Near out-going edges and corners aggressive substances 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 under the prevailing environment, this will lead to early development of damage at the out-going corners and along such edges, i.e. a so-called corner effect develops.

Near in-going edges and corners stress singularities in loaded structural components occur, and the risk of cracking is increased. This may increase the rate at which aggressive substances, including water, locally penetrate into the concrete.

Cyclic wetting and drying effects will strongly accelerate the rate at which dissolved aggressive substances enter the concrete and concentrate 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. Examples are: round columns or columns with rounded side faces in an aggressive environment are more durable than square or rectangular columns; the same goes for rectangular beams.

The exposed edges should be rounded with a radius of curvature of minimum three times the nominal concrete cover, but depending on the bar diameters. A similar curvature should be found in the transverse reinforcement. The corresponding longitudinal corner reinforcement should also be distributed over the rounded edges.

Watertight joints and seals cannot be considered fully watertight throughout the service life of the structure. Consequences of temporary leaks, e.g. at time up to maintenance or replacement, should be foreseen in the secondary protection or drainage of underlying elements.

This may lead to slopes draining the upper surface of supporting beams or columns. Also special water protection or drainage may be applied on these areas. Where de-icing salts are used, e.g. on bridges, parking decks or balconies, this secondary protection is particularly important due to the risk of chloride corrosion following a leak.

Drainage of water over concrete should be limited as much as possible, and drainage over joints and seals should be avoided.

Roofs with large eaves provide valuable protection of the façades against wetting. Bands of balconies and galleries may have a similar effect.

Care should be taken in the detailing of façades of buildings and structures in order to allow easy drainage of water and to facilitate clean-washing.

Retaining walls and bridge piers close to traffic roads may profit from having a larger distance to the road than the required minimum. This will reduce the splash water and fog spray caused by the traffic. This is especially

Surface areas subjected to wetting, splashing or water accumulation should be kept as small as possible.

valuable in the case of frequent use of de-icing salts on the road. The possible increase in cost for the structure may well turn to an economic advantage in the long run.

Alternatively special protection may be applied, see subsection 8.4.6. If the special provision shall protect against corrosion risk due to penetrating chlorides, epoxy coated reinforcement or cathodic protection (sacrificial anode or imposed current systems) are options to be considered.

Most special provisions seem to have a service life of their own which is shorter than the stipulated service life of the structure they are to protect. Consequently such provisions may only be relied upon to ensure the required service life if regular inspection and maintenance is carried out as an integral part of the protection.

Placing and special protection of anchorages for prestressing tendons should be selected considering the local exposure.

Horizontal faces where ponding may take place, and vertical faces on which water may drain are especially risky zones for placing anchorages for prestressing tendons. Even if the anchorage zone is protected by seals or watertight joints etc. it should be foreseen that seals and joints may leak and shed water over the anchorage zone. This should be foreseen in the selection of anchorage protection system.

If possible, internal holes and voids in the structure should be drained and ventilated.

Unintentional voids shall be avoided. This is especially important for post-tensioned concrete structures where lack of grout or insufficient grouting of the ducts may leave ungrouted portions of the tendons.

Water may accumulate in any void present within an exposed structure. Leaks, cracks and condensation may contribute to this accumulation. This water accumulation may cause corrosion on exposed reinforcement. Prestressed reinforcement in grouted ducts especially prone to such unwarned corrosion in the case of unintentionally ungrouted parts of the ducts.

Good and complete grouting of prestressing ducts should be ensured.

Water may also fill up the void or the ungrouted portion of ducts. When later subjected to freezing, severe spalling or bursting of the element may occur.

Structural conditions likely to lead to large local cracks should be avoided. Risks of such cracking should be handled by an appropriate detailing of the reinforcement.

Abrupt deviation of forces in a structure and abrupt changes in section causes stress concentrations likely to cause single large cracks. Concentrated forces due to anchoring of prestressing tendons or due to reactions from supports create large local splitting forces which may cause large cracks. Restraining forces due to differential settlements, shrinkage and temperature effects may also cause large cracks.

8.4.3. Concrete materials, cover thickness and prestressing tendons

Whenever a specially conceived and approved durability assurance scheme is not available, the following rules are deemed to provide a highly impermeable concrete in accordance with subsection 8.1.1.

(a) Concrete composition
The requirements stated in d.6 in Appendix d shall be followed. The exposure classes of subsection 1.5.2 determine the corresponding requirements to be followed.

Appendix d gives the elements among which the design shall choose a combination able to satisfy the specific requirements of the individual structure.

Usually the choice may comprise the following elements

- types and strength class of cement, see d.6.3.3.1
- additions, see d.6.3.3.2 and d.6.5.3
- aggregates, see d.6.3.3.3 and d.6.5.1
- cement contents and water/cement ratio see d.6.3.4, Table d.2 and d.6.5.2.

Concretes with special properties are dealt with in d.6.6.

Special care shall be taken to ensure a high quality impermeable concrete in the outer layer—or 'skin'—of the structure.

Casting and curing conditions have a decisive influence on the permeability of this 'skin'-concrete.

The requirements differ depending on the type of structure, it being

- plain concrete
- reinforced concrete
- prestressed concrete.

The sensitivity to deterioration mechanisms may increase going from plain concrete to prestressed concrete.

Admixtures containing harmful chloride contents should not be applied to reinforced or prestressed concrete structures (see Appendix d).

For indoor structural parts and for structures in prevailing dry environments the relative humidity may be kept permanently at a level where chloride induced corrosion may occur. However, short-term, temporary or local admittance of moisture or water to the concrete may suffice to initiate corrosion and later maintain a corrosion process. Such cases may occur during construction, in kitchen and bathroom environments, at leaks in pipes and after change of use of the structure. In warm dry environments this may occur during occasional rains, due to irrigation, near water outlets, from air-conditioners, and during the fall of dew.

(b) Spacers

Spacer material shall be selected with due regard to the aggressivity of the environment.

In exposure class 3–5 spacer material should preferably have good adhesion to the concrete.

Requirements to concrete quality in the outer layer—or 'skin'—of the structure shall also be satisfied for concrete spacers.

(c) Minimum cover

The minimum distance between any concrete surface and the nearest reinforcement bar, or the nearest prestressing tendon or the sheathing for such tendons, shall be obtained from Table 8.4.1.

The values are absolute minimum values with no downward tolerances allowed and no upward tolerance being specified.

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

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

Tolerance should be taken as 10 mm unless in the individual case it can be demonstrated that a lower value is obtainable. The tolerance should not be less than 5 mm.

Spacers shall be designed according to c_{nom}.

The spacing of spacers shall be adequate to ensure the required cover.

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

Table 8.4.1. Minimum cover, $c_{\min}$

Exposure class	$c_{\min}$ (mm)
1	10
2	25
3, 4	40
5	*

*Depends on the individual type of environment encountered.

The values in Table 8.4.1 refer only to corrosion protection of reinforced and prestressed concrete structures.

Other reasons (see chapter 9) may warrant larger covers such as

- ensuring bond strength
- ensuring fire protection
- use of larger aggregate sizes.

(d) Protection of bonded prestressing tendons

In accordance with subsection 8.1.1 the corrosion protection of prestressing tendons may consist of several successive protection measures such as

- quality and diameter of prestressing steel
- grouting of ducts or sheathing
- type of sheathing

- quality of concrete cover
- thickness of concrete cover
- special surface protection
- sealing of anchorages.

Each step may be individually chosen and designed to obtain optimal combined protection for a given environment, in order to obtain the required service life with an acceptable level of reliability.

Prestressing steel and sheathings shall be adequately protected against environmental influences and mechanical damage during transport and storage and during the period between assembly and grouting, according to subsections 11.5.1–11.5.5.

A distinction should be made between internal and external unbonded tendons.

For internal tendons the properly specified and executed concrete cover provides the primary corrosion protection. Supplementary corrosion protection barriers are provided by the sheathing and the long-term cement grout, grease, wax or other suitable agent used as sheath filling material.

For external tendons the sheathing provides the primary corrosion protection. A supplementary corrosion protection barrier is provided by the long-term cement grout, grease, wax or other suitable agent used as sheath filling material.

The principles for selecting protective measures are similar for monostrands and for multiple tendons.

(*e*) *Protection of unbonded tendons*

In accordance with subsection 8.1.1 the corrosion protection of unbonded tendons may consist of several successive protection measures such as

- quality and dimension of prestressing steel
- type of sheath filling material such as grease, wax or other suitable agent
- type of sheathing, depending on whether tendons are internal or external
- for internal tendons: quality and thickness of concrete cover
- special surface protection
- sealing of anchorages.

Each step may be individually chosen and designed to obtain optimal combined protection in a given environment in order to obtain the required service life with an acceptable level of reliability.

The prestressing steel shall be effectively protected over its entire length including the anchorages and any coupler present considering the chosen exposure class and the intended service life of the tendons.

When various types of materials are combined it shall be verified that they do not contain any substance harmful to the prestressing steel and that materials which come into contact with each other are compatible.

Special attention is to be paid to the corrosion protection of the anchorage. The continuous protection of the prestressing steel shall be ensured under due consideration of the relative movement between prestressing steel and anchor head during the stressing operations.

The sheathing shall be completely waterproof and continuous for the full length of the tendon, and shall prevent the intrusion of cement paste during concreting.

The sheathing material should be of a type which will not react with cement, grease or steel. It should be durable and resistant to damage and abrasion and it should remain stable and flexible during handling, storing on site and in service for the range of temperatures likely to be experienced.

The sheathing should maintain sufficient strength and flexibility to bridge any fine cracks which may appear in the concrete.

The sheathing material shall be resistant to ageing by exposure to UV-light.

The shape of prestressing strands should not be reproduced on the outside of the sheath.

Sheathing may not contain PVC.

PVC can produce chlorides and may increase the corrosion risks.

The anti-corrosion grease, wax or similar void filling agent shall be impervious to moisture, non-absorbing and not capable under static conditions of forming an emulsion with water.

The force required to move the prestressing steel in the finished protected sheathing in its final location in the structure shall be limited to allow subsequent stressing to take place.

The force required to move the prestressing steel in a 1 m long sample at room temperature should not exceed 75 N.

Corrosion protection filler materials injected into the sheathings or pipes (multiple tendons) shall satisfy the following requirements

- easy to inject into the pipes in more or less fluid form
- able to restabilize in a way that no leakages occur under the prevailing conditions
- have a known thermal expansion coefficient
- be of good tightness and provided with vents in sufficient number and at locations to prevent air pockets and voids.

A higher temperature of the filler should neither adversely affect the duct nor the prestressing steel.

It shall be decided whether expansion chambers are needed to cope with higher ambient temperatures.

The tightness of the tendon envelope, especially at the anchorages, and the ambient temperature shall be taken into account.

Unbonded tendons and sheathings shall be adequately protected against environmental and mechanical damage during transport and storage, according to subsections 11.5.6 and 11.5.7.

8.4.4. Detailing

Structures should be designed taking into account the combined effects of the relevant exposure class of subsection 1.5.2, the detailing of the structural form including joints, connections and supports as stated in subsection 8.4.2 and the detailing of the reinforcement layout.

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.

Bundled reinforcement should comply with the requirements for cover, spacing and bundling as stated in subsection 9.1.5.

Where possible the reinforcement layout should constitute a closed three-dimensional reinforcement cage. When using links or stirrups, these should have a geometric form which complies with the three-dimensional cage action.

Congestion of reinforcement may lead to difficult casting conditions where the concrete is segregated by being sieved through the reinforcement, thus causing bad compaction and honeycombing.

The three-dimensional closed reinforcement cage action provides a confinement of the concrete, and can increase the structural reliability in the case of any deleterious expansive reactions within the concrete causing delamination and splitting of the concrete. This increases the overall robustness of the structure.

In the case of slabs which may need no shear reinforcement a minimum number of uniformly distributed links may be provided to ensure a sufficient three-dimensional confinement of the concrete in case of any splitting or delamination of the bulk concrete.

Structural components exposed permanently or temporarily to moist environments, such as steel inserts, temporary hooks, fasteners etc. cast into the concrete, shall not be in electric contact with any cast-in reinforcements.

In moist environments the cast-in steel area determines the rate of corrosion of the exposed part of the steel.

The corrosion rate of the exposed part of the steel may be further reduced by coating the steel item, either the cast-in part only or, if possible, the total steel item.

These requirements are also valid for joints and bearings.

Alternatively stainless steel may be applied, in which case electric contact to the reinforcement is admitted.

In exposed structural parts construction joints should be selected with due regard to the

— type of exposure and aggressiveness of the environment at the joint

— the stress level and stress variations foreseen at the location of the joint

— the impact of the joint on the visual appearance of the structure.

Construction joints should be located in parts of the structure where the exposure to water and dissolved aggressive substances is minimal. Any water expected to run on the surface should preferably run parallel to the joint.

If possible, construction joints should be located in zones with no or minimal tensile stresses in the concrete and with minimal stress variations. Stress reversals in joints should be avoided, if possible.

In accordance with the basic requirements of subsection 1.5.1, and as stated in section 8.1, acceptable appearance is one of the design objectives. Construction joints are clearly visible and, where relevant, care should be taken in selecting their location and in selecting the formwork determining the visual transition in the surface texture.

Due to the different sensitivity of prestressing steel and ordinary reinforcing steel to corrosion induced failures, different corrosion protection needs to be applied.

This has been shown by laboratory tests and practical experience.

Prestressing steel may fail due to stress corrosion cracking (SCC) or hydrogen embrittlement (HE) in a very brittle manner and without warning even at rather little degrees of corrosion. The design strategy therefore, shall be to avoid any depassivation of the prestressing steel surface if the environmental conditions may lead to corrosion.

Zinc coated or galvanized sheathings may not be used for grouted ducts as hydrogen develops and greatly increases the risk of HE.

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.

If the prestressing steel is located in deeper sections cracking of the outer concrete layer is acceptable if the crack depth is limited to an extent that ensures an uncracked cover over the prestressing steel according to the defined nominal values given in Table 8.4.1.

Test results and practical experience show that crack width does influence the time to depassivation, however a limitation of crack width to avoid depassivation during the whole service life is impossible for ordinary reinforced elements. After depassivation crack width practically does not influence the corrosion rate. Therefore, a differentiation of permissible crack widths depending on environmental classes is not considered necessary for ordinary reinforced concrete elements.

Very severe chloride attack occurs in, for example, unprotected park decks with restraint induced cracking due to shrinkage and/or temperature. Chloride containing water penetrates through those cracks even at very low crack widths. Temperature changes may avoid healing of even small cracks.

8.4.5. Nominal crack width limitations

Basically different strategies need to be applied for prestressed and ordinary reinforced concrete elements.

Corrosion risk and corrosion rate in the region of cracks depend predominantly on the impermeability and the thickness of concrete cover. Therefore, the requirements of subsection 8.4.3 are of special importance to ensure sufficient corrosion protection of the reinforcement in cracked regions.

For structural elements exposed to environments according to exposure class 2 to 5 (see Table 1.5.1 in section 1.5) crack width limitation for prestressed elements shall ensure that the prestressing steel will not be depassivated during the anticipated service life. The limitations given in Table 7.4.1 (section 7.4) normally are satisfactory.

For structural elements or sections being affected by imposed deformations, e.g. temperature gradients, not covered by the frequent load combination and leading to cracking across the prestressing steel, additional measures compared to Table 7.4.1 are necessary. Additional measures could be

- impermeable ducts or coating
- compression within the whole section.

Depassivation, primarily due to carbonation, cannot be totally avoided in the region of cracks crossing ordinary reinforcement. Crack width is of minor importance with respect to corrosion rate in the range to be expected, if the design is in accordance with the Model Code.

In the case of very severe chloride and unfavourable structural conditions (e.g. cracks due to tension in horizontally oriented elements with chloride attack on the top side) special protective measures like coating of concrete or coating of reinforcement should be taken. Limitation of crack widths to lower values than $w = 0.3$ mm cannot prevent corrosion damage under those unfavourable conditions.

8.4.6. Special protective measures

The required service life of concrete structures should primarily be obtained (see subsection 8.1.1)

- through a robust structural design taking the specific environmental aggressivity into account when selecting the structural form
- by selecting an appropriate concrete mix
- by ensuring an adequate compaction and curing of the concrete
- by selecting an inspection and maintenance strategy which will reveal the condition of the structure in use in due time for preventive maintenance or minor repairs to be performed before costly repairs are needed.

In case of especially aggressive environments where the normal provisions to ensure the required service life cannot suffice, and in cases where insufficient durability has resulted in damage to an existing structure, special protective measures may be applied to obtain the required service life.

The special protective measures are of the following types.

- Provide smooth surfaces and minimize the area exposed to the environmental aggressivity.
- Provide structural protection such as
  — roof, eaves or similar to protect concrete surfaces against rain
  — surface protection in the form of a water repellant impregnation, a thin or thick film coating, tanking (of e.g. foundations), membrane and lining

Surface coatings may be selected barriers against penetrating substances such as

- water (H_2O)
- water vapour
- carbon dioxide (CO_2)
- chloride ions (Cl^-)
- oxygen (O_2).

The consequences of selecting such barriers should be carefully evaluated, especially the risks of possible accumulation of moisture in concrete with a water impermeable membrane. As an example alkali-aggregate reactions may develop even in old structures following surface treatment, if the concrete contains alkali-reactive aggregates.

Chlorides enter into the concrete dissolved in water; this is why water repellant or water impermeable membranes should be used to prevent the ingress of Cl^-.

Theoretically corrosion can be stopped by preventing access of O_2 to the reinforcement. However, in general this is an unreliable procedure. In practice corrosion inhibition can be obtained only in fully submerged structures. The ageing of surface coatings and their ability to be re-done at regular intervals should be carefully evaluated prior to selecting products.

The colour stability of coatings may vary, and should be considered prior to selecting products.

A moisture membrane may be protected by a separate overlay, e.g. as on traffic loaded elements subjected to de-icing salts.

— increased concrete cover; provide special skin reinforcement if $c_{nom} \geqslant 70\,mm$

Two different strategies may be followed when selecting increased concrete cover with a special skin reinforcement.

(*a*) The cover on the skin reinforcement is considered a sacrificial cover which may spall, if this is acceptable, some time in the future if the skin reinforcement corrodes. In the latter case the skin reinforcement acts as a sacrificial anode protecting the main reinforcement provided they are connected electrically. The outer part of the cover is not taken into account in the load carrying capacity. Stiffness and restraining forces are calculated with and without this extra cover.

(*b*) The skin reinforcement is specially protected, e.g.

(i) *By polymeric coating*. It should be ensured, that there is no electric contact between the coated skin reinforcement and the uncoated main reinforcement.
By maintaining uncoated main reinforcement (coating of single bars) a future installation of cathodic protection is a valuable option, should this prove necessary some time in the future (Multi-Stage Protection Strategy, see subsection 8.1.1).

(ii) *By selecting specific stainless steel.* There is no restriction in electric connections to the main reinforcement.

— reduce environmental aggressivity by, for example, surface insulation, thus controlling heat and moisture conditions in the concrete (housing).

- Provide special protection of the reinforcement, such as

— placing prestressed reinforcement in sheathings (metallic or plastic) with special corrosion protective grout or void filler

Polymeric coating of reinforcement may provide a long-term reliable barrier against corrosion of reinforcement due to either carbonation or chloride ingress, provided the coating

- has sufficient thickness
- provides a total coverage, allowing only minimum amounts of pinholes and holidays (see special standards)
- has a high and lasting bond to the reinforcement
- is undamaged in bent bars
- is patch repaired sufficiently at cut ends.

and provided coated bars are not mixed with uncoated bars in the same structural component where there is a risk of electric contact between the two types of bars.

Coated bars have different bond and anchorage characteristics which should be taken into account in the design and detailing of the reinforcement.

Galvanizing may prolong the service life of cast-in reinforcement if the concrete carbonates, but galvanizing is not considered to provide a long-term reliable corrosion protection of reinforcement in chloride contaminated concrete.

— coating of reinforcement

Cathodic protection of reinforcement may provide a reliable protection against corrosion even in cases where very high chloride concentrations may occur. Such protection may also be achieved for reinforcement where corrosion has started.

Cathodic protection of immersed or buried parts of a structure can be achieved by traditional local anodes placed in water or in moist soil and the protection based on either sacrificial anodes or on an impressed current system.

For structural parts in the air, anodes should be placed on the concrete surface either distributed or localized, and the protection based on an impressed current system. Several surface mounted anodes need a conductive overlay to ensure the current distribution to the reinforcement. The dead load of such overlays should be considered in the design.

Cathodic protection of prestressed structures is considered possible, but due to the increased risk of hydrogen embrittlement of the prestressing steel, care should be taken in the design and especially in the monitoring of the system to avoid overprotection and hydrogen development at the reinforcement.

— cathodic protection.

Cathodic protection tends to increase the concentration of alkalis near the reinforcement. This may increase the risk of alkali-aggregate reactions if the concrete contains alkali-reactive aggregates.

For new structures anodes may be placed in the formwork and cast integral with the structure. Any short circuits with the reinforcement will be detrimental to the system.

Initial electric continuity of the reinforcement will greatly facilitate the installation of cathodic protection some time in the future, should this prove necessary.

The probability of achieving robustness and good quality concrete in the most exposed parts of a structure, such as in the outer concrete skin, is increased considerably by selecting structural form and detailing which facilitates the execution and curing process. This includes

- adequate dimensions to ensure easy casting and compaction of concrete
- formwork which, where relevant, provides smooth and pleasant surface texture
- detailing of reinforcement to ensure high quality, well compacted concrete in the concrete cover, free of any honeycombing, see sub-section 8.4.4
- predetermined positioning of construction joints to match the exposure class and the foreseen structural performance.

In general the adequacy and workability of the concrete mix for a specific job should be verified by performing trial castings under conditions which simulate actual conditions during construction. Full verification testing should be made on test specimens from trial castings in accordance with the concrete specifications for the work.

For structures in chloride containing environments (exposure class 3 and 4) all reinforcement should be electrically continuous for the whole structure or individually for selected structural components

— select non-corroding reinforcement (specific stainless steel).

- Provide special monitoring systems (e.g. a warning system) to follow the condition of the structure.
- Provide intensified inspection and maintenance routines to cope with early deterioration.

8.4.7. Prerequisites related to execution and maintenance

Structures should be designed and detailed, with due regard to the relevant exposure classes, and with due account of the execution procedures fore-seen, such as to

- facilitate execution
- be adequately inspectable
- be maintainable.

The design should be based upon an explicitly stated maintenance strategy.

A quality assurance level for execution and maintenance should be determined prior to finalizing a design.

The execution process should ensure a concrete mix which limits bleeding and should ensure adequate moisture and temperature curing of the concrete to avoid plastic shrinkage, cracking and thermal cracking (see chapter 2 and Appendix d).

Such vulnerable parts may typically be associated with

- joints
- bearings
- drainage systems
- miscellaneous installations
- areas where water and dirt accumulate.

Visual inspections can only reveal ongoing active deterioration such as

- abrasion and erosion
- cracking and spalling
- disintegration of concrete surfaces due to freeze-thaw attack, sulphate attack, alkali aggregate attack, acid attack etc.
- rust staining and other miscolouring.

Inspection should include selective sampling of information regarding, where relevant

- carbonation depth relative to prevailing concrete covers
- chloride profiles relative to prevailing concrete covers
- potential mapping with excavations ('windows') to calibrate measurements
- coring including thin section microscopy and petrographic analysis to determine concrete quality such as compaction, curing, microcracking, water–cement ratio, air content, reactivity of aggregates
- surface tapping to reveal surface delaminations.

Such testing selected from the above list and performed prior to any deterioration visible with the naked eye may reveal oncoming deterioration in due time for preventive maintenance to be performed.

It is current experience that in the majority of cases the costs for preventive maintenance may be substantially lower than the corresponding repair once visible damage is developing.

Inspections and maintenance procedures should take due account of the relevant deterioration mechanisms as revealed by the testing.

During design and execution the more vulnerable parts of the structure which may be expected to need intensified inspection and maintenance should be identified.

Regular and systematic inspection and maintenance routines are integrated parts of ensuring the required service life of structures. Inspectors should determine the current condition of the structure and its components, and can in general not rely solely on visual inspections.

In order to minimize future maintenance and repair costs the inspection routines should, to the extent possible, reveal approaching deterioriation or lack of adequate performance in due time for preventive maintenance to be applied, i.e. reveal the transition point between initiation and propagation as presented in subsection 8.1.1, Fig. 8.1.1.

9. DETAILING

Whereas the amount of steel reinforcement and prestressed tendons is determined by applying the models included in the previous chapters, this chapter contains information related to limit values, location and arrangement of steel elements, dictated by considerations other than those accounted for by means of calculation.

Such other considerations may be

- conditions of validity of simplified calculation procedures
- minimum ductility conditions
- qualitative reliability requirements
- functional requirements
- practical durability measures
- friendliness of execution, etc.

A complete design should always account for these considerations, be it by means of sound empirical rules, whenever respective more specific models are not used.

9.1. ANCHORAGES, SPLICES, ARRANGEMENT

The rules in subsections 9.1.1 to 9.1.5 apply to reinforcing steel. Subsections 9.1.6 and 9.1.7 apply to prestressing steel. To ensure that bond forces are safely transmitted and to prevent spalling of the concrete, the minimum cover of any bar, stirrup, tendon or sheathing of diameter ϕ should be at least equal to ϕ.

Detailed guidance is given in the CEB-Application Manual on 'Concrete Reinforcement Technology' (Bulletin 140) and CEB Bulletin 164 'Industrialization of Reinforcement'.

Subsections 9.1.1 to 9.1.3 are applicable in particular to bars with diameter $\phi \leqslant 32$ mm.

Supplementary rules are given

- for high-bond bars of $\phi > 32$ mm, in subsection 9.1.4
- for bundled bars in subsection 9.1.5.

9.1.1. Anchorages

9.1.1.1. General

The normal methods of anchorage are

- straight anchorages (Fig. 9.1.1(a))
- curved anchorages:
  - — hooks (at 150° to 180°) (Fig. 9.1.1(b))
  - — bends (at 90° to 150°) (Fig. 9.1.1(c))
  - — loops (Fig. 9.1.1(d))
- anchorages with at least one welded transverse bar within the design anchorage length (Fig. 9.1.1(e))
- anchorage by mechanical devices.

Straight anchorages or anchorages with bends are not allowed for plain bars in tension.

Local transverse reinforcement should be provided if bends or hooks are used for compression reinforcement (see clause 9.1.2.1).

For the shear strength of welds, see clause 2.2.5.1b.

For welded meshes made of plain or indented wires, see subsections 6.9.8 and 6.9.9.

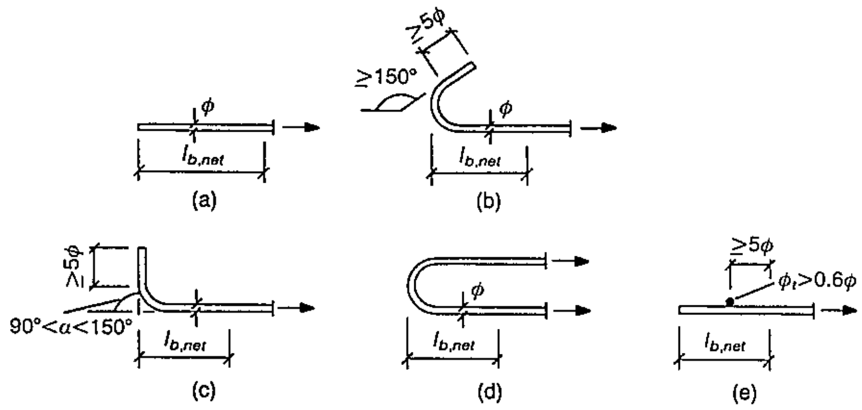


Fig. 9.1.1. Recommended detailing of anchorages

The purpose of transverse reinforcement is to avoid

- longitudinal cracking under the effect of transverse tensile stresses originating in the anchorage zones
- bursting of the concrete due to pressure exerted at the end section of a compression bar (see clause 9.1.2.1).

Normally the minimum shear reinforcement confining the anchored bars (see clause 9.2.2.2) or the minimum transverse reinforcement in slabs (see clause 9.2.1.1), in walls (see clauses 9.2.4.2 and 9.2.4.3) and in columns (see clause 9.2.3.2) fulfils the requirement for minimum transverse reinforcement.

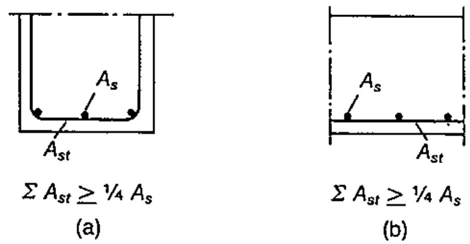
For bars in compression, the transverse reinforcement should surround the bars, being concentrated at the end of the anchorage and extend beyond it to a distance of at least 4ϕ.

Special care is necessary where the bars end near a concrete surface (see Fig. 9.1.3) or are provided with bends or hooks if they are subjected to high compression forces. A minimum cover of 2ϕ is normally required.

Transverse reinforcement should be provided

- for anchorages in tension, if there is no compression transverse to the plane of splitting (e.g. due to a support reaction)
- for anchorages in compression, in all cases.

The minimum area of transverse reinforcement (one leg) is 25% of the area of one anchored bar (see Fig. 9.1.2).



ΣA_{st} is the area of the transverse reinforcement along the anchorage length

Fig. 9.1.2. Minimum transverse reinforcement: (a) beams; (b) slabs

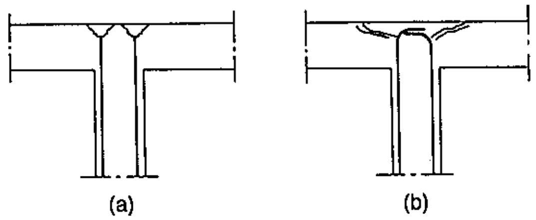


Fig. 9.1.3. Possible damage caused by compression bars ending near concrete surface

9.1.1.2. Permissible radii of bends

Bars

The diameter of the mandrel used should satisfy the following conditions.

In general for ambient temperature, the minimum mandrel diameter should not be less than the diameter required to satisfy the bend–rebend test for the reinforcement.

(*a*) The steel bar shall not fail or crack during its bending around the mandrel.

Thus, the minimum mandrel diameter depends on the plastic uniform elongation of the steel under flexural conditions, taking into account rate effects and temperature effects.

Besides, the presence and form of the ribs may also influence the minimum diameter of the mandrel. Because of the high sensitivity of the bars with regard to the parameters mentioned hereover, the relevant indications of the approval documents will apply.

(*b*) Crushing or splitting of concrete, due to the pressure occurring inside the bend, should be avoided under the ULS conditions.

In the absence of transverse reinforcement, the following simplified model may be used, where the local compression capacity (section 3.3) is reduced by 30% to account for concrete skin deficiencies

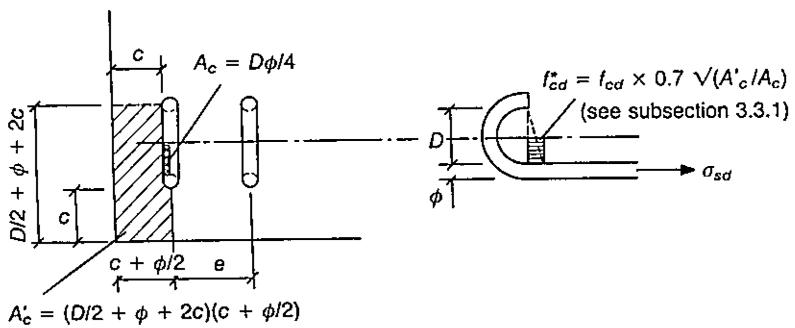


Fig. 9.1.4. Simplified model

The relevant minimum mandrel diameter depends on concrete tensile and steel strengths, on the thickness of concrete cover (or the distance of consecutive bars) perpendicular to the plane of curvature, as well as on the anchorage scheme (hairpin, hook, stirrup angle). Besides, the design value of the steel stress σ_{sd} to be anchored by means of the bent end of the bar has to be taken into account. Last but not least, transverse reinforcement reduces the allowable mandrel diameter (increasing the local compressive strength of concrete).

$$\tfrac{1}{2}D\phi f_{cd}^{*} = (\pi\phi^2/4)\sigma_{sd}$$

$$D/\phi \approx \delta/\sqrt{(1 + 2c/\phi)}(\sigma_{sd}/f_{cd})$$

where $\delta \approx 1.6$. This value (reflecting the f_{cc}/f_{ct} ratio, see subsection 3.3.1, left hand side) should be taken higher for higher concrete strengths.

In the expression above, c = min (cover, e/2), and f_{cd} takes one of the two values defined in clause 6.2.2.2 (generally f_{cd1}).

Similar approximate expressions for other anchorage schemes may be found.

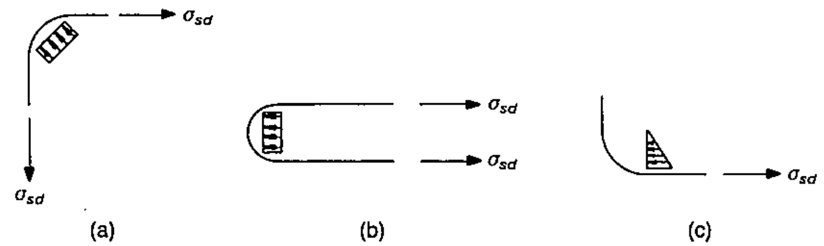


Fig. 9.1.5. δ-values for various anchorage schemes: (a) $\delta \approx 1.6$; (b) $\delta \approx 1.8$; (c) $\delta \approx 2.6$

Vertical forces and moments needed for equilibrium are not represented on the sketches.

Because of the large number of parameters influencing the diameter of the bend, which may lead to concrete crushing, conservative values based on long experience are suggested by National Codes, accounting for specific conditions.

Indicatively, the following minimum values of D/ϕ may be used for 90° bends if 60% of the yield stength of a deformed bar is anchored by the bend. In a conservative way the beneficial role of high strength concrete was not considered in the following table.

Table 9.1.1. Values of D/ϕ for 90°-bends anchoring 60% of the yield strength of a deformed bar—S400; concrete C20

c/ϕ	D/ϕ
$c/\phi > 7$	10
$3 < c/\phi < 7$	15
$c/\phi = 1$	25

c = min (cover thickness, half the free distance between consecutive bars)

If $f_{yk} \neq 400$ MPa the above values of D/ϕ should be multiplied by $f_{yk}/400$.

However, smaller D/ϕ-values may be used if transverse reinforcement is present.

The minimum diameters given in Table 9.1.1 can be used for welded reinforcement bend after welding, provided the distance between the point of welding and the beginning of the curvature is not less than 4ϕ. This distance may be reduced or the welding point may fall in the curved part of the reinforcement if, for structures subjected to predominantly static loading, the mandrel diameter is at least 20ϕ.

For the transmission of the concentrated anchorage force to the concrete, see sections 3.3 and 3.5.

Welded mesh

The diameter of mandrels of welded mesh bends should respect the two requirements mentioned above, taking into account the additional force transfer capacity due to welded transverse bars.

These rules do not cover cases under predominant dynamic loading.

9.1.1.3. Mechanical anchorage devices

The effectiveness of mechanical anchorages should be demonstrated by suitable tests. The slip, recorded during the test, between the bar and the concrete at the loaded end, shall not exceed

- 0.1 mm under 70% of the ultimate force
- 0.5 mm under 95% of the ultimate force.

The design value of the anchorage resistance should not be larger than

- 50% of the ultimate force of the anchorage, when fatigue loads are negligible
- 70% of the experimental fatigue strength for 10^7 load cycles.

For permissible curvatures, see clause 9.1.1.2.

9.1.1.4. Anchorages of stirrups and shear assemblies

The anchorage of stirrups, shear assemblies and confining reinforcement is normally obtained by means of hooks, bends, or welded transverse bars.

The type of anchorage used should not induce splitting or spalling of the concrete cover.

Anchorages by hooks (150° to 180°) are required for plain bars.

For arrangement of shear reinforcement, see clause 9.2.2.2.

Anchorages by bends (90° to 150°) are only allowed for high-bond bars. The anchorage is considered to be fully efficient

- if the curved section of the bar is extended by a straight length of at least
  - 5ϕ or 50 mm following an arc of 150°, or more (Fig. 9.1.6(a))
  - 10ϕ or 70 mm following an arc of 90° (Fig. 9.1.6(b))
- or if on the length of the anchorage there are
  - either two welded transverse bars of diameter not less than 0.7 times the diameter of the stirrup (Fig. 9.1.6(c))
  - or a single welded transverse bar, of diameter not less than 1.4 times the diameter of the stirrup (Fig. 9.1.6(d)).

Shear resistance of welds shall be equal to at least $0.3 f_{sy} A_s$ of the anchored bar, when tested naked and $0.5 f_{sy} A_s$ of the biggest bar, when tested in concrete.

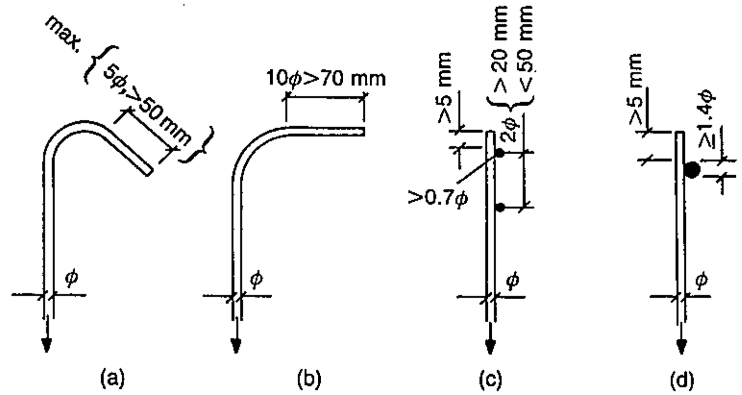


Fig. 9.1.6. Anchorages of links, stirrups and shear assemblies

Confinement should be used if bends or hooks are used for compression reinforcement (see clause 9.1.2.2.3b and subsection 6.9.5).

9.1.2. Splices

9.1.2.1. General requirements for bars

Forces may be transmitted from one bar to another

Splicing of all reinforcing bars of an element at a single cross-section should preferably be avoided (see, however, clause 9.1.2.2.2).

The splices in a section should, if possible, be arranged symmetrically and parallel to the outer face of the element.

- by lapping the bars, with or without hooks, bends or loops
- by welding
- by mechanical devices assuring load transfer in tension–compression or in compression only.

9.1.2.2. Splices by overlapping

9.1.2.2.1. General requirements

Lapped splices should preferably not be placed in zones where the reinforcement is utilized to its design strength.

Force transfer between spliced bars should be safely secured. To this end

- If the transverse distance between two spliced bars does not exceed 4ϕ, the anchorage length should be taken from eq. (6.9–7) (see Fig. 9.1.7(b)).
- If the transverse distance between spliced bars is larger than 4ϕ, the following measures should be taken.
  - — The anchorage length should be increased by a distance equal to the spacing of the bars.
  - — The transverse reinforcement needed should be calculated by a strut and tie model according to Fig. 9.1.7(a); however, this reinforcement should not be lower than foreseen in clause 9.1.2.2.3.

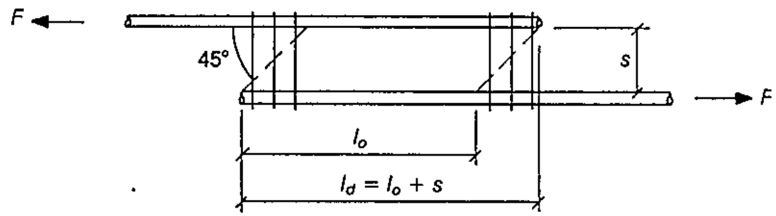


Fig. 9.1.7(a). Splice when $s > 4\phi$

- In principle, lapped splices should be staggered; exceptions are allowed under the conditions of clause 9.1.2.2.2.

  If two adjacent laps are arranged in different sections, the distance separating the ends of bars of adjacent laps should not be smaller than (see Fig. 9.1.7(b))
  - — in the transverse direction 2ϕ and not less than 20 mm (clear distance)
  - — in the longitudinal direction $0.3 l_o$ (l_o from eq. (6.9–7)).

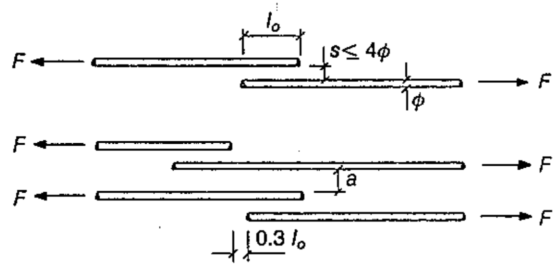


Fig. 9.1.7(b). Staggering and spacing of splices: $s \leqslant 4\phi$; $a \geqslant 2\phi$; $a \geqslant 20\,mm$

- Transverse tensile stresses of concrete should be safely taken by transverse reinforcement when needed (see clauses 9.1.2.2.2 and 9.1.2.2.3).

9.1.2.2.2. Permissible percentage of spliced reinforcement

(*a*) When the provisions of clause 9.1.2.2.1 are respected, the rules of Table 9.1.2 apply for bars in tension.

When transverse reinforcement is provided in excess of the basic requirements (clause 9.1.2.2.3), higher percentages of lapped bars may be used provided that appropriate justification is given.

For secondary reinforcement see clause 9.1.2.3.1b.

Table 9.1.2. Permissible percentage of lapped bars in tension in one section, if transverse reinforcement is not provided

Type of bar	Monotonic load effects	Repeated load effects
High-bond bars		
If only in one layer	100%	100%
If in several layers	50%	50%
Plain bars		
ϕ < 16 mm	50%	25%
$\phi \geqslant$ 16 mm	25%	25%

Secondary ('distribution') reinforcement may be lapped in one section.

(*b*) The percentage of lapped bars in compression in any section may be 100% of the total reinforcement.

9.1.2.2.3. Transverse reinforcement in the splicing region

Transverse reinforcement is required to resist transversal forces developed by inclined concrete struts assuring the transmission of longitudinal force from one bar to the other.

(a) Transverse reinforcement for bars in tension

If the diameter ϕ of the lapped bars is smaller than 16 mm or if the percentage of lapped bars in one section is equal to or less than 25%, then the minimum transverse reinforcement provided for other reasons (e.g. shear reinforcement, distribution bars) is considered to be sufficient.

If $\phi \geqslant 16$ mm or the percentage of lapped bars in one section is more than 25%, then

(i) the transverse reinforcement should have a total area (sum of all transverse bars perpendicular to the layer of the spliced reinforcement) of not less than the area of the spliced bar (Fig. 9.1.8)

(ii) if $a \leqslant 10\phi$ (Fig. 9.1.8), transverse bars should be formed by stirrups (a being the transverse distance between consecutive lap splices, see Fig. 9.1.7b).

The amount of the transverse reinforcement can be reduced in the case of transverse compression.

The confinement needed can also be obtained by sufficient concrete mass or by stirrups not necessarily in contact with the lapped bars section.

The transverse reinforcement should be placed along the two outer sections of the splice ($l_o/3$) (Fig. 9.1.8(a)), between the longitudinal reinforcement and the concrete surface.

(b) Transverse reinforcement for bars permanently in compression

One bar of the transverse reinforcement should be placed outside of each end of the lap length and within 4ϕ of the ends of the lap length (Fig. 9.1.8(b)).

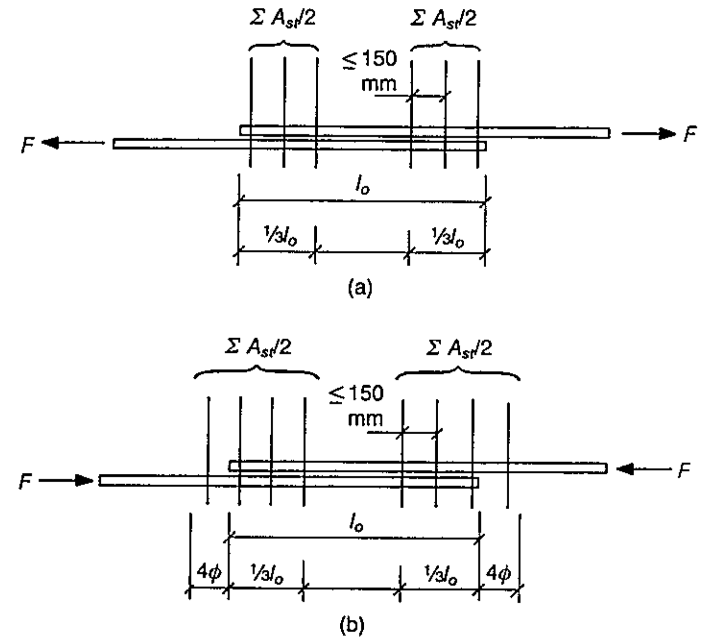


Fig. 9.1.8. Detailing rules and notation for lapped splices (plan view): (a) in tension; (b) in compression

9.1.2.3. Overlapping of welded fabric

9.1.2.3.1. General requirements

Welded fabric can only be spliced by lapping.

(a) Main reinforcement

The splices can be made either by intermeshing or by layering of the fabric (Fig. 9.1.9).

The term 'main reinforcement' is used for reinforcement, irrespective of its direction, when it is provided by calculation to resist action effects (e.g. in two-way spanning slabs).

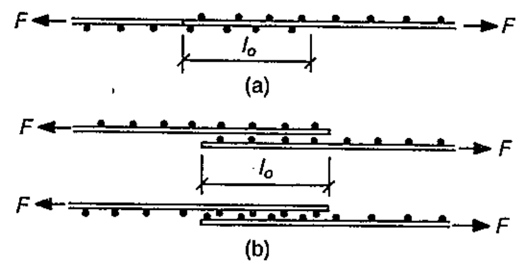


Fig. 9.1.9. Lapping of welded fabric: (a) intermeshed fabric (longitudinal section); (b) layered fabric (longitudinal section)

Where fatigue loads occur, intermeshing should be adopted (clause 1.6.4.2a).

For intermeshed fabric, the lapping arrangements for the main longitudinal bars should conform with clause 9.1.2.2 and subsection 6.9.5 (neglecting the favourable effect of the transverse bars: thus taking $\alpha_2 = 1.0$).

For layered fabric, the splices of the main reinforcement should generally be situated in zones where the design stress of the reinforcement for the ultimate limit state is not more than 80% of the design strength.

Where this condition is not fulfilled, the effective depth of the steel taken into account in the calculations, in accordance with section 6.3, should apply to the layer furthest from the tension face; in addition, due to the discontinuity at the end of splices, when carrying out a crack-verification (Table 7.4.3), next to the joint, the steel stress used in Table 7.4.3 should be increased by 25%.

A reinforcement is called 'secondary' when it is provided transverse to the main steel in order to undertake specific (not calculated) action effects, such as local transverse moments, temperature or shrinkage effects etc. For this reinforcement, the term 'distribution reinforcement' is sometimes used.

(b) Secondary reinforcement

Where the reinforcement acts as 'secondary reinforcement' both arrangements (i.e. wires in the same plane or in different planes) are allowed irrespective of the type of loads (fatigue loads or not).

9.1.2.3.2. Permissible percentages of lapped welded fabric reinforcement

(a) Main reinforcement

For intermeshed fabric, the values given in Table 9.1.2 are applicable.

For layered fabric the permissible percentage of the main reinforcement that may be spliced by lapping in any section depends on the specific cross-sectional area A_s/s of the welded fabric

(i) if $A_s/s \leqslant 1200\,mm^2/m$, then 100%
(ii) if $A_s/s > 1200\,mm^2/m$, then the welded fabric may be spliced only if it is in an interior mesh of multiple layers. The permissible percentage of bars to be spliced by lapping in one section is 60% of the total reinforcement area.

The joints in multiple layers should be staggered by $1.3l_o$ (l_o from eq. (6.9–8)).

(b) Secondary reinforcement
When the reinforcement acts as 'secondary reinforcement', splices in the transverse reinforcement can be placed in the same cross-section and may extend to the whole (100%) of the total area of the reinforcement in that section.

(c) Laps of the secondary reinforcement
For intermeshed fabric, clause 9.1.2.2.3 applies.
For layered fabric, the length of the lap is chosen from Table 9.1.3.

Table 9.1.3. Required lap lengths for splices of the secondary reinforcement (layered fabric)

Diameter of wires (mm)	Lap lengths
$\phi \leqslant 6$	$\geqslant 150$ mm; at least 1 wire pitch within the lap length
$6 < \phi \leqslant 8.5$	$\geqslant 250$ mm; at least 2 wire pitches
$8.5 < \phi \leqslant 12$	$\geqslant 350$ mm; at least 2 wire pitches

9.1.2.4. Splices by mechanical devices

Mechanical connections of bars are based on couplers.

A coupler is a prismatic or cylindrical steel device which allows the joining of two bars end to end. The force in a bar is transferred to the other bar through the coupler.

9.1.2.4.1. Classification of mechanical connections

Mechanical connections are characterized according to their properties with regard to load transfer: compression-only connections and tension-compression connections.

Connections may need additional materials (e.g. filling materials) or necessitate end preparation of the bars (e.g. threading).

In this category the following types are available

- threaded sleeve
- wedge-locking sleeve
- bolted steel sleeve (solid type or strap type)
- metal-filled sleeve.

The tension-compression connections may be based

- on threading, like
  - — steel sleeve threaded for special bars
  - — threaded sleeve, rolled threads on bars
  - — taper-threaded steel coupler
- on material filled, like
  - — metal filled sleeve
  - — mortar filled sleeve
- or on sleeve deformation, like
  - — forged steel sleeve
  - — cold swaged steel sleeve
  - — cold swaged steel sleeve with threaded ends.

In accordance with these data, appropriate consequences in design of reinforced concrete should be considered; such as design strength values, allowable redistribution and type of analysis to be used.

(*a*) *Compression-only connections* are based on compressive stress transfer by coaxial contact from one bar to the other. The ends of the bars shall be square cut and in concentric contact. Alternatively, a compression transfer based on a metal-filled sleeve or a threaded sleeve may be applied.

(*b*) *Tension-compression connections* shall be designed for full tension and compression capability and the connection has to meet certain requirements regarding its elongation and ductility properties.

In all cases, the recommendations of appropriate technical approvals should be used.

9.1.2.4.2. Requirements

Technical approvals should specify the following characteristics of connections

- characteristic values of yield and rupture strengths
- deformation properties of the connections
- fatigue characteristics.

9.1.3. Arrangement of the longitudinal reinforcement in a cross-section

9.1.3.1. Simultaneous use of steel of different types

The simultaneous use of steels of various types is allowed on condition that this is taken into account in the design and that no confusion is possible during execution.

It shall be possible to distinguish clearly between

- bars of various grades
- reinforcement that is weldable and that which is not
- type of steel with respect to ductility (see section 2.2).

Special attention, both in design and construction, is needed in cases of simultaneous use of various steels in the same element (e.g. S220 and S400 as main reinforcement in slabs).

9.1.3.2. Clear distance in the horizontal and vertical direction

The bars in the various horizontal layers should be arranged in vertical planes, leaving sufficient space between them to allow for internal vibration.

The intermediate horizontal or vertical free space between parallel single bars or horizontal layers of parallel bars, should be at least equal to the largest bar diameter but not less than 20 mm.

Bars spliced by lapping may be in contact along the lap length.

The maximum size of the aggregate should be chosen to facilitate concreting and adequate compaction of the concrete surrounding the bars.

9.1.4. Additional rules for high-bond bars of large diameter

For high-bond bars of diameter $\phi > 32$ mm, the following rules supplement those given in subsections 9.1.1 and 9.1.2.

Splitting forces are higher and dowel action is greater in case of large-diameter bars; hence reinforcement should be located in the corners of stirrups. A stirrup can surround at most three bars per layer in beams and in slabs where shear reinforcement is necessary.

Where there are more than two layers, the bars situated near the walls of the section should be surrounded by additional stirrups with ends parallel to the planes of the layers of reinforcement

(*a*) When large diameter tension bars are used, a skin reinforcement is needed to keep the crack widths within acceptable limits, unless a verification by calculation is carried out in accordance with section 7.4.

The area of skin reinforcement referable to the skin concrete section $A_{ct,ext}$ (see Fig. 9.1.10) should be not less than

- 0.01 in the direction perpendicular to large diameter bars
- 0.02 parallel to those bars.

The skin reinforcement can be taken into account in the design, provided that it meets the requirements for the arrangement and anchorage of these types of reinforcement.

Skin reinforcement shall have adequate concrete cover (see clause 8.4.3d).

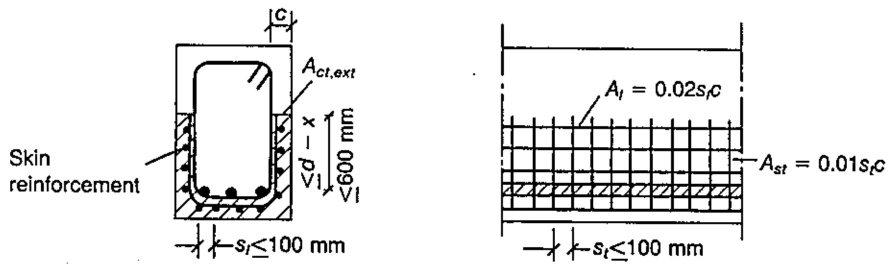


Fig. 9.1.10. Skin reinforcement (*x is the depth of the neutral axis at the ultimate limit state*)

In the absence of transverse compression, additional transverse reinforcement is required in the anchorage zone (see section 9.1.2). For straight anchorages (see Fig. 9.1.11), the area of this additional reinforcement should not be smaller than

- in the direction parallel to the tension face

$$A_{st} = 0.25\, A_s n_l \tag{9.1–1}$$

- in the direction perpendicular to the tension face

$$A_{sv} = 0.25\, A_s n_2 \tag{9.1–2}$$

where

A_s denotes the area of an anchored bar
n_l is the number of layers with anchored bars in a section
n_2 is the number of anchored bars in one layer.

Transverse reinforcement should be uniformly distributed in the anchorage zone with spacings which should not exceed five times the diameter of the longitudinal reinforcement.

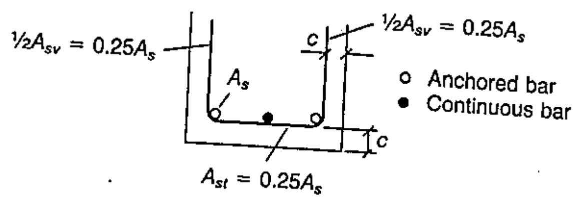


Fig. 9.1.11. Additional reinforcement in an anchorage zone with bar diameter greater than 32 mm and no transverse compression: in this case, $n_l = 1$, $n_2 = 2$, and $A_{st} = \frac{1}{2}A_{sv} = 0.25A_s$

(b) Additional care should be taken in the anchorage zones of large diameter bars which should be anchored as straight bars or through mechanical devices.
(c) Lapped splices of large diameter bars are not recommended.

Sections 6.9 and 7.4 and subsections 9.1.3 and 9.1.4 concerning free space, bond, crack control and skin reinforcement, apply, taking into account the equivalent diameter.

Not more than two bars in contact may be placed in a single plane.

9.1.5. Additional rules for bundled bars

9.1.5.1. General

The number of bars in the bundle should be limited to

- $n \leqslant 4$ for vertical bars in compression and for the bars of a lapped splice
- $n \leqslant 3$ for all other cases.

9.1.5.2. Equivalent diameter

For design purposes, bundles of bars containing n bars having the same diameter are replaced by a single notional bar having the same centroid, and an equivalent diameter

$$\phi_n = \phi\sqrt{(n)} < 55\,\text{mm} \qquad (9.1\text{–}3)$$

For bundles comprising n bars of different diameters, ϕ_n is the diameter of a notional bar with the same area and the same centroid as the bundle of bars under consideration.

9.1.5.3. Minimum concrete cover

The equivalent diameter ϕ_n is taken into account in the evaluation of the minimum cover. However, the cover provided should be measured from the actual outside contour of the bundle of bars.

9.1.5.4. Horizontal and vertical free distances

Subsection 9.1.3 applies, taking into account the equivalent diameter ϕ_n.

It may be necessary to adopt spacings greater than the minimum spacings to allow concreting and the passage of an internal vibrator (especially when a large number of bundles is used), unless special measures are taken to obtain good compaction of the concrete surrounding the bars.

9.1.5.5. Anchorage

The anchorages of the individual bars of a bundle should be straight.

The anchorages should be staggered: for bundles of 2, 3 or 4 bars, the staggering should be respectively 1.2, 1.3 or 1.4 times the anchorage length of the individual bars.

The anchorage length of the whole of the bundle may also be determined on the basis of the equivalent diameter ϕ_n. This method may be used for anchorages over supports.

9.1.5.6. Lapped joints

Joints can be made on only one bar at a time but at any one section. The laps of the individual bars should be staggered in accordance with clause 9.1.5.5.

In the case of laps using sleeved compression joints for bundles, the distance between adjacent sleeves can be reduced to twice the diameter of the sleeve.

9.1.6. Detailing rules for zones of introduction of prestressing forces

The rules presented in this subsection make use of those given in sections 3.3 (local compression), 6.3.4 (longitudinal shear in flanged sections), 6.8 (discontinuity regions), 7.3 (stress limitations) and 7.4 (limit state of cracking).

The quantitative rules of this subsection are meant for anchorages of post-tensioned tendons. They have to be supplemented in the case of couplers. They remain qualitatively valid (needing however some specific supplements) for zones of anchoring of pretensioned tendons and zones of deviation of external tendons.

9.1.6.1. Spreading of the prestressing force

A zone close to any anchorage is a discontinuity region.

It extends on both sides of the anchorage and in all directions.

The extent of this discontinuity zone in front of the anchorage (i.e. conventionally on the same side as the tendon) is larger than (commonly twice) its extent behind the anchorage.

The main part of the stress field in the discontinuity region consists of the spreading of the prestressing force by compression from the anchorage up to the end (in front of the anchorage) of the discontinuity region. A simplified model is based on the assumption that the force generates compressive stresses uniformly spread from the anchorage device within an angle 2β, where $\tan 2\beta = 2/3$ (Fig. 9.1.12).

In fact, $\tan 2\beta$ may range between 2/3 and 1 (see clause 9.1.6.2c).

For a T-beam it is assumed that the dispersion of the prestress is effectuated

- in the middle plane of the web, starting from the anchorage device, within an angle 2β
- in the middle plane of the top slab, if the spreading in the web reaches it, on both sides of the rib, following an angle β.

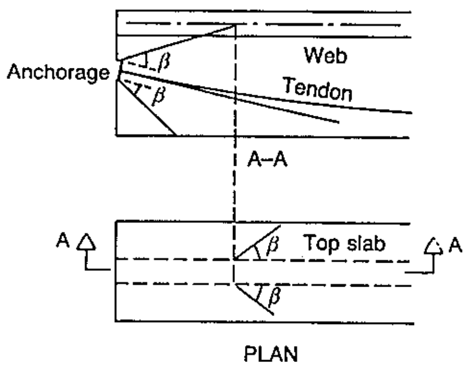


Fig. 9.1.12. Dispersion of prestress

For example, box girders.

For other shapes of members more detailed models should be used.

9.1.6.2. Local analysis of the discontinuity region

(*a*) The spreading of the prestressing force in a field of compressive stresses results, for equilibrium

(i) in tensile spalling and bursting stresses perpendicular to the compressive stresses

(ii) in the case of T-beams, in shear stresses between web and flange.

Other tensile or shear stresses may be necessary for equilibrium for other member shapes.

The location in space of all these stresses may be assessed approximately.
The resultants of these stress fields shall be superimposed on the other action effects according to section 6.3, in order to establish a more complete truss model to be used for the verification of the resistance of the region (ultimate limit state).

(*b*) Appropriate models should be used to control the widths of cracks developed in the discontinuity region due to local compression deformation.

In cases where several anchorages are considered in the same region, specific schemes of cracks due to the effects of these anchorages may occur and, if the crack widths are not controlled, they may result in ultimate failure (see CEB Bulletin 163).

9.1.6.3. Additional reinforcement in the discontinuity region

The limit measures relating to reinforcement are applicable to all the following types of reinforcement.

(*a*) Reinforcement shall be provided and designed for a sufficient resistance (ULS) in all places where tensile or shear internal forces are put in evidence by the model obtained by superimposition according to clause 9.1.6.2(a).

More concentrated prestressing forces (stronger anchorage devices) may necessitate supplementary measures for crack control.

The location of the reinforcement may be found approximately.
Examples are the reinforcement A_{s1} and A_{s2} in Fig. 9.1.13 and a shear reinforcement between web and flange in the case represented in Fig. 9.1.12 (compare also clause 6.9.12.1).

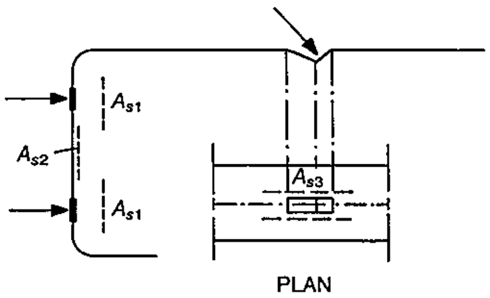


Fig. 9.1.13. Local reinforcement in the vicinity of anchorages of tendons

(*b*) Closely spaced reinforcement is necessary where internal tensile deformations are identified according to clause 9.1.6.2(b), in order to reduce these deformations and spread them over several small-width cracks.

(*c*) Sufficient reinforcement is necessary for crack control (ductility requirement) in all other parts of the discontinuity region where it is envisaged that tensile stresses may occur. In all these parts a minimum reinforcement defined by eq. (7.4–16) should be provided.

These bars shall be sufficiently long in order to cover the uncertainty of the models used.
Examples are the reinforcements A_{s2} and A_{s3} in Fig. 9.1.13.

Because of the high uncertainty of the local analysis, small compressive stresses may in reality turn out to be tensile stresses.

9.1.7. Horizontal and vertical clear distance for internal prestressing steels

9.1.7.1. General

The tendons for internal post-tensioning and pretensioning should be placed, considering simultaneously the sheathing and ordinary reinforcement where relevant, in such a manner that the concrete may be easily placed between them.

9.1.7.2. Post-tensioning

The sheathings shall be located so that

- the concrete can be safely placed without damaging the sheathings
- the concrete can resist the forces from the sheathings in the curved parts under and after tensioning
- no grout will leak into other sheathings during the grouting process.

The minimum horizontal and vertical free spacing for the tendons are given in Fig. 9.1.14.

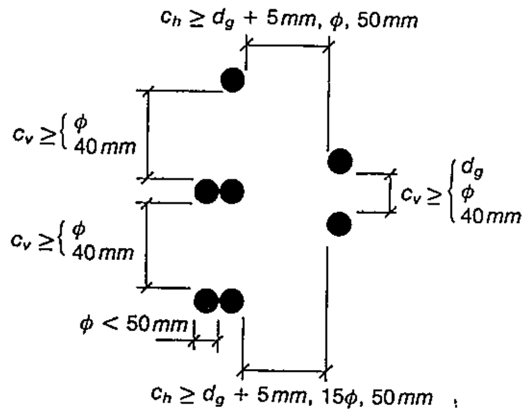


Fig. 9.1.14. Minimum clear spacing for sheathings (where d_g is the maximum aggregate size), see Appendix d.6.5 and ISO 565

When bundles of sheathings are used, the horizontal free spacing between two bundles should be calculated on the basis of the equivalent diameter of bundles.

9.1.7.3. Pretensioning

The minimum horizontal and vertical free spacing of tendons are given in Fig. 9.1.15.

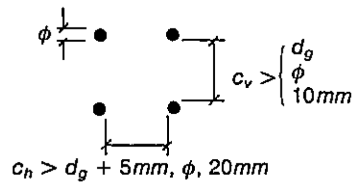


Fig. 9.1.15. Minimum clear spacing for pretension tendons (where d_g is the maximum aggregate size)

Other layouts are acceptable in special cases, provided that test results show satisfactory ultimate behaviour with respect to

- the concrete in compression at the anchorage
- the spalling of concrete
- the anchorage of pretensioned tendons
- the danger of corrosion of the tendons at the end of elements
- the placing of the concrete between the tendons.

9.2. DETAILING OF STRUCTURAL MEMBERS

The quantitative rules of this section are not valid for prestressed elements. However, they are qualitatively valid also for prestressed elements. In case of prefabrication see chapter 14.

9.2.1. Slabs

This subsection applies to rectangular solid slabs, supported by beams and cast in situ, which satisfy the conditions given in section 5.5 and for which the smallest dimension is not less than five times the effective depth.

Slabs may be classified as

- one-way slabs, which are calculated to resist flexural stresses in only one direction
- two-way slabs, which are calculated for flexure in more than one direction.

Conventionally, terms used to describe the geometry (e.g. lateral, horizontal) refer to a slab with a horizontal midplane. The 'top' face of a slab is that on which a load is applied, which is assumed to exert a pressure.

This subsection does not apply to one-way slabs which can be assimilated to beams, i.e. slabs where

- the two free edges are quasi-parallel, at a distance at least equal to three times the total depth
- the main bending moment quasi-parallel to the free edges is much higher than the other main bending moment having the same sign.

9.2.1.1. Main flexural reinforcement

9.2.1.1.1. General

Appropriate minimum reinforcement percentages are needed in order to satisfy several requirements exceeding those covered by calculation.

In general, a slab may be reinforced with four layers (two 'lower' layers and two 'upper' layers). The given rules apply separately to each layer.

The reinforcement is called 'main reinforcement' irrespectively of its direction when it is provided by calculation to resist normal action effects (see section 6.4).

The reinforcement is called 'secondary' reinforcement when it is provided transverse to the main one to undertake specific (not calculated) action effects such as local transverse moments, temperature or shrinkage effects.

In two-way slabs, the reinforcement provided in both directions is considered as 'main' reinforcement; the reinforcement perpendicular to each direction should also satisfy the rules given for secondary reinforcement.

The secondary reinforcement should be provided even if reinforcement is required in one direction only (e.g. flange beams, see clause 9.2.2.3).

The maximum spacing of bars is recommended as follows

- for main reinforcement $s_{max} < 1.2h$ or 350 mm whichever is the less
- for secondary reinforcement $s_{max} < 2h$ or 350 mm whichever is the less

where h denotes the total depth of the slab.

The yield force provided by the secondary reinforcement should not be less than 0.2 times that of the main reinforcement at any section.

For high concentrated loads this ratio should be at least equal to 0.33.

9.2.1.1.2. Partially restrained edges

If the edge of a slab is partially restrained and this restraint has not been considered in the analysis, top reinforcement should be able to balance at least one-quarter of the absolute value of the maximum moment in the corresponding span. This reinforcement should extend from the face of the support over a distance of at least 0.2 times the corresponding span.

Corner lift creates tensions in the 'upper' face which act roughly in the direction of the bisector of the corner angle, and create tensions in the 'lower' face at right angles to that bisector.

9.2.1.1.3. Corner top reinforcement

If the corner of a slab formed by two simply supported edges is prevented from lifting and such restraint is not taken into account in the analysis, and if the existing upper reinforcement is not capable of resisting a moment at least equal to the value of the maximum moment in the span, additional reinforcement should be provided at the corner.

If at the corner, one edge is simply supported and the other restrained, the total top orthogonal reinforcement should be capable of resisting a moment equal to at least one-quarter of the maximum moment in the span.

Restrained corner reinforcement can be omitted if the consequence of cracking in the corner of the slab is not important.

The corner top reinforcement should extend from the face of the support over a distance of at least 0.2 times the smaller span.

Shear reinforcement is not necessary in slabs if $V_{Sd} \leqslant V_{Rd1}$ (see subsection 6.4.2).

9.2.1.1.4. Staggering rule for slabs without shear reinforcement

One half of the maximum area of reinforcement needed in the span should be extended as far as the supports, where it should be suitably anchored.

9.2.1.2. Shear reinforcement

9.2.1.2.1. General

A condition which is analogous to that in clause 9.2.1.1.4 is given in the rule on staggering (see clause 9.2.2.5).

The stirrups should surround the bars (i.e. the top longitudinal reinforcement and the bars of the bottom reinforcement). In general, their slope to the middle plane of the slab should lie between 45° and 90°.

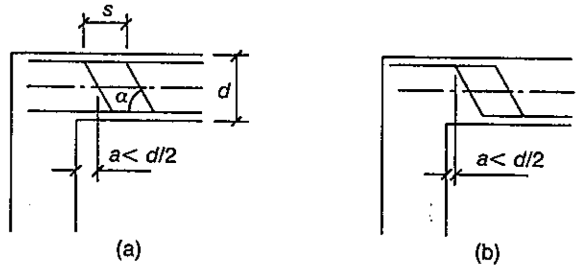


Fig. 9.2.1. Recommended spacing of shear reinforcement: (a) stirrups; (b) bent-up bars

Generally, the angle of bent-up bars to the horizontal should not be less than 30°.

The spacings of the various layers of shear reinforcement should satisfy the condition

$$s \leqslant 0.75d(1 + \cot\alpha) \qquad (9.2\text{–}1)$$

The distance between the face of a support and the nearest layer of shear reinforcement should not exceed $d/2$. That distance is to be taken

- for stirrups, at the middle plane of the slab
- for bent-up bars, at the level of the top reinforcement in bending.

9.2.1.2.2. Zones close to linear supports

The shear reinforcement should be arranged in zones where V_{Sd} is greater than V_{Rd1} (see clauses 6.4.2.3 and 6.4.2.5). A minimum percentage of reinforcement is required.

The transverse spacing of bars in the same layer of shear reinforcement should not exceed $1.5d$ or 800 mm whichever is the smaller.

The shear reinforcement may consist solely of bent-up bars if

$$V_{Sd} \leqslant F_{Rcw}/3$$

where F_{Rcw} is the resistance of web concrete in compression as calculated by eq. (6.3–11).

If not, the stirrups should by themselves satisfy the minimum percentage rule.

9.2.1.2.3. Staggering rule and anchorage lengths of main reinforcement

Clause 9.2.2.5, which gives staggering rule for beams, applies to one-way and two-way spanning slabs.

See subsection 6.4.3.

9.2.1.2.4. Punching shear reinforcement

For punching shear reinforcement account can be taken of

- stirrups contained in a zone at a distance not exceeding $2d$ and 800 mm from the loaded area; the condition resulting from eq. (9.2.–1) should be respected in all directions
- bent-up bars passing above the area defined by a contour line located a distance $d/4$ away from the contour line of the loaded area.

9.2.1.3. Free edges

Along the length of a free edge, a slab should contain (see Fig. 9.2.2)

- reinforcement parallel to the edge consisting of at least two bars, one in the top corner and the other in the bottom corner
- reinforcement perpendicular to the edge, and of which the free ends extend up to a distance of at least $2h$ from the edge.

In thick slabs, the reinforcement along the length of a free edge is to be distributed over the slab thickness.

The reinforcement perpendicular to the edge may consist of U-shaped stirrups enclosing the longitudinal bars along the edge.

The ordinary reinforcement of the slab may form an edge reinforcement.

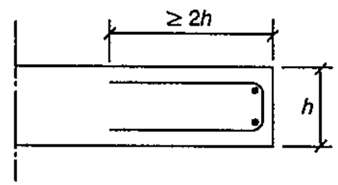


Fig. 9.2.2. Reinforcement along a free edge

9.2.1.4. Hollow or ribbed slabs

The rules given in clauses 9.2.1.1 to 9.2.1.3 can be applied where appropriate.

The top slab of ribbed or hollow block slabs should be reinforced with a mesh providing, in each direction a cross-sectional area not less than 0.001 of the section of the top slab.

If the rib spacing exceeds 1 m, reinforcement in conformity with sub-section 6.3.4 should be provided.

See commentary to section 9.2.

9.2.2. Beams

9.2.2.1. Longitudinal reinforcement

A minimum area of longitudinal bonded reinforcement should be provided to avoid brittle failure in case of unforeseen loss of concrete tensile strength.

If a specific study is not carried out in this respect, the area of longitudinal tensile bonded reinforcement provided should be at least taken equal to

- $0.0015 b_t d$ for steel grades S400 and S500
- $0.0025 b_t d$ for steel grade S220

where b_t is the average width of the concrete zone in tension.

In a T-beam, if the neutral axis in the ULS is located in the flange, the width of the latter is not taken into account in evaluating b_t.

See clause 9.2.3.2.

A minimum transverse reinforcement could also be required to prevent buckling of the longitudinal compression reinforcement.

If a specific study is not carried out in this respect, this maximum ratio may be taken equal to 4% other than at laps.

Appropriate maximum values of tensile reinforcement ratio should be respected in order to ensure a minimum level of ductility.

9.2.2.2. Shear reinforcement

Closed stirrups can be arranged as shown in Fig. 9.2.3. The lap splice of stirrup in the web shown in Fig. 9.2.3(d) is allowed only for high-bond bars, provided that there is no risk of corrosion of the stirrup.

The shear reinforcement should form an angle of 90° to 45° with the axis of the beam.

In most cases, the shear reinforcement in beams consists of vertical stirrups enclosing the longitudinal tensile reinforcement (Fig. 9.2.3) and anchored according to clause 9.1.1.4.

It may also consists of a combination of stirrups and

Attention is drawn to the risk of splitting cracks to open when the longitudinal bars, which are near the outer faces of elements, are bent-up.

- bent-up bars
- shear assemblies in the form of cages or ladders of high-bond bars that are cast in without enclosing the longitudinal reinforcement (see Fig. 9.2.4). Their anchorage should comply with the provisions of clause 9.1.1.4 (see Fig. 9.1.6(c) and (d).

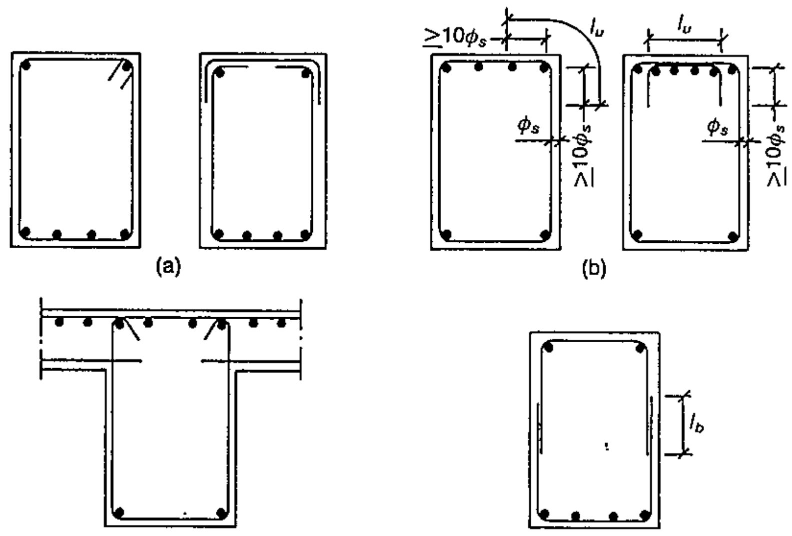


Fig. 9.2.3. Possible layout of stirrups: (a) closing in compression zone; (b) closing in tensile zone; (c) closing in compression zone; (d) this detail has been used but does require particularly good workmanship to achieve its design purpose

For bent-up-bars, the minimum mandrel diameter should be not less than the values given in Table 9.1.1.

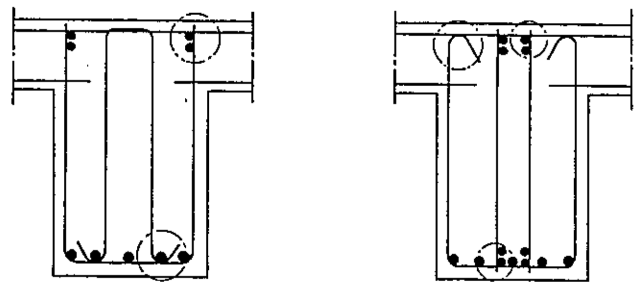


Fig. 9.2.4. Examples of combination of stirrups and shear assemblies: for detailing of the anchorages in the circled zones, see Fig. 9.1.6

The values given in Table 9.2.1 should be taken as minimum reinforcement percentage.

$$\rho_w = \frac{A_{sw}}{s b_w \sin\alpha} \qquad (9.2\text{–}2)$$

See clause 6.3.3.1.

Except for the cases listed in clause 6.3.3.1 any beam should have a minimum number of ties or stirrups over its entire length. This minimum reinforcement is required to ensure that the failure load is higher than the cracking load.

Smooth shear reinforcement bars should not exceed 12 mm in diameter.

The maximum spacing s_{max} of various layers of shear reinforcement is defined by the following conditions

$$s_{max} = 0.7d < 300\,\text{mm} \quad \text{for } F_{Scw} \leqslant \tfrac{1}{5}F_{Rcw}$$

$$s_{max} = 0.6d < 300\,\text{mm} \quad \text{for } \tfrac{1}{5}F_{Rcw} < F_{Scw} \leqslant \tfrac{2}{3}F_{Rcw}$$

$$s_{max} = 0.3d < 200\,\text{mm} \quad \text{for } \tfrac{2}{3}F_{Rcw} < F_{Scw}$$

where F_{Scw} is the acting compression force of web concrete (see eq (6.3–10)) and F_{Rcw} is the resistance of web concrete in compression as calculated by eq (6.3–11).

The transverse spacing of legs in each group should not exceed $2d/3$ or 800 mm, whichever is smaller.

Table 9.2.1. Minimum reinforcement values of ρ_w in percent

Concrete	Minimum reinforcement [%]		
	steel S220	steel S400	steel S500
C12	0.15	0.08	0.06
C20	0.20	0.11	0.09
C30	0.26	0.15	0.12
C40	0.32	0.18	0.14
C50	0.37	0.21	0.16
C60	0.42	0.23	0.18
C70	0.46	0.26	0.20
C80	0.51	0.28	0.22

9.2.2.3. Beam flanges

For the area of transverse reinforcement (tying a beam flange to the web) calculated in accordance with subsection 6.3.4, a minimum reinforcement is normally required.

See subsection 6.3.4 for the calculation of this reinforcement. The minimum percentage of the total reinforcement crossing the connection, calculated by eq. (9.2–2) replacing b_w by the thickness of the top slab, can be taken equal to the values in Table 9.2.1.

9.2.2.4. Torsional reinforcement

Clauses 9.2.2.1 and 9.2.2.2 also apply to the longitudinal bars and stirrup of beams subjected to torsion, except the stirrup arrangements of Fig. 9.2.4 which are not allowed in the case of torsion.

The spacing of stirrups should not exceed a value $u_{ef}/8$, where u_{ef} denotes the length of the perimeter of the stirrups.

The longitudinal bars shall be placed so that at least one bar is located in each corner of the stirrup, the other bars being distributed uniformly along the internal perimeter of the stirrups, at a spacing not exceeding 350 mm.

The values indicated in clauses 9.2.2.1 and 9.2.2.2 can be adopted for the torsional reinforcement as well.

For notation, see subsection 6.3.5.

9.2.2.5. Curtailment of bars

The tensile envelope should be calculated from eq. (6.3–4). Beyond the section, where the reinforcement is no longer required to carry the full force, the reinforcement can be curtailed with an anchorage length of $l_{b,net}$ + 100 mm.

Unless the section of zero moment is exactly known, a horizontal range equal to the possible variation range of the zero moment shall be assumed.

Since shear forces are taken into account when calculating the tensile chord forces (section 6.3), there is no need for any additional 'moments' staggering rule.

For the definition of $l_{b,net}$ see Fig. 6.9.6.

9.2.3. Columns

9.2.3.1. Longitudinal reinforcement

(*a*) Unintended eccentricities and the need for controlling creep deformations leads to the requirements of a minimum percentage of longitudinal reinforcement.

(*b*) In order to ensure easy compaction of concrete and safe splicing of longitudinal bars, an appropriate maximum steel percentage should be respected.

(*c*) The minimum number of longitudinal bars is four for rectangular columns and six for circular columns.

For some other limit measures, see chapter 10.

If a more specific study is not carried out in this respect, a value equal to $0.008A_c$ may be used.

In the absence of a specific study on the matter, the area of the longitudinal reinforcement should not exceed $0.04A_c$, with the exception of regions of lap splicing where it can reach $0.08A_c$.

The diameter of longitudinal bars should preferably not be less than 12 mm.

9.2.3.2. Transverse reinforcement

Generally, the transverse reinforcement consists of stirrups surrounding the longitudinal reinforcement.

The diameter of stirrups shall not be less than 5 mm or one-quarter of the maximum diameter ϕ_l of the longitudinal bars. Their spacing

- shall secure longitudinal compression bars against local buckling
- shall ensure that stirrup legs intersect at least one possible shear crack under the most adverse condition.

The transverse reinforcement shall be so arranged that each bar or group of bars placed in a corner, and one of every two intermediate bars of the outer layer of reinforcement are held. Bars can be considered to be held if they are not located at a distance of more than 150 mm from a held bar.

Correct lateral tying of circular columns can be achieved with the aid of hoops or helices around the longitudinal bars or groups of bars.

All transverse reinforcement (ties, stirrups or helices) should be appropriately anchored (see clause 9.1.1.4).

If a more specific study is not carried out, the following values may be used

$$\text{spacing } s = 12\phi_l$$
$$s = \min\{h_c, 300\,\text{mm}\}$$

where h_c denotes the smallest dimension of the column.

Under certain circumstances, it may be necessary either to increase the stirrup diameter or to decrease spacing, to prevent bursting or any local secondary damage; special precautions are to be taken

- in zones located on either side of a beam or a slab, over a height equal to the larger dimension of the column section
- at changes of direction of the longitudinal bars.

9.2.4. Reinforced concrete building walls

This subsection deals with reinforced concrete walls of which the length measured horizontally is at least equal to four times the thickness.

For some other limit measures, see chapter 10.

9.2.4.1. Vertical reinforcement

The area of the vertical reinforcement shall lie between $0.004\,A_c$ and $0.04\,A_c$, where A_c is the required sectional concrete area.

In general, half of this reinforcement should be located at each face.

The distance between two adjacent vertical bars should not exceed twice the wall thickness or 300 mm, whichever is lesser.

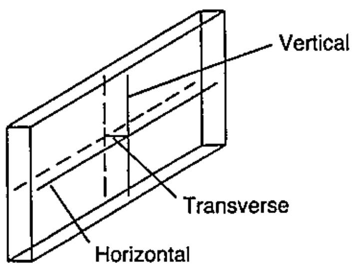


Fig. 9.2.5. Terminology of wall reinforcement

9.2.4.2. Horizontal reinforcement parallel to the wall faces

Horizontal reinforcement running parallel to the faces of the wall (and to the free edges) should be provided and arranged at each surface. Its minimum section should not be less than 30% of that of the vertical reinforcement. The spacing between two adjacent horizontal bars on the same surface should not be greater than 300 mm. The diameter should not be less than one-quarter of that of the vertical bars.

The necessary reinforcement for the control of cracking is given in section 7.4.

9.2.4.3. Horizontal reinforcement perpendicular to the wall faces (transverse reinforcement)

If the area of the load carrying vertical reinforcement exceeds 0.02 A_c then clause 9.2.3.2 applies.

9.2.5. Deep beams

9.2.5.1. Simply supported deep beam on two supports

9.2.5.1.1. Longitudinal reinforcement

The main longitudinal reinforcement corresponding to the ties considered in the design model should be uniformly distributed over a depth measured from the lower face of the beam of about 0.12h or 0.12l whichever is lesser where

- h is the total height of the beam
- l is the design span.

It should be fully extended from one support to the other.

At supports, the anchorage should be obtained by using horizontal hooks or U-loops or by anchorage plates, unless the length between the support and the end of the beam is greater than the anchorage length of $l_{b,net}$ (see subsection 6.9.1).

Because of the danger of splitting, anchorage by means of vertical hooks should be avoided.

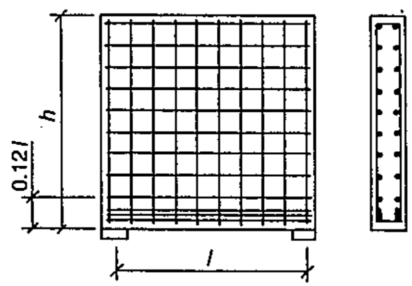


Fig. 9.2.6. Distribution of longitudinal reinforcement in a deep beam in case of direct loading (schematic)

9.2.5.1.2. Additional reinforcement

(a) Direct loading (the load is applied on the top of the beam)

In this case, an additional reinforcement in the form of a mesh of orthogonal reinforcement consisting of a horizontal layer surrounded by a vertical layer should be arranged.

The total percentage of the bars in each direction should not be smaller than 0.2% (i.e. 0.1% for each face).

(b) Suspended loading (the load is applied at the bottom of the beam)
In this case, the orthogonal mesh described in (a) above should be supplemented by additional stirrups to transmit the total load between its application level and the level corresponding to the lesser of h and l.

The reinforcement should surround the bars of the main reinforcement and be fully extended over a depth equal to the lesser of h and l.

Near the supports the height of the stirrups may be slightly reduced (by about 20%).

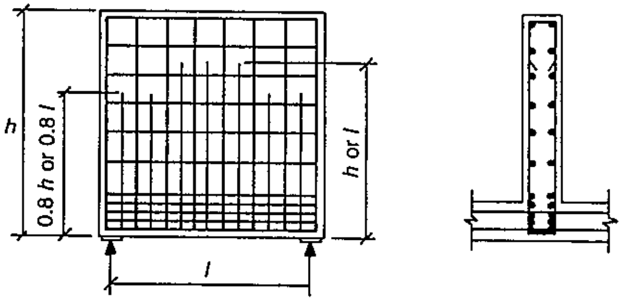


Fig. 9.2.7. Recommended reinforcement layout for suspended loading (schematic)

(c) Vertically distributed load
According to the chosen design model, the force transmitted to the deep beam should be resisted by an additional reinforcement (suspension reinforcement) made either of vertical stirrups extended without cut-off near the common volume over a length equal to the lesser of h and l (Fig. 9.2.8(a)), or by bent-up-bars (adequately anchored) resisting about 60% of the load placed symmetrically to the line of action of the load, and by complementary stirrups (Fig. 9.2.8(b)).

This case corresponds to a load applied over the total depth of the beam by means of a transverse perpendicular wall or by a column of large cross-section which is extended down the lower part of the beam.

For the minimum diameter of the curvature of the bars, see Table 9.1.1.

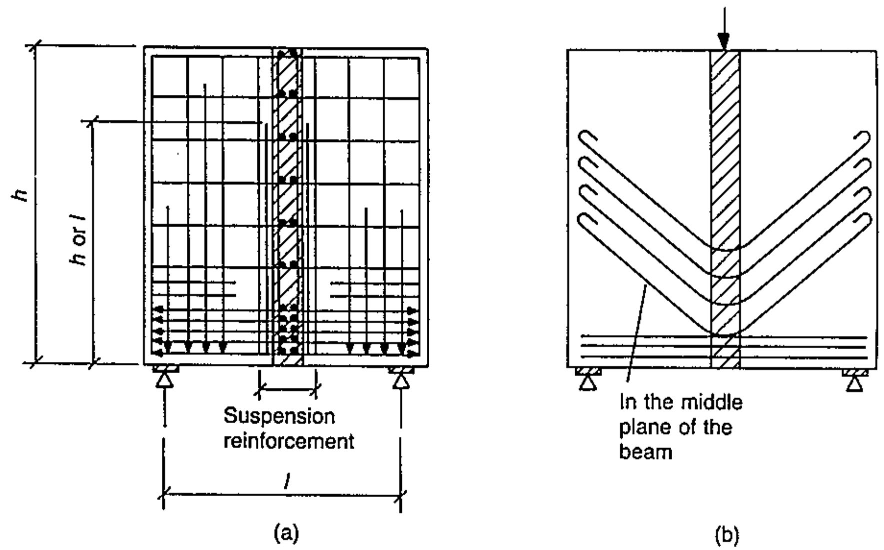


Fig. 9.2.8. Reinforcement for vertically distributed load

9.2.5.2. Continuous deep beams

(a) In the span (i.e. in the positive moment area)
For the main reinforcement as well as for shear reinforcement, clause 9.2.5.1 applies.

(b) Over the supports
For the main horizontal tensile reinforcement

- a fraction $[(l/h) - 1)]/2$ of the total required cross-sectional area of reinforcement should be placed in the upper strip which extends to $0.2h$ or $0.2l$ whichever is the less (see Fig. 9.2.9)
- the remaining cross-sectional area should be uniformly distributed within the lower strip just below, which extends to $0.6h$ or $0.6l$, whichever is the less (see Fig. 9.2.9)

Where $h > l$ supplementary longitudinal reinforcement should be placed near the upper face of the beam.

- one bar on two may be stopped symmetrically at a distance from each face of the support equal to $0.4h$ or $0.4l$, whichever is the less.

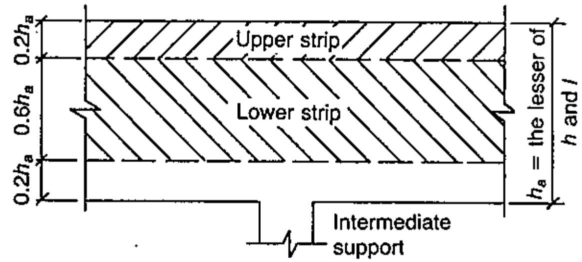


Fig. 9.2.9. Distribution of reinforcement over the supports

10. LIMIT MEASURES

10.1 INTRODUCTION

This chapter contains information related to required 'limit measures' (i.e. minimal and/or maximal rules) regarding

- the materials to be used
- the dimensions of concrete cross-sections and covers
- the cross-sections and arrangement of steel.

This information is mainly a summing up of relevant clauses from all the Model Code chapters, completed in some cases with guidance based on previous experience. Further limit measures may be needed for other cases not covered by this Code.

These limit measures may have been dictated by several conditions and requirements such as

(*a*) conditions for the validity or the avoidance of calculations
(*b*) minimum ductility needed
(*c*) qualitative reliability and insensitivity requirements
(*d*) functional demands
(*e*) durability
(*f*) friendliness of execution.

It should be noted, however, that a limit measure may serve, simultaneously, more than one of the above listed conditions and requirements.

The application of the Code is only possible if all these conditions are appropriately considered by the designer (see also subsection 1.1.5), even in case this Model Code does not foresee explicit limit measures for all of them.

On the other hand, when such explicit measures are foreseen, the designer may also make use of alternative means to face the above mentioned six conditions or requirements, provided that full evidence is available on the matter.

This chapter intends to assist the designer in the first step of design, i.e. in conceptual design (before a detailed analysis and verification are performed). After the selection of a structural system, dimensions have to be preliminarily given to concrete and eventually steel; thus, stiffnesses may be determined for the analysis, and verifications may be carried out at the end.

These preliminary dimensions are given out of the experience of the designer; however, a certain guidance may be offered by summing up all information related to 'limit measures' required throughout the Code. These limit measures may have to do with the selection of the quality of materials to be used, as well as with concrete dimensions and steel sections; all may influence predimensioning as well as final verifications.

Limit measures (see subsection 1.1.5) may have been foreseen in a code for the following possible reasons.

(*a*) The validity of the engineering models or of the practical rules suggested for design may be secured only under conditions regarding the quality of materials or the dimensions of structural elements (e.g. maximum distance of stirrups in beams, maximum concrete class, conditions of release of calculating secondary reinforcement, etc.). In the same category belong the conditions allowing for the avoidance of some calculations.

(*b*) A minimum level of ductility is needed, even in cases this fundamental property is not explicitly checked; and such minimum ductility may need certain limit measures (e.g. maximum steel ratio) in order to be implemented.

(*c*) Qualitative reliability and insensitivity requirements may dictate some minimal measures (e.g. a minimum thickness of elements or the need for at least two tendons available in each concrete section).

(*d*) Functional requirements may also be covered by minimal measures (e.g. the non-damageability of non-structural elements is secured by restricting the deformability of slabs).

(*e*) Practical durability measures may directly concern minimum covers, minimum bar-diameters, minimal concrete classes, etc.

(*f*) Friendliness of execution may dictate minimum values of thickness of concrete sections, maximum steel percentage, etc. Finally, some inspectability considerations may influence dimensioning.

This is mainly the case for those measures which are not compulsory.

10.2. QUALITY OF MATERIALS

10.2.1. Concrete grades

(*a*) A minimum concrete grade is specified to ensure the validity of models and to ensure good durability.

(i) Only grades C16 and above shall be used for reinforced concrete (clause 2.1.1.2).

(ii) Only grades C25 and above shall be used for prestressed concrete (clause 2.1.1.2).

(iii) For plain precast wall panels ($\rho < 3‰$) grade C16 and above shall be used, while for reinforced precast wall panels ($\rho \geqslant 3‰$) grade C20 and above shall be used (clause 14.2.5.6).

For compression joints (precast, subsection 14.3.2) the strength of joint mortar material should not be less than 70% of the adjacent precast concrete strength (if the mortar is not transversely confined), unless the joint is designed accounting for actual mortar strength.

For segmental construction, the compressive strength of the material of wide joints should be not less than 25 MPa or that of the joined segments (clause 14.7.1.3), while the compressive strength of the material of match joints should be at least equal to that of the concrete of the joined segments (clause 14.7.1.4).

(*b*) Concrete constitutive laws and models should be used with caution for $f_{ck} > 50$ MPa (clause 2.1.1.1); the relation between shear resistance and concrete strength depends (for higher concrete grades) upon the characteristics of the aggregates (clause 6.4.2.3).

In addition, an upper concrete grade should be foreseen taking into account the conditions of minimum ductility and ease of execution. In this Model Code this limit is C80.

10.2.2. Reinforcing steel

(*a*) Grades higher than S500 require further consideration concerning the validity of the given rules, i.e. $\sigma - \varepsilon$ diagrams, bond, etc. (clause 2.2.4.2).

Adequate margins of safety and low local damageability are ensured by these minimal measures.

(*b*) Regarding ductility, necessary whether or not moment redistribution is taken into account (clause 2.2.4.4), three classes are foreseen

class A: $(f_t/f_y)_k \geqslant 1.08$, $\varepsilon_{uk} \geqslant 5.0\%$
class B: $(f_t/f_y)_k \geqslant 1.05$, $\varepsilon_{uk} \geqslant 2.5\%$
class S: $(f_t/f_y)_k \geqslant 1.15$, $\varepsilon_{uk} \geqslant 6.0\%$.

To be used where high ductility of structural elements is required (i.e. in seismic regions).

In seismic regions, additional requirements for Class S may be introduced, e.g. in order to reduce uncertainties in applying the capacity design criterion.

$$(f_{yk})_{act}/(f_{yk})_{nom} \leqslant 1.30$$

10.2.3. Prestressing steel

(*a*) Grades

Lower limiting values for the characteristic tensile strength of prestressing steels are dictated by the applicability of prestressing, where upper limiting values are imposed by the possibilities of production of prestressing steels.

Other conditions are foreseen as well, regarding reverse bending behaviour, constriction, etc.

(*b*) Regarding ductility (clause 2.3.4.4), the unit elongation at maximum load (ε_{uk}) shall be at least equal to 0.035.

*Table 10.2.1. Limit measures concerning quality of materials**

	Validity of models/ calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Concrete grade Minimum	RC: C16 PC: C25 Precast: C16/C20	—	•	—	•	—	2.1.1.2 14.2.5.6
Maximum	C50	•	—	—	—	•	2.1.1.1 6.4.2.3
Reinforcing steel Max. f_{yk}	S500	—	—	—	—	—	2.2.4.2
Ductility	—	3 classes $\varepsilon_{uk} \geqslant \ldots$ $(f_t/f_y)_k \geqslant \ldots$	—	—	—	—	2.2.4.4
Prestressing steel Max. f_{pyk}	—	—	—	—	—	—	—
Ductility	—	$\varepsilon_{uk} \geqslant 0.035$ and other conditions	—	—	—	—	2.3.4.4

*Among the rules not explicitly mentioned in this Code, the most important ones of which the designer should take note are marked by a circle.

10.3. CONCRETE DIMENSIONS

10.3.1. Support widths

In clauses 6.4.2.3 and 14.4.2.1 minimum lengths are foreseen for steel bars beyond the centre-line of supports as well as for bearing of floors made of hollow-core units.

For concrete elements (slabs and beams) supported by concrete walls and columns or by masonry, minimum support widths should be foreseen, in order to ensure safe and reliable transfer of forces and to prevent anchorage failure of the main longitudinal reinforcement.

10.3.2. Span to depth ratios

If the deflections of slabs or beams are not numerically checked, the height of the flexural element should follow the minimal rule

$$d \geqslant l/\lambda$$

where λ is taken from Table 7.5.2 and eq. (7.5–5) of clause 7.5.2.3.

This rule intends to ensure low damageability of non-structural elements.

10.3.3. Slenderness

The dimensions of the cross-section of vertical structural elements may be influenced by slenderness considerations if second order effects are to be disregarded. The corresponding limit value of slenderness is given in subsection 5.4.2. See also clause 6.6.3.1.

10.3.4. Minimum dimensions

It is assumed that the minimum thickness of concrete in any part of a linear element is

- 80 mm for reinforced concrete and prestressed concrete (subsection 6.3.1)
- 50 mm for precast concrete (clause 14.2.5.3).

Minimal geometrical conditions are given

- in clause 14.4.2.2 for composite floors with precast slabs
- in clause 14.4.2.3 for composite floors with ribs and blocks
- in clause 14.2.5.6 for precast wall panels
- in clause 14.7.1.4 for joints in precast/segmental structures.

In addition the minimum thickness of beams cannot be less than five times the maximum size of aggregates or five times the diameter of sheathing or tendons.

However, use of such extremely small thicknesses should be made only under special conditions securing against unforeseen loading or environmental conditions.

10.3.4.1. Stress limitations

Excessive compressive stress in the concrete under service load may lead to longitudinal cracks in the concrete or higher levels of creep. If the proper functioning of a member is likely to be adversely affected by these, measures should be taken to limit the stresses to an appropriate level.

(*a*) If the stress does not exceed $0.6 f_{ck}(t)$

(i) under the rare combination, longitudinal cracking is unlikely to occur

(ii) under the quasi-permanent combination, creep and the corresponding prestress losses can be correctly predicted.

(*b*) If under the quasi-permanent combination the stress exceeds $0.4 f_{ck}(t)$, the non-linear model shall be used for the assessment of creep (see clause 2.1.6.4.3d).

For further details see section 7.3.2 and clause 1.6.6.2.

(*c*) Durability requirements for prestressed concrete may impose other limits on stresses in the concrete, e.g. that the section should remain in compression (see clause 7.3.1.1).
In addition, creep effects may also be checked at transfer zones in prestressed beams.

Where fire resistance is needed, other limits may apply.

10.4. CONCRETE COVER

Cover minimal values may be required for several reasons, e.g. as a measure to secure

- steel-to-concrete bond resistance
- protection of steel against external corrosive agents
- ease of concrete pouring.

Similarly, in the absence of additional skin reinforcement, maximal values may be needed in order to limit surface cracking and spalling.

Depending on the requirement under consideration, the appropriate cover values should be retained.

(*a*) To ensure that bond forces are safely transmitted and to prevent spalling of the concrete, the maximum cover of any bar, tendon or sheathing of diameter ϕ should be at least equal to ϕ (section 9.1).
In case of bundled bars, the equivalent diameter ϕ_n is taken into account in evaluating the minimum cover. However, the cover provided should be measured from the actual outside contour of the bundle (clause 9.1.5.3).
When anchoring is made by means of bends, hoops or loops, it is recommended that in the anchorage area, the thickness of the cover would be equal to 3ϕ (see also clause 9.1.1.2b).

The equivalent diameter ϕ_n (< 55 mm) is calculated according to clause 9.1.5.2.

This rule is also applicable in the case of beam top bars bent inside the column in a beam-column connection.

(*b*) To ensure good durability (section 8.4), the minimum distance between any concrete surface and the nearest reinforcement bar or the sheathing for such tendons, shall be obtained from Table 8.4.1.

Table 10.4.1. Minimum cover, c_{min}

Exposure class	c_{min} (mm)
1	10
2	25
3, 4	40
5	*

*Depends on the individual type of environment encountered.

The values are absolute minimum values with no downward tolerances allowed and no upward tolerance being specified.

For precast elements, nominal values of concrete cover may be reduced by 5 mm with respect to the values specified in Table 8.4.1, if accurate control on bar position and on concrete compaction is carried out.

(*c*) To prevent cracking and spalling of thick concrete cover, in cases enhanced durability is desired (subsection 8.4.6) or high strength/high bond and large diameter bars are used ($\phi > 32$ mm, subsection 9.1.4), special skin reinforcement shall be provided if $c_{nom} \geqslant 70$ mm.

10.5. STEEL CROSS-SECTIONS AND ARRANGEMENTS

In Tables 10.5.1 to 10.5.5 limit provisions mainly for reinforcing steel are summarized separately for each category of building elements.

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 = 10 mm with normal quality control
tolerance = 5 mm with intensified quality control.

Reduction of concrete cover is related to better compaction in precast units than those specified in Table 8.4.1.

Where fire resistance is needed, other limits may apply.

However, depending on local conditions, lower limit values of cover thickness may lead to the need of additional skin reinforcement.

The arrangement of information in these tables makes clear that several requirements do not always lead to a limit measure. However, for some of those 'empty' cases, several National Codes do foresee respective rules.

*Table 10.5.1. Limit measures concerning concrete dimensions/covers**

	Validity of models/ calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Concrete dimensions							
Minimum support widths	•	—	•	—	—	—	—
Max. l/d (for buildings)	—	—	—	$\lambda_o k_T k_t \left(\frac{400}{f_{yk}}\right)$	—	—	7.5.2.3
Columns max. λ	6.6.3.1	—	—	—	—	—	5.4.2 6.6.1.3
Minimum dimensions: beams	RC, PC: 80 mm Precast: 50 mm	—	—	—	—	$5d_{aggr}$, 5ϕ	6.3.1 14.2.5.3
slabs	•	—	•	—	•	—	—
columns	•	—	•	—	—	•	—
walls	—	—	•	—	—	•	—
Compressive stress limit	$0.4f_{ck}$ creep	—	—	—	$0.6f_{ck}$ longitudinal cracks	—	7.3 1.6.6.2
Cover							
Minimum	ϕ bond	—	—	—	10–(40) mm	$2d_{aggr}$	9.1 8.4
Maximum	—	—	—	—	For $c_{NOM} \geqslant 70$ mm skin reinforcement	—	8.4.6

*Among the rules not explicitly mentioned in this Code, the most important ones of which the designer should take note are marked by a circle.

*Table 10.5.2. Slabs**

	Validity of models calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Shear reinforcement (if needed)							
Minimum mechanical ratio ($\omega_{sw} = A_{sw}f_{yk}/(b_w s f_{ctm})$)	$\geqslant 0.20$	—	—	—	—	—	6.4.2.4
Spacing	$< 0.75d\ (l + \cot\alpha)$	—	—	—	—	—	6.4.2.4
Inclination to the horizontal							
stirrups	$\alpha \geqslant 45°$	—	—	—	—	—	6.4.2.4
bent-up bars	$\alpha \geqslant 30°$	—	—	—	—	—	6.4.2.4
Longitudinal reinforcement							
Maximum bar diameter related to the steel stress (under service loads)	—	—	—	See Table 7.4.3	See Table 7.4.3	—	—
Maximum bar spacing (in a cross-section) related to steel stress (under service loads)	—	—	—	See Table 7.4.4	See Table 7.4.4	—	—
Minimum reinforcement area within the tensioned zone	$A_{s,\min} = k_c k f_{ctmax} A_{ct}/\sigma_{s2}$ (see eq. (7.4–16))	—	—	—	—	—	—
Tensile stresses in steel (under the rare combination)	$< 0.8 f_{yk}$ $< f_{yk}$ (when stresses are due only to imposed deformation)	—	—	—	—	—	7.3.3
Clear distance in the horizontal or vertical direction	—	—	—	—	—	$\geqslant$ largest bar dia. $\geqslant 20$ mm	—
Main flexural reinforcement							
Minimum bar diameter (mm)	—	•	—	—	•	—	—
Maximum bar spacing (mm)	—	Min (1.2h, 350 mm)	—	—	—	—	9.2.1.1.1, lhs
Secondary flexural reinforcement							
Ratio of secondary to main reinforcement:							
distributed loads	$\geqslant 0.20$	—	—	—	—	—	9.2.1.1.1
concentrated loads	$\geqslant 0.33$	—	—	—	—	—	9.2.1.1.1
Maximum bar spacing (mm)	Min (2h, 350 mm)	—	—	—	—	—	9.2.1.1.1, lhs

* Among the rules not explicitly mentioned in this Code, the most important ones of which the designer should take note are marked by a circle.

*Table 10.5.3. Beams**

	Validity of calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Longitudinal reinforcement (bending)							
Minimum bar diameter (mm)	•	—	•	—	•	—	—
Minimum number of bars (or tendons)	•	—	•	—	—	—	—
Minimum percentage of reinforcement: $\rho_s = A_s/(b_w d)$							
S400, S500	—	0.0015	—	—	—	—	9.2.2.1, lhs
S220	—	0.0025	—	—	—	—	
Maximum percentage of reinforcement	—	0.040 (outside zones of splices)	—	—	—	—	9.2.2.1
Maximum bar diameter related to the steel stress (under service loads)	—	—	—	See Table 7.4.3	—	—	—
Minimum reinforcement area within the tensioned zone	—	—	—	See eq. (7.4–16)	—	—	—
Tensile stresses in steel (under the rare combinations of actions)	—	—	—	$< 0.8 f_{yk}$ $< f_{yk}$ (when stresses are due only to imposed deformation)	—	—	7.3.3
Maximum bar spacing (within a cross-section) related to steel stresses	—	—	—	See Table 7.4.4	—	—	—
Clear distance in the horizontal and vertical direction	—	—	—	—	—	⩾ Largest bar dia. ⩾ 20 mm	9.1.3.2 9.1.3.2
Longitudinal reinforcement (torsion)							
Minimum bar diameter (mm)	$S/16$, where S is spacing of stirrups	—	—	—	—	—	6.3.5.2
Minimum reinforcement	One bar at each corner of the stirrup + bars uniformly distributed along the internal perimeter of the stirrup at a spacing < 350 mm	—	—	—	—	—	9.2.2.4
Transverse reinforcement (shear)							
Minimum bar diameter	—	—	—	—	—	—	—
Maximum bar diameter	12 mm (for smooth bar)	—	—	—	—	—	9.2.2.2

Table 10.5.3. cont

	Validity of calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Minimum percentage $\rho_w = A_{sw}/(sb_w \sin\alpha)$	—	See Table 9.2.1					
Minimum mechanical ratio of stirrups $\omega_{sw} = A_{sw}f_{yk}/(b_w s f_{ctm} \sin\alpha)$	$\geqslant 0.20$	—	—	—	—	—	6.4.2.4
Inclination of transverse reinforcement to the axis of the member	$\geqslant 45°$ (stirrups) $\geqslant 30°$ (bent-up bars)	—	—	—	—	—	6.4.2.4
Spacing at stirrups longitudinal	—	—	Depending on value of F_{Scw}/F_{Rcw}	—	—	—	9.2.2.2
transverse	—	—	Min $\{2d/3, 800\,\text{mm}\}$	—	—	—	9.2.2.2
Transverse reinforcement (torsion-closed stirrups)							
Minimum bar diameter (mm)	—	—	—	—	—	—	9.2.2.4
Minimum percentage of reinforcement	—	—	—	—	—	—	9.2.2.4
Maximum spacing longitudinal	Min $\{0.5b, 0.5d\} < u_{ef}/8$, where u_{ef} is the perimeter of the stirrup	—	—	—	—	—	9.2.2.4
transverse	$0.75d$	—	—	—	—	—	6.3.5.4

*Among the rules not explicitly mentioned in this Code, the most important ones of which the designer should take note are marked by a cross.

Table 10.5.4. Columns

	Validity of models/ calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Longitudinal reinforcement							
Minimum number of bars:							
rectangular columns	—	—	4	—	—	—	9.2.3.1
circular columns	—	—	6	—	—	—	9.2.3.1
Minimum bar diameter (mm)	12	—	—	—	—	—	9.2.3.1, lhs
Minimum percentage of reinforcement	—	—	0.008	—	—	—	9.2.3.1
Minimum area of reinforcement	$A_s = 0.15 N_{Sd}/f_{yd}$	—	—	—	—	—	6.6.2.4
Maximum percentage of reinforcement (in zones of splices)	—	0.08	—	—	—	—	9.2.3.1
Transverse reinforcement							
Minimum bar diameter (mm)	$\geqslant$ max. $\{\phi_l/4, 5\,\text{mm}\}$, where ϕ_l = diameter of longitudinal bars	—	—	—	—	—	9.2.3.2
Maximum bar diameter (mm)	—	—	—	—	—	—	9.2.3.2
Maximum spacing of stirrups (mm)	—	—	$12\phi_{l,\min} b$, 300 mm, where b is smallest dimension of concrete section	—	—	—	9.2.3.2
Minimum spacing (mm)	—	—	—	—	—	—	9.2.3.2
Maximum distance of longitudinal bars not held by a stirrup	—	—	—	150 mm in plane (buckling)	—	—	9.2.3.2

Table 10.5.5. Walls

	Validity of models/ calculations	Ductility requirements	Reliability requirements	Functional demands	Durability requirements	Execution requirements	Reference
Vertical reinforcement							
Minimum bar diameter	—	—	—	—	—	—	9.2.4.1
Minimum percentage of reinforcement ρ_v	—	—	0.004	—	—	—	9.2.4.1
Maximum percentage	—	0.04	—	—	—	—	9.2.4.1
Maximum spacing of bars	Min {$2t$, 300 mm} where t is wall thickness	—	—	—	—	—	9.2.4.1
Horizontal reinforcement							
Minimum bar diameter	—	—	—	—	—	—	9.2.4.2
Minimum percentage of reinforcement	—	—	$\geqslant 0.30\rho_v$	—	—	—	9.2.4.2
Maximum percentage	—	—	—	—	—	—	9.2.4.2
Maximum spacing of bars	$\not>$ 300 mm	—	—	—	—	—	9.2.4.2
Transverse reinforcement (for ρ_v > 0.02)	—	—	—	—	—	—	9.2.4.3

PART III. CONSTRUCTION AND MAINTENANCE

11. PRACTICAL CONSTRUCTION

11.1. GENERAL

This section is essentially drafted from the designer's viewpoint to ensure that the assumptions behind the design requirements are not invalidated in construction. It includes a series of minimum specification requirements for the standard of workmanship.

In essence the designer specifies the requirements for materials and execution for the type of structure to be built. Such requirements are to be given on drawings and in specifications. It is the task of the contractor to check the given instructions, to implement them, and to ensure in his work the required level of quality.

The content is not extensive enough to be used as a contract document. More information for carrying out construction (including safety on site) should be taken from manuals such as the FIP Guide to Good Practice —Basic reinforced concrete and prestressed concrete construction, FIP London, 1978.

Durability and service life aspects are treated in section 8.1.

In the implementation of the material in this chapter, an appropriate quality assurance plan as in chapter 12 should be adopted.

11.2 SITE

11.2.1. General

Responsibilities should be clearly defined. Decision and information charts should be established at the beginning of the work.

Referring to concrete construction, the following fields of activities may appear

- representative of the owner
- project manager, overall responsible for design and execution
- resident engineer, representing the design manager
- site manager, who is responsible for all activities on the site
- contractor, who is responsible for the execution of the work in general
- sub-contractor for specific work, e.g. related to prestressing
- supplier of materials
- co-ordinator for design and construction in a particular field, e.g. falsework and centring.

In ordinary construction where standard practice is ensured, these charts may not be needed.

It should be noted that the list of activities is an example; in some countries other definitions may be specified.

These activities may be assumed by different persons (in case of large or complicated structures); some of them may be assumed by the same person depending on the size of the work and accepted standard practice.

11.2.2. Project management

The following tasks may pertain to the project management

- co-ordination of design, construction and maintenance; definition of responsibilities; official filing

- overall construction programme including commencement of works, excavation, falsework, erection sequences
- communication between design and construction, including inspections and feedback, in particular
  - — to control completeness of designer's documents for construction as drawings, material characteristics, reinforcement lists, specifications
  - — to inform designer about construction documents elaborated from site engineers, about actual soil conditions and site records.
- Relevant Quality Management Systems and Quality Assurance should be considered.

11.2.3. Site management

The following tasks may pertain to the site management

- check of basic design assumptions
- organization of control procedures on site
- minutes of co-ordination meetings
- records of site operations
- as-built documentation.

Examples of design assumptions to be verified

- soil conditions
- temporary excavation
- centring design.

11.2.4. Preparation work for site

The check list of the preparation of the site should include

- existing installations
- excavation
- site equipment
- erection procedures
- working places and their protection
- safety on site.

For detailed information refer to: Bygghälsan Report 'Survey of Working Environment in Concrete Construction', FIP Stockholm, 1982; FIP Guide to Good Practice 'Prestressed concrete—Safety precautions in post-tensioning', Thomas Telford, London, 1989.

11.2.5. Inspections

The inspection should cover materials, records, workmanship and construction.

Tests concerning reinforcement, constituent materials and production of concrete should be defined.

Modifications of the structural design and the specifications should not be introduced except with the approval of the designer or of the supervisory authority.

11.3. FORMWORK, FALSEWORK AND CENTRING

For major projects a co-ordinator for design and construction of falsework and centring should be nominated.

For detailed information refer to CIB/FIP/CEB Manual on formwork technology 'Coffrage', CIB Rotterdam, 1985.

For bridge construction, the following check list is recommended: Traggerüste, Check-Liste, Ingenieurbüro Krebs und Kiefer, Darmstadt, 2nd edition 1982.

11.3.1. Basic requirements

(*a*) Give shape to concrete structure within the prescribed tolerances (see Chapters 1 and 4).
(*b*) Give desired surface texture.
(*c*) Allow thorough placing of reinforcement, tendons and concrete (and its compaction).
(*d*) Allow space for prestressing jacks.
(*e*) Resist actions during construction, including those due to prestressing (equipment, deformations during tensioning).
(*f*) Allow striking without damage (see section 11.9).

11.3.2. Design

(*a*) *Principles*. Design of falsework and centring to be based on codes with a safety concept equal or comparable to the present Model Code for Concrete Structures.
(*b*) *Actions*

(i) Vertically: permanent and variable loads during construction; wind (uplift); support reactions due to prestressing, especially in construction stages (e.g. displacement of dead load effects during tensioning of tendons).
(ii) Horizontally: pressure of fresh concrete (depending on method of placement), effect of included formwork, wind (bracing), swing of crane load (accidentally), local deviation force in curved pipes for concrete pumping.
(iii) Dynamic action: impacts (e.g. due to waves), vibration (compaction of concrete).
(iv) Effects of ground settlements.
(v) Thermal effects (concrete hardening, heat treatment).

(*c*) *Choice of structure*. Should be rather stiff to reduce deformations during erection; should allow unrestricted deformation during tensioning of tendons. Special attention to be given to proper (ground) supports and joints (e.g. mortar bed).

Checks can be made by calculation, testing or accepted standard practice.

Use of standardized material according to manufacturer's instructions and approval documents.

(*d*) *Structural calculation*. Verify resistance (load-bearing capacity) and deformations (sufficiently stiff to ensure that limits of tolerances for concrete structures are satisfied) with special attention to the compatibility with concrete operations (concreting programme) and to the interaction between concrete and falsework during tensioning of tendons (if necessary, remove part of falsework, e.g. to avoid 'springing-up' due to resilient effect of falsework). Camber according to design calculations of the concrete structure and designer's drawings.

(*e*) *Tightening of formwork*. Prevent harmful loss of material during concreting (proper joints, if necessary, to be sealed).

(*f*) *Formwork remaining in concrete structure*. Check durability and compatibility with concrete; if it is acting as a structural element, ensure stability and correct anchorage.

(*g*) *Formwork spacers left in concrete structure*. Should not impair durability and appearance.

(*h*) *Accelerating curing*. Ensure uniform distribution of temperature through the mass of concrete.

(*i*) *Cooling equipment for concrete in structures with large dimensions*

(*j*) *Striking* (see section 11.9). Formwork, falsework and centring to be so designed that they can be correctly struck without damaging the concrete structure (if necesary, provide devices for lowering the centring).

11.3.3. Erection

(*a*) *Principles*. Ensure compliance with prepared drawings and specifications (including surface finishes). Not to be removed until sufficient strength of concrete (see section 11.9).

(*b*) *Supports and structural joints*. Ensure correct positioning and force transmission.

(*c*) *Assembly of formwork panels*. Joints should be mortar-tight.

(*d*) *Preparing formwork faces*. Surfaces in contact with concrete should be clean (all rubbish to be removed). Approved mould-release agents to be applied in a thin uniform layer without contaminating the reinforcement; on vertical or sloping surfaces the material should have sufficient adherence. Concrete to be placed soon after application of the release agent.

(*e*) *Temporary works inserts*. Temporary works inserts may be necessary to assist in keeping formwork, reinforcement, prestressing ducts and other similar items in place until the concrete has hardened. Such inserts should not introduce unacceptable loading into the structure, should not react harmfully with the constituents of the concrete, the reinforcing or prestressing steel and should not produce unacceptable features.

(*f*) *Checks before and during concreting*. See section 11.6 and chapter 12.

11.3.4. Re-use of material

Proper maintenance in order to be equivalent to new products; if used successively, the reduced bearing capacity should be taken into account.

11.4. REINFORCEMENT

For detailed information refer to: CEB Manual on Concrete Reinforcement Technology, Georgi, St-Saphorin (CH), 1983; CEB Bulletin 164 'Industrialization of Reinforcement in Reinforced Concrete Structures', CEB Lausanne, 1985.

11.4.1. Transportation and storage

Steel bar reinforcement, welded mesh reinforcement and prefabricated reinforcement cages should be so transported and stored, that they do not suffer any damage (mechanical or corrosion).

The surface condition of the reinforcement should be examined prior to use to ensure that it is free from mechanical damage (e.g. notches) and from surface deposits damaging bond properties. Overall reduction of the section through corrosion may be tolerated within certain permissible limit values; pitting is not allowed.

11.4.2. Identification

If means of identification has been lost, acceptance tests on samples should be required.

11.4.3. Cutting and bending

Reinforcing steel should be cut and bent in accordance with accepted standard practice.

Bending should be carried out by mechanical methods, at constant speed without jerking, with the aid of mandrels, so that the bent part has a constant curvature.

11.4.4. Welding

Welding shall not be carried out on reinforcing steel unless the steel is suitable for welding (see clause 2.2.5.3).

In general, reinforcing bars should not be welded at or near bends of the bars.

Where a risk of fatigue exists, the welding of reinforcement should conform to special requirements.

The production and check of the welded connections should comply with the relevant requirements.

11.4.5. Joints

The length and position of lapped joints should be in accordance with the design and the drawings.

Joints made with mechanical connecting devices should be carried out in accordance with the specified standards or approval documents.

11.4.6. Assembly

The assembly of the reinforcement should be robust enough to ensure that the bars do not shift from their prescribed position in the course of transport, placing and concreting.

11.4.7. Placing

The specified minimum cover to the reinforcement should be maintained by the use of approved chairs and spacers.

The reinforcement should be secured against any displacement and the position of the reinforcement should be checked before concreting. Tolerance limits are given in clause 1.4.5.2.

In areas of congested reinforcement sufficient spacing should allow for insertion of needle vibrators.

This point should already be implemented during design and detailing of the reinforcement.

11.5. TENDONS

11.5.1. Prestressing steel (transportation and storage)

The following items should be observed

- use of clean containers; suitable protection against environmental influences in case of water transport

- protection (during transport to the site and while on site) against mechanical damage, detrimental corrosion and deposits damaging the bond properties
- storage under solid cover (against rain, ground, aggressive atmosphere) and with ventilation
- no welding close to stored steel without protection from splashes
- periodic examination on site
- very thin film of rust may be accepted if it can be removed by a dry cloth.

Refer also to the forthcoming FIP report on 'Corrosion protection of prestressing steel (pretensioning, post-tensioning and stay cables)'.

The detrimental influence of rust on the friction behaviour should be taken into account.

11.5.2. Sheathing (ducting)

In addition to subsection 11.5.1, the following should be considered

— check aspect (no local damage), no corrosion inside, use of end caps
— leak-proof, water tightness
— sheathing should resist mechanical and chemical attack
— empty ducts (steel inserted later): immediately after concreting, the free passage in the sheathing should be checked, e.g. by pulling-through a conical gauge (dolly).

Empty ducts should be sufficiently rigid or should be stiffened during construction by a temporary mandrel.

11.5.3. Anchorages, couplers

The requirements of the approval documents should be checked.

11.5.4. Fabrication of tendons

(*a*) *General*. Tendons can be fabricated on-site or off-site in permanent installations. In both cases the procedures shall ensure that the final product meets the requirements.

(*b*) *Identification*. Maintain identification of all materials. If identification cannot be established, acceptance tests on samples have to be performed.

(*c*) *Cutting*. Normally by shear, saw or abrasive disc. If by oxy-acetylene flame, excess length will be necessary (use sufficient oxygen to ensure cutting, rather than melting).

(*d*) *Bending of bars*. Not allowed in vicinity of anchorages or couplers. Reverse bending not allowed.

(*e*) *Assembly*

(i) Materials to be checked before use, especially after long storage period (identification, not wet, etc.).

(ii) Fabrication area protected (clear of ground and under cover, the latter if required on account of environmental conditions). Prestressing steel for a tendon, from the same delivery. Anchorage plate perpendicular to tendon axis.

(iii) Sheathing to be jointed by couplers which are taped at both ends for watertightness. Joints in adjacent sheathings to be staggered at least 300 mm apart.

(iv) Provide vents (with possibility to be closed) at both ends (threaded for connection to pump), peak points and all points where air or water may accumulate, especially in long tendons (maximum interval 40 m), vertical tendons and significantly inclined tendons.

In case of large duct diameters ($\geqslant 80$ mm), near to the peak points, additional holes for later secondary injection should be provided.

(*f*) *Transportation.* To be transported and handled with care. Not to be dragged across the ground. When lifting by crane, avoid local crushing and restrict bending.

(*g*) *Placing* (tolerance limits: see section 4.1)

(i) Careful inspection of sheathings before and after placing, and particularly at construction joints.

(ii) Accurate fixing according to designer's specifications (drawings) of dimensions (and tolerances), spacers and supports; prevent empty ducts from floating; ease of casting concrete.

(iii) Secure position of sheathings containing prestressing steel, the use of welding is not allowed for corrugated sheathings; it can be authorized, when sheathing consists of steel tubing. During placing of tendons in the formwork the minimum radii of prestressing steel and sheathing should be respected. All vents and other openings of the tendons shall be closed and marked immediately after placing of the tendon.

(iv) Pull-in or push-in of strands and wires should be made with proper equipment by using procedures which are not harmful to the prestressing steel and the ducts.

(v) Prevent flow of grout between crossing ducts (if insufficient concrete between ducts) by metal strips or other barriers, especially when high grouting pressure is used.

11.5.5. Temporary protection of tendons

The period between assembly and grouting should be kept to a minimum: in a non-corrosive environment maximum 12 weeks, but not more than 4 weeks inside the formwork (unstressed) and 2 weeks after tensioning.

In dry environments, the given periods may be extended.

Use of products only with strict respect of producer's instructions for the application. When water-soluble products are used as temporary protection, the bond properties should not be affected harmfully.

Otherwise corrosion protection should be provided, e.g. by sealing, blowing out the ducts with pre-dried air, use of vapour phase inhibitors or emulsifiable oils.

Suitable protection for threaded ends of bars should be provided.

Protective wrapping should be chemically neutral.

FIP Recommendations 'Design of flat slabs in post-tensioned concrete', May 1980 and 'Corrosion protection of unbonded tendons', Thomas Telford, London 1986.

11.5.6. Unbonded tendons

The use of unbonded tendons in practical construction generally does not differ from practice with bonded tendons. Therefore the relevant sections of this chapter remain valid. It is, however, essential that sufficient care is taken during execution so as not to damage the protection system of such tendons.

FIP Recommendations 'External prestressing' (under preparation).

11.5.7. External tendons

The use of external tendons in practical construction generally does not differ from practice with bonded tendons inside the concrete. Therefore the relevant sections of this chapter remain valid.

11.6. CONCRETE

11.6.1. General

Refer to Appendix d on concrete technology.

11.6.2. Measures to be taken before concreting

Measures to be taken in case of unforeseen stops in concreting or freezing etc. should be duly planned.

The concreting dates shall be fixed well in advance to allow control of geometry, block-outs, inserts, reinforcement and tendons (checking of space for needle vibrator).

Shortly before casting, uncoated timber formwork shall be watered and the position of the void formers checked. Empty ducts should be secured against uplift.

11.6.3. Concreting programme

The following items should be considered in the concreting programme

- concrete pour areas, timetable (sequence of pours)

Examples: roughing measures, use of retarders.

Manuals for extreme weather conditions:

FIP Guide to Good Practice—Concrete construction in hot weather, Thomas Telford, London, 1986; STUVO/FIP Report 'Concrete in hot countries', STUVO, 's-Hertogenbosch (NL), 1985; RILEM Recommendations for winter concreting, RILEM Bulletin, Paris, 1963.

- construction joints: location, surface
- particular types of surface finish.

11.6.4. Measures to be taken after concreting

The fresh concrete should be protected against premature drying out.

11.7. TENSIONING OF TENDONS

11.7.1. General

To secure a correct and safe tensioning of tendons it is necessary to provide

- skilled personnel, trained for the purpose
- solid and safe equipment (regular calibration)
- written instructions and pre-arranged prestressing programme
- suitable safety precautions during prestressing.

11.7.2. Instructions to the site

(a) Basic information
- (i) post-tensioning system, tendon units and prestressing equipment; anchorages and couplers used
- (ii) minimum concrete strengths prior to stage and final tensioning
- (iii) tensioning from one or both ends (at the same time or at one end after the other); order of stressing successive tendons
- (iv) sequences of successive stages of stressing tendons and striking falsework, if such stages are envisaged
- (v) expected values for pressure in the jack, force to be developed at the jack, elongation of steel, draw-in at non-jacking end and slip of steel during anchoring
- (vi) any tests to be performed (e.g. friction).

(b) Detailed programme
- (i) increments in the stressing force of each tendon, including influence of striking falsework
- (ii) corresponding steel elongations, including effects of immediate and time-dependent losses

(iii) permissible deviations from expected elongations should be indicated
(iv) corrective measures to be taken, if the above deviations are not respected.

Suggested values are: 15% for a particular tendon, but not more than 5% for the sum of all values of tendons in the same section.

(c) Methods of measurement

(i) pressure
(ii) force
(iii) elongation
(iv) draw-in or slip.

Refer to the FIP State of the Art Report: 'Tensioning of tendons: force–elongation relationship', Thomas Telford, London, 1986.

(d) Records

(i) observations to be recorded during tensioning
(ii) site stressing records to be transmitted to and signed by the design engineer in view of any necessary adaptation of above instructions.

11.7.3. Tensioning operations

(a) Before stressing

(i) confirm that above instructions can be met
(ii) check concrete strength
(iii) check that structure is free to move
(iv) check that vent holes are not blocked
(v) inspect exposed tendon ends and surfaces on which anchorages and stressing equipment will bear
(vi) calibrate prestressing devices.

(b) During stressing

(i) stresses should increase at a gradual and steady rate; for long elongation: temporary anchoring and regripping
(ii) tensioning log with all data and anomalies observed
(iii) full records of pressures at jack or load cell and of steel elongations corresponding to each increment of the tension force, including temporary overstress, release, draw-in or slip at the ends
(iv) compare measured and calculated values
(v) observe instructions, if deviation is greater than permitted
(vi) maintain tendons to allow eventual restressing; cut-off and grouting only after final approval.

(c) After stressing

(i) visual inspection of concrete and anchorages
(ii) supervision of possible temporary protection of tendons
(iii) for trimming surplus tendon material, apply clause 11.5.4(c) on cutting of tendons.

Where surplus tendon material is burned off, the anchorage devices should be protected from excessive heat development.

11.7.4. Temporary protection after tensioning

A temporary protection may be necessary, if grouting takes place more than two weeks after tensioning, unless it is demonstrated that other measures effectively prevent corrosion.

Protective material shall allow later sufficient bond and shall have no deteriorating effect on the grout. Only approved products should be used.

Refer also to the forthcoming FIP report on 'Corrosion protection of prestressing steel (pretensioning, post-tensioning and stay cables)'.

Use of products only with strict respect of producer's instruction for the application.

11.8. GROUTING OF TENDONS

11.8.1. General

(a) The main objectives are

(i) to prevent corrosion of the prestressing steel
(ii) to provide an efficient bond between the prestressing steel and the concrete.

For detailed information refer to the FIP Guide to Good Practice 'Grouting of tendons in prestressed concrete', Thomas Telford, London, 1990.

(b) Basic requirements

(i) all voids in ducts and anchorages should be filled with a suitable grouting material (usually cement grout)
(ii) the above objectives are met by using approved grout materials (remain alkaline, no harmful components) and by filling the ducts and anchorages completely, including voids between tendons, with a grout which after hardening fulfils structural requirements (strength, bond)

(c) subsection 11.7.1 (General on tensioning) applies for grouting as well.

11.8.2. Cement grout

(a) Properties

(i) adequate fluidity and cohesion when plastic
(ii) low bleed, and shrinkage compensated when hardening
(iii) adequate strength and resistance to freezing when hard.

(b) Materials

(i) ordinary Portland cement or, in particular cases, other cements
(ii) water
(iii) admixtures: plasticizing agents, viscosity modifying agents, gas generating admixtures, retarders (for long parts of ducts to be grouted)
(iv) chlorides from all sources should not exceed 0.1% by mass of cement.

Other cements than Portland should be used only if their suitability has been verified by tests.
The use of silica fume or similar material is in development, but its benefits have not yet been confirmed.

(c) Mixing

(i) all materials to be batched by mass
(ii) water–cement ratio should not exceed 0.40
(iii) admixtures according to manufacturer's instructions
(iv) general procedure: firstly water in mixer, add cement, after thorough mixing (to get homogeneous colloid) add other materials
(v) time of mixing according to manufacturer's instruction: not more than four minutes; after mixing grout should be kept in continuous agitation, until pumping into the duct (within 30 minutes of mixing, unless retarder is used); temperature of fresh grout should not exceed 35°C.

Lower values than 0.40 are recommended (refer to the above FIP Guide) but in special cases, higher values, up to 0.45, may be used.

11.8.3. Instructions to the site

(a) Preconditions

(i) equipment operational (including 'standby' grout pump to avoid interruptions in case of malfunction)
(ii) permanent supplies of water under pressure and of compressed air
(iii) materials batched (excess to allow for overflow)
(iv) ducts free of harmful material (e.g. water, ice)
(v) vents prepared and identified
(vi) preparation of control tests for grout
(vii) in case of doubt, grouting trial on representative ducts.

(b) Grouting programme

(i) characteristics of grout (including periods available for use, hardening time)
(ii) characteristics of equipment (including pressures, injection rates)

(iii) blowing of air through the ducts, or clearing by methods defined in the approval documents

In hot climate, alternatively, it may be advisable to flush water through the ducts prior to starting the grouting operation. However, the consequences of remaining water on the water–cement ratio should be considered.

(iv) order of injection operations and fresh grout tests (fluidity, segregation)
(v) manufacture of test specimens (bleeding, shrinkage, strength)
(vi) grout volume to be prepared
(vii) instructions in case of incidents (e.g. fault during injection: remove grout from duct and repeat injection) and harmful climatic conditions (e.g. after and during periods with temperature lower than 5°C).

If measuring equipment (temperature gauges) is available, the 5°C value may be referred to the temperature of the structure in the vicinity of the tendon.

(c) Records to be established on all operations, measures, tests and particular events; to be transmitted to the design engineer.

11.8.4. Grouting operations

(a) Before injection, check preconditions and confirm that grouting programme can be fulfilled.

For details refer to the above FIP Guide.

(b) During injection

(i) process to be carried out at a continuous and steady rate (sufficiently fast to prevent segregation at points where flow is restricted; but slow enough in corrugated ducts to prevent entrapping of air in downward stream of grout)

Grouting of a duct should be done without interruption. The speed at which the grout is pumped through the duct can vary from 5 to 15 m/min. The maximum grouted length should not exceed 120 m.

(ii) grouting shall commence at the lowest grouting point (or from the lowest cable end) and continue until the fluidity or density of the grout flowing from the free ends and the vent openings is about the same as that of the injected grout; after closing the last vent, pressure to be held at 0.4 to 1.0 MPa for some minutes
(iii) in some circumstances (large diameter of vertical or inclined ducts, at the highest points of the ducts) post-injection is necessary to replace bleed water by grout
(iv) after completion of grouting, losses of grout from the duct should be prevented
(v) to allow expansion of grout during hardening and to displace bleed water, appropriate vents may be opened.

(c) After injection

(i) all equipment to be thoroughly washed very shortly after grouting, followed by thorough draining of pump, mixer and pipelines

(ii) if large voids are suspected, take precautions for regrouting
(iii) in case of doubt, control with an endoscope or with vacuum.

11.8.5. Sealing

Once the grout has hardened, all openings, grouting tubes and vents have to be hermetically sealed to prevent penetration of water and harmful products (e.g. anti-freeze or de-icing agents).

To obtain a good seal of anchorage recesses, a preparatory treatment to the surface around the anchorages may be applied; in large recesses, connection reinforcement should be provided.

11.8.6. Other protection

Refer also to the forthcoming FIP report on 'Corrosion protection of prestressing steel (pretensioning, post-tensioning and stay cables)'.

Tendons may be protected by materials based on bitumen, epoxy resins, rubber, etc., provided that the effects on bond and fire resistance are not important.

11.9. STRIKING

11.9.1. General

Formwork, falsework and centring should remain undisturbed until concrete has achieved sufficient strength to withstand the stresses and deformations to which it will be subject (with an acceptable margin of safety).

Special consideration should be given to

(a) the weight of the concrete (especially if this is the major part of the full design load)
(b) imposed loads (e.g. from falsework placed for higher elements, before their hardening)
(c) sequence of striking, eventual temporary jacking and temporary support
(d) the need of certain elements being retained for reducing time-dependent deformations (e.g. auxiliary props) or for the stability of the whole structure (e.g. wind-bracing)
(e) prestressing and grouting operations (see sections 11.7 and 11.8)
(f) particular striking operations (e.g. at re-entrant angles of formwork)
(g) environmental conditions (e.g. freezing) and measures available to protect the concrete once the formwork is removed
(h) subsequent surface treatment requirements.

Striking operations to be executed without shock (e.g. by sudden removal of wedges) and by using only static forces.

Supporting elements (columns, walls) should be struck first.

Lowering of centring by respecting the structural system and the prestressing programme.

11.9.2. Minimum periods before striking

The minimum periods before striking depend upon several influences such as strength development, curing, deformation behaviour or dead load ratio.

For prestressed concrete structures striking may be done after tensioning of tendons, which is related to a required minimum strength of concrete.

In the absence of other information the following periods are recommended for reinforced concrete structures, if normal hardening cement is used.

Table 11.9.1. Recommended minimum periods before striking

	Surface temperature of concrete			
	⩾24°	16°	8°	2°
Vertical formwork	9 h	12 h	18 h	30 h
Slabs				
soffit formwork	3 days	4 days	6 days	10 days
props	7 days	10 days	15 days	25 days
Beams				
soffit formwork	7 days	10 days	15 days	25 days
props	10 days	14 days	21 days	36 days

The above values for vertical formwork demand that striking is immediately followed by appropriate curing or protection from low or high temperatures.

If, during the hardening of concrete, freezing periods occur, the above values are to be increased, at least by the duration of these periods; for winter concreting in general, refer to the RILEM Recommendations for winter concreting, RILEM Bulletin, Paris 1963.

The values in Table 11.9.1 may have to be increased, if special consideration is given to the limiting of early cracking (especially in elements with different thicknesses or temperatures), or to the reduction of creep deformations.

The values in Table 11.9.1 may be reduced if accelerated curing methods or special formwork (e.g. sliding forms) are used.

12. QUALITY ASSURANCE AND QUALITY CONTROL

Due to practical and historical reasons, the terminology used in this chapter is not in full accordance with the terms defined in ISO 8402—'Quality—Vocabulary' from 1986.

For example, in ISO 8402

- quality assurance plan is denoted quality plan
- quality control means the operational techniques and activities used to fulfil requirements for quality
- inspection is the term used for activities such as measuring and testing.

12.1. QUALITY ASSURANCE

12.1.1. Quality assurance requirements

The subsequent description covers the phases detailed design, production and execution. The preliminary phases (design brief, basic engineering) and the operation and maintenance phases are not covered.

In this chapter only the requirements related to a specific project are specified.

Requirements of a company's quality system are not covered. It is anticipated that each company has established and implemented a quality system covering its basic organization.

12.1.2. Quality assurance plan

For the whole building process, the quality assurance plan (as well as the set of individual quality assurance plans) should at least include, with the appropriate degree of detail, the following elements as suitable for the tasks to be undertaken.

The content of a quality assurance plan depends on the extent of the quality system in the basic organization. The described quality assurance plan assumes that a full quality system is established and implemented in the basic organization. Deliveries and services of sub-contractors shall be covered with reference, where relevant, to their quality assurance plans.

(*a*) *Organization*. The responsibility, authority and the interrelation of all personnel who manage, perform and verify work affecting quality should be defined. The names and qualifications of the personnel responsible for tasks requiring special skill and experience should be stated.

External organization and key personnel involved should be defined.

Internal and external lines of communication should be defined.

(*b*) *Planning*. It should be described how the taks undertaken will be planned in order to achieve a systematic and orderly performance.

(*c*) *Design control*. Procedures to

(i) identify input requirements
(ii) perform the design
(iii) verify the design
(iv) control design changes

should be established and maintained.

The verification measures may include

- design reviews
- discipline check of individual documents
- alternative calculations
- qualification testing.

(*d*) *Document control.* Establish measures to ensure that all essential quality related documents are reviewed and approved by competent and authorized personnel prior to issue.

The control should ensure that the right documents are at the right places at the right time.

(*e*) *Procurement.* Establish measures to ensure that services procured, products purchased and items subcontracted are in accordance with specified requirements.

(*f*) *Production and construction.* Establish measures to ensure that production and construction activities are performed under controlled conditions.

This may include appropriate controls of materials, equipment, processes and procedures, computer software, personnel and associated supplies, utilities and environments.

The inspection, testing and supervision activities necessary to fulfil the specified requirements should be defined.

All inspection, testing and supervision activities should be documented.

(*g*) *Records.* Performance quality records to be maintained as evidence that the service provided and the items delivered/constructed meet contractual or other applicable technical requirements. These records should be delivered to the client or preserved for an agreed period after completion of the project.

12.2. QUALITY CONTROL

12.2.1. Classification of control procedures

Basically two types of control are distinguished and each of them may be performed at various levels.

Although distinguished, the two kinds of control are normally interconnected.

Other distinctions of control methods can be made (e.g. visual or test-based, statistical or total). Refer to Bulletin 191.

12.2.1.1. Types of control

Types of control are

- *production control*, which relates to production and execution processes and includes preliminary tests where relevant; production control applies to materials, components and production and execution activities at least

- *compliance control*, which relates to the results of production and execution processes; it is applicable to all stages of the building process, i.e. to promoting, design, materials and components, production and execution, and use of the completed structures.

Compliance control is normally concluded by the acceptance of products relating to the corresponding stage.

12.2.1.2. Control levels

According to the body which performs a control or is responsible for it, distinction is made between internal and external control.

Internal control may be carried out either by the producer himself, or by another body acting for the producer.

An internal control made by the producer may be performed by the production team or by a specialized service of the producing firm. In case of certified production, a high degree of independence of the controlling body is of paramount importance (see clause 12.2.2.3).

Internal control relates to a production control and possibly (in the case of certified production) to a compliance control, final acceptance excluded.

External control may be carried out either by the client himself, or by another body acting for the client.

Here a contractor is normally considered as the client with regard to his suppliers and subcontractors.

External control is necessary to conclude to a final acceptance of products. In the case of reliable certified production the external compliance control can be radically reduced (merely up to an identification of the product and checking the certificate).

12.2.2. Control systems

12.2.2.1. General

Control systems are rational combinations of production controls and compliance controls. They shall be in accordance with the following rules and subsections 12.2.3 to 12.2.6.

The choice of an appropriate control system for a particular project depends on several parameters. The total control system shall be consistent with the quality measures defined in the quality assurance plan of the project.

The main parameters are

- technical capacity, availability and degree of independence of the participants
- current practice concerning the quality system of each participant (described in his own Quality Manual)
- intended degree of assurance established by the client (see clause 12.2.2.2)
- pre-existing organizations able to be integrated into the system
- more generally, technological level within the area, past practice and experience.

This variety of conditions and constraints does not make it possible at the present time to define control classes for whole building processes. Degrees of control may, however, be distinguished and even standardized (see clause 12.2.2.2).

12.2.2.2. Variation of degrees of control

Control systems can be widely differentiated. Some common ways to vary the degree of control are now given.

These various ways generally do not provide the same effects.

(*a*) More or less complete and detailed planning of the control system within the quality assurance plan. The first purpose of such planning is to avoid omissions and ambiguities. It may also specify, for example

(i) how each control should be formalized
(ii) control stops where the building process can be subdivided, mainly at points where responsibility is transferred, from one party to another or where one phase of the process gives way to another.

A precise formalization is highly motivating and constitutes by itself a guarantee. See the General Principles on quality assurance for structures, clause 5.2 (published in 1981 as IABSE Report No. 35).

(*b*) More or less specifying the independence of controllers.

Independence is a relative and imprecise property.

(*c*) Selecting sufficiently qualified controllers, with regard to the activity to be controlled and to the control criteria.

(*d*) Combining, if appropriate, several control methods, exerted by different bodies, for the same purpose.

A duplication of the same controls made by two different persons may be demotivating for both. Conversely a good example consists of supplementing controls of execution by controls of the completed structure, such as defined in subsection 12.2.6.

(*e*) Differentiating the intensities of controls; for example, in the case of statistical control, it may consist of differentiating the definition of batches or units, the sampling procedure, the sample size.

This differentiation directly modifies the filtering effect of control, but also indirectly influences the strategy of the producer, because he considers the costs of rejection or correction to optimize the level of quality of his supplies. This indirect effect is commonly the main one. Hence, the main effect of a control is its mere existence.

(*f*) Severity of acceptance criteria.

In the case of statistical control, this severity cannot be measured only by a level of confidence, because it depends also on the reliability (and hence on the consistency) of the prior information upon which the acceptance criteria are based.

(*g*) Severity of actions taken in case of non-compliance.

12.2.2.3. Certification

Certification may be envisaged for design, materials, components and execution.

ISO has defined several types of certification. When these types were established no particular consideration had been given to construction activities. Other definitions and terminologies are in discussion within the EC.

The various types of certification give unequal degrees of reliability. More precisely these differences result from the general possible degrees of differentiation listed in clause 12.2.2.2, and more specifically from

(*a*) who certifies (the producer or another body)
(*b*) who designates a certifying body external to the producer
(*c*) who approves the quality system of the producer (the producer, the certifying body or another body such as e.g. a Commission or a public authority)
(*d*) whether, how and by whom the internal control is supervised
(*e*) whether sufficient sanctions can be expected if it is discovered that certain products do not comply with the certification requirements.

The permanent character of a certification is by itself favourable to its reliability.

A designation by the public authorities gives generally the best independence.

A higher reliability is obtained if the supervising body from time to time samples certified products from the stock and tests them.

12.2.3. Control of planning and design

This control basically consists of

- verifying that all necessary requirements and conditions are satisfied, for the completed structure and for its execution
- checking that the calculation models and methods are appropriate and that the numerical calculations are carried out correctly
- checking that the drawings and descriptions are clearly understandable and that they comply with the design calculations and with the established specifications.

For this checking three general methods are defined in Bulletin 191. They are described and commented in detail in IABSE Surveys S 27-484.

According to the contract, the control may have

- to verify also that the required quality of planning and design is reached
- to be exerted globally or at each of a series of predefined phases.

This quality shall be specified where appropriate. For example various degrees of quality of designs are defined in general terms in the Proceedings of the IABSE Workshop RIGI 1983 and in CEB Bulletin 184, par. 4.1.2.

A control exerted at successive phases is more rigorous than a control exerted globally. In the first case the content of the individual phases should be rationally chosen (see, for example, IABSE 27-84).

12.2.4. Control of materials and structural components

12.2.4.1. General

The materials and components considered in this clause are

- all steels, anchorage devices, ducts
- primary (raw) materials for concrete, mortar, grout, i.e. cement, aggregates, water, admixtures
- concrete (industrially produced, 'ready-mix concrete', or produced on site), components if industrially produced.

Although not explicitly considered in this clause, the other structural materials to be used (e.g. bearing pads, glue, epoxy, products for curing, mortar, grout, materials for formwork and falsework) should be controlled by following the same principles. Their control should, however, be specifically differentiated.

Among these materials and components a distinction is made between those which are covered by an approved Quality System (QS), those that are not and those which are produced on site by a contractor or subcontractor.

For components produced on the site, subsection 12.2.5 is relevant.

12.2.4.2. Materials and components covered by an approved QS

The quality control made by the producer shall be supplemented by a limited series of controls intended to cover the interface with the supplier and the transferring phase.

If relevant, properties not covered by the approval shall be identified and controlled as in clause 12.2.4.3.

(*a*) For all steels the origin and identity of every delivery shall be controlled. This should be made by referring to the documents of certification carried on the delivery, to labels and to rolling marks (where relevant). The shape or straightness and the surface conditions shall also be controlled.

(*b*) For anchorage devices, cement, admixtures and aggregates the origin and identity of every delivery shall be controlled, along with

(i) for anchorage devices the surface conditions
(ii) for cement the temperature and, if relevant, the colour
(iii) for admixtures the limit date of use, if relevant
(iv) for aggregates, if not standardized, the grade (dimensions), cleanness and other measured properties mentioned on the delivery note.

(*c*) For ready-mix concrete, the following should be controlled for every delivery

(i) the origin and identity of the mix (components, notably additive, shall be defined in a delivery note established by the producer)

It is recommended to control the absence (or better the impossibility) of addition of water during transport or at delivery.

(ii) the interval of time between mixing and delivery

Controls of consistence on specimens taken from the delivery are recommended.

(iii) sometimes the temperature (especially during severe climatic conditions).

The differentiation to concrete produced on site (no approved quality system) shall be established in terms of a lower number of tests and acceptance criteria based on more favourable prior information, and not in terms of a lower γ_c-factor.

Strength tests shall be made for compliance control, as defined in subsection 12.2.4, with an appropriate frequency.

The quality system rules applicable to ready-mix concrete are widely applicable for the production of industrialized structural components.

(*d*) For structural components, the following are to be controlled for every delivery

(i) the origin and identity, with special regard to the reinforcement if the factory produces components having the same external dimensions but different reinforcement; this shall be made by reference to the delivery note and to marks
(ii) the indication, on the delivery note, of all required information on the time and conditions of production
(iii) the dimensions and deformations, the absence of spalling or cracks
(iv) the colour, if relevant.

12.2.4.3. Materials and components not covered by an approved QS

A detailed programme of control shall be defined.

For materials to be controlled statistically, some controls at least shall be made at the factory or the production site, in order to ensure that the production is not excessively heterogeneous rendering the programme of control not significant.

In the absence of any reliable prior information, no statistical control can be simultaneously operational and reliable. A total control is possible only if non-destructive test methods are available and practical.

12.2.4.4. Structural concrete produced on the site

For structural concretes, the properties likely to be specified are various. The most common are the size (or maximum size) of aggregates, the consistence and the required characteristic compressive strength $f_{ck\,req}$. For lightweight and heavyweight concretes, the unit mass is generally specified.

There are also various ways to specify these properties. Hence, the corresponding quality controls can be very varied.

For non-structural concretes, which commonly are a non-negligible part of the total production, controls should be alleviated.

For example concretes may be specified as designed mixes or prescribed mixes (see for the definition Appendix d, section d.3).

In the case of production certified in certain precise conditions, tests made on concrete properties may be used for compliance control (see section 12.1).

For concrete not produced on the site, production control may have to cover transport to the site, depending on the delivery conditions.

As measures to be taken may be different for various structural elements it is recommended to avoid that a lot includes elements of different kinds such as beams and columns. For practical reasons, it is recommended to define the lots such that the same workmanship and climatic conditions can be presumed for each lot.

Unless a low degree of control is accepted, the production control should include

- the production equipment (weighing and gauging equipment, mixer and control apparatus)
- the production process
- some concrete properties, first as preliminary tests and then during production.

The term 'same conditions' implies that any important variation in the quality of the constitutive materials (for example due to different deliveries, or different climatic conditions) should be excluded.

For compliance control the total production should be divided into lots which are considered to be produced under the same essential conditions and can be judged separately.

For each lot several batches are sampled, and from each batch one or several specimens are made for tests.

The specimens are made, preserved and tested according to standardized conditions.

For many properties, e.g. consistence, the conformity of the lot under consideration is recognized only if all individual test results comply with the specified values of these properties.

Irrespective of the number of specimens taken from each batch, the test result x_i to be used in the compliance strength criterion will be only one for each batch, equal to the mean value if more than one specimen is taken from the batch.

The first inequality generally is the main one. The second one is useful to check whether an important anomaly has occurred during the production of the lot, and may be dominating for small n.

For concrete strength, the compliance control generally shall be statistical, and the criterion is defined by the two following inequalities

$$\bar{x} \geqslant f_{k\,req} + k_1$$

$$x_{\min} \geqslant f_{k\,req} - k_2$$

where

$\bar{x}$ is the mean value of the n test results $x_1, x_2 \ldots x_n$
$x_{\min}$ is the minimum value of these test results
k_1, k_2 are specified coefficients; for $n \geqslant 6$, k_1 is taken equal to λs_n where

$$s_n = \sqrt{\left[\frac{\sum_i (x_i - \bar{x})^2}{n-1}\right]}$$

No final generally agreed decision has been taken on the limits of n; $n \geqslant 6$ is taken from ENV 206; greater values such as $n = 15$ in MC 78 are hardly compatible with any practical division in lots.

The distribution within a lot may generally be considered to be Gaussian. The prior information on the parameters of this distribution may generally be put together in a hierarchical model (see JCSS reports) which is based

k_1 (or λ) and k_2 may be dependent on n. Their values should be deteron practical experience and is different for different standardized conditions, relating to production and production control.

After the standardized conditions have been defined and the associated prior information has been numerically assessed, combinations of this information with k_1 (or λ) and k_2 values make it possible to draw 'OC lines' giving the probability of acceptance of the lot as a function of the actual ratio of defectives.

mined on the basis of statistical studies taking into account prior information on the type of distribution, on the standard deviation and on the mean value. This information is in every case associated with the observance of standardized conditions.

12.2.5. Control of execution

12.2.5.1. General

The following shall be controlled

- the interfaces with design and supplies of materials and components
- the workmanship.

These activities are not able to be fully standardized. The corresponding controls are either purely qualitative, or, if there are quantitative references (e.g. geometrical tolerances) cannot be based on statistics, so that their conclusions generally are qualitative.

For the same reasons the intensity of these controls can only be differentiated

- either qualitatively (defining those tasks that will be controlled), or
- indirectly by the means devoted to inspection (e.g. permanent presence of a surveyor from the controlling body).

Each individual control activity shall be documented by a record, a note or a report.

For concrete structures the main activities to be controlled relate to

- concrete
- formwork and falsework
- reinforcement
- prestressing steel and devices
- precast units.

All differentiations in quality control shall be defined within the frame of the quality assurance measures. Although widely recommended, establishing a more or less detailed site journal cannot be considered as an alternative to control reports. It gives only an indirect differentiation.

12.2.5.2. Concrete

The main activities to be controlled are

- transport and placing (limit duration and avoid segregation)
- compacting
- curing (ensure a sufficient duration)
- surface finishing.

For production refer to subsection 12.2.4.

Other activities to be controlled, if relevant, are protection with regard to frost and steam curing.

12.2.5.3. Formwork and falsework

If the materials to be used for these are new, they should be controlled according to the principles defined in subsection 12.2.4. If not, the verification, selection and restoring of the delivered materials are activities to be controlled.

The other main activities or qualities to be controlled are

- erection (geometrical and mechanical aspects)
- tightness
- internal surface
- removal.

12.2.5.4. Reinforcement

The main activities to be controlled are

- handling and storage
- cutting and shaping (if not made in a factory before delivery)
- assembly (including laps, joints and welding if relevant)
- positioning.

12.2.5.5. Prestressing steels and devices

The main activities to be controlled are

- handling and storage
- cutting and shaping, if relevant, of ducts and tendons
- positioning
- tensioning (a detailed record shall in any case be established)
- grouting (for the materials to be used and the production of the grout, the control shall be made by following the principles defined in subsection 12.2.4).

12.2.5.6. Precast units

The main activities to be controlled are

- handling and storage
- transport on site
- positioning
- assembly.

12.2.6. Control of the completed structure

12.2.6.1. Control for final acceptance of the structure

This control consists of

- a general supervision of the structure and of the control documents produced during execution
- if required by the contract, controls of some performances of the structure
- unless a low degree of control during the use of the structure is accepted, collecting and ordering all existing documentation likely to be used during the lifetime of the structure for use, maintenance and repair.

This includes mainly the control of measures taken in case of non-compliance.

They are for example control of serviceability performance by loading tests, control of watertightness, control of aesthetic required qualities, etc.
In order not to damage the structure or reduce its life duration, controls by loading tests should be limited, commonly below serviceability limit states.

This documentation shall include, among others, as-built drawings.

12.2.6.2. Organization of later controls

For these controls the following should be established

- a utilization plan for the structure
- a programme for inspection and maintenance, to be carried out where long-term compliance with the basic assumptions for the project is not ensured.

The utilization plan should be established during the design phase, and the programme for inspection and maintenance may be defined by a pre-existing standard. These documents may, however, have to be revised and updated on the basis of the documentation mentioned in clause 12.2.6.1.

13. MAINTENANCE

13.1. GENERAL

Design should be based on available information regarding the foreseen inspection and maintenance policy, so that relevant decisions may be taken on various design parameters; such as construction joints, form and section of less accessible elements of the structure, type and quality of basic building materials, detailing, sensitivity of the structural concept etc.

Due to the character of the subject, this chapter is not meant to be operational.

13.2. INSPECTION

Structures designed and constructed in conformity with the provisions of this Code should be inspected and maintained as frequently and carefully as possible, so that they will continuously fulfil all requirements related to the intended service and safety.

Particularly, structures of major importance or under adverse service conditions should necessarily be inspected periodically, adopting appropriate in-situ testing and monitoring strategies.

National authorities should decide on inspection and maintenance policies to be imposed on concrete structures under well defined conditions.

For conventional concrete structures under normal service conditions, the following time periods between successive competent inspections could be suggested

for houses, offices, etc.	10 years
for industrial buildings	5–10 years
for highway bridges	4 years
for railway bridges	2 years
for road bridges	6 years

13.3. REPAIR

All minor defects or light damage (non-structural) impairing the performance of elements or parts of the structure should be systematically rehabilitated.

If serious damage is observed or major defects are suspected (with possible structural consequences) an appropriate assessment and redesign procedure should be followed.

Relevant information is included in CEB Bulletin 162.

PART IV. DESIGN FOR PARTICULAR TECHNOLOGIES

14. PRECAST CONCRETE ELEMENTS AND STRUCTURES

14.1. DESIGN BASIS

14.1.1. General

This chapter deals with the design and detailing considerations special to structures made partly or entirely of precast elements, with basic reference to ordinary buildings, structured in wall systems, beam-column systems, or dual systems.

The term 'precast element' refers to any structural concrete element manufactured in purpose built technical facilities which are protected from adverse weather conditions by appropriate means. Composite elements are precast in part, then completed with cast in situ concrete.

Structures are wholly precast, when made of precast elements, or partly precast, when some elements are cast in situ. Both are characterized by the presence of joints, providing mutual connection between elements in the overall structural behaviour.

In general the rules regarding resistance and serviceability requirements of design for in situ cast concrete structures apply also to precast and composite constructions, unless modified or supplemented in this chapter.

Precasting of structural elements derives from a philosophy of industrialization of production processes, aimed at improving economy and quality of buildings.

Whereas the precast units are produced at a high technological level in plants, the connections will be executed on the site, possibly in simple and quick operations.

Production and assembly needs give rise to various solutions for connecting structural parts, differing from those of 'monolithic' cast in situ structures. Thus fulfilment of the common performance requirements may be obtained by ad hoc design criteria, including structural modelling, use of materials and details, which may differ from the traditional ones.

The great variety of problems related with production and assembly of precast structures is not dealt with extensively in this Code, being a matter of specific recommendations. However, this chapter points out some criteria consistently with the principles of the Code, which apply to general aspects of precast elements and structural systems pertaining to ordinary buildings. Durability considerations are also included.

14.1.2. Structural arrangement

The layout of the structure and the interaction between the structural members should ensure a robust and stable behaviour.

Deformaton due to differential loading, shrinkage,,creep, thermal effects and possible differential settlements of foundations should be accounted for.

Floors and walls composed of adequately tied precast elements may be used as wind-bracing horizontal and vertical diaphragms, provided that the corresponding in-plane forces can be appropriately transferred to the supporting structural elements.

Joints in precast structures may provide 'monolithic' connection by in situ casting of concrete over reinforcement protruding from units; thus realizing full continuity in a very similar way to cast in situ structures. The same may be obtained by post-tensioning steel reinforcement across grouted, glued or even dry joints with little interspace, or by means of only metal devices, although this might affect local stiffness and ductility. Structures with joints as above may behave as continuous frames.

Other joints may realize hinge-like restraints, able to transmit mainly shear and compression. In structures connected with such joints, bracing is needed.

The necessary interaction between elements is obtained by tying the structure together, using acting horizontal (peripheral and internal) and vertical ties.

Structural integrity (with particular regard to progressive collapse) shall be provided by an adequate 'strategy', choosing active and passive measures for controlling the occurrence and the propagation of damage. Engineering judgement is required in assessing these strategies for particular structures.

Elements themselves may be partly precast and completed in situ (composite elements) with large surfaces as interface connections, with or without transverse reinforcement, to give monolithic behaviour.

In general, precast elements have limited shrinkage and creep deformations after assembly. However, effects of differential values, particularly in composite members, should be evaluated. This may affect, among others, the spacing of expansion joints.

14.1.3. Analysis and design

The analysis of a precast structure should be based on assumptions compatible with the structural layout and with the actual detailing of the structure. Possible changes of the statical system during the different stages of construction should be assessed.

In addition to the limit-state requirements, the design (especially of connections) should account for an easy and reliable assembly and maintenance.

Joints should be adjustable during assembly and—especially those made of materials other than concrete—be protected from aggressive agents. They should furthermore, if necessary and possible, be inspectable and replaceable.

The analysis should account for

- the behaviour of structural units as such, in subsequent functional phases (including transitory situations: see subsection 14.1.4)
- 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; some non-conventional conditions may be envisaged (e.g. for dimensioning against overall instability, explosions, impacts, progressive collapse).

When the production is industrialized and continuously monitored and a complete quality assurance system is supervised and certified by an independent body, which implies systematic rejections in case of non-compliance, the partial safety factors γ_c and γ_s may be chosen between 1.5 and 1.4 and between 1.15 and 1.10, respectively, depending on the reduction (up to 50%) of the tolerances defined in subsection 1.4.5.

No reduction of γ_c is justified by the mere use of prefabrication, nor by a reduction of the standard deviation of concrete strength, γ_c being intended to be applied to an unchanged specified 5% fractile. A smaller standard deviation implies a smaller mean strength, and the fractile 5% is such that the safety degree is not significantly modified. However, a known smaller standard deviation may be taken into account in the acceptance criteria of concrete.

Usually the quality assurance system can ensure better compliance with the specified tolerances on formwork and reinforcement for precast than for cast in situ elements.

The reduction of γ_c is applicable only to the precast elements, and not to the joints, for which section 6.10 is applicable.

Under the same conditions, in case of continuous production of identical elements and making a direct statistical assessment of the performance of the whole production possible,

- either another reduction of γ_c may be made by dividing it by 1.1η if the mean ratio η of the strength of drilled cores and standard specimens is greater than 0.9

This other reduction of γ_c implies that it has been experimentally and statistically demonstrated that the quality of workmanship justifies a reduction of the mean conversion factor η, normally taken equal to 1.1, included in γ_c. In the direct assessment of R_d a possible reduction of the dispersion of the conversion factor is implicitly taken into account.

The two possible reductions of γ_c may be accumulated.

- or the design resistance R_d may be directly assessed according to the rules relating to statistical interpretation in the case of design by testing.

All reductions of partial factors envisaged in this subsection may be taken into account in the design on the basis of prior information if statistically demonstrated.

14.1.4. Transitory situations

Precast units should be designed to make all operations of demoulding, handling, storage, transport and erection safe and to avoid undesirable effects on their future behaviour within the structure.

All static and dynamic transitory situations the units may undergo should be verified in the design, accounting for actual material properties at their time of occurrence. These may be determined by means of reduced sampling and simplified criteria. Partial safety factors may be reduced for verifications in transitory situations during handling and transportation.

Lifting devices and loops must have ductile behaviour and be dimensioned accounting for possible uneven lifting, for dynamic actions, and for lower bond capacity in the early stages.

The verification of safety of the structure during assembly should be performed for a combination of actions, such as wind, dead weight and possible superimposed load, with their respective eccentricities, expected to act during assembly.

The edges of elements containing supports for security devices (such as railings and lifelines) should be checked for a nominal load of 2 kN in the most unfavourable position. Floor elements should allow for passage of workers, and be able to carry a nominal vertical load of 1.4 kN in dwellings and 2 kN on any 200 × 200 mm accessible square.

The sampling of concrete at the time of particular operations, like prestressing or demoulding, may be less extended than normally required. Thus, conventional characteristic values may be worked out on the basis of empirical formulae.

Reduced safety factors may be justified by the limited probability of actions exceeding the nominal values and the limited consequences of failure before normal use (see section 2.2 of CEB Bulletin d'Information No. 191 'General Principles on Reliability for Structures'). On the other hand, effects on the normal safety requirements must be considered.

Possible reduction of steel ductility at low temperatures should be taken into account.

Unless more accurately determined, dynamic vertical effects of lifting and transport may be accounted for by treating the self-weight with a factor 1 ± 0.2.

Overloads due to demoulding might be measured.

Dynamic horizontal effects of transport should be prevented by bracing devices.

14.1.5. Tolerances

Allowance for construction inaccuracies should cover deviations in the production and the erection of the elements. They may be assessed from a statistical analysis of measured or predicted deviations.

It should be checked that the tolerances in the actual execution are compatible with the tolerances assumed in the analysis and design.

Tolerances affecting structural behaviour should be pointed out in the design documents.

For tolerance limits, refer to subsection 1.4.5.

Production of prefabricated units may be more accurate and may be subject to stricter quality control than in situ cast structures.

14.2. ELEMENTS

14.2.1. General design considerations

Dimensioning of concrete elements is performed in general according to the methods of dimensioning and detailing of concrete members presented in this Code.

Where such methods are not applicable, particular models may be used. Performance should be verified by means of behaviour models worked out on the basis of experimental tests, or by referring to acknowledged technical recommendations and technical approvals. Testing procedures and results should be adequately documented.

Production and assembly considerations may lead to innovative arrangements, in which sufficient safety and serviceability is obtained in a manner different from those normally utilized in cast in situ structures. Such deviations from traditional dimensioning and detailing rules require specific justification.

Especially connections may require global behaviour models (e.g. moment rotation, shear-displacement) rather than models on stress–strain relationships. Justification by testing may often be needed. Uncertainties of behaviour should be accounted for by adequate idealizations and safety factors.

In the design stage, appropriate action should be taken to facilitate correct execution.

Good design should minimize likelihood of errors on site, foresee assembling operations apt for visual control, and prescribe realistic tolerances. Further requirement may be a proper demountability of the structure.

14.2.2. Execution

(*a*) *Manufacture*

Installations, raw materials, working processes, etc. should satisfy the assumptions used in the design.

In order to avoid harmful chemical reactions during the manufacture of units, the temperature of concrete should not exceed specific limits appropriate to the concrete mix used. Different properties of concrete in different zones of the element, according to the type of casting should not impair structural behaviour.

(*b*) *Handling, storage and transportation*

Each structural element should have an identification mark.

Identification marks should give all necessary information with respect to identification and erection.

Handling, storage and transportation of the elements should conform to the design specifications.

Care should be taken to avoid differential deformations during storage.

(*c*) *Erection and assembly*

During erection, adequate safety against instability should be provided.

Erection of elements should be in accordance with design assumptions; e.g. minimum support length, contact stresses, bearing material, etc.

Components should not be installed if they have been damaged in a way which affects their structural performance.

Connections should satisfy the design functions with regard to safety and serviceability (including durability considerations).

Considerably damaged components should be reassessed for use in normal or reduced performance, repair, or rejection.

14.2.3. Reinforcement detailing

(a) Concrete cover

Nominal values of concrete cover may be reduced by 5 mm with respect to the values specified in Table 8.4.1, if accurate control on bar position and on concrete compaction is performed.

Reduction of concrete cover is related to better compaction in precast units than those specified in Table 8.4.1. Where fire resistance is needed, other minima may apply.

(b) Detailing of support areas

Detailing of bars near the bearings of precast elements shall be carefully done to ensure that the shapes prescribed can be placed in the positions required by the design.

14.2.4. Composite elements

The design of a composite element should be based on the material properties of the combined materials (strength, shrinkage, creep, thermal deformations).

Composite members may be considered monolithic, with respect to their resistance, when shear transfer can be performed safely (without discontinuous deformations at the interface), by pure interface bond or by interaction with connecting reinforcement.

The effectiveness of connecting reinforcement and its contribution to the failure mechanism in ultimate limit states shall be verified.

A composite element is a structural member composed of a precast part of reinforced or prestressed concrete, and a cast in situ part, connected to the precast part by bond, with or without reinforcement connectors.

The properties which affect shear transfer at the interface surface are: surface roughness and cleanness, concrete strength, strength of connectors, deformability and bond properties. The shear reinforcement should be anchored properly on each side of the interface. The splitting force caused by the shear reinforcement has to be considered, when significant. For composite floor members with flat and wide interfaces subjected to low shear stresses, it is allowed to rely upon friction at the interface only. Connecting devices across it may be omitted, see clause 14.4.2.2. However, the interface should be rough and properly cleaned before cast in situ concreting, the top layer adequately compacted and cured in order to prevent high early shrinkage.

For design in low shear, reference is made to FIP Guide to good practice on 'Shear at the interface of precast and in situ concrete'.

14.2.5. Construction details

14.2.5.1. Bearings

The integrity of bearings for precast members should be ensured by the effectiveness of reinforcement on both sides of the reinforced bearings and by a suitable limitation of the bearing stress.

Account should be taken of possible horizontal or flexural actions at the bearing due to creep, shrinkage and temperature effects, unforeseen eccentricities, etc., by either the provision of sliding and rotation bearings or suitable reinforcement.

In order to satisfy the requirements of stability and resistance, the support arrangements should consider

- limitation of the contact stresses (a_1)
- additional support lengths taking into account concrete cover on the main reinforcement of both supporting and supported element ($a_2 + a_3$).

When horizontal reactions cause friction movements, the possible accumulation of non-reversible shifts due to uneven behaviour under cyclic actions (e.g. thermal) should be prevented.

Support zones should be dimensioned and detailed to assure correct positioning and resistance, accounting for production and assembly tolerances.

Possible local effects of prestressing anchorages should be considered.

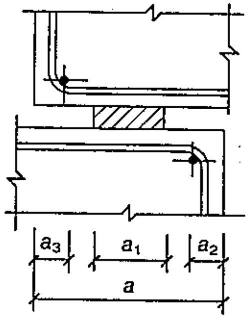


Fig. 14.2.1. Support length

The support length should normally satisfy the condition (net of tolerances)

$$a \geqslant a_1 + a_2 + a_3$$

Anchorage length of reinforcement should also be accounted for.

For masonry or plain concrete supports, it may be assumed that $a_2 = 25$ mm.

The shape of the sections of reinforced and prestressed concrete floors should be adequate to fulfil the requirements related to strength and serviceability (including durability considerations), as well as fire resistance.

14.2.5.2. Floor elements

Prestressed floor units should be designed and detailed with due regard to the risk of splitting and bursting in the anchorage zones of the prestressing tendons.

The overall depth and the thickness of flanges should comply with punching resistance, transverse distribution of loadings and transverse deflection limits.

14.2.5.3. Beams

The cross-section thickness of beams should not be less than 50 mm in their final presentation.

Simply supported beams should have sufficient torsional restraint to withstand horizontal actions due to wind, unsymmetrical loadings, imperfections and impacts.

The support, both on beam and on column or wall side, should be designed for an unintentional additional torsional moment

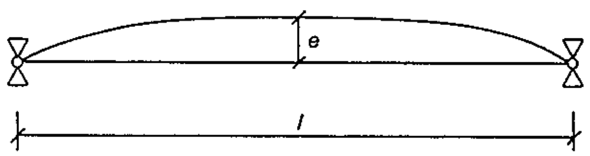


Fig. 14.2.2. Horizontal eccentricity e *at midspan*

Erection loads are often more critical because before grout is placed under the metal footing, all the loading is taken via the anchor bolts and/or shims.

The necessary anchorage depends on the design forces on the column base only.

The main tensile reinforcement of concrete corbels should be anchored positively near the extreme outer face, e.g. by welding cross bars or by welding to confinement angles. The eccentricity of the loading should be kept to a minimum.

In case of embedded structural steel corbels, the resistance of the concrete against crushing should be verified.

$$T_{ad} < V_{Sd} l/300$$

where

l is the beam span
V_{Sd} is the design vertical reaction (shear force)

and for a horizontal force

$$H_{sd} > 0.2V_{Sd} > 30\,\text{kN}$$

Lateral instability of the beam due to bending and torsion should be studied accounting for an unintentional horizontal misalignment at midspan

$$e > l/500$$

in addition to external actions.

Accurate account must be taken of realistic unfavourable restraint conditions.

14.2.5.4. Columns

(a) Longitudinal reinforcement
Pretensioned reinforcement may be accounted for in computing minimum reinforcement (see clause 9.2.3.1); minimum bar diameter does not apply.

(b) Reinforcement details
The provision of reinforcement against possible splitting at column ends should be considered.

(c) Metal footings
Column metal bases should be designed for both erection loads and loads which occur in service.

The column base should be anchored to the main column reinforcement.

(d) Corbels
Corbels should be capable of resisting vertical and horizontal actions due to loading, creep, shrinkage, thermal movements, eccentricities due to tolerances etc. (see also clause 14.2.5.1).

14.2.5.5. Foundation elements

Concrete sockets should be able to transfer vertical loads, bending moments and horizontal shear from columns to the soil. The joint between the column and the socket should be large enough to enable a good concrete filling below and around the column.

The joint-filling under the column base should enable good force transfer between the column and the foundations.

Vertical force is assumed to be transmitted entirely through the column base unless the lateral surfaces of column and socket are provided with adequate roughness or keys.

Bending and shear are assumed to be transmitted entirely to the socket walls.

14.2.5.6. Wall panels

(a) Plain concrete panels

Minimum concrete grade of panels should be C16.

Concrete panels should be designed as plain elements if the vertical reinforcement is less than $\rho = 0.003$. Panels shall contain boundary reinforcement to control shrinkage and thermal cracking.

Reinforcement to control cracking caused by in-plane shear shall be placed in the centre plane, or symmetrically.

The precast elements may contain also structural continuity reinforcement to be joined in situ with reinforcement of adjacent elements in order to produce a wall system, as required in section 14.5.

Local reinforcement of wall panels comprises the lifting devices and reinforcement to prevent structural damage during handling and transportation.

The thickness of panels for bearing walls should normally not be smaller than 100 mm, if made of normal concrete and not smaller than 140 mm, if made of lightweight concrete. Shear walls and flank walls made of normal concrete may not be less than 80 mm thick.

The above values should be increased by 20 mm for external walls.

It is recommended that connections for lifting panels from the top edge should be anchored to a depth taking into account dynamic and demoulding overloads, as well as concrete strength at demoulding time. Very severely loaded devices are to be anchored according to structural analysis. Unless analysed, the depth can be taken as 0.75 of the panel height.

Lifting points in the base of the panels should have adequate bearing capacity.

(b) Reinforced panels

Minimum concrete grade of reinforced panels should be C20. Wall panels may be considered in the structural analysis as reinforced elements if the reinforcement ratio is $\rho \geqslant 0.003$ in both directions.

The main reinforcement should consist of two meshes, placed near the external surfaces of the panel when its thickness is $\geqslant 150$ mm.

(c) Sandwich panels

Sandwich panels designed as composite elements should be verified by analysis or by testing.

The connectors between the external leaf and the bearing leaf should be made of corrosion resistant material. The design should account for possible different kinds of physical and chemical aggression, and for fatigue actions.

A sandwich panel normally consists of an internal bearing leaf, designed in accordance with the previous structural requirements (see (a) and (b)), and of a thermal insulation layer and of an external leaf. The latter should be free to deform independently; otherwise the curvature due to temperature and shrinkage shall be accounted for.

Care should be taken in controlling mesh position and concrete quality of the external leaf with respect to durability problems.

14.3. JOINTS

14.3.1. General

Joints must be designed to transmit all action effects implicit in the assumptions made in the analysis of the structure as a whole and in the design of the individual members to be joined. The design should ensure that

Particular attention should be paid to the detailing of the joint, in order to prevent premature splitting of spalling of concrete at the ends of precast units. Tolerances and fitting requirements, as well as construction requirements with respect to convenient completion and work inspection, are to be taken into account when dimensioning the joint.

- the joint is able to accommodate the relative displacements needed to mobilize 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 the structure as a whole
- account is taken of the tolerances anticipated in manufacture and erection.

When verifying the ultimate limit state of the joint, the resistance of the in situ portion, as well as the adjacent portions of the precast elements, is to be considered.

The resistance and the stiffness of the joints may be based on analytical methods, or on laboratory tests. The influence of workmanship imperfections at the site is to be taken into account. Unfavourable deviations from testing conditions should be evaluated when using 'design by testing'.

Compression, flexural and shear joints are defined by the primary action effect they sustain.

Tests may be especially useful to determine the sensitivity of strength and stiffness parameters to the expected variations in geometry and in workmanship.

The design should allow for effective assembly and filling operations; solutions that are highly sensitive to variations that cannot be controlled and which might impair the durability should be avoided.

14.3.2. Compression joints

This term is used for joints designed to transmit axial, or slightly eccentric compressive forces between precast units (such as columns, wall panels or struts) and other building elements.

The strength of mortar joint material should not be less than 70% of the adjacent precast concrete strength, if the mortar is not transversely confined, unless the joint is designed accounting for actual mortar strength.

Relatively weak joint material can initiate cracking in precast element, parallel to the direction of loading, and reduce the capacity of the joint.

The shape of the joint may affect its resistance.

Dry joints with no intermediate padding material may only be used where great accuracy in manufacture and installation is obtained.

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

The minimum and maximum dimensions, for joints filled after the unmatched elements are positioned, are chosen taking into consideration proper filling and compaction.

Account should be taken of unintentional eccentricities.

In assessing the resistance of compressive connections, the interaction between axial and shear forces should be accounted for.

Additional care is needed for appropriate detailing of longitudinal bars across the joint.

(*a*) *Joints between plain concrete elements*

The resistance depends upon the eccentricity of the normal load, the strength and confinement of the in situ concrete and/or of the mortar, and the fixing moment from the floor. This applies also to hinged joints between reinforced elements. If $V_{Sd} > 0.1A_j f_{cd}$ the reduction of N_{Rd} should be evaluated by means of interaction diagrams (see Bulletin No. 169).

Interaction of N and V may be disregarded if

$$V_{Sd} \leqslant 0.1A_j f_{cd}$$

(*b*) *Continuity joints*

Continuity joints between reinforced elements are the most frequent type of horizontal joints of columns in precast structures.

In this kind of joint, the vertical compression force N is transmitted through the direct contact of the connected elements and/or through the connection of reinforcement.

14.3.3. Shear joints

Shear joints appear in wall systems, in bracing walls interacting with beam and column systems, in prestressed segmental members, in floors for horizontal diaphragm action, as joints between precast and cast in situ concrete in composite members, and as beam to beam, column to column or beam to column joints in systems of prefabricated linear elements.

Plane and keyed joints are distinguished.

The structural requirements for shear joints refer also to flexural joints acting as shear joints as well.

Shear joints with negligible normal force are recommended to be keyed joints.

14.3.3.1. Ultimate resistance

For verification of the ultimate limit state of a shear joint it may approximately be assumed that the stress distribution is constant along a definite section of the joint length.

The design resistance V_{Rd} of joints which do not allow inspection for proper filling with concrete, is to be reduced by applying a supplementary $\gamma_{Rd} > 1.0$.

For γ_{Rd} the following values may be used

$\gamma_{Rd} = 1.3$ for open keyed joints or composite elements
$\gamma_{Rd} = 1.5$ for closed keyed joints
$\gamma_{Rd} = 1.6$ in the general case of plain joints.

Possible interaction with tensile forces should be accounted for.

The design resistance of the shear joint may be calculated, making use of the formula

$$V_{Rd} = \min \left\{ \begin{array}{c} \{\mu[A_s f_{yd}(1 + \cot\alpha)\sin\alpha + N_d] + 0.1 A_k f_{cd}\}/\gamma_{Rd} \\ 0.3 A_j f_{cd} \end{array} \right\} \qquad (14.3\text{–}1)$$

where

μ is 0.5 for smooth plane surfaces and 0.9 for rough or keyed surfaces

A_s, f_{yd}, α are the cross-sectional area, design yield stress and inclination of the steel bars crossing the joint and being well anchored on both sides

N_d is the design (most unfavourable) normal force acting on the joint section (positive when compressive)

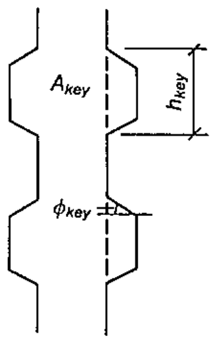
A_j is the cross-sectional area of the joint under compression

A_k, f_{cd} are the cross-sectional area of the portion of the joint keys interacting in the resistance, and the design strength of concrete in the considered joint portion ($A_k = 0$ for plain joints or when $A_k/A_j < 0.2$).

The simplified verification formula is based partly on a shear-friction mechanism, it accounts for the additional contributions of possible keys, and it is derived from series of tests on overall joint samples.

In case of tensile forces acting on the joint, N_d in eq. (14.3–1) is given a negative sign. When compressive, the most unfavourable N_d should be used.

More sophisticated models for specific cases including also dowel effect, may be found in chapter 3.



$A_j = b_j l_j$

$A_{key} = b_{key} h_{key} n$

l_j – joint length (e.g. storey height)

n – number of keys at length l_j

$h_{key} \geqslant 50$ mm and $t_{key} \geqslant 10$ mm

$h_{key}/t_{key} < 8$

$\phi_{key} \leqslant 30°$

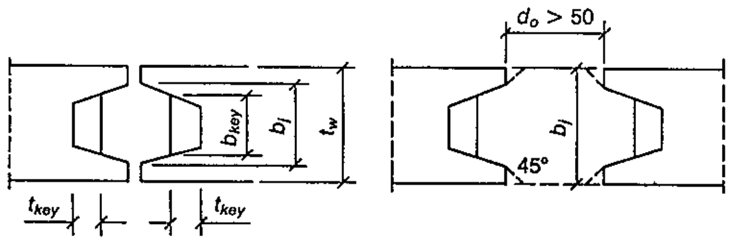


Fig. 14.3.1. Definitions—keyed joints

A minimum reinforcement ratio is advised (see Bulletin No. 169)

$$\frac{A_s f_{yd}}{A_j f_{cd}} \geqslant 0.01$$

An appropriate value of minimum transverse reinforcement is needed in order to secure a minimum ductility and to counterbalance uncertainties related to the determination of action effects and to the accuracy of models.

14.3.3.2. Stiffness

The determination of load effects may assume for simplification elastic behaviour of the structures. Individual parts of the structures, however, may be assigned a stiffness corresponding to the actual load level.

In the absence of more detailed information, the elastic shear stiffness C_j of the joint, applicable for linear analysis of a precast structure under normal actions, may be assumed as

$$C_j = 5 \times 10^3 \, V_{Rd}/A_j \text{ (MPa/m)}$$

where V_{Rd}, A_j are the design resistance and cross-section area of the shear joint.

14.3.4. Flexural and tensile joints

The ultimate limit state is to be verified neglecting the tensile strength of the concrete.

Besides the resistance to bending moment and/or to axial force, the resistance of the joint to shear is to be verified. The requirements given in subsection 14.3.3 then apply.

Continuity of reinforcement through the joint shall be secured. The jointing method used should be such that the assumptions made in the analysis of the structure are realized.

The reinforcement should be anchored in such a way that its full capacity can be mobilized.

Care should be taken to avoid bond failures of the bars also under accidental loadings.

Any type of connection that may not be justified on the basis of provisions of this Model Code shall be checked by testing.

Connections transmitting bending moments between floor slabs and precast walls, as well as beams and columns are categorized as flexural joints.

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

Continuity may be obtained by

- lapping of bars
- welding of bars
- reinforcement grouted into apertures
- overlapping reinforcement loops
- sleeving
- threaded couplers

or any other type of connection that can be shown capable of performing adequately.

14.3.5. Ties

The diaphragm action of floor and wall as required in subsection 14.1.2 should be secured by ties (in two orthogonal directions) effectively continuous and anchored at the perimeter of the structure.

Ties may be provided entirely within in situ concrete toppings or connections, partly within in situ concrete and partly within precast members or entirely within precast members.

Lapping to provide continuity of bars used for ties should be avoided if possible. Where lapping is used, the lap should be established as far away as possible from critical joints between elements and should be spiral—confined within the bond length.

Ties are reinforcements (concentrated or not) running in walls or floors from end to end, to realize the tensile bars of ideal horizontal and vertical truss systems, providing for normal and accidental situations.

Ties may be post-tensioned.

Connecting all the bars in one zone (plane) of the joint is permitted. The influence of such an arrangement on the structural reliability should be appropriately taken into account, see also subsections 14.4.1 and 14.5.1.

14.4. FLOOR SYSTEMS

14.4.1. General

The main structural requirements of floors are span load bearing, transverse load distribution, diaphragm distribution of horizontal actions, as well as resistance against accidental actions affecting the floor or its supporting structures.

Most common precast floor types are

- hollow-core reinforced or prestressed slabs, with or without in situ topping
- composite floors made of precast plates, completed with in situ solid or hollowed concrete grouting
- solid slabs
- ribs with in situ topping grouted on thin transverse slabs or on hollow infilling blocks
- double tees, with or without topping.

Floor ties are dimensioned according to the structural analysis, particularly with regard to diaphragm action.

Some specific design criteria (when needed) rely partly on special primary resisting schemes, justified by redundancy of the structure and by the provision of subsidiary resisting schemes, mobilizing the resistance of ties.

However, appropriate minimum ties should be provided in order to counterbalance uncertainties related to the determination of action effects and to the accuracy of models.

If not otherwise calculated, the following minimum design ultimate forces should be provided

- peripheral ties: 60 kN,
- internal ties: 20 kN per metre width.

14.4.1.1. Common design criteria

(a) Support arrangements

The precast floor units may be simply or continuously supported. The connections at the supports should be designed and detailed accordingly. Unintentional restraint effects at the support of simply supported floors should be appropriately considered.

The top reinforcement of continuous or cantilevering precast floors can be anchored in the precast elements or in a structural topping layer. In the ultimate limit state, transfer of stresses between the top layer and the precast units should be appropriately assured.

For floor elements manufactured by long-line methods, the way of cutting the elements may affect the support area and should be considered when the normal support length is prescribed.

(b) Longitudinal joints

When transverse load distribution or diaphragm action is assumed, longitudinal joints between precast floor elements should be designed to transfer the vertical and longitudinal shear forces from one element to the other.

The shear forces can be transferred along joint interfaces (by means of concrete or mortar), or by concentrated shear connectors, or in cast in situ toppings.

The transfer of vertical and horizontal shear forces can be realized in different ways

- grouted joints
- welded connections
- reinforced topping on the floor elements.

Minimum joint width and minimum joint opening in the top of the floor should be designed considering the actual type of grout and grouting method. If tie bars are placed and anchored in the joint, the joint width at the tie bar level should not be less than twice the bar diameter, or 25 mm.

The strength of the joint fill, the quality of the grout, the size of the joint and the joint filling operation may affect the shear capacity and should be considered in the design and detailing.

When the connections at the joints are not designed for bending moment capacity and provided with reinforcement or metal devices accordingly, they may be assumed to act as shear transfer hinges.

In most cases, with properly distributed bracing members, a regularly shaped floor diaphragm as a whole may be considered as rigid in the horizontal plane. However, particular cases might require an analysis accounting for the in-plane deformability of the floor diaphragm.

The ties can be distributed along the edges of the precast floor elements or concentrated in the joints. Ordinary reinforcement or prestressing tendons in the precast floor units may be part of the tying system if properly connected across the joints by special tie arrangements. Special tying members can be incorporated in the floor system at intermediate joints or at the edges if anchored to the floor by proper tie arrangements.

Corners and voids of the diaphragm must be properly detailed so that the continuity of the tying system is assured.

When a cast in situ topping is provided, this should normally have a thickness on average not less than 40 mm, with minimum actual local thickness not less than 30 mm.

Vertical keys, suited for horizontal shear, may be of various shapes, dimensions and spacing; the depth is normally not less than 8 mm. Also proper artificial roughness of the joint interfaces can produce effects of indentations.

The connections at the supports should be designed and detailed aiming at structural integrity and ductility in collapse situations. The connections should withstand large imposed deformations.

Possible combinations of horizontal and vertical shear should be considered in the design of the connections.

The joint faces of the precast members should be suitably rough or indented in order to make shear transfer possible by friction or interlocking effects.

(*c*) *Transverse load distribution*

Transverse load distribution between adjacent floor elements under action of live loads across the joints should be ensured by appropriate shear transferring connections. The shear capacity should be determined.

(*d*) *Diaphragm action*

Precast floors can act as diaphragms for transferring horizontal forces to the bracing vertical elements when the following conditions are satisfied

- the completed floor is analysed under realistic assumptions of the deformability of the bracing members, the precast elements and the connections
- the elements are connected and the completed floor is provided with a tying system so that lateral force transfer is possible by arch or truss action
- the tying system is able to resist all tensile forces deriving from the in-plane actions (bending, shear, tension)
- force transfer capacity is provided to the bracing vertical elements in order to realize the necessary interaction; possible stress concentrations at the joints should be considered in detailing
- in shear joints formed by longitudinal grouted joints, the average shear stress should not exceed 0.10 MPa in the ultimate limit state, if the joint faces are not provided with vertical indentations; if indented or keyed joint faces are used, higher values may be adopted, based on adequate experimental data.

(*e*) *Structural integrity*

If the floor system is providing the integrity of the structure as a whole, precast floor elements should be directly or indirectly tied to the support at both ends.

14.4.2. Specific design criteria

For specific types of precast floor units, deviations from the general code criteria can be permitted under certain conditions. Additional design criteria and minimum specifications on cross-section geometry, reinforcement, capacity and detailing of connections are given accordingly for each specific type of precast floor.

For the following types of precast floors, special rules for the design and detailing are presented in clauses 14.4.2.1 to 14.4.2.3

- floors with hollow core units
- composite floors with thin precast slab elements
- composite floors with precast rib and block systems.

14.4.2.1. Floors made of hollow-core units

(a) Support design

The bearing length should normally not be less than 55 mm. An even contact zone along the support should be provided.

The bearing length may be reduced at intermediate wall–floor connections if the design shear force is 50% or less of the shear capacity of the floor elements and the contact stress at the support is less than $0.25 f_{cd}$. However, the reduced value should not be less than 40 mm.

The design and detailing of the connections at the support should prevent a possible restraint initiating a shear failure near the support. Crack formation in the top layer should be prevented.

Shear design of units without shear reinforcement in the supporting zone should take into account the following modes of failure:

— shear compression failure;
— shear tension failure;
— anchorage failure of the main reinforcement.

Ties can be indirectly anchored to hollow core floor elements in concreted cores or in grouted joints, taking due account of reduced bond stresses that might arise from site practice and/or reduced cover in joints.

For anchorage in grouted joints, measures are required for the anchorage length of end anchors to make it possible to utilize the full strain capacity of the steel.

Axial tensile forces may occur due to restrained deformations at the support.

Models for verifying different failure modes are contained in FIP Recommendations for Hollow Core Slabs.

The tensile strength of concrete may be taken into account to determine the shear resistance on condition that

- sufficient load distribution capacity is available in the direction perpendicular to both the load and the span
- no significant axial tensile forces occur.

Any possible local damage has to be compensated by the redistribution capacity of the member itself or the structural system as a whole.

(b) Transverse load distribution

Transverse distribution is possible under the following conditions:

— longitudinal joints are able to transfer the vertical shear forces;
— a tying system prevents relative lateral displacement of adjacent precast units.

Under any rare combination of actions, the transverse flexural stresses occurring at the bottom of the precast elements should be limited to

$$\sigma_{ct,fl} < f_{ctk}/1.5$$

The transverse load distribution capacity can be calculated on the basis of slab theory assuming that the joints between slab units behave as linear hinges.

The reason for taking into account the axial tensile strength instead of the flexural tensile strength in the transverse direction is due to the hollow cores.

The ultimate load bearing capacity can be determined assuming that one longitudinal crack appears in the slab and cannot transmit any bending moment but only shear force, similar to the joints between the slabs.

Blocks have a minimum punching resistance of 1.5 kN with reference to a load applied on a 50 mm × 50 mm surface in the most unfavourable position.

14.5. WALL SYSTEMS

14.5.1. General

Wall systems consist of structural walls stiffened at each floor level by floors acting as horizontal diaphragms. The floors may be supported by the walls (load bearing wall system) or by beams and columns (dual system). In both cases the stability of the structures against horizontal actions is assured by the structural response of the wall system.

The structural requirements on overall integrity and robustness of the structure are to be met by tying the wall system together i.e. by providing the reinforcement at each floor level or similar arrangements, and by possible vertical ties.

When tensile stresses are expected in the wall, vertical tie reinforcement should be provided along the edges. This reinforcement crosses the horizontal joints and is an integral part of them.

Post-tensioning may be applied as well, to counteract the tensile stresses in the wall system.

When detailing the wall system, the difference in deformation and in displacement of the individual differently loaded wall portions (including deformations due to temperature and shrinkage) should be taken into account.

14.5.2. Structural analysis

Structural analysis of the wall system subjected to vertical and horizontal actions may be carried out separately for each action and the safety requirements may be checked for the sum of the resulting structural responses.

When the wall is subject to normal force combined with in-plane bending, the vertical strips as shown in Fig. 14.5.1 should be verified conventionally for compression, under the equivalent normal force

$$N_d = \sigma_d^* bt$$

where

t is the wall thickness
σ_d^* is the average design normal stress on the strip area tb.

The tensile part should be treated as a tensile member subjected to the action $N_d(+)$.

The above action effects are to be applied both to panels and joints.

The safety requirements regarding the normal force N_d acting in the wall are to be verified in two critical zones

Large deformations in the joints, exceeding those corresponding to the resistance of the joints (assumed for verifying the safety of the structures under normal actions) may be taken into account in the analysis of secondary stabilizing systems.

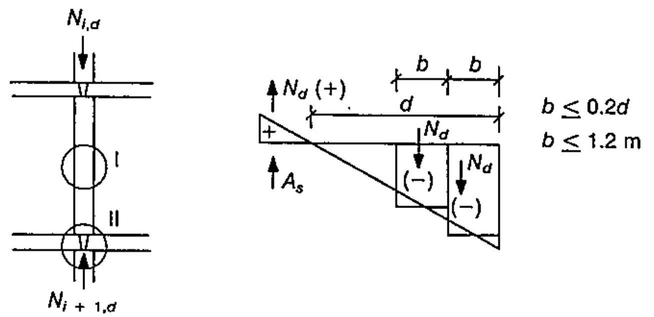


Fig. 14.5.1. Wall subject to normal forces and in-plane bending

14.4.2.2. Composite floors with precast plates

For the design of composite floors, the completed floor can be considered monolithic, if the following conditions are fulfilled

- the method of construction of the precast plates ensures the roughness of the interface
- a bonding reinforcement is provided, with a correct anchorage in precast and cast in situ layers, if the design shear stress is higher than $0.3 f_{ctd}$ or if dynamic or impact loadings are to be considered
- a transverse connection is achieved at the joints between precast plates.

For floors with cores of non-structural materials, the following conditions should be met

- maximum spacing of ribs: 700 mm
- minimum thickness of precast plates and in situ concrete: 50 mm.

The bearing capacity and the deflections of the prefabricated plates during construction phases have to be verified by calculation or tests, taking into account the temporary supports.

The prefabricated plates are simply reinforced, or prestressed slabs. They are used as the shutter for the in situ concrete topping during construction.

Bonding reinforcement may be realized by ordinary stirrups, lattice trusses, ladders of reinforcement steel or adequate baskets.

Bonding reinforcement is necessary only in the areas where the limit shear stress can be exceeded. Thus, precast plates may have bonding reinforcement only near the bearings.

The transmission of shearing forces on a bearing may be realized by bars projecting from the edge of the plate, and bonded in in-situ concrete; if there are no projecting bars at the edge, this transmission may be achieved by stirrups close to the bearing zone.

A transverse connection across joints may be made with reinforcement bars placed within the cast in situ topping.

14.4.2.3. Composite floors with ribs and blocks

The transverse loadbearing capacity of this type of floor is based on shear transmission in the compression flange and on stiffening ribs in transverse direction. For spans exceeding 6 m, at least one transverse rib is required.

The width of ribs should be at least 50 mm.

If ribs are widened at the bottom to resist negative bending moments, the increase in the width is limited to 1 : 3 splay.

The spacing of ribs should not exceed 700 mm.

The bond capacity of the interface between the prefabricated part of the longitudinal rib and in situ concrete has to be verified in accordance with subsection 14.3.3.

Additional reinforcement in the anchorage zone of prestressing steel is generally required.

The ribs should be designed also for resisting loads applied before the hardening of in situ concrete. Otherwise they should be supported by intermediate props. Deflections at that stage should be checked.

Composite floors with precast ribs and blocks consist of partly prefabricated longitudinal ribs, intermediate prefabricated blocks or shutters and in situ concrete topping.

Part of the web and the topping are completed with in situ concrete. Infill blocks may function as simple shutter; in such a case, blocks typically have a modulus of elasticity

$$E_c < 8000 \text{ MPa}$$

A topping acts as a compression flange, if

$h_0 \geqslant 40$ mm and $i/12$ for dwellings
$h_0 \geqslant 50$ mm and $i/8$ for other buildings,

where h_0 is the thickness (i is the clear distance between ribs) and transverse reinforcement is provided consisting of at least $\phi 6$, $a \leqslant 330$ mm.

If infill blocks act as structural part, they typically have a modulus

$$8000 < E_c < 25000 \text{ MPa}$$

and shall provide all structural performance.

zone I in the middle part of the wall, where the total eccentricity e_{tot} of the normal force N_d is affected by the second order eccentricity e_2 due to the slenderness of the wall,

zone II in the joint, where the wall resistance is reduced due to the disturbances of the forces.

Models for verifying the resistance of zone II are given in CEB Bulletin No. 169—'Draft Guide for the Design of Precast Walls Connections'.

The bearing capacity of the wall in zone I is to be verified according to the general rule for slender concrete members. In the verification (according to section 6.6) the additional eccentricity e_a covering execution inaccuracies should be taken into account.

The initial eccentricity e_0 of the normal force resulting from the vertical actions may be assumed *a priori* (hinged model of the wall systems) or calculated making use of a continuous frame model. The restraint to bending of floor–wall joints is to be taken into account in the second case.

14.6. BEAM AND COLUMN SYSTEMS

14.6.1. General

Building structures made of linear precast elements (beams and columns) may be designed in order to meet one of three basic structural schemes

(*a*) continuous framework
(*b*) cantilevered continuous columns
(*c*) 'hinged joints' plus bracing walls.

Bracing walls may also be used combined with schemes (*a*) and (*b*) (dual schemes).

Normally, all schemes are associated with horizontal floor decks acting as diaphragms.

The behaviour of the three schemes differs essentially in the distribution of horizontal actions.

Scheme (b) is normally associated with low-rise buildings such as industrial halls. Often floor connections in the roof provide only axial (no in-plane shear) action.

The conventional lateral force as overall means for verifying stability is assumed to be 1% of vertical loads acting at each floor.

Regularity of layout and dispersed bracing is recommended.

Structural integrity and limitation of damage propagation after accidental events should be provided depending on the particular layout (see sub-section 14.1.2).

14.6.2. Structural analysis

Continuity may be assumed for connections concreted in situ and fully respecting the detailing of monolithic frame joints or for connections that have been proved continuous by means of accurate testing of resistance and stiffness under unfavourable conditions.

Testing of connections for rigid beam-column joints should consider effects of repeated and alternate loadings, of internal actions (shrinkage and temperature constraints), and possible degradations. Seismic requirements are not included in this code.

In case of absence or discontinuity of diaphragm (e.g. hall roofs with simply supported, non-mutually connected elements), effects of displacement on stability members and compatibility of bearings should be verified.

Connections should otherwise be considered as hinged in the analysis related to lateral actions, and for the assessment of a realistic stiffness for other actions.

Particular attention should be paid in diaphragm detailing of structures pertaining to scheme (*c*).

This is a method of assembling precast segments by means of post-tensioning.

14.7. SEGMENTAL CONSTRUCTION

The deformation line of the assembled structure is to be computed and checked at every construction stage.

14.7.1. Joints

14.7.1.1. Orientation

An inclination of less than 20° is acceptable if the shape of the surface is such that the existence of sufficiently high friction forces can be counted on.

In principle the middle surface of a joint should be perpendicular to the post-tensioning force.

14.7.1.2. Types of joints

Two types of joints are commonly used

- wide joints (with cast in situ concrete, dry-pack mortar, or grout joint filler)
- match cast joints (with epoxy bonding agent or dry joints), including wide joints made to match at a later stage.

The recommended approximate widths are

$\geqslant$ 80 mm for cast in situ joints
$\geqslant$ 25 mm for tamped joints
$\leqslant$ 15 mm for grouted joints

The obtained accuracy of structure depends mainly on the preparation and casting of the joints during erection rather than on the precision of the segments themselves.

14.7.1.3. Wide joints

The width of cast in situ joints should permit effective vibration of the concrete and allow for any necessary welding of reinforcement or of metal inserts.

The strength of the joint material should be not less than 25 MPa or that of the joined segments.

14.7.1.4. Match cast joints

The compressive strength of the material of the joint should be at least equal to that of the concrete of the joined segments.

If necessary, consideration should be given to the creep of this material.

The joint surfaces fit each other perfectly because each element is cast against those with which it will be in contact in the structure. The surface can comprise indentations, which ensure correct positioning when they are erected and adequate resistance to load effects.

The recommended width of the epoxy resin layer is 1–3 mm.

14.7.2. Structural analysis

When calculating the load effects and strength and drawing up the detailed design, the structural components obtained by these methods may be considered as monolithic for serviceability limit states.

No tension is acceptable in the joints under rare combinations of actions.

The shear in joints without shear keys (wide joints only) should satisfy $V_{Sd} \leqslant V_{Rd}$.

For verifying limit states of decompression, an appropriate temperature gradient between opposite layers of the assembled structure should be accounted for.

The characteristic values of μ_k can be found by tests.

V_{Rd} may be calculated according to clause 14.3.3.1.

The shape of the keys may be chosen to suit the particular application; they can be broadly distinguished into two categories

- single keys, generally large and localized
- multiple keys, generally covering as much of the joint surface as practical.

Shear keys located in the compressive zone under bending (ULS) are of major importance.

Appendices

APPENDIX A

NOTATION

The aim of this appendix is merely to describe the symbols to be used without prejudging the exact definition of each term.

Notation should be in accordance with ISO 3898.

a.1. Construction of symbols

A symbol to represent a given quality or term is constructed as follows.

(1) The main letter of the symbol is chosen from a.2, a.3, a.4, or a.5 on the basis of its dimensions and its use, as given in Table a.1.
(2) Descriptive subscripts may be chosen at will. When subscripts other than those appearing in a.6, a.7 and a.8 are used, a clear definition of their meaning shall be given.
(3) In constructing symbols, the first subscripts shall indicate the location and the following ones the cause (nature, location, etc.). When necessary to avoid confusion, the use of a comma between the two categories of subscript is recommended.
(4) When there is no risk of confusion, all or some of the descriptive subscripts, may be omitted.
(5) Numbers may be used as subscripts if necessary.
(6) An apostrophe (') representing compression is to be added to symbols representing geometrical quantities, if necessary.

Table a.1. Guide for the construction of symbols

Type of letter	Dimensions	Use
Roman capital	Force; force times length; length to a power other than 1; temperature	1. Actions and action effects; work; energy 2. Area; volume; first and second moments of area 3. Temperature 4. Moduli of deformation (exception to the general rule)
Roman lower case	Length; quotient of length and time to a power; force per unit length or area; except when used as a subscript	1. Linear dimensions (length, width, thickness, etc.) 2. Velocity; acceleration; frequency 3. Actions and action effects per unit length or area 4. Strengths 5. Descriptive letters (subscript) 6. Mass 7. Time
Greek capital	—	Reserved for mathematical symbols
Greek lower case	Dimensionless	1. Coefficients and dimensionless ratios 2. Strains 3. Angles 4. Densities (related to mass or weight) (exception to the general rule) 5. Stresses

Note: concepts not included in Table a.1 should be classified in the nearest category.

To avoid confusion, the following precautions should be taken.

(a) The possibility of confusing 1 (numerical) with l (letter) in some typed documents has been recognized. L will therefore be used in place of l (letter) when there would be a risk of ambiguity in typed documents.

(b) Roman upper and lower case letter O shall not be used as a main letter owing to the possibility of confusion with other symbols. For the same reason, it is recommended that kappa (κ) and chi (χ) should be avoided as far as possible. Lastly, if the lower case Greek letters eta (η) and omega (ω) are used, care must be taken in writing them to avoid confusion with the lower case Roman letters n and w.

a.2. Meaning of Roman capital letters*

A area
B
C torsional moment of inertia
D fatigue damage factor; diffusion coefficient
E modulus of elasticity; earthquake action
F action in general; local loading
G permanent action; shear modulus
H horizontal component of a force
I second moment of a plane area
J creep function
K (permeability) coefficient
L can be used for 'span; length of an element' in place of l
M bending moment; coefficient of water absorption
N axial force
O (void)
P prestressing force
Q variable action
R strength (resisting load effect); reaction at a support; resultant
S load effect (M, N, V, T); static moment of a plane area
T torsional moment; temperature
U
V shear force, volume
W modulus of inertia
X reaction or force in general, parallel to x-axis
Y reaction or force in general, parallel to y-axis
Z reaction or force in general, parallel to z-axis

* Roman capital letters can be used to denote types of material, e.g. C for concrete, LC for lightweight concrete, S for steel, Z for cement.

a.3. Meaning of Roman lower case letters

a deflection; distance; acceleration
b width
c concrete cover
d effective height; diameter (see also h and a.5)
e eccentricity (see also a.5)
f strength of a material
g distributed permanent load; acceleration due to gravity
h total height or diameter of a section; thickness
i radius of gyration
j number of days
k all coefficients with dimension
l span; length of an element

m bending moment per unit length or width; mass; average value of a sample
n normal (longitudinal, axial) force per unit length or width
o (void)
p (void)
q distributed variable load
r radius
s spacing; standard deviation of a sample
t time; torsional moment per unit length or width; thickness of thin elements
u perimeter
v velocity; shear force per unit length or width
w width of a crack
x co-ordinate; height of compression zone
y co-ordinate; height of rectangular diagram
z co-ordinate; lever arm

a.4. Use of Greek lower case letters

alpha α angle; ratio; coefficient
beta β angle; ratio; coefficient
gamma γ safety factor; density; shear strain (angular strain)
delta δ coefficient of variation; coefficient
epsilon ε strain
zeta ζ coefficient
eta η coefficient
theta θ rotation
iota ι (void)
kappa κ (to be avoided as far as possible)
lambda λ slenderness ratio; coefficient
mu μ relative bending moment; coefficient of friction; mean value of a whole population
nu ν relative axial force; Poisson's ratio
xi ξ coefficient; ratio
omicron o (void)
pi π (mathematical use only)
rho ρ geometrical percentage of reinforcement; bulk density
sigma σ axial stress; standard deviation of a whole population
tau τ shear stress
upsilon υ (void)
phi ϕ creep coefficient
chi χ (to be avoided as far as possible)
psi ψ coefficient; ratio
omega ω mechanical percentage of reinforcement

.5. Mathematical symbols and special symbols

Σ sum
Δ difference; increment (enlargement)
$\varnothing$ diameter of a reinforcing bar or of a cable
' (apostrophe) compression (only in a geometrical or locational sense)
e base of Naperian logarithms
exp power of the number e
π ratio of the circumference of a circle to its diameter

n number of . . .
w/c water/cement ratio
$\not>$ not greater than: *indicates the upper bound in a formula
$\not<$ not smaller than: *indicates the lower bound in a formula
$<$ smaller than
$>$ greater than

*These symbols placed at the end of an expression indicate that where the result to which it leads is higher (or lower) than the limit given, then the values given should be taken into account and not the result obtained from the formula.

a.6. General subscripts

a	support settlement; additional; accidental load
b	bond; bar; beam
c	concrete; compression; column
d	design value
e	elastic limit of a material
f	forces and other actions; beam flange; bending; friction
g	permanent load
h	horizontal; hook
i	initial
j	number of days
k	characteristic value
l	longitudinal
m	mean value; material; bending moment
n	axial force
o	zero
p	prestressing steel
q	variable load
r	cracking
s	ordinary steel; snow; slab
t	tension;* torsion;* transverse
u	ultimate (limit state)
v	shear; vertical
w	wind; web; wire; wall
x	linear co-ordinate
y	linear co-ordinate
z	linear co-ordinate
1, 2, 3 . . .	particular values of quantities
∞	conventional asymptotic value

*When confusion is possible between tension and torsion, the subscripts *tn* (tension) and *tr* (torsion) should be used.

a.7. Subscripts for actions and action effects

$a(A)$	support settlement; accidental action
cc	creep of concrete
cd	delayed elasticity of concrete
cf	delayed plasticity of concrete
cs	shrinkage of concrete
ep	earth pressure
$eq(E)$	earthquake; seismic
ex	explosion; blast
$f(F)$	forces and other actions
$g(G)$	permanent load
im	impact
lp	liquid pressure

$m(M)$	bending moment
$n(N)$	axial force
$p(P)$	prestress
$q(Q)$	variable load
$s(S)$	snow load
$t(T)$	torsion; temperature
$v(V)$	shear
$w(W)$	wind load

a.8. Subscripts obtained by abbreviation

abs	absolute
act	acting
adm	admissible, permissible
cal	calculated, design
crit (or *cr*)	critical
ef	effective
el (or *e*)	elastic
est	estimated
exc	exceptional
ext	external
fat	fatigue
inf	inferior
int	internal
lat	lateral
lim	limit
max	maximum
min	minimum
nec	necessary
net	net
nom	nominal
obs	observed
pl	plastic
prov (or *pr*)	provisional (stage of construction), provided
red	reduced
rel	relative, relaxation
rep	representative
req	required
res	resisting, resistant
ser	serviceability, service
sup	superior
tot	total
var	variable

APPENDIX B
TERMINOLOGY ON CONSTRUCTION WORKS

In order to avoid some ambiguities which are common in the usual language, it is recommended to use the following terms in accordance with the definition given below.

- *Construction works*: Everything that is constructed or results from construction operations. In accordance with ISO 6707 Part 1, this term covers both building and civil engineering works. It refers to the complete construction comprising both structural and non-structural elements.
- *Execution*: The activity of creating a building or civil engineering works. The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.
- *Structure*: Organized combination of connected parts designed to provide some measures of rigidity. This term refers only to load carrying parts.
- *Type of building or civil engineering works*: Type of 'construction works' designating its intended purpose, e.g. dwelling house, industrial building, road bridge.
- *Form of structure*: Structural type designating the arrangement of structural elements, e.g. frame, beam, triangulated structure, arch, suspension bridge.
- *Construction material*: A material used in construction work, e.g. concrete, steel, timber, masonry.
- *Type of construction*: Indication of principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction, composite construction.
- *Method of construction*: Manner in which the construction will be carried out, e.g. cast in place, prefabricated, cantilevered.
- *Structural system*: The load-bearing elements of a building or civil engineering works and the way in which these elements are assumed to function, for the purpose of modelling.

APPENDIX C

DESIGN BY TESTING

This appendix has a provisional character. It may be amended when the relevant basic documents of the JCSS become available.

c.1. Scope

This appendix contains guidance for the experimental assessment of the response of structural members not included in the Model Code's range of application. In very particular cases only (see section c.3) the content of this appendix may be used in order to modify Code provisions.

In this context 'response' may mean deformation or strength characteristics.

Examples of such structural members may be

- assemblages of reinforced concrete prefabricated elements
- units with special shape (polyhedric panels, bunkers).

Examples of special design problems studied experimentally may be

- prestress losses
- special production processes (accelerated curing etc.).

However, this appendix covers only those cases in which a statistical interpretation of results is possible.

Examples of tests which generally cannot be statistically interpreted

- tests on frictional losses of prestress made by measurements of forces transmitted by tendons on the structure during construction
- tests on uncertainties related to static equilibrium made by measurements on the smaller reaction of supports.

The development of computer calculations has considerably reduced the interest of tests to this purpose.

This appendix does not contain information about experimental investigation used for structural analysis.

Tests to be made for Agrément would need further developments and are not fully covered by this appendix.

c.2. Definition

Design by testing (DbT) is considered here as a procedure where loading tests on limited series of representative specimens are used for the determination of the response of structural members.

In a wide sense design by testing may cover all kinds of load tests on either scaled-down models, on special specimens or on the entire structure itself; these tests are expected to give additional information to the designer.

In this respect, two particular cases may be distinguished in practice

(*a*) *Small structural elements repeatedly produced under factory conditions.* An improvement of the existing design model is sought; a considerable number of full scale specimens are tested, taking into account all actions and influences during the lifetime of the structure where these elements will be incorporated.

(*b*) *Larger structural elements to be constructed under site conditions.* An analytical model may not be initially available or considerable modification of existing models is sought. A comparatively lower number of tests are carried out simulating the real life conditions. This case is meant to be the main case addressed by this appendix.

However, the following cases are not covered by this appendix:

- non-destructive test-loadings of single finished structures
- wind tunnel or earthquake simulator tests
- routine control tests on reinforced concrete industrial products
- accelerated durability tests.

c.3. Aims of design by testing

The aim of the design by testing is to obtain design values for the response of structural members under specified load conditions with respect to a certain limit state.

Design by testing may be pursued instead of design by calculation when

(*a*) the calculation models are insufficient or out of the Code's range of application

Such cases may be deviations from the provisions of the present Code with respect to, for example

- geometric shapes
- combination of new materials
- detailing and/or connections
- severe actions.

(*b*) design by testing may also be used in some particular cases and with special care, either when the calculation models given in the present Code are thought to lead to uneconomic results, or where greater precision is desirable.

Such cases may be considered where, for example

- the economic importance of a project or of an element to be mass produced justifies the effort of design by testing
- the codified model, in some particular conditions is judged to be clearly too rough.

...ial d... s ne...ary for several reasons.

- Computational design versus ultimate limit states often covers indirectly serviceability as well and intermediate limit states (which for simplicity are not codified): a modification of the codified provisions for ULS might result in deficiencies in this respect.
- Design by testing is not able to account for some uncertainties (e.g. on structural analysis or on parasitic phenomena due to the environment) which are covered by code provisions, nor to justify a reliability format more precise than the codified one.

c.4. Requirements

The design-by-test procedure should ensure that the design leads to the same level of reliability as the Code.

This is particularly important when design by testing is to be permitted for financial reasons (see section c.3).

To this end all possible conditions, actions and possible influences expected during the lifetime of the structure shall be appropriately reproduced in laboratory.

In this respect, see *inter alia*

(*a*) the in-time variation of the basic variables (subsection c.5.2)
(*b*) the condition affecting the conversion factor η (clause c.9.1.1).

If some of these variations and conditions are expected to have a systematic effect on the structural response under investigation, their complete reproduction in the laboratory shall be secured.

The whole procedure shall be developed by the designer and be approved by the relevant counterpart.

The experimental assessment shall be performed by qualified institutions with staff experienced in planning, executing and evaluating tests.

c.5. Planning

c.5.1. Calculation model—limit states

A plan should be drafted by the designer, which shall contain the objective of testing and all indications necessary for the sampling or manufacturing of the specimens, the execution of the tests and their evaluation.

The experimental procedure may cover either an ultimate limit state or a serviceability limit state. In either case, a minimum knowledge of the relevant response mechanism and parameters is sought. It is desirable that on the basis of this knowledge, an empirically or physically based calculation model is available prior to testing, in order to evaluate the response. This model is referred to as 'prior calculation model' g_R

If such a model is not available prior to testing, trial preliminary tests should be carried out in order to facilitate parameter identification.

$$R_t = g_R(X, W, D) \qquad (c\text{–}1)$$

where

R_t is the response of the available analytical model
X is the vector of basic variables
W is the vector of nominal variables
D is the vector of unknown coefficients to be determined by the testing.

The response depends in general on a set of measurable quantities. The quantities which are random with respect to an elementary population are referred to as basic variables X. The quantities which are considered as deterministic, with respect to an elementary population, are referred to as nominal variables W. Nominal variables may be constant within a population or vary in a predetermined manner.

The terminology used in this clause and the following ones, with regard to the variables, is not exactly the same as in Bulletin 191 and in chapter 1 (section 1.3 and subsequent) of this Model Code because the character of some variables (fundamental or not) is not yet established at this stage. Some variables indeed, although random, may have very little influence on the response.

All quantities affecting the response must be present in the model either as basic variables or as nominal variables.

In case of unidentified significant parameters, a larger scattering of results is expected.

c.5.2. Information on basic variables

For the basic variables included in the calculation model, statistical parameters have to be known. If these values are not known, it is recommended to estimate them by preliminary tests. Where information is available only from a limited population, variances need to be increased accordingly.

Special care is necessary because statistical parameters of basic variables have not an intrinsic character. They generally depend on the considered statistical population, which may be very different at the level of the Code, at the level of the specimens and at the level of the expected application of the test result.

Special mention should be made of the quasi-deterministic concrete strength of the specimens which exhibit a very low variability of concrete strength. Thus, in the relevant model, the characteristic strength of in-structure concrete should be made equal to the characteristic strength of the specimen concrete.

The in-time variation of the basic variables should be taken into account in the model.

For example

- in-time development of the concrete strength positive or negative because of some additives
- possible decrease of the tensile strength of concrete due to hydro-thermal cyclic conditions
- possible decrease of steel ductility or fatigue performance due to minor corrosion conditions.

Gross-errors cannot be covered by this procedure.

c.5.3. Number of specimens

The number of tests carried out should be sufficiently large in order to lead to results with satisfactory small confidence interval (see section 8.3).

The necessary number of tests is higher where not only mean values but also standard deviations are to be statistically assessed.

c.5.4. Scale effects

The specimens should preferably be dimensioned as close to full scale as possible, so that the scale effects do not increase the model uncertainties. Otherwise, similitude laws and fracture mechanics shall be appropriately taken into account.

The dimensions of the specimens should, if possible, cover the entire range of the probable variation of the dimensions of the structural element.

When scaled-down specimens are used, special attention should be paid to the influence of aggregate size, re-bars size, concrete workability etc. on the response.

c.5.5. Actions

The actions may be direct forces (loads in general), or imposed deformations or other influences varying in space and/or in time. The scheduled actions process must be fully determined by a set of load (or occasionally of other influence) parameters. The actions process shall be selected so that they are representative for the anticipated scope of application of the structural member.

Each specimen may be subjected to the same or different action processes.

Other influences may be temperature, humidity etc., conditioning the behaviour of the specimens and their response.

The actions process ('load path') should also include data regarding rate effects (e.g. load application velocity, the order of application of each influence).

c.5.6. Origin of specimens

The specimens should be specifically manufactured for the testing.

Specimens taken from production may be used either for preliminary tests or in the case (a) defined in section c.2.

c.6. Testing conditions and measurements

Special care should be taken in order to check that all assumptions made in the planning (see section c.5) are satisfied.

Quantitative and qualitative observations are highly recommended in order to check these assumptions, made implicitly or explicitly, or to allow alternative evaluations afterwards.

If, during testing, a given condition is found to influence considerably the results, this condition shall be more systematically studied as a new basic variable of the model g_R.

c.6.1. Basic and nominal variables

The actual values of all basic and nominal variables included in the calculation model should be, as far as practicable, determined by direct measurements for each specimen and each experiment.

When a direct measurement is not possible, an indirect measurement is allowed. Then the respective conversions should be used, by introducing the necessary new variables of the conversion factors or by increasing the variance of the relevant variable.

In the case of indirect measurement of a variable, the difference of the statistical properties between the populations in the laboratory and on site shall be taken into account.

c.6.2. Actions

The actual values of the imposed actions have to be recorded during the test and especially at the 'critical point' of the limit state considered.

Since the response of the structural member is in general identified in terms of load intensity at which the limit state is reached, the actual limit value of the limit load should be measured with additional care.

c.6.3. Deformation—structural behaviour

During the tests, systematic measurements shall be carried out concerning the deformations (elongation, deflection, rotation).

It is recommended that measurements are redundant in order to make a mutual checking of the results possible.

c.7. Laboratory report

The laboratory report should contain at least

- the name of the part asking for the design by testing
- the names of the laboratory staff involved in the DbT
- the scope of the DbT
- technical description of the specimens (dimensions, materials, fabrication technique, number of specimens, etc.)
- testing procedure
- all measurements of basic and nominal variables
- the actions process
- the deformation of specimens
- crack pattern
- the failure mode and the critical material
- photos and/or video recordings
- the prior calculation model used
- laboratory comments on the obtained results.

Further useful guidance on the matter may be found in the Recommendation of RILEM TC 125.

c.8. Statistical analysis of test results

c.8.1. Estimation of the coefficients D

The estimation of the coefficients D (eq. (c–1)) may be based on

- least square methods
- maximum likelihood methods.

The coefficients D may also be considered as random variables, but in order to check the validity of the equation (c–1), in a first step, the coefficient D should be given the best value D_m.

For linear models or models which can be transformed to linear, coefficients generally should be assumed as normally distributed.

Where no linear models are obtained, a Taylor expansion in the vicinity of the expected design point may be used to avoid non-linear regression analysis.

For linear models of the type

$$Y = d_1 x_1 + d_2 x_2 + \ldots + d_n x_n = DX'$$

the joint distribution of D is a multivariate central t-distribution. For details, reference is made to the relevant literature.

When coefficients of mechanical models are to be estimated, prior information on these coefficients should be derived from physical considerations.

Often it is known that coefficients can only take values larger or smaller than unity.

c.8.2. Correlation between experimental and theoretical values

In general, the observed/experimental values R_e of the response will be different from the corresponding theoretical values $R_t = g_R(X_m, W, D_m)$ (predicted by the model). This scatter is usually measured by the correlation coefficient ρ $(0 \leqslant \rho \leqslant 1)$.

When the correlation coefficient is high the correlation is considered to be sufficient.

$\rho = 0.85$ to 0.90 is considered to be a sufficient value.

High correlation does not necessarily imply a good model. In fact, if the number of unknown parameters tends to the number of observations, then ρ tends to unity. This is for example the case when for one basic variable, a polynomial of $(n - 1)$ degree has been considered as prior calculation model

$$Y = d_0 + d_1 x + d_2 x^2 + \ldots + d_{n-1} x^{n-1}$$

where n is the number of test results.

In such a case $\rho = 1$.

A graphic representation of the differences $(R_e - R_t)$ as a function of R_t is necessary to verify the validity of the prior calculation model.

c.8.3. Characteristic value

The characteristic value $R_k = g_R(X_k, W, D_k)$ (5% fractile or other) can be calculated from test results by means of appropriate statistical analysis.

The characteristic value R_k generally is defined as the value for which

$$P(R_t < R_k) = 0.05$$

with a level of confidence that normally is taken about 0.75 (unilateral limit).

c.9. Design procedure

c.9.1. Design values

c.9.1.1. Design values for ULS

The design value of the response is given by the expression

$$R_d = \eta R_k / \gamma_{Rd} \gamma_m$$

where

R_k is the characteristic response defined statistically on the basis of the test results

γ_m is the material partial safety factor adopted according to the failure mode of the material decisive for the bearing capacity

If relevant, the characteristic values X_k are substituted in R_k by other representative values depending on the combination and/or the limit state.

The material safety factor γ_m is to be adopted in accordance with this Code as

$$\gamma_m = \gamma_s \quad \text{or} \quad \gamma_m = \gamma_c$$

see section 1.6 for numerical values of the γ-factors according to the failure mode and to the material decisive for the failure.

The differences covered by the factor η may, for example, be

- loading time (when concrete strength is critical, a factor equal to 0.85 should be applied at least to account for sustained loads)
- support conditions
- humidity conditions and their alternations
- differences in geometry
- differences in workmanship and/or curing conditions.

Influences of systematic character (e.g. a brittleness factor such as $1 - f_{ck}/250$) should be covered by the model and not by η.

Those conversion conditions included in γ_m should not be duplicated when assessing η.

Only influences of secondary importance may be accounted for by means of conversion factors estimated in a more or less empirical way.

The basic value of γ_{Rd} results from a comparison to be made between $D_k \eta / D_m \gamma_m$ adopted above with the η/γ_m given in the Code in a similar case of design by calculation. γ_{Rd} should be increased according to experience in cases where the scatter of D is high (e.g. its coefficient of variation is greater than 0.15) or the expected variability of the basic variables is significantly higher on site than in specimens (see subsection c.5.2), or the value of the conversion factor on site conditions seems to be less than the already adopted η value.

Experts' opinions may be used in evaluating η and γ_{Rd}, provided that all systematic differences are taken into account in the tests.

η is a conversion factor, taking into account the differences between testing conditions and conditions in the actual structure.
If a more detailed model is known, accounting separately for the role of both materials, separate conversion and safety factors may be used accordingly.

γ_{Rd} is a complementary model-uncertainty-factor intended to cover differences between the testing conditions and the actual ones in the structure, which cannot be accounted by D_k, by γ_m or by the conversion factor η.

γ_{Rd}-values for SLS are generally different than for ULS; some minor aspects of a conversion factor may as well be accounted for by this γ_{Rd} factor.

c.9.1.2. Design values for SLS

The design values for the serviceability limit states are given by the expression

$$R_d = R_k / \gamma_{Rd}$$

with R_k and γ_{Rd} as defined in clause c.9.1.1.

c.9.2. Verification

The most frequent verification inequality is

$$S_d \leqslant R_d$$

where

S_d denotes the design value of the load effects
R_d denotes the design value of the the response.

Basic documents

MENEGOTTO, M. Justification of structural components and assemblages by testing—Relationship with analytical code design. FIP Commission on Prefabrication. April 1983.

THORENFELDT, E. Design by testing of concrete structures; CEB Commission I.

TASSIOS, T.P. Design based on testing; Scientific Papers, Faculty of Civil Engineering, N.T.U.A., Vol. 2, April–June 1983.

CEB/ECCS. Note No. 10 'General rules for design by testing', July 1985.

CEB/ECCS. Note No. 11 'Evaluation of test results', July 1985.

LEWICKI, B. 'Some remarks in connection with the anticipated reviewing of the Model Code 1978', CEB Commission I, April 1985.

LEWICKI, B. 'Example to Note No. 11', 'Evaluation of test results', August 1986.

MAIER, W. 'Design by Testing', April 1986.

EC3. 'Background document for Chapter 9 of Eurocode 3', July 1988.

JCSS. 'Estimation of structural properties by testing for use in limit state design', August 1989.

LEWICKI, B. 'Design by testing', ISO, TC98, SC2, November 1989.

APPENDIX D
CONCRETE TECHNOLOGY

d.1. Scope

This appendix gives information on the technical requirements to be satisfied by the constituent materials of concrete, concrete composition, the properties of fresh and of hardened concrete, the production, placing and curing of concrete and the verification of the required properties. Concrete which satisfies these requirements will have mechanical and durability properties generally in accordance with the properties on which the design rules of this Model Code are based. This appendix does not deal with special concretes such as fibre concrete, heavyweight concrete, polymer concrete, repair mortars and organic compounds other than admixtures. However, it includes information on lightweight aggregate concrete and on concrete with a compressive strength in excess of 50 MPa, often referred to as high strength concrete.

d.2. Reference documents

This appendix should be interpreted in conjunction with other international documents such as the standards and recommendations prepared by

CEN:	European Committee for Standardization
ISO:	International Organization for Standardization
RILEM:	International Union of Testing and Research Laboratories for Materials and Structures

Particular attention is drawn to the following documents though other national or international standards may be of equal relevance.

prENV 197	Cement; composition, specifications and conformity criteria, June 1989.
ENV 206	Concrete—Performance, production, placing and compliance criteria.
ISO 1920	Concrete tests—Dimensions, tolerances and applicability of test specimens.
ISO 2736/1	Concrete tests—Making of test specimens—Part 1: Sampling of fresh concrete.
ISO 2736/2	Concrete tests—Making of test specimens—Part 2: Making and curing of test specimens for strength tests.
ISO 4012	Concrete—Determination of compressive strength of test specimens.
ISO 4013	Concrete—Determination of flexural strength of test specimens.
ISO 4108	Concrete—Determination of tensile splitting strength of test specimens.
ISO 4109	Fresh concrete—Determination of the consistency—Slump test.
ISO 4110	Fresh concrete—Determination of the consistency—Vebe test.
ISO 4111	Fresh concrete—Determination—Degree of compactibility (Compaction index).
ISO 4848	Concrete—Determination of air content of freshly mixed concrete—Pressure method.
ISO 6275	Concrete, hardened—Determination of density.
ISO 6276	Concrete, compacted fresh—Determination of density.
ISO 6782	Aggregates for concrete—Determination of bulk density.

ISO 6783	Coarse aggregates for concrete—Determination of particle density and water absorption—Hydrostatic balance method.
ISO 7031	Concrete, hardened—Determination of the depth of penetration of water under pressure.
ISO 7033	Particle density and water absorption of fine and coarse aggregates for concrete (pycnometer method) (at present at the stage of draft standard).
ISO 7034	Cores of hardened concrete—Taking, examination and testing in compression (at present at the stage of draft standard).
ISO 8045	Concrete, hardened—Determination of rebound number using the rebound hammer (at present at the stage of draft standard).
ISO 8047	Concrete, hardened—Determination of ultrasonic pulse velocity (at present at the stage of draft standard).
ISO 9812	Fresh concrete—Determination of consistency—Flow test (at present at the stage of draft standard).
RILEM CPC7	Direct tension (final recommendation, 1975).

In this document frequently reference is made to 'Durable Concrete Structures—CEB Design Guide' CEB Bulletin d'Information No. 182, 1989.

d.3. Definitions

Some of the technical terms relating to concrete technology and used in this appendix are defined as follows.

(1) *Concrete*: Material formed by mixing cement, coarse and fine aggregate and water and produced by the hydration of the cement paste (cement and water); in addition to these basic components, it may also contain admixtures and/or additions. If the maximum particle size of the aggregate is 4 mm or less, the resulting material is generally termed mortar, not concrete.

(2) *Fresh concrete*: Concrete still in the plastic state and capable of being compacted by normal methods.

(3) *Hardened concrete*: Concrete which has hardened and developed strength.

(4) *Site mixed concrete*: Concrete batched and mixed on or near the construction site by the user.

(5) *Ready-mixed concrete*: Concrete batched in a plant outside or on the construction site, mixed in a stationary mixer or a truck mixer and delivered by the producer to the user as fresh concrete ready for use either on the construction site or into a vehicle of the user.

(6) *Batch*: The quantity of concrete mixed in one cycle of operations of a batch mixer, or the quantity of concrete conveyed as ready-mixed concrete in a vehicle, or the quantity discharged during approx. 1 min from a continuous mixer.

(7) *Normal weight concrete*: Concrete having an oven-dry (105°C) density greater than 2000 kg/m^3 but not exceeding 2800 kg/m^3.

(8) *Lightweight aggregate concrete*: Concrete having an oven-dry density of not more than 2000 kg/m^3. It is entirely or partly produced by the use of aggregate that has a porous structure (lightweight aggregate).

(9) *Heavyweight concrete*: Concrete having an oven-dry density greater than 2800 kg/m^3.

(10) *Admixture*: Product which is added in quantities generally less than or equal to 5% by mass of the cement before or during mixing or during an additional mixing operation, causing the required modifications to the normal properties. In special cases amounts $> 5\%$ may be added.

(11) *Addition*: Finely divided inorganic material that may be added to the concrete in order to improve certain properties or to achieve special properties. There are two types of inorganic additions: nearly inert additions (Type I) and pozzolana or latent hydraulic additions (Type II).

(12) *Aggregate*: Concrete component consisting of uncrushed and/or crushed natural and/or artificial mineral substances with particle sizes and shapes suitable for the production of concrete.

(13) *Normal weight aggregate*: Aggregate with a particle density between 2000 and 3000 kg/m^3, the particle density being determined, for example, according to ISO 6783 or ISO 7033.

(14) *Lightweight aggregate*: Natural or artificial aggregate consisting of particles with a porous structure and with a particle density less than 2000 kg/m^3, the particle density being determined, for example, according to ISO 6783 or ISO 7033.

(15) *Heavyweight aggregate*: Aggregate having a particle density larger than 3000 kg/m^3, the particle density being determined, for example, according to ISO 6783 or ISO 7033.

(16) *Cement*: A hydraulic binder, i.e. a finely ground inorganic material which when mixed with water, forms a paste which sets and hardens by hydration reactions and processes and which, after hardening, retains its strength and stability even under water.

(17) *Effective water content*: Mixing water plus water already present on the surface of the aggregates or in the admixtures and additions.

(18) *Water/cement ratio*: Ratio of effective water content to cement content of the concrete (refer also to clause d.6.3.3.2).

(19) *Entrained air*: Microscopic air bubbles intentionally incorporated in concrete during mixing, usually by use of surface active agents; typically between 10 and 1000 μm in diameter and spherical or nearly so.

(20) *Entrapped air*: Air voids in concrete which are not purposely entrained and which are significantly larger and less useful than those of entrained air, 1 mm or larger in size.

(21) *Designed mix*: A mix for which the user specifies the required performance of the concrete and any additional characteristics, and the producer is responsible for providing a mix which complies with the required performance and additional characteristics.

(22) *Prescribed mix*: A mix for which the user specifies the composition of the mix and materials to be used. The producer is responsible for providing the specified mix but is not responsible for the performance of the concrete.

(23) *High strength concrete*: Concrete with a characteristic strength higher than 50 MPa.

d.4. Constituent materials

d.4.1. General requirements

The basic materials (i.e. cement, aggregates, water, admixtures and additions) shall be suitable for making concrete which will attain and retain the required properties. Therefore, the materials should meet certain requirements as to composition and physical properties. Moreover, their quality should be consistent, and no impurities shall be introduced during transport and storage.

Even small traces of foreign materials such as gypsum, lime, zinc or sugar can have an adverse effect on the essential properties of concrete, such as the setting, hardening or dimensional stability. Constituents shall not contain ingredients which may cause corrosion of embedded reinforcement. Only such basic materials should be used whose properties are verified by an appropriate certification body and which are subject to continuous quality control with the exception of water. General instructions for the choice and evaluation of the constituent materials are given in the following. Further information may be sought in international and national standards or regulations.

d.4.2. Cement*

d.4.2.1. Types and requirements

The cements shall satisfy the requirements of national or international standards. The most important standards for all applications are those related to setting behaviour, strength development, dimensional stability and, for some special applications, heat of hydration and resistance to internal and external chemical attack.

The following classification is widely accepted and proposed, e.g. in prEN 197

Portland cements	(CE I)
Portland composite cements	(CE II)
blast furnace cements	(CE III)
pozzolanic cements	(CE IV)

In all cases, for the purposes of calculation, cement is understood to exclude calcium sulphate and additives.

As a guideline the following limits for the composition of the various types of cement may be given which are identical to those presented in prEN 197. Portland cement should have a clinker content of at least 95%. Portland composite cement should consist of at least 65–80% Portland cement clinker and up to 35% ground granulated blast furnace slag, up to 28% natural or artificial pozzolans and up to 20% ground limestone filler. Blast furnace cements should have a minimum of 20% of Portland cement clinker and up to 80% of ground granulated blast furnace slag. Pozzolanic cements should have a minimum of 60% of Portland cement clinker and up to 40% of natural or artificial pozzolans.

In addition, cements may differ with respect to their strength class as well as with respect to their rate of strength development. Also cements with special properties may be used for special applications such as cements with a low heat of hydration, sulphate resistant cements or cements with a low effective alkali content.

* Since the approval of this Model Code by the CEB General Assembly a CEN standard for cement ENV 197 has been approved by the CEN member countries. The types and strength classes as well as the composition of cements given in ENV 197 differ from those referred to in this Model Code.

As an example in pr EN 197 distinction is made between the four types of cement listed above (CE I to IV), between 2 strength classes (32.5 and 42.5 MPa) and between normal and rapid hardening cements (R). Many national standards also specify a third strength class around 52.5 MPa, which is referred to in subsequent sections.

In this Model Code reference is made to cements in more general terms

slowly hardening cements	(SL)	e.g. CE 32.5
normal hardening cements	(N)	e.g. CE 32.5 R; CE 42.5
rapid hardening cements	(R)	e.g. CE 42.5 R
rapid hardening high strength cements	(RS)	e.g. CE 52.5

National or international standards may contain restrictions concerning the type of cement to be used for certain applications and additional requirements for the minimum strength and composition of cements used for pretensioned prestressed concrete or for cement grout used for prestressing tendons.

d.4.2.2. Handling and storage

Cement should be protected from moisture and impurities during transportation and storage. Cement stored in the open absorbs moisture and carbon dioxide from the air, which causes the cement to cake and impair its hydration capacity. Finely ground, rapid hardening high strength cements are particularly sensitive in this respect. Except in very dry climates paper bags do not provide sufficient protection for prolonged storage even under cover. Special packaging measures are required for cement expected to be stored for prolonged periods or at a high relative humidity. Before using cement, the consumer should make sure that it still complies with the applicable standard specifications.

The various types of cement should be clearly marked and so stored that the wrong type cannot be used by error.

d.4.3. Aggregates

d.4.3.1. Types

The aggregates used for concretes covered by this Model Code can be natural or artificial mineral substances, either crushed or uncrushed and with particle sizes, grading and shapes such that they are suitable for the production of concrete. The individual particles generally have a range of sizes and have a dense or a porous structure. Further definitions are given in section d.3.

Aggregates can be further distinguished according to their rock and mineral type, their constituent materials, or according to some properties of practical importance, e.g. the shape and surface texture of the particles and their strength and durability.

d.4.3.2. General requirements

Normally, aggregates shall satisfy the requirements in the national standards for the particular applications envisaged. In special cases, it is possible to use aggregates which do not comply with the standards or aggregates for which the available standards are not applicable, provided that there are sufficient data or experience to show that concrete made with these materials is satisfactory and that the suitability of the aggregate for the particular conditions of use has been checked.

It is a general requirement that aggregates shall not become soft and shall not be excessively friable or expand excessively when wetted. They shall not be liable to decomposition, shall not combine with the products of hydration of the cement to form compounds which damage the concrete, e.g. alkali-silica reaction, and shall not cause corrosion of embedded steel reinforcement.

In order to satisfy these requirements the amount of harmful substances in the aggregates should be limited. Harmful in particular are clay and silt, i.e. particles with a diameter less than about 0.06 mm; organic compounds such as humus, coal or wood; sulphates and salts which may be aggressive to embedded steel such as nitrates, halogenides, in particular chlorides. Some standards limit the amount of silt in sand to 3 or 4 percent by weight and in coarse aggregates to 1 to 2 percent by weight. Sulphates, expressed as SO_3, often are limited to 1 percent by weight. With regard to the limitation of chlorides refer to clause d.6.3.5. Reactive aggregates are dealt with in clause d.6.3.3.3.

Depending on their application, the aggregates shall also satisfy certain requirements as to grading, purity, strength, shape of the particles, surface texture and resistance to frost and abrasion. In the case of lightweight aggregates, the porosity and water absorption as a function of time are also of special interest (refer to section d.16).

For the design and construction, account shall be taken of any unusual effects which the aggregates might be known to have on certain properties of the concrete, e.g. on its strength, density, shrinkage, moisture movement, thermal expansion, elastic modulus or durability. For example, some dolerites and sandstones shrink when drying; concrete made with these can shrink significantly more than estimated from section 2.1 of this Model Code.

d.4.3.3. Aggregates from marine sources

Aggregates from marine sources may be used provided that their chloride content complies with national standards or the information given in clause d.6.3.5, and that they satisfy all other requirements for the particular application of the aggregates.

Large quantities of hollow or flat shells in marine aggregates can have an adverse effect on the workability of the fresh concrete and on the properties of the hardened concrete.

d.4.3.4. Handling and storage

The aggregate must not become contaminated with other materials during transport or storage. If aggregates of different gradings or of different types are delivered separately, they should not be mixed inadvertently. Segregation of the different sizes of aggregates within a size range should be prevented.

d.4.4. Mixing water

Water to be used as mixing water should comply with national standards and shall not contain harmful ingredients in such quantities as to affect the properties of the fresh or hardened concrete or to reduce the protection of the reinforcement against corrosion.

In case of doubt, the water should be tested beforehand. Water containing oil, fat, sugar or a high amount of humic acids is unsuitable. Seawater shall not be used for the production of prestressed concrete and in general of reinforced concrete. Its use for reinforced concrete is permissible in some countries in exceptional cases subject to restrictions in the choice of cements and the total chloride content of the concrete. In such cases only a part of the total mixing water should be seawater.

Water previously used to remove remnants of concrete from installations of ready-mixed concrete plants and from transport vehicles may be recycled and used as mixing water subject to a number of precautions.

- Recycling water has to satisfy the general requirements for mixing water of concrete.
- Special measures are necessary if part of the recycling water has been used to clean machinery or the outside of transport vehicles so that the water is contaminated with grease or oil.

- Silt or fine aggregate particles in the water should either be distributed uniformly or separated from the water by sedimentation.
- The content of silt or fine aggregate particles should be determined on the basis of density measurements of the water, and has to be taken into account in mix design.
- Recycling water should not be used for air-entrained concrete or for concrete with a high sulphate resistance unless it is shown that the recycling water does not contain too high amounts of C_3A. Limits for allowable C_3A content as given in national standards should be followed.
- Recycling water should be used with caution for architectural concrete since its use may lead to a non-uniform appearance of the concrete surfaces.

d.4.5. Admixtures and additions

d.4.5.1. Admixtures

Admixtures as defined in section d.3 are added to the concrete to improve certain properties of the concrete by their chemical and/or physical effects. In general, they are added in small quantities less than approx. 50 g or 50 cm^3/kg of cement. Depending on the amount and composition of an admixture its water content may have to be taken into account in calculating the water/cement ratio. In special cases cements containing admixtures may be used instead of adding admixtures to the mix.

Types of admixtures include plasticizing water reducing admixtures, high range water reducing superplasticizing admixtures, water retaining admixtures, air entraining admixtures, set accelerating admixtures, hardening accelerating admixtures, set retarding admixtures and water repellent admixtures. Whereas the efficacy of these admixtures is well established and accepted it is still controversial for corrosion inhibiting admixtures.

The user should have all necessary information on the constituents and properties of the admixtures and their effect on concrete and reinforcement including information on the typical dosage, the detrimental effects of under-dosage and over-dosage, whether or not the admixture contains substances enhancing steel corrosion such as chlorides and, if so, the chloride content and the permissible duration and conditions of storage.

Admixtures should be transported and stored so that their quality is not affected by physical and chemical influences such as frost or high temperatures. They should be clearly marked and so stored that error is excluded.

The admixtures shall not adversely affect those properties of the concrete which are important for a particular application. They shall neither impair the durability of the concrete nor combine with the ingredients to form harmful compounds or endanger the protection of the reinforcement against corrosion. Chlorides, in particular, may increase the risk of corrosion. For this reason, the use of admixtures containing chlorides for reinforced concrete or for concrete which might come into contact with reinforced concrete is prohibited outright in some countries and strongly restricted in others. With regard to the total chloride content of a concrete mix refer to clause d.6.3.5.

Whilst improving certain properties of the concrete, an admixture can adversely affect others, and it is therefore necessary to carry out tests to verify the suitability of the admixture. These tests should also provide data on the quantity of admixture to be added in an individual case.

d.4.5.2. Additions

Additions as defined in section d.3 are added to the concrete in order to improve certain properties or to achieve special properties. Because of their larger amounts, the volume of additions always has to be taken into account in mix design.

Inert fines, pigments etc. are classified as Type I additions. Type II additions include latent hydraulic or pozzolanic substances such as granulated blast furnace slag, pulverized fuel ash or silica fume. Additions should comply with national standards, and their effectiveness should be checked by trial mixes.

Additions must not be harmful and shall be compatible with the other ingredients of the concrete. This means, for example, that their contents of chloride, sulphur or magnesium and their loss on ignition must not exceed certain limiting values; otherwise they can exert a harmful effect (e.g. on the durability of the concrete or on its ability to protect the reinforcement against corrosion) which may not be detected by short-term tests. The user should have the necessary information regarding the nature of the additions and their permissible content in concrete.

d.4.5.3. Handling and storage

The transport and storage of admixtures and additions should be so arranged that their quality is not affected by physical and chemical influences. The packing materials and/or the delivery papers should clearly indicate the type of admixture or addition concerned, together with the conditions of storage and use.

d.5. Classification of concrete

In design concrete is generally classified on the basis of its compressive strength. For particular applications it may also be classified on the basis of its density or potential durability.

d.5.1. Classification by strength

Concrete may be classified on the basis of its characteristic compressive strength f_{ck} as given in subsection 2.1.1 of this Model Code.

The characteristic strength is defined as that value of strength below which 5% of the population of all possible strength measurements of the specified concrete are expected to fall. This code is based on the uniaxial compressive strength of cyclinders, diameter 150 mm, height 300 mm, stored in water at 20 $\pm$ 2°C until tested at an age of 28 days in accordance with ISO 1920, ISO 2736/2 and ISO 4012. Where specimens other than cylinders 150/300 mm are used and where they are stored in other than the standard environment conversion factors for the compressive strength of specimens of different size, shape and storage conditions should be determined experimentally or according to rules given in national standards. Clause 2.1.3.2 of this Model Code gives such conversion factors for cubes 150/150/150 mm made of normal weight concrete.

For special cases, it may be necessary to define a minimum strength limit at an earlier age in addition to the governing 28-day strength, e.g. to enable the forms to be struck earlier, for earlier prestressing or for earlier transportation of precast elements. In special cases it is also permissible for the required strength to be attained at a later age, e.g. where slowly hardening cements are used, provided that the expected loading history as well as the durability of the structure are taken into consideration.

d.5.2. Classification by density

Concrete may be classified on the basis of its density as defined in section d.3 (7), (8) and (9)

- normal weight concrete having an oven-dry density greater than 2000 kg/m^3 but not exceeding 2800 kg/m^3
- lightweight aggregate concrete having an oven-dry density less than 2000 kg/m^3
- heavyweight concrete having an oven-dry density larger than 2800 kg/m^3.

The treatment (placement, compaction, etc.) of heavyweight concrete demands observance of additional rules not given in this appendix.

d.5.3. Classification by durability

The durability of concrete is understood to be its resistance to physical and chemical attack such as frost or elevated temperatures, carbonation, sulphate attack etc. The resistance of concrete to such actions is governed primarily by its resistance to the ingress of aggressive media and thus by the capillary porosity of the hydrated cement paste as well as by entrapped air. A dense paste with a low capillary porosity is in most instances more durable than a paste with a high capillary porosity and a coarser pore system.

There is no generally accepted method to characterize the pore structure of concrete and to relate it to its durability. However, several experimental investigations have indicated that concrete permeability both with respect to air and to water is an excellent measure for the resistance of concrete against the ingress of aggressive media in the gaseous or in the liquid state and thus is a measure of the potential durability of a particular concrete.

There are at present no generally accepted methods for a rapid determination of concrete permeability and of limiting values for the permeability of concrete exposed to different environmental conditions. However, it is likely that such methods will become available in the future allowing the classification of concrete on the basis of its permeability. Requirements for concrete permeability may then be postulated; they would depend on exposure classes i.e. environmental conditions to which the structure is exposed.

Though concrete of a higher strength class is in most instances more durable than concrete of a lower strength class, compressive strength per se is not a complete measure of concrete durability, because durability primarily depends on the properties of the surface layers of a concrete member which have only a limited effect on concrete compressive strength.

d.6. Concrete performance requirements

d.6.1. General considerations

The composition of a particular concrete should be so chosen that a required performance both with regard to strength and to durability is assured. However, not all concrete performance requirements can be evaluated sufficiently on the basis of direct experiments. Therefore, until adequate test methods are developed, the performance requirements—particularly those for concrete durability—have to be expressed on the basis of certain rules with regard to concrete composition and choice of materials.

Consequently, the components of the concrete, cement, aggregate, water, admixtures and additions must be selected and the mix proportions so chosen that all relevant performance criteria are met. In particular consistence and resistance to bleeding of the fresh concrete, density, strength and other mechanical properties of the hardened concrete as well as its durability and ability to protect embedded steel against corrosion are of importance.

d.6.2. Requirements for strength

The compressive strength of concrete depends primarily on the water/cement ratio, on the degree of hydration and, therefore, on age and curing of the concrete as well as on the strength class of the cement used. It is also influenced by type and strength of the aggregates as well as by type and amount of additions.

The relations between water/cement ratio and concrete compressive strength are not unique. Therefore, they have to be determined at least for each combination of type of cement, type of aggregate and for a given concrete age. As a rough guideline the relations between water/cement ratio and characteristic compressive strength f_{ck} at an age of 28 days for concretes made with cements of different strength classes are given in Fig. d.1. Relations between concrete compressive strength and other mechanical properties are given in section 2.1 of this Model Code. With regard to the effect of additions refer to clause d.6.3.3.2.

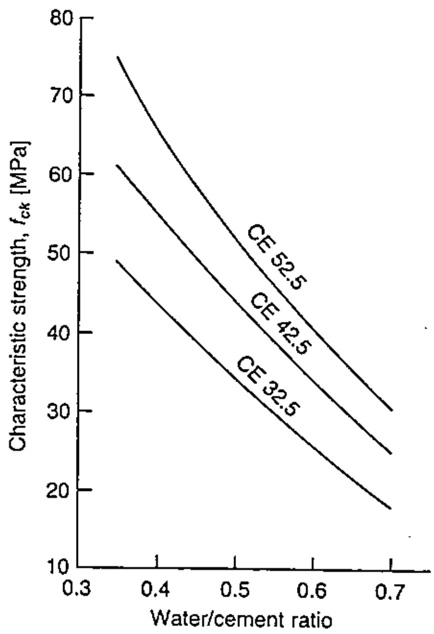


Fig. d.1. Approximate relations between the water/cement ratio and the characteristic compressive strength f_{ck} of concrete for various strength classes of cement and quartzitic aggregates

d.6.3. Requirements for durability

d.6.3.1. General considerations

To produce a durable concrete, which will withstand exposure to certain environmental conditions as given in clause d.6.3.2 and which protects the reinforcing steel against corrosion during the intended lifetime, the following aspects should be taken into account

- choice of suitable constituent materials containing no ingredients harmful to the durability of the concrete or causing corrosion of the steel
- choice of a concrete composition such that the concrete
  — satisfies all specified performance criteria for fresh and hardened concrete
  — can be placed and compacted to form a dense cover to the reinforcement
  — avoids harmful internal actions, e.g. alkali-silica reactions
  — withstands external actions, e.g. environmental influences such as weathering, gases, liquids and soil
  — withstands mechanical attacks, e.g. abrasion
- mixing, placing and compacting of the fresh concrete such that the concrete constituents are distributed uniformly in the mix and are not segregated and that the concrete achieves a closed structure
- curing of the concrete such that especially the surface zone and the cover of the reinforcement obtain the desired properties to be expected from the mix.

d.6.3.2. Exposure conditions

The requirements to be met by the concrete depend on the environment to which the concrete is exposed. Environment in this context implies chemical or physical actions resulting in effects which are not considered as loads in structural design.

In many instances different parts of a structure may be subjected to different exposure conditions. In some cases local conditions, i.e. the 'micro-climate' may be decisive for the durability of the entire structural member. The major environmental conditions for concrete are classified according to Table 1.5.1 in subsection 1.5.2 of this Model Code as well as in Table d.1 of this appendix. When selecting an exposure class for a particular application from Table d.1 it should be borne in mind that for some elements more severe conditions may prevail during construction than assumed for the completed structure.

The exposure conditions given in Table d.1 apply to concrete. A somewhat different classification is necessary to describe the environmental conditions aggressive to the reinforcement (refer to 'Durable Concrete Structures—CEB Design Guide').

d.6.3.3. Choice of materials

1. Types and strength classes of cement

Already at the design stage of a concrete structure the following aspects, which are relevant for the choice of type and strength class of the cement to be used should be kept in mind:

- required characteristic strength of the concrete
- required rate of strength development
- dimensions of the structure including cover to reinforcement, with regard to the rate of hydration
- environmental conditions during casting
- exposure conditions of the finished structure, in particular aggressive environments such as sulphate attack, exposure to freezing and thawing with or without de-icing chemicals
- curing of the concrete.

If the required characteristic strength of the concrete is high, the use of cements of a high strength class may be advantageous. Other parameters being equal, Portland cements (CE I according to clause d.4.2.1), other

Table d.1. Exposure classes for concrete related to environmental conditions (from ENV 206)

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

* E.g. in commercial laundries.
† Chemically aggressive environments are classified further in clause d.6.6.4, Table d.3. They may occur alone or in combination with exposure classes 1–4.

types of cement with a high content of Portland cement clinker and in particular R-cements as defined in subsection d.4.2 exhibit a strength development which is more rapid than that of cements having a higher content of additional constituents. Therefore, they are especially suitable where a concrete strength at a concrete age less than 28 days is specified.

Where the dimensions of a concrete member are large, cements generating low amounts of heat of hydration should be used in order to avoid thermal cracking. Many slowly hardening cements containing higher contents of constituents other than Portland cement (CE II, CE IV according to clause d.4.2.1) satisfy this requirement.

Low ambient temperatures during casting may require rapid hardening cements whereas low heat cements are advantageous where the ambient temperatures during casting are high.

Special cements may be required for certain exposure conditions such as sulphate resistant cements if the concrete is exposed to water with a sulphate content exceeding approx. 600 mg/kg or to soil with a sulphate content exceeding approx. 3000 mg/kg. Portland cements with a C_3A content less than approx. 3% by mass as well as blast furnace slag cements with a slag content > 65% are generally considered sulphate resistant. Also some Portland composite cements may have a high resistance to sulphates even if their C_3A content is higher than 3%. Although seawater may contain a substantial amount of sulphates, the use of sulphate resistant cements is not mandatory for concrete exposed to seawater.

In cases where the use of reactive aggregates cannot be avoided, special cements with a low content of Na_2O and K_2O result in more durable concrete.

The effectiveness of air-entraining agents in protecting concrete against freezing, thawing and de-icing agents depends to some extent on the type of cement. For reinforced concrete members exposed to de-icing agents which contain chlorides, the resistance of the concrete to the penetration of chlorides is of significance which may be influenced also by the type of cement. Concrete made of blast furnace cements (CE III) may have a particularly high resistance to the penetration of chlorides; however, for high slag contents their scaling resistance may be reduced.

It is also worth noting that, other parameters being equal, i.e. water/cement ratio, cement content and curing, concretes made with cements having substantial amounts of constituents other than Portland cement (CE II–CE IV) carbonate at a higher rate than concretes made of Portland cement (CE I). This effect can be offset by the use of lower water/cement ratios or more intensive curing.

Depending on the type and strength class of the cement more or less intensive curing of the concrete may be required. Many cements containing larger amounts of constituents other than Portland cement clinker (CE II–CE IV) are generally more sensitive to curing than most Portland cements of the same strength class. However, after prolonged curing or in a humid climate their pore structure is particularly dense and impermeable resulting in concretes of high durability (also refer to section d.12).

For these reasons not all types of cement should be used for all conditions to which a concrete member may be exposed. National standards which are based on local experiences with certain types of cement should, therefore, be followed. This is particularly true for cements used for prestressed concrete structures.

2. Additions

Most additions with hydraulic properties, Type II according to d.3 (11), may influence the durability of concrete in a way similar to that of additional constituents of cements as described in the preceding sections.

National standards or regulations may permit consideration of some additions as part of the total cement content and taking them into account when calculating an effective water/cement ratio $(w/c)_{ef}$

$$(w/c)_{ef} = \frac{w}{c + \alpha f}$$

where

w is the water content
c is the cement content
f is the content of addition
α is the efficiency coefficient.

The efficiency coefficient α is not a unique value even for a given type of addition. It may depend on concrete age, amount of the addition f, type of cement, the particular combination of addition and cement, and may be different when considering the effect of water/cement ratio on strength or on durability.

For fly ash obtained from the flue gases of furnaces fired with pulverized anthracite or bituminous coal, values of α may range from $\alpha \cong 0$ to $\alpha \cong 0.5$ for durability considerations. As long as the ratio of fly ash content to cement content $f/c < 0.15$ $\alpha \cong 0.4$ may apply for strength considerations. For some silica fumes values of α up to 0.7 may be valid.

The total amount of pozzolanic additions has to be limited since, particularly at higher ages, such additions may react with the calcium hydroxide formed by the hydration of Portland cement clinker. This may impair the alkalinity of the concrete and thus its ability to protect embedded steel against corrosion. Where fly ash or natural pozzolans are used, the content of Portland cement clinker given as a percentage of the sum of cement and additions should not be less than 60%. For cements containing Portland cement clinker and ground granulated blast furnace slag as the only main constituents, the content of clinker should not be less than 20%.

The content of silica fume should not exceed 15% of the content of cement.

Currently, CEN-Standards are under preparation in which requirements to be satisfied by additions are specified.

3. Aggregates

Aggregates should satisfy the rules set out in subsection d.4.3.

Where the use of reactive aggregates is inevitable, one or a combination of the following precautions should be taken

- limit the total alkali content of the concrete mix
- use a cement with a low content of effective alkali
- limit the degree of saturation of the concrete, e.g. by impermeable membranes.

For further details the requirements of national standards or regulations should be followed taking account of previous long term experience existing with a particular combination of cement and aggregate.

d.6.3.4. Cement content and water/cement ratio

So long as concrete cannot be characterized in terms of durability classes as indicated in subsection d.5.3, durability of a particular concrete may be related to its strength class or its composition, in particular type of cement, minimum cement content and maximum water/cement ratio. In addition, minimum strength classes may be required for a given exposure class.

To ensure a high resistance to the penetration of aggressive substances, the water/cement ratio should be the lower, the more severe the exposure of the concrete member. A minimum cement content should be maintained in order to guarantee the alkalinity of the concrete required for corrosion protection of embedded steel reinforcement and to ensure workability of the fresh concrete for a given water/cement ratio. Recommended values of minimum cement content and maximum water/cement ratios for different exposure classes together with other requirements are given in Table d.2.

The values for minimum cement content given in Table d.2 apply for concrete with a maximum aggregate size of 32 mm. The amount of cement paste required for a given workability of the fresh concrete, generally decreases as the maximum aggregate size increases. Therefore, for concretes with a maximum aggregate size larger than 32 mm, values of the minimum

Table d.2. Durability requirements related to environmental exposure (based on ENV 206, Table 3)

	Exposure class according to Table d.1								
	1	2a	2b	3	4a	4b	5a	5b	5c†
Max. w/c ratio* for									
plain concrete	–	0.70	–	–	–	–	–	–	–
reinforced concrete	0.65	0.60	0.55	0.50	0.55	0.50	0.55	0.50	0.45
prestressed concrete	0.60	0.60	–	–	–	–	–	–	–
Min. cement content* in kg/m^3 for									
plain concrete	150	200	200	–	–	–	200	–	–
reinforced concrete	260	280	280	300	300	300	280	300	300
prestressed concrete	300	300	300	–	–	–	300	–	–
Min. air content of fresh concrete in % for nominal max. aggregate size of:‡									
32 mm	–	–	4§	4§	–	4§	–	–	–
16 mm	–	–	5§	5§	–	5§	–	–	–
8 mm	–	–	6§	6§	–	6§	–	–	–
Frost resistant aggregates	–	–	Yes	Yes	–	Yes	–	–	–
Impermeable concrete according to d.6.6.1	–	–	Yes	Yes	Yes	Yes	Yes	Yes	Yes

* Max w/c may be replaced by $(w/c)_{cf}$ and min. cement content may be replaced by $(c + \alpha f)$ where national standards or regulations allow. Refer to clause d.6.3.3.2.
† In addition, the concrete should be protected against direct contact with the aggressive media, e.g. by coatings.
‡ With a spacing factor of the entrained air void system $\leqslant 0.20$ mm.
§ In cases where the degree of saturation is high for prolonged periods of time, e.g. for structures in the tidal zone, parts of locks, etc.

cement content lower than those given in Table d.2 apply, whereas higher values are required for concretes with a maximum aggregate size smaller than 32 mm.

The values for maximum water/cement ratio, w/c, and minimum cement content, c, given in Table d.2 are mean values. According to ENV 206 for maximum w/c conformity is assumed if the mean value of w/c is not higher than the values given in Table d.2, and if single values do not exceed these values by 0.02. For minimum c conformity is assumed if the mean value of c is not less than the values given in Table d.2 and if single values are not lower than 5% by weight of the specified value.

The durability requirements laid down in Table d.2 apply to conventional concretes. For some prefabricated members or for high strength concretes, deviations from these requirements may be justified; e.g. for high strength concrete to be used within exposure classes normally requiring entrained air this provision may not be necessary, provided adequate frost resistance of the particular concrete is documented by experiments.

d.6.3.5. Maximum permissible quantity of deleterious substances in the concrete

The durability of the concrete and the protection afforded to embedded reinforcement can be adversely affected by any excess of deleterious substances in the components of the mix. Generally applicable limits cannot be quoted, because the permissible amount of harmful substances depends on the particular use of the concrete, on the ambient conditions to which the concrete is exposed and on the type and composition of all concrete constituents.

Excessive amounts of sulphate and free magnesium oxide in the constituent materials can lead to deterioration by expansion cracks. Free chloride ions Cl^- in the concrete may be particularly harmful in relation to corrosion protection of embedded reinforcement. Experience has shown that a content of free chlorides exceeding 0.35 to 1.0% of the weight of cement in the concrete may cause corrosion. Even lower chloride contents may be harmful, e.g. in carbonated concrete or for prestressing strands. Therefore, expert opinion should be obtained in cases where the acceptable limit of chlorides is in doubt. Also refer to 'Durable Concrete Structures—CEB Design Guide'.

Also national and international standards provide information; e.g. ENV 206 gives limits on the Cl^- content by weight of cement, determined in accordance with EN 196, Part 21, of 1.0% for plain concrete, 0.4% for reinforced concrete and 0.2% for prestressed concrete.

d.6.4. Requirements for workability of fresh concrete

Workability of the fresh concrete i.e., its coherence and its placing, compacting and finishing properties may be expressed in terms of its consistence. The level of consistence must be such that the fresh concrete is easily workable without becoming segregated and that it can be fully compacted with the equipment available.

The consistence required for a particular application depends on the size of the structural member, the presence and spacing of reinforcement, the available equipment for compaction and the environmental conditions.

The consistence of the fresh concrete depends on the water content, the fineness and the quantity of the fines, and on the grading and the nature of the aggregates. It can be influenced by certain admixtures and additions.

To ensure proper compaction of concrete cast in situ, it is recommended that the consistence of the concrete at the time of placing is high, e.g. classes S3 or F3 as defined in clause d.7.1.1 unless other measures are taken.

For special applications, such as for some production procedures of precast concrete elements, very dry concrete mixes–zero-slump concretes—not covered by the consistency ranges given in clause d.7.1.1 may be particularly suitable.

Attention should be paid to a possible reduction of consistence of the fresh concrete soon after mixing. It may occur in a dry environment and at high ambient temperatures or when certain cements or admixtures, such as high range water reducing superplasticizing admixtures, are used.

The consistence may be measured by means of various standard tests which are given in clause d.7.1.1.

d.6.5. Mix design

d.6.5.1. Grading of aggregates

1. General considerations

The aggregate for concrete consists of a mixture of particles of different sizes which are combined in accordance with certain requirements. In order to achieve the required or prescribed grading and to keep it within acceptable limits, the aggregate in the concrete should be composed from materials having a narrower range of particle sizes.

The grading of individual materials and of the combined aggregate is established with test sieves on the basis of national or international standards. ISO 565 proposes the following sieve sizes: 0.125, 0.25, 0.5, 1.0, 2, 4, 8, 16, 31.5 and 63 mm.

The principal aspects concerning overall aggregate gradings are as follows.

(*a*) Optimum cement content and low water demand. In this respect, it is advantageous and useful to have aggregate gradings with a relatively low sand content, a high proportion of coarse particles and a small amount of interstitial voids.

(*b*) The concrete should not become segregated during handling, placing or compaction, should be sufficiently workable to be compacted with the available equipment and should attain a close textured surface after finishing. To meet this requirement, an optimum content of fines is needed, this requirement being partly in conflict with clause (*a*). The optimum grading will depend on the handling, placing, compacting and finishing conditions, on the type of structure and on the properties of the aggregates to be used (e.g. particle shape, surface texture and available gradings).

(*c*) The maximum particle size depends on the dimensions of the concrete member, the thickness of the concrete cover, the spacing of the reinforcing bars, the handling and placing conditions, and sometimes, in the case of lightweight aggregate concrete, on the strength class. It should not exceed one-third of the smallest dimension of the member and should generally be less than the spacing of the bars and their distance from the shuttering. It is recommended to use a nominal maximum particle size which corresponds to one of the mesh sizes specified in the applicable national or international standard.

The grading of aggregates can be represented by grading curves. It may also be sufficient to specify only the ratio of fine to coarse aggregate. In this case, the grading of the fine and the coarse aggregate should comply with certain requirements. In all cases, the essential criterion for the composition of the aggregate mix is the absolute volume and not the weight of the different grading groups.

An aggregate may be continuously graded or gap graded. A continuously graded aggregate includes all particle sizes from the finest to the largest. A gap graded aggregate is one in which one or more of the intermediate grading groups are missing.

2. Continuous grading curves

Figures d.2 to d.5 show examples of grading curves. For natural sand and gravel gradings in zone 3 are preferred although gradings in zone 4 are also acceptable. Aggregates in zone 1 tend to produce harsh concretes which are difficult to work and liable to bleed: these difficulties can be minimized by blending the aggregate with other aggregates, if available, or by the use of air entrainment or additions (refer to clause d.6.5.1.4).

Aggregates in zone 5 may require large amounts of water and in consequence high cement contents for a given water/cement ratio to achieve a

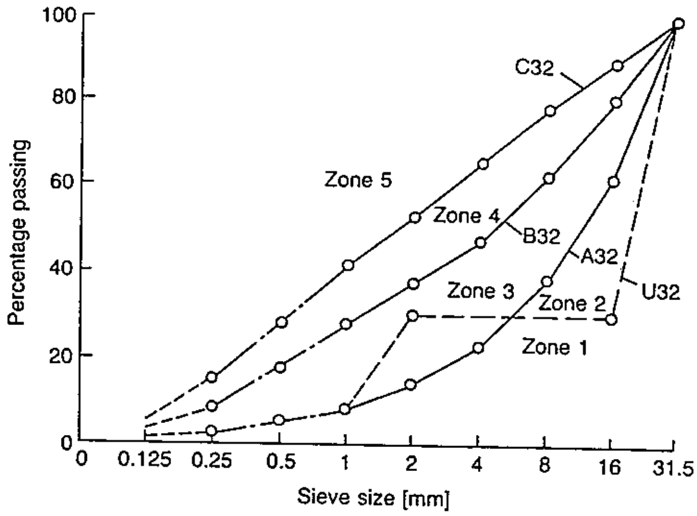


Fig. d.2. Examples of grading curves for a maximum aggregate particle size of 8 mm

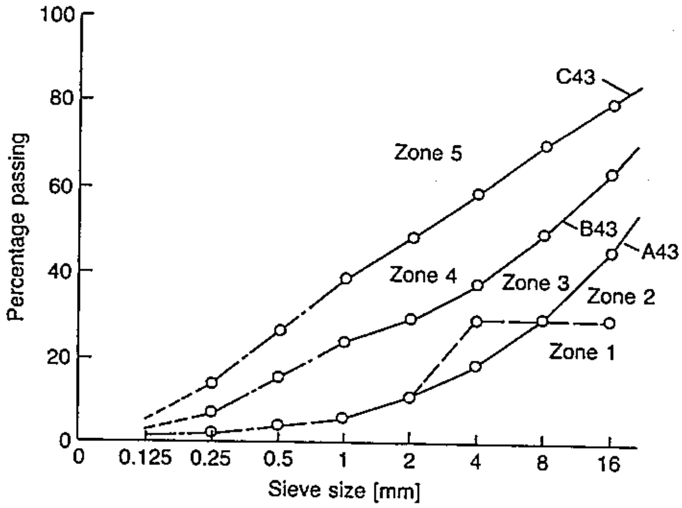


Fig. d.3. Examples of grading curves for a maximum aggregate particle size of 16 mm

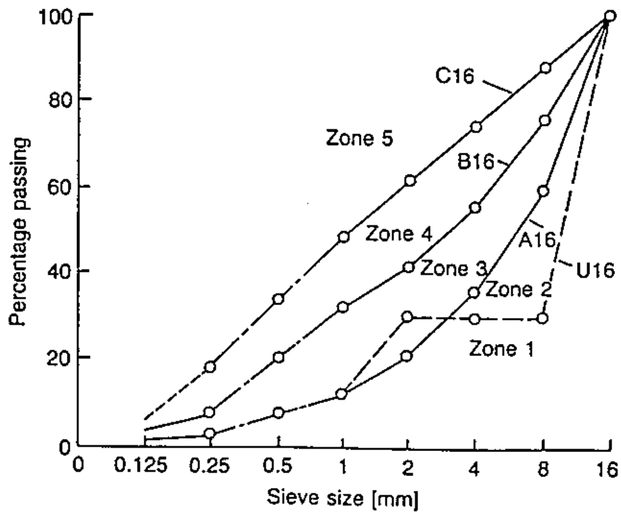


Fig. d.4. Examples of grading curves for a maximum aggregate size of 32 mm

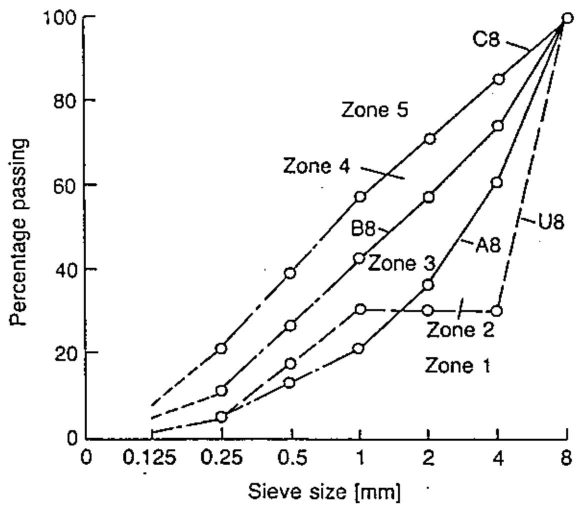


Fig. d.5. Examples of grading curves for a maximum aggregate size of 63 mm

given strength. This can be modified by blending the aggregates with other aggregates if available.

Slightly different rules may have to be applied for crushed sand and crushed coarse aggregates.

3. Gap graded mixes

Gap graded aggregates with grading curves between the grading curves (U) and (C) in Figs. d.2 to d.5 are generally suitable.

4. Content of ultrafine material

Ultrafine material comprises all components having particle sizes smaller than 0.25 mm including the cement and the very fine particles in the aggregate and in the additions, if any. Particles smaller than 0.125 mm in size are particularly effective.

In order that concrete may be cohesive, making it easier to work, it should contain a sufficient quantity of ultrafine material. This is especially important for concrete which is to be transported over long distances or conveyed through pipes, for concrete in thin walled structural components with closely spaced reinforcing bars and for concrete having a high resistance to water penetration (refer to clause d.6.6.1).

If the cement content is low and if the sand contains only small amounts of ultrafine particles very fine grained mineral substances, non-deleterious for concrete may be added. To verify that the ultrafine particles do not contain harmful amounts of fine substances such as clays, organic material or iron hydroxides they should be tested according to national standards, e.g. the methylene blue test or the sand equivalent test. Entrained air tends to reduce the necessary amount of ultrafine material. As a guidance it may be assumed that approx. 10 cm^3 of entrained air replace approx. 5 cm^3 by absolute volume of ultrafines.

The optimum amount of ultrafines depends on the grading of the aggregates. However, the following aspects should be taken into account.

- Too high an amount of ultrafines will reduce the workability of the fresh concrete unless water reducing or high range water reducing superplasticizing admixtures are used.
- A replacement of cement by ultrafines beyond the limits given in clause 6.3.3.2 can adversely affect concrete durability.
- If the addition of ultrafines allows a reduction of cement content without an increase of the water/cement ratio, shrinkage and creep of the concrete are reduced.

d.6.5.2. Cement content and water/cement ratio

In many instances the water/cement ratio of a mix is governed by the strength requirements as indicated in subsection d.6.2. Even if the strength requirement would allow a higher water/cement ratio it should not exceed the values given in Table d.2 for different exposure conditions.

The cement content is controlled by the water demand of the aggregates to achieve a certain consistence of the fresh concrete together with the water/cement ratio. Where no previous experience for the water demand of a given type and grading of aggregates is available it has to be determined by means of trial mixes. In no case should the cement content fall below the minimum values given in Table d.2, taking into account the first footnote.

d.6.5.3. Admixtures and additions

Admixtures and additions should only be added to the mix in such quantities that they do not reduce the durability of the concrete and do not cause corrosion of the reinforcement.

Therefore, as a guide the total amount of admixtures, if any, should not exceed 50 g/kg or 50 cm^3/kg cement. On the other hand, the amount of each admixture added should not be less than 2 g/kg cement in the mix unless it is dispersed in part of the mixing water prior to adding it to the mix. This is necessary to ensure uniform distribution of the admixtures within the mix.

Liquid admixtures in quantities exceeding 3 dm^3/m^3 of concrete should be taken into account when calculating the water/cement ratio.

A certified product declaration based on tests and including instructions for the admixture used should be available at the place where the concrete is produced.

The total amount of additions, particularly of highly reactive pozzolans should be limited as indicated in clause d.6.3.3.2.

d.6.6. Concrete with special properties

The general requirements regarding concrete composition to meet both strength and durability requirements have been dealt with in subsections d.6.2 and d.6.3. In the subsequent paragraphs additional information is given on the production and composition of concrete subjected to special service conditions. In this context also refer to the information given in 'Durable Concrete Structures—CEB Design Guide'.

d.6.6.1. Concrete with high resistance to water penetration

The resistance of concrete to water penetration may be tested e.g. in accordance with ISO 7031 (refer to clause d.7.2.5).

The resistance of fully compacted concrete against penetration and saturation by water depends primarily on the porosity of the hardened cement paste. The water/cement ratio should, therefore, be low and should not exceed 0.55 in structural components with a thickness of 100–400 mm or 0.60 in thicker components. In addition, careful curing is required.

d.6.6.2. Concrete with high resistance to freezing and thawing

Concrete which is critically saturated will be damaged by frequent cycles of freezing and thawing. Such damage can be prevented by limiting the likelihood of critical saturation i.e. low water/cement ratio and sufficient curing, and by the use of air-entraining agents.

In Table d.2 minimum values of the air content of fresh concrete are given for concretes made of aggregates with different maximum sizes. The effectiveness of an air-entraining agent is characterized by the spacing-factor, i.e. the largest distance of a point within the cement paste from an air void and by the microporosity of the cement paste. The spacing factor should not exceed 0.20 mm. The microporosity, which may be defined as the content of pores smaller than 300 μm, should not be less than 1.5%. In addition, the frost resistance of the aggregates should be checked by suitable tests.

d.6.6.3. Concrete with high resistance to freezing and thawing and de-icing chemicals

The effects of freezing and thawing of concrete generally become more severe under the simultaneous action of freezing and thawing and de-icing chemicals.

Under such conditions the use of air-entraining agents is vital, and lower values of the water/cement ratio should be used (refer to Table d.2).

Some cements with a low percentage of Portland cement clinker have had an unsatisfactory service record in some countries when used for concretes exposed to de-icing salts unless low water/cement ratios and very good curing prevented critical saturation of the concrete. Therefore, such cements should be used cautiously under such conditions, and local experience as well as national standards and regulations should be observed.

d.6.6.4. Concrete with high resistance to chemical attack

The severity of chemical attacks depends mainly upon the nature of the aggressive substances (chemical composition and whether solid, fluid or gaseous), their pressure and their rate of flow and the ambient temperature. In the case of solids or gases the presence of moisture is of importance.

It is often difficult to judge whether a chemical attack will occur or not and to choose the necessary precautions. Therefore, in case of doubt, it is recommended that an expert opinion is obtained.

Generally, distinction can be made between three types of chemical attack

- leaching of soluble constituents from the concrete
- chemical reactions with formation of soluble compounds
- chemical reactions, where the products occupy a larger volume than the reacting constituents of the concrete, thus damaging the concrete by their expansion.

The severity of the attack of various substances in water and soils may be assessed from Tables d.3 and d.4, respectively. There, distinction is made between three degrees of severity

- slightly aggressive (exposure class 5a according to Table d.1)
- moderately aggressive (exposure class 5b according to Table d.1)
- highly aggressive (exposure class 5c according to Table d.1).

Generally, the chemical attack is more severe at higher temperatures, at higher pressure or if the concrete is exposed to mechanical abrasion due to fast flowing water or to alternating freezing and thawing. The severity can be assumed to be lower if the temperature of the water is consistently low, if the quantities of water are small or if the rate of flow of the water is slow such as in soils with a permeability coefficient $k > 10^{-5}$ m/s.

The resistance of concrete to chemical attack depends primarily on its impermeability. Therefore, the maximum water/cement ratios given in Table d.2 should not be exceeded. In addition, particularly careful curing is necessary. In cases of exposure to highly aggressive chemicals, additional protective measures such as impermeable coatings are required.

For a sulphate content exceeding 600 mg/kg in water and 3000 mg/kg in soil, the use of sulphate resistant cements is mandatory.

In soils with high contents of both sulphates and chlorides, attention should be paid to the observation that concrete made of sulphate resistant

Table d.3. Limiting values of deleterious substances in water of predominantly natural composition for the assessment of the severity of chemical attack

	Degree of severity*		
	Slight	Moderate	High
1. pH value	6.5–5.5	5.5–4.5	< 4.5
2. Carbonic acid dissolving lime (CO_2) in mg/dm^3 determined by marble test according to Heyer	15–40	40–100	> 100
3. Ammonium (NH_4^+) in mg/dm^3	15–30	30–60	> 60
4. Magnesium (Mg^{2+}) in mg/dm^3	300–1000	1000–3000	> 3000
5. Sulphate (SO_4^{2-}) in mg/dm^3	200–600	600–3000	> 3000

*The degree of severity is valid for stationary or slowly moving water under moderate climatic conditions (e.g. Central Europe). It depends upon the highest degree of severity even if it is reached only by one of the five criteria listed in the lines 1 to 5. If two or more values are found to lie in the upper quarter of a range (in the case of the pH value in the lower quarter) then the degree of severity is deemed to be raised by one step. This increase does not, however, apply to seawater.

Table d.4. Limiting values for the assessment of the deleteriousness of soils

	Degree of severity*	
	Low	Medium
1. Degree of acidity according to Baumann-Cully	> 20	–
2. Sulphate (SO_4^{2-}) in mg/kg according to Baumann-Cully	2000 to 5000	> 5000

*The degree of severity is valid for soils which are saturated at frequent intervals. It can be reduced with decreasing permeability of the soils. If the content of sulphur from sulphides exceeds 100 mg S^{2-} per kg of air-dry soil (more than 0.01% S^{2-}) or if dumps of industrial waste products are concerned, the deleteriousness should be assessed by an expert.

Portland cements generally has a lower resistance to the penetration of chloride ions, than concrete made of blast furnace slag cements with a high slag content.

In spite of the high sulphate content of seawater, the use of sulphate-resisting cement for structures exposed to seawater is not required in some countries because, provided that it is sufficiently impermeable, concrete has proved to have adequate resistance to the effects of seawater.

Apart from these general recommendations, the resistance of concrete to chemical attack and the precautionary measures to be taken should be examined in detail in every country on the basis of local experience and technology. In addition, reference is made to 'Durable Concrete Structures —CEB Design Guide'.

d.6.6.5. Concrete with high resistance to wear

Concrete exposed to highly abrasive conditions shall be adequately resistant to wear. Such an attack can occur, for example, as a result of heavy traffic, impact or sliding loads caused by poured material or of the rapid flow of water containing solids such as sand or ice.

Concrete for use under these conditions should have a strength not less than C30; high strength concrete proved to have a particularly high abrasion resistance. The fine aggregate should consist mainly of quartz or of materials of at least the same hardness; the coarse aggregate should consist of stone or of artificial material having a correspondingly high abrasion resistance. In the surface directly exposed to the abrasive forces, the aggregates shall be tightly packed, and the coarse particles must be solidly held in the mortar to prevent their being torn out. It is recommended that the aggregate should include a high proportion of coarse aggregate and have a grading which enables the grains to be closely packed with few intergranular voids (e.g. near the grading curve A or, in the case of gap graded mixes, between B and U in Figs. d.2 to d.5). The surface texture of the aggregate particles should be fairly rough and the shape of the particles should not be elongated.

At the surface of the concrete, bleeding should be minimized, and there should not be a marked laitance layer. It is, therefore, preferable to use concrete of stiff-plastic to stiff consistence or, alternatively, to undertake a special additional treatment of the concrete surface. Such measures could be applied when the concrete is in either the fresh or in the hardened state. In the fresh state, it is possible to apply vacuum treatment according to subsection d.11.4 or to generate additional compaction by power trowelling. Spraying in a dry cement-sand mix may cause surface cracking.

To ensure that the surface of the concrete is not friable, the concrete should be cured for a sufficient period i.e. at least twice as long as proposed in section d.12.

Where the abrading action is particularly severe, the provision of a special wearing surface is advisable (e.g. partial or total replacement of the aggregate by silicon carbide or carborundum).

In the hardened state, the weaker surface zone may be removed by grinding, especially for interior floors which are not exposed to frost action.

d.6.6.6. Concrete subjected to elevated temperatures

Concrete should not be exposed for prolonged periods to temperatures higher than 250°C, otherwise there may be a serious loss of strength and harmful effects on other important concrete properties. High temperatures below 250°C can have an influence on certain properties, such as creep, shrinkage or the modulus of elasticity as indicated in section 2.1 of this Model Code. Only aggregates should be used which have proved to be suitable for concrete exposed to elevated temperatures. Where concrete is exposed to high temperatures, it is important to make sure that the concrete attains a high degree of hydration, and has dried out before it is heated for the first time. Rapid changes of temperature should be avoided.

Special concretes are required in conditions of prolonged exposure to temperatures in excess of 250°C.

d.7. Verification of concrete properties

d.7.1. Fresh concrete

d.7.1.1. Consistence

The consistence of fresh concrete may be determined by means of standard tests such as the slump test (ISO 4109), the Vebe test (ISO 4110), the compaction test (ISO 4111) or the flow table test (ISO 9812). The classifications obtained from the various test methods are given in Tables d.5 to d.8.

Table d.5. Slump classes according to ISO 4109

Class	Slump in mm according to ISO 4109*
S1	10–40
S2	50–90
S3	100–150
S4	> 160

* The slump measured is to be rounded off to the nearest 10 mm.

Table d.6. Vebe classes according to ISO 4110

Class	Vebe seconds according to ISO 4110
V0	> 31
V1	30–21
V2	20–11
V3	10–5
V4	< 4

Table d.7. Compaction classes according to ISO 4111

Class	Degrees of compactability to ISO 4111
C0	> 1.46
C1	1.45–1.26
C2	1.25–1.11
C3	1.10–1.04

Table d.8. Flow classes according to ISO 9812

Class	Flow diameter in mm according to ISO 9812
F1	< 340
F2	350–410
F3	420–480
F4	490–600

The slump test is not the most suitable test for non-cohesive concrete or for concrete with a dry consistence. If the slump is less than 10 mm the test result should be recorded as less than 10 mm. It should also be stated that the concrete has a consistence stiffer than that for which the slump test is suitable.

For concrete of fluid consistence, e.g. when high range water reducing admixtures are used, the flow table test gives the most reliable results.

Since there is no generally valid relationship between the different methods of measuring consistence, the method should be agreed upon in each particular case.

d.7.1.2. Air content

The air content of the fresh concrete may be determined by the pressure method, e.g. according to ISO 4848.

d.7.1.3. Wet density

Where a determination of the wet density of the compacted fresh concrete is required it may be measured e.g. according to ISO 6276.

d.7.1.4. Water/cement ratio and cement content

Currently, no universally applicable methods are available to determine water/cement ratio and cement content of the fresh concrete on the site within a sufficiently short period of time unless additional information such as type of cement, content of ultrafines etc. is available.

The cement content has to be controlled by careful measurement of the weight of cement added to the mix. The water/cement ratio has to be calculated from the weight of water added to the mix taking into account the water content of liquid admixtures and the effective water content of the aggregates, and from the weight of cement.

Also the 28-day compressive strength may be used to verify the water/cement ratio if relations between compressive strength and water/cement ratio are available for a particular concrete mix.

d.7.2. Hardened concrete

d.7.2.1. Compressive strength

Concrete compressive strength as defined in subsection d.5.2 should be determined on cylinders, diameter 150 mm, height 300 mm, which have been stored in water at 20 ± 2°C up to the time of testing at an age of 28 days and which are tested in accordance with ISO 4012. If national standards require different types of specimens or different storage conditions up to the time of testing, conversion factors which have been verified by tests have to be applied. In clause 2.1.3.2 of this Model Code conversion factors are given which relate the strength of cubes 150/150/150 mm to the strength of cylinders 150/300 mm made of normal weight concrete. For lightweight aggregate concretes other conversion factors may be valid.

d.7.2.2. Tensile strength

The tensile strength of concrete may be determined by various methods

axial tensile strength	according to RILEM CPC 7
splitting tensile strength	according to ISO 4108
flexural tensile strength	according to ISO 4013

In clause 2.1.3.3.1 of this Model Code relations are given to estimate upper and lower limits of the tensile strength from compressive strength as well as relations between the results of the three test methods to determine the tensile strength. If different test methods are used, other correlations have to be used.

d.7.2.3. Strength development

The development of strength with time may be verified by testing concrete specimens according to clause d.7.2.1 at suitable time intervals. If the strength development of the concrete in the structure is to be verified, curing conditions simulating site exposure and deviating from those specified in clause d.7.2.1 should be employed.

d.7.2.4. Resistance to abrasion

Currently, there exists no international standard to determine the abrasion resistance of concrete. Therefore, testing should be done according to national standards or regulations.

d.7.2.5. Resistance to water penetration

The resistance of concrete to water penetration may be tested, for example, according to ISO 7031. Currently, few data are available to relate the results of experiments carried out in accordance with ISO 7031 to the long term performance of structures. If, deviating from the curing method specified in ISO 7031, the specimens are stored in water up to the time of testing, concrete may be considered to have a high resistance to water penetration if the maximum values of penetration are less than 50 mm and if the average penetration is less than 20 mm.

d.7.2.6. Density

The oven-dry density of hardened concrete is used for the classification of concrete by density. The apparent density may be determined, e.g. according to ISO 6275.

d.8. Specification of concrete

This Model Code distinguishes between two modes of concrete specification

- designed concrete mix, section d.3, definition (21)
- prescribed concrete mix, section d.3, definition (22).

d.8.1. Designed concrete mix (C I)

d.8.1.1. General requirements

The composition of designed mixes may be based on previous average production data for the materials and of the plant producing the concrete over a period exceeding at least one month and not exceeding one year. In the absence of suitable previous data, trial mixes should be made. Information on the composition of designed mixes should be made available to the user on request. The producer of the concrete is responsible for its composition, which must be such that the fresh concrete is sufficiently workable, does not segregate and can be virtually fully compacted. The resulting concrete, when hardened, must comply with the specification—usually a prescribed compressive strength. For the trial mixes, therefore, a sufficiently high strength margin should be obtained in the tests. Furthermore, the concrete must be durable and afford adequate protection to the reinforcement against corrosion. To achieve this, its composition should be in accordance with the instructions given in this appendix.

d.8.1.2. Data for specifying designed mixes

The specification of designed mixes should contain the following basic data

(*a*) strength class
(*b*) nominal maximum size of aggregate
(*c*) consistence of the fresh concrete
(*d*) limitations for the composition depending on the use of the concrete according to Table d.2 (e.g. plain concrete, reinforced or prestressed concrete and environmental conditions).

Additional data to be verified in trial tests may be specified if required for special conditions such as

(*a*) characteristics of the hardened concrete, e.g.

(i) density
(ii) high resistance to water penetration
(iii) resistance to freezing and thawing
(iv) resistance to chemical attack
(v) resistance to abrasion
(vi) resistance to high temperatures

(*b*) characteristics of the composition, e.g.

(i) type of cement
(ii) air content
(iii) accelerated strength development
(iv) heat development during hydration
(v) retarded hydration
(vi) special requirements for aggregates
(vii) special requirements concerning resistance to alkali-silica reaction
(viii) special requirements for the temperature of the fresh concrete.

d.8.2. Prescribed concrete mix (C II)

d.8.2.1. General requirements

In this case, the structural designer and/or the user assume responsibility for the composition of the concrete. They should check the suitability of the concrete by tests on trial mixes, unless adequate knowledge is available from previous use of similar mixes.

The trial mixes are governed by the same criteria as those which apply to the production of designed mixes (C I).

d.8.2.2. Data for specifying prescribed mixes

The specification of prescribed mixes should contain at least the following basic data

- cement content per cubic metre of compacted concrete
- cement type and strength grade
- consistence class or water/cement ratio of the fresh concrete
- type of aggregate
- nominal maximum size of aggregate
- type and, if necessary, quantity of any admixture or addition.

Additional data may be specified such as

- data with regard to concrete composition
  - — additional requirements for aggregates including any special grading
  - — air content of fresh concrete
  - — special requirements regarding the temperature of the fresh concrete on delivery
- data with regard to the transportation and the procedures on site
  - — delivery rate
  - — limitation of type (agitating/non-agitating equipment), size or height of transport vehicle.

d.9. Batching and mixing of fresh concrete

d.9.1. Batching

For the batching of the basic materials the accuracies given in Table d.9 are recommended. These values apply for the total batch volume.

To achieve these accuracies for batching the measuring equipment has to be sufficiently accurate and comply with the relevant national standards or regulations.

When batching cement, the required accuracy can usually be achieved only by weighing. Alternative systems may be used, provided their reliability has been proved in practice, and the accuracy of measurement, when checked by field trials, is within the required tolerances.

Generally, aggregates should also be weighed. (For the batching of lightweight aggregates see section d.16.) If batching is carried out by volume, it is necessary to confirm frequently that the dry weight of the measured particle groups is accurate. In particular, it is necessary to carry out frequent checks and make corresponding corrections on aggregates containing variable amounts of moisture because the bulk density (unit weight) particularly of the finer particle groups will vary considerably with the water content.

Table d.9. Recommended accuracies for batching of constituent materials

Constituent	Accuracy
Cement Water Total aggregate Additions	±3% of required quantity
Admixtures	±5% of required quantity

The added water can be batched by weighing or by volume. The largest permissible quantity of water depends on the required consistence. Water shall not be added to the concrete after it has left the mixer. The cement content of concrete, for which limits of the water/cement ratio are recommended (e.g. according to Table d.2), shall be sufficient to ensure that this condition is also satisfied. In this case, account shall be taken of moisture on the surface of the aggregates, the amount of water absorbed prior to placing the concrete and where appropriate, the water content of additions and admixtures if they contain water in noticeable amounts.

Admixtures in the form of powders and additions should always be measured by weight. Fluid or paste-like admixtures and additions can be measured by volume or by weight.

d.9.2. Mixing

Concrete can be mixed either in a stationary mixer or in a truck mixer. Such mixers shall be capable of achieving a uniform distribution of the constituent materials and a uniform workability of the mix within the mixing time and at the mixing capacity.

Truck mixers shall be equipped such that the concrete can be delivered in a homogeneously mixed state. In addition, they have to have suitable measuring and dispensing equipment if mixing water or admixtures are to be added on the site.

Mixing of the various constituents should continue until a uniform mixture is obtained. The duration of mixing depends on the type and composition of the concrete and also on the type and condition of the mixer. It must not be excessive so as to break up aggregates.

The temperature of the fresh concrete before casting should, if possible, not exceed 30°C and should not fall below 5°C in cold weather or frost. Frozen aggregates should be completely thawed before or during mixing. Also refer to sections d.13 and d.14.

If the temperature of the fresh concrete exceeds 30°C (e.g. when using mixing processes with steam or in hot climates) special measures are required to ensure that the concrete can still be completely compacted. Such measures include the use of a retarder or the use of a cement which is particularly suitable for these conditions. The introduction of steam at the mixer necessitates the use of special equipment and special experience.

In principle, the composition of the fresh concrete must not change after leaving the mixer.

d.9.3. Ready-mixed concrete

d.9.3.1. General requirements

When the concrete is delivered its consistence and composition shall be in accordance with the specification. It should not have segregated and must not be at an unacceptably high or low temperature (refer to section d.9.2). Furthermore, it shall remain workable for an adequate period. The period between the addition of the water and the discharge of the transport vehicle, therefore, should not be excessive.

In general, truck mixers or agitators should be completely unloaded not more than 90 minutes and other vehicles not more than 45 minutes after the addition of the mixing water. If accelerated setting of the concrete is likely to occur (e.g. in hot weather), then the period until the completion of unloading should be correspondingly shortened, unless the period in which the concrete can be satisfactorily cast is extended by the use of suitable admixtures.

If the duration of transport is very long, it may be advisable to add the water shortly before delivery. In this case, any liquid admixtures must not be introduced prior to adding the water. However, all batching of water and admixtures at a later stage in transport should be demonstrated to be adequate by trial mixing and trial casting. The period until completion of unloading can be longer if the temperature is low or if retarders are used. Details should be taken from the appropriate national standards or regulations.

Where high range water reducing admixtures are used, the admixtures can only be added shortly before casting because of the limited duration of the fluidifying effect of such admixtures. After adding the admixture, the concrete should be re-mixed until the high range water reducing admixture is uniformly distributed and has become fully effective.

d.9.3.2. Information for user

The user of ready-mixed concrete needs information from the producer of the ready-mixed concrete on the composition of the concrete, to permit proper placing and curing of the fresh concrete as well as to assess the strength development and the durability of the concrete in the structure. This information should be given to the user either beforehand on request or at delivery.

Therefore, before discharging the concrete, the producer of the ready-mixed concrete should provide the user with a delivery ticket for each load of concrete, which contains the following basic information

- name of ready-mixed concrete plant
- serial number of delivery ticket
- date and time of loading, i.e. time of first contact between cement and water
- truck number
- name of user
- name and location of the site
- specification of the concrete as well as details or references to specifications, e.g. code number, order number
- amount of concrete in m^3
- name or mark of the certification body where appropriate.

For a designed mix the delivery ticket should, in addition, contain

- strength class
- exposure class for which the concrete is suitable
- consistence class
- type and strength class of cement
- type of aggregate
- type of admixtures and additions, if any
- special properties of the concrete.

This information may also be provided by reference to the producer's catalogue of concrete mixes which should contain information on strength and consistence class, batch weights and other relevant data.

For a prescribed mix the delivery ticket should give

- details of the composition, e.g. cement content, type of admixtures and additions, if any
- consistence class.

d.10. Handling, placing and compaction of fresh concrete

d.10.1. Handling

The method of handling (e.g. in skips, on conveyor belts or through pipes) and the composition of the concrete have to be co-ordinated in such a way that segregation is prevented.

Whereas practically any concrete can be transported in suitable skips, concrete that is to be pumped through pipes shall satisfy certain requirements as to its mix design and workability. (For pumped concrete refer to subsection d.11.5.)

Concrete moved on a conveyor belt shall also be cohesive. It is preferable to provide ploughs and devices for holding the concrete together at the point of discharge.

Concrete which is cast under water should be composed and handled in accordance with the recommendations given in subsection d.11.2.

d.10.2. Time of placing

Concrete should be placed as soon as possible after mixing or, in the case of ready-mixed concrete after delivery, so as to minimize any reduction in workability and changes in its composition.

d.10.3. Placing

Concrete should not segregate when being placed. Chutes used, when filling tall column or wall forms, should end a short distance above the place where the concrete is deposited in order to keep the concrete together. The concrete should be placed in uniform layers the thickness of which depends on the method and effectiveness of the method of compaction. Dumping the concrete in large heaps and then distributing it with vibrators is not advisable, because the concrete is liable to segregate. To avoid the formation of horizontal layers of laitance the concrete should be deposited in a sufficiently continuous manner to ensure that, while being compacted, it will be completely bonded to the previous layer of concrete.

d.10.4. Compaction

The concrete should be compacted as completely as possible, so that it contains a minimum amount of entrapped air. Depending upon the workability of the concrete and the type of structural component, the concrete may be compacted by vibration, rodding, stamping, hammering of the formwork etc. Concrete of stiff, plastic or semi-fluid consistence should generally be compacted by vibration. The compaction of fluid concrete should be carried out by rodding or, if segregation is not probable, by light vibration.

When vibrators are used vibration should be applied continuously during the placing of each batch of concrete until the expulsion of air practically ceases and in a manner which does not promote segregation.

Particular attention should be paid to ensure that the reinforcing bars are embedded in dense concrete. While placing and compacting the fresh concrete care should be taken to avoid displacement and damage of reinforcement, tendons, ducts, anchorages and formwork.

Concrete which has already been compacted can be improved by revibration at a later time, as long as it is still workable. Revibration tends to close plastic shrinkage and settlement cracks and cavities under horizontal reinforcing bars.

Where a special surface finish is required, this should be specified additionally.

d.10.5. Construction joints

Construction joints are formed in places where the concreting work has to be interrupted for practical reasons. The number of joints should be kept to a minimum since they can have low tensile and shear strengths thus reducing the load-bearing capacity near the joint. Furthermore, there is a risk that faulty workmanship may impair the watertightness of the concrete near the construction joint; this can also reduce the protection of the reinforcement against corrosion.

If possible, construction joints should be located at places where they will not be heavily stressed or where a joint is required for other reasons. Horizontal joints should not be placed in positions intermittently below the water level.

To provide a good bond between the old and the new concrete any laitance on the hardened concrete should be removed before the fresh concrete is cast.

The older concrete, if dry, should be wetted before further concrete is placed. At the time of concreting, however, the older concrete should be somewhat dry on the surface and slightly absorptive, whereas the core should still be wet, i.e. the concrete should be saturated but surface dry.

d.11. Concrete for special manufacturing or casting conditions

d.11.1. Concrete containing a combination of admixtures

For some applications (e.g. for mass concrete, for concrete with a high resistance to freezing and thawing and de-icing chemicals or for concreting at a very slow rate) frequently the simultaneous use of two or more different admixtures is advantageous. Different admixtures may influence each other's effectiveness with regard to the properties of the fresh or hardened concrete, and in some instances may not be compatible, e.g. some high range water reducing admixtures and air entraining admixtures.

Also setting time and strength development may be significantly altered by combinations of admixtures. Therefore, in such cases special knowledge is required, and trial tests with the intended combination of admixtures are mandatory. Furthermore, the manufacturers of the admixtures should be consulted if any doubts exist concerning the compatibility of the admixtures.

d.11.2. Concrete cast under water

Concrete placed under water to form load bearing or impermeable structures should flow as a coherent mass, so that it will attain a dense structure even without compaction and does not segregate.

The following recommendations apply to the composition of concrete cast under water.

(*a*) *Consistence.* A semi-fluid consistence corresponding to the slump classes S3 or S4 in Table d.5 should be used. A slump of about 150 mm or greater is advisable. The optimum consistence also depends on the manner in which the concrete is cast. Concrete intended for pumping, normally should be somewhat stiffer to prevent a blockage of the pipes. Fluid mixes, however, can be pumped satisfactorily if suitable admixtures (high range water reducing admixtures, water retaining admixtures) are added.

(*b*) *Aggregates.* To achieve good workability combined with a low water/cement ratio and close, compacted texture without additional means of compaction, the use of aggregates of rounded or cubic particle shapes and a smooth surface is preferable. In general, continuous gradings are preferred, because the danger of segregation is less than with gap graded mixes. Grading curves near the centre of zone 3 in Figs d.2 to d.5 are suitable while the proportion of fines up to 4 mm in size should lie near line B. Maximum coarse aggregate sizes exceeding approx. 32 mm can lead to difficulties in casting. In some cases a maximum aggregate particle size of less than 32 mm is advisable.

(*c*) *Cement.* Recommendations on the required cement content have not yet been agreed on. High cement contents of about 350–400 kg/m^3 are often recommended in the literature to improve the cohesion of the fresh concrete and thus diminish the danger of loss of cement by leaching, as well as to ensure that an adequate amount of cement remains in the hardened concrete, in spite of the leaching which will inevitably occur. It also has been proposed that the cement content should be limited to 325 kg/m^3 because high cement contents can encourage the formation of wide thermal cracks. If the aggregate does not contain a sufficient quantity of ultrafines, it may be necessary to introduce additional ultrafine material to optimize the cohesion of the fresh concrete. The use of cements other than Portland cements (types CE II–IV) may be appropriate for concrete cast under water, in order to increase its resistance against chemical attack and to reduce heat of hydration.

Unless special admixtures or additions are used which improve concrete cohesion in the fresh state, concrete cast under water should not fall freely through the water. Otherwise it may be leached out and become segregated. The following processes have been particularly effective when casting concrete under water.

(*a*) *Tremie (pipe) method.* The concrete is placed through vertical pipes, the lower end of which is always inserted sufficiently deep into the concrete which has been placed previously but has not set. The concrete emerging from the pipe pushes the material that has already been placed to the side and upwards and thus does not come into direct contact with the water.
(*b*) *Direct placement with pumps.* This is a further development of the tremie method. As in the case of the tremie method the vertical end piece of the pipe line has always to be inserted sufficiently deep into the previously cast concrete and should not move to the side during pumping.
(*c*) *Special mix design.* This includes the use of admixtures or additions which assure sufficient cohesion of the fresh concrete. Under these conditions, the concrete may be freely dropped through the water.

Such methods and concrete compositions may also be used for casting concrete in bentonite.

d.11.3. Shotcrete

Shotcrete, or gunite, is the process by which concrete or mortar is sprayed onto a surface to produce compacted material which is self-supporting. There are two basic systems. In the dry process the pre-mixed dry materials are mixed with the water at the spray gun whereas in the wet process the mixing takes place earlier.

The advantage of shotcrete is that it requires only an inner form (or an existing surface). Therefore it is particularly suitable for the construction of curved sections, for tunnel linings or coatings and for the repair or strengthening of structural elements.

When defining the composition of shotcrete, distinction has to be made between

- the initial mix which corresponds to the original shotcrete mixture before it is transported to the shotcreting equipment
- the jetted mix which is the shotcrete which leaves the nozzle
- shotcrete, which is the material in place; it differs from the jetted mix by the rebound which occurs during jetting.

Careful consideration of the mix design is necessary by the shotcrete contractor to ensure good adhesion, proper compaction and satisfactory properties of the hardened material. The water/cement ratio of the in-place concrete or mortar is usually in the range 0.35–0.5. Maximum aggregate size for sprayed concrete is about 20 mm. The density of the in-place concrete is similar to equivalent, well compacted, conventional concrete.

Because of the high particle velocities involved in this technique, special consideration has to be given to the safety of personnel.

The various steps in producing and placing shotcrete and the substantial differences between the composition of shotcrete and the initial mix require careful control and testing during the various steps of the entire operation.

d.11.4. Vacuum dewatering

Vacuum dewatering is a technique for removing water from the surface of freshly cast concrete. It results in much faster stiffening of the fresh concrete and considerable improvement in the physical properties of the hardened concrete.

Vacuum dewatering is most commonly used in conjunction with power trowelling for the production of smooth, hardwearing floor surfaces. In this application, vacuum dewatering enables the usual delay between casting and surface finishing to be reduced from several hours to less than one hour, which can be a considerable advantage in cold weather.

Vacuum dewatering is particularly useful with concrete of fluid consistence but is not suitable for use with air-entrained concrete.

d.11.5. Pumped concrete

Materials for pumpable concrete should be batched consistently and uniformly, and should be thoroughly mixed. Quality control of the ingredients and of the batching and mixing is essential for smooth-running pumping operations and for good quality of the pumped concrete.

The maximum particle size of angular aggregates should be limited to one-third of the internal diameter of the pipe or hose, and, in the case of well rounded aggregate, to 40% of the pipe diameter.

Adequate and uniform grading of the aggregate during the whole pumping operation is very important. Grading should preferably be continuous and should follow the grading curve B in Figs. d.2 to d.5 as closely as possible. Gap-graded aggregate should be used with caution, since it is prone to segregation.

The content of ultrafine materials should be high enough to provide good cohesion, but the excess may have deleterious effects on the fresh or hardened concrete (see clause d.6.5.1.4). Attention should also be paid to the portion of the fine aggregate passing sieve size 0.25 mm, which should be between 15 and 30%, and to that passing sieve size 0.125 mm, which should not be less than 5 to 10% of the total volume of the sand. If the fine aggregate is deficient in either of these two sizes the above recommendation can be met by blending with selected finer sands or with an adequate type of addition.

Lightweight aggregates should be pre-wetted in order to offset excessive absorption under the pressure exerted by pumping and the stiffening of the concrete during pumping operation.

The required cement content depends on the same basic principles valid for conventionally placed concrete, though slightly higher amounts of cement may be necessary due to the higher consistence classes and the higher amount of ultrafines used for pumping.

Admixtures which increase workability usually improve pumpability by providing additional lubrication, reduce segregation and decrease bleeding.

Air-entraining admixtures also improve workability, but too large a quantity of entrained air may make pumping difficult. Set-retarding admixtures may be used when pumping long distances and in hot weather.

The consistence of fresh concrete for pumping, measured by the slump test, should be in the plastic range i.e. in classes S2 and S3. Too fluid a consistence may produce excessive bleeding, and, therefore, cause blocking in the pump line. Maintaining control of proper and uniform consistence is essential for a smoothly running pumping operation.

The suitability of the concrete mixture should be verified by trial mixes and by performing a pumping test.

Pipes for pumping should not be made of materials having a deleterious effect on concrete; aluminium alloy, for example, may cause the generation of hydrogen bubbles and thus a reduction of concrete strength.

Before starting the pumping operation the pipe line should be lubricated with a properly designed mortar, which should be wasted and not used in the concrete placement, or with a batch of the regular concrete with the coarse fraction omitted.

d.12. Curing and protection

d.12.1. General considerations

In order to obtain the potential properties to be expected from the concrete, thorough curing and protection over an adequate period of time are vital. It is essential that curing and protection start immediately after compaction of the fresh concrete.

In this context curing is a measure to avoid premature drying of the concrete and to provide the cement paste in the concrete with a sufficient amount of water over a sufficiently long period of time to achieve a high degree of hydration within its mass and particularly in its surface layers. In addition, curing includes measures against the effects of sunshine or wind and the prevention of cracking due to early shrinkage.

Insufficient curing often has only a minor effect on the strength development of the concrete in the structure with the exception of thin sections because the core of a thicker concrete section maintains a sufficiently high moisture content for a prolonged period of time even without curing. However, lack of curing is detrimental to the durability of a concrete structure which is primarily controlled by the properties of the surface layers. Premature drying results in a permeable surface layer with little resistance to the ingress of aggressive media.

Protection is a measure against other external effects which may harm the young concrete such as leaching due to rain or flowing water, rapid cooling or freezing, thermal stresses due to heat of hydration, vibration or impact. Protection against freezing is dealt with in section d.13.

d.12.2. Methods of curing

The principal methods for the curing of the concrete are

- keeping the formwork in place
- covering with plastic films
- placing of wet coverings
- sprinkling with water (at temperatures above freezing)
- application of curing compounds which form protective membranes.

These methods may be used separately or in combination.

Not all methods of curing are equally effective. Therefore, it is essential that the effectiveness of the method chosen is controlled e.g. by frequent inspection. In general, those methods where water is added result in a denser pore structure of the concrete than methods which only prevent the concrete from drying. Nevertheless, sprinkling of cold water on a concrete surface which is warm due to the heat of hydration may cause severe temperature stresses and early cracking of the concrete surface layers.

d.12.3. Duration of curing

d.12.3.1. Parameters influencing the duration of curing

The concrete has to be cured until its surface layers are sufficiently impermeable. This generally means that it will also attain the required strength. Therefore, the duration of curing depends on the following.

- *Curing sensitivity of the concrete as influenced by its composition.* The most important characteristics of concrete composition with respect to curing are the water/cement ratio, the type and strength class of cement, as well as type and amount of additions. Concrete with a low water/cement ratio and made of a rapidly hardening cement e.g. R- or RS-cements according to clause d.4.2.1 reach a required level of impermeability more rapidly and, therefore, need less curing than concretes with a higher water/cement ratio and made of cements which hydrate slower such as SL-cement according to clause d.4.2.1 or concretes containing higher amounts of Type II-additions.

  Also the resistance of the concrete to a particular exposure condition as influenced by the type of cement or additions should be taken into account. A lower resistance to e.g. carbonation which is characteristic for some of the blended cements CE II–CE IV according to clause d.4.2.1 can be offset by the choice of a lower water/cement ratio or prolonged curing.
- *Concrete temperature.* Due to the heat of hydration generated by the reaction between cement and water the concrete temperature may increase, thus accelerating hydration. Therefore, the higher the temperature particularly of the surface layers of the concrete, the shorter the required duration of curing. The temperature of the concrete depends on the temperature of the ambient air, strength grade and amount of cement, the dimensions of the structural member and the insulation properties provided e.g. by the formwork. Thus thin concrete sections without thermal insulation exposed to low ambient temperatures during curing and made of cements with a low heat of hydration need particularly careful curing.
- *Ambient conditions during and after curing.* A low relative humidity of the ambient air, sunshine and high winds accelerate drying of the unprotected concrete at an early stage of hydration. Therefore, under such conditions prolonged curing is required, because after termination of curing the surface layers of the concrete dry out rapidly, and hydration will no longer continue. On the other hand when concreting in a humid environment at moderate temperatures curing will at least partially be provided by the surrounding atmosphere.
- *Exposure conditions of the finished structure in service.* The more severe the exposure conditions as given in Table d.1, the longer the required duration of curing.

Thus, an estimate of the required duration of curing is a complex problem. The best approach is to define limiting values of permeability of the concrete surface layers which have to be reached before curing can be terminated. These value should depend on the exposure condition of the structure in service as well as on the type of cement but not on water/cement ratio, strength class of the cement and concrete temperature. At this time neither methods to measure surface permeability nor limiting values of permeability are generally accepted. Therefore, the required duration of curing has to be estimated on the basis of the parameters given above.

Some of these parameters are interrelated particularly with regard to concrete temperature so that a reliable estimate of the required duration of curing necessitates preliminary experiments on the concrete in question and continuous measurements of concrete temperature on site. Such an approach which is based also on theoretical considerations is given in 'Durable Concrete Structures—CEB Design Guide'. Rules of thumb for a quick estimate are given in the following section.

d.12.3.2. Estimates of the duration of curing

In Table d.10 minimum durations of curing are proposed for concrete members subjected to exposure condition 2a; 2b; 4a and 5a according to Table d.1. In Table d.10 distinction is made according to expected ambient conditions during curing and during the period immediately after curing and according to the rate at which the concrete reaches a certain impermeability. This rate depends on the water/cement ratio and strength class of cement as proposed in Table d.11. There, reference is made to the general cement classification given in clause d.4.2.1.

Table d.10. Minimum duration of curing in days for $T > 10°C$ for exposure classes 2a; 2b; 4a and 5a, according to Table d.1

Expected ambient conditions during and after curing	*Rate of development of impermeability of concrete*			
	Very rapid	*Rapid*	*Medium*	*Slow*
I No direct sunshine, relative humidity of surrounding air RH > 80%	*1*	*2*	*3*	*4*
II Exposed to medium sunshine or medium wind velocity or relative humidity RH: 50% < RH < 80%	*2*	*3*	*4*	*5*
III Exposed to strong sunshine or high wind velocity or relative humidity RH < 50%	*3*	*4*	*6*	*8*

Table d.11. Rate of development of impermeability of concrete

Rate of development of impermeability	*w/c*	Class of cement*
Very rapid	0.5–0.6 < 0.5	RS RS; R
Rapid	0.5–0.6 < 0.5	R N
Medium	0.5–0.6 < 0.5	N SL
Slow	All other cases	

* Refer to clause d.4.2.1.

Table d.10 is valid for concretes made of Portland cements (CE I) and for a reference temperature of the concrete of 20°C. For temperatures of the ambient air during curing < 10°C the duration of curing should be increased. Then the required increase of curing time may be determined on the basis of the maturity concept given in clause 2.1.8.2 of this Model Code. As a rough guide for a concrete temperature of 10°C the required duration of curing is about twice the required curing time for a concrete temperature of 20°C. For a concrete temperature of 30°C it is only about half the curing time at 20°C. Therefore, thermal insulation of the concrete during curing may be an effective method to reduce curing times, particularly for concrete made of slowly hydrating cements. In such cases, however, attention has to be paid to thermal stresses which may be developed when the thermal insulation is removed. Refer to subsection d.12.4.

Concretes made of cements containing high amounts of constituents other than Portland cement clinker (CE II 32.5; CE III 32.5; CE IV 32.5) and concretes containing Type II additions in amounts which approach the maximum values given in clause d.6.3.3.2 are more sensitive to curing than concretes made of Portland cements with the same water/cement ratio. Therefore, for such concretes the duration of curing should be increased by 1 to 2 days beyond the values given in Table d.10 if during curing the concrete is exposed to conditions II and III. For condition I the rate of drying after curing is so low, that even the surface layer of the concrete will continue to hydrate for some time after the termination of curing.

Depending on the type and use of the structural element (e.g. the intended finish) the minimum duration of curing given in Table d.10 should also be applied for exposure class 1. Where the concrete is exposed to severe abrasion or to severe environmental conditions (exposure classes 3, 4b, 5b and 5c according to Table d.5) the length of curing given in Table d.10 should be increased by 3 to 5 days depending on the ambient conditions during curing according to Table d.10.

d.12.4. Protection against thermal cracking of surface

The hardened concrete has to be protected against damaging effects caused by heat of hydration generated in the concrete.

Where no cracking is permitted, adequate measures should be taken to ensure that the tensile stresses caused by temperature differences are less than the tensile strength at the time these stresses occur. This generally requires a careful numerical analysis.

To avoid cracking caused by a temperature rise of the concrete at its surface, the temperature difference between the centre and the surface should be less than 20°C. Details on measures to prevent cracking by protection of the young concrete are given in 'Durable Concrete Structures —CEB Design Guide'.

d.12.5. Heat treatment

The rate at which concrete hardens can be accelerated by heat treatment, because a rise of temperature during hardening, within certain limits, increases the early strength; the final strength, however, may be somewhat less than that of concrete stored at normal temperatures. Decisive factors in this respect are the age of concrete at the commencement of heat treatment, the rate of temperature rise, the level of temperature, the duration of heat treatment and the rate of cooling. The successful application of heat treatment depends on the type of cement used, but generally applicable rules cannot be given. Hence, before commencing production, the materials to be used and the method of treatment employed should be checked for their suitability through trial mixes.

Unless there is sufficiently documented positive experience with the conditions of a particular heat treatment of a concrete of given composition and constituent materials, particularly the cement, it is recommended that for curing of concrete members which will be subjected to exposure classes 2 to 5 (Table d.1), the following limitations with regard to heat treatment (steam curing) are observed.

- Concrete temperature during the first 3 hours after mixing should not exceed 30°C and should not be higher than 40°C during the first 4 hours.
- The rate of temperature increase should not exceed 20 K/h.
- The average maximum temperature of the concrete should not exceed 60°C (individual values < 65°C).
- The concrete should be cooled at a rate not exceeding 10 K/h.
- The concrete should be protected against moisture loss throughout the curing procedure and while cooling.

These limitations in the temperature regime are given to prevent chemical reactions some time after the heat treatment, generally referred to as secondary ettringite formation. They are associated with a volume increase and in turn cracking and disruption of the concrete. Since these reactions depend on the composition of a particular cement a more severe heat treatment than given above does not necessarily lead to such damaging reactions.

Precautions required when introducing steam into the mixer are discussed in section d.9.2.

Heat treatment can influence the final properties of the hardened concrete (e.g. ratio of tensile strength to compressive strength, deformation properties, durability). This should be taken into account in design.

d.13. Concreting in cold weather or frost

The hydration of the cement paste is retarded at lower temperatures. Apart from this, frost can permanently impair immature concrete if the water contained in the pores freezes and disrupts the concrete. Therefore, at low ambient temperatures appropriate measures should be taken to ensure that the rate of hardening is adequate and that frost damage is prevented.

Whenever there is a risk of frost or prolonged freezing, the fresh or very young concrete should be protected from freezing by arrangements for covering and/or insulating the newly placed concrete or by enclosures for heating the air surrounding the newly cast structural element. Especially in the latter case, adequate provisions for maintaining proper moisture conditions are essential.

Before being exposed to freezing temperatures, concrete should have a compressive strength of at least 5 MPa. The time required to reach this minimum freezing strength will be the shorter the higher the temperature of the concrete and the lower the water/cement ratio. For a more accurate estimate of this time refer to 'Durable Concrete Structures—CEB Design Guide'.

The drop in temperature of any portion of the concrete should be gradual, not exceeding 10°C in massive structures and 20°C in non-massive structures throughout the first 42 hours after the end of protection. The safe removal of formwork and its supports is controlled by the proportion of design strength which should be attained at the time of stripping forms or removing their supports.

The minimum temperature of fresh concrete at the time of placing should not be lower than + 5°C. However, when concrete is cast in thin sections this minimum temperature should preferably be at least 10°C. The initial temperature of fresh concrete as mixed should be raised to offset heat loss between concrete mixing and placing, but it should in no case be higher than 30°C.

The desired temperature of the fresh concrete can be achieved by heating the constitutive materials. Mixing water may be heated up to 100°C, and the average temperature of aggregates for an individual batch should not exceed 65°C. To avoid the possibility of flash set, water and aggregate should be first mixed together in such a way that the temperature of the mixture is reduced to less than 40°C, and only after that may cement be admitted into the mixer. The aggregate should be free of lumps of ice and snow.

The subgrade on which fresh concrete is placed should not be frozen, and its minimum temperature should be at least 3°C.

For details reference is made to the appropriate provisions of the various national standards.

d.14. Concreting at high temperatures

The properties of concrete can be influenced unfavourably by hot weather. In concrete technology hot weather is defined as any combination of high temperature, low relative humidity and high wind velocity tending to impair the quality of fresh and hardened concrete or otherwise resulting in abnormal properties. High temperatures of fresh concrete accelerate setting, increase the rate of hydration and the water demand, and lead to diminished final strength. Hot weather also causes difficulties during placing and the formation of plastic shrinkage cracks in young concrete.

Therefore, every effort should be made to ensure that the temperature of the concrete at the time of placing is lower than 30°C to 35°C for normal structures, and less than 15°C for mass concrete. In cases where severe internal stresses or restraint may develop due to differences between the increased temperature caused by heat of hydration and the temperature of the surrounding air, the temperature of the fresh concrete at the time of casting should be controlled so as to limit this temperature difference. Also refer to subsection d.12.4.

This should be achieved by

- cooling of the mixing water
- reducing the aggregate temperature by shading the stockpiles and/or sprinkling or spraying coarse aggregate fractions
- keeping the cement temperature low; if the initial temperature of the cement is sufficiently low, this may be achieved by insulating the walls of the silos
- shading and/or sprinkling the batching, mixing, delivery and placing equipment
- good co-ordination and speeding-up of placement and finishing operations
- proper planning of the time of casting, so that the concrete is placed when the ambient temperature is as low as possible.

Curing should be carried out in accordance with section d.12. When concreting at high temperatures moist curing is particularly suitable. It should continue without interruption for a minimum of 7 days. For flat structures (slabs, pavements), however, the use of an appropriate membrane-forming curing compound which is evenly applied immediately after completion of the finishing operations often is more practical.

During placing and finishing operations, particularly flat structures should be protected from wind and sunshine by windbreaks and/or shades, and any excessive evaporation of water causing early shrinkage cracking should be prevented by cooling and moistening or fogging the surrounding air.

d.15. Retempering

Retempering is the addition of water or cement paste with an appropriate water/cement ratio, or of a high range water reducing superplasticizing admixture, and the remixing of concrete which, on arrival at the construction site, has lost so much workability that it has become unplaceable or unusable.

Retempering with water shall be prohibited outright since it is very likely to increase the original water/cement ratio, thus reducing the quality of the hardened concrete. It may be tolerable only if the effective water/cement ratio remains within specified limits after taking into account estimated amounts of water already chemically combined or evaporated.

Retempering with a high range water reducing superplasticizing admixture is much more convenient, because it may even allow a reduction of the water/cement ratio of the initial mix. In such cases a proper amount of admixture should be added directly into the truck mixer where mixing should continue until a uniform consistence of the batch is achieved. Repeated retempering is feasible, but its effectiveness diminishes. Therefore, it is not advisable to retemper more than twice. Attention should be paid to the fact that the use of high range water reducing superplasticizing admixtures may cause a loss of entrained air.

d.16. Structural lightweight aggregate concrete: special factors

d.16.1. Requirements for aggregates

Lightweight aggregates should satisfy the requirements contained in the relevant standards.

Water absorbed by the aggregates can affect the resistance of the concrete to damage due to freezing and thawing, its fire resistance and thermal resistivity. Where these properties are of importance, suitable measures should be taken to prevent the absorption of excessive quantities of water by the aggregate as it may take a prolonged period, several years in some cases, before the absorbed water can be liberated.

d.16.2. Mix design

The composition of lightweight aggregate concrete should be based invariably on trial mixes, unless the design is derived from previous experience with the same constituent materials (C I according to subsection d.8.1 or C II according to subsection d.8.2). For acceptance of a mix design the results of the tests on the trial mixes should indicate that a strength appropriate for the prescribed strength class, the required density and other required properties can be attained with adequate certainty and that the workability of the concrete will be satisfactory.

d.16.2.1. Grading of aggregates

Lightweight aggregates shall be delivered and stored separately in a sufficient closed graded range of sizes to allow for the fact that lightweight aggregate particles may vary in some of their properties with the size of the aggregate particles (e.g. particle density, strength, water absorption). Accidental mixing of different gradings or segregation within a group may affect the workability, strength and density of lightweight aggregate concrete. It is recommended that the maximum particle diameter should not exceed 25 mm, in order to avoid segregation and as a consequence reduction in the strength of the concrete.

Advice on the grading of the aggregates is given in clauses d.6.5.1.2 and d.6.5.1.3 for continuous and gap-graded mixes.

d.16.2.2. Cement content

The required cement content can be determined from trial mixes. For reinforced and prestressed lightweight aggregate concrete the cement content used should not fall below certain minimum values (e.g. 300 kg/m^3). Nor should the cement content be allowed to be too high (more than 500 kg/m^3) since—as for all other types of concrete—this can lead to microcracking due to excessive heat development, especially in large components and possibly to excessive shrinkage and creep, in the absence of adequate curing and other preventative measures.

d.16.3. Consistence

In general it is recommended that, immediately before the concrete is cast, its consistence should lie in the plastic range. Lightweight aggregate concrete having a stiff consistence should only be used in special circumstances since it tends to be more difficult to place and compact.

On the other hand it should be borne in mind that semi-fluid and fluid lightweight aggregate concrete tends to segregate owing to rising of the lighter coarse aggregate particles.

d.16.4. Prewetting and batching of aggregates

Water absorption by the aggregates may cause a reduction of the effective water content of the cement paste. The effective water content of the cement paste is important with respect to the evolution in time of workability, to strength and to durability requirements. In order to fix the total amount of mixing water (w_{tot}) and the effective water content (w_{ef}) or the water/cement ratio (w/c) one of the following procedures is proposed.

Dry aggregates

In this case two methods can be applied.

- The dry aggregates and the sand are mixed during 1 min with 40–60% of the total amount of water. This total amount is determined by adding conventionally to the effective water in the cement paste, the amount of water absorbed by the aggregates in 30 minutes (measured on a separate sample).

$$w/c \;=\; (w_{tot} - w_{30\,\mathrm{min}})/c$$

- The dry aggregates remain in water during 30 minutes. Before mixing the aggregates are drained for 5 minutes. Only the supplementary water is added during mixing.

$$w/c \;=\; w_{add}/c$$

Wet aggregates

After the determination of the moisture content (m), comparison with the absorption after 30 minutes indicates whether supplementary water has to be added

$$w/c \;=\; [w_{tot} - (w_{30\,\mathrm{min}} - m)]/c \quad \text{if } m < w_{30\,\mathrm{min}}$$

$$w/c \;=\; w_{add}/c \quad \text{if } m > w_{30\,\mathrm{min}}$$

Trial mixes with a particular aggregate are recommended to fix the total amount of mixing water leading to the desired workability and strength.

If the moisture content of the aggregate is highly variable, batching the coarse aggregate by volume may be preferable. The moisture held in the aggregate particles need not be taken into account but of course, the mixing water (i.e. the quantity of water to be added at the mixer) must be considered. However, when batching by weight, the water held in the aggregate should be taken into account.

d.16.5. Mixing

The mixing process should continue long enough to ensure that the resulting fresh concrete is homogeneous. The period required may be longer than that needed for normal weight concrete. However, in judging the duration of the mixing process, the enhanced friability of some types of lightweight aggregate (e.g. most foamed slags) should not be overlooked.

Admixture should not be added to the mix before the aggregate particles are sufficiently wetted, to prevent a part of the admixture being absorbed by the aggregate particles, thus reducing the effect of the admixture.

d.16.6. Ready-mixed concrete

In practice lightweight aggregate concrete should be transported in truck mixers so that it can be intensively re-mixed immediately before delivery on the site.

Owing to the higher water absorption of lightweight aggregates, loss of workability during transport and handling can be higher for some types of lightweight aggregate concrete than for concrete made with normal weight aggregate.

It is recommended that trial mixes should be produced if a particular lightweight aggregate is to be used for the first time for a ready-mixed concrete. For these trial mixes account should be taken of the proposed working procedure, including length of haul, handling conditions and casting procedures.

d.16.7. Handling

When handling lightweight aggregate concrete, the greater tendency of some types of lightweight aggregate to segregate compared with normal weight aggregate should be taken into account. This tendency will be enhanced where the concrete is of fluid consistence and the density of the aggregate particles is low (less than 1000 kg/m^3). The cohesion of fresh concrete can be improved, where necessary, by using admixtures, such as water reducers, air-entraining agents and stabilizers or by the use of certain additions.

When pumping lightweight aggregate concrete, careful attention should be given to the mix design and the type of pumping equipment and aggregate to be used (e.g. natural sand should preferably be used when pumping is considered), in order to reach the required distance and lift. Depending on the type of pumping equipment and aggregate used, the effective pumping distance and lift can be less than those achieved in the case of concrete with normal weight aggregate.

d.16.8. Casting

For some types of very lightweight aggregate concretes compaction may be more difficult to achieve, than for concrete made with normal weight aggregates. Consequently the positions where concrete is placed in the formwork and the points where vibration is effected should be more closely spaced. Lightweight aggregate concrete should always be compacted by means of vibration, as indeed is the case for normal weight concrete.

In most cases the effective radius of action for poker vibrators in lightweight aggregate concrete is only about half that observed for concrete made with normal weight aggregates. Consequently the customary distances between immersion points should be approximately halved. For some types of lightweight aggregate concrete, it may be more difficult to obtain a satisfactory surface finish because the coarse lightweight aggregates particles tend to float. This can be alleviated by using surface vibrators or by treating the surface with a roller fitted with a perforated drum, thus pushing the larger aggregate particles down while allowing the fines to rise.

d.16.9. Curing

Because of the large quantity of water absorbed by lightweight aggregates, concrete made with such aggregates responds well to curing. As evaporation takes place on the surface of the concrete, water absorbed by the aggregate is released and transferred to the matrix. Thus, water is automatically available for hydration by continuous replacement for a period which will depend on the ambient conditions.

In temperate climates it is sufficient to ensure proper hydration without the use of external devices to prevent evaporation, such as damp sacks or a plastic membrane, usually required in the case of concrete made with normal weight aggregates. In hot climates, however, careful attention should be given to the protection of the concrete surface against rapid drying out.

Curing of lightweight aggregate concrete may need to be more closely controlled than that of concrete made with normal weight aggregates, particularly if the lightweight aggregate is very dry and continues to absorb much water even after finishing. With regard to the duration of curing the recommendations given in section d.12 are applicable.

The temperature increase due to heat of hydration tends to be higher for lightweight aggregate concrete, but owing to the lower thermal expansion coefficient of lightweight aggregates and closer compatibility of the moduli of elasticity of aggregate and matrix, the extent of microcracking is generally less in lightweight aggregate concrete.

d.17. Production of high strength concrete

d.17.1. Principles

The scope of the previous model code MC 78 was limited to strength grades up to C 50. In the following sections some specific facts relating to the production of cement based concrete of higher strength, frequently referred to as high strength concrete, are dealt with. Such concretes may be advantageous not only because of their higher strength, but also because of their lower permeability thus enhancing the durability of concrete structures.

A strength of concrete in excess of 50 MPa is reached by a reduction of the water/cement ratio or of the water/cement + effective pozzolan ratio to values below 0.40. This is accomplished by the use of high range water reducing superplasticizing admixtures. Sometimes highly reactive additions such as silica fume or some pulverized fuel ashes substantially contribute to the development of high strength.

In general no particular equipment. batching and mixing methods or unusual admixtures or additions are necessary to produce such concretes.

d.17.2. Choice of materials

d.17.2.1. Cements

The cements used for high strength concrete must satisfy the general requirements for ordinary concretes.

Since in many instances high amounts of high strength Portland cements are used, particular attention should be paid to the development of heat of hydration and its consequences.

d.17.2.2. Aggregates

The aggregates for high strength concrete should satisfy the requirements laid down for aggregates for ordinary concrete. When choosing suitable aggregates, particular attention should be paid to possible alkali-silica reactions because of the high cement contents of high strength concretes.

d.17.2.3. Admixtures

In most instances, water reducing or high range water reducing superplasticizing admixtures are used to reduce the water demand for a given consistence of the fresh concretes.

d.17.2.4. Additions

In many instances, condensed silica fume or highly reactive pulverized fuel ashes with a sufficiently high specific surface area are used as additions.

d.17.2.5. Mixing water

The requirements are identical to those for ordinary concrete.

d.17.3. Consistence of fresh concrete

Whereas pulverized fuel ash in many instances reduces the water demand, the use of silica fume results in a substantial increase of the water demand. For this reason high range water reducing superplasticizing admixtures are often used in concretes containing silica fume.

For the characterization of consistence the Vebe test proved to be particularly suitable. In many practical applications it became apparent that high strength concrete can also have a fluid consistence in the fresh state. For such concrete the flow table is particularly suitable to measure concrete consistence.

d.17.4. Concrete composition

In principle, there are no differences between the principles for the composition of ordinary and of high strength concretes. However, particular attention should be paid to the proper grading of aggregates.

d.17.5. Batching and mixing

For batching and mixing of high strength concrete the rules laid down for ordinary concrete apply (section d.9).

d.17.6. Handling, placing and compaction

Silica fume reduces the tendency of concrete to segregate. In many instances high strength concrete is very suitable for pumping. Otherwise, the same rules for handling and placing apply as for ordinary concrete (section d.10).

In order to achieve the required high strength, proper vibration to remove entrapped air is of particular significance.

d.17.7. Curing

Because of the low water/cement ratio early loss of water may stop hydration of high strength concrete, and the low bleeding rates may cause plastic shrinkage. Therefore, the concrete has to be cured carefully, employing curing methods where water is added rather than using methods which only prevent moisture loss. Irrespective of subsection d.12.3 a minimum curing period of 3 days is recommended in all cases.

d.18. Personnel, equipment and installations

d.18.1. General requirements

Batching and mixing, handling and placing, curing as well as testing and the quality control of concrete require reliable supervising personnel with a successful service record in concrete and reinforced concrete construction, as well as sufficient knowledge and experience, so that the successful execution of construction work is ensured. Making and placing the concrete should be carried out by trained and qualified personnel (concretors).

d.18.2. Supervisor on the construction site

During the course of construction, the contractor or the supervisor assigned by the contractor or his/her knowledgeable representative have to be present on the site during construction. The supervisor has to make sure that the work is executed according to the applicable specifications, drawings and regulations. In particular he/she is responsible for

- the dimensions of the structural members as planned
- the safe construction and stiffening of the formwork
- quality of the constituent materials as specified
- conformity of the reinforcement with the drawings
- the proper choice of the time of stripping the forms
- prevention of overloading finished structural members
- elimination of damaged prefabricated members which could reduce the load carrying capacity of the structure
- correct installation of auxiliary supports where needed.

For the batching and mixing, handling and placing, compaction as well as curing, all installations and equipment, which are required for a high quality concrete with uniform strength as pointed out in section d.9 through d.17, have to be available and properly maintained.

d.18.3. Supervisor for precast concrete and ready-mixed concrete plants

During the working hours the supervisor or his/her knowledgeable representative have to be present on the plant. He/she has to fulfil similar tasks as have been pointed out in subsection d.18.2 as far as they apply to the work in a plant.

In addition, the supervisor is responsible for

- additional requirements regarding equipment and installations in such plants
- observation that only such ready-mixed concrete or precast concrete units leave the plant which satisfy all technical requirements
- delivery tickets which have to contain all necessary information.

d.18.4. Permanent concrete laboratory

Contractors who use concrete with special properties according to subsection d.6.6 or according to section d.11, as well as ready-mixed concrete and precast concrete plants should have access to a permanent concrete laboratory, which is equipped with all testing apparatus and installations necessary for preliminary experiments, as well as for quality control. The laboratory has to be situated such that a close co-operation with the construction site is possible. If the contractor makes use of an external testing laboratory, a contract specifying all testing and quality control measures to be conducted by the laboratory has to be issued. Such contracts should be valid for a prolonged period of time.

Production control of a plant or of a site should not be carried out by a concrete laboratory which at the same time carries out production control measures for the constituent materials.

The concrete laboratory has the following tasks

- trial testing of the concrete
- quality control of the concrete, including strength development, as far as these tests are not conducted by the personnel on the site
- control of equipment on the site and in the plant prior to commencement of the concrete work
- continuous control as well as advice during manufacturing, handling and curing the concrete; records have to be kept about the results achieved
- evaluation and judgement of the results of all tests carried out for the individual plants and sites, and reports of the results to the contractor and his supervisor or the supervisor of a plant
- training of personnel on the site and of the plant.

The concrete laboratory has to be headed by an expert knowledgeable in concrete technology and concrete production. The expert should have a certificate indicating such knowledge.

The contractor is responsible that the supervisor as well as the personnel responsible for manufacturing and testing the concrete are trained and instructed at suitable intervals, so that they are able to cary out all measures for the concrete work, including testing and quality control. The supervisor of a concrete laboratory should keep records on the training of the personnel.