Alfred Steinle, Hubert Bachmann, Mathias Tillmann (Ed.)

Precast Concrete Structures

- introduction to this subject and as a practical resource with examples for both structural engineers and architects
- history of this construction method and the status of European standards are also included
- completely revised by a new group of authors for this edition

Building with precast concrete elements is one of the most innovative forms of construction. This book serves as an introduction to this topic, including examples, and thus supplies all the information necessary for conceptual and detailed design.

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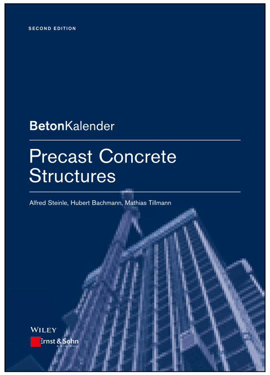

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ABOUT THE BOOK

Der Betonfertigteilbau ist eine der innovativsten Bauweisen - hier werden neue Betone, Bewehrungen und Herstellverfahren erstmals angewendet, denn das Fertigteilwerk bietet hervorragende Voraussetzungen für die industrielle Herstellung.
Dieses Buch führt in die Bauweise ein und vermittelt alles notwendige Wissen für die Konstruktion, Berechnung und Bemessung. Auch die geschichtliche Entwicklung und der Stand der europäischen Normung werden aufgezeigt. Der Dreh- und Angelpunkt für den wirtschaftlichen und fehlerfreien Einsatz von Betonfertigteilen ist der fertigungs- und montagegerechte Entwurf. Neben den zu beachtenden Randbedingungen werden typische Fertigteilkonstruktionen zur Diskussion gestellt. Die Verbindungen der Betonfertigteile sind gerade bei Horizontallasten besonders zu beachten, daher wird die Aussteifung von Fertigteilgebäuden ausführlich behandelt. Besonderheiten der Bemessung, z. B. Lager, Konsolen und Stützenstöße, werden detailliert dargestellt. Ein zunehmend wichtiger Anwendungsbereich für Betonfertigteile ist der Fassadenbau, welchem ein eigenes Kapitel gewidmet ist. Abschließend wird auf die Fertigung eingegangen, um beim Leser das Verständnis für die Bauweise zu vertiefen.
Für die vorliegende 2. Auflage wurde das Werk vom erweiterten Autorenteam komplett überarbeitet. Das Buch ist eine Einführung und ein praktisches Arbeitsmittel mit Beispielen für Bauingenieure und Architekten gleichermaßen.

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Foreword

The *Concrete Yearbook* is a very important source of information for engineers involved in the planning, design, analysis, and construction of concrete structures. It is published on a yearly basis and offers chapters devoted to various, highly topical subjects. Every chapter provides extensive, up-to-date information written by renowned experts in the areas concerned. The subjects change every year and may return in later years for an updated treatment. This publication strategy guarantees that not only is the latest knowledge presented, but that the choice of topics itself meets readers' demands for up-to-date news.

For decades, the themes chosen have been treated in such a way that, on the one hand, the reader gets background information and, on the other, becomes familiar with the practical experience, methods, and rules needed to put this knowledge into practice. For practising engineers, this is an optimum combination. In order to find adequate solutions for the wide scope of everyday or special problems, engineering practice requires knowledge of the rules and recommendations as well as an understanding of the theories or assumptions behind them.

During the history of the *Concrete Yearbook*, an interesting development has taken place. In the early editions, themes of interest were chosen on an ad hoc basis. Meanwhile, however, the building industry has gone through a remarkable evolution. Whereas in the past attention focused predominantly on matters concerning structural safety and serviceability, nowadays there is an increasing awareness of our responsibility with regard to society in a broader sense. This is reflected, for example, in the wish to avoid problems related to the limited durability of structures. Expensive repairs to structures have been, and unfortunately still are, necessary because in the past our awareness of the deterioration processes affecting concrete and reinforcing steel was inadequate. Therefore, structural design should now focus on building structures with sufficient reliability and serviceability for a specified period of time, without substantial maintenance costs. Moreover, we are confronted by a legacy of older structures that must be assessed with regard to their suitability to carry safely the increased loads often applied to them today. In this respect, several aspects of structural engineering have to be considered in an interrelated way, such as risk, functionality, serviceability, deterioration processes, strengthening techniques, monitoring, dismantlement, adaptability and recycling of structures, and structural materials plus the introduction of modern high-performance materials. The significance of sustainability has also been recognised. This must be added to the awareness that design should focus not just on individual structures and their service lives, but on their function in a wider context as well, i.e. harmony with their environment, acceptance by society, responsible use of resources, low energy consumption, and economy. Construction processes must also become cleaner and cause less environmental impact and pollution.

The editors of the *Concrete Yearbook* have clearly recognised these and other trends and now offer a selection of coherent subjects that reside under the common 'umbrella' of a broader societal development of great relevance. In order to be able to cope with the corresponding challenges, the reader can find information on progress in technology, theoretical methods, new research findings, new ideas on design and construction, developments in production and assessment and conservation strategies. The current selection of topics and the way they are treated makes the *Concrete Yearbook* a splendid opportunity for engineers to find out about and stay abreast of developments in engineering knowledge, practical experience and concepts in the field of the design of concrete structures on an international level.

TU Delft

Prof. Dr. Ir. Dr.-Ing. h. c. Joost Walraven
Honorary president of the international concrete federation fib

Preface to the Third German Edition

Building with precast concrete components is as old as building with reinforced concrete itself, for the very first reinforced concrete element, Joseph Monier's flower tub (c. 1850), was, in essence, a *precast* concrete item.

It was only in the second half of the twentieth century, however, that this form of construction took on its industrialised form. Factors that contributed to this were, in particular, the development of heavy lifting equipment, the use of mechanised steel moulds, and, more recently, automated manufacturing systems, for producing suspended floor elements especially.

This book on precast concrete construction is based on the manuscript written by Prof. Dr.-Ing. Volker Hahn (former director of Ed. Züblin AG) for his lectures at the University of Stuttgart in the early 1970s, which was recast as a book by Dr.-Ing. Alfred Steinle. The manuscript rewritten by Alfred Steinle and Volker Hahn first appeared in *Beton-Kalender 1988*. That article was reprinted in 1995 and in revised form in 2009 and 2016. It was in 1998 that the information first appeared as an actual book as part of the *Bauingenieur-Praxis* series. The second edition was published in 2009 and now it is time for a new edition.

With modern methods of construction making use of industrial methods of manufacture, which includes construction with factory-precast concrete components, the design of the individual elements, and also the entire structure, is heavily influenced by the factory production. On the manufacturing side, the growing trend towards mechanisation and automation in production is evident.

The development of high-performance concretes provides us with the chance to employ these for precast concrete construction in particular because factory production presents excellent conditions for their use. The first precast concrete components made from ultrahigh-strength concrete for bridges and façades are already in use, the latter also making use of glass fibres or carbon inlays. Besides the industrial production of batches and series of components, we are seeing more and more one-offs being produced, which take advantage of the excellent production options in order to achieve a high standard of quality. These tendencies will become even more obvious as more and more progress is made in the development of concrete as a building material.

The authors' aim in writing this book is to map out the boundary conditions of factory prefabrication for architects and structural engineers and also to demonstrate the opportunities presented by this method of construction – and thus contribute to the ongoing development of precast concrete structures.

November 2018

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Stuttgart/Bonn

Contents

2.2 Tolerances and Calculations for Fit

2.2.1 General

Construction work is characterised by many different manual activities where deviations between required and actual dimensions are unavoidable. In order that structures and components of the structural carcass and interior fitting-out can be assembled properly without reworking, it is necessary to consider tolerances.

The topics of 'tolerances' and 'joints' come into play in precast concrete construction when joining together individual components. It must be ensured that the precast concrete elements can be fitted into the structure on the building site. As dimensional deviations in the precast concrete elements can no longer be compensated for on the building site, appropriate joints must be provided (see Section 2.2.3).

Joint widths between precast concrete elements are influenced by the following factors:

- Changes in length of the elements, e.g. due to temperature fluctuations or shrinkage
- Deformability of joint seals
- Dimensional deviations resulting from production and erection
- Measuring and workmanship inaccuracies on site.

Deformations of concrete components are unavoidable. However, they are not covered by tolerance standards, instead can be calculated related to a particular structure in the course of the structural design work, although such calculations can only be as accurate as the accuracy of the input values.

The magnitude and sign (+ or −) of some deformations change during the period of use, e.g. due to temperature changes, whereas other deformations, e.g. due to creep and shrinkage, are generally irreversible. Deformations that change with time and load must be taken into account in calculations for fit where they are important for the proper assembly of components.

Measuring and workmanship inaccuracies are random variables whose magnitude and sign cannot be predicted at the design stage, instead only ensue during the production, construction, and erection processes. For this reason, the tolerance standards specify maximum dimensional deviations, so-called limits of size. The task of a joint is therefore to compensate for both unavoidable dimensional deviations and also unavoidable changes in length.

Furthermore, when specifying joint widths, it is necessary to consider that the maximum deformability of a joint seal may not be exceeded. For example, in DIN 18540 the maximum value for the permissible deformability of joint sealants is given as 25% of the total deformation, i.e. the joint may widen or narrow by max. 25%. As façade panels in particular are subjected to considerable temperature fluctuations, the width of the joint will vary by at least these temperature-induced changes in length. Where joints are too narrow, the 25% figure is exceeded, meaning that the joint material tears or is excessively compressed (see [8]).

Regular joint layouts or equal joint widths between all precast elements are not necessary from the technical viewpoint and also contradict the aforementioned basic principle of a joint. Where a regular joint layout is required for purely aesthetic reasons, the joint no longer serves to compensate for changes in length and tolerances, instead is there purely to satisfy such aesthetic aspirations. As, however, in these situations, too, random dimensional deviations continue to occur and have to be compensated for, these must be given due attention. The measures required for this can exceed the normal duty of care of manual activities and hence lead to additional costs [9].

2.2.2 Tolerance Standards

The following dimensional deviations can occur when building with precast concrete elements:

- Dimensional deviations during the production of the element in the precasting plant
- Dimensional deviations during erection or during the handling of the elements
- Dimensional deviations on the building site, e.g. when measuring or setting out or due to prior trades.

Every individual operation results in dimensional deviations, which means that the dimensional accuracy of the structure in its finished state depends on the dimensional deviations of the individual operations (see ISO 1803).

In Germany, two standards in particular are relevant for tolerances for buildings:

- DIN 18202:2013-04 Tolerances in building construction – Structures
- DIN 18203-1:1997-04 Tolerances in building construction – Part 1: Prefabricated components made of concrete, reinforced concrete and prestressed concrete.

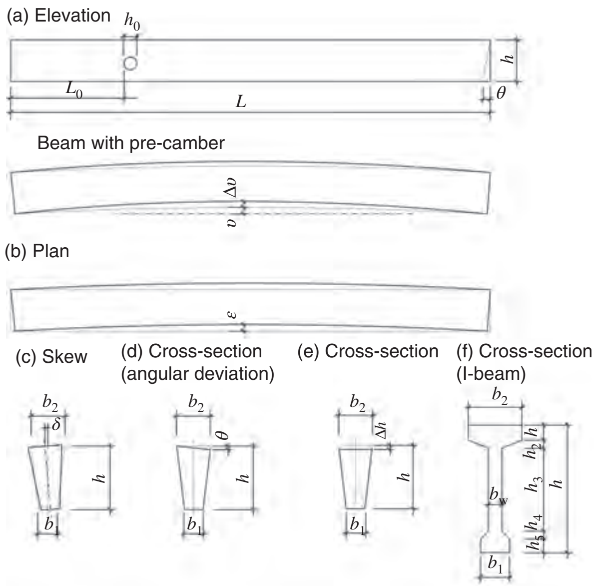
General production tolerances for precast concrete elements are specified in DIN 18203-1, although this standard has been formally withdrawn from the active set of DIN standards because it contradicts the applicability of the European product standards. However, the tolerances given in DIN 18203-1 still correspond to the state of the art and ensure that the standard of production in Germany accepted by all those involved is retained, thus reducing the probability of problems of fit.

Figure 2.4 shows examples of the production tolerances for beams.

When precast concrete elements are integrated in a structure, they fall within the remit of DIN 18202. The requirements specified in DIN 18202 are independent of material and type of construction and also include measuring and erection tolerances. Two key tasks of DIN 18202 are therefore to ensure that all the parts of the structure fit together and to regulate the interfaces between individual trades and parts of the structure.

DIN 18202 specifies the following limits of size for structures:

Limits of size for dimensions (see Table 2.1)
Limit values for angular deviations (see Table 2.2)
Limit values for flatness deviations (see Table 2.3)
Limit values for alignment deviations (see Table 2.4).



Limits of size for length dimensions

Component	Limits of size ΔL in (mm) for nominal dimensions L in (m)							
	≤1.5	>1.5 ≤3.0	>3.0 ≤6.0	>6.0 ≤10.0	>10.0 ≤15.0	>15.0 ≤22.0	>22.0 ≤30.0	>30.0
Length, reinforced concrete beams	±6	±8	±10	±12	±14	±16	±18	±20
Length, prestressed concrete beams	—	—	—	±16	±16	±20	±25	±30

Limits of size for cross-section dimensions

Component	Limits of size Δh, Δb in (mm) for nominal dimensions h, b in (m)					
	≤0.15	>0.15 ≤0.30	>0.30 ≤0.60	>0.60 ≤1.0	>1.0 ≤1.5	≥1.5
Cross-section dimensions, beams	±6	±6	±8	±12	±16	±20

Limit values for angular deviations

Component	Limit values θ in (mm) for nominal dimensions in (m)		
	≤0.40	>0.40 ≤1.0	≤1.50
Cross-section dimensions, beams	±4	±6	±8

If the limits of size for length or cross-section dimensions are fully exploited, the limit values for angular deviations may not be exceeded. The stricter criterion governs in each case

Other limit values

Limit value for curvature ε at every level of construction: $\varepsilon = \pm L/700$
Limit values for deviations from pre-camber $\Delta\upsilon = \pm L/700$ (for prestressed concrete beams: $\Delta\upsilon = \pm L/500$)
Skew of longitudinal axis: $\delta = \pm L/700$
Limits of size for openings: – Position of opening: ΔL_0 as for ΔL (length) – Size of opening: Δh_0 to DIN 18202, table 1 and DIN 18202, table 2

Figure 2.4 Limits of size for beams according to DIN 18203-1 and DIN EN 13225.

Table 2.1 Limits of size for dimensions (to DIN 18202, table 1).

	Limits of size in (mm) for nominal dimensions in (m)					
	≤1.0	>1.0 to ≤3.0	>3.0 to ≤6.0	>6.0 to ≤15.0	>15.0 to ≤30.0	>30.0[a)]
Related to dimensions on plan, e.g. grid dimensions	±10	±12	±16	±20	±24	±30
Related to dimensions on elevation, e.g. storey heights	±10	±16	±16	±20	±30	±30
Related to clear dimensions on plan, e.g. dimensions between columns	±12	±16	±20	±24	±30	—
Related to clear dimensions on elevation, e.g. below floor slabs and downstand beams	±16	±20	±20	±30	—	—
Related to openings, e.g. for windows, external doors	±10	±12	±16	—	—	—
Related to openings as above, but with finished reveals/jambs	±10	±10	±12	—	—	—

a) These limits of size can be used for nominal dimensions of up to about 60 m. Special considerations will be necessary for larger nominal dimensions.

Table 2.2 Limit values for angular deviations (to DIN 18202, table 2).

	Perpendicular offsets in (mm) for nominal dimensions in (m)						
	≤0.5	>0.5 to ≤1.0	>1.0 to ≤3.0	>3.0 to ≤6.0	>6.0 to ≤15.0	>15.0 to ≤30.0	>30.0[a)]
Related to vertical, horizontal and sloping surfaces	3	6	8	12	16	20	30

a) These limits of size can be used for nominal dimensions of up to about 60 m. Special considerations will be necessary for larger nominal dimensions.

In the course of the planning process (Section 2.2.3), these limits of size must be taken into account as an allowance in addition to the production tolerances. It is necessary to comply with limits of size for length and cross-section dimensions as well as limit values for angular deviations. The stricter criterion governs in each case.

Flatness deviations for components are considered separately from limits of size or angular deviations. The figures given in tolerance standards regarding the flatness of planar components such as floor and wall elements relate to the individual elements. Offsets and projections at the edges of these components, i.e. between adjoining precast concrete elements, are not covered by DIN 18202. It is not possible to specify generally applicable figures for such offsets and projections. Limit values for offsets and projections as well as the compensatory

Table 2.3 Limit values for flatness deviations (to DIN 18202, table 3).

	Perpendicular offsets in (mm) for measuring point spacings in (m)				
	≤0.1	≤1.0	≤4.0	≤10.0	≤15.0[a)]
Related to unfinished top surfaces of ground or suspended floor slabs with low requirements (e.g. precast floor plates with in situ concrete topping)	10	15	20	25	30
Related to unfinished top surfaces of ground or suspended floor slabs to be covered by floor finishes with normal requirements	5	8	12	15	20
Related to finished top surfaces of ground or suspended floor slabs with low requirements (e.g. basements, storage rooms)					
Related to finished top surfaces of ground or suspended floor slabs with normal requirements	2	4	10	12	15
Related to finished top surfaces of ground or suspended floor slabs with higher requirements	1	3	9		
Related to unfinished wall surfaces and soffits of suspended floor slabs	5	10	15	25	30
Related to finished wall surfaces and soffits of suspended floor slabs with normal requirements	3	5	10	20	25
Related to finished wall surfaces and soffits of suspended floor slabs with higher requirements	2	3	8	15	20

a) These limits of size can also be used for measuring point spacings >15 m.

Table 2.4 Limit values for out-of-plumb deviations for columns (to DIN 18202, table 4).

	Perpendicular offsets as limit values in (mm) for measuring point spacings in (m)				
	≤3.0	>3.0 to ≤6.0	>6.0 to ≤15.0	>15.0 to ≤30.0	>30
Related to permissible deviations from alignment	8	12	16	20	30

measures necessary in each case must therefore be specified separately by the designer. Reference [9] contains information on this.

Tolerance standards were not drawn up to satisfy aesthetic requirements or to assess the visual appearance of a component or structure (see Section 2.2.1), but rather for purely technical reasons, to ensure that components fit together

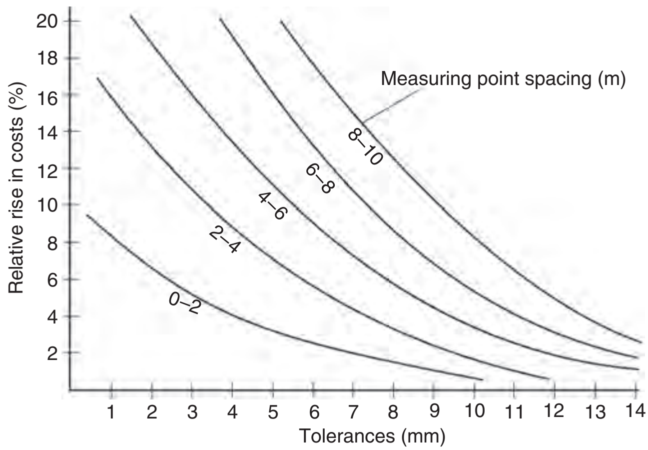


Figure 2.5 Relative rise in costs depending on construction tolerances according to [10].

properly. Limit values and suitable compensatory measures for achieving visual and aesthetic requirements must therefore be agreed in each individual case.

Tolerance standards define a framework that can be expected for components or structures with typical forms of construction and typical dimensions built with customary care and attention. Beyond the scope of that framework, the permissible dimensional deviations and the associated methods of measurement will have to be specified separately. This also applies when greater precision is required. In principle, when specifying 'tighter' tolerances, it is advisable to weigh up the technical feasibility, the functional requirements, the additional work, and the associated additional costs. Figure 2.5 shows the approximate rise in costs to be expected when specifying higher precision.

Cast-in parts and connectors are crucial for the proper fit of components to comply with functional requirements. Some cast-in parts and connectors can be adjusted for height, lateral position, or spacing, which simplifies the fitting together of elements. Tolerances for cast-in parts and connectors are not explicitly specified in the tolerance standards. The values given in Tables 2.5 and 2.6 can be generally assumed.

2.2.3 Calculations for Fit

Tolerance standards specify limits of size for individual components, but compliance with those does not automatically guarantee that several components will fit together. In order to ensure that components can be assembled with the necessary joint widths, the final fit must be considered, i.e. calculations for fit carried out.

In the course of considering this final fit, it is necessary to establish whether it is reasonable to apply relevant tolerance standards or, for reasons of function or appearance, other, possibly greater, precision will have to be specified. Agreement on tolerances and interfaces between the different specialists and trades involved must therefore be reached at the earliest possible opportunity.

Table 2.5 Limits of size for positions of cast-in parts and connectors in precast concrete elements (to DIN 18203, table 1).

	Limits of size **Δ*L*** in (mm) for nominal dimensions ***L*** in (m)							
	≤1.5	**>1.5 to ≤3.0**	**>3.0 to ≤6.0**	**>6.0 to ≤10.0**	**>10.0 to ≤15.0**	**>15.0 to ≤22.0**	**>22.0 to ≤30.0**	**>30.0**
Cast-in parts in linear concrete elements	±6	±8	±10	±12	±14	±16	±18	±20
Cast-in parts in prestressed concrete elements	—	—	—	±16	±16	±20	±25	±30
Cast-in parts in floor and wall elements	±8	±8	±10	±12	±16	±20	±20	±20
Cast-in parts in façade panels	±5	±6	±8	±10	—	—	—	—

Table 2.6 Limits of size for positions of cast-in parts and connectors in the structure (to DIN 18202, table 1).

	Limits of size **Δ*L*** in (mm) for nominal dimensions ***L*** in (m)					
	≤1.0	**>1.0 to ≤3.0**	**>3.0 to ≤6.0**	**>6.0 to ≤15.0**	**>15.0 to ≤30.0**	**>30.0**
Cast-in parts on plan[a)]	±10	±12	±16	±20	±24	±30
Cast-in parts on elevation[a)]	±10	±16	±16	±20	±30	±30
Cast-in parts on plan between two components	±12	±16	±20	±24	±30	—
Cast-in parts on elevation between two components	±16	±20	±20	±30	—	—

a) Related to global dimensions, e.g. grid dimensions.

Calculations for fit can be carried out in various ways:

– Calculations for fit according to the additive method

The maximum values of all individual tolerances are added together. This results in a maximum probability of fit, but also a maximum chance of error, e.g. a maximum joint width.

$$\delta_{\text{comb}} = \sum \delta_i \tag{2.1}$$

where

δ_{comb} total design tolerance

δ_i all tolerances in the process chain

– Calculations for fit taking into account a statistical allowance for the propagation of errors

The more individual tolerances meet at one point, the lower is the probability that the maximum values of all individual tolerances will occur at that point. Therefore, tolerances can be added geometrically according to a Gaussian distribution of propagation of error to produce a total design tolerance taking into account the statistically probable coincidence of individual tolerances.

$$\delta_{\text{comb}} = \sqrt{\sum (\delta_i)^2} \tag{2.2}$$

The following approach according to [5] combines both methods, and experience shows that it supplies sufficiently accurate results:

$$\delta_{\text{comb}} = \delta_{\max}\sqrt{\sum (\delta_i)^2} \tag{2.3}$$

where

δ_{comb} total design tolerance
$\delta_{\max}$ maximum tolerance across the entire process chain
δ_i every other tolerance in the process chain

A suitable method should not be chosen arbitrarily, instead according to the actual situation. Where many steps in the process chain are carried out by one company, the recommendation is to perform calculations for fit according to the statistical propagation of errors. But where each individual step is carried out by a different company (contractor, precast concrete supplier, erection specialist), it is only possible to apply the propagation of errors method when agreement between all those involved is reached beforehand regarding the interfaces and tolerances of the individual trades.

Calculations for fit must take into account not only the production inaccuracies for the precast concrete elements themselves, but also inaccuracies in the measurement and construction of the structural carcass. Inaccuracies in prior trades may only be omitted from the calculation when the as-built structural carcass is measured and these measurements are taken into account in the production of the precast concrete elements. As that approach has a considerable influence on the whole sequence of construction, carrying out such measurements must be carefully coordinated with all those involved in advance.

Figure 2.6 (with on-site measurement) and Figure 2.7 (without on-site measurement) show the results of calculations for fit to determine joint widths between façade panels. The joints in Figure 2.7 are wider because in this case the inaccuracies of the structural carcass are unknown and hence must be taken into account in the calculations for fit.

2.3 Production

Modern production methods in conjunction with CAD/CAM support permit good flexibility and variability combined with short production processes. Properly developed standardisation of components and connections helps to avoid misunderstandings and achieve cost-optimised building operations. Repetition

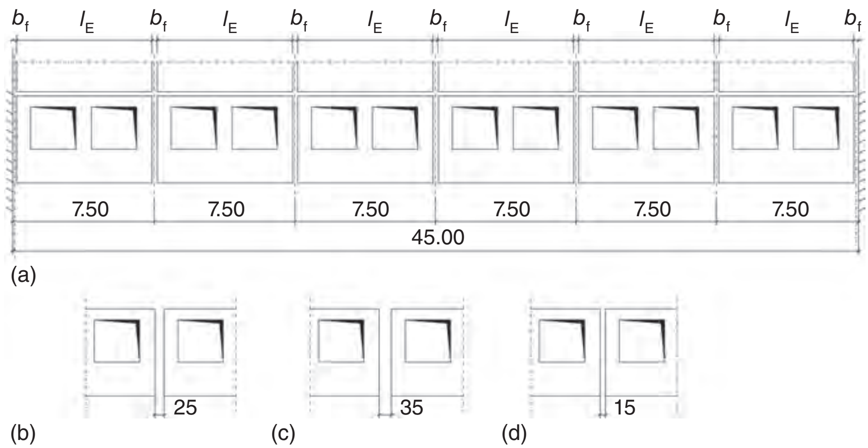


Figure 2.6 Joint widths following measurement on site: (a) elevation on and dimensions of façade, (b) nominal joint width, (c) maximum joint width, and (d) minimum joint width. Source: Taken from Ref. [5].

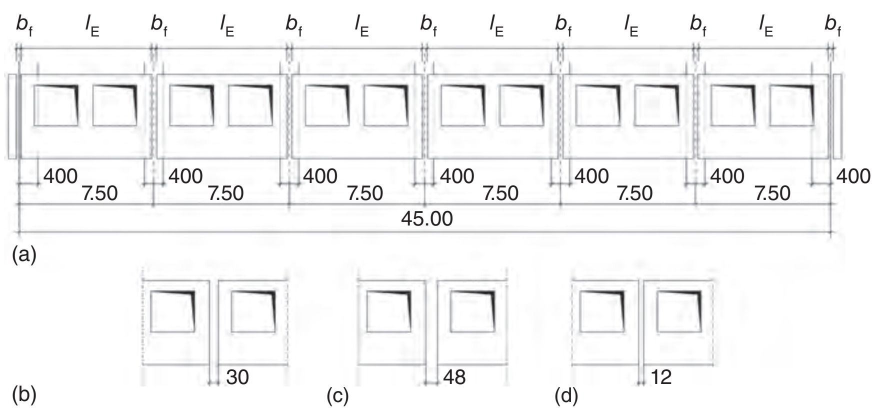


Figure 2.7 Joint widths without prior measurement on site: (a) elevation on and dimensions of façade, (b) nominal joint width, (c) maximum joint width, and (d) minimum joint width. Source: Taken from Ref. [5].

and learn effects plus experience and routine lead to enormous time-savings at the factory and on site, which not only reduce the costs but also improve quality and reliability and, in the end, simplify transport and erection.

Standard solutions could be

- The use of standardised cross-sections and connections
- Standardised production sequences
- Modular forms of construction (e.g. unitised or room module principles).

Over the decades, certain component cross-sections have proved to be particularly advantageous and versatile (see Chapter 4). These cross-sectional forms, and hence the corresponding rationalisation effects, have therefore become established. However, precast concrete elements are not mass-produced goods, instead bespoke components, because even the smallest changes (cross-section dimensions, lengths, cast-in parts, or openings) lead to different mould and reinforcement requirements.

For this reason, the terms ‘series’ or ‘batch’ should not be misunderstood and not confused with the serial production of other branches of industry (see [11, 12] for system building and industrialisation).

It is difficult to specify figures for minimum batch sizes in order to achieve an economic construction project. In particular, forming batches has a great influence on technical working requirements and mould costs. In turn, the shapes of precast concrete elements, and hence the cost of their development, mould assembly and the production itself, have a considerable influence on batch sizes. The most favourable conditions occur with an uninterrupted production flow (Figure 2.8). When using cast-in parts, standard parts should be used as far as possible because these can be obtained from stocks at short notice.

The production processes at the precasting plant differ fundamentally from conventional construction on the building site in many ways. The use of system moulds reduces the costs of moulds and production and increases productivity. The production work required for every individual precast concrete element, and hence the risk of potential errors, is thus minimised.

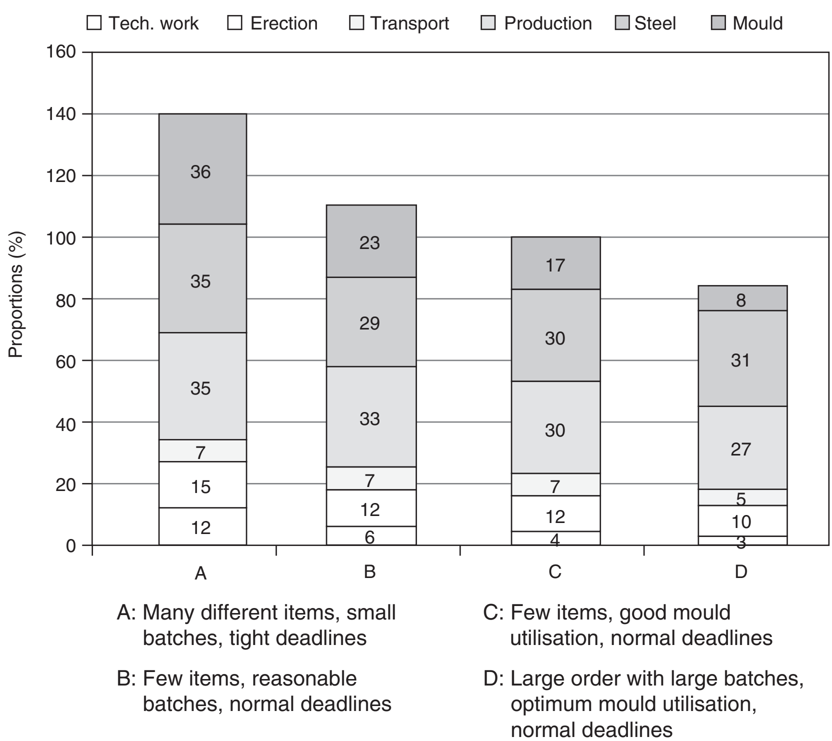


Figure 2.8 Costs structure of a precast concrete multistorey building depending on batch sizes and timetable. Source: Taken from Ref. [1].

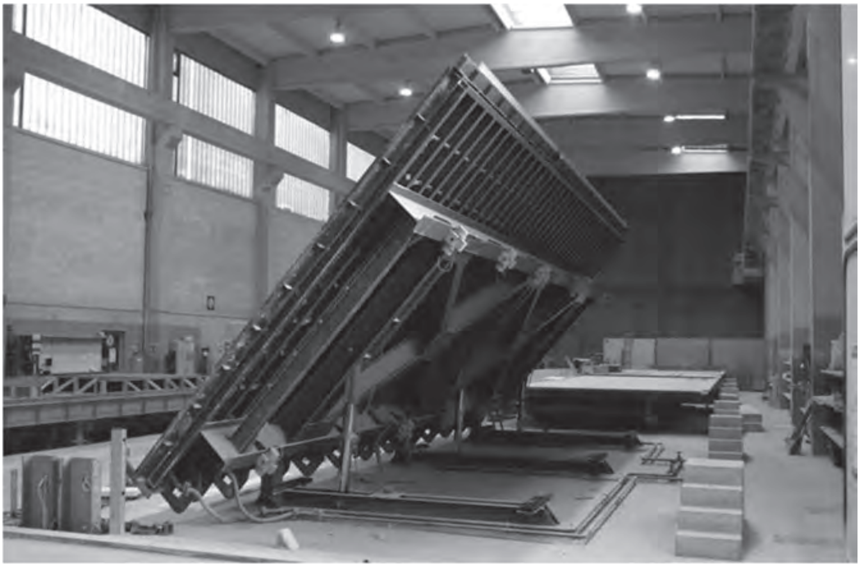


Figure 2.9 Tilting table.

Surfaces cast against the sides of the moulds of course differ from surfaces exposed to the air during casting, irrespective of whether timber or steel moulds or form liners are used. Therefore, surfaces not cast against mould sides often require additional work in the form of floating, rubbing, trowelling, or rolling.

Planar precast concrete elements such as those for floors or walls are mostly cast on tilting tables, with only one side cast against the mould (Figure 2.9). Battery moulds, which enable both sides of a wall element to be cast against the mould, are not common in precasting plants any more.

The joint between the side and base of a mould must be sealed to prevent concrete seeping into this joint. Triangular plastic fillets are normally used for this, which is why the edges along the undersides (in the sense of the production process) of precast concrete elements are generally chamfered. There must be a clear indication on the drawings if the top edges (in the sense of the production process) are to be chamfered as well.

Rectangular beams or T-beams are often cast in rigid moulds. In these cases, the sides of rectangular beams or the webs of double-T sections are inclined slightly outwards so that such elements can be easily lifted out of the mould once they have cured (Figure 2.10). Where connections are to be left visible, then such production-related properties of precast concrete elements must be considered at the design stage.

Columns are mostly cast lying horizontal in a mould so that the side from which the column is concreted is generally the open side of the mould. Where a column has corbels facing in different directions, then coordination with the factory is required to establish from which side the column can or should be concreted (Figure 2.11).

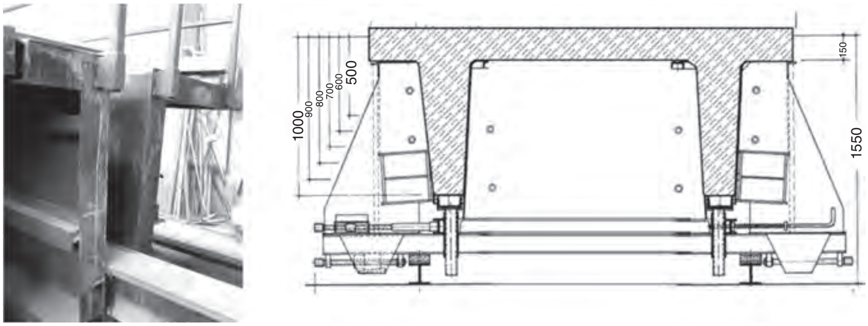


Figure 2.10 Mould for a double-T section.

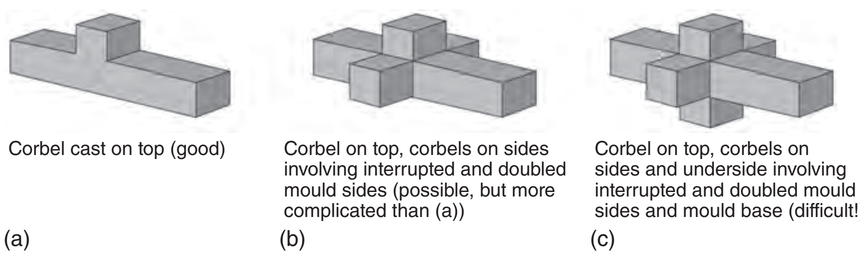


Figure 2.11 Horizontally cast columns: (a) with corbel on top only, (b) with corbels on three sides, and (c) with corbels on all four sides.

Reference [1] contains a comprehensive range of standard forms plus tables of load-carrying capacities for precast concrete elements. Therefore, right from the design phase, it is possible to determine the necessary cross-section dimensions quickly and economically, and these can then serve as the basis for the costing (Figure 2.12).

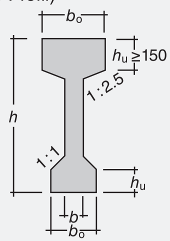
Corbels, notched beam ends, forked supports, shear dowels, and elastomeric bearings are construction details that recur constantly in precast concrete construction, which results in another repetition effect and reduces the amount and cost of the work. However, standardised, categorised connections such as those used in structural steelwork cannot be implemented in precast concrete construction. Full information and details plus typical examples of connections between precast concrete elements can be found in [1, 3].

2.4 Transport and Erection

2.4.1 General

The production, transport, and erection of a structure plus the way it is subdivided into prefabricated parts all influence each other and therefore must be taken into account right from the start of the design work. A building or part of the building must be subdivided into producible, transportable, and erectable units. Important factors here are the maximum transport dimensions, transport

Blatt 3: Binder (I-Profil)



Ausführungen als Parallel-Binder oder als Satteldach-Binder mit 5% Neigung. im Normalfall ohne Auflagervouten
Abfasungen: gebrochen. Katheten je 10 mm für untere Untergurtkanten
Alle Abmessungen ausreichend für Feuerwiderstandsklasse F 90-A nach DIN 4102-4

Querschnittswerte (mm)			
h	b_o	b	h_u
800	400	120	150
1000	400	120	150
1200	500	120	160
1400	600	120	250
1600	700	120	250
1800	800	120	250
200	800	150	350
2200	800	150	350
2400	800	150	350

Spann-weite l	Abstand a	Binderhöhe h (mm) bei Einwirkungen $g_{k,j} + q_{k,l}$ (kN m^{-2})								
(m)	(m)	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
15.0	5.0									
	6.0		600				800		1000	
	7.5					1000			1200	
	10.0		800	1000		1200			1400	1600
20.0	5.0				1000				1200	
	6.0		800						1400	
	7.5				1200		1400		1600	
	10.0		1000	1200	1400			1600	1800	2000
25.0	5.0				1200		1400		1600	
	6.0		1000					1600		1800
	7.5				1400	1600	1800		2000	
	10.0		1200	1400	1600	1800	2000			
30.0	5.0					1600		1800		2000
	6.0		1400				1800	2000		
	7.5		1600		1800	2000				
	10.0			1800	2000					
35.0	5.0					1800	2000			
	6.0				1800	2000				
	7.5		1600	1800	2000					
	10.0		1800	2000						
40.0	5.0									
	6.0									
	7.5		2000			I-Binderprofil wählen (s. nächste Seite)				
	10.0									

Figure 2.12 Extract from FDB (Fachvereinigung Deutscher Betonfertigteilbau e.V., German Precast Concrete Construction Association) range of standard elements and table of load-carrying capacities. Source: Taken from Ref. [1] (available in German only).

(a) (b)

Figure 2.13 (a) Unhindered erection with mobile and crawler-mounted cranes and (b) erection in the city with tower cranes. Photo: Architekturforum.

weights, and transport routes plus the feasible erection weights and the lifting equipment available at the factory and on the building site.

The smaller the elements or the larger the batches, the greater are the costs of transport and erection, for fixing and connecting the individual elements together and for producing the joints. Therefore, elements in the maximum sizes possible should be the goal in order to minimise the aforementioned work in the factory and on the building site and avoid unnecessary costs.

Local conditions and methods of erection are extremely important. On greenfield sites, constraints on the building operations will be minimal (Figure 2.13a). But when working in heavily built-up urban areas, the confines of the site might make it necessary to use a tower crane for all lifting operations, which means that the design will have to be based on smaller elements owing to the lower safe working loads of such cranes (Figure 2.13b).

2.4.2 Transport

In Germany, the maximum dimensions (length, width, height) and maximum total weights of vehicles or combinations of vehicles are laid down in the *Straßenverkehrs-Zulassungs-Ordnung* (StVZO, road traffic licensing regulations; Table 2.7). Taking the permissible values given in those regulations, the result is the following maximum possible dimensions for precast concrete elements:

– Max. length for conventional tractor units = approx. 12.5 m
– Max. width = 2.50 m

Table 2.7 Permissible dimensions and total weights for road transport (depends on the respective approving authority).

	Without special permit (to StVZO art. 32)	**With annual permit (to StVO art. 29 and art. 70)**
Width (m)	2.55	3.00
Height (m)	4.00	4.00
Length (m)	15.50	24.00
Total weight (t)	40 (to art. 34)	48 (tractor unit with self-steering trailer)

4

Precast Concrete Elements

4.1 General

Over the decades, special forms and systems have been developed for many components of precast concrete construction. Those described in this chapter have proved to be very economical and flexible. The Fachvereinigung Deutscher Betonfertigteilbau e.V. (FDB, German Precast Concrete Construction Association) has developed a range of standard elements that are recommended for tenders and planning purposes [1]. The figures in this section illustrate the typical cross-sectional dimensions of this range of standard elements.

4.2 Floor and Roof Elements

4.2.1 General

Many systems have been devised for precast concrete floors, e.g. solid slabs, slabs with voids, floor plates with in situ concrete topping or floor elements with ribs or webs. Figure 4.1 shows a general overview.

4.2.2 Solid Slabs

Solid slabs are frequently employed as balcony slabs and in residential buildings in the form of room-sized floor elements with lengths ≤4.5 m and widths ≤3.0 m. Precast solid floor elements are designed and reinforced in the same way as in situ suspended floor slabs and are hence analysed according to DIN EN 1992-1-1. Section 9.3.1.1 (NA.5) of that standard specifies 70 mm as the minimum depth of a solid slab although this value is considerably exceeded in most cases. The disadvantage of solid slabs for transport is their high self-weight.

Connections between floor elements are dealt with in Sections 6.12 and 6.13.

4.2.3 Hollow-Core Slabs

4.2.3.1 General

Hollow-core slabs are industrially manufactured floor systems that are very popular for precast concrete construction and prove very economical in the case of larger batch sizes. The circular, oval, or even rectangular voids result in savings

Precast Concrete Structures, Second Edition. Alfred Steinle, Hubert Bachmann, and Mathias Tillmann.

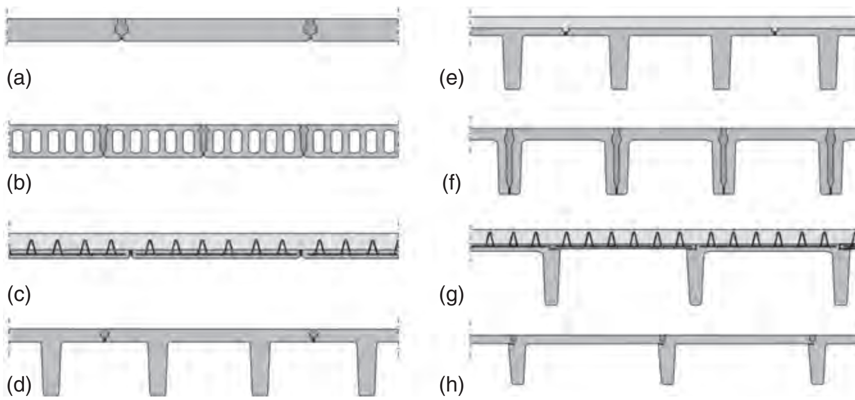


Figure 4.1 Overview of various precast concrete floor systems.

in materials and weight of up to 50% compared with solid slabs, which means that longer spans and/or thinner slabs, and hence lower overall depths, can be achieved. We distinguish between conventionally reinforced and prestressed concrete hollow-core slabs, with the latter being very popular in Germany.

4.2.3.2 Prestressed Hollow-Core Slabs

Prestressed concrete hollow-core slabs are manufactured on prestressing beds up to 150 m long with strands of prestressing steel as the sole means of reinforcement. They are cast using slipformers or extruders that provide formwork, concreting, and compacting functions simultaneously. Concrete strength classes of C45/55 are typically achieved. After curing, the individual elements are cut to length from the long ribbon of material (see also Section 8.1.2).

Prestressed hollow-core slabs can be used for many applications and include multistorey car parks as well as office and residential buildings. Owing to the method of manufacture, prestressed hollow-core slabs span in one direction. Spans of up to about 18 m are possible with a slab depth of about 400 mm. Table 4.1 shows an overview of prestressed hollow-core slabs with various depths and load-carrying capacities.

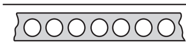
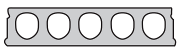
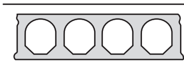
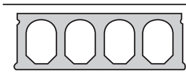
The method of manufacture results in a standard width of 1.20 m. However, make-up slabs with a width $b < 1.20$ m can also be produced. Angled cuts on plan are possible up to an angle of 60° with respect to the longitudinal axis. Cantilevers in the direction of the span are normally limited to eight times the slab depth. Figure 4.2 shows a prestressed hollow-core slab being installed.

The maximum permissible uniformly distributed imposed load is normally 10 kN m^{-2}. This may be increased to 12.5 kN m^{-2} for a slab depth $d > 250$ mm. In emergency situations, prestressed hollow-core slabs can also support the weight of heavy fire-fighting vehicles.

Openings are cast directly in the slabs at the precasting plant and must be checked for their effects on the structure. Larger openings will require steel trimmer beams. Holes drilled after casting or openings cut for services at a later date are only permitted in the areas of the voids because otherwise there is a risk of damaging, even severing, individual prestressing strands.

The eccentric prestressing causes a pre-camber of the slab; the maximum deviation from the calculated pre-camber is specified as ±1/1000 (1 = span). Different

Table 4.1 Range of prestressed concrete hollow-core slabs according to [2].

Floor type					
Cross-section	***h* (cm)**	**Self-weight[a)], *g* ($kN\,m^{-2}$)**	**Fire resistance rating**	**Bending moment[b)], M_{Rd} ($kNm\,m^{-1}$)**	**Shear force, V_{Rd} ($kN\,m^{-1}$)**
	15–16	2.70	F30	76	62
			F90	66	55
	20	3.30	F30	140	63
			F90	119	60
	26–27	4.00	F30	250	75
			F90	242	68
	32	4.70	F30	333	91
			F90	333	86
	40	5.30	F30	460	126
			F90	460	145

a) Guidance figures for the self-weight of the finished floor including grouted joints.
b) Guidance figures for the ultimate limit state for exposure class XC1.

Figure 4.2 Positioning a prestressed hollow-core slab unit.

deformations of neighbouring elements can occur with thin cross-sections in particular. Special measures (special alignment tools) can reduce such vertical misalignment, but not eliminate it completely. One solution is to choose thicker elements with a correspondingly lower prestress to reduce the pre-camber and hence any vertical misalignment. In principle, the recommendation is to agree permissible limits of size for vertical misalignment with the manufacturer beforehand.

The omission of conventional reinforcement because of the method of production means that the tensile strength of the concrete has to be included in the calculation in order to achieve the necessary shear strength, especially at the supports, and to distribute the loads in the transverse direction. This applies, in particular, when supporting precast concrete floors on a flexible support, e.g. steel beams or reinforced concrete downstand beams, because high transverse tensile stresses ensue as a result of the deflection of the beams.

When using flexible supports, the applied shear force V_{Ed0} may not exceed 50% of the design value of the shear capacity $V_{Rd,c}$:

$$V_{Ed0} \leq 0.5 \cdot V_{Rd,ct} \tag{4.1}$$

Furthermore, the deflection of the flexible support for actions applied with a safety factor $\gamma_f = 1.0$ may not exceed 1/300. Bearing strips on flexible supports must be at least 35 mm wide and 10 mm thick. Studies of flexible supports can be found in [3–6].

Owing to the high level of prestress in the elements designed to be free of cracks at the serviceability limit state, the concrete tensile strength $f_{ctk;0.05}$ should generally not be exceeded within the transmission length 1_{pt2} at the ultimate limit state. Therefore, no further analyses of the anchorage are required (see Section 6.18.7).

Table 4.2 summarises the main features of prestressed hollow-core slabs [7].

Until now, national technical approvals for the use of prestressed hollow-core slabs were required in Germany in addition to the European product standard DIN EN 1168 ‘Precast concrete products – Hollow-core slabs’. Following a ruling by the European Court of Justice, these approvals are no longer possible since October 2016 (see Section 4.4). However, the technical content of the national technical approvals may still be taken into account when assessing usability [8]. In addition, an industrial directive [9] is intended to provide information on which analyses are required for assessing prestressed hollow-core slabs.

4.2.3.3 Conventionally Reinforced Hollow-Core Slabs

Conventionally reinforced hollow-core slabs are produced in widths of up to 2.50 m on steel pallets in the desired lengths in a special plant in which augers

Table 4.2 Features of prestressed hollow-core slabs.

Floor elements	Design advice
 Prestressed hollow-core slabs	– Spans up to about 18 m – Standard width 1.20 m – No temporary propping required – Suitable for foot traffic immediately after erection – Flexible routing of services and flexible usage options – Available as slim-floor slab (with beams within the depth of slabs) – Low self-weight – Finished, smooth soffit – Penetrations and trimmer beams must be carefully planned – Make-up elements required for irregular plan forms

push the concrete through a rectangular die matching the dimensions and geometry of the slab cross-section. In contrast to prestressed hollow-core slabs, longitudinal and transverse reinforcement and even shear stirrups are possible.

Spans of 6–7 m supporting loads of 5 kN m^{-2} are possible for slab depths between 140 and 200 mm. Spans of up to 10 m are possible with a slab depth of 300 mm. The longitudinal edges are cast with keyed joints where necessary so that in-plane and out-of-plane forces can be transferred across the joints between elements. Conventionally reinforced hollow-core slabs can always be erected without the need for any temporary propping.

Essentially, they can be analysed according to Eurocode 2. Until now, principles for structural verification were provided by the DIBt on the basis of DIN 1045-1 [10]. A revised edition of a directive containing design and detailing information for conventionally reinforced hollow-core slabs is to be drafted on the basis of DIN EN 1992-1-1.

Reference [11] describes tests on joints between conventionally reinforced hollow-core slabs. These floor systems were designed for loads that are not predominantly static, for uniformly distributed loads of up to $q \leq 12.5$ kN m^{-2}, and forklifts up to 35 kN.

4.2.4 Precast Floor Plates With In Situ Concrete Topping

4.2.4.1 General

Precast floor plates with in situ concrete topping are traditional precast concrete items that are completed on site with an in situ concrete topping. The intention here is to combine the advantages of 'pure' precast concrete construction with those of in situ concrete construction. Those advantages are as follows:

- Factory production of the precast concrete floor plates with in situ concrete topping
- Eliminating or reducing the cost of formwork on site
- Construction of a simple floor diaphragm and simple transverse distribution of the loads by adding additional reinforcement within the in situ concrete topping
- Creation of surfaces without joints
- Creation of a monolithic component
- Owing to the lower self-weight of the precast concrete floor plates with in situ concrete topping, the lifting plant needed is lighter than that for 'pure' precast concrete construction.

However, the advantages of a 'pure' precast site ('dry' site) are lost, meaning that, for example, winter conditions might prevent concreting.

The rational production of precast plank floors on automated production lines and their good adaptability, with almost any geometry possible, has made them very popular in Germany. The elements consist of a precast concrete plank at least 50 mm thick, which functions as permanent formwork for the in situ concrete topping. To simplify handling of these thin units during transport and erection, they are provided with rigid steel reinforcement in the form of lattice beams.

The top chord of each lattice beam functions as a compression member in the temporary erection condition, but is not normally classed as top tension reinforcement in the final condition. Where tension reinforcement in the top of the slab is required to achieve a continuity effect, additional reinforcement must be laid above the top chord of each lattice beam. The bottom chords can be classed as bottom tension reinforcement. The diagonal bars connecting the top and bottom chords serve as shear connectors and, if necessary, as shear reinforcement. Any additional reinforcement required for the diaphragm action of the slab is laid in the in situ concrete topping, making perimeter tie beams unnecessary.

Depending on the surface finish of the joint between the precast and in situ concrete, precast plank floors can be classed as monolithic slabs in the final condition designed according to the provisions of DIN EN 1992-1-1. National technical approvals or European technical approvals (ETAs) are available for various types of lattice beam, and these approvals may need to be taken into account in the design.

Precast plank floors can also be designed as floors spanning in two directions. In these cases, the longitudinal reinforcement in the direction of the primary span, and which is installed in the precast plank, is supplemented by transverse tension reinforcement on top of the precast plank. However, according to DIN EN 1992-1-1/NA, section 10.9.3 (NA.14), only reinforcement that is continuous or has adequate laps can be considered for action effects at 90° to the joint. The conditions for considering spliced reinforcement are as follows:

- Reinforcing bar diameter $Ø \leq 14$ mm
- Reinforcement cross-section $a_s \leq 10\,cm^2\,m^{-1}$
- Design value of shear force $V_{Ed} \leq 0.3\ V_{Rd,max}$.

Furthermore, the joint must be reinforced with stirrups or lattice beams, for example, at a spacing $\leq 2h$ (h = slab depth).

When determining the internal forces, then according to DIN EN 1992-1-1/NA, section 10.9.3 (NA.15), the beneficial effect of the torsional rigidity may only be considered when there are no butt joints between precast concrete floor plates with in situ concrete topping within the torsion zone of 0.3 l from the corner or when lattice beams are positioned max. 100 mm from the edge of the joint. Capacity to resist the twisting moment only has to be verified if the slab is rigidly connected to edge beams or neighbouring floor bays.

Precast plank floors are frequently provided in conjunction with conventionally reinforced or prestressed concrete beams in the form of T-beams (Figure 4.3). However, slabs supported at discrete points are also possible, which requires special lattice beams to resist punching shear.

Precast plank floors require temporary propping during erection, which means that one key advantage of precast concrete construction is lost. Prop spacings exceeding 5 m are usually only possible with very closely spaced lattice beams or special lattice beams where the top chord is replaced by a channel-shaped buckling-resistant sheet metal profile (Figure 4.4). This type of suspended floor is especially economical with high storey heights if the extra cost of the lattice beam is lower than the cost of providing temporary propping. The system is also approved for dynamic actions [12].

Design for the temporary erection condition cannot be carried out according to the conventional design principles of Eurocode 2, instead must be considered as

Figure 4.3 Precast concrete floor plates with in situ concrete topping supported on primary and secondary downstand beams.

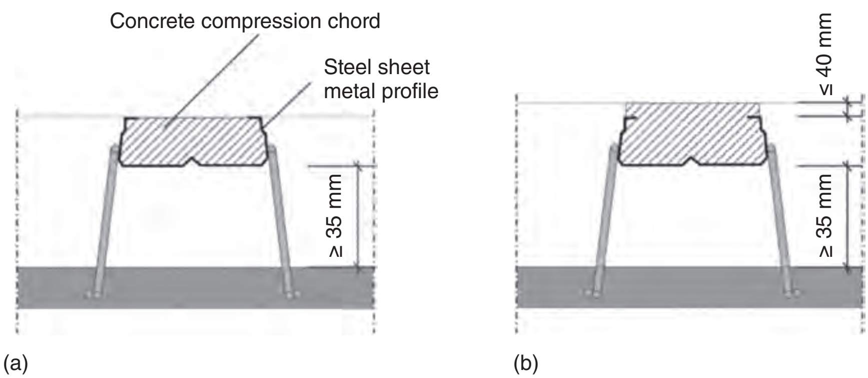


Figure 4.4 Lattice beams for floor elements that remain rigid during erection. Source: Montaquick system.

a combination of the truss action of the lattice beam and the structural response of the precast concrete plank. An analytical model is not yet available. The prop spacings given in the approvals are therefore based on tests [13]. The European product standard DIN EN 13747 describes tests for determining prop spacings. However, these have not been applied in Germany so far.

Table 4.3 summarises the main features of precast floor plates with in situ concrete topping [7].

4.2.4.2 Prestressed Precast Floor Plates With In Situ Concrete Topping

Prop spacings of up to 8 m can be achieved when using prestressed precast floor plates with in situ concrete topping – a special form of the precast concrete plank. To do this, the floor plates with in situ concrete topping are prestressed with strands or profiled wires approved for pretensioning and positioned at roughly

Table 4.3 Features of precast floor plates with in situ concrete topping.

Floor elements	Design advice
 Precast concrete floor plates with in situ concrete topping	– Spans up to about 8 m (prestressed floor plates with in situ concrete topping up to about 15 m) – Often suitable for irregular plan layouts and/or large openings – Suitable for dynamic loads (only with special lattice beams) – In situ concrete topping results in floor surface without joints – Temporary propping generally required – Finished, smooth soffit – Low self-weight of floor plates with in situ concrete topping (for transport and erection) – Large proportion of in situ concrete (progress on site)

mid-depth. Such precast floor plates with in situ concrete topping are at least 50 mm thick, but normally 80–100 mm.

Owing to the longitudinal expansion, prestressed precast floor plates with in situ concrete topping with lattice beams result in restraint effects, which means that their use is restricted to lattice beams in segments with a length of max. 1.0 m or lattice beams with an interrupted top chord. Alternatively, conventional shear stirrups can be used. The shear connection is achieved via bond, although at supports, structural shear connectors amounting to 2 $cm^2\ m^{-1}$ must be provided over a length of 0.5 m. Prestressed precast floor plates with in situ concrete topping are regulated exclusively via national technical approvals.

4.2.5 Ribbed Elements

Ribbed elements, either conventionally reinforced or prestressed, are used for heavy loads and long spans (Figure 4.5). They are cast in long steel moulds or on a prestressing bed and can be used with or without an in situ concrete topping (Figure 4.6).

Ribbed elements are produced in widths of up to 3.0 m, overall depths of 300–1100 mm and lengths of up to 25 m (Table 4.4). The webs, usually at a spacing of 1.20 m, have a 1 : 20 slope so that the elements can be easily lifted out of the rigid moulds after curing without adhering to the sides. The side panels of the moulds can be adjusted to suit the respective slab width.

Ribbed elements designed for use with an in situ concrete topping are cast with a 60-mm-deep flange that serves as permanent formwork for the concrete added