (longitudinal) tension reinforcement ($= A_s/bd$)
$\rho_{s,ef}$ ..... effective reinforcement ratio ($= A_s/A_{c,ef}$)
ρ_t ..... relaxation after t hours
ρ_w ..... ratio of web reinforcement ($= A_{sw}/b_w s \cdot \sin\alpha$)
σ ..... stress
$\sigma_1, \sigma_2, \sigma_3$ .. principal stresses
σ_c ..... concrete compression stress

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σ_{cd} ..... design concrete compression stress
σ_{ct} ..... concrete tensile stress
$\sigma_{c,cf}$ ..... compression stress of confined concrete
$\sigma_{c,max}$ ..... maximum compressive stress
$\sigma_{c,min}$ ..... minimum compressive stress
$\sigma_{p0}(x)$ ..... initial stress in prestressing reinforcement at a distance x from anchorage device
$\sigma_{p0,max}$ ..... maximum tensile force in prestressing reinforcement at tensioning
σ_{pcs} ..... tendon stress due to prestress after all losses (due to creep and shrinkage)
σ_{pd} ..... tendon stress under design load
$\Delta\sigma_{Rsk}(n)$ .. stress range relevant to n cycles obtained from a characteristic fatigue strength function
σ_s ..... steel stress
σ_{s2} ..... steel stress in the crack
σ_{sE} ..... steel stress at the point of zero slip
σ_{sr2} ..... steel stress in the crack under crack loading (σ_{ct} reaching f_{ctm})
$\Delta\sigma_{Ss}$ ..... steel stress range under the acting loads
τ_b ..... local bond stress
τ_{bm} ..... mean bond stress
$\tau_{fu,d}$ ..... ultimate design shear friction capacity
τ_{max} ..... maximum value of bond stress
τ_{Rd} ..... resistance to shear stress (design value)
τ_{Sd} ..... applied shear stress (design value)
$\psi(t,t_0)$ ..... relaxation coefficient
ω ..... mechanical reinforcement ratio
ω_{sw} ..... mechanical ratio of stirrup reinforcement
ω_v ..... volumetric ratio of confining reinforcement
ω_w ..... volumetric mechanical ratio of confining reinforcement
ω_{wd} ..... design volumetric mechanical ratio of confining reinforcement

Roman capital letters:

A ..... total area of a section or part of a section (enclosed within the outer circumference)
A_1 ..... section area in state I (taking into account the reinforcement)
A_c ..... area of concrete cross section or concrete compression chord
$A_{c,ef}$ ..... effective area of concrete in tension
A_{core} ..... effectively confined area of cross-section in compression
A_{ef} ..... area enclosed by the centre-lines of a shell resisting torsion
A_p ..... area of prestressing reinforcement
A_s ..... area of reinforcement
A_s' ..... area of compressed reinforcement
A_{sh} ..... area of hoop reinforcement for torsion
A_{sl} ..... area of longitudinal reinforcement
A_{st} ..... area of transverse reinforcement
A_{sw} ..... area of shear reinforcement
$A_{s,cal}$ ..... calculated area of reinforcement required by design
$A_{s,ef}$ ..... area of reinforcement provided
$A_{s,min}$ ..... minimum reinforcement area
D ..... fatigue damage, diffusion coefficient
D_{lim} ..... limiting fatigue damage
E ..... modulus of elasticity

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E_c ..... reduced modulus of elasticity for concrete
$E_c(t_0)$ ..... modulus of elasticity at the time of loading t_0
E_{ci} ..... tangent modulus of elasticity at a stress σ_i (at T = 20°C)
$E_{c,sec}$ ..... secant modulus of elasticity at failure for uniaxial compression ($E_{c,sec} = f_{cm}/|\varepsilon_{c0}|$)
E_p ..... modulus of elasticity of prestressing steel
E_s ..... modulus of elasticity of steel
$E_{s,sec}$ ..... secant modulus of elasticity of steel
F ..... force, applied load or load effect
F_b ..... bond force transmitted along the transmission length
F_c ..... strut force (compression force)
F_d ..... design value of action
F_{pt} ..... tensile load of prestressed reinforcement
$F_{p0,1}$ ..... characteristic 0,1% proof -load
$F_{Sd,ef}$ ..... effective concentric load (punching load enhanced to allow for the effects of moments)
F_t ..... tie force (tension force)
F_{ud} ..... ultimate dowel force
G ..... permanent action
G_F ..... fracture energy of concrete
G_{F0} ..... base value of fracture energy (depending on maximum aggregate size)
G_{inf} ..... favourable part of permanent action
G_{sup} ..... unfavourable part of permanent action
H ..... horizontal force, horizontal component of a force
I ..... second moment of area
I_1 ..... second moment of area in state I (including the reinforcement)
I_2 ..... second moment of area in state II (including the reinforcement)
I_c ..... second moment of area of the uncracked concrete cross-section (state I)
$J(t,t_0)$ ..... creep function or creep compliance representing the total stress dependent strain per unit stress
K_g ..... coefficient of gas permeability
K_w ..... coefficient of water permeability
L ..... span, length of an element
M ..... bending moment; maturity of concrete
M_r ..... cracking moment
M_{Rd} ..... design value of resistant moment
M_{Sd} ..... design value of applied moment
M_u ..... ultimate moment
M_y ..... yielding moment
N ..... axial force, number of cycles to failure (fatigue loading)
N_r ..... axial cracking force
N_{Rd} ..... design value of resistance to axial force
N_{Sd} ..... design value of applied axial force
P_{d0} ..... design value of prestressing force (initial force)
$P_{k,inf}$ ..... lower characteristic value of prestressing force
$P_{k,sup}$ ..... upper characteristic value of prestressing force
P_m ..... mean value of prestressing force
Q ..... variable single action; volume of a transported substance (gas or liquid)
R ..... resistance (strength); bending radius; universal gas constant
R_d ..... design resistance
RH ..... ambient relative humidity

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RH_0 ..... 100% relative humidity
S ..... load effect (M, N, V, T); absorption coefficient
ΔS_{cd} ..... stress range under fatigue loading
$S_{cd,max}$ ..... design value of maximum compressive stress level (fatigue loading)
$S_{cd,min}$ ..... design value of minimum compressive stress level (fatigue loading)
$S_{c,max}$ ..... maximum compressive stress level (fatigue loading)
$S_{c,min}$ ..... minimum compressive stress level (fatigue loading)
S_d ..... design load effect (M, N, V, T)
T ..... temperature, torsional moment
ΔT ..... temperature change
T_{Rd} ..... design value of resistance to torsional moment
T_{Sd} ..... design value of applied torsional moment
$T_{Sd,eff}$ ..... effective design value of applied torsional moment
V ..... shear force; volume of gas or liquid
V_{Rd} ..... design value of resistance to shear force
V_{Sd} ..... design value of applied shear force
V_u ..... ultimate shear force
W_1 ..... section modulus in state I (including the reinforcement)
W_2 ..... section modulus in state II (including the reinforcement)
W_c ..... section modulus of the uncracked concrete cross-section (state I)
$W_{c,cf}$ ..... volume of confined concrete
W_e ..... external work
W_i ..... internal work
$W_{s,trans}$ ..... volume of closed stirrups or cross-ties

<u>others:</u>

ø ..... nominal diameter of steel bar
$ø_n$ ..... equivalent diameter of bundles containing n bars
$ø_p$ ..... diameter of prestressing steel (for bundles equivalent diameter)
$\phi\ (t,t_0)$ ..... creep coefficient
ϕ_0 ..... notional creep coefficient
Θ_{pl} ..... plastic rotation capacity
$\sum U$ ..... total perimeter of rebars

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Statistical symbols

Roman lower case letters:

$f_r(r)$ ..... probability density function (of log-normal distribution)
$f_R(r)$ ..... probability density function of resistance
$f_S(s)$ ..... probability density function of action
$f_x(x)$ ..... probability density function (of normal distribution)
k ..... normalised variable or fractile factor
m_R ..... mean of resistance
m_S ..... mean of action
m_x ..... mean (same meaning as $\bar{x}$)
p_f ..... failure probability
$\breve{x}$ ..... median
$\hat{x}$ ..... modal value
$\bar{x}$ ..... mean (same meaning as m_x)
x_p ..... p-%-fractile

Greek lower case letters:

α ..... sensitivity factor
β ..... reliability index
γ ..... (partial) safety factor
σ_R ..... standard deviation of resistance
σ_S ..... standard deviation of action
σ_x ..... standard deviation
σ_x^2 ..... scattering or variance

Roman capital letters:

$F_r(r)$ ..... probability distribution function (of log-normal distribution)
$F_x(x)$ ..... probability distribution function (of normal distribution)
R ..... resistance
S ..... action
V_x ..... coefficient of variation
Z ..... safety zone (difference of R and S)

others:

$\Phi(k)$ ..... normalised function

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ISSN-1562-3610
ISBN 2-88394-043-6

Structural Concrete

Volume 3

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