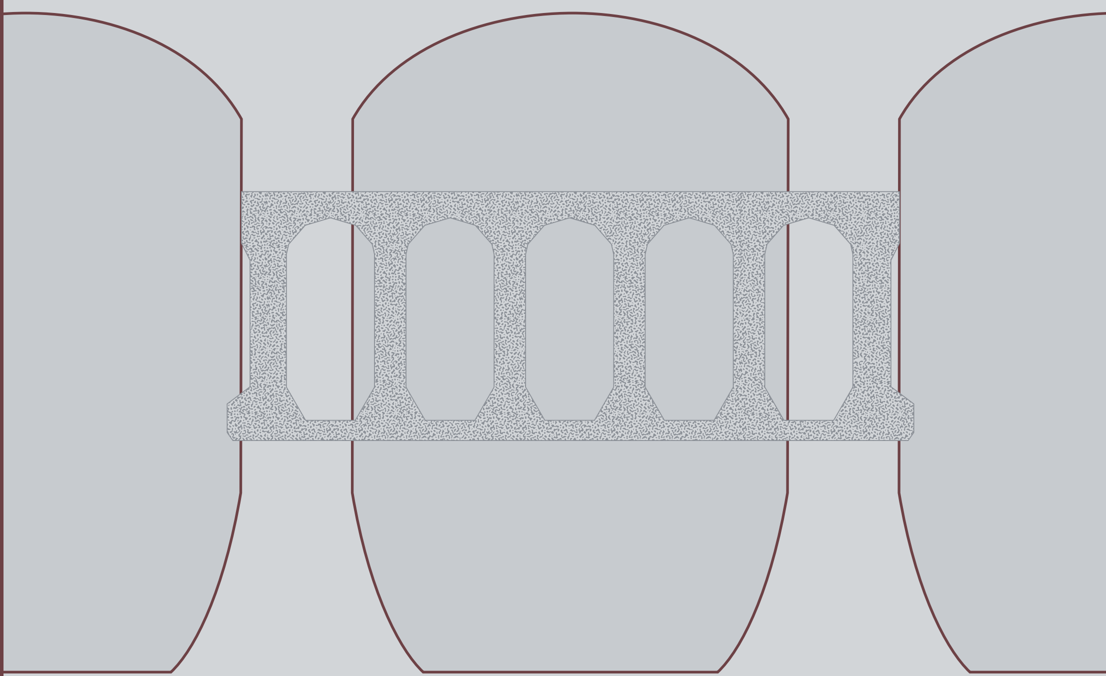
The Hollow Core Floor Design and Applications



ASSAP
hollow core slabs

Association of Manufacturers of Prestressed Hollow Core Floors

The Hollow Core Floor Design and Applications

Manual ASSAP

1st Edition

ASSAP

hollow core slabs

ASSOCIATION OF MANUFACTURERS OF PRESTRESSED HOLLOW CORE FLOORS

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ASSAP

THE ASSOCIATION OF MANUFACTURERS OF PRESTRESSED HOLLOW CORE FLOORS

"A non-profit Association for the promotion, safeguarding and defence of the hollow core floor and the legitimate interests of associated producers" (from Article 2 of the Articles of Association).
"Associates, at the time they are admitted, commit themselves to orient their company's policies in the direction of quality and to respect the technical and ethical criteria established by the Association" (from Article 4 of the Articles of Association).

ASSAP was founded in June 1982 in Ponte Taro (Parma, Italy) with the participation of almost half of the producers then present on the Italian market. The guiding idea was to promote and enhance the prestige of the prestressed hollow core floor.
The members of ASSAP, in alphabetical order, are the following companies, some of which (in italics) no longer exist or have left the Association for having ceased the production of hollow core floors:
ANTARES in Frosinone, *BONETTI* Prefabbricati in Castenedolo (Brescia), *CEMENTEDILE* in Lauriano Po (Turin), CENTRO ITALIA PREFABBRICATI in Frosinone, *CONCARI* Prefabbricati in Parma, DIGNANI Prefabbricati in Montecassiano (Macerata), *EDILCEMENTO* in Gubbio (Perugia), EDILGORI Precompressi in Terni, E.P. EDILIZIA PREFABBRICATA in Pomezia (Rome), ESSE SOLAI in Dueville (Vicenza), EUROPREFABBRICATI in Castellalto (Teramo), GIULIANE SOLAI in Ruda (Udine), HORMIPRESA in Tarragona (Spain), IAPITER in Avellino, ICIENNE in Arezzo, IMMOBILIARE CENTRO NORD in San Martino B.A. (Verona), INPREDIL in Masserano (Biella), *INPREVIB* in Chivasso (Turin), LATERIZI FAUCI in Sciacca (Agrigento), MARCHETTI & MORANDI in Ponte Buggianese (Pistoia), *MUBEMI* in Valencia (Spain), PAVICENTRO in Aveiro (Portugal), PAVINORTE in Penafiel (Portugal), PRECOMPRESSI CENTRO NORD in Cerano (Novara), *PRECOMPRESSI METAURO* in Calcinelli di Saltara (Pesaro), PRETENSADOS INDUSTRIALES in Santo Domingo (Rep. Dominicana), R.D.B. in Piacenza, S.G.C. in Taranto, *S.I.C.S.* in Lodi, *SUN BLOCK* in Kuala Lumpur (Malaysia), *VIBROCEMENTO SARDA* in Cagliari.

Soon after its creation, ASSAP turned to Prof. Franco Levi of the Politecnico of Turin, for his expert advice. He strengthened the scientific basis of the engineering techniques and applications that the Association's proposers, belonging to the Gruppo Centro Nord, had previously developed and shared with all ASSAP members.
From 1982 to 1986 the testing laboratory of the Politecnico of Turin directed by Prof. Pier Giorgio Debernardi devoted its energies to the experimental testing of the restraint of continuity established between hollow core floors on several supports by means of normal reinforcement resistant to negative moment and inserted in situ in the slab ends prepared specifically for the purpose.

The second task was the study of the mechanical model to explain the unexpected experimental behaviour of the restraint of continuity between hollow core slabs during the cracking phase. Effectively, once the positive and negative moments of cracking had been reached and passed experimentally in the laboratory, it was noted that these cracks never joined one another and thus caused no structural collapse.
It was found that cracks remained separate owing to the presence at the ends of the arch and tie system of compression struts in the concrete (see Fig. 4.10 in Chapter 4) which inhibited their coming together. Thus collapse is avoided in the cracking phase.

Prof. Franco Levi - Politecnico of Turin (Italy)

Thanks to this formidable and reassuring scientific diagnosis, Prof. Levi opened the doors to Italian, and later European, Codes dealing with hollow core floors laid in continuity.

Among the many innovative applications introduced by ASSAP we also find the clear span connection between hollow core slabs and bearing beams cast in situ (see paragraphs 4.4.2 and 4.4.3).
What are the conditions within which these connections can be assured without support?

Once again it was the testing laboratory of the Politecnico of Turin that addressed this new research challenge through the construction of beam models, both depressed and in floor thickness, cast in situ, with hollow core slabs in continuity but not lying on the beam itself.
Results of the tests confirmed the validity of the engineering idea, although with the limits and precautions dictated by Prof. Levi (see paragraph 4.4.4.).

The last research project, which dealt with spalling stresses (see paragraph 3.5.2), required a three-year effort. If in normal prestressed beams vertical tensions in the web-end are absorbed by the specific stirrups, in hollow core slabs they must be opposed by the tensile strength of the concrete alone.
Spalling stresses must also be specifically restricted if the hollow core slab is inserted as a clear span between bearing structures cast in situ.

This "manual", which represents "Self-Regulation Document" for companies producing prestressed hollow core slab floors and ASSAP associates, is an instrument containing the knowledge acquired by the Association through specific studies and research and which has supplied to associates the know-how necessary not only for the production, but also for the design of hollow core slab floors on innovative and precise technological and scientific bases.

FOREWORD

After thirty years of continuous and enthusiastic work in a specific field, a technician unknowingly and inevitably becomes a specialist in that sector and finds what he has been dealing with for many years so obvious that he or she is dumbfounded when professional colleagues do not show the same level of expertise in such a congenial subject.
In the case of the technicians who formed the nucleus that led to the founding of ASSAP, they were too often perplexed by the inaccuracy of some producers and many designers in the specific field of the production and application of hollow core floors.
For these reasons, starting from the 1980s, ASSAP began thinking of writing a "manual" in which to state the principles for the correct design and application of this universally known component, which is sometimes not fully appreciated owing to preconceptions and improper applications.

The sum of the experience gained by the technicians of the ASSAP committee was found to be so vast that it could not be contained in a sort of "instant guide to..." because while putting it into hard copy form it became more like a "treatise". The obvious consequence was that its preparation would require far more time and many more revisions than were originally planned.
The book you are now reading is thus a complete compendium, perhaps even too detailed, but undoubtedly useful, of important information providing in-depth knowledge of the hollow core floor and its prefabricated component which is the prestressed hollow core slab.

The purpose of this publication is thus to provide designers, producers and users of hollow core floors with an instrument to assist them in finding solutions to problems they come across professionally, problems that must be solved by bringing together theory and codes with correct constructive intuition taking into account the real necessities of practical construction work.
Over the years, designers have developed many innovative engineering solutions in the use of this prefabricated element. These must be well understood before its special characteristics can be fully exploited while

maintaining safe structural conditions and complying with the rules of good building practice.

With this publication, ASSAP, the Association of Manufacturers of Prestressed Hollow Core Floors, has brought together the general design criteria, which have been amply verified experimentally, to provide designers with a practical instrument for use when dealing with all morphological types of hollow core slabs. Methods of calculation are standardized and practical rules for implementation in conformity with Italian and European codes now in force are given.

With great dedication, the following technicians participated in the preparation of this publication. In doing so they have earned the unconditional gratitude of ASSAP:
Gennaro Capuano, Bruno Della Bella, Pierluigi Ghittoni, Piercarlo Morandi and Stanislaw Pereswiet-Soltan.

ASSAP offers special thanks to Prof. Franco Levi, Prof. Pier Giorgio Debernardi, Prof. Crescentino Bosco, Prof. Piero Contini of the Structural Engineering Department of the Politecnico of Turin and the late Renzo Perazzone who, starting from 1982, conducted many experiments to verify a large amount of the technical and engineering formulations contained herein.
Many thanks to Bruno Della Bella for having completed the chapter 5th, with the deformations argument about hollow core floors.

Deserving of special mention are Prof. Antonio Migliacci of the Politecnico of Milan who, as far back as 1967, formulated on an experimental basis the theory of transverse transmission of concentrated loads, and Prof. Marco Menegotto of the University La Sapienza of Rome who conducted many experimental investigations on extruded hollow core floor slabs, with special emphasis on diaphragm behaviour.

Verona, September 2002

Giorgio Della Bella
ASSAP, Chairman

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NOTATIONS

Symbols used in this text comply with EC2 EUROPEAN CODE ENV 1992-1-1.

$\mathbf{A_c}$	Total cross-section area
$\mathbf{A_{fl}}$	Area of reinforcement bars
$\mathbf{A_p}$	Area of prestressing tendons
E	Modulus of elasticity; effect of action
F	Action (general)
G	Permanent action
I	Moment of inertia
M	Bending moment
P	Prestressing force
Q	Variable action
R	Structure internal resistance
S	Effect of action
V	Shear force
$\mathbf{V_{Rd}}$	Design value of the internal resistance to shear force
$\mathbf{V_{Ed}}$	Design value of the applied shear force
a	Distance
b	Width
$\mathbf{b_c}$	Width of a hollow core full of concrete
$\mathbf{b_i}$	Width of a single web
$\mathbf{b_w}$	Total width of slab webs
c	Distance; concrete cover of tendons
d	Effective depth of a cross-section
$\mathbf{e_o}$	Prestressing tendon eccentricity
v	Deflection
h	Depth of a cross-section
$\mathbf{h_f}$	Depth

i	Distance between reinforcing bars or prestressing tendons
k	Coefficient; factor
l	Lenght; span
$\mathbf{l_{bp}}$	Transmission length of prestressing tendons
n	Number
t	Time
$\boldsymbol{\alpha}$	Angle; ratio
$\boldsymbol{\beta}$	Angle; ratio
$\boldsymbol{\beta_b}$	Factor for transmission length of prestressing tendons
$\boldsymbol{\gamma}$	Partial safety factor
$\boldsymbol{\gamma_c}$	Partial safety factor for concrete material properties
$\boldsymbol{\gamma_g}$	Partial safety factor for permanent actions G
$\boldsymbol{\gamma_p}$	Partial safety factor for prestressing actions P
$\boldsymbol{\gamma_q}$	Partial safety factor for variable actions Q
$\boldsymbol{\gamma_{sp}}$	Partial safety factor for spalling tensions
$\boldsymbol{\delta}$	Coefficient
$\boldsymbol{\varepsilon}$	Elongation
$\boldsymbol{\mu}$	Coefficient of friction
$\boldsymbol{\nu}$	Coefficient
$\boldsymbol{\varrho}$	Reinforcement ratio
$\boldsymbol{\sigma}$	Stress; tension
$\boldsymbol{\sigma_I}$	Design principal stress
$\boldsymbol{\sigma_d}$	Design compressive stress
$\boldsymbol{\sigma_{po}}$	Prestressing tendon tension at time 0
$\boldsymbol{\sigma_{sp}}$	Spalling tension
$\boldsymbol{\sigma_{spi}}$	Spalling tension at time of prestression application
$\boldsymbol{\tau}$	Shear stress
$\boldsymbol{\tau_{Rd}}$	Basic design shear strength
$\boldsymbol{\tau_{Sd}}$	Design value of shear stress
θ	Temperature
ϕ	Diameter of a reinforcing bar or prestressing tendon
$\boldsymbol{\psi}$	Coefficient for combination of actions

Concrete

C Strength class of concrete

$\mathbf{f_c}$ Cylinder compressive strength

$\mathbf{f_{ck}}$ Characteristic compressive cylinder strength at 28 days

$\mathbf{f_{ck,\ cube}}$ Cube characteristic compressive strength at 28 days

$\mathbf{f_{cd}}$ Design value of cylinder compressive strength ($= f_{ck}/\gamma_c$)

$\mathbf{f_{ct}}$ Tensile strength

$\mathbf{f_{cfm}}$ Mean value of flexural tensile strength

$\mathbf{f_{cfd}}$ Design value of flexural tensile strength ($= f_{cfm}/\gamma_c$)

$\mathbf{f_{ctk}}$ Characteristic axial tensile strength

$\mathbf{f_{ctk\ 0.05}}$ Lower characteristic tensile strength (5% fractile)

$\mathbf{f_{ctk\ 0.95}}$ Upper characteristic tensile strength (95% fractile)

$\mathbf{f_{ctm}}$ Mean value of axial tensile strength

$\mathbf{f_{ctd}}$ Design value of axial tensile strength ($= f_{ctk0.05}/\gamma_c$)

Normal reinforcement

$\mathbf{f_{yk}}$ Characteristic yield strength

$\mathbf{f_{tk}}$ Characteristic tensile strength

$\mathbf{f_{sd}}$ Design tensile strength ($= f_{yk}/\gamma_s$)

$\mathbf{f_{0.2k}}$ Characteristic yield strength at 0.2%

$\boldsymbol{\varepsilon_{uk}}$ Ductility; elongation at maximum load

Prestressing steel

$\mathbf{f_{pk}}$ Characteristic tensile strength

$\mathbf{f_{p0.1k}}$ Characteristic 0.1% proof-stress

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Chapter 1
HOLLOW CORE SLAB FLOORS

1.1. Historical background

In the 1930s the German Wilhelm Schaefer, together with a colleague named Kuen, laid the foundations for the realisation of something quite similar to what today we call the "hollow core slab".
It was an insulated structural slab made up of a hollow core layer of pumice concrete enclosed within two layers of normal reinforced concrete.
At the end of the 1940s and beginning of the 1950s, after years of production line changes based on trial and error, the "Schaefer" plant began meeting with some success.

Production licences were sold to five companies in East and West Germany and one in the United States.

The most important of West German producers, BUDERUS'SCHE EISENWERKE, was the first to introduce prestressing in hollow core slabs in its plant in Burgsolms, which is still in operation. Static calculations were studied by Prof. Friedrich at the Technical University of Graz (Austria).

Soon afterwards, around 1955, the layer of pumice concrete was abandoned to allow the production of hollow core slabs in monolithic concrete with spans and capacities less limited by the poor shear strength of pumice.

In the same years the American company that had purchased the Schaefer plant introduced prestressing and developed to such a point that it also became the producer of patented plants under the name SPANCRETE.

Spancrete plants call for a casting machine on a bridge crane. Hollow core slab casting takes place with the laying of layers one on top of another, separated by a simple sheet of plastic.

Surface flatness is not perfect, but it is acceptable, as can be seen in many parking silos in the United States.

Once the upper casts of a pile of slabs have hardened naturally, a diamond disk sawing machine is mounted on the same pile of slabs and hollow core slabs are cut and removed.

The plant, with the use of a vibrating slipform machine on the single casting beds, as is now the most common configuration, was designed in 1955 by Max Gessner of Lochham (Munich).

In 1957, the West German companies MAX ROTH KG and WEILER KG purchased Gessner's patent and in 1961 began the gradual expansion throughout Europe and the world of hollow core slabs produced with slipform machines.

In 1960 the SPIROLL Company in Canada developed an original machine for the production of hollow core slabs by means of a screw-feeder that extrudes the concrete.

With this new procedure concrete with a low water/cement ratio was compacted and vibrated. The cores were characterized by a typical circular section quite different from the usual oblong one produced with slipform machines.

The extrusion procedure was also received favourably, especially in Northern Europe and the Soviet bloc and, as is always the case with two competing systems, the race for supremacy between the slipform system and the extrusion system produced great benefits in the development of the prefabricated hollow core slab all over the world.

The Italian company NORDIMPIANTI SYSTEM, which since 1974 has specialized in the construction of slipform machinery and plants, deserves special mention owing to the impulse it gave to increasing the dimensions of hollow core slabs.
In 1987, NORDIMPIANTI earned praise for the successful construction of machinery for the production of an important series of hollow core slabs with three cores having depths of 50, 60, 70 and 80 cm; the latter three depths are still a record today.

1.2. General information

Hollow core slab floors represent a special kind of floor totally made of concrete lightened by hollow cores. They can be prestressed or with normal reinforcement.
Since there is very little production of hollow core slabs with normal reinforcement worldwide, from this point on we shall speak only of the prestressed type.

Slabs are lightened by leaving longitudinal voids (cores) of suitable size to create webs. The intrados and extrados flanges of these webs form the concrete section to be prestressed using embedded steel tendons.

Tensioned steel is the only reinforcement in the hollow core slab, which is without reinforcement against shear.
The structure's resistance to shear thus depends entirely on the tensile strength of the concrete. For this reason concrete quality must be constant, controlled and certified at all stages of production.

Such a precast, prestressed structural component for the laying of bearing floors proved to be quite reliable from the very beginning. It has been widely employed internationally, as can be seen from the fact that almost all national building codes devote at least one paragraph to hollow core slabs and exempt them from the generic obligation to use reinforcement against shear.

As concerns shear strength, which depends on the concrete alone, there is an enormous mass of scientific documents on research, studies, laboratory tests, in situ testing and codes.
Among these, special importance is attributed to the following documents owing to their seriousness and depth of analysis:

FIP "Recommendations on Precast Prestressed Hollow Core Slab Floors", 1988.
FIP "Quality Assurance of Hollow Core Slab Floors", 1992.
FIB (CEB-FIP) **"**Special design considerations for precast prestressed hollow core floors", 1999
P.C.I. "Manual for the Design of Hollow Core Slabs" (U.S.A.), 1985 and 1998.
EUROPEAN STANDARD pr. EN 1168/1 "Prestressed hollow core slabs for floors" 1998.

Fig. 1.1. Cross sections of hollow core slabs for floors.

The latter document takes into consideration hollow core slabs with depths up to 44 cm.

In today’s reality, such slabs are being produced with depths of 60, 70 and even 80 cm, but for safety's sake they must have vertical stirrups in the webs and at least the bottom side reinforced with continuous welded mesh or at least placed in correspondence to the ends of each element.

In the preparation of this text we chose 50 cm (but not in all cases) as the upper limit for a producible hollow core slab without vertical and horizontal reinforcement.

1.3. Reasons for the choice of hollow core floors

There are many reasons why hollow core slabs have met with such a warm reception and have spread to all continents. It can rightly be defined the most “cosmopolitan” of prefabricated components in the building industry worldwide.

Among the many advantages they offer, three are especially important:

Technical advantages

Hollow core slabs are produced in well-equipped, up-to-date plants using advanced technologies requiring little labour. They are produced on casting beds, usally steel, and made with slipform machines or by extrusion. Concrete batching plants with automatic control of weights and the water/cement ratio and, almost universally, equipment for the hot curing of concrete in controlled conditions of temperature and humidity are the other essential components.

Thus, the production of hollow core slab floors has always been accompanied by continuous quality control very close to the directives of the ISO 9001 Standards.

Technically, this means that:

– concretes are made with selected aggregates and with controlled grain-size curves which are particularly constant in time, with a low water-cement ratio, well-compacted and with high physical and mechanical characteristics, $f_{ck} \geq 45 \div 60$ MPa;
– prestressing tendons possess certified strengths and characteristics of relaxation and constantly controlled concrete cover, and are thus well protected from aggressive outside elements and fire.
  The compactness of the concrete, the low water/cement ratio and the integral prestressing of the section, besides inhibiting cracking, also greatly slow down the velocity of concrete carbonation, thus assuring durability and allowing its use even in highly aggressive environments so long as standard concrete cover is assured.

The class of concrete also guarantees a high elasticity modulus, equal to at least 1.3 ÷ 1.5 times that of concrete normally cast in situ.
It follows that installed floors are quite rigid and show very slight elastic deflection under loads applied during inspection.
For this reason it is possible to install thinner slabs for the same span and loads compared to floors that are similar but not entirely prefabricated and prestressed.

The use of modern slide mould machines and extruders, which assure very advanced performance, allow the obtaining of slabs that are structurally and geometrically well formed, such as to supply certain elements for quality evaluation by immediate visual control of webs, lateral profiles and ends cut with diamond disks.
Steel casting beds, suitable to ensure perfect flatness and well-shaped edge lines, form a perfectly smooth surface with well-finished edges at the intrados of the slabs; these are details that produce the excellent aesthetic effect of hollow core floors with "exposed" concrete ceilings.

No ends of steel reinforcement protrude from prestressed hollow core slabs for connection to surrounding structures in cast concrete. Such indispensable connecting reinforcement is inserted *in situ* in the longitudinal joins or in specially provided open cores at the ends, of suitable number and lengths, for joining the set in row slabs.

These efficacious connections with adjoining structures, which make the entire floor monolithic, allow the use of hollow core slabs in all structural applications, even in seismic areas, and together with all kinds of bearing structures, whether cast in situ, precast or steel.

The efficacy of such connections has been demonstrated in innumerable tests in the testing laboratories of prestigious universities and assures a level of structural solidity which is never below what is offered by more traditional floors requiring more abundant in *situ* casts of concrete.

Economic advantages

There is a substantial reduction in building times and thus large savings in machinery and labour.

In fact, labour is kept to a minimum at all stages of production, stocking, transport, erection and completion of the finished floor at the site.

This very low incidence of labour provides the user with a substantial economic advantage, but requires the producer to make large capital investments and employ qualified personnel, since the entire manufacturing process is characterized by a very high technological content so as to guarantee high yield in a continuous cycle and at the same time maintain the high quality standards required by product codes.

Versatility in application

Up to the 1970s hollow core floors were used almost exclusively with the simple support of steel beams, precast concrete beams and bearing walls.
They were often used as the simple covering of prefabricated industrial sheds.

The low depths of slabs then produced (10 ÷ 15 ÷ 20 ÷ 25 cm) did not allow long spans or heavy loads; however, it was in those very years that the most openminded builders began to insert hollow core slabs in buildings structured with reinforced concrete cast in situ.

The positive union of hollow core slab floor and reinforced concrete beam cast in such a way as to englobe the slab ends led to unexpected developments in applications and to the generalized use of hollow core floors in all kinds of buildings.

Today, hollow core slabs of large depths allow construction of floors with spans up to 20 metres under industrial loads, no longer with simple support, but with restraints of structural continuity and even perfectly fixed ends. Further advantages of these slabs come from the possibility of their use as a clear span between beams cast in situ having the same depth as the floor.

These possible applications have favoured the adoption of hollow core floors in underground construction works where it is of primary importance for the structure to be monolithic.

Fig.1.2 Hollow core slab floors in a multistorey underground parking garage

The great versatility of hollow core slabs allows their use not only as floors, but also as walls of tanks for hydraulic plants, as earth retaining walls in civil and road works and efficaceously as external and bearing walls for civil and industrial buildings of all heights.

Fig.1.3 The hollow core walls of a water treatment tank

Fig.1.4 Hollow core bearing walls and floors in a multistorey residential building

Numerous examples of multistorey buildings advantageously erected with such bearing walls demonstrate that all the possible uses of this very special precast element still have not been exploited fully. Its development worldwide must therefore be considered as still at the beginning; the future will certainly see its use in applications that have not yet been conceived.

1.4. Reference to codes

1.4.1. Italian building Standards

The characteristic cross section of hollow core slabs shows some parts where the concrete is thinner than required by Italian regulations for reinforced and prestressed concrete.

This, as well as other waivers allowed by the Italian code, is justified by the special production technologies and materials that go into their production and as long as the producer constantly meets the quality requisites of the Italian Ministry of Public Works through "Production in Controlled Series".

Following is a list of rules in force and the specific items dealing with hollow core slab floors:

- ITALIAN NATIONAL APPLICATION DOCUMENT (N.A.D.) FOR EUROCODE 2 ENV 1992-1-1 ACCEPTANCE. Ministerial Decree dated 9 January 1996, Section III.

par. 2.3.3.2.	schedule 2.3	- Safety factor for prestressed reinforced concrete.
par. 4.1.3.3.	schedule 4.2	- Minimum cover of prestressing tendons.
par. 4.2.3.5.6.	schedule 4.7	- Length of anchoring zone of prestressing tendons.

– “BUILDING RULES FOR CALCULATION, EXECUTION AND FINAL INSPECTION OF REINFORCED AND PRESTRESSED CONCRETE CONSTRUCTION WORKS” (Italian building standard).
Ministerial Decree dated 14 February 1992 for calculation according to the Allowable Stresses method.
Ministerial Decree dated 9 January 1996, Section I and Section II for calculation according to the method of Limit States.
Ministerial Explanatory Memorandum dated 15 June 1996

par.	6.2.2.	Minimum cover for reinforcing steel.
chapt.	7	Supplementary rules concerning floors.
par.	7.0.a	Obligatory use of additional bottom reinforcing in supports of floors capable of absorbing a tensile stress equal to shear.
par.	7.1.4.6	Waiver of transversal reinforcement (final paragraph).
par.	7.3.3.	Specific provision for hollow core floors.
par.	7.1.6.	Provisions also valid for hollow core floors.
par.	7.1.4.2.	(second paragraph). Provision valid also for hollow core floors with concrete topping (minimum depths).
par.	7.3.2.	(fourth paragraph). Minimum depth for hollow core floors without topping.
par.	7.3.4.	Provision for hollow core floors with concrete topping.

– “BUILDING RULES FOR DESIGN, EXECUTION AND FINAL INSPECTION OF PRECAST CONSTRUCTION WORKS” (Italian Prefab Regulations).
Ministerial Decree dated 3 December 1987 and Ministerial Memorandum no. 31104, dated 16 March 1989.

par. 2.11.1.3.	Floors. "Production in Controlled Series" obligatory for prefabricated elements without shear reinforcement or with thicknesses below 4 cm at any point.

par. 2.2 In calculations of prestressed elements produced in "Controlled Series" the coefficient $\gamma_c = 1.42$ is assumed in the method at Limit States just as a 5% increase in tensions is assumed in verifications with the Allowable Stresses method.

– "ANALYTICAL RULES TO EVALUATE THE RESISTANCE TO FIRE OF REINFORCED CONCRETE AND PRESTRESSED SUPPORTING MEMBERS".
Memorandum C.N.R.-V.F. UNI 9502.
This is a fundamental document for the analytical calculation of a structure's resistance to fire.

– "TECHNICAL RULES AND STANDARDS FOR ACTIONS ON BUILDING SAFETY VERIFICATION"
Ministerial Decree dated 16 January 1996 and Ministerial Explanatory Memorandum dated 4 July 1996.
This is the Italian document for the application of EUROCODE 1 EN 1991-1 "BASIS OF DESIGN AND ACTIONS ON STRUCTURES".

– TECHNICAL RULES FOR CONSTRUCTIONS IN SEISMIC AREAS, Ministerial Decree dated 16 January 1996.
This is the Italian document for application of EUROCODE 8 EN 1998 "DESIGN PROVISIONS FOR EARTHQUAKE RESISTANCE OF STRUCTURES".

– INSTRUCTIONS CNR 10025/1998 "INSTRUCTIONS FOR THE DESIGN, EXECUTION AND CONTROL OF PRECAST CONCRETE STRUCTURES".
These Instructions dated 10^{th} december 1998, were prepared by the Working Group "Prefabrication" of CNR updating the previous Instructions CNR 10025/1984 in conformity with most recent international recommendations relevant to precast concrete structures.

– EN ISO 9000:2000 Standard "QUALITY MANAGEMENT SYSTEMS-FUNDAMENTALS AND VOCABULARY" (December 2000).

This indicates the objectives that a company must set itself in order to satisfy the Customer with continuity, to ensure company management that the pre-established quality standard has been reached and to assure the purchaser that the specified quality will be delivered.

– EN ISO 9001:2000 Standard “QUALITY MANAGEMENT SYSTEMS - REQUIREMENTS” (December 2000).
  It promotes the adoption of a process approach to enhance customer satisfaction by meeting customer requirements. Producers of prefabricated components are involved. Indeed, this Standard considers all operating stages of a job, from designing to implementation, erection and servicing once the structure has come into use.

– EN ISO 9004:2000 Standard "QUALITY MANAGEMENT SYSTEMS - GUIDELINES FOR PERFORMANCE IMPROVEMENTS" (December 2000).
  This Standard gives guidance on a wider range of objectives than does EN ISO 9001:2000, particularly for the continual improvement of an organisation’ performance and efficency. The effective application of the system aims to enhance not only customer satisfaction and product quality. It is extended to include the satisfaction of other interested parties: collaborators, community, associates, organization partners, suppliers.

– EEC DIRECTIVE 89/106 “EC CONFORMITY MARK ON PRODUCTS FOR THE BUILDING INDUSTRY AND RELATIVE APPLICATION DOCUMENTS”.
  It will come into force as law when ratified by the Council of Europe as implementation of EEC Directive 89/106. The EC Conformity Mark will become obligatory for all building products (as for all other products in circulation in countries belonging to the European Community). The certificate of conformity will be issued by national certification and inspection bodies which shall assess the compliance of the product with the European Product Standard by carrying out inspection and surveillance of production control.

To obtain the Certificate of Conformity it will be indispensable for producers to adopt a Factory Production Control System.

1.4.2. European building Standards

The geometric and strength characteristics of sections as well as calculating, designing, verifying and acceptance methods refer to European standards in force at the time this text is being written and are listed below.

– EN 206-1 “CONCRETE: SPECIFICATION, PERFORMANCE, PRODUCTION AND CONFORMITY”.
This is a most precise and detailed description of the production of concrete in order to assure the necessary durability as well as quality. The new version of this Standard also applies to all cases concerning prefabrication. Deviations are admitted for special elements made with concrete having a low water/cement ratio, such as hollow core slabs, if foreseen in specific product standards.

– ENV 1991-1 (EUROCODE 1) “BASIS OF DESIGN AND ACTIONS ON STRUCTURES”.
This Standard was adopted by Italy with Ministerial Decree dated 16 January 1996.

– ENV 1992-1-1 (EUROCODE 2) “DESIGN OF CONCRETE STRUCTURES - PART 1: GENERAL RULES AND RULES FOR BUILDINGS”.
This is the General Standard addressing the needs for strength, behaviour when installed and durability of structures made of reinforced and prestressed concrete. It does not deal with specific fields but contains the values of safety coefficients approved by CEN-TC 250 and the general principles of design valid also for prefabricated components in general. This Standard is applicable in Italy as long as the substitute, integrating and suppressive prescriptions contained in the General Part and in Sections I and III of Ministerial Decree dated 9 January 1996 are complied with.

– ENV 1992-1-3 (EUROCODE 2) “DESIGN OF CONCRETE STRUCTURES” - PART 1-3: “PRECAST CONCRETE ELEMENTS AND STRUCTURES”.
This supplies a general basis for the design and building details of the structures of buildings partly or entirely constructed with prefabricated components.
This part supplies the principles and rules that supplement those found in ENV 1992-1-1 concerning prefabricated components and therefore also hollow core slabs.

– pr EN 1992-1 (EUROCODE 2 Part 1-2001) “DESIGN OF CONCRETE STRUCTURES” – PART 1 – “GENERAL RULES AND RULES FOR BUILDINGS”.
Updated Provisional European Norme covering both ENV 1992-1-1 and ENV 1992-1-3.

– ENV 1992-1-2 (EUROCODE 2 Part 1-2) “DESIGN OF CONCRETE STRUCTURES” – PART 1-2 “STRUCTURAL FIRE DESIGN”.
General rules to value fire resistance of reinforced or prestressed concrete structures are supplied by this standard.

– ENV 1992-1-4 (EUROCODE 2 PART 1-4) “DESIGN OF CONCRETE STRUCTURES” PART 1-4 “STRUCTURAL LIGHTWEIGHT AGGREGATE CONCRETE WITH CLOSED STRUCTURES”.
At the moment it is not suitable for hollow core slabs.

– Pr. EN 1168/1 “PRECAST CONCRETE PRODUCTS – HOLLOW CORE SLABS FOR FLOORS" (provisory European Standard).
Some details of hollow core floors, for example the absence of normal transverse reinforcement, make it necessary to apply some specific rules in addition to ENV 1992 -1-3. This standard thus supplies the rules for special designs not contemplated by ENV 1992 -1-1 and 1-3, but in perfect agreement with their principles of calculation. This standard belongs to a series of product standards dealing

with concrete prefabricated structures and concerns the characteristics that producers of hollow core slabs must assure in order to respond to the essential requisites as defined by the Directive on Building Materials EEC 89/106. As concerns fire-resistance, the standard refers to ENV 1992 -1-2, (Eurocode 2, Part 1-2). Given the great importance of this Standard Pr- EN 1168, which deals specifically with hollow core floors, it will be included in the next ASSAP publication as it will appear in its final version.

– ISO 140-3 / IS0 717-1 / ISO 717-2 Standards "ACOUSTICS – MEASUREMENTS AND RATING OF SOUND INSULATION IN BUILDINGS AND OF BUILDING ELEMENTS".
These standards concern hollow core floors for quality assurance of comfort in the buildings.

– ISO 6946 Standard – "BUILDING COMPONENTS AND BUILDING ELEMENTS – THERMAL RESISTANCE AND THERMAL TRANSMITTANCE – CALCULATION METHOD".
This standard is important in determining fire resistance of buildings with hollow core floors or walls.

1.4.3. Important international documents

Here we mention four documents that are quite important owing to their authoritative value for consultation in the hollow core floor sector.

– MANUAL FOR THE DESIGN OF HOLLOW CORE SLABS.
U.S.A. Prestressed Concrete Institute P.C.I., 1985 and subsequent 1998 second edition. This is the first manual devoted to prestressed hollow core slab floors. It describes the various production systems and the different kinds of slabs. It indicates the methods for calculation according to ACI Standards, illustrated with meaningful examples and gives full details on design and application criteria to be followed. Structural continuity between slabs and negative moments

are not provided. It deals with resistance to fire, acoustical behaviour and quality and tender specifications.

- FIP Recommendations "PRECAST PRESTRESSED HOLLOW CORE SLAB FLOORS" (1988).
  It represents the first important European document containing principles for the calculation and structural design of hollow core slab floors on the basis of experience gained in northern Europe with extruded slabs. No structural restraint is foreseen beyond the simple support.

- FIP Guide to good practice "QUALITY ASSURANCE OF HOLLOW CORE SLAB FLOORS" (1992).
  It gives numerous specific rules for acceptability of hollow core slabs for floors. It is a document of great importance as a reference work for the acceptability or non-acceptability of the slabs in case of controversy.

- FIB (CEB-FIP) Guide to good practice “SPECIAL DESIGN CONSIDERATIONS FOR PRECAST PRESTRESSED HOLLOW CORE FLOORS” (1999).
  The purpose of this Guide is to supplement the existing FIP Recommendations (1988) in which some rules for design were incomplete or missing. Much scientific research on different aspects has been carried out since 1983 at important European universities and has produced further knowledge on the behaviour of hollow core floors. Chapter 2 deals with restrained composite supports and other ASSAP specific application technologies.

Chapter 2
PRODUCTION

2.1. Notes on production technologies

The production of prestressed hollow core slabs takes place in works on long iron beds (120 ÷ 150 m) on which prestressing wires are positioned and stretched.
Casting of concrete is continuous and done with machinery designed specifically for the purpose. Generally speaking, three production procedures are employed:

– The "slipform" procedure with slide mould machines in which concrete is directed into mobile sectors and vibrated by batteries of vibrators at different frequencies. In slide mould machines casting takes place in three stages: intrados, webs and extrados to arrive at completion of the finished slab (Fig. 2.1).

– The "extruder" procedure with the use of extruding machines in which the concrete is forced by special screw-feeders to compact in a single stage to produce the finished slab section (Fig. 2.2).

– A third procedure can be classified as "slipform" even though it does not employ slipform machines but batteries of vibrating tubes which are extracted from the artifact in a single stage.

Fig. 2.1 Slide-mould machine (slipform procedure)

Fig. 2.2 Extruder

Fig. 2.3 Prestressing reinforcement stretched on the casting bed

Fig. 2.4 Continuous casting of concrete

All production procedures require concrete of the highest quality and homogeneity both in grain size composition and the cement/water ratio. They must assure instantaneous stability in shape so as to form the voids, high initial mechanical strength to allow prestressing and removal from the bed after a short time, and finally optimum adherence of prestression reinforcement and any normal reinforcement included in the casting.

Curing is accelerated by homogeneously heating the concrete until the required degree of strength for release of prestression reinforcement is reached ($f_{ck} > 30 \div 35$ MPa). This strength is determined experimentally through the breaking of test pieces that have received the same vibratory and thermal treatment.

At the time of the compression test (28 days from casting), the concrete will have cubic strength above $f_{ck,cube}$ 55 MPa.

Once the artefact begins to be cast continuously over the entire length of a bed, operating immediately on still fresh concrete, the cut-outs required by the design or the holes for vertical canalization are added manually.
In this phase grooves are made in the slab ends for anchoring tie bars, as well as the transversal holes that may be needed for lifting.

When the concrete is sufficiently hardened and strands are released from their anchorages, slabs are then cut to the required length with abrasive or diamond disks.
This is the moment of concrete prestression in each slab.

On removal from the casting bed, the hollow core slabs present the intrados which is smooth from having been in contact with the metal surface, while the side faces and the extrados are rough. This assures a good bond with in situ castings of joins or structural topping.

Fig. 2.5 Cutting of slabs and their removal from casting bed

Fig. 2.6 Hollow core slab storage yard

In all production processes of hollow core slabs the following stages are present:

– preparation of the bed, cleaning and treatment with demoulding oil;
– laying of steel reinforcement, wires or strands for prestressed concrete (Fig. 2.3);
– stretching of reinforcement with systematic control of tension and elongation;
– continuous casting of concrete (Fig. 2.4);
– manual or mechanized intervention on each slab to meet design functions and sizes;
– marking of slabs with design mark, order number, date of production and weight;
– covering of the cast bed with waterproof sheets and possible heating to accelerate hardening;
– systematic control of concrete strength before releasing stretched wires to prestress the artifact;
– transversal cutting to isolate the single slabs (Fig. 2.5);
– removal of the slabs from the bed and transportation to storage area (Figs. 2.5 and 2.6).

Fig. 2.7 Slip-formed slabs with widened webs or special shaped webs (grandstands for a stadium)

2.2. Cross section geometry

2.2.1. Types of hollow cores

There are different kinds of hollow core slabs that vary in lateral profile or design of voids, which may be perfectly round, elliptical or, more often, with a composite profile.

In general there are voids similar to circles in shallow slabs and elongated holes with rectilinear sides and joining curves for slabs of greater depth (see Fig. 2.8).

In the case of elongated forms, special attention is given to the upper and lower fillets to avoid concentration of stresses and to limit the thickness of the concrete arches above and below the voids.
As stated previously, the depth of slabs now being produced varies from 12 to over 80 cm.
On the average, voids represent about 50% of the total slab volume.

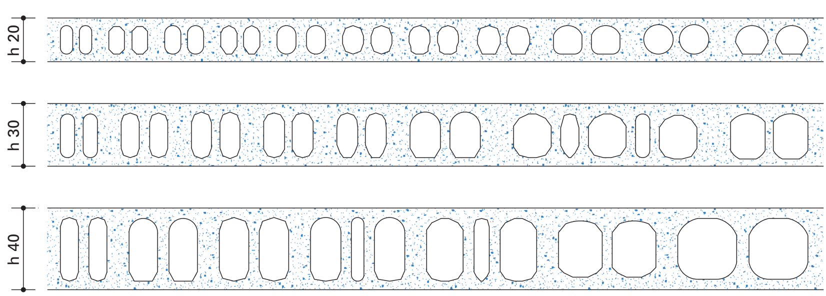


Fig. 2.8 Types of voids in hollow core slabs

For elements up to 20 cm in depth voids represent not more than 40%.
With greater depths, the void percentage is between 50 and 70%. Slabs thus produced are quite low in weight.

2.2.2. Typical shapes of lateral join faces

The lateral profile of the different slabs can assume quite variable configurations (see Figs. 2.9 and 2.10).
Floor slabs possess longitudinal joins open at the top and slot-shaped to allow grouting in a longitudinal shear-key form to assure transversal transmission of loads and deformations, even with heavy, concentrated loads.

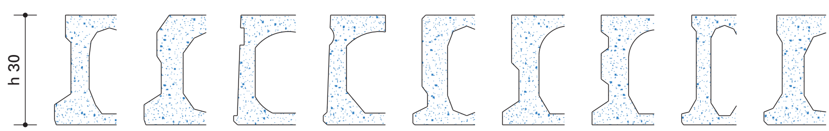


Fig. 2.9 Lateral profiles in hollow core floor slabs

When the longitudinal join receives and englobes normal tie reinforcement, it must present two minimum dimensions:

– Minimum upper aperture 3 cm wide; if the join must also act as an edge beam the minimum aperture must be 5 cm (see Italian Prefab Regualtions, par. 2.11.2.b);

– The width of the zone where reinforcement is positioned must be greater than or equal to three times the diameter of the bar and compatible with the maximum diameter of the grouting granules (not less than 6 cm is recommended). When the join also acts as an edge beam the minimum width of the positioning zone of the reinforcement must be 8 cm (see same, par. 2.11.2.b).

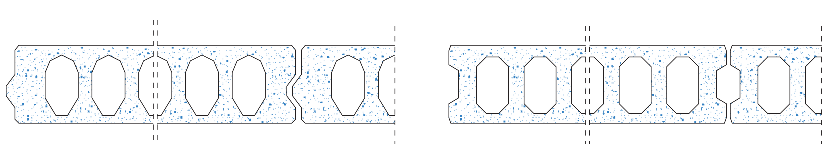


Fig. 2.10 Lateral profiles in hollow core wall slabs

On this subject, see also Figs. 3.3 and 3.5 in paragraph 3.3 below.

The longitudinal join faces may also have vertical indentation to improve the bond of the cast concrete and consequently its diaphragm behaviour (see next ASSAP Volume).

Slabs used as walls are produced with lateral male-female shapes or with female-female shapes to allow the proper placing on both faces depending on how they are to be used.

2.2.3. Thickness of webs and flanges

The design of concrete cross-sections of hollow core slabs is of the utmost importance. It must start from a careful analysis of the different economic, technical and regulatory aspects.

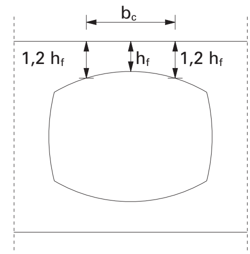
After optimization of the cross-sections from the cost and weight viewpoints, which must also take into account the technology of the machines used in their casting, regulations in force and good building practice, it is important to maintain constant control over all stages of production to avoid costly wastes of concrete due to overthickness or dangerous underthickness causing weakness of the section.

Minimum depths are set in Chapter 4.3.1 of the EN 1168 Standard. These must be increased by the amount of specific tolerance of each producer:

Webs $$b_{i\,min} \geq \begin{cases} h/10 & [mm] \\ \sqrt{2h} & [mm] \\ 20 & [mm] \\ d_g + 5 & [mm] \end{cases}$$ (not less than the largest value)

Flanges $$h_{f\,min} \geq \begin{cases} \sqrt{2h} & [mm] \\ 17 & [mm] \\ d_g + 5 & [mm] \end{cases}$$ (not less than the largest value)

Upper flange $h_f \geq b_c/4$



h (mm) = depth of the slab

d_g (mm) = maximum nominal dimension of aggregate

b_c (mm) = width of the portion of the upper flange between the two sections having a thickness 1.2 times the smallest thickness of $h_{f\,sup}$

Generally speaking, the thickness of the vertical webs between voids is never less than 30 ÷ 35 mm and it increases in slabs of greater depth or more subject to shear stress.
Slabs of the slipform type can be produced with some wider webs at the expense of other voids or even by eliminating some of them completely to increase shear strength (see Fig. 2.7).
The minimum thickness of flanges above and below the voids is usually not less than 25 or 30 mm.

2.2.4. Distribution and cover of prestressing strands

Attention is drawn to the special care that must go into the study of the zones in which prestressing reinforcement is placed: the durability and especially the fire resistance of the slab require strategies that are in contrast with the exploitation of the maximum reinforcement that can be inserted in the section.
The problem is addressed in Italian and European regulations for the sole purpose of assuring the proper distribution and protection of reinforcement, which must such as to guarantee the functional durability of the structure once completed (see Fig. 2.11).

Prestressing wires must be positioned below the webs where the concrete section is such as to assure effective covering. Furthermore, they must be

distributed in such a way as to be uniform and symmetrical in the cross section.
In par. 4.3.1.2, the EN 1168 Standard recommends minimum reinforcing of at least four strands or wires for each element having a width of 1.20 m.

Distance between reinforcing strands or wires

The minimum distance between the surfaces of the strands is not specifically mentioned in the Italian Standard which, in par. 6.1.4, fixes the centre distance between strands only for normal reinforcement, which is

$$i \geq \begin{cases} \phi \\ 20\ \text{mm} \\ d_g \end{cases} \quad \text{(not less than the largest value)}$$

ϕ = diameter of the normal steel bar or nominal diameter of strand
d_g = max. nominal dimension of aggregate

Paragraph 4.3.1.2 of EN 1168 Standard and paragraph 5.3.3.1 of European Standard EC2 ENV 1992-1-1 prescribe the following distances for strands:

minimum horizontal distance $$i_h \geq \begin{cases} \phi \\ 20\ \text{mm} \\ d_g + 5\ \text{mm} \end{cases} \quad \text{(not less than the largest value)}$$

minimum vertical distance $$i_v \geq \begin{cases} \phi \\ 10\ \text{mm} \\ d_g \end{cases} \quad \text{(not less than the largest value)}$$

For unstirruped structures (hollow core slabs) EC 2 ENV 1992-1-1, par. 4.1.3.3. Point 11, and Standard EN 1168, in par. 4.3.1.3, prescribe the following limit values for the concrete covering prestressed tendons, including the allowable tolerance (see Fig. 2.11):

c = 2 ϕ if the distance between centres of the tendons is $\geq 3\ \phi$

c = 3 ϕ if the distance between centres of the tendons is $< 2.5\ \phi$

ϕ = diameter of strand or wire; in the case of varying diameters it is necessary to consider the mean value.

For ribbed wires, the concrete cover shall be increased by 1 ϕ.

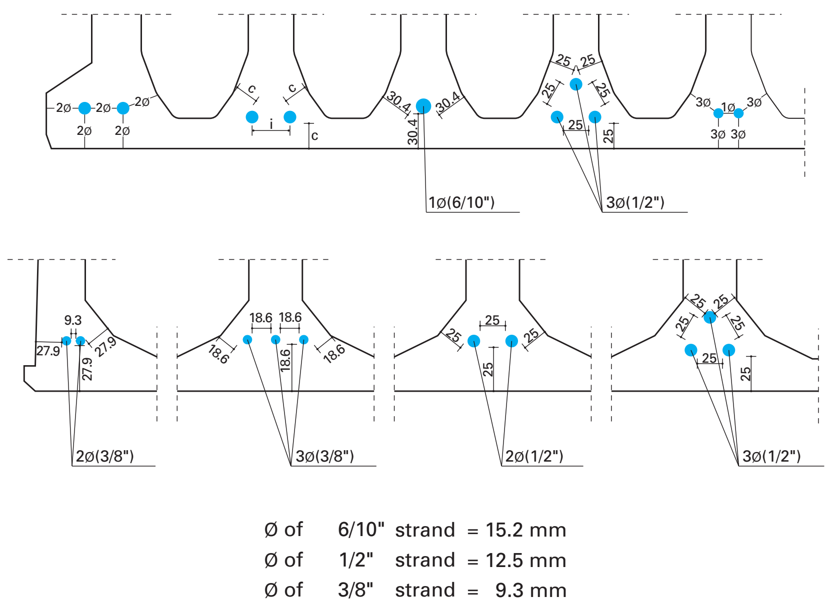


Fig. 2.11 *Correctly placed reinforcement. Having assured control of production, the minimum design values for a single strand indicated above for "c" admit the maximum negative tolerance of – 5 mm (EN 1168 par. 4.3.3.1 and ENV 1992-1-1 par. 4.1.3.3. Point 8).*
In pr EN 1992-1 (Section 4), the updated version of Eurocode 2, new different values for concrete cover and minimum distance of strands are given. They will be subjected to specific National Application Document by each CEN member State.

Table 2.1
Concrete cover for Class of cylinder/cubic strength $\geq$ C 40/50 N/mm^2 in accordance with Eurocode 2 ENV 1992-1-1 par. 4.1.3.3.

Exposure classes		**Examples of environmental conditions**	**Design cover thicknesses c (mm) including a default tolerance up to –5 mm**
1 dry environment		interior of buildings for normal habitation or offices (commercial, public buildings). Internal, non-aggressive environments: storehouses, garages, etc.	2 ϕ 25
2 humid environment	a without frost	- interior of buildings with high humidity (laundries, etc.) - external components - components in non-aggressive soils and/or water	30
	b with frost	- external components exposed to freezing temperatures - components in non-aggressive soils and/or water subject to frost - internal components with high humidity and exposed to frost	35
3 humid environment with frost and deicing salts		Internal and external components exposed to frost and de-icing agents	50
4 seawater environment	a without frost	- components totally or partially immersed in sea water or exposed to splash. - components in saturated salt air (coastal areas)	50
	b with frost	- components partially immersed in sea water or exposed to splashing and freezing - components in saturated salt air and exposed to frosts	50
The following classes may occur alone or in combination with the classes mentioned above			
5 Aggressive chemical environment	a	Slightly aggressive chemical environment (gases, liquids, solids)	35
	b	Moderately aggressive chemical environment (gases, liquids, solids)	40
	c	Hightly aggressive chemical environment (gases, liquids, solids)	50

As concerns protection against corrosion, it is to be kept in mind that the minimum thickness for concrete covering prestressing reinforcement depends on several factors such as maximum dimensions of the aggregate, mixing water/cement ratio, concrete strength and its chemical and physical composition. Last but not least, the aggressiveness of the environment in which the structure is erected must be considered.

Italian National Application Document of European ENV 1992-1-1 Code: in paragraph 4.1.3.3., Table 4.2., takes into account the strength class of the concrete and the aggressiveness of the environment in fixing cover thickness. Six classes of exposure are indicated. Table 2.1 gives the values for Italy. Tolerance up to -5 mm is included.

It is also specified that class of exposure - 1 - may be adopted for cover in the direction of hollow cores.

Different definitions of exposure classes are given in the new Standard EN 206-1 (December 2000) as well as in pr EN 1992-1 (new version of Eurocode 2) and are reported in Table 2.2 here below.

A correlation between exposure classes in accordance with ENV 1992-1-1 (see Table 2.1) and updated EN 206-1 (see Table 2.2) is reported in Table 2.3 below.

Further EN 206-1 prescriptions deal with the kind of aggregate and cement, minimum cement content and the maximum water/cement ratio.
Other, even more severe limits can be imposed for fire safety as indicated in the chapter to which the reader is referred for further details on the subject.

Table 2.2
Exposure classes related to environmental conditions in accordance with EN 206-1 (December 2000).

CLASS DESIGNATION	DESCRIPTION OF THE ENVIRONMENT	INFORMATIVE EXAMPLES WHERE EXPOSURE CLASSES MAY OCCUR
1. NO RISK OF CORROSION OR ATTACK		
XO	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack. For concrete with reinforcement or embedded metal:very dry	Concrete inside buildings with very low air humidity
2. CORROSION INDUCED BY CARBONATION		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air hymidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3. CORROSION INDUCED BY CHLORIDES		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4. CORROSION INDUCED BY CHLORIDES FROM SEA WATER		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. FREEZE/THAW ATTACK		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agent	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. CHEMICAL ATTACK		
XA1	Slightly aggressive chemical environment according to EN 206, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206, Table 2	Natural soils and ground water

Table 2.3

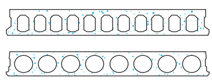
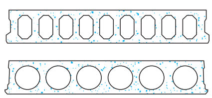
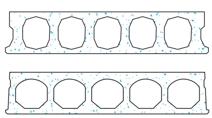
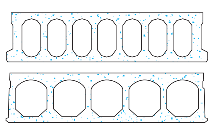
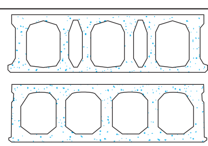
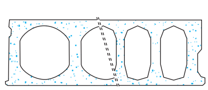
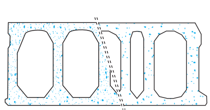
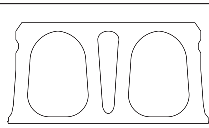
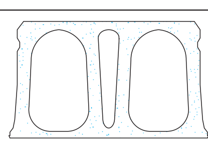
Linking table for exposure classes according to ENV 1992-1-1 and both updated EN 206-1 and pr EN 1992-1.

ENVIRONMENTAL CONDITION		EXPOSURE CLASSES		MINIMUM PRESCRIPTION	
		ENV 1992-1-1	EN 206-1	Water/cement ratio max.	Minimum concrete class
DRY		**1**	**XO**	0.65	C 20/25
HUMID without frost		**2a**	**XC1 - XC2**	0.60	C 25/30
WITH FROST	MODERATE ATTACK without de-icing salts	**2b**	**XF1**	0.55	C30/37 and frost proof aggregates
WITH FROST	AGGRESSIVE ATTACK without de-icing salts	**2b**	**XF3**	0.50	aerated C30/37 and frost proof aggregates
WITH FROST	MODERATE ATTACK with de-icing salts	**3 - 4b**	**XF2**	0.50	aerated C30/37 and frost proof aggregates
WITH FROST	HIGHTLY AGGRESSIVE ATTACK with de-icing salts	**3 - 4b**	**XF4**	0.45	aerated C35/45 and frost proof aggregates
SLIGHTLY AGGRESSIVE		**5a**	**XC3 - XA1 XD1**	0.55	aerated C30/37
MODERATE AGGRESSIVE		**4a - 5b**	**XC4 - XA2 XD2 - XS1**	0.50	C30/37
HIGHTLY AGGRESSIVE		**5c**	**XA3 - XD3 XS2 - XS3**	0.45	C5/45

2.2.5. Examples of cross-section of hollow core slabs, relevant weights and geometric characteristics with simple support and without resistance to fire

The range of existing hollow core slabs is so wide in shapes and cross sections that it is not possible to make a list.
Table 2.4 below offers a short summary of hollow core slabs having same standard width and typical exemplification of depths.
Typical mean values are given for static and geometric characteristics.
They refer to concrete class C45/55 of the prefabricated slab and C25/30 of the corroborant topping.

Table 2.4

The static characteristics are given for the standard slab width of 1200 mm	Depth **h** mm	Slab weight kN/m^2	In situ dead weight kN/m^2	Max reinforcement mm^2	**I** without topping cm^4	**M** max without topping kNm	**V** max without topping kN	**h** of topping cm	**I** with topping cm^4	**M** max with topping kNm	**V** max with topping kN
	**150**	2.30	2.45	900	27,400	65	40	4	52,000	80	45
	**200**	2.80	3.00	1,130	66,000	115	50	4	110,000	140	60
	**250**	3.30	3.50	1,180	120,000	160	70	4	190,000	185	80
	**300**	3.60	4.00	1,450	205,000	230	80	4	310,000	275	90
	**350**	4.00	4.40	1,600	315,000	320	90	6	520,000	380	100
	**400**	4.70	5.30	1,900	465,000	420	105	6	700,000	480	120
	**500**	5.70	6.50	2,300	900,000	630	135	8	1,400,000	720	155
	**600**	6.40	7.40	2,500	1,450,000	760	140	8	2,200,000	870	170
	**700**	7.30	8.40	2,650	2,200,000	980	170	8	3,200,000	1080	205

2.3. Production details

The casting machine (extruder or slip-former) produces hollow core slabs while advancing at the speed of 1.10 - 1.50 m/min.
Workmen following the machine perform the manual operations on the fresh concrete needed to meet design specifications (see Fig. 2.12).

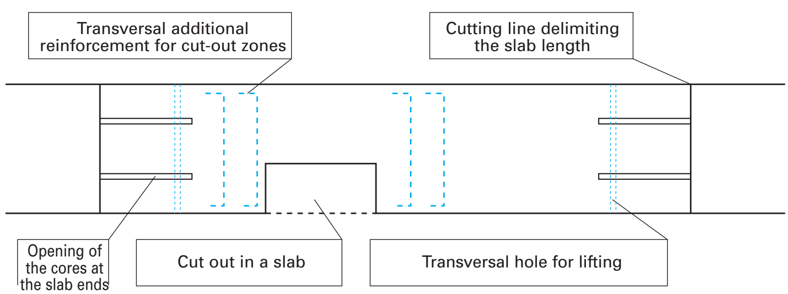


Fig. 2.12 Manual operations on a slab immediately after casting.

The first operation is the tracing of the cutting line delimiting the length of the slab with the immediate application of its identifying mark.

Lines for any shaping to be done on the fresh concrete are also drawn. Following this, some cores at the slab ends are opened, holes for lifting are drilled and any supplementary normal reinforcement required is added.

Tracing operations are usually performed manually by a qualified operator.

Electronic plotter machines are now in use. These are based on the CAM (Computer Aided Manufacturing) method and are expected to become very common in the near future.

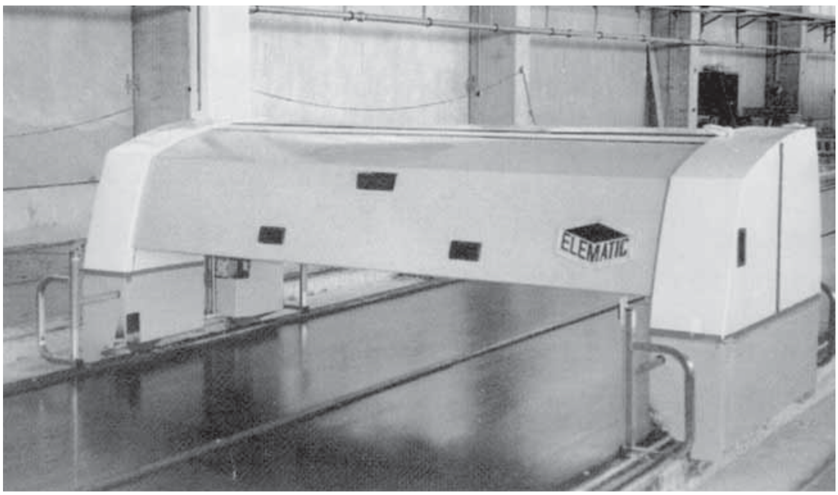


Fig. 2.13 CAM plotter machine equipped with an automatic marking device.

Direct interventions on the concrete to make cut-outs or add extra reinforcing bars will continue to be done by hand for a long time to come, possibly with the aid of hydropneumatic tools for the removal of still-fresh concrete.

When the concrete has hardened sufficiently and the slabs have been removed from the casting bed, other operations are performed: holes for draining rain water are drilled and plugs are applied to the ends of the cores as described in the following paragraphs.

2.3.1. Open cores at slab ends

The opening of the cores illustrated in Fig. 2.14 is to house and anchor the normal reinforcing bars required by the design for connection at slab supports, for the resistance to negative moments and for the absorption of shearing-bending forces on slab ends.

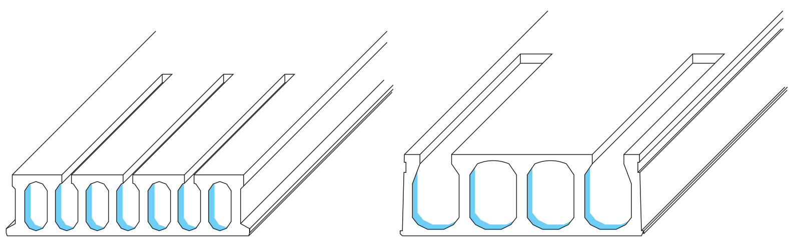


Fig. 2.14 The open cores at the ends of hollow core slabs

The open cores may vary in number and length depending on static needs when the longitudinal joins between adjacent slabs, which are usually placed every 120 cm, by themselves do not allow proper distribution of reinforcement.
It is important to note that the connection bars at slab ends can be dislocated every 120 cm only in presence of very moderate loads (roofs and so on) or in the case of a large support and thus of verified anchoring of the prestressing strands.

Continuity between hollow core slabs requires a distribution of connecting bars at least every 60 cm as a safeguard against negative moment.
Continuity between clear span floors requires an even thicker distribution of connecting bars, at least one every 30 or 40 cm.

The length of the open cores must be calculated in such a way that shear tension at the interface between the concrete filling the cores and that of the prefabricated element meet specified values (see paragraph 3.4.1. below, values of τ_{Rdj}).

It is to be noted that in the case of a filled core, in which both the continuity reinforcement of the floor and the tie bars are anchored, verification of bonding between the precast and the filling concrete must be performed for the sum of the stresses involved.

2.3.2. Sheaths for strand neutralization

In some cases with strong reinforcement the neutralization of some tendons up to about 70 cm in correspondence to the ends of the single hollow core slabs may be required.

The production cycle, whether by extrusion or slipforming, allows the use of sheaths to neutralize prestressing strands.

Along the casting bed, at the points at which each slab is to be cut, the segments of tubular sheathing have to be fixed on the strands which have not yet been stretched (for the operator's safety) taking into account the amount of displacement of each fixing point caused by subsequent stretching of the strand.

This operation must be performed under the supervision of experts for the reasons given below.

- On advancing, the casting machine tends to push the sheaths forward if they are not perfectly attached to the strand to be neutralized.

- The use of open sheathing to be inserted on already stretched strands is not recommended because the concrete would be forced into the sheath by the machine, thus making it ineffective.

- The use of rigid plastic tubes of the kind used for electric cable ducting, is recommended. Of course the tube must be placed on the strand before it is anchored for stretching.

– Also not recommended are alternatives such as greasing strands in the parts to be neutralized or painting strands with substances inhibiting adhesion of concrete since the advancing of the machine pushes such substances into areas where they should not be.

2.3.3. Additional reinforcement bars

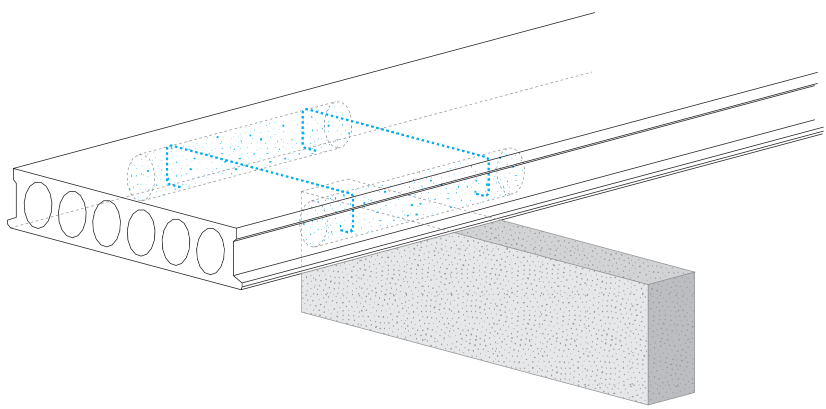


Fig. 2.15 Example of transverse reinforcement inserted manually into fresh concrete of a partially supported slab.

There are many reasons for inserting additional reinforcement bars into the still-fresh concrete of the slabs as shown in Figures 2.12; 2.15; 2.16 and 2.17. The designer must realize that these operations, although easily implemented, are costly and can be performed only on a limited number of slabs for a given order.

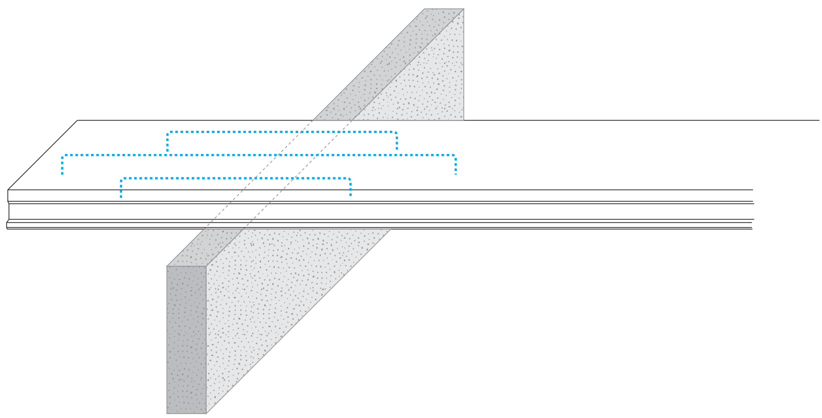


Fig. 2.16 Example of longitudinal reinforcement inserted manually in the still-fresh hollow core slab in correspondence to a support with a heavily loaded cantilever

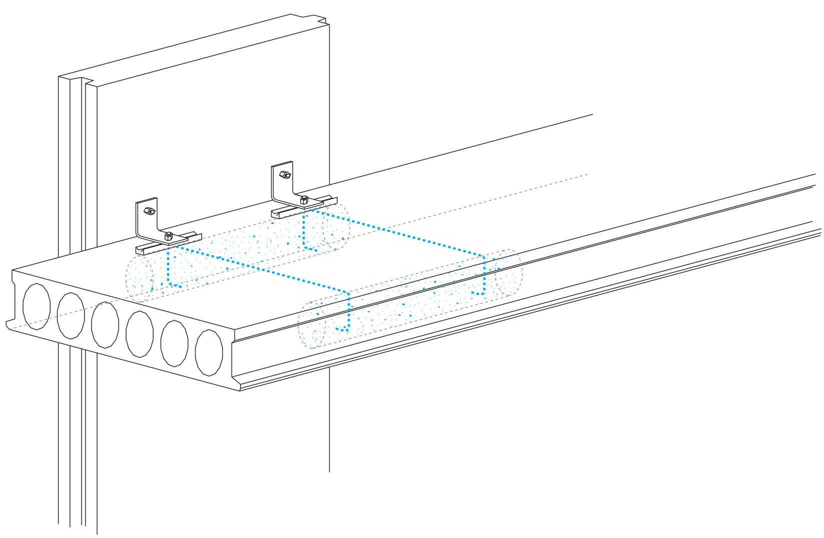


Fig. 2.17 Example of the proper anchoring of boltable profiles on the outer edge of a hollow core slab.

2.3.4. Cut-outs in hollow core slabs

Very often hollow core slabs must be cut or notched for adaptation to design geometry.
These operations are performed immediately after casting, when the concrete is still fresh, with removal of the concrete around the stretched strands so as to allow the cutting of the strands following hardening.

In any case, the needs of the building design must be compatible with slab bearing capacity, which can stand limited cut-outs. They must be agreed upon with the designer and one of the manufacturer's engineers.

The cut strands in the zone in which the concrete has been eliminated apply prestressing in parts of the slab that are far from the ends. This creates internal stresses which may lead to longitudinal cracking of the concrete which compromises the integrity of the slab during storage, transportation and assembly of the structure.
Production managers thus must rely on all their experience when applying the indispensable precautions to assure the integrity of notched slabs, their respondence to the needs of dry assembly and finally to maintain the load-bearing capacity of the slab, even in the places weakened by cuts.

Normally speaking, small cut-outs in slab ends do not create any special problems so long as they do not exceed 40 cm in length or width, just as the openings in the body of the slab that involve the strands of one or two webs must not exceed 60 cm in length.
It is in all cases necessary to verify the residual load-bearing capacity of the notched slab.

In correspondence to larger cut-outs, it is almost always necessary to reinforce the slab with normal reinforcement embedded in the concrete while still fresh (see Fig. 2.18).

Special attention is required for slabs placed at the two extremities of a floor. Lateral cut-outs in edge slabs are to be avoided unless proper stiffening is applied.

It is also necessary to prepare for the lifting of the cut out slab by providing points that assure proper balance when lifted.

Fig. 2.18 gives examples of holes and cut-outs that can be made with the necessary precautions, which are highlighted below.

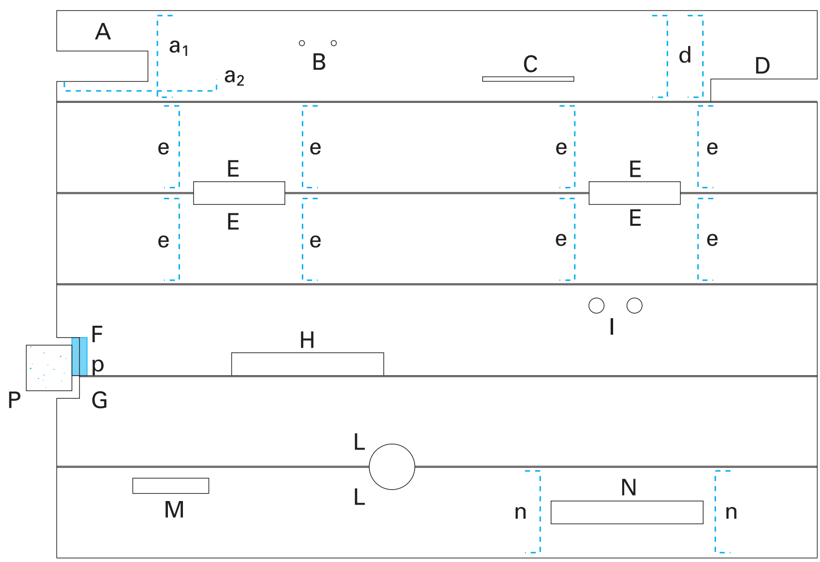


Fig. 2.18 *Several possibilities of cut-outs and holes that can be made in 120 cm-wide hollow core slabs using the proper techniques*

Notch A - It can be up to 40 cm wide and must be at a distance of not less than 30 cm from the lateral edge of the slab. When its length is more than 50 cm transverse reinforcement -a_1- and longitudinal reinforcement -a_2- are required. Verification of shear is necessary.

Holes B - when their width involves only one core there are no problems.

Slot C - when its width involves only one core there are no contraindications.

Cut-out D	-	it must not be wider than 40 cm; transverse reinforcement -d- may be necessary and verification of shear is indispensable.
Notches E	-	the width of each half-notch shall never be more than 40 cm; transverse reinforcement -e- and verification of loadbearing capacity are indispensable.
Cut-out F	-	in correspondence to prefabricated pillars - P - the large cut-outs - F - must lie on metal brackets - p - joined to the pillar by means of expansion pins.
Cut-out G	-	when the size of the offset is below 40 x 40 cm no special measures are called for, with the exception of shear checking.
Cut-out H	-	when the width is ≤ 20 cm no special measures are required except for calculation of loadbearing capacity.
Holes I	-	holes bored in situ are very useful for the passage of last-minute piping. It is indispensable to recalculate loadbearing capacity owing to interrupted strands.
Half-holes L	-	the radius shall not be greater than 40 cm. They are to be bored in the fresh concrete. Verify residual loadbearing capacity.
Holes M and N	-	width shall not be greater than 30 cm. Residual loadbearing capacity shall be verified. If length is above 60 cm, it is necessary to add reinforcement - n n - These holes can also be made in situ with diamond disks.

2.3.5. Ways of lifting

Fig. 2.5 shows the lifting of hollow core slabs by means of a rocker arm with grip cheeks. The alternative system of lifting with cables and rods (see Fig. 2.19) is quite useful when delivering a limited number of hollow core slabs to many different construction yards.

The transversal hole for insertion of the rod has an inner diameter of ~ 40 mm and weakens, although only slightly, section shear strength.
It is recommended to bore the transversal hole in the lower part of the slab and to use this type of lifting device with slabs having a thickness of no less than 20 cm.

In the case of slabs having a maximum thickness of 20 - 24 cm and with a weight normally limited to 2.4 tons, lifting with forks made of special steel with a certified maximum load is sometimes resorted to (see Fig. 2.19).

As prescribed by Italian prefab Regulations (Art. 2.2.1.), it is recommended to verify the shear and cantilever tension at the lifting section by considering the weight of the slab multiplied by the minimum dynamic coefficient 1.15.

Given the lack of transverse reinforcement in the upper part of the slab undergoing lifting stresses, it is advisable, for safety's sake, to use emergency cables with rapid-release spring catches just before their final laying in place (see Fig. 2.19).

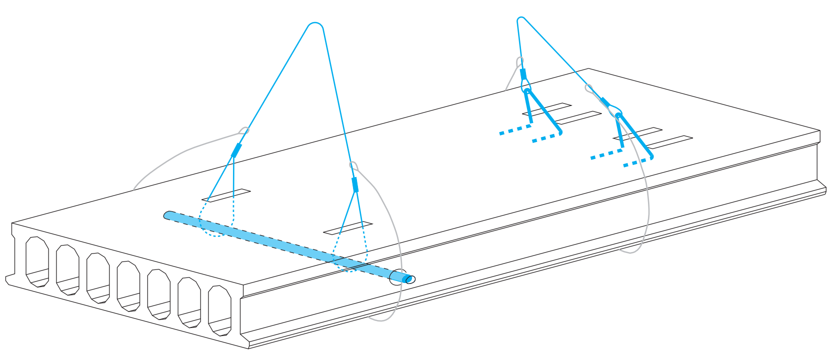


Fig. 2.19 Holes for lifting with cables and blocking rod, or with tested and certified steel forks. Note the safety cables to be detached just before placing the slabs in position

2.3.6. Holes for draining rainwater

It is known that hollow core slabs, after being placed in position, but before they receive their waterproofing cover, are exposed to the elements and a great deal of rainwater may accumulate in them.
Indeed, the upper surface of the slabs, even when supplied with concrete topping, always present discontinuities due to differential shrinkage of additional casts or to microcracking. For this reason, it is more permeable than the uniformly compact bottom surface, which is made even more impermeable by prestressing.

In regions with temperate or warm climates the presence of water in the hollow cores can create the bothersome problem of dripping water, which may begin even after a long time from completion of the work.
In colder regions, the englobed water may freeze and cause ugly longitudinal lesions in the concrete on the lower surface.

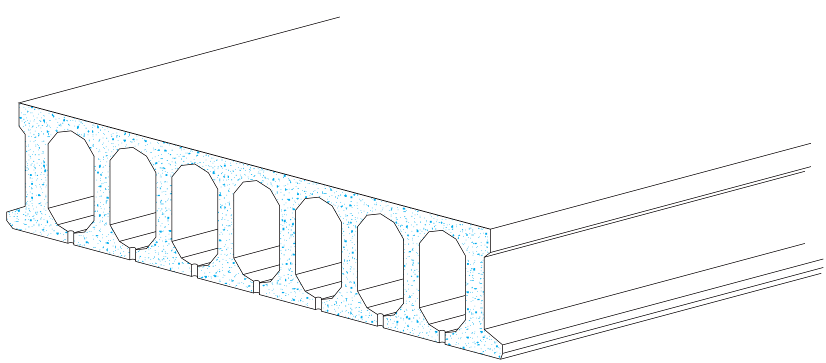


Fig. 2.20 Boring for the drainage of rainwater. Diameter about 6 - 8 mm

It is to be pointed out that even in this case the damage does not involve the statics of the floor because there is no variation in the prestressed surface.
These problems can be avoided by boring a hole in the intrados of each slab in correspondence to each void immediately after removal from the casting bed (see Fig. 2.20).
For this purpose a battery of percussion drills is used when required by customers.

2.3.7. Plugs for hollow cores

When hollow core slabs having a depth above 20 - 25 cm in the presence of beams cast in situ or tie beams at intermediate supports, it is recommended to infill the cores at the slab ends and at the end of the open cores so that the cast concrete does not penetrate into them, thus causing an unnecessary increase in weight due to wasted concrete.

Plugs, which may be of polystyrene, pressed plastic or spongy plastic, are usually placed flush with the end of the floor slab.

Only in the case of clear span floors on beams cast in situ or having the same depth as the floor, it is indispensable for the plug to be inside the core at a distance from the slab end at least equal to the depth of the floor, for the hanging of the slabs.
During vibration of the cast the plug must not be pushed inside the core. For this reason it must be compressed at the time it is placed in the core or held in position with a blocking device of some kind.

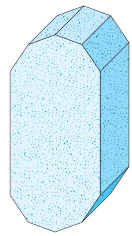
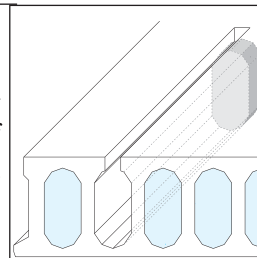
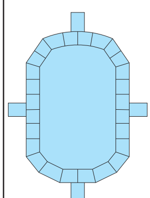
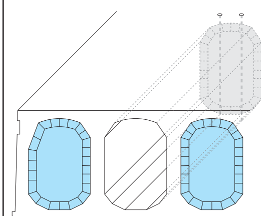
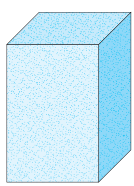
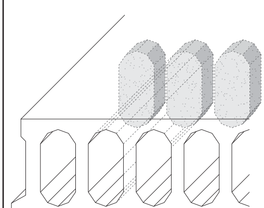
	Polystyrene plug. It is positioned flush with the outside of the end of the slab and in such a way as to close the open cores.	
	Pressed plastic plug. It is positioned flush with the outside of the slab end or, if placed inside the core in depth, it is held in place by two metal bars placed vertically from above in the fresh slab.	
	Spongy polyurethane plug used exclusively inside the core. It holds its position owing to the force of expansion it exerts.	

Fig. 2.21 Plugs for hollow cores.

2.3.8. Devices for eliminating camber deviations

Because of normal production tolerances, different production, and therefore hardening times, and sometimes because of the different positions in which slabs are stored, there may be different prestressing and creep values in slabs of the same kind with identical reinforcement. Thus, when they are mounted they may show different cambers which show up at the intrados of adjacent slabs.

These eccessive differences can be corrected and brought to acceptable values by means of special devices that must be applied prior to the casting of longitudinal joins (see Fig. 2.22).

The devices illustrated here work best with the thinnest slabs and usually consist of screw clamps or steel rod struts.

In the case of thick slabs it is necessary to apply two or even three clamps to reduce a single defect in camber. Naturally it is necessary to isolate the clamp from the concrete so that it can be removed freely as soon as the surrounding cast is sufficiently hardened, which is to say after about 2 or 3 days.

As an alternative to clamps it is possible to use steel rod struts to hold the lateral profile of the unaligned slab in the correct position. The lower slab must be pushed upwards and held there by one or two struts. The conformation of the profile must be such as to facilitate blocking.

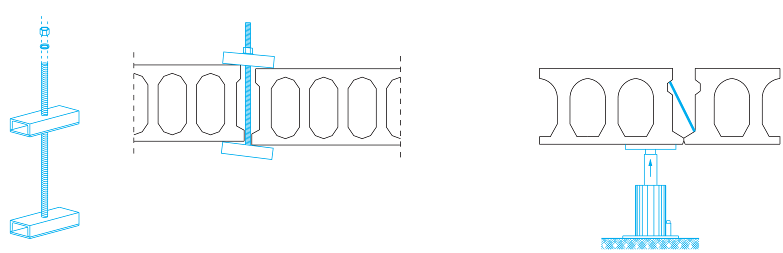


Fig. 2.22 Example of a screw clamp and a 16 mm steel rod strut to reduce defects in camber.

2.4. Dimensional tolerances

The geometric dimensions of the single hollow core slabs and the spatial coordinates of their relative positioning may be slightly different from nominal design values so long as the dimensional variations are acceptable for the structure being built. These dimensional variations are called tolerances.

For components having a static function, tolerances must be defined in the plans of the project so as not to compromise the safety of the structure in the various stages of its life.

There are three groups of tolerances which sometimes tend to add to one another, thus making them more critical:

- tolerances in the production of the artefact (or dimensional tolerances);
- tolerances in assembly of the artefact;
- tolerances of centre distances between loadbearing structures prepared in situ to receive the component.

All these tolerance limits must be clearly indicated by the designer as a function of the type of loadbearing structure; these values must be taken into account in checking safety of the structure.

As a rule, the following tolerances for hollow core slabs will be used. They have been taken from Eurocode 2 ENV 1992-1-1, from the standard EN 1168 and from the FIP document entitled QUALITY ASSURANCE OF HOLLOW CORE SLAB FLOORS.

Designers or manufacturers may adopt tolerance values different from the ones indicated herein so long as they are clearly indicated in the plans and so long as the entire project conforms to planned values.

As concerns other problems in the acceptability of hollow core slabs, designers may consult FIP document: "QUALITY ASSURANCE OF HOLLOW CORE SLAB FLOORS" which is sometimes used as a contract reference document in the supplying of hollow core slabs for floors.

2.4.1. Tolerances in dimensions and assembling

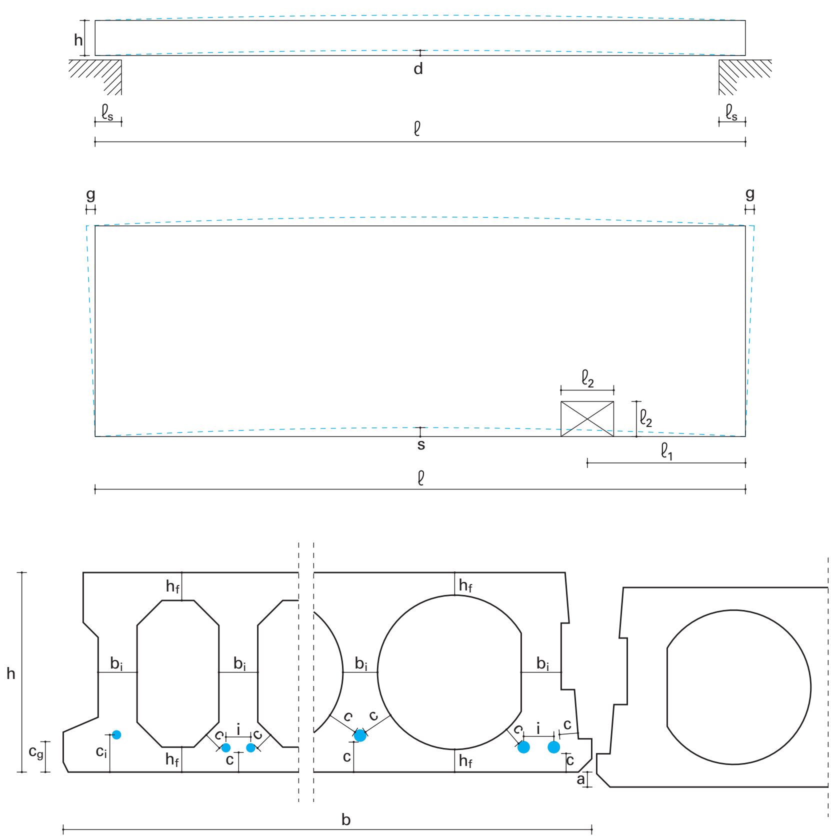


Fig. 2.23 Geometric references for dimensional and assembly tolerances

Maximum deviations:

1) Length (ℓ) of slab (EN 1168 par. 4.3.1.5.2) ± 25 mm
When the support lenght is minimum, a smaller deviation value is recommended.
(FIP ASSURANCE Table 4 point 1)

2) Width (b) for a standard slab (EN 1168 par. 4.3.1.5.2) ± 5 mm
For slabs under standard size ± 15 mm ÷ ± 25 mm
(FIP ASSURANCE Table 4 point 2)
Width is measured at the widest point of the slab

3) Slab height (h) for h $\leq$ 150 mm -5 mm + 10 mm
for h = 440 mm ± 15 mm
For other heights linear interpolation is applied
(EN 1168 par. 4.3.1.5.1.a)

4) Nominal minimum web thickness.
Individual web (b_i) - 10 mm
Total per slab (b_w) - 20 mm
(EN 1168 par. 4.3.1.5.1.b)

5) Nominal minimum flange thickness above and below the cores.
Individual flange (h_f) - 10 mm + 15 mm
Mean value (above or below) - 5 mm
(EN 1168 par. 4.3.1.5.1.c)

6) Position of lower prestressing steel reinforcement.
Individual strand or wire (c_i)
For h $\leq$ 200 mm ± 10 mm
For h > 200 mm ± 15 mm
Mean value per slab (c_g) ± 7 mm
(EN 1168 par. 4.3.1.5.1.d)

7) Concrete covering tendons (c) and centre distance (i) between strands.
Deviations with respect to minimum design values - 5 mm
if verified by quality control of production
(EC2 - ENV 1992 -1-1 par. 4.1.3.3. Point 8)

8) Misalignment (g) of cut with disk saw 10 mm
(FIP ASSURANCE Table 4 point 6)

9) Sweep (s)
(FIP ASSURANCE Table 4 point 7)
for $l \leq 12$ m 5 mm
for $l > 12$ m 10 mm

10) Camber (d) (see par. 5.7 Chapter 5).
Mean deviation from calculated value $\pm \ell/1000$
Maximum deviation for singular slabs $\pm \ell/500$
Maximum calculated camber shall be limited to $\ell/300$
(FIP ASSURANCE Table 4 point 8)

11) Openings, blockouts and fixing plates (l_1 and l_2).
Cut-outs in fresh concrete ± 25 mm
Holes bored in hardened concrete ± 15 mm
Fixing plates moored at works ± 20 mm
(FIP ASSURANCE Table 4 point 9)

12) Lack of flatness in single slab due to helicoidal torsion.
Maximum deviation from flatness 15 mm
(FIP ASSURANCE Table 4 point 10)

13) Slippage of strands into slab end Δl_o (See Table 2.3)
(for calculation see EN 1168 par. 4.2.3.2. and EC2 ENV 1992-1-1 par. 4.2.3.5.6. point 4 and Table 4.7).
Values given here are applicable in the following circumstances:

- concrete strength on release of strands $f_{ck} = 30$ N/mm^2
- steel stress following release $\sigma_{pmo} = 1250$ N/mm^2
- upper bound value of transmission length $l_{bpd} = 84\,\phi$

Table 2.3 (See example of calculation 3.5 in paragraph 3.5.4.)

Tendon	Nominal ϕ	Mean value Δl_o	Single tendon 1.3 Δl_o
Strand 3ϕ3 mm	ϕ 6.5 mm	1.4 mm	1.8 mm
Strand 3/8"	ϕ 9.3 mm	2.0 mm	2.6 mm
Strand 1/2"	ϕ 12.5 mm	2.6 mm	3.4 mm
Strand 0,6"	ϕ 15.2 mm	3.1 mm	4.1 mm

14) Minimum nominal design support length $l_{s\ min}$
EC2 ENV 1992-1-3 Art. 4.5.5.2.: 65 mm ÷ 100 mm
Maximum deviation on assembly -25 mm
ITALIAN prefab Regulations (par. 2.4.1.):
minimum final support length following assembly 50 mm
minimum temporary support length during assembly 30 mm

15) Defect in camber (a) between adjacent slabs.
The defect is acceptable when it is less than $\ell/1000$
Or when it is lower than a contractually defined value chosen between the limit values of 8 mm and 15 mm
(FIP ASSURANCE par. 6.1.3.)
Such values are also acceptable as residual defect in camber following adjustment during assembly.

Chapter 3
STATIC PECULIARITIES

3.1. Introduction

Prestressed hollow core slabs, as they leave the manufacturer's plant, are self-bearing horizontal structures capable of bearing, even just placed on their supports, the overloads for which they have been designed.

The simplest and most widespread use of hollow core slabs is in the formation of simply supported floors, with no in situ casting, with the exception of longitudinal joins between adjacent slabs.

Many applications, however, require more highly advanced designs.
It is possible, for example, to connect slabs to bordering structures in such a way as to obtain more complex static functions which are applicable to buildings that need monolithic structures and prolonged durability in time.

Thus it is possible to design and implement fixed ends, static continuity between floors, transverse diffusion of concentrated loads, the support of the floor as a clear span between beams cast in situ, the diaphragm action even in anti-seismic buildings and so on.

The detailed study of static peculiarities and connections that are presented in this chapter and the following one are fundamental in dealing with special cases in which a specific, high-level engineering solution is required.

3.2. Floor depth

Under the heading TECHNICAL CONSIDERATIONS (see paragraph 1.3 above) it was illustrated that the hollow core floor, with the same inertial moment as other kinds of floors, is more rigid and much less prone to long-term deformation owing to the very high strength of concrete and thus its greater modulus of elasticity.

For these reasons hollow core floors can be much thinner than other kinds of floors with the same span and loadbearing capacity.

The availability of thin floors is of primary importance in buildings where there are severe limits on height and volume, and also in underground works where every extra centimetre of excavation means an increase in costs.

However, special care must be used when designing particularly thin hollow core floors to avoid the following aesthetic and technical problems:

– excessively accentuated camber;

– defects in camber visible at the intrados between adjacent slabs due to inevitable differences;

– possibility of separation between slabs that are too thin and the concrete topping (as discussed in paragraph 3.4.1 below);

– possibility of cracking in rigid or glass partition walls;

– possibility of poor working of frames.

To safeguard the proper working and the aesthetics of structures, both Italian Standard (Chapt. 7) and Instructions CNR 10025/98 (par. 2.2.1) are oriented towards the prescription of a ratio between design span and floor thickness of l_c/h which is deemed adequate to meet rigidity requisites.

For hollow core floors without a concrete topping (Italian Standard Art. 7.3.2.) the following slenderness ratios are to be complied with:

– floors with simple support $l_c/h \leq 35$

– partially restrained or continuous floors $l_c/h \leq 42$

For hollow core floors with concrete topping of thickness **s** it is possible to consider the contribution of the topping even if its concrete class is usually C25/30 with respect to the precast concrete class C45/55. In this case the following slenderness ratios are to be complied with CNR 10025/98 Instructions, par. 2.2.1:

– floors with simple support $\dfrac{l_c}{h + s/2} \leq 35$

– partially restrained or continuous floors $\dfrac{l_c}{h + s/2} \leq 42$

Both Standards admit a waiver from above prescriptions for floors used as roofs only, so long as they are not flat covers of civil buildings with internal partitions.

Both Standards admit exceptions from the limits given above even when proper experimentation on prototypes is supported by calculations that take into account nonlinear behaviours, cracking and creep.

In such a case, both experimentation and calculation shall demonstrate that:

a) the incremental instantaneous elastic deflection $\boldsymbol{\upsilon}$ caused by rare combination of actions is $\upsilon \leq l_c/1000$

b) the incremental deflection at infinite time $\boldsymbol{\upsilon}_\infty$ caused by quasi-permanent combination of actions is $\upsilon_\infty \leq l_c/500$

Naturally the incremental deflections considered here will be counted starting from the situation of the unloaded assembled floor and will be measured from the lowering undergone by the middle section of the floor under permanent and accidental actions foreseen in a) and b), neglecting any initial camber of the floor without loads.

As concerns minimum thicknesses of simple penthouse roofs, even though not contemplated by regulations, the following slenderness ratios are recommended:

– for roofs with simple support $l_c/h \leq 50$

– for continuous roofs $l_c/h \leq 55$

The attention of the designer is drawn to the question of flat roofs that are too thin which, in the presence of very occasional loads, may become catchbasins unable to discharge rainwater.

For this kind of roof the same limits required for loadbearing floors are recommended.

The slenderness ratio of hollow core slabs used
as infill walls may be $l/h \leq 60$

3.3. Longitudinal join shape

The proper static behaviour of the hollow core floor depends to a large degree on the efficiency of the longitudinal joins, which are made by means of the in situ casting of the raceways created by placing side by side two slabs whose sides are shaped for the purpose (see Fig. 3.1).

This union effectively contributes to assuring the monolithic state of the floor. The capacity of the floor to distribute concentrated loads transversally depends on it through the obtaining of the corroboration of adjoining slabs along experimentally verified lines of influence.

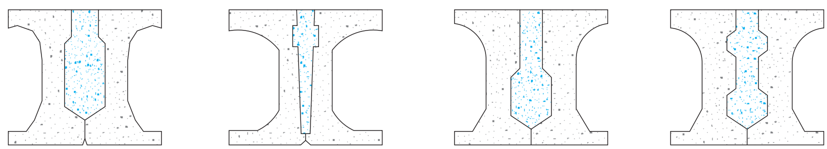


Fig. 3.1 Longitudinal joins

The longitudinal join acts as a cylindrical hinge capable of transmitting vertical shearing stresses but not flexural moments.
As can be understood intuitively, the hollow core floor can be considered theoretically as a series of beams joined together by means of cylindrical hinges.

This theoretical consideration describes the behaviour of the structure with an accuracy that is directly proportional to how closely the single slabs can be considered as beams.

On the other hand, when the single hollow core slabs are more similar to plates, it is better to consider them as a series of plates joined together by means of cylindrical hinges (see Fig. 3.2).

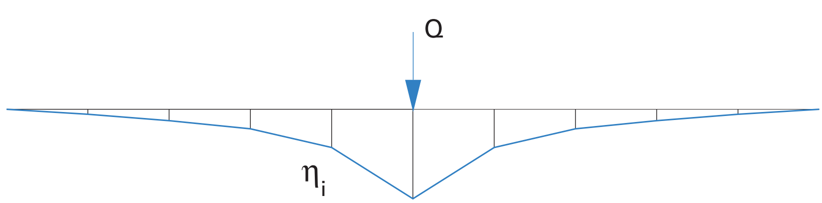


Fig. 3.2 Lines of influence of a concentrated load on a hollow core floor considered as a series of hinged plates.
(Prof. A. Migliacci - 1967 - Politecnico of Milan)

The transverse distribution of loads also depends on the number of slabs adjacent to the directly loaded union: distribution is poor if the load does not have a sufficiently wide length of floor on both sides.

When a hollow core floor has a corroborant concrete topping reinforced with a resistance-welded mesh, the longitudinal join acts as a cylindrical hinge only in the case of downward deflection, while upward deflection is inhibited by the tensile strength of topping reinforcement.

In this case the join, besides transmitting shear stresses, also transmits transverse negative moments (see Fig. 3.3).

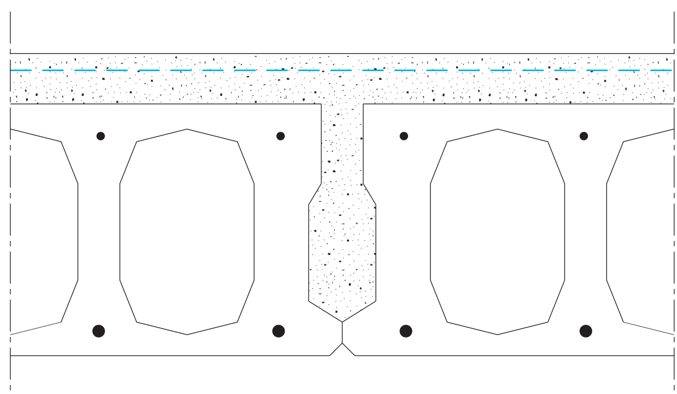


Fig. 3.3 Longitudinal join with reinforced or non-reinforced topping

When the longitudinal join must contain tie reinforcement, according to the Italian Prefab Regulations (Art. 2.11.2.b), it is considered as an edge beam and must have a minimum width of 5 cm and an average width of 8 cm (see Fig. 3.4).

These dimensions of the longitudinal join acting as the housing for normal reinforcement are valid especially in seismic areas, but are also recommended for non-seismic areas for the following reasons:

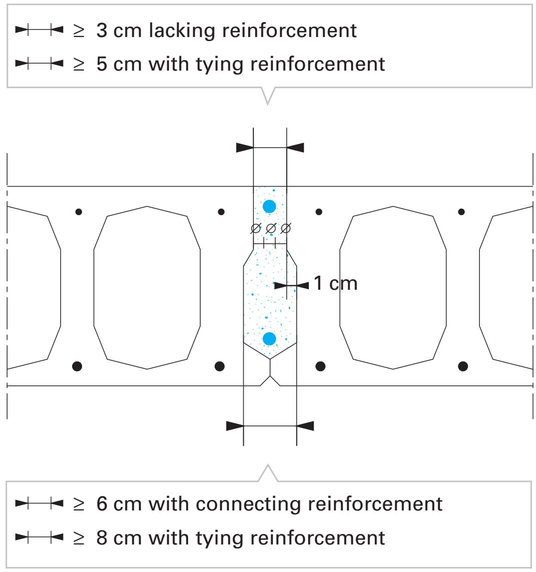


Fig. 3.4 *Longitudinal join: minimum dimensions in presence of connecting reinforcement and when tying reinforcement is called for (Italian Prefab Regulations, par. 2.11.2 b point 3)*

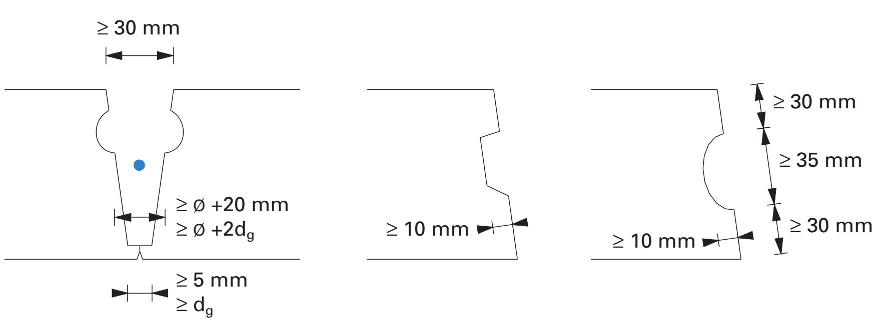


Fig. 3.5 *Longitudinal join: shapes and minimum dimensions (proposed in the European Standard EN 1168 Art. 4.3.1.4 and Informative Annex B)*

- normally, a part of the connecting reinforcement of the floors is placed in the upper part of the join; this reinforcement must be completely embedded in the concrete;

- the connecting reinforcement between the hollow core floor and beams cast in situ without support for the floor is placed in both the upper and lower parts of the join; the embedding of this reinforcement must assure the greatest functionality and protection;

- in correspondence to the supports, however they may be made, a part of the reinforcing bars for the transmission zone must be placed at the lowest possible level in the join raceway (Italian Standard, Art. 5.3.1.). This reinforcement must be well embedded in the concrete.

Special attention is required during the casting of concrete in longitudinal joins to make sure that shear stresses are transmitted properly.

The FIP Guide “Quality assurance of hollow core slab floors” (point 6.3), prescribes the minimum class of the casting (C20 – C25), the maximum diameter of the aggregate (not above 8 mm), the consistency of the casting, the cleanliness of the raceway and especially that the conglomerate must not present cracks due to shrinkage later on.

In presence of special cyclic or vibrating loads it is necessary to use more resistant concrete, possibily with the predisposition of pockets for the inclusion of transverse connecting stirrups.

It is to be pointed out here that the term "longitudinal join", is used in this book in place of the more common "longitudinal joint" since the term "join" means connection between structural components capable of transmitting stresses.
On the other hand, by "joints" is meant spaces between structural components allowing mutual shifts without the transmission of stresses.

3.4. Concrete topping on the hollow core floor

Hollow core floors usually do not require topping. But it is called for when it is necessary to provide a loadbearing capacity greater than that allowable for a floor without topping, or when plans require discrete or continuous transverse reinforcement.

Usually reinforcement embedded in the topping is in the form of a resistance-welded mesh.

In reality, the topping, even when made of concrete having a characteristic strength of only half that of the concrete used in the precast slab, increases the inertia of the floor, thus making it capable of supporting loads that are greater than those that can be borne by the floor without topping.

Topping improves the overall rigidity of the floor and, especially when reinforced, notably increases the transversal distribution of concentrated loads.

It also favours the plate or diaphragm action of the floor (see Italian Standard, par. 7.3.1.). Consequently, hollow core floors used in the construction of small bridges are usually designed with a reinforced topping.

In the hollow core floors of industrial buildings and garages, where loads are mobile, the reinforced concrete topping is often quartz smoothed so as to have a low-cost finish.

Italian Standard (par. 7.3.4.) states that to be statically corroborant, the cast in situ topping must have a thickness of at least 4 cm, and be reinforced with a resistance-welded mesh.

The Italian Prefab Regulations (par. 2.11.1.3) make it obligatory in seismic areas for floors composed of precast elements to have transversal reinforcement capable of transmitting horizontal stresses.

For hollow core floors in seismic areas it is thus convenient to include in the topping the transversal reinforcement connecting the peripheral ties.

3.4.1. Interface shear capacity between in situ topping and precast slab

It is essential to be attentive to problems of solidation between the cast in situ concrete topping and the upper surface of the slab which has already hardened for some time. This is also true of the reinforced cast of an open core or a longitudinal join and the slab itself.

The Italian Standard (par. 7.1.6.2.) imposes verification that serviceability limit state for shear stress between a precast slab and concrete cast in situ shall be less than 0.30 N/mm^2 for a smooth contact surface and less than 0.45 N/mm^2 for a rough surface.

The European Standard ENV 1992-1-3 and the updated pr EN 1992-1 are far more cautious on this subject since they are quite similar in prescribing that between precast concrete and concrete cast in situ (thus also between the topping and the hollow core slab or between the casting of an open core and the precast slab, the longitudinal shear stress τ_{sdj} calculated at the interface shall not be above the following value at the Ultimate Limit State (ULS):

$$\tau_{sdj} \leq \tau_{Rdj}$$

with

$$\tau_{Rdj} = k_t \tau_{Rd} + \mu \sigma_N + \rho f_{yd} (\mu \sin \alpha + \cos \alpha)$$

and, in any case, when stirrups connect concrete topping to the slab, it is also necessary to verify the following condition:

$$\tau_{Rdj} \leq 0.5\, \nu\, f_{cd}$$

where

τ_{Rdj} = design shear resistance per surface unit at the ULS.
ν = efficiency factor ~ 0.5
f_{cd} = design value of cylinder compressive strength of topping concrete or concrete cast in the join between two slabs and in open cores (see Table 5.1 in Chapter 5, below)
k_t = 1.4 and $\mu = 0.6$ for extruded or slipformed surfaces (ENV 1992-1-3 par. 4.5.3.3 Tab. 4.115)

k_t = 1.8 and $\mu = 0.7$ for surfaces raked to a depth of at least 3 mm (same Table).

τ_{Rd} = $0{,}25\ f_{ctk\ 0.05} / \gamma_c$ design shear strength (values are given in Table 5.1, Chapter 5, below)

σ_N = tension per surface unit of an external vertical force (if this is present); σ_N is positive if the vertical force causes compression and negative if it causes traction; in any case $\sigma_N \leq 0.6\ f_{cd}$

ρ = A_s/A_c with A_s = area of the stirrups crossing the joining surfaces (if present)
and A_c = area taken into account for shearing stress

f_{yd} = calculated value of tensile strength of reinforcement

α = angle between reinforcement and the surface of the join, but ≥ 45° and ≤ 90°

μ = friction coefficient.

Lacking metal links and external vertical forces it must therefore be

$\tau_{Sdj} \leq \tau_{Rdj} = k_t\, \tau_{Rd}$

This signifies that with a concrete topping having characteristic cylinder/cube strength of C 25/30 we have (Table 5.1 Chapter 5, below being $\gamma_c = 1.6$):

$f_{ck} = 25\ N/mm^2$

$\tau_{Rd} = 0.28\ N/mm^2$

$f_{cd} = f_{ck}/\gamma_c = 15.6\ N/mm^2$

Thus the maximum interface shear resistance τ_{Rdj} between concretes at ULS will be:

- for extruded or slipformed surfaces ($k_t = 1.4$)
  $\tau_{Rdj} = 0.39\ N/mm^2$
- for surfaces raked to a depth of at least 3 mm ($k_t = 1.8$)
  $\tau_{Rdj} = 0.50\ N/mm^2$
- with metal connections it may be
  $\tau_{Rdj} = 3.90\ N/mm^2$

The maximum value $\tau_{sdj} \leq 0.39$ N/mm^2 may be considered acceptable even for cast open cores containing normal reinforcement, so long as there is no possibility of longitudinal cracking. In the case of a longitudinal join cast between two adjacent slabs and containing connecting reinforcement the average shear value must be limited to:

- $\tau_{Rdj} \leq 0.1$ N/mm^2

as prescribed in paragraph 4.5.3.3, point 106, of the ENV 1992-1-3 Standard.

Coming back to the case of the interface between the cast topping and the top of the hollow core slab, it is so vast that in practical cases the shear value is always $\tau < 0.20$ N/mm^2 in the Serviceability Limit State.

Static behaviour would thus appear to be safeguarded in all cases, even without the rough surface of the slab. But in some cases there may be detachment of the topping from the slab in the presence of special conditions:

– floor too thin deflected by the frequent passage of mobile loads;

– presence of vibrating or cyclic loads inducing a loss of adherence due to fatigue and/or a different elastic response between the concrete of the topping and that of the slab;

– presence of dust or impurities on the surface of the slab prior to the casting of the topping;

– presence of stagnant water on the prefabricated floor prior to casting of the topping.

To eliminate the possibility of detachment it is therefore always preferable to corrugate the top surface of the slab by means of a special rake applied to the casting machine and to wash down the surface prior to cast the topping avoiding puddles.

In presence of shear stress $\tau > 0.20\ N/\ mm^2$ in S.L.S. it is also recommended to insert joining stirrups in the longitudinal joins and open cores at the ends of the floor slab.
As indicated in Fig. 3.6, these stirrups must emerge from the extrados of the hollow core slabs for anchorage of the topping.

Further verification of the mesh-reinforcement of the topping and the linking stirrups may be made necessary by so-called second-order tensions when shear stresses or axial compression stresses in the topping exceed mean values. These actions may be met when:

- tractions in the topping due to its shrinkage with respect to the already-hardened concrete may require additional reinforcement at supports (these tractions may however be compensated for by creep in the concrete of the precast slab and by the consequent greater compression of the topping under service loads;

- elastic instability of the topping excessively compressed by service loads may require some additional stirrups between topping and slab in correspondence to the floor centre line.

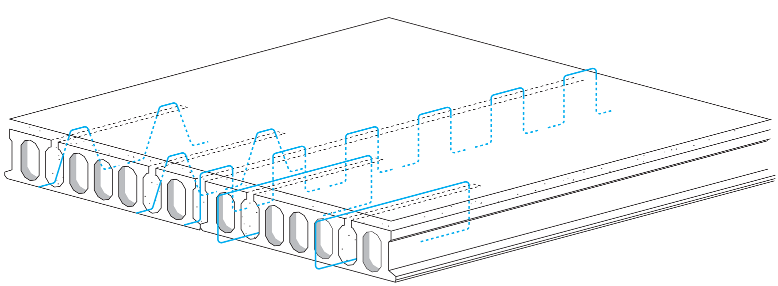


Fig. 3.6 Possible connecting reinforcement between hollow core floor and the corroborant cast in situ topping

3.5. Prestressing

The main reinforcement of hollow core slabs is composed of steel wires or strands with a high elastic limit prevalently positioned underneath the vertical webs, where the section of concrete allows optimum covering of the steel.

The pattern of the reinforcement remains rectilinear for the entire length of the artifact.

The wires or strands are strained prior to casting. When released, they exercise a force which, through the steel-hardened concrete bond, stresses the floor slab, thus inducing higher compression stresses on the bottom side and lower stresses, or even traction, on the upper side.

Prestressing contrasts positive flexural moments acting on the structure, thus neutralizing tractions which would be induced on the bottom side of the precast slab. At the same time it reduces compressional stresses on the upper side if there are pre-existent tractions in it.

In thinner hollow core slabs (up to about h = 20 ÷ 25 cm), the centre of gravity of the prestressing steel often remains within the inertial core of the transverse section and therefore prestressing does not generate tractions in the concrete at the upper side.

In hollow core slabs of 25 or more cm in thickness the centre of gravity of strands always remains below the inertial web core, thus generating tractions at the upper side. The greater the eccentricity and diameter of the tendons the greater the traction generated (see Fig. 3.7).

In such a case there may be cracking during handling, transportation and the lifting into place of the slab when tractions due to the weight of slab ends that jut out from the lifting devices are added to those of prestressing tractions.

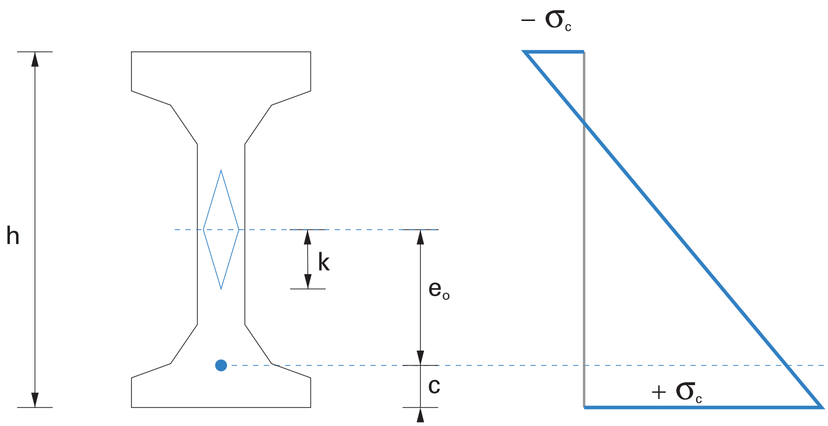


Fig. 3.7 The inertial core and the eccentricity of a prestressing tendon

Eccessive tractions in the concrete on the upper side also generate accentuated cambers in the slabs. These can be reduced by adding prestressing reinforcement in the upper part of the element.

The design of a prestressed hollow core structure considers the whole cross section as entirely reacting and it limits tractions in the concrete to acceptable values in the elastic field.

At the two extremities of the slab, for a length called the "anchorage zone" or "transmission zone" of the tendons, we have the development of prestressing from zero up to the value of total prestressing (see Fig. 3.8).

The transmission length is equal to 70 diameters of the strand for the Italian Standard (Art. 3.2.9), while for Eurocode 2 ENV 1992-1-1, (Art. 4.2.3.5.6. schedule 4.7) it depends on the surface characteristics of the tendons and the real characteristic strength of the concrete at the time of the application of prestressing. For this reason it may even be greater than 70 diameters.

For the Italian Standard (Art. 3.2.9) the "anchorage" or "transmission" zone is to be considered not prestressed and thus must be verified for shear and flexural moment like a normal partialized cross section of normal reinforced concrete whose traction strength is assured by the addition of normal reinforcing bars which, in the hollow core slab, are embedded in the longitudinal joins and in the open cores.

For the Italian Standard as for Eurocode 2 (ENV 1992-1-1, Art. 4.2.3.5.6), prestressing in the anchorage zone develops from zero to 100% following a parabolic law which, for conveniency's and safety's sake can be assimilated to a linear growth having the length of anchorage as shown in Fig. 3.8.

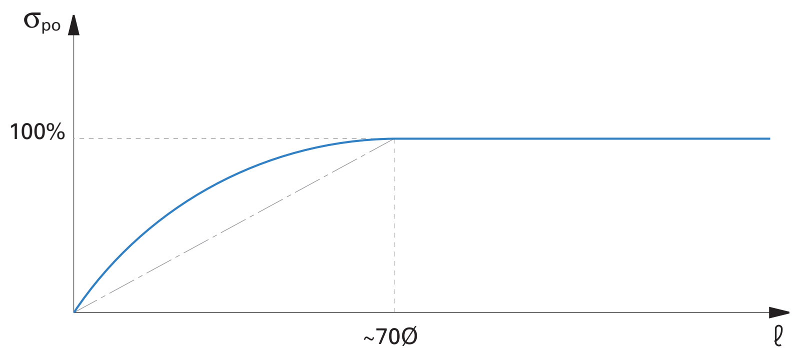


Fig. 3.8 Development of prestressing in the anchorage zone

3.5.1. Tensile forces in the transmission zone

Anchorage by bonding of the prestressing tendons is greatly influenced by the presence or absence of cracking in the transmission zone.

To avoid all cracking it is therefore indispensable to verify traction stresses in the anchorage zone as prescribed by Eurocode 2 ENV 1992-1-1, Art. 4.2.3.5.6. points 7, 8 and 9 and by the Italian Standard.

Following is an analysis of the main traction stresses in the anchorage zone of the prestressed hollow core slab dealt with in the Standard EN 1168, Art. 4.3.1.6 and in the FIP Quality Assurance document, Fig. 14 and Table 3.

Cracking situations in the anchorage zone are caused by tensile forces named bursting, splitting and spalling.

Bursting tensile force

Bursting is generated by strand slippage into the slab end. The end of the stretched strand may, on being cut, widen slightly to become a wedge which, on shrinking into the concrete, even slightly, may create traction stress.

Incorrectly positioned strands in too-narrow concrete sections may cause bursting as shown in Fig. 3.9.

The length of cracks of this kind is usually no more than 8 ÷ 10 cm and the only consequence is the lengthening of the anchorage zone to the length of the crack.
These cracks may favour the slippage of strands beyond acceptable limits (see paragraph 3.5.4 below).
These problems do not arise when the thicknesses of the covering of strands of 2 ϕ are complied with as indicated in Fig. 2.11 and paragraph 2.2.4 herein, since in these cases bursting is easily contained by the concrete and causes no problems.

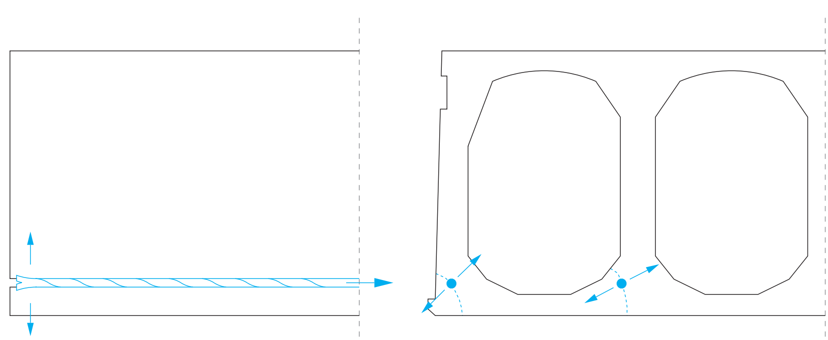


Fig. 3.9 Bursting may cause small cracks to appear in the hollow core slab end

Splitting tensile force

These stresses are caused by the development of prestressing in the anchorage zone as shown in Fig. 3.10.

The progressive anchorage of the tensioned strand gradually transfers prestressing forces to the concrete.

These forces, since they have a sloping trend, generate traction stresses in the concrete.

If the latter are greater than the chararacteristic tensile strength of the concrete, they may generate a typical horizontal crack going from one strand to another to the point of almost causing detachment of the lower edge of the slab.

These lesions, like those caused by bursting, are of limited depth and represent a problem of aesthetics at the slab end.

As in the case of bursting, cracks may lengthen the anchorage zone to the length of the cracks and may also favour strand slippage into the slab end concrete.

Splitting is avoided by maintaining the distance between strands and cover requirements indicated in paragraph 2.2.4 above.

These lesions, if any, involve the support zone of the slab. Thus, since their extension is quite limited, the slab remains acceptable.

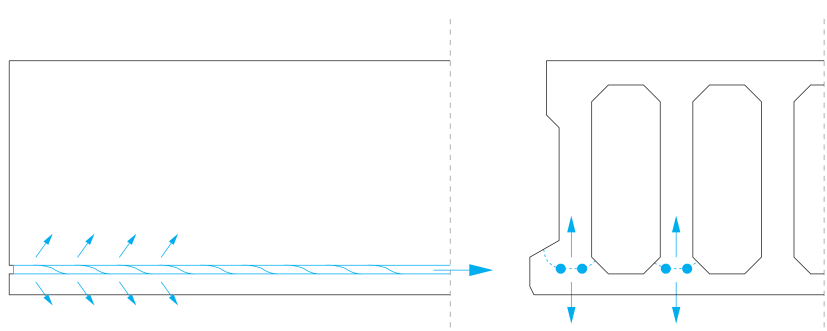


Fig. 3.10 *Splitting at the slab end may cause a small crack that unites strands that are too close together.*

Spalling tensile force

This is not to be confused with splitting since it occurs above the axis of the strands in the zone of the hollow core slab end where the webs are at its minimum width (see Fig. 3.11).

These stresses are also caused by development of prestressing in the concrete of the slab ends where only the lower part holding the strands begins to be prestressed.

This anchorage zone undergoes a combined compressive and bending stress which tends to detach it from the still nearly inert upper part.

The combined compressive and bending stress takes place through stress flow lines whose direction is not parallel to the axis of the slab (see Fig. 3.12).

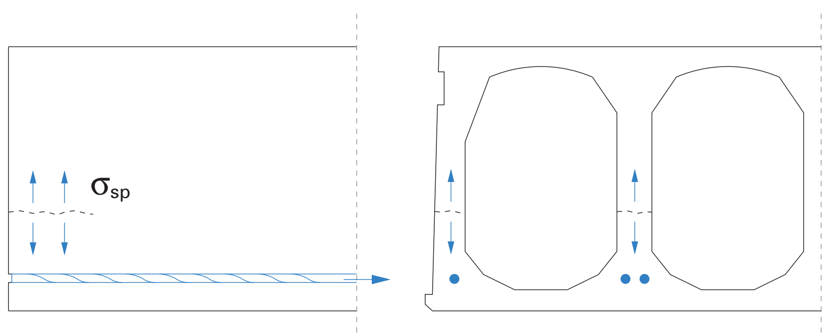


Fig. 3.11 Spalling may cause lesions in the prestressed slab end

Vertical components of the stress are tensile forces quite accentuated near the slab headpieces and are rapidly damped in the adjoining cross sections.

In presence of excess tendons, tensile stresses may be greater than tensile strength of the concrete at the time of sectioning the single slabs. This in turn may lead to horizontal cracks which are commonly known by manufacturers as "wolf's mouths" or "crocodile mouths".

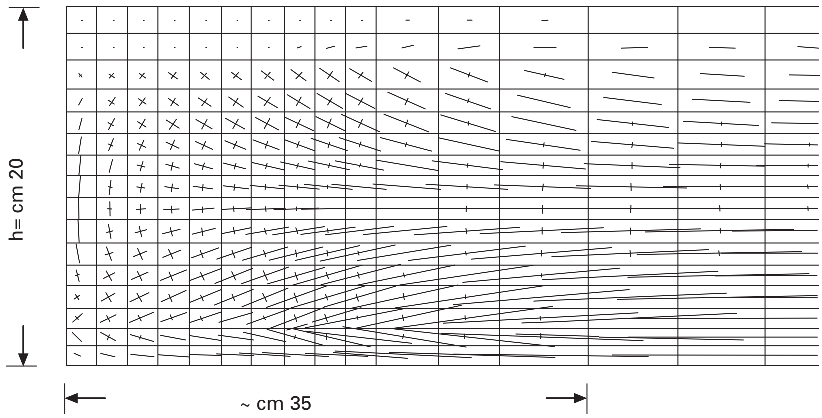


Fig. 3.12 *Principal stresses in the end of a prestressed hollow core slab having depth 20 cm with 3 ϕ 3 mm strands.*
Finite element modelling with prestressing introduced on the basis of the experimental adherence curve of 3 ϕ 3 mm strand.
(Prof. Franco Levi and Renzo Perazzone, April 1983)
The vertical or sloped segments in boldface indicate spalling stresses on a proportional scale.
The horizontal or thin sloped segments indicate prestressing stresses on a proportional scale

It has been observed that when fissures begin to appear in the webs, they quickly extend to great length.

This has been verified and confirmed in calculations with a finite element model of the opening of a horizontal fissure gradually advancing and by calculating stresses at different points of the structure (see Fig. 3.13).

When a hollow core slab exhibits a horizontal crack in one web only, it may still be accepted following an evaluation of possible consequences. When a spalling crack appears in two or three webs the slab has to be rejected (FIP Quality Assurance document, Table 3 point 4).

Horizontal cracks at extremities may progress during transportation and lifting and this reduces shear strength.

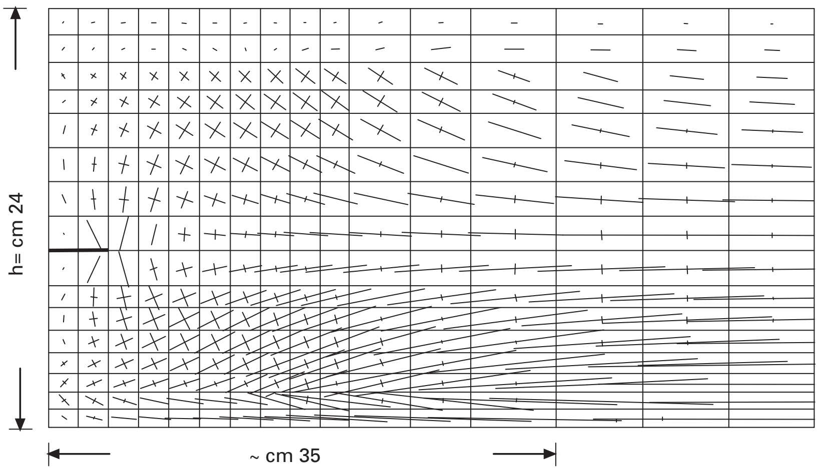


Fig. 3.13 Principal stresses at the end of a prestressed hollow core slab having depth 24 cm and tendons 3 ϕ 3 mm.
Finite element model of the triggering of a crack in the first cross section of the slab end.
(Prof. F. Levi and R. Perazzone, April 1983

To reduce spalling tensile stresses in webs reinforced with two or more strands, a strand is normally sheathed, thus neutralizing it, for a length of 50 ÷ 70 cm from the ends.

3.5.2. Control of spalling tensile stress in the webs

The European Standard EN 1168, Art. 4.3.1.6, requires calculation of the spalling stress in the webs most prestressed.
This is even more important in the presence of suspended floors or those with a clear span between beams, also when they have same depth as the floor. Indeed, suspension stress is added to that of spalling in the webs (see paragraph 4.4.4 below). Spalling cracks must obviously be avoided.

The same article of Standard EN 1168 gives the rule to apply, which is also given here, to verify this requisite.

Spalling shall be checked for the most reinforced web.

If the web is reinforced with strands of different diameters, the resulting stress to be considered is the sum of the single spalling stresses.
In the calculation of spalling the reinforcement near the upper side is disregarded and only the lower reinforcement of a single web is taken into account (see Fig. 3.14).

Spalling σ_{sp} must satisfy the condition:

$\sigma_{sp} \leq f_{ctk0.05}$ (Standard EN 1168 Art. 4.3.1.6)

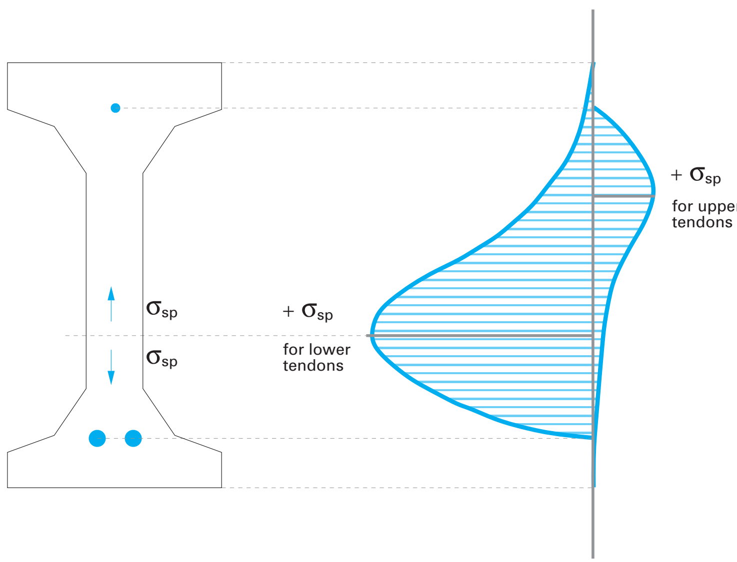


Fig. 3.14 *Representation of spalling intensity σ_{sp} generated by the lower tendons and the upper tendon in a web of the hollow core slab.*

in which:

$f_{ctk0.05}$ = lower characteristic value of concrete tensile strength at the time that the tendons' prestress is released (see Table 5.1 in Chapter 5, below).

$f_{ctk0.05} = 0.7\ f_{ctm} = 0.7 \times 0.3\ f_{ck}^{2/3}$ [N/mm²]

f_{ck} = Characteristic compressive cylinder concrete strength [N/mm²]

$$\sigma_{sp} = \frac{P_0}{b_i e_0} \cdot \frac{15\,\alpha_e^{2,3} + 0.07}{1 + \left(\frac{l_{bp}}{e_0}\right)^{1,5} (1.3\alpha_e + 0.1)} \qquad \textbf{[SP]}$$

where:

b_i = minimum web width;

P_o = $\sigma_{po} A_p$ = force transmitted by the steel in the considered web;

σ_{po} = stress in the steel at the time of verification;

A_p = area of the prestressing steel;

e_o = eccentricity of the prestressing steel;

α_e = $(e_o - k)/h$ = eccentricity ratio;

k = W/A = inertial core radius;

h = height of the web;

l_{bp} = mean value of the length of transmission = $\beta_b\ \phi$
(see EC2 - 4.2.3.5.6);

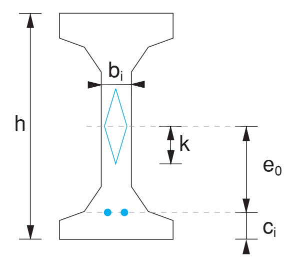
β_b = transmission coefficient;

ϕ = nominal diameter of the strand or wire.

Examples of calculation

Example 3.1
Let us consider, at the time of prestressing, a web of a slip-formed hollow core slab having depth h = 300 mm, as in the section shown below.
Reinforcement consists of two 0.5” strands the axis of which is placed at 30 mm from the intrados.
Calculation is made for one 0.5” strand only; then the value is doubled to obtain total σ_{sp}.

We have:



h	=	300 mm
c_i	=	30 mm
b_i	=	42.5 mm
C 30/37	=	class of concrete on prestressing release
σ_{poj}	=	1250 N/mm^2 = experimental tensile value in the strands at the time of releasing
A_p	=	93 mm^2 (one strand 0.5")
P_o	=	$\sigma_{po}\, A_p = 116{,}250$ N
e_o	=	150 - 30 = 120 mm
k	=	W/A = 72.3 (from the geometry of the cross section)
α_e	=	$(e_o - k) / h = 0.159$
ϕ	=	12.5 mm
l_{bp}	=	70 ϕ = 875 mm (see EC2 Table 4.7.)
$f_{ctk0.05}$	=	2.03 N/mm^2 (see Table 5.1 in Chapter 5, below)

On applying the formula **[SP]** the result is:

$$\sigma_{sp} = 0.935 \text{ N/mm}^2$$

Considering that there are two strands:

$$2\,\sigma_{sp} = 1.87 \text{ N/mm}^2 < f_{ctk0.05} = 2.03 \text{ N/mm}^2$$

so the value is acceptable.

Example 3.2

If in the same web the two 0.5" strands are placed at 35 mm from the lower edge, recalculation of σ_{sp} gives:

σ_{sp} = 0.815 N/mm^2

2 σ_{sp} = 1.63 N/mm^2 < $f_{ctk0.05}$ = 2.03 N/mm^2

which is even more favourable.

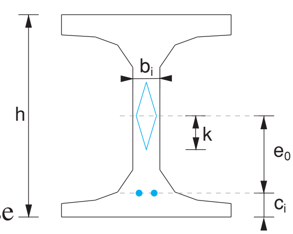
Example 3.3

Let us consider the web of an extruded slab of depth h = 400 mm, as in the section shown below.

Reinforcement consists of two 0.6" strands the axis of which is placed at 35 mm from the lower edge.

Again calculations are performed for only one of the strands and the result is doubled at the end.

We have.



h = 400 mm

c_i = 35 mm

b_i = 53 mm

C 30/37 = class of concrete on prestressing release

σ_{po} = 1250 N/mm^2 = experimental tensile value at the time of releasing

A_p = 139 mm^2 (one 0.6" strand)

P_o = $\sigma_{po} A_p$ = 17,375 N

e_o = 174 mm

k = W/A = 108 mm

α_e = $(e_o - k)/h = 0.1625$

ϕ = 15.2 mm

l_{bp} = 70 ϕ = 1064 mm. (see EC2 Table 4.7)

$f_{ctk0.05}$ = 2.03 N/mm^2. (Table 5.1 at the Cap. 5° par. 5.4.1.)

On applying the formula **[SP]** the result is:

$\sigma_{sp} = 0.99 \text{ N/mm}^2$

Considering that there are two strands:

$2\,\sigma_{sp} = 1.98 \text{ N/mm}^2 < f_{ctk0.05} = 2.03 \text{ N/mm}^2$

the value is at the limit of acceptability.

Example 3.4

If in the same web the two 0.6" strands are placed at 40 mm from the lower edge, recalculation of σ_{sp} gives:

$\sigma_{sp} = 0.89 \text{ N/mm}^2$

$2\,\sigma_{sp} = 1.79 \text{ N/mm}^2 < f_{ctk0.05} = 2.03 \text{ N/mm}^2$

the value is certainly acceptable.

Concluding observations

a) The analysis with finite elements gives the trend of tensile stress σ_{sp} in the longitudinal sense, which can be illustrated as in Fig. 3.15.

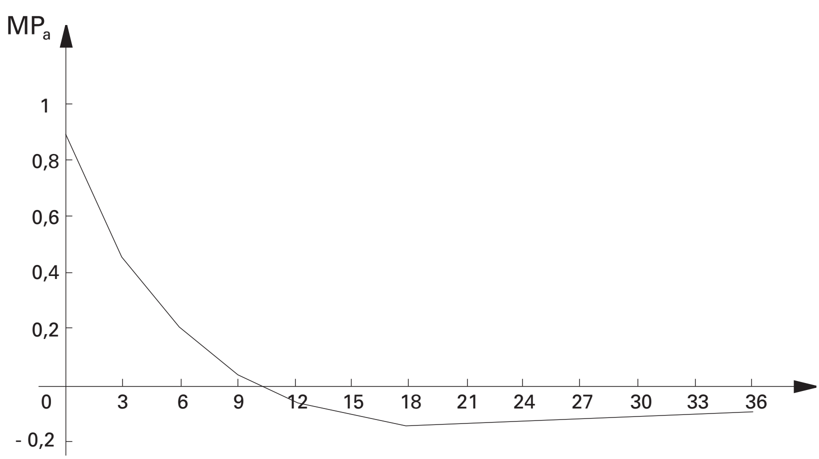


Fig.3.15 Trend of spalling tensile stress in the longitudinal sense at half-height of the web (FIP Recommendations Precast Prestressed Hollow Core Floors 2.2.1)

The maximum stress σ_{sp} is in correspondence to abscissa zero (end of the slab). After a few centimetres σ_{sp} is annulled and afterwards one notes the presence of an inversion of sign.

b) By decreasing eccentricity e_o of the tendon, the value of σ_{sp} decreases notably; thus the greater the distance of strands from the lower edge, the more contained are spalling stresses σ_{sp}. However, the floor's loadbearing capacity is reduced because the amount of prestressing of the lower edge is reduced.

c) Every increase in thickness b_i of the web reduces spalling stress in the web itself. It is important to keep this principle in mind while designing a section of hollow core slab.

d) With the same total area of tendons in a web, stress σ_{sp} is minimum when strands of larger diameter are used. It is thus convenient for reducing σ_{sp} (and at the same time for saving labour) to use only one 0.6" strand instead of obtaining the same steel area with a 1/2" strand plus a 3/8" strand.
In fact σ_{sp} decreases with the increase in strand anchoring length l_{bp} the value of which is 1060 mm for strand ϕ 0.6" while it amounts to only 763 mm on considering the mean value of the transmission length of the 3/8" and 1/2" strands.

e) Should it be necessary to further reduce stress σ_{sp} due to the presence of more than one strand in each web, the only solution is to sheath a strand to neutralize its adherence near the slab end, keeping in mind the precautions in paragraph 3.5.3 below.

f) By applying the previous numerical examples, every producer of hollow core slabs can calculate beforehand the values of stress σ_{sp} generated by the different kinds of strands (see Fig. 3.16) for each depth of slab produced.

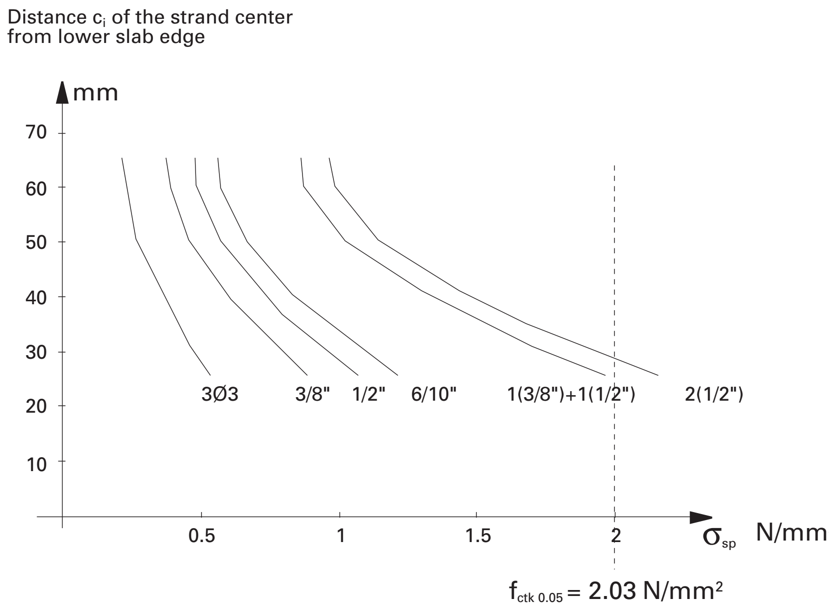


Fig. 3.16 *Values σ_{sp} for each cross section of hollow core slab can be supplied by each producer already in table form for the different types of strand.*

With more than one strand in each web, the relative σ_{sp} are summed to obtain the total spalling stress acting in each web.

The diagram in Fig. 3.16 was constructed on the basis of the following hypothesis:

h = 300 mm = Web of the slip-formed hollow core slab in example 3.1.

C 30/37 = Characteristic strength of concrete at prestressing.

f_{ck} = 30 N/mm^2

$f_{ctk0.05}$ = 2.03 N/mm^2 (Table 5.1 in Chapter 5, par. 5.4.1.)

$l_{bp} = 70\ \phi$	=	mean transmission length (EC2 table 4.7)	
		plait $3\ \phi\ 3$	$l_{bp} = 455$ mm
		strand 3/8"	$l_{bp} = 650$ mm
		strand 1/2"	$l_{bp} = 875$ mm
		strand 6/10"	$l_{bp} = 1060$ mm

The curves shown in Fig. 3.16 highlight the decrease in the value of σ_{sp} with the increase in the distance of strands from the lower edge.

g) Care is needed to avoid overestimating the class of concrete of the vertical webs on release of prestressing. Class C 30/37 may not always be the real strength class of the webs. This may present a defect in compacting and thus be of a lower class.

h) In the calculation of σ_{sp} with the formula **[SP]** one must consider only the action of the lower strands of the web and disregard the upper strand in the same web for the reason illustrated in Fig. 3.14.

i) The calculation of σ_{sp} in hollow core slabs once in use must take into account the time elapsed from the application of prestressing and then the completion of concrete shrinkage and creep and the relaxation of prestressing steel.
The final spalling stresses are reduced compared to the initial $\sigma_{sp\,(i)}$ according to the relation:

$$\sigma_{sp\,(\infty)} = \sigma_{sp\,(i)} \frac{\sigma_{p\,\infty}}{\sigma_{p\,0}}$$

in which

$\sigma_{p\,0}$ = steel stress immediately after the release $\cong 1250$ N/mm^2

$\sigma_{p\,\infty}$ = final steel stress $\cong 1100$ N/mm^2

thus we have the spalling decrease

$$\sigma_{sp\,(\infty)} \cong \sigma_{sp\,(i)} / 1.1$$

When the floor is in use, it must also be taken into account that stresses $\sigma_{sp\,(\infty)}$, besides being reduced, act in concrete with a final characteristic strength $f_{ctk0.05\,(\infty)}$ that is much greater than the initial strength $f_{ctk0.05\,(i)}$ at the time of applying prestressing.

Indeed, (see Table 5.1 in Chapter 5, par 5.4.1):

in concrete final class	C 45/55	this is:	$f_{ctk0.05\,(\infty)}$	=	2.85 N/mm^2
at time of release the concrete class was	C 30/37	and:	$f_{ctk0.05\,(i)}$	=	2.03 N/mm^2

The ratio is at least:

$$f_{ctk0.05\,(\infty)} / f_{ctk0.05\,(i)} = 1.4$$

and the final spalling safety factor of the hollow core floor once in use is:

$$\gamma_{sp} \cong 1.1 \times 1.4 > 1.5$$

Safety is always assured.

3.5.3. Reduction of prestressing by means of sheaths

In consideration of what has been stated about spalling tensile stress in the two preceding paragraphs, to reduce it to within acceptable limits recourse is often had to the neutralization of some strands for a length of some tens of centimetres in correspondence to the slab ends.
Normally the length of sheaths is not more than 50 ÷ 70 cm and their extremity remains visible in the end section of each slab.

It is necessary to make sure that the sheathed strand in the slab end begins to be 100% efficient only after about 70 ϕ from the end of the sheath, which is to say starting from a cross section in which bending moments are already abundantly present.

It is necessary to perform specific calculations to make sure that wherever the sheathed strand is not yet fully efficient, the prestressing induced by unsheathed strands alone must be sufficient to oppose tractions generated by the bending moments present in that length of the slab.

In current manufacturing practice there is another good reason for reducing excess prestressing by means of total neutralization of one or two strands in a certain number of slabs.
This is due to the fact that the fixed length of prestressing and casting beds often imposes the production of slabs which require little reinforcement in completing a bed with more reinforcement.

Excessive prestressing of these slabs almost always accentuates camber and this is visible only at the time of their being assembled. Antiaesthetic defects in camber at the intrados between slabs with too much reinforcement and others properly reinforced will appear.
In such cases one obtains better results by sheathing, and thus totally neutralizing, the entire length of excess strands in the portion of the bed occuped by slabs requiring less reinforcement.

3.5.4. Slippage of strands into slab ends

Anchoring of the strands to the concrete is to a great extent dependent on the bonding capacity of the different concretes which vary from one manufacturer to another, even though they possess the same characteristic strength.

It has also been observed that even when the aggregate is kept unchanged, cements of different origins influence the degree of bonding of strands.

In any case, the proper compacting of concrete around strands assures their good anchorage. But this cannot prevent visible slippage of strands into slab ends once the strands are cut.
This slippage must remain within acceptable limits as illustrated in Table 2.3 (paragraph 2.4.1, point 13).

Immediately after sawing the slabs it is necessary to inspect visually the slippage of the strands at both ends of all slabs.
The effective slippage of a strand can be measured as the mean value of slippage of the two ends of that strand at opposite ends of the slab (FIP QUALITY ASSURANCE document, par. 3.5.3.).

If slippage values found are above acceptable limits, that slab or batch of slabs must be discarded, or at least declassed.
In such a case the loadbearing capacity of that slab must be reduced by considering strands with excessive slippage as not being in the slab.

The European Standard EN 1168, at point 4.2.3.2, illustrates how to calculate maximum allowable slippage Δl_o of strands which is to be considered the mean value of the three strands that have shrunk the most in the same slab end.

$$\Delta l_o = 0.4\, l_{bpd}\, \sigma_{pmo} / E_p \qquad \textbf{[a]}$$

where:

l_{bpd} = upper limit of transmission length (see EC2 ENV 1992-1-1, Art. 4.2.3.5.6, Table 4.7 and point 4)

σ_{pmo} = stress in prestressing steel immediately after release

E_p = modulus of elasticity of prestressing steel

The maximum allowable slippage of a single strand shall not exceed the value of 1.3 Δl_o.

Values given in Table 2.3 (par. 2.4.1 above) were calculated by applying the formulation shown above, as specified in the following Example of calculation 3.5.

The following ways of measuring strand slippage are expressed by Standard EN 1168 at point 5.2:

– the mean value of slippage for a given slab shall be calculated on the basis of measurements of the three most slipped strands;

- each single value shall be measured with the accuracy of 0.5 mm and then compared with 1.3 Δl_o. The mean value shall be compared with the acceptable Δl_o.

Such meticulousness in prescriptions indicates the importance of the value of wire slippage as concerns Controlled Quality of prestressed hollow core slabs.

Example of calculation

Example 3.5

Article 5.2 of the European Standard EN 1168 prescribes calculation of mean prestressing wire slippage according to the expression **[a]** given above.

For EC2 ENV 1992-1-1, Art. 4.2.3.5.6. points 3) and 4) we have:

l_{bpd} = 1.2 l_{bp} and $l_{bp} = \beta_b \, \phi$

ϕ = nominal diameter of prestressing steel

It is assumed that at the time of applying prestressing, the real compressive strength of the slab's concrete is $f_{ckj} = 30$ N/mm^2.
Schedule 4.7 of Article 4.2.3.5.6. gives the corresponding value of the coefficient

β_b = 70

thus we have

$l_{bpd} = 1.2 \; \beta_b \; \phi = 84 \, \phi$

σ_{pmo} = prestressing steel stress immediately after release to be measured experimentally. In calculating it is assumed to be 1250 N/mm^2

E_p = modulus of elasticity of prestressing steel which for this calculation is assumed to be 196000 N/mm^2

Thus for the 3/8" strand whose nominal ϕ is 9.3 mm we have

$$\Delta l_o = 0.4 \times 84\,\phi \frac{1{,}250}{196{,}000} = 2.0 \text{ mm.}$$

For the most common prestressing strands we repropose the values of Δl_o indicated in Table 2.3 par. 2.4.1 above.

3.6. Rules and devices for the support of hollow core slabs

Generally speaking, it is necessary to distinguish between temporary support during assembly to be completed and consolidated by in situ casting, and simple support which will be permanent even when the building is completed.

A further distinction concerns the support surface, which may be more or less irregular and may thus offer discontinuous support points.

On considering floors composed of precast elements, Italian Prefab Regulations prescribe that permanent simple support after assembly shall be at least 5 cm, while a temporary support during assembly and prior to the in situ cast to make it permanent may be a minimum of 3 cm.

Eurocode EC2 ENV 1992, Part 1.3, Art. 4.5.5.2 and also pr EN 1992-1, Section 10 which deals specifically with prefabricated floors, are much more detailed.

In the case of permanent simple support, it prescribes different lengths depending on the nature of the support surface, the effective bearing width, support pressure and also takes into account building tolerances and length of the precast element.

For the FIP Guide “Quality Assurance”, par. 6.1.2., in the case of final simple support, the hollow core slab may be placed directly on the support surface only when this is represented by a steel beam or another perfectly clean and smooth equivalent surface.

If the slab is to be supported with simple final support on a concrete surface that is not perfectly smooth and coplanar, it is indispensable to interpose a 60 Shore neoprene tape with a minimum width of 30 mm for the Italian Standard and 40 mm for ENV 1992, Part 1-3, and having a minimum thickness of 5 mm.

If the support surface is not regular, lodging with mortar is indispensable.
No special rules are laid down for the support of hollow core slabs, even with direct support on not perfectly regular surfaces, when the project calls for in situ reinforced casts that englobe the ends of the slabs.

In such a case, in fact, all loads applied later on with the positioning of the slab do not increase pressure on the irregularities because it is lying on the casts, which make it one with the support.

It is good practice to keep in mind during the in situ positioning of the hollow core slabs that if there are support lengths of less than:

3 cm	for slabs up to 5 m in length
4 cm	for slabs up to 8 m in length
5 cm	for slabs up to 12 m in length

it is necessary to temporarily prop the slabs at the extremities until the completing casts have hardened sufficiently.

3.6.1. Minimum design support length

The minimum nominal length for simple final support of floors is prescribed in Part 1.3, "Prefabricated Concrete Elements and Structures", of EC2 ENV 1992, Art. 4.5.5.2.
Similar prescription is also reported in pr EN 1992-1, Section 10.

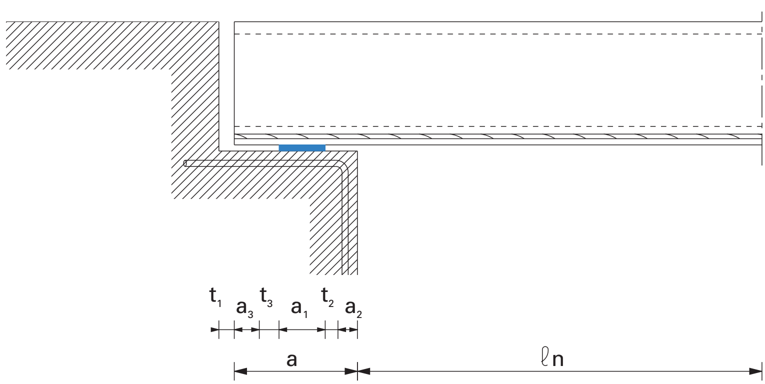


Fig. 3.17 Nominal length "a" of support

$$\mathbf{a} = \mathbf{a}_1 + (\mathbf{a}_2^2+\mathbf{a}_3^2+\mathbf{t}_2^2+\mathbf{t}_3^2)^{1/2}$$

where

$$a_1 = \frac{V_{max}}{b_n \sigma_{Rd}}$$

width of rubber or neoprene tape;
it must never be $a_1 < 40$ mm.

V_{max} = calculated value of support reaction

b_n = bearing width of support; it must never be more than 600 mm.

σ_{Rd} = calculated value of concrete compressive strenght

$\sigma_{Rd} \leq 0.6\ f_{cd}$ for dry bearing support (concrete on concrete);

$\sigma_{Rd} \leq 0.7\ f_{cd}$ for support on rubber or neoprene tape;

$\sigma_{Rd} \leq 0.8\ f_{cd}$ for lodging on mortar or for concrete on steel;

f_{cd} = design value of concrete compressive strength. See next Chapter 5, schedule 5.1, (it is the lesser of the strengths of the slab or the bearing structure);

a_2 = length of possible yield of the corner in the bearing structure with support pressure $\sigma_{sd} > 0.4\ f_{cd}$

a_2 = 0 in the case of steel structures;

a_2 = 25 mm in the case of masonry or unreinforced concrete;

a_2 = nominal cover of reinforcement bar if this is $\phi \leq 12$ mm;

a_2 = nominal cover of the bar (if the bar is $\phi > 12$ mm) + bar diameter + inner radius of bend;

a_3 = length of possible yield of the end of the slab with bearing pressure $\sigma_{sd} > 0.4\ f_{cd}$

a_3 = 0 in the case of straight strands or bars exposed at the ends of the slab;

a_3 = greater value between cover of reinforcement at the ends or 10 mm (in the case of end reinforcement $\phi \leq 12$ mm);

a_3 = 15 mm (in the case of end reinforcement $\phi > 12$ mm);

t_2 = 15 mm for steel or precast concrete supports;

t_2 = 20 mm for masonry or in situ cast concrete supports;

t_3 = $l_n / 2500$ where l_n is the clear span between supports;

t_1 = maximum manufacturing deviation of the length of slab + maximum construction tolerance.

Example of calculation

Example 3.6

The lower floor-bearing support edge of a prestressed concrete "T" beam is 150 mm wide.

The beam's concrete strength class is C 35/45. The stirruping of the floor-bearing edge is composed of 10 mm ϕ bar with a nominal cover of 15 mm.

This must support, by means of 60 Shore rubber tape, a floor made up of hollow core slabs 1200 mm wide whose concrete strength class is C 45/55.

The clear distance between faces of the supports is $l_n = 11.10$ m and the calculation span (distance between centres of supports) is $l_c = 11.30$ m.

Each slab end discharges onto the support the load

$V_{max} = 130$ kN and $V_{min} = 90$ kN

The question is:

Is the residual nominal space t_1 between the end of the slab and the web of the beam sufficient to remain within normal assembling tolerances?

Calculation of the minimum support length:

$$\mathbf{a}_{min} = \mathbf{a}_1 + (\mathbf{a}_2^2 + \mathbf{a}_3^2 + \mathbf{t}_2^2 + \mathbf{t}_3^2)^{1/2}$$

where

$$a_1 = \frac{V_{max}}{b_n \sigma_{Rd}}$$

$V_{max} = 130$ kN

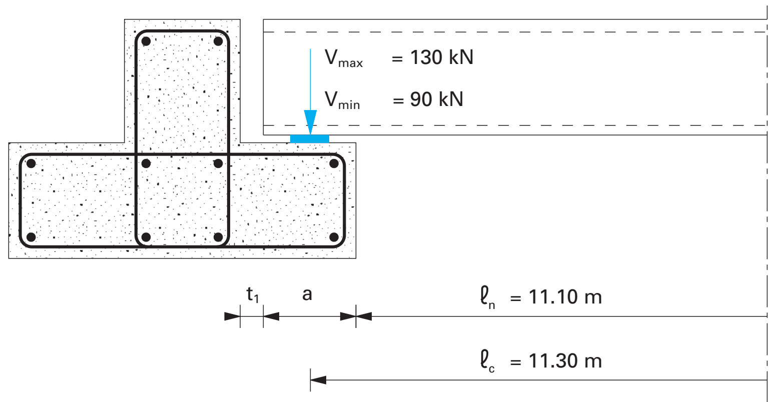


Fig. 3.18 *Nominal length "a" of support of a hollow core floor on a precast inverted "T" beam*

b_n = 600 mm

σ_{Rd} = $0.7\ f_{cd} = 16.31\ N/mm^2$

f_{cd} = is calculated for the concrete of the beam. For Table 5.1 in Chapter 5 it is f_{cd} =23.3 N/mm^2

$$a_1 = \frac{130{,}000}{600 \times 16.31} = 13.28\ mm.$$

Since the calculated value is less than the width of the rubber tape, this value is assumed for a_1 (which by definition must always be ≥ 40 mm)

a_1 = 40 mm;

a_2 = 15 mm external nominal cover of 10 mm ϕ bar;

a_3 = 0 mm since the hollow core slab is reinforced with strands emerging at the ends;

t_2 = 15 mm since the supporting structure is of precast concrete;

t_3 = $l_n/2{,}500$ = 4.44 mm (where l_n = 11,100 mm);

the minimum nominal length of the support is

$$a_{min} = 40 + (15^2+15^2+4.44^2)^{1/2} = 61.6 \text{ mm}$$

It follows that it is quite possible to consider in the design the support length a = 100 mm still having the available space t_1 = 50 mm to absorb the sum of tolerances for in situ building assembly and length of the slab.

3.6.2. Additional reinforcement in the transmission zone for flexural and shear capacity

Article 3.2.9 of the Italian Standard considers not prestressed the final lengths of prestressed structures with bonded tendons for a length equal to 70 times the largest nominal diameter of the prestressing tendon.
This anchorage zone must therefore be verified with respect to shear with the rules for normal reinforced concrete.

Article 7.0.a of the Italian Standard requires at the ends of prestressed floors with bonded wires the placing of sufficiently diffuse normal reinforcement at the bottom side to assure absorption of tensile stress equal to shear value to safeguard tractions in the concrete induced by the bending moment present near the support.

This requirement, when applied to hollow core floors, can be expressed as follows:
In correspondence to supports, however they may be made, suitably anchored and conveniently distributed reinforcing bars, such as to assure the absorption of tensile stress equal to shear value, shall be placed in the structure at the lowest level possible.

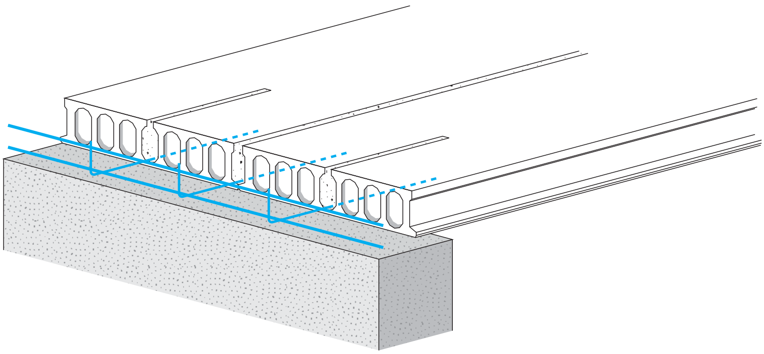


Fig. 3.19 The reinforcement at the supports of the hollow core floor as for Art. 7.0.a of Italian Standard

The length of reinforcing bars on the floor side shall be calculated on the basis of good bond conditions of the cast in situ concrete (Article 3.1.4 and 5.3.3 of Italian Standard).

As concerns the anchorage of concrete filling the open cores in the precast slab, reference is made to the preceding paragraph 3.4.1.

3.6.3. Prestressing in the transmission zone for flexural and shear capacity

Article 4.2.3.5.6 of EC2 ENV 1992-1-1 takes into account the linear increase in the resistant force supplied by prestressing tendons in the anchorage zone to oppose tractions in the concrete generated by shear and bending moments.

This resisting force may be null for a few millimetres starting from the end of the slab up to the beginning of the real bonding of prestressing tendons; it then increases linearly up to full prestressing.

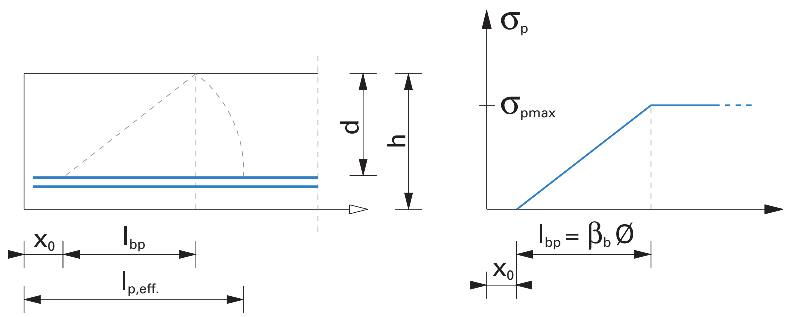


Fig. 3.20 The transfer of prestressing in hollow core slabs

Transmission length l_{bpd} depends on the kind and diameter of tendon and on the real effective strength of the concrete in the element at the time of prestressing.

The neutral length x_o indicated in Fig. 3.20 can be evaluated from time to time and depends on any sudden release of prestressing steel, on slippage of strands at the limit of acceptability and on the maximum diameter of the aggregate used.

It can also be deliberately caused by the neutralization of bonding of the ends of strands with sheaths.

In practice, if a structure is designed for construction outside Italian jurisdiction (for which it is possible not to comply with Art. 7.0.a of the Italian Standard) and the inclusion of terminal reinforcement in the slab ends, as seen in paragraph 3.6.2, is not desired, it is necessary to make sure that in the meaningful cross sections of the slab near supports the conditions expressed in the following points are verified:

a) shear-flexural capacity as expressed in the point 4.3.2.3 of ENV 1992-1-1 or 6.2.2 of the updated pr EN 1992-1

b) shear-traction capacity as expressed in point 4.3.2.3. of ENV 1992-1-3 (Part 1.3) or 6.2.2 of pr EN 1992-1

c) resistance against loss of the anchorage as indicated in Article 4.2.3.5.6. of ENV 1992-1-3 (Part 1.3)

For details on these verifications see next ASSAP publication specific for calculation.

3.7. Increase in shear capacity with concrete filled cores

Additional concrete casting in situ to complete hollow core floors generally consists of filling the longitudinal joins and the open cores at slab ends in which the connecting bars are embedded or in the laying of a corroborating concrete topping where considered necessary.

These castings are made of concrete which increases the cross section of the hollow core slab and therefore considerably increases the shear capacity of the end sections of the floor once in use.

In situ concrete is normally classified as C20/25 ÷ C30/35, whereas the prefabricated slab has a minimum classification of C45/55. This must be taken into consideration when homogenizing the slab end section in order to calculate shear capacity. For the theory and calculation procedures see next ASSAP publication.

Chapter 4
CONNECTIONS AND STRUCTURAL SCHEMES

4.1. Connections and ties

Hollow core floors can be used together with any kind of bearing structure, whether it is made of cast in situ reinforced concrete, precast beams and pillars or steel structures.

As we shall see in this chapter they can easily be designed, case by case, so as to assure specific metal links to structures surrounding them, even to the point of realizing true structural restraints and even structural ties.
With these premises, there is no obstacle to the forming of fixed ends in hollow core floors or to their use in seismic zones.

The word “connection” is generally employed when referring to reinforcement made of steel bars, which may be different in shape, that create any sort of union between two or more adjoining structural elements.
Ties on the other hand are enchaining reinforcements, made effectively continuous, passing through walls and/or slabs to safeguard integrity of the entire building structure.

Connecting reinforcement between precast slabs and bearing beams is to be considered essential for the structural integrity of the floor.
Just as essential for the structural integrity of the entire building are the continuous tying reinforcements embedded in the edge beams.

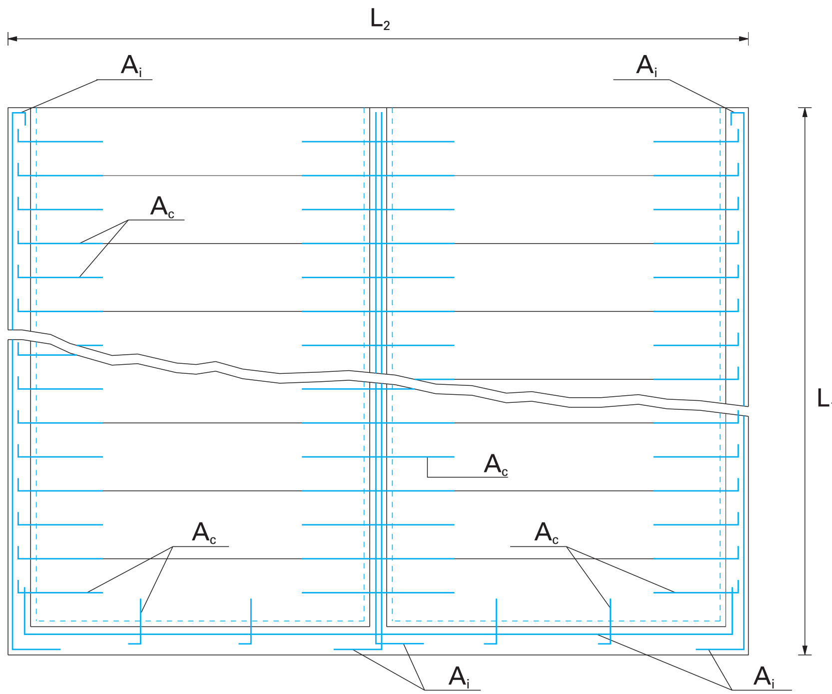


Fig. 4.1 *Connections and ties in a floor with hollow core slabs.*

According to Italian Prefab Regulations, the minimum values of Ai and Ac are (Art. 2.11.2. b/e):

- *minimum total area of the perimeter ties of the floor:*
  in a non-seismic zone $A_i \geq 300\ mm^2$
  in a seismic zone $A_i \geq 400\ mm^2$
- *minimum connecting bar area for longitudinal and lateral connections:*
  it must absorb at least 1% of axial stresses and in any case must be $A_c \geq 33.3\ mm^2/m$

According to EC2 ENV 1992-1-3 Part 1-3 (Art. 5.5.2.a) and also pr EN 1992-1 (Section 9.10):

- *perimetric ties shall be capable of supporting traction* $F_i = L \times 10\ kN/m \leq 70\ kN$
  where L is the total length of span (L_1 *o* L_2)
- *perimetric ties can be arranged within a band of 1.2 m from the edge. This means that they can also be inserted in the longitudinal union between the next-to-last and last hollow core slab at the floor edge.*

4.1.1. Connections in hollow core floors

Because of the typical manufacturing process, hollow core slabs never leave the factory with reinforcing bars protruding from the concrete. But they do offer many natural, easy-to-reach housings in which to place many different kinds of reinforcing bars with end hooks for all kinds of connections.

The most obvious of these housings is the longitudinal union between slab and slab in which the connecting bars can be anchored and, if the minimum dimension of the section allows (see par. 3.3 above), so can the longitudinal tie bars.

These longitudinal unions between slabs are almost always too few and too far one from the other to meet the design requirement of having connections distributed in a sufficiently continuous way.

For this reason other grooves, the "open cores" mentioned in paragraph 2.3.1, are made in the ends of the hollow core slabs in the factory. It is possible to have one, two or even three open cores in every slab end with a width of 120 cm, so as to offer housings that are more numerous and closer together for the connections required by the design.

When the floor is delimited laterally by tie beams or edge beams, as shown in Fig. 4.1, transverse connections for slabs without topping are made by placing hooking bars inside special laterally opened cuts in the body of the slab (see Fig. 4.2).
The lateral cuts have a distance between centres of about 1 or 2 metres and are small in size.
Where a concrete topping is required, this offers a natural housing for a resistance-welded mesh which is suitable for creating the transverse connections at the edges (see Fig. 4.3).

According to EC 2 ENV 1992-1-3 (Art. 5.5.2.a) and also pr EN 1992-1 (Section 9.10) perimetric ties can also be inserted in the longitudinal union between the next-to-last and last hollow core slab as long as they are within 1.2 m from the free edge of the floor.

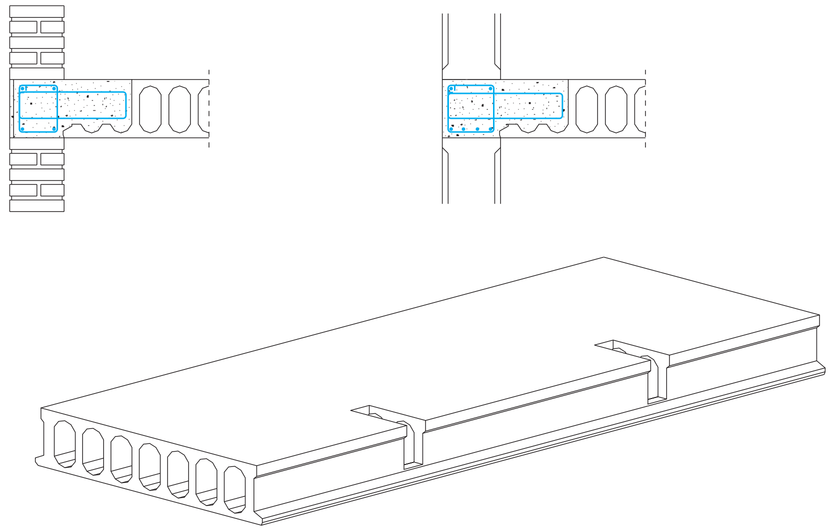


Fig. 4.2 *Lateral cuts in a slab without topping for hooking bar housing*

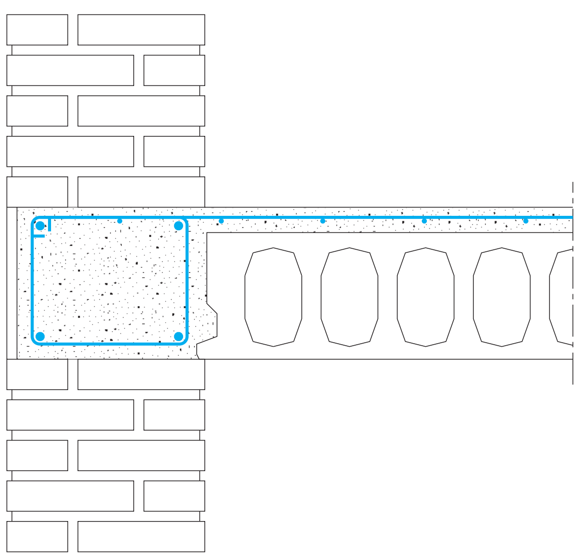


Fig. 4.3 *Lateral connection with a resistance-welded mesh inside the concrete topping.*

Another housing for longitudinal tie bars may be the outermost open core of the floor as shown in Fig. 4.4 and 4.5.
Transversal tying reinforcement A_i will be bent and anchored in the same core.

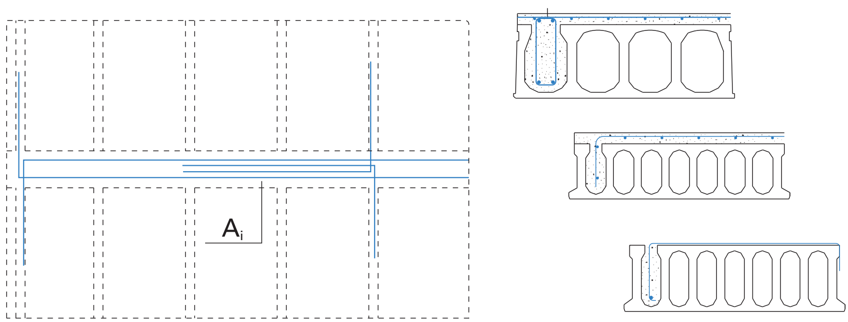


Fig. 4.4 *Edge tie bars incorporated in the hollow core slab or in the final longitudinal join in presence and absence of concrete topping.*

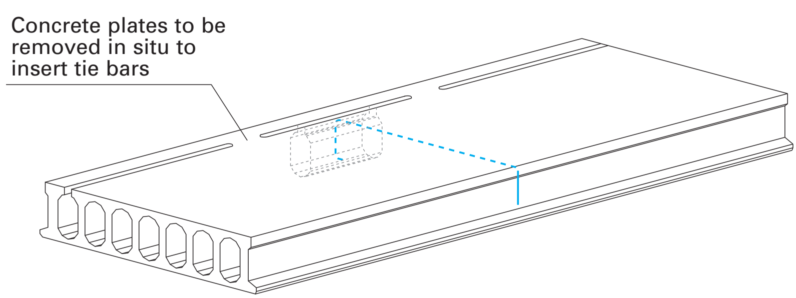


Fig. 4.5 *Slab prepared to house perimetric tie bars.*
The transverse bars anchored in the hollow core slab that restrain the floor to the perimetric tie in seismic zones shall be at least two for a floor span of l > 6.0 m and at least one for spans of 4.0 < l < 6.0 m and shall have area of at least 33.3 mm²/m (Italian Prefab Regulations 2.11.2. e).

In slabs prepared to incorporate the edge connecting bars, the aperture of the core is made in the factory on lengths of not more than 2 m, leaving the plates of the cover to be removed in situ (see Fig. 4.5).

4.1.2. Anchoring capacity of connecting bars in the hollow core slab

The efficiency of each connecting reinforcement between the end of the hollow core slab and any other structure around it lies in the perfect anchoring of the bars into the core and/or longitudinal union.
To be sure of the effectiveness of the connection it is therefore necessary to take into account that:

- the in situ casting that fills the open core or longitudinal union shall be properly anchored to the prefabricated slab;

- the connecting bars shall be solidly anchored in the in situ casting that fills the core and the longitudinal union.

To be sure to guarantee such functions it is necessary to observe the following points:

a) the class of the cast concrete must be at least C 20/25 (but C25/30 is preferred, see paragraphs 3.3 and 3.4 above); the casting should be properly compacted and vibrated;

b) shear stress τ_{sd} between the concrete of the slab (extruded or slipformed) and the concrete cast in situ, under the maximum traction of the reinforcing bars anchored in the same core, for the European Standard ENV 1992-1-3 (Art. 4.5.3.3) as for the updated pr EN 1992-1 (Section 6.2.5), for concrete class C 25/30 shall be at the Ultimate Limit State:
   - in the open cores $\tau_{sd} \leq 0.39$ N/mm^2
   - in the longitudinal unions between slabs $\tau_{sd} \leq 0.10$ N/mm^2

while for the Italian Standard (par. 7.1.6.2.) it can be $\tau_{sd} \leq 0.30$ N/mm^2 in the cores and the unions at the Serviceability Limit State;

c) the surface of the prefabricated element shall be carefully cleaned and the cast in situ concrete shall be made with cement that is not fast-hardening and is limited in shrinkage;

d) the lower part of the open core, where removed concrete is laying, normally shall not be considered effective contact surface.

As concerns anchorage of the bars, it is necessary to verify that:

e) the anchorage length of the connecting bars (preferably with ribbed surface) shall comply with the values established by ENV 1992-1-1 (Art. 5.2.3.4) and pr EN 1992-1 (Section 8.0);

f) the entire length of the bars shall be properly embedded in the compacted concrete;

g) the final part of the open core shall be closed with a plug so that casting is held in place during vibration (this recommendation is especially valid for hollow core slabs with depth $h \geq 250$ mm).

4.2. The execution of structural restraints

Starting from the 1970s, the hollow core floor developed to the benefit of "prefabrication", which prevalently involved dry-assembled structures and were thus simply laid on supports.

Thus, up to a short time ago an overwhelming majority of the hollow core floors produced in the world were applied with simple support for reasons of economy and speed in construction.

From the 1970s on, "prefabrication" of buildings has been gradually transformed into "industrialization of building by components" and greater attention has been focused on connections and integrative in situ casting to restore the traditional monolithic characteristic of buildings. Hollow core slab floors are now in demand to assure better structural performance.
Below is an analysis of the peculiar characteristics of restraints applied to hollow core floors and the specific ways in which they are to be manufactured, apart from dimensioning and calculation of the forces involved, which will be dealt with in the next ASSAP publication.

4.2.1. Simple support

Each span of floor with simple support must be free to deflect under the action of both permanent and occasional loads.
It is also indispensable to assure the necessary links to the bearing structures.

According to the Italian Standard (Articles 5.3.1. and 7.0.a) and, in special cases, also for the European Standard (ENV 1992-1-1, par. 4.2.3.5.6), specific reinforcing bars positioned as close to the bottom as possible and well anchored, capable of supporting the positive moment of the span which develops in the transimission zone, are required.

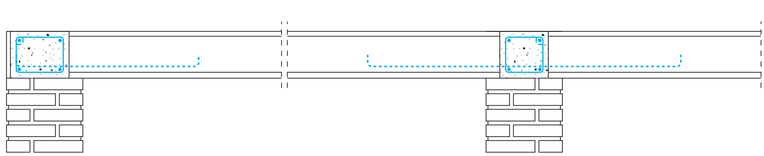


Fig. 4.6 A simply supported hollow core floor

To that end, in cases of simple support, the reinforcement shall be capable of absorbing at the ULS a tensile stress equal to total shear at the support.

A distance between centres for such shear-resistant bars no greater than 60 cm is recommended, with an exception for slabs used as coverage for which a distance between centres of 120 cm is normally used because of the low level of shear stresses.

For the Eurocode in the anchorage zone of strands at slab ends there is prestressing that varies linearly from zero to 100%. Thus it allows the positive moment in the anchorage zone to be supported also by the amount of prestressing developed in that section.

More precisely, ENV 1992-1-1 (point 4.2.3.5.6.9) and pr EN 1992-1 (Section 8.10), prescribe that if the envelope of traction forces acting in the anchorage zone (a combination of tractions due to shear and those due to the bending moment) is greater than $f_{ctk0.05}$, it is necessary to make sure that such envelope is not greater than the traction capacity supplied both by prestressing reinforcement and by reinforcement bars that may have been included in the anchorage zone.

In the case of simply supported floors special attention must be given to avoiding the development of unintended negative moments caused by carelessness in building which may lead to floor cracking (see Standard EN 1168 Informative Annex E.

Eurocode ENV 1992-1-1 (point 5.4.2.1.2.1) and pr EN 1992-1 (Section 9.0), prescribe that in buildings with diffuse in situ casting, even when simple support has been assumed in the design, it is in any case necessary to insert additional reinforcement capable of absorbing a negative moment deriving from a partial fixed end restraint equal to at least 25% of the maximum bending moment of the span.

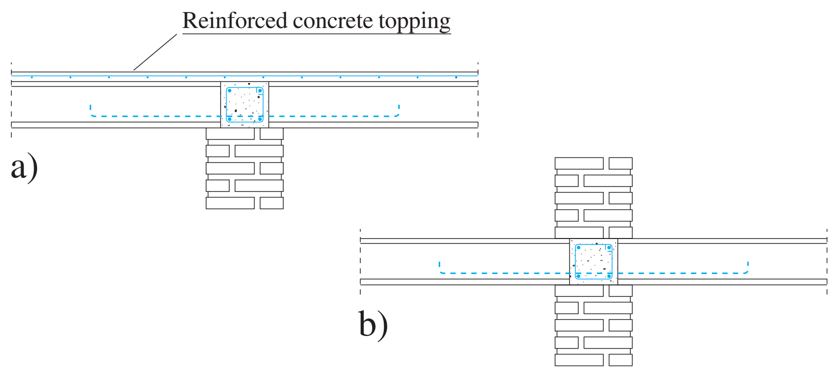


Fig. 4.7 *False simple supports with the development of unexpected negative moments:*
a) the concrete topping reinforced with resistance welded steel mesh creates continuity and opposes the free deflection of the floors;
b) the weight of the wall above blocks the free rotation of floor ends.

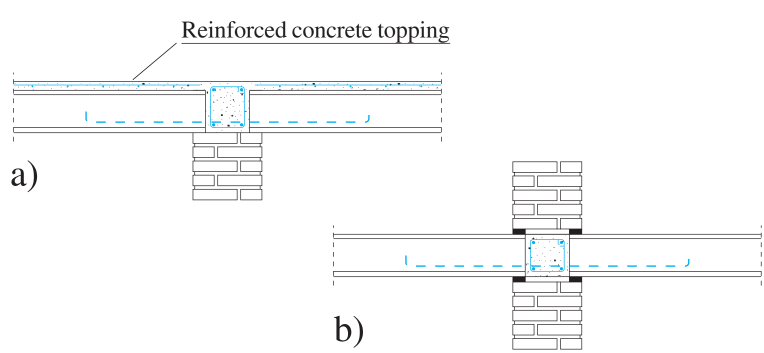
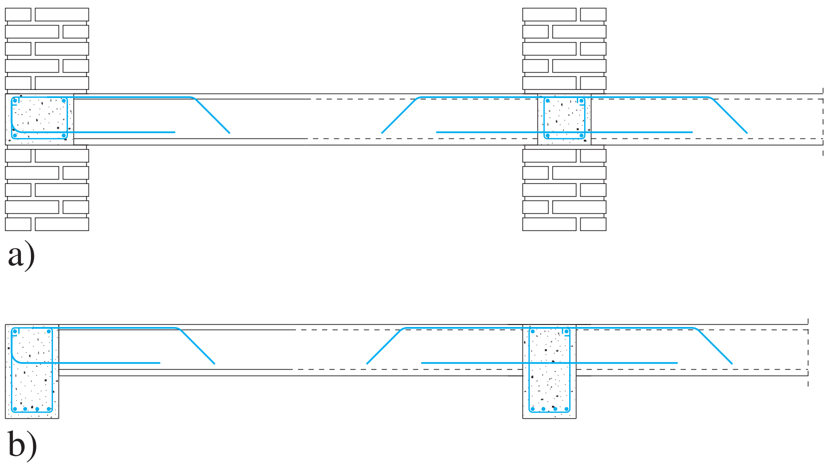


Fig. 4.8 *Correct building steps that give the floor the freedom of simple support:*
a) interruption of the reinforcement of the concrete topping above a middle support;
b) interposition of strips of rubber allows free rotation of floor ends.

4.2.2. Continuity in a multispan floor

In Italy today, this kind of structural restraint is used in most applications of hollow core floors with cast in situ beams and is also applied with precast beams when they have to be completed by in situ casting.



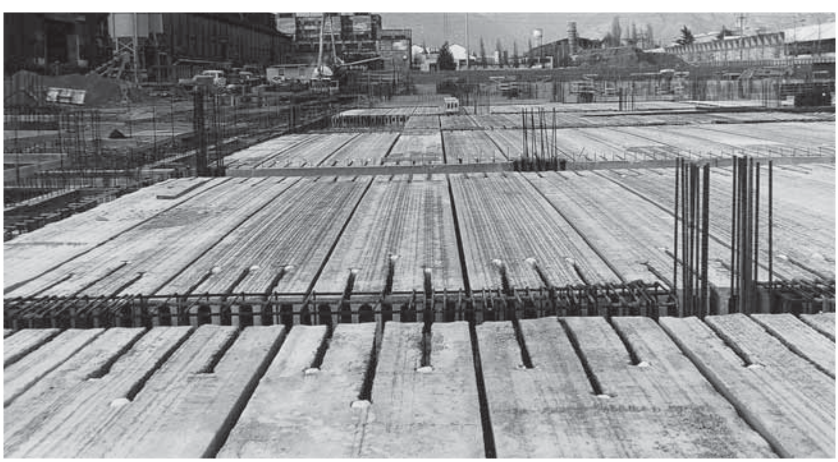


Fig. 4.9 *Floor in continuity on several supports:*
a) with total block of the connection (fixed end restraint);
b) with possible rotation of connections (partially fixed end).

In fact, in cases of this kind, retaining walls, staircases and pillars are always cast in situ. In garages the design often calls for the finishing of floors with smooth concrete and it represents no problem for the builder to add steel reinforcement to make the hollow core floors continuous.

Continuity becomes "indispensable" when the hollow core floor is inserted as a clear span in a structure that is entirely cast in situ, whether it is with pillars and beams or loadbearing walls.
In these cases hollow core slab floors are connected as a clear span to the loadbearing structure without direct support. Thus the simple support restraint is structurally inadmissible (see Fig. 4.9.b).

The continuity restraint is "indispensable" also when floor finishing does not allow visible cracks in the vicinity of the support and when deflection under long-lasting and occasional loads must be kept to a minimum.

Continuity is "recommended" when the compressed flange of the beam must structurally involve the portion of the hollow core slab floor adjacent to it.
This portion of the floor acts as an integral part of the beam itself.
This allows the obtaining of a thinner bearing beam (see Fig. 4.16).

Finally, the continuity restraint is an "undesirable consequence" that the designer must keep in mind in the case of special building conditions of the kind illustrated in Fig. 4.7: concrete topping reinforced with resistance welded steel mesh or loadbearing walls that grip the ends of the hollow core floor and impede its free rotation.

A simply supported hollow core slab is in a stress state near the support that must receive special attention. This is due to the contemporary presence of spalling stresses and stresses dependent on the diffusion of prestressing (splitting), as well as shear stresses.

Opposed to spalling and splitting there is the positive action exerted by the final compressed strut inside each web.

In the case of slabs for which the continuity restraint is made in situ, the internal stress situation is substantially improved in all senses by the presence of compression stresses at the lower flange. They are very important in many cases (see Fig. 4.10).

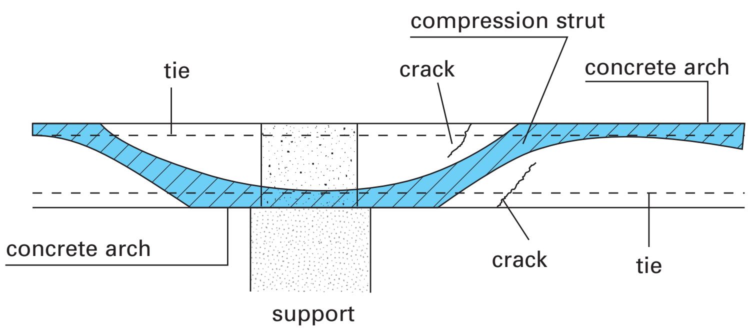


Fig. 4.10 The arch-tie mechanisms created in correspondence to supports of hollow core floor made continuous (Prof. Franco Levi - Politecnico of Turin 1983).

In effect, the possibility of the formation of cracks in the zone subject to negative moment in elements made continuous absolutely can in no case impede the setting in of two arch mechanisms in concrete with steel ties whose concavities face in opposite directions (upwards on the support and downwards in the span).
The contemporary presence of compression struts in the two systems stops the cracks induced by the two flexural moments of opposite sign from joining one another. This can be seen quite clearly in Fig. 4.10.

Before designing hollow core floors without negative moments at supports it is well to keep in mind the technical and economic aspects listed in Table 4.1 and evaluate convenience of the continuity restraint in relation to the more simplified design method based on the simply supported floor slabs.

Table 4.1

Design requirements	Advantages of continuity	Disadvantages of continuity
1. Resistance to deflection (and shear) at the S.L.S. and U.L.S.	- with the same hollow core slab depth and prestressing reinforcement it is possible to obtain resisting moments up to 30% higher. Shear strength depends on the number of cores filled at slab ends.	- higher cost of additional reinforcing bars (up to 3 kg/m^2) and concrete cast in situ (up to 20 $litres/m^2$ for connections embedded in slab ends; - it is necessary to use prestressing reinforcement even at the top side of the slabs and some cores must be open at the top of slab ends; - it is necessary to place plugs in cores at a set distance from slab ends; - it is necessary to check the maximum prestressing of slabs on the bottom side to avoid excessive compression forces due to the negative moment.
2. Fire resistance	- with the same slab depth and prestressing reinforcement, strength may increase by 30%	- see point 1
3. Seismic design	- the amount of design horizontal forces is decreased consequent to a higher value of the "q" factor of behaviour of the structure owing to higher ductility and energy dissipation	- see point 1
4. Minimum elastic and long-term deformation under permanent and occasional loads	- with the same slab depth deformations decrease at least 2 folds and up to 5 folds	- see point 1
5. Elimination of cracks visible in correspondence to supports of floors with more than one span	- the requirement is met in all cases through careful planning of the number, diameter and distance between centres of the reinforcing bars for negative moments	- see point 1
6. Reduction of depth of the loadbearing beam	- this is obtained by widening the compressed flange of the beam to the floor	- see point 1
7. Hollow core slabs supported by beams cast in situ with same depth as slabs	- this application is possible	- see point 1; - the cross section of the hollow core slab having a width of 1.20 metres must be such that the overall thickness of webs is about 40 cm so as to minimize suspension stresses; - prestressing of the bottom side must not be excessive in order to keep spalling stresses in all webs under control.

Floors made with hollow core slabs can be considered continuous for loads acting after their assembly if the following conditions are met:

a) continuity reinforcement bars are placed at distances between centres of no more than 60 cm;

b) rules on adherence between precast concrete and in situ casts are complied with (see paragraph 3.4.1 above) as well as between in situ casts and the embedded continuity bars. Maximum stress in the bar equal to the value it is subjected to in the end section of the floor must be considered (Italian Standard par. 5.3.3. and 7.3.3.; EC2 ENV 1992 -1-1 par. 5.4.2.1.3; pr EN 1992-1 Section 9);

c) continuity reinforcement bars are anchored at the ends to the lower edge of the slab by means of a hook or bend;

d) the depth of the slab is no less than 12 - 15 cm.

The imposed conditions tend to satisfy the following requirements:

a) the continuity reinforcement must be sufficiently diffuse and well anchored;

b) the bond between the precast element and the integrative concrete must be assured as indicated under points a), b) and c), of paragraph 4.1.2. above and the bond between the steel and the corroborating concrete complies with points d), e) and f) in the same paragraph;

c) the terminal anchorage at the lower flange must eliminate possible detachment of the concrete of the stressed upper flange of the floor (see Fig. 4.11);

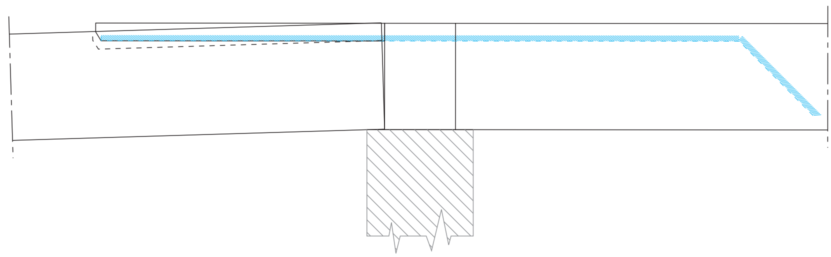


Fig. 4.11 Lacking terminal anchorage at the lower edge, the stressed steel may cause the problem seen here.

d) it is difficult for a hollow core slab floor with a depth of less than 12 - 15 cm to be reinforced to the point of assuring valid structural continuity.

In the case of a floor made continuous, reinforcing bars placed at the bottom edge in correspondence to supports must absorb the tensile stress equal to shear calculated only for its own weight and that of the casts made during completion.
In fact, permanent loads and occasional loads applied subsequently generate negative moments on supports and thus compression, and not traction, at the bottom edge of the floor in the support zone.

Apart from the rheological effects of viscosity and shrinkage, as well as the redistribution of moments, the positive moment of span of the floor is obtained by adding:

in the 1st phase: the moment due to the weight of the hollow core floor itself completed with in situ casting, calculated for simple support;

in the 2nd phase: the maximum positive moment due to permanent loads plus occasional loads calculated for continuos spans in the worst conditions of load.

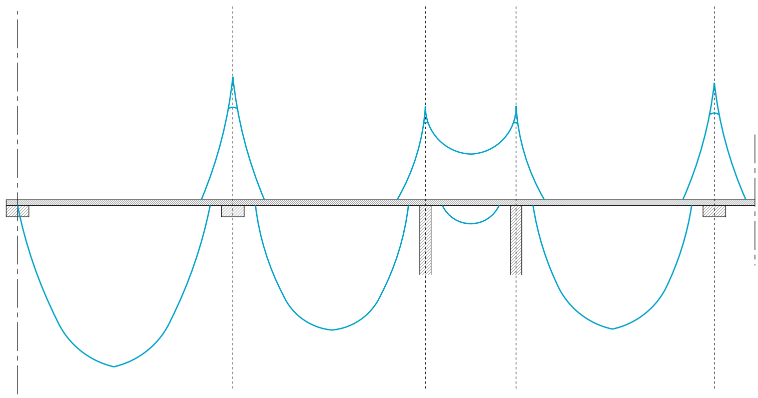


Fig. 4.12 *Trend of the maximum negative and positive moments in a continuous floor due only to permanent and occasional loads in the 2nd phase:*

- *the negative moments to consider for supports are those in the most unfavourable load conditions;*
- *the positive span moments are due to the worst load conditions and are to be added to the positive moments of the 1st phase (weight of the floor itself with simple support);*
- *the rigidness of the several floor spans must be constant in calculating the continuous beam;*
- *it is necessary to take into account the negative span moments generated by special load conditions and, for safety's sake, provide for enough normal or prestressed reinforcement.*

Consequently, the maximum negative moments are obtained from calculation for continuos spans of the 2nd phase only, permanent plus occasional loads, in the most unfavourable conditions.

As concerns the rheological effects of viscosity and shrinkage, see the discussion in the next ASSAP publication.

As indicated in Fig. 4.12 and 4.13, the vertex of the cusp of negative moments is damped in correspondence to the width of the connection, keeping in mind that the same beam already absorbs a part of negative moment in correspondence to the cusp on the support (rounding to a parabola).

4.2.3. Redistribution of the moments due to connection ductility

After calculating the maximum negative moments at supports as described in the previous paragraph, and after rounding the cusps to a parabola as indicated in Fig. 4.12 and 4.13, it is opportune to take into account a further decrease in maximum negative moments at the expense of the maximum positive moments of the span by applying the so-called redistribution of the moments.

The greater ductility of the connection cast in situ, which is clearly lower in strength class than the concrete of the prefabricated element, causes redistribution of a certain amount of the negative moment with an increase in the positive moment of the span (see Fig. 4.13).
Moment redistribution must be performed taking into account the sum of the different ductility factors in the connection:

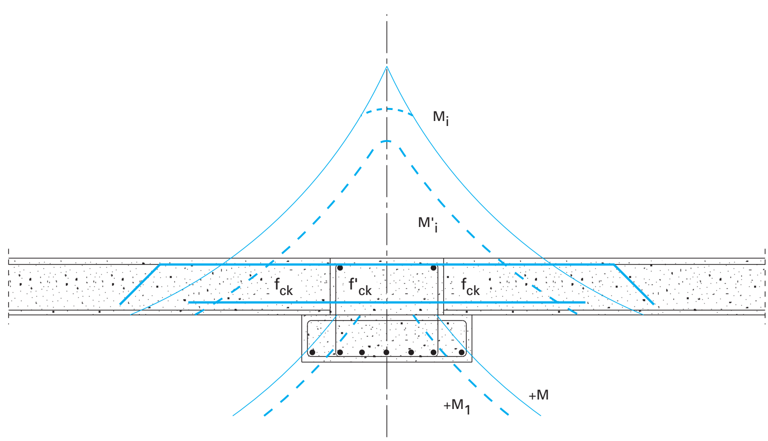


Fig. 4.13 *Negative moment M_i decreases to M'_i due to greater deformability, both elastic and viscous, of concrete with strength f'_{ck}.*
Consequently the positive moments of the span increase by the same amount.

- **in a purely elastic regime** the concrete in the connection has a modulus of elasticity E'_{cm} which is less than the modulus of elasticity E_{cm} of the prefabricated concrete. Redistribution depends on ratio E'_{cm}/E_{cm} (see next ASSAP publication).

- **in a viscous regime** it is necessary to take into consideration the viscous deformation component caused by external actions (loads) and the prestressing component; the effects may even be of opposite sign (see next ASSAP publication); for this reason this component can be considered null in a first approximation.

To conclude, if we indicate with δ the relation between effective negative moment M'_i after redistribution and theoretical negative moment M_i prior to redistribution, which is to say

$$M'_i \quad = \quad \delta\, M_i$$

the value δ, equal to the root of the ratio between the characteristic strengths of the two concretes, is not very far from the truth

$$\delta \quad = \quad \sqrt{f'_{ck} / f_{ck}}$$

so long as one satisfies the condition imposed by the Italian Standard (Art. 4.1.1.3), the European Standard (ENV 1992-1-1, Art. 2.5.3.4.2.) and also pr EN 1992-1 (Section 5.5), in which, given:

x = depth of the neutral axis at the ULS after redistribution
d = effective height of the section

the result must be:

$$\delta \quad \geq \quad 0.44 + 1.25\ x/d \qquad \text{(for concrete } f_{ck} \leq \text{C35/45)}$$
$$\delta \quad \geq \quad 0.56 + 1.25\ x/d \qquad \text{(for concrete } f_{ck} > \text{C35/45)}$$

In any case, with highly ductile steel of the FeB 44k type δ must be between 0.7 and 1.0.

$$0{,}7 \leq \delta \leq 1{,}0.$$

Once the real amount of negative moments on supports has been ascertained, it is necessary to plan a sufficient number of traction-resisting bars anchored for the regular length of bonding, starting from the point at which they are no longer stressed (see ENV 1992-1-1, points 5.2.2.3; 5.2.3.4; 5.4.2.1.3 and also pr EN 1992-1 Section 9.0).

It will also be necessary to make sure that compression stresses induced on the lower flange of the hollow core floor by the negative moment, added to prestressing stresses in that section, are not above the maximum value admissible for the concrete class of the prestressed element.

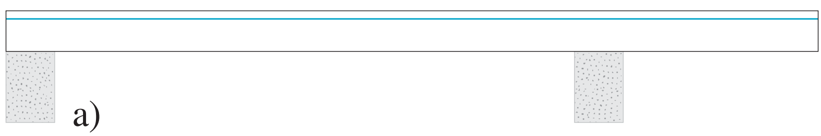
It must always be kept in mind what was stated at the beginning of this paragraph, that is, that redistribution of moments must increase the positive moment of the span to the same extent that the negative moments on supports decrease.

4.2.4. Restraint for cantilevers

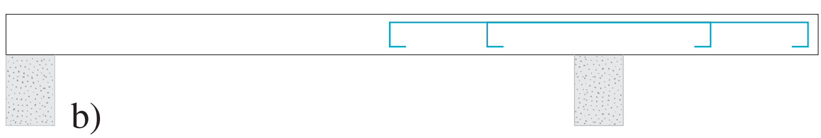
Hollow core floors with cantilevers can be made in different ways as shown in Fig. 4.14.

The way shown in Fig. 4.14 a) is usually applied in presence of cantilevers of less that 1.20 - 1.50 m with light loads.
The prestressing steel in the upper part is normally put there by manufacturers who handle hollow core slabs with fork lifts, which inevitably cause overhangs with dynamic stresses.
Eaves and other small overhangs are made with this kind of reinforcement, but not in the case of heavy loads.

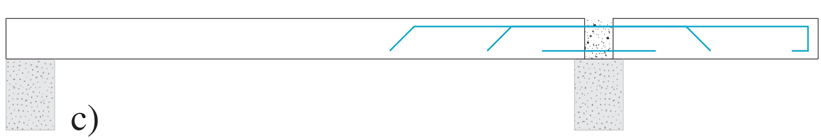
In fact, the anchorage of prestressing steel in concrete in the upper part of the hollow core slab, which is somewhat less compact than the corresponding intrados zone, must be checked in all cases.
Moreover, this prestressing reinforcement of the cantilever is not very expedient since it detracts from loadbearing capacity of the hollow core slab in the span between the two supports.



Prestressed tendons for prestressing of the upper part.



Reinforcing bars inserted into the slab when concrete is still wet or in situ in the cores opened for the purpose.



Reinforcing bars inserted in situ with temporary shoring of the overhanging slab.

Fig. 4.14 Three different ways of producing a hollow core floor with cantilever.
In the case of overhanging floors with corroborant concrete topping, reinforcement of the cantilever is normally inserted in the topping cast in situ and solution b) is normally preferred.

The way shown in Fig. 4.14 b) is the one most commonly used with medium cantilevers and loads since it offers assurance of excellent static performance.
In any case, it is essential to verify compression stresses in the concrete on the lower side of the floor in correspondence to the support of the cantilever since the compression due to the negative moment is added to the prestressing of the prestressed structure itself.
For this reason it is best also to verify the presence of any ugly elastic and viscous downward deflections of the cantilever caused by excessive compressional stresses on the bottom side.

The way shown in Fig. 4.14 c) is the best in the presence of long cantilevers (even up to 5 m) and/or very heavy loads.
Also in this case it is necessary to verify compression stresses in the concrete on the bottom side of the slab on two supports because of pre-existent prestressing in the slab end zone.
This solution is excellent also from the aesthetic viewpoint because shoring allows the raising of the extremity of the cantilever, thus preventing sagging caused by elastic and viscous deformation of the fixed end of the cantilever.

4.3. The beam-floor connection.

4.3.1. Premise

Here we examine one by one the several kinds of connection between bearing beams and hollow core floors.

In all the examples of construction that follow, it is of fundamental importance to keep in mind that the end of any hollow core slab filled to the desired depth with concrete cast in situ, properly vibrated and having the desired characteristic strength, works quite well as a structurally corroborant flange of the bearing beam to which it is connected by appropriate reinforcement.

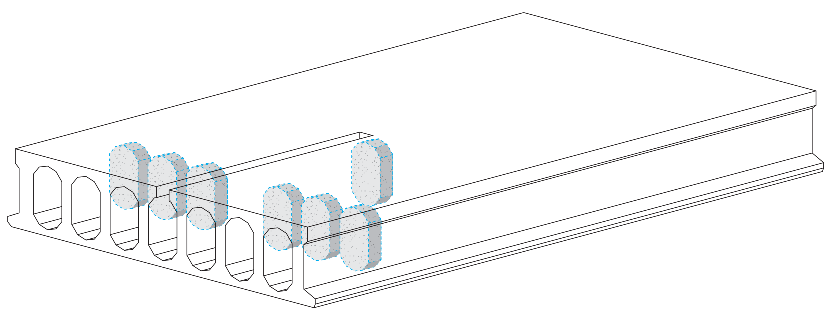


Fig. 4.15 *Slab end prepared for casting of a small flange corroborant with the bearing beam.*

The inertial moment of the bearing beam is greatly increased when the flanges of the corroborant floor are taken into account. For simplicity's sake they are calculated as a full rectangular section composed of concrete having the same strength class as the concrete cast in situ.
To obtain that flange function from the hollow core slab end it must be prepared for this in the planning stage as illustrated in Figs. 4.15 and 4.16.

It is also indispensable to provide upper and lower linking reinforcement between hollow core slabs whose ends represent flanges of the bearing beams so that there is no formation of cracks caused by negative moments and excessive transversal shear stress near the extremities of the floor not adequately connected to the beam by appropriate reinforcement.

If the width of the flange is limited to about 1.5 - 2.0 times the depth of the slab, the cores not touched on by opening must be plugged up at the desired depth as described in paragraph 2.3.7. and illustrated in Fig. 4.15.

In this case the concrete cast in situ and well vibrated is capable of penetrating into the cores to the depth of the plugs and compacting properly up to the point of filling all the empty space available.

If the width of the flange is greater, up to the point of reaching the widest extension allowed by the codes (Italian Standard or ENV 1992-1-1, Art. 2.5.2.2.1.) it is essential to prepare the slab end with all cores open at the top to the necessary lenght (see Fig. 4.16), to allow the filling of the flange with concrete cast and vibrated in situ.

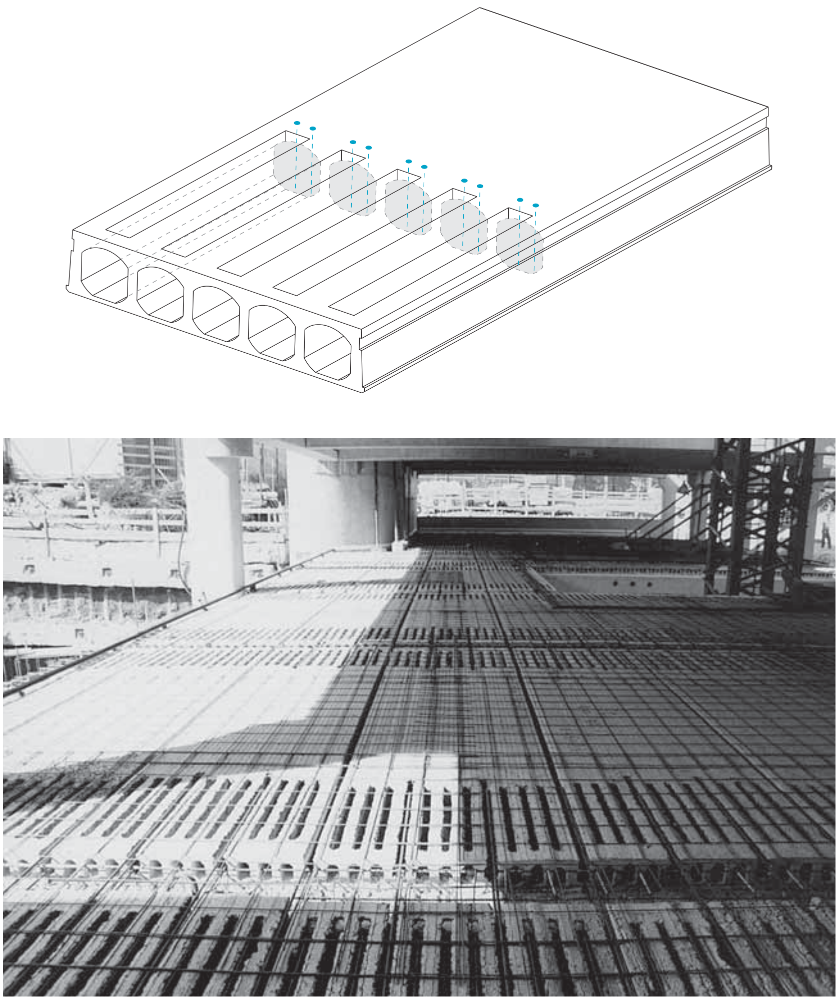


Fig. 4.16 Slab end prepared for casting of a fairly large corroborant flange

4.3.2. Inverted T – and L – shaped precast beams

Beams of this type are usually self-bearing and there is no corroborant flange. The hollow core floor must almost in all cases be considered as having simple support owing to the presence of the vertical nucleus of the beam which does not facilitate casting in situ of the beam-floor connection (see Fig. 4.17).

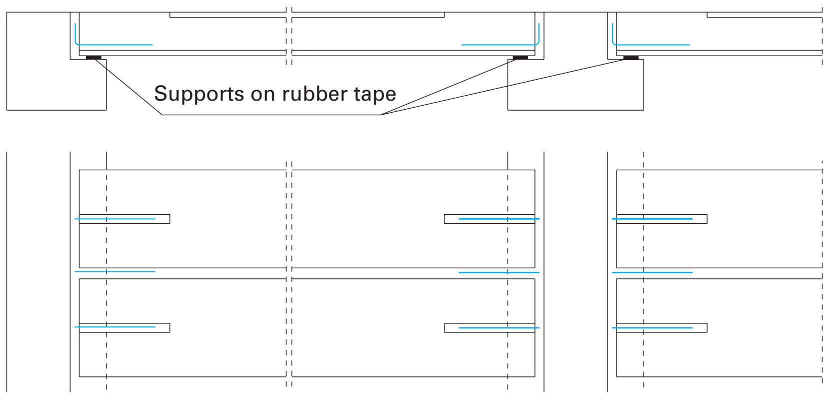


Fig. 4.17 Hollow core floor with simple support on inverted T and L beams.

It is thus a good idea to prepare the support of the hollow core floor on special rubber tape (see paragraph 3.6.) since the restraint remains a simple support even for permanent loads and occasional loads applied subsequently.

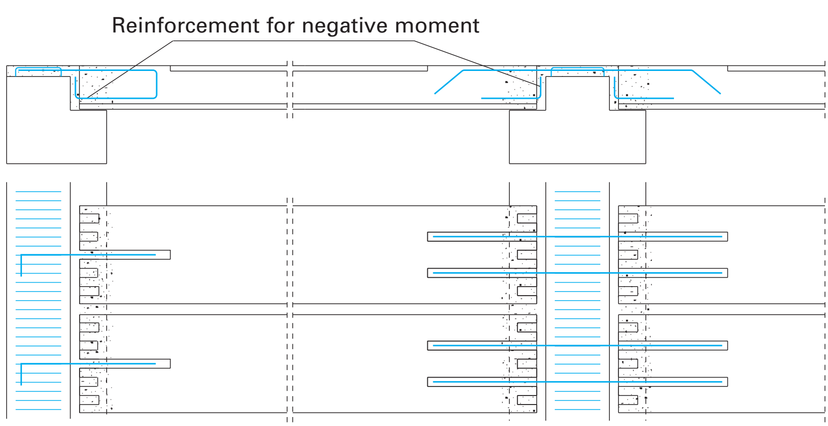


Fig. 4.18 Hollow core floor in continuity on an inverted T beam.

The restraint of continuity between hollow core floors can be accomplished only if all, or at least an adequate number, of cores at the slab ends are open at the top to allow good casting in situ between the end of each slab and the nucleus of the beam.

This cast concrete, if well compacted, supplies the indispensable horizontal opposition to compression forces generated on the lower side by negative moments of continuity (see Fig. 4.18).

4.3.3. Precast I beams

These beams are almost always prestressed and are always self-bearing, even for the weight of the floor itself and the completing castings.

After the hollow core floor has been laid, reinforcing bars that assure the type of restraint chosen for permanent and occasional loads applied subsequently are positioned.

A corroborant flange, if present, made with in situ casting of the beam-floor connection, can be taken into account only in the case of a floor made continuous, or at least partially continuous. The corroborant flange comes into play only for permanent and occasional loads.

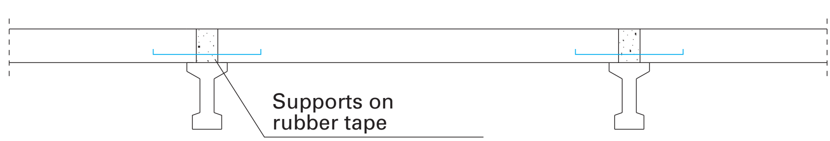


Fig. 4.19 Precast I beams with simply supported floor.

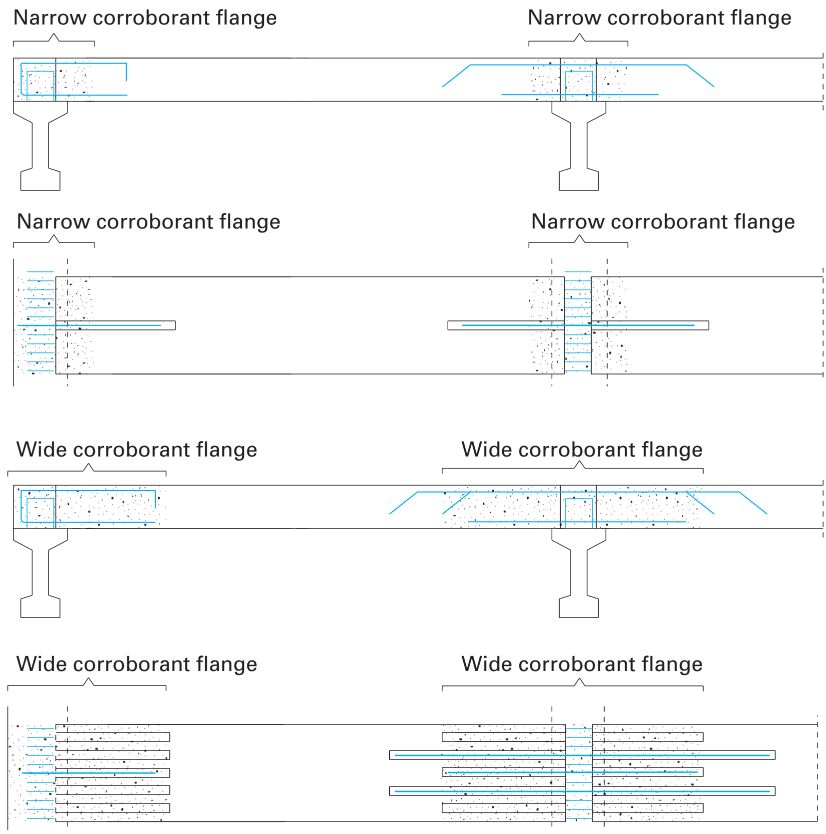
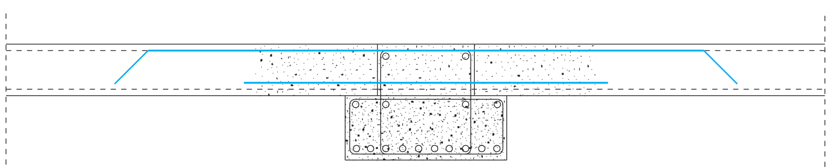


Fig. 4.20 *Precast I beams with hollow core floor in continuity and some corroborant flanges:*

- *a narrow corroborant flange is obtained by inserting plugs into the cores inside the slab end;*
- *a wide corroborant flange is obtained by opening all cores in the slab end.*

4.3.4. Semi-precast beams

These are composed of a precast bottom part which may also be prestressed. They are provided with stirrups pointing upwards.
Sometimes these partially precast beams are self-bearing only for their own weight.
In such cases they must be temporarily shored to support the weight of the floor and the in situ castings which, on penetrating into the end of the hollow core floor, form the corroborant flange of the beam, if required.
To assure structural corroboration of the flange with the beam, the floor must be restrained to the beam with reinforcement both at top and bottom approximately every 40 cm and the linking stirrups must be verified to make sure they can withstand shear stresses. When the semi-precast beam is shored prior to the laying of the slabs, all weights of the beam, the floor and the in situ casts, besides permanent and occasional loads, are supported by the completed beam with corroborant flange.



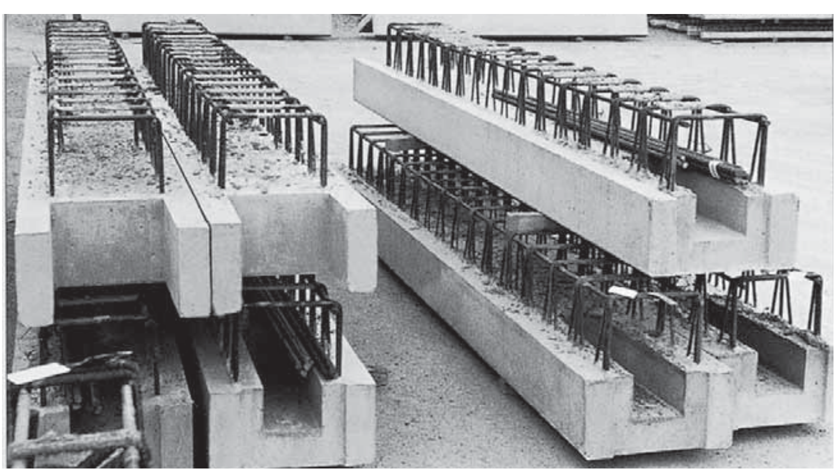


Fig. 4.21 Semi-precast beams of normal or prestressed concrete. The beam-floor connection must always be a continuous, or at least partially continuous, restraint.

4.3.5. Steel H-beams

The combination of hollow core floors with steel beams is quite common in North America and northern Europe. It is less frequent in Italy owing to the limited use of steel bearing structures.
The hollow core floor is almost always placed on the upper flange of the beam, on which the connecting rivets that assure structural corroboration between steel beams and in situ castings are welded.

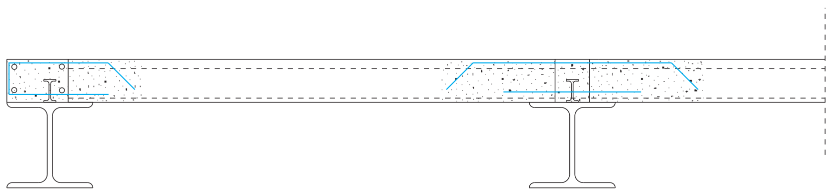


*Fig. 4.22 Hollow core floors on steel beams.
Connecting rivets can be induction-welded to the upper flange of the beam even after the laying of the slabs.*

If the hollow core floor is made continuous, or at least partially continuous, on the steel beam with rivets, it is advantageous to take into account the corroboration of the concrete of the ends of the floors that constitute the compressed upper flange of the resulting composite beam.

Steel beams are always sized to support self-bearingly the weight of the floor and in situ castings. Thus the composite beam must support only permanent and occasional loads applied subsequently.

The hollow core slab floor is rarely placed on the bottom flange of the steel beam because of the difficulties encountered in assembly.

In cases of this kind it is almost impossible to obtain a continuous restraint between floors because of the difficulty in achieving a perfectly compacted casting between the end of the floor and the vertical ribbing of the beam.

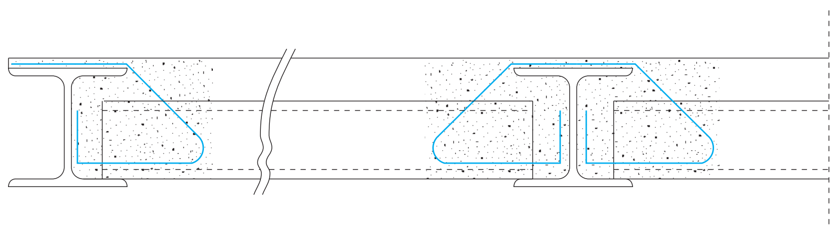


Fig. 4.23 The hollow core floor placed on the bottom flange of the steel beam.

4.3.6. Steel reticular beams

Steel reticular beams are composed of a bottom plate provided with upper trestlework.
The hollow core floor lies on the bottom plate and the in situ casting involves the entire depth of the beam and also penetrates into the cores to the desired depth.
The floor is always linked to the beam with diffuse reinforcement assuring the restraint of continuity or at least partial continuity.
Prior to the laying of the hollow core slabs on the reticular beams, except in special cases of self-bearing expressly called for in the design, it is essential to shore the beam to support the weight of the floor during assembly and also to avoid possible warping or even tilting under eccentric loads represented by slabs not laid symmetrically.

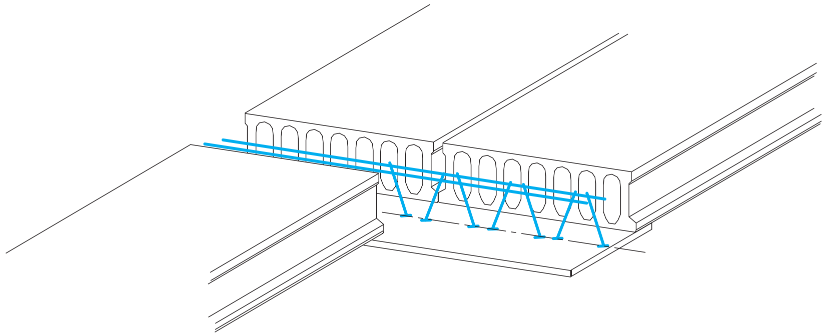


Fig. 4.24 The hollow core floor supported by a reticular steel beam.

The composite beam formed after the in situ castings is traction-reinforced by the bottom plate and the compression resistant zone is represented by the concrete flange in the hollow core slab floor widened to the extent necessary to support all weights and loads present.

The reinforcing bars linking the hollow core floor to the composite beam must be well diffused to act as the stirruping of the wide-flanged beam that results.

4.4. Beams cast in situ

At least one third of hollow core floors produced in Italy are used together with cast in situ bearing beams having normal reinforcement.

The hollow core slabs are almost always laid prior to casting of the beams and are thus temporarily supported by the timber of the formwork for the casting of the beams themselves.
The reinforcement of the beam is bound to the hollow core floor by means of the continuity reinforcement of the floor.

The in situ castings form the actual body of the bearing beam and extend into the hollow core slabs to complete the corroborant flange and seal the longitudinal unions between slab and slab.

The resulting beam, configured with a widened upper flange, supports all loads composed of the dead weight of the beam itself, the floor and subsequent loads.

Essentially, the following three paragraphs describe two types of connection between a hollow core floor and a beam cast in situ:

- hollow core floor with length of support on the beam cast in situ;
- clear span hollow core floor without length of support on the beam.

The latter case, which is encountered quite frequently, requires in-depth knowledge of the beam-floor connection and of the series of verifications analysed in point 4.4.4 below.

4.4.1. Floor with support on beam

In presence of heavy overloads on floors, or loads of the kind found on roads, recourse is had to a beam with a cross section with a widened base allowing the floor to find adequate support in the body of the beam.

The stirruping of the beam must be designed as shown in Fig. 4.25.

This kind of beam can also be cast in two stages in the presence of special conditions such as large distances between floors when the cost of shoring is very high.
In such cases (see Fig. 4.26) the bottom of the beam is cast first after verifying its self-bearing capacity to support the dead weight of the floor.
Then the hollow core slabs are laid and the finishing casts capable of supporting subsequent occasional overloads in the beam's final configuration are made.

Casts extend into the hollow core slabs to form the corroborant flanges of the beam which are properly linked to the stirrups of the beam with diffuse reinforcement

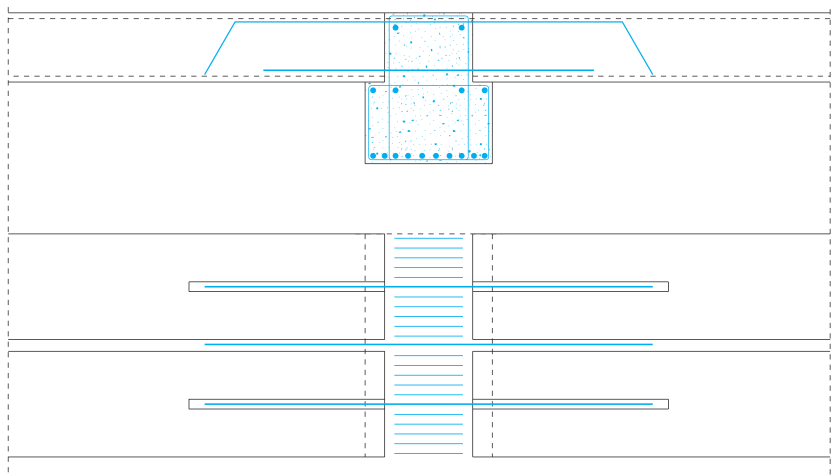


Fig. 4.25 *The floor-beam connection cast in situ with length of support of the floor.*

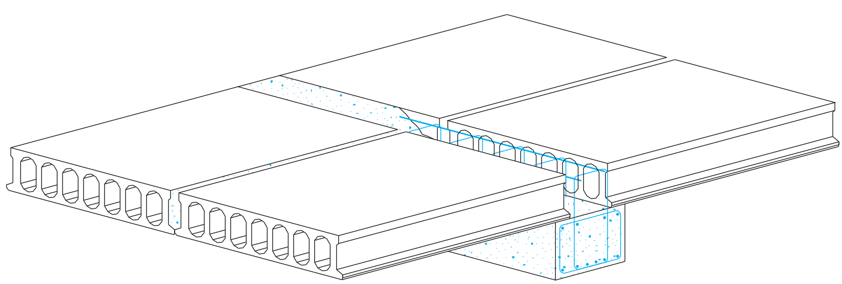


Fig. 4.26 *Beam previously cast in situ to be self-supporting of the dead weight of the floor itself and the finishing casting.*

4.4.2. Clear span floor without support on beam

It is often the case that widening the base of the beam to support the hollow core slabs is not statically justified.

As illustrated in Fig. 4.27, the beam can support the floor owing to the in situ casting of concrete, which from the beam penetrates into the cores of the floor, and to the continuity and linking reinforcement.

Although calculation and verification of this section will be discussed in paragraph 4.4.4., we observe here that if the concrete nuclei cast in the cores are not capable of supporting shear stresses from the floor, it is necessary to insert a shear-resistant stirrup in each nucleus (see Fig. 4.28).

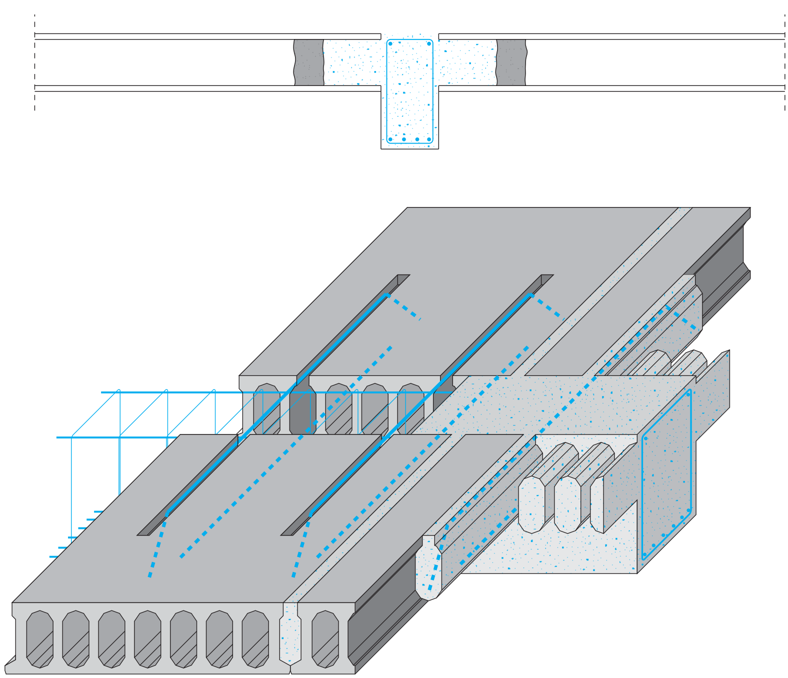


Fig. 4.27 The beam-floor connection with a clear span

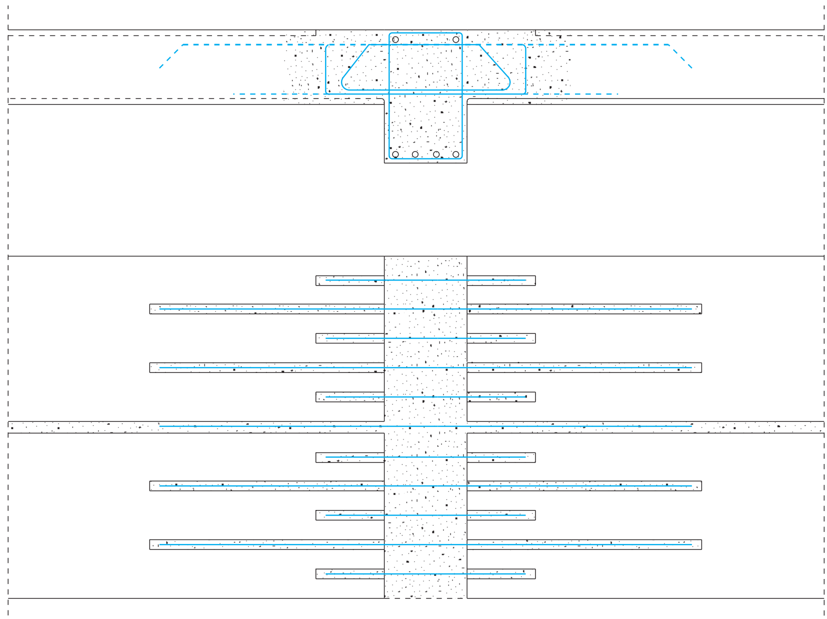


Fig. 4.28 *An example of stirruping of the in situ cast nuclei in the cores and in the longitudinal union.*

In cases of this kind the floor must always be designed with the continuity restraint and it must therefore provide for double reinforcement distributed every 30 - 40 cm in correspondence to the top and bottom sides of the floor in cores open for the purpose.

In this particular beam-floor connection, each hollow core slab is suspended from the beam by means of the upper portion of the single vertical web between core and core.

Since these webs are not stirruped, the vertical traction stress that is created in each web due to suspension tends to increase spalling stress, described in paragraph 3.5.1., and in particular in paragraph 3.5.2.

It is thus indispensable to perform verification as described in paragraph 4.4.4., which may supply stress values $\boldsymbol{\sigma}_{sp}$ that satisfy the requirements of European Standard EN 1168, Eurocode 2 ENV 1992-1-1 and the Italian Building Standard.

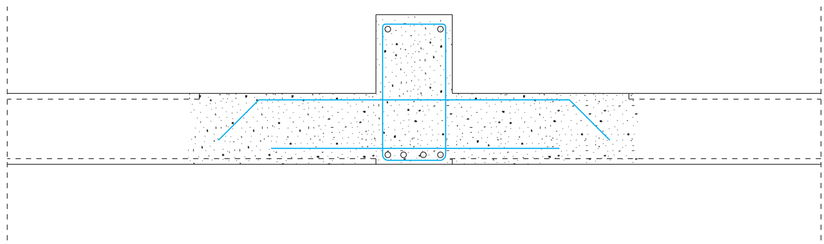


Fig. 4.29 Clear span floor suspended from a raised beam.

A variation of the connection discussed in this paragraph is represented by the less frequent situation illustrated in Fig. 4.29.

Obviously the static functioning of the beam-floor connection is the same as the one illustrated in Fig. 4.27.

4.4.3. Flat beam having depth equal to that of the hollow core floor

In building practice, this case is met with quite frequently, especially when there are no excessive overloads (residential buildings, services and also parking silos).
Also with this kind of beam the width of the compressed concrete flange to be used in calculations includes the hollow core slab ends to the length delimited by plugs previously inserted in the cores.

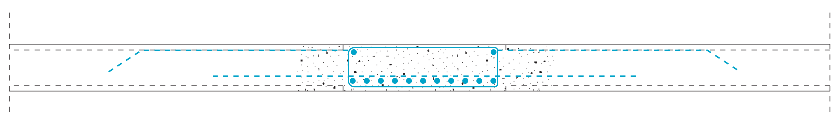
All cores must be filled with concrete in situ to a length at least equal to the depth of the floor.

Very special mention must be made of hollow core floors supported by steel reticular beams (see par. 4.3.6 above) or flat beams or beams of the kind shown in Fig. 4.29.

In all these cases, every deflection of the beam is met by an identical transverse deflection of the hollow core slab ends bound to it. If deflection accentuates to the point of causing cracking in the beam concrete (cracking admitted in the presence of normal reinforcement) there will also be cracking in the hollow core slabs, as in any other kind of floor. Cracks start from the slab ends and continue for a short distance.

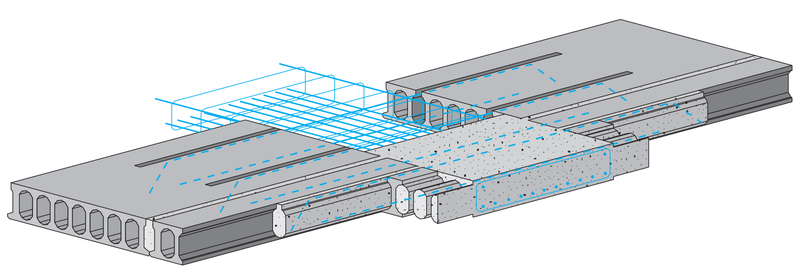
It is to be noted that these longitudinal cracks, when they appear, always open below the cores and never reach the zone below the webs in which the prestressing strands are anchored.

However, these cracks, which detract from appearance, must be avoided and it is therefore a good idea to design beams that are sufficiently rigid to avoid cracking.



a)

The double linking reinforcement is placed every 30 to 40 cm with a suitable bonding length, which must be at least equal to the length of transfer of prestressing



b)

All cores must be filled with concrete in situ to a length at least equal to the depth of the floor.

Fig. 4.30 a) Connection of the hollow core floor in continuity to a flat beam.
b) Conformation of the in situ casting of the flat beam.

4.4.4. Design of the composite connection between cast in situ beam and hollow core slab without direct support

Since no stirrups in the webs of prefabricated slabs are called for, linking in continuity between hollow core slabs and beams cast in situ without support for the floor must conform to the following prescriptions formulated in conformity with the European Standard EN 1168, Art. 4.3.1.6, with Eurocode 2 and the Italian Building Standard. The prescriptions thus formulated are confirmed both by experimental results obtained in research conducted specifically at the Politecnico of Turin (Prof. F. Levi, Mr. R. Perazzone, Prof. P.G. Debernardi from 1982 to 1985), and by static tests of structural applications performed in Italy in the last two decades.

Prescriptions

a) hollow core slabs must be at least 15 cm deep.

b) All cores must be plugged in such a way as to assure that they are filled with compacted concrete up to a distance from the end of the slab at least equal to the depth of the slab itself.

c) Floors shall be made effectively continuous for moments induced by occasional and permanent loads applied after execution of the structure.

d) The continuity reinforcing bars on the top and bottom sides shall be placed with a distance between centres not above 30 - 40 cm on the average and shall comply with bonding conditions between in situ casts and slab concrete as discussed in paragraph 3.2 herein.

e) Shear stresses flush with the beam in the connection section of the nuclei in concrete cast in situ to fill the cores shall be less than those allowed for concrete without shear reinforcement (according to the Allowable Stresses method) or the value of the calculation of V_{Sd} at the ultimate limit state shall be less than value V_{Rd} of the section (according to the Limit State method). Otherwise, it is necessary to verify the shear-resisting reinforcement inserted in the open cores at the end of the hollow core slab (see Fig. 4.28).

f) The shear stresses mentioned above are partially reduced by the bond between the beam cast in situ and the end section of the slab, as long as it is rough (not cut with a diamond disc).

g) The maximum vertical tensile stress σ_{spi} (spalling) verified on removal from the bed at slab ends, due to anchoring stresses of prestressing reinforcement, shall be less than the tensile strength of the concrete $f_{ctk\,0,05}/\gamma_{sp}$ calculated at the time of applying prestressing with $\gamma_{sp} = 1{,}2$.

h) The principal tensile stress in the most stressed web, due to the contemporary presence of spalling $\sigma_{sp,d(t)}$ (calculated at the time of being put to service for utilization with the formula $\sigma_{sp,d(t)} = \gamma_p \sigma_{spi} P_{m(t)}/P_{m,o}$) and stresses due to the suspension of the floor τ_{sd} (calculated for the dead weight of the floor and all overloads multiplied by the relative coefficients γ_G and γ_Q) shall be less than the final strength of the concrete f_{ctd} .

Comments on prescriptions

a) The cores of a hollow core floor less than 15 cm deep are too small to be filled with any certainty to the desired depth with concrete cast in situ.

b) The nuclei in the cores and open cores containing reinforcement must be created by casting concrete coming from the beam. It must be well vibrated and then held in place by tightly fitting plugs in the cores.
The length of the nuclei in the cores, which must be at least equal to the depth of the hollow core slab, also assures a contact surface between the in situ casting and webs of the prefabricated slab sufficient to hold by bonding the dead weight of the floor and the relative loads.

c) The monolithic character of the beam-floor connection is fundamental for the functionality of the composite connection without direct support.

d) The continuity reinforcement bars must therefore be sufficiently diffuse. A distance between centres of the bars not above 30 - 40 cm requires that at least half of the cores shall contain reinforcement. In practice, the cores filled by casting without reinforcement alternate with open cores containing reinforcement. Reinforcing bars may be counted in calculating the suspension of the floor.

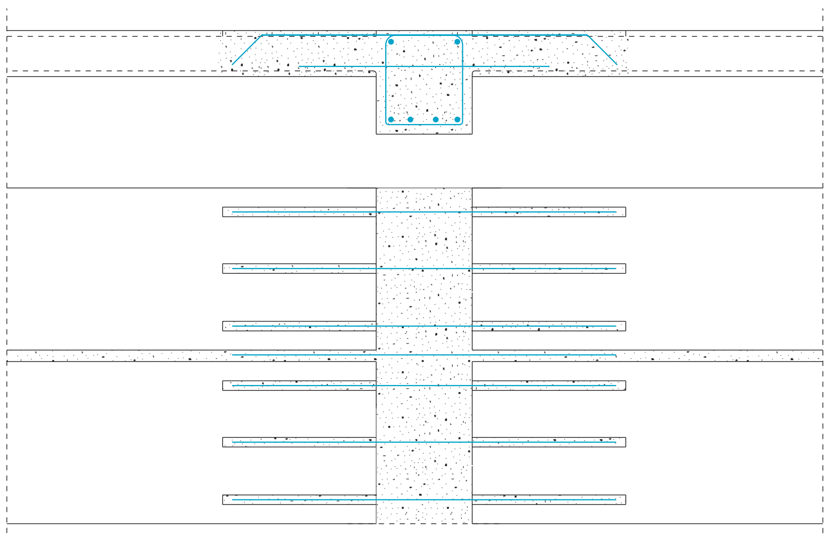


Fig. 4.31 Continuity reinforcing bars are provided with a distance between centres which on the average is not more than 30 - 40 cm.

e) The attachment section of the concrete nuclei to the beam must be able to support, even without stirruping, the shear stress generated by its part of the load (see Fig. 4.32).

The design shear resistance has to satisfy the prescriptions contained in the Codes also taking into account the reinforcing bars for continuity restraint. (Italian Building Standard Art. 3.1.4.; Eurocode 2 ENV 1992-1-1, Art. 4.3.2.3 and also pr EN 1992-1 Section 6.2).

Failing this shear capacity it is necessary to insert stirruping in the open cores (see Fig. 4.28).

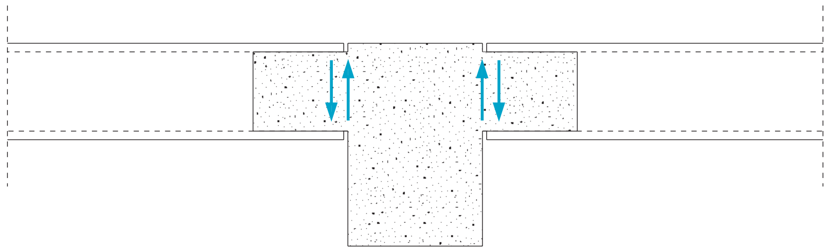


Fig. 4.32 Shear stresses in the attachment section of concrete nuclei to the beam when shear reinforcement is missing.

f) The bond between beam and the rough end of the hollow core slab, when not cut with a disc, corroborates the attachment section of the nuclei and participate actively in supporting the floor (see Fig. 4.33).

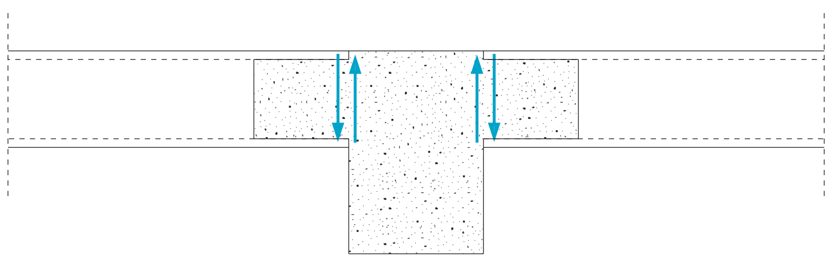


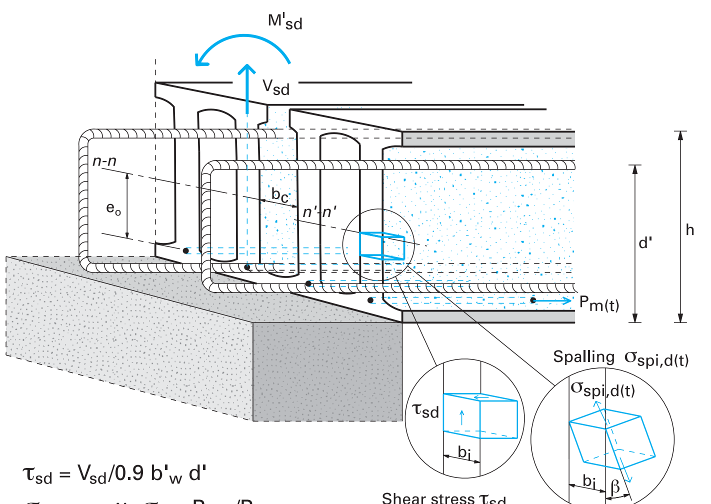
Fig. 4.33 If the ends of the hollow core slab are rough, shear stresses at the interface with the beam are distributed throughout the entire end section of the slab.

g) It is necessary to ensure that the hollow core slabs for this kind of beam-floor connection are laid in situ without any sort of crack already open at the ends.

It is thus necessary to make sure that vertical spalling stress $\sigma_{\mathbf{spi\ max}}$ (see paragraph 3.5.2. above) at the time of the application of prestressing is definitely within the limits prescribed by Standard EN 1168, Art. 4.3.1.6, by taking into account the safety coefficient $\gamma_{sp} = 1.2$.

h) Safeguarding against the appearance of cracks in hollow core floor ends suspended in the operating phase requires the maximum principal stress at the end of each web to remain below threshold f_{ctd} calculated on the final strength of the concrete of the slab. Indeed, when the building comes into use, vertical spalling stress σ_{sp} is reduced with respect to the time of the application of prestressing (see point "i" of the concluding observations in paragraph 3.5.2.), but it combines with stress τ_{sd} due to the suspension of the slab on the nuclei of concrete protruding from the beam (see Figs. 4.27 and 4.30).

The prescription is effective according to both Limit States and Allowable Stresses methods, depending on the designer's choice.



$$\tau_{sd} = V_{sd}/0.9\, b'_w\, d'$$

$$\sigma_{spi,d(t)} = \gamma_P\, \sigma_{spi}\;\; P_{m(t)}/P_{mo}$$

$$b'_w = \sum b_i + n, b_c\, E_{cm}/E_{pm}$$

$$\sin 2\beta = \cos 2\beta \cong 0.7$$

$$\sigma_{I,d} = \frac{\sigma_{spi,d(t)} + \tau_{sd}\cos 2\beta}{2} + \sqrt{(\tau_{sd}\sin 2\beta)^2 + \left(\frac{\sigma_{spi,d(t)} + \tau_{sd}\cos 2\beta}{2}\right)^2} \leq f_{ctd}$$

Fig 4.34 *Principal stress σ_{Id}, that is, the combination of spalling stress with suspension stress in the end of web "i" of the hollow core slab when the ordinary support is lacking.*

The verification indicated in prescription **h)** consists of calculating principal tensile stress $\sigma_{I,d}$ which must not be greater than the tensile strength of the prefabricated concrete in the most stressed web (see Fig. 4.34).

$$\sigma_{I,d} = \frac{\sigma_{spi,d(t)} + \tau_{sd} \cos 2\beta}{2} + \sqrt{(\tau_{sd} \sin 2\beta)^2 + \left(\frac{\sigma_{spi,d(t)} + \tau_{sd} \cos 2\beta}{2}\right)^2} \leq f_{ctd}$$

with

τ_{sd} = calculated value of shear stress $\tau_{sd} = V_{sd} / 0.9\, d'b'_w$;

$\sigma_{spi,d(t)}$ = calculated value of σ_{sp} at time (t) in the most stressed web: $\sigma_{spi,d(t)} = \gamma_p\, \sigma_{spi}\, P_{m(t)} / P_{m.o}$;

f_{ctd} = design tensile strength of the slab concrete;

V_{sd} = shear in the interface section between prefabricated slab and in situ cast;

β = angle between spalling stress and shear (approximately $\beta \cong 20° \div 25°$ and thus $\cos 2\beta \cong 0.7$);

b'_w = total width of the webs in the composite end section of slab also taking into account the number of cores filled with concrete coming from the beam: $b'_w = b_w + n\, b_c\, E_{cm} / E_{pm}$;

d' = effective height of the composite reinforced section;

n, b_c = number and width of cores filled with concrete coming from the beam;

E_{cm} / E_{pm} = ratio between the elastic moduli of cast concrete and prefabricated concrete;

γ_p = partial safety factor of prestressing forces for ULS: $\gamma_p = 1.2$;

b_i = width of web "i" undergoing the greatest spalling stress in the hollow core slab;

b_w = total width of the webs of the prefabricated hollow core slab;

σ_{spi} = spalling in the webs at the time of prestressing, calculated according to point 4.3.1.6. of the European Standard EN 1168 (see paragraph 3.5.2.above);

$P_{m,o}$ = mean force of prestressing at time of application to the slab;

$P_{m(t)}$ = mean force of prestressing at time (t): $P_{m(t)} = P_{m,o} - \Delta P_t$;

ΔP_t = loss of prestressing force at time (t): $\Delta P_t = (P_{m,o} - P_{m,\infty})\, \alpha_t$;

$P_{m,\infty}$ = final force of prestressing after deduction of all losses;

α_t = non-linear coefficient in order to obtain effective losses as a function of time;

t = time elapsed from application of prestressing to application of actions

t	=	2 months	α_t	=	0.5
t	=	3 months	α_t	=	0.6
t	=	6 months	α_t	=	0.7
t	=	1 year	α_t	=	0.8

Concluding remarks

The prescriptions listed clearly indicate that hollow core slabs with the largest number of cores and webs offer the best assurance of good suspension on beams cast in situ owing to the good diffusion of continuity reinforcement and the higher value of b_w.

That is to say, the greater the sum of thicknesses of webs in a hollow core slab the greater is its suspension loadbearing capacity, while value $\sigma_{I,d}$ is greatly reduced.

To keep spalling stresses low it is a good idea for the hollow core slabs not to be excessively prestressed and therefore for their slenderness not to be extreme.

For the safe suspension of a slab having a width of 120 cm it is best for the sum of thicknesses of webs to be $b_w \geq 38 \div 40$ cm and for the l/h (slenderness) ratio to be kept below 30 ÷ 35 for floors with normal civil loads. With heavier overloads (8.0 - 10.0 kN/m^2) it must remain below 30.

Examples of connection verifications

Example 4.1

A beam cast in situ carries two spans of floor in continuity: h = 300 mm having width b = 1.2 m and clear span l = 9.60 m for both spans.

The dead weight of the assembled floor is	4.0 kN/m^2
Total overload capacity is	8.0 kN/m^2

The characteristics of the cross section and reinforcement in the hollow core slab are those indicated in Fig. 4.35 (cross section of the floor near the slab ends).

Prescriptions **a), b), c), d)**, are complied with. Condition **f)** is considered not applicable since the hollow core slab ends were cut with a diamond disc.

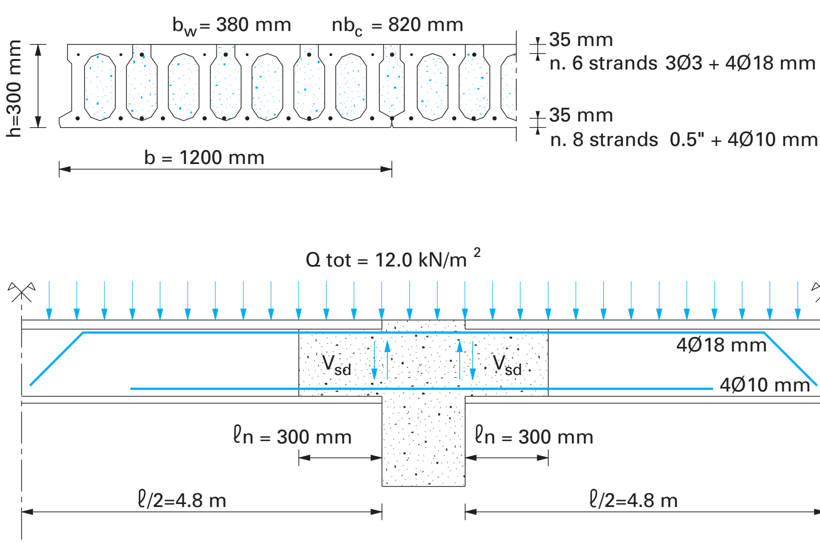


Fig. 4.35 *Verification of a slip-formed hollow core floor of depth h = 300 mm as a clear span between beams cast in situ (floor without direct support on the beam).*

A check is made on prescription **e)** to see that it is verified.
In correspondence to the attachment of each hollow core slab to the beam, the shear value with a slab 1.20 m wide at the ULS is:

$V_{sd} = (\gamma_G\, G + \gamma_Q\, Q)\; b\; l/2$

$V_{sd} = (1.4 \times 4.0 + 1.5 \times 8.0)\; 1.2 \times 4.8 = 101.38$ kN

Lacking stirruping in the nuclei of cast concrete, it is necessary to verify the calculated shear strength of the section of attachment of the nuclei to the beam.
It must be:

$V_{Rd} \geq V_{sd}$

with

$V_{Rd} = \tau_{Rd}\; k\; (1.2 + 40\rho)\; n\; b_c\; d'$ (EC2 ENV 1992-1-1 Art. 4.3.2.3.)

The following value is attributed to Class C25/30 concrete with $f_{ck} = 25$ N/mm^2 and with $\gamma_c = 1.6$:

$\tau_{Rd} = 0.28$ N/mm^2 (see table 5.1 Cap. 5 below)

it is also:

nb_c = 820 mm total width of nuclei cast in situ in the slab end;

d' = 245 mm the effective height of nuclei cast in situ;

k = 1.6 - d' [m] = 1.355 dimensional coefficient (ENV 1992-1-1, Art. 4.3.2.3)

A_{fl} = 1018 mm^2 cross-sectional area of the upper four bars Ø18 mm;

$\rho = A_{fl}/n\; b_c\; d' = 0.00506$ reinforcement coefficient;

thus

$V_{Rd} = 0.28 \times 1.355\; (1.2 + 40 \times 0.00506) \times 820 \times 245 =$

$= 106892$ N $= 106$ kN $> V_{sd} = 101.38$ kN

which means that verification is satisfactory, although at the limit of acceptability.

Prescription **g)** was verified in paragraph 3.5.2. by example of calculation 3.2.

It is seen (formula [SP]in paragraph 3.5.2.) that for webs reinforced with one 0.5" strand the value of vertical tension stress at the end of the slab (spalling) at the time of application of prestressing is:

$\sigma_{spi} \quad = \quad 0.815 \text{ N/mm}^2$

Since we considered that the concrete of the slab at the time of applying prestressing was Class 30/37, we have:

$f_{ctk\ 0.05} \quad = \quad 2.03 \text{ N/mm}^2$

Stress σ_{spi} is admitted at the time of applying prestressing and it is also accepted in the case of a future link of the floor without direct support on the beam. It must be:

$\sigma_{spi} \quad \leq \quad f_{ctk0.05} / \gamma_{sp}$

with

$\gamma_{sp} \quad = \quad 1.2$ [as in prescription **g)** above]

In reality we have

$\sigma_{spi} \quad < \quad 2.03/1.2 \text{ N/mm}^2 = 1.69 \text{ N/mm}^2$

Now we must verify prescription **h**).

The prefabricated slab is made of concrete having a final class of C45/55 for which we have:

$f_{ctd} \quad = \quad 1.87 \text{ N/mm}^2$

(see Table 5.1 in paragraph 5.4.1., the value for Controlled Mass-Production.

It is assumed that the application of overloads with the building in use takes place three months after the laying of the slab and four months after applying prestressing at manufacturer's works.

This gives us for a time t = 4 months the assumption of the value

$\alpha_t \quad = \quad 0.65$ [see Comments, point **h)** above]

Let us consider that on applying prestressing the stress in the strands was

$P_{m,o} \quad = \quad 1250 \text{ N/mm}^2$

while on final loss of prestressing it is

$P_{m,\infty} \quad = \quad 1100 \text{ N/mm}^2$

from previous data we deduce that at time t = 4 months the stress in the strands was

$$P_{m(t)} = P_{m,o} - (P_{m,o} - P_{m,\infty})\, \alpha_t = 1152 \text{ N/mm}^2$$

Spalling stress at time t = 4 months is thus

$$\sigma_{spi,d/(t)} = \gamma_p\, \sigma_{spi}\, P_{m(t)} / P_{m,o} = 1.2 \times 0.815 \times 1152/1250 = 0.90 \text{ N/mm}^2$$

We saw previously that shear at the end section of the slab is:
$V_{sd} = 101.38$ kN.

The calculated shear stress τ_{sd} is

$$\tau_{sd} = V_{sd} / 0.9\, d\, b'_w$$

with

d = 265 mm effective height of the hollow core slab

$b'_w = b_w + n\, b_c\, E_{cm}/E_{cp} = 380 + 820 \text{ x } 30500/35700 = 1080$ mm

from which

$$\tau_{sd} = 101380/(0.9 \times 265 \times 1080) = 0.39 \text{ N/mm}^2$$

Furthermore, since angle $\beta \cong 22.5°$ and $2\,\beta = 45°$ we have:

$$\sin 2\beta = \cos 2\beta = 0.7$$

From this it follows that

$$\sigma_{I,d} = (\sigma_{spi,d(t)} + \tau_{sd} \cos 2\,\beta)/2 + \sqrt{(\tau_{sd} \sin 2\,\beta)^2 + [(\sigma_{spi,\,d(t)} + \tau_{sd} \cos 2\,\beta)/2]^2}$$

$$\sigma_{I,d} = [0.90 + 0.39 \times 0,7]/2 + \sqrt{(0.39 \times 0.7)^2 + [(0.90 + 0.39 \times 0.7)/2]^2}$$

$$\sigma_{I,d} = 0.586 + 0.646 = 1.232 \text{ N/mm}^2$$

This value is much lower than $f_{ctd} = 1.87$ N/mm² and the possibility of using the floor as a clear span is amply verified.

Example 4.2

The previous example is repeated but instead of considering a slipformed slab we now consider an extruded one of the same depth and having the same reinforcement (eight 0.5" strands).

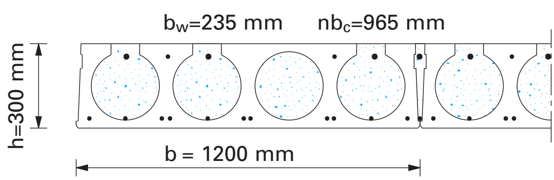


Fig. 4.36 *Verification of an extruded slab h = 300 mm in exactly the same conditions as the previous example with identical reinforcement.*

This slab has four cores and five webs. Thus the webs most stressed by spalling are reinforced by two 0.5" strands and have a maximum width of 49 mm.

Spalling at the time of applying prestressing (formula [**SP**] of paragraph 3.5.2.) is

$\sigma_{spi} = 1.32 \text{ N/mm}^2$

This stress, since it is less than 1.69, satisfies the prescription **g)**:

$\sigma_{spi} < f_{ctk\,0.05} / \gamma_{sp}$

with

$f_{ctk\,0.05} / \gamma_{sp} = 2.03/1.2 = 1.69 \text{ N/mm}^2$ as in the previous example

In order to verify prescription **e)** as well, we must have:

$V_{Rd} \geq V_{sd}$

having as in Example 4.1:

$V_{sd} = 101.38 \text{ kN}$

and

$V_{Rd} = \tau_{Rd}\, k\, (1.2 + 40\, \rho)\;\; n\;\; b_c\; d'$

where

τ_{Rd} = 0.28 N/mm^2

nb_c = 965 mm

d' = 245 mm

k = 1.35

A_{fl} = 1018 mm^2

ρ = $A_{fl} / n\ b_c\ d'$ = 0.0043

we have

V_{Rd} = $0.28 \times 1.35\ (1.2 + 40 \times 0.0043)\ 965 \times 245 =$

= 122613 N = 122 kN

and effectively we have

V_{Rd} > V_{sd} = 101.38 kN

We shall now verify prescription **h)**: it must be:

$\sigma_{I,d} \leq f_{ctd}$ with f_{ctd} = 1.87 N/mm^2

Assuming the same characteristics relating to the time of application of loads as in the previous example, we have:

$P_{m,o}$ = 1250 N/mm^2

$P_{m(t)}$ = 1152 N/mm^2

and consequently we have:

$\sigma_{spid(t)} = \gamma_p\, \sigma_{spi}\, P_{m(t)} / P_{m,o}$

$\sigma_{spid(t)} = 1.2 \times 1.32 \times 1152 / 1250 = 1.46$ N/mm^2

The shear value remains as in the previous example

V_{sd} = 101.38 kN

To calculate the value of shear stress τ_{sd} we consider that

d = 265 mm

$b'_w = b_w + n\ b_c\ E_{cm}/E_{pm} = 235 + 965 \times 30500 / 35700 = 1059$ mm

from which

$\tau_{sd} = V_{sd} / 0.9\ d'\ b'_w$

$$\tau_{sd} = 101380 / (0.9 \times 265 \times 1059) = 0.40 \text{ N/mm}^2$$

thus

$$\sigma_{I,d} = (\sigma_{spi,d(t)} + \tau_{sd} \cos 2\beta)/2 + \sqrt{(\tau_{sd} \sin 2\beta)^2 + [(\sigma_{spi,d(t)} + \tau_{sd} \cos 2\beta)/2]^2}$$

$$\sigma_{I,d} = [1.46 + 0.4 \times 0.7]/2 + \sqrt{(0.4 \times 0.7)^2 + [(1.46 + 0.4 \times 0.7)/2]^2}$$

$$= 1.78 \text{ N/mm}^2$$

This value is acceptable because $< f_{ctd} = 1.87$ N/mm^2. However, it is at the limit of acceptability in "Controlled Mass-production" while it is not acceptable in "Normal Production" in which the limit is $f_{ctd} = 1.77$ N/mm^2 (see Table 5.1).

Example 4.3.

A beam cast in situ without direct support for a slipformed hollow core floor h = 400 mm with a clear span l = 12.0 m.

The dead weight of the assembled floor is 4.8 kN/m^2

Total overload capacity is 6.0 kN/m^2

In Fig. 4.37 the characteristics of the cross section of the hollow core slab (section of the floor near the ends) are shown.

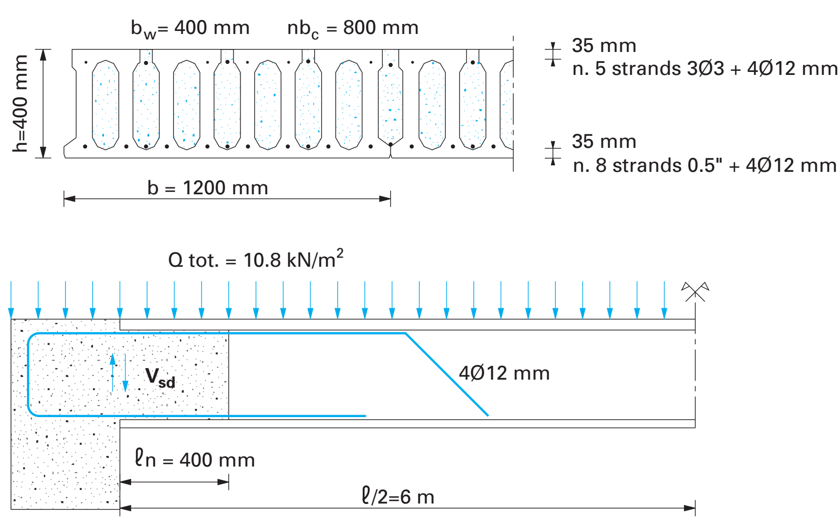


Fig. 4.37 Slipformed hollow core slab h = 40 cm, reinforced with eight 0.5" strands.

As in the previous examples, prescriptions **a), b), c), d)**, are satisfied and condition **f)** is not considered applicable since the slab ends were cut with a diamond disc.

Prescription **e)** is here below verified.

The shear value of a slab 1.20 m wide is

$$V_{sd} = (\gamma_g G + \gamma_Q Q)\ b\ l/2$$
$$V_{sd} = (1.4 \times 4.8 + 1.5 \times 6.0)\ 1.2 \times 12.00/2 = 113.18 \text{ kN}$$

Lacking stirruping, in the attachment section of the nuclei cast with Class C 25/30 concrete it must be

$$V_{Rd} \geq V_{sd}$$

where, as in the previous examples

$$\tau_{Rd} = 0.28 \text{ N/mm}^2$$
$$V_{Rd} = \tau_{Rd}\ k(1.2 + 40\ \rho)\ n\ b_c\ d'$$

The geometric characteristics of the beam-floor attachment section are:

nb_c	=	800 mm	total width of nuclei cast in situ
d'	=	340 mm	effective height of nuclei cast in situ
k	=	1.6 - d'[m] = 1.26	
A_{fl}	=	452 mm^2 (4 ϕ 12 mm)	
ρ	=	$A_{fl} / n\ b_c\ d'$ = 0.00166	

from which we have

$$V_{Rd} = 0.28 \times 1.26\ (1.2 + 40 \times 0.00166)\ 800 \times 340$$
$$= 121530 \text{ N} = 121.53 \text{ kN}$$

We have $V_{Rd} > V_{sd}$ for which there is no need for additional stirruping.

Verification of prescription **g)**

Each web is reinforced with a 0.5" strand placed at 35 mm from the bottom side. By applying the formula [**SP**] in paragraph 3.5.2. we obtain the value of spalling stress σ_{spi} at the time of application of prestressing.

We have:

$\sigma_{spi} = 1.11\ N/mm^2$

and such stress is acceptable, both on applying prestressing with class C 30/37 concrete and $f_{ctk0.05} = 2.03\ N/mm^2$, as in the case of suspension of the floor, since $\sigma_{spi} < f_{ctk0.05} / \gamma_{sp}$

with $\gamma_{sp} = 1.2$

and $f_{ctk0.05} / \gamma_{sp} = 1.69\ N/mm^2$

We shall now verify prescription **h)**

The final class of the concrete in the slab is C 45/55, for which we have

$f_{ctd} = 1.87\ N/mm^2$

We assume that time t of application of the overload is still four months from the prestressing of the slab for which we have

$\alpha_t = 0.65$

Stresses in the strands are the following:

$P_{m,o} = 1250\ N/mm^2$ on application of prestressing

$P_{m,\infty} = 1070\ N/mm^2$ when prestressing losses are finished

on application of overloads at time t = 4 months we have

$P_{m(t)} = P_{m,o} - (P_{m,o} - P_{m,\infty})\ \alpha_t$

$P_{m(t)} = 1250 - 180 \times 0.65 = 1133\ N/mm^2$

Spalling stress at time t = 4 months is thus

$\sigma_{spid(t)} \quad = \quad \gamma_p \, \sigma_{spi} \, P_{m(t)} / P_{m,o}$

$\sigma_{spid(t)} \quad = \quad 1.2 \times 1.11 \times 1133/1250 = 1.21 \text{ N/mm}^2$

If, as seen previously, the value for shear on the slab end section is

$V_{sd} \quad = \quad 113.2 \text{ kN}$

we calculate shear stress

$\tau_{sd} \quad = \quad V_{sd} / 0.9 \; d \; b'_w$

in which we have

$d \quad = \quad 365$ mm effective height of the hollow core slab

$b'_w \quad = \quad b_w + n \, b_c \, E_{cm}/E_{cp}$

$b'_w \quad = \quad 400 + 800 \times 0.82 = 1056$ mm

from which we obtain

$\tau_{sd} \quad = \quad 113{,}200 / (0.9 \times 365 \times 1056) = 0.33 \text{ N/mm}^2$

We thus find the principal tensile stress

$$\sigma_{I,d} = (\sigma_{spi,d(t)} + \tau_{sd} \cos 2\beta)/2 + \sqrt{(\tau_{sd} \sin 2\beta)^2 + [(\sigma_{spi,d(t)} + \tau_{sd} \cos 2\beta)/2]^2}$$

$$\sigma_{I,d} = [1.21 + 0{,}33 \times 0.7]/2 + \sqrt{(0.33 \times 0.7)^2 + [(1.21 + 0.33 \times 0.7)/2]^2}$$

$$\sigma_{I,d} = 0.725 + 0.760 = 1.48 \text{ N/mm}^2$$

The result is $\sigma_{I,d} < f_{ctd} = 1.87 \text{ N/mm}^2$ and therefore the suspension is feasible and perfectly safe.

Example 4.4

We repeat the case proposed in Example 4.3, but this time we replace the slipformed slab with an extruded one having the same depth.

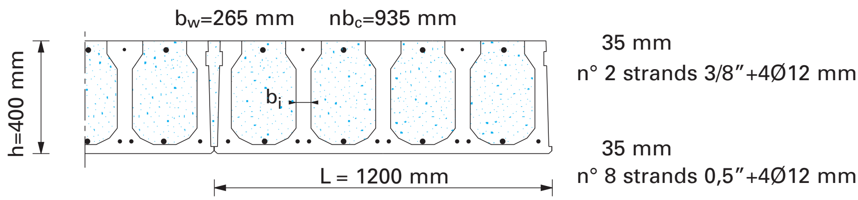


Fig. 4.38 *Extruded hollow core slab, h = 40 cm. in exactly the same conditions as in the previous example and with the same reinforcement.*

This slab has webs among which the most exposed to spalling stress are reinforced with two 0.5" strands.
The formula [**SP**] given in paragraph 3.5.2. supplies initial spalling stress

$\sigma_{spi} \quad = \quad 1.69\ N/mm^2$

This value, being lower than $f_{ctk\ 0.05} = 2.03\ N/mm^2$ and equal to $f_{ctd}/\gamma_{sp} = 1.69\ N/mm^2$ just satisfies the condition foreseen in point **g)** even though it is at the limit of acceptability and thus of safety.

We shall now verify prescription **e)**

$V_{Rd} \quad \geq \quad V_{sd}$

As in the previous example

$V_{sd} \quad = \quad 113.18\ kN$

$\tau_{Rd} \quad = \quad 0.28\ N/mm^2$

$V_{Rd} \quad = \quad \tau_{Rd} k\,(1.2 + 40\rho)\, n\ \ b_c\ d'$

with

nb_c = 935 mm

d' = 340 mm

k = $1.6 - d'\,[m] = 1.26$

A_{fl} = $452\ mm^2$ $(4\ \phi\ 12\ mm)$

ρ = $A_{fl} / n\ b_c\ d' = 0.00142$

from which

V_{Rd} = $0.28 \times 1.26\ (1.2 + 40 \times 0.00142)\ 935 \times 340$

= $140956\ N = 141\ kN$

The result is $V_{Rd} > V_{sd}$, and the condition is satisfied.

Verification of prescription **h)**

For the final prefabricated concrete class C 45/55 it is still

f_{ctd} = $1.87\ N/mm^2$

Keeping the times of overload application and stresses in the prestressing strands the same, we have

$\sigma_{spid(t)}$ = $\gamma_p\,\sigma_{spi}\,P_{m(t)} / P_{m,\,o}$

$\sigma_{spid(t)}$ = $1.2 \times 1.69 \times 1133 / 1250 = 1.84\ N/mm^2$

The shear value remains the same

V_{sd} = $113.2\ N/mm^2$

Shear stress is

τ_{sd} = $V_{sd} / 0.9\ d\ \ b'_w$

with

d = 365 mm

b'_w = $b_w + n\ b_c\,E_{cm} / E_{cp}$

b'_w = $265 + 935 \times 0.82 = 1032\ mm$

from which we obtain

$$\tau_{sd} \quad = \quad 113200 / (0.9 \times 365 \times 1032) = 0.33 \text{ N/mm}^2$$

The principal tensile stress is

$$\sigma_{I,d} \quad = [1.84 + 0.33 \times 0.7] /2 + \sqrt{(0.33 \times 0.7)^2 + [(1.84 + 0.33 \times 0.7) / 2]^2}$$

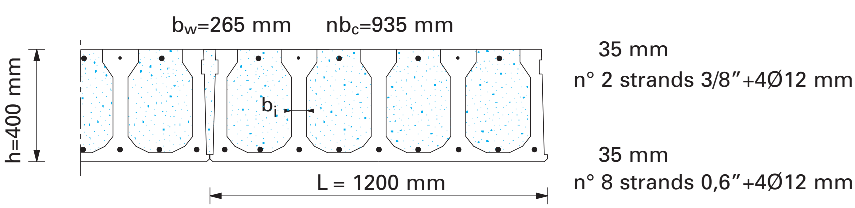
$$= 2.09 \text{ N/mm}^2$$

We find $\sigma_{I,d} = 2.09$ N/mm$^2 > f_{ctd} = 1.87$ N/mm^2 and as a result suspension is not feasible.

Even if we brought the eight prestressing strands to 40 mm from the intrados instead of 35 mm, thus obtaining more favourable spalling amounting to $\sigma_{spi} = 1.54$ N/mm^2 calculation of the principal stress $\sigma_{I,d} = 1.91$ N/mm^2 would still not be acceptable.

Example 4.5

Let us reconsider the previous example with prestressing reinforcement composed of five 0.6" strands placed at 35 mm from the intrados instead of eight 0.5" strands. This reinforcement is 6.5% less than that of the previous exercise, but it is still sufficient to hold the floor with the foreseen span and overload.



The example of calculation 3.3 in paragraph 3.5.2 supplies the initial spalling for a 0.6" strand in the same section

$$\sigma_{spi} \quad = \quad 0.99 \text{ N/mm}^2$$

and therefore the tension conditions in the webs are clearly more favourable.

We verify only prescription **h)**

$\sigma_{spid(t)} = \gamma_p \sigma_{spi} P_{m(t)} / P_{m,o}$

$\sigma_{spid(t)} = 1.2 \times 0.99 \times 1133 / 1250 = 1.08 \ N/mm^2$

$V_{sd} = 113.2 \ N/mm^2$ as in the previous example

$\tau_{sd} = 0.33 \ N/mm^2$ as in the previous example

The principal tensile stress is

$$\sigma_{I,d} = [1.08 + 0.33 \times 0.7]/2 + \sqrt{(0.33 \times 0.7)^2 + [(1.08 + 0.33 \times 0.7)/2]^2}$$

$$= 1.35 \ N/mm^2$$

we have $\sigma_{I,d} = 1.35 \ N/mm^2 < f_{ctd} = 1.87 \ N/mm^2$ thus the value is fully acceptable for suspension.

4.5. The connection between hollow core floor and reinforced concrete loadbearing wall

The hollow core floor is sometimes bound to the multistorey loadbearing parting walls cast in situ by means of a fixed restraint (see Fig. 4.39).
The parting wall must be sufficiently wide since the hollow core slabs must be laid with an overlap of at least 3 cm prior to the in situ casting that forms the structural joint (see paragraph 3.6).

Moreover, the width of the tying beam between the hollow core slab ends shall not be less than 8 cm (Italian Prefab Regulations Art. 2.11.2.b).

The loadbearing strength of the vertical section of the floor in correspondence to the union shall be not less than that of the wall. It is thus necessary to fill the cores carefully to the entire length of the support on the loadbearing parting wall.

Fig. 4.39 illustrates the problems of the floor - parting wall connection and suggests the adoption of solution **c)** whenever possible.

This solution refers back to the ways of suspending the floor as a clear span from beams cast in situ that were discussed in detail in paragraphs 4.4.2, 4.4.3 and 4.4.4.

Much more frequent is the case of a hollow core floor with a fixed end in the top of a reinforced concrete wall.

It is often the case of the cover of a tank or underground storage areas where there is the possibility of very heavy overloads weighing on the floor or when they are used to cover canals or tunnels on which there may be the passage of road traffic.

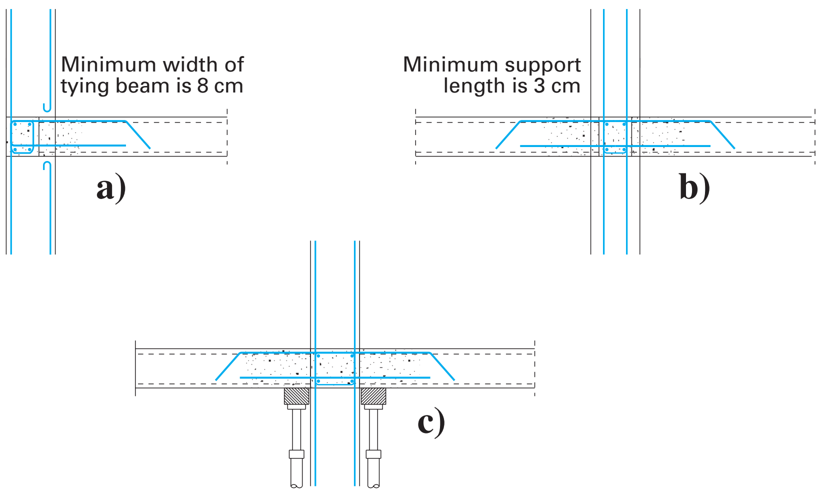


Fig. 4.39 *The connection between hollow core floor and reinforced concrete parting wall.*

a) Suitable support of the floor on the parting wall interrupts the reinforcing running up the loadbearing wall.

b) The minimum allowable support may not interrupt the reinforcing running up the wall, but it is too small during the laying. To be on the safe side it is necessary to put up temporary timbering to support the floor prior to the casting of the tying beam.

c) When overloads are not excessive, it is expedient to suspend the floor as a clear span following support on timbering temporarily.

As illustrated in Fig. 4.40, in all these cases special care must go into the realization of an effective fixed joint by means of very diffuse and suitable reinforcement to transfer traction from the hollow core floor extrados to the taut edge of the bearing wall.

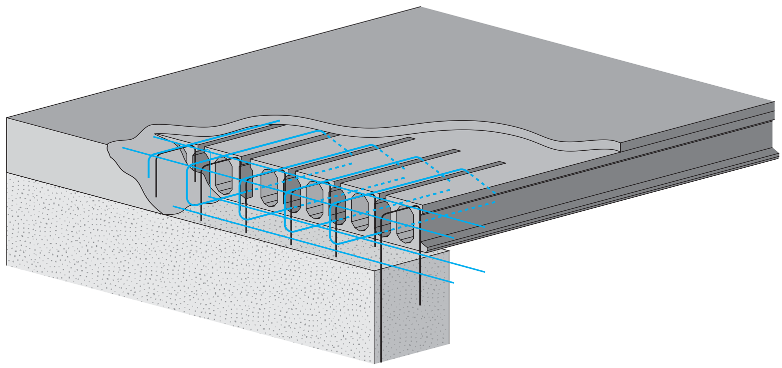


Fig. 4.40 Connection between hollow core floor and retaining wall, with fixed end.

Some loadbearing walls cast in situ or even prefabricated are equipped with a corbel to support the hollow core floor as illustrated in Fig. 4.41.
In such cases the bond between floor and wall is almost always a case of simple support.

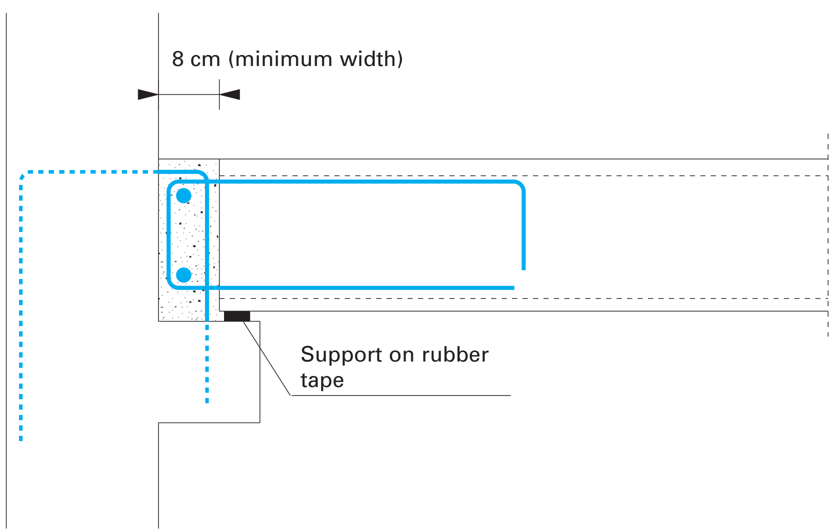


Fig. 4.41 The wall - hollow core floor connection with a bond of simple support.

To assure a metal link between wall and floor it is necessary for a series of stirrups of small diameter to overhang from the supporting corbel enough to connect a narrow, tying beam to which the floor is linked.

4.6. The large holes in hollow core slab floors

In some cases the design of the floor to be laid calls for hollow core slabs with holes larger than the notches discussed in paragraph 2.3.4.

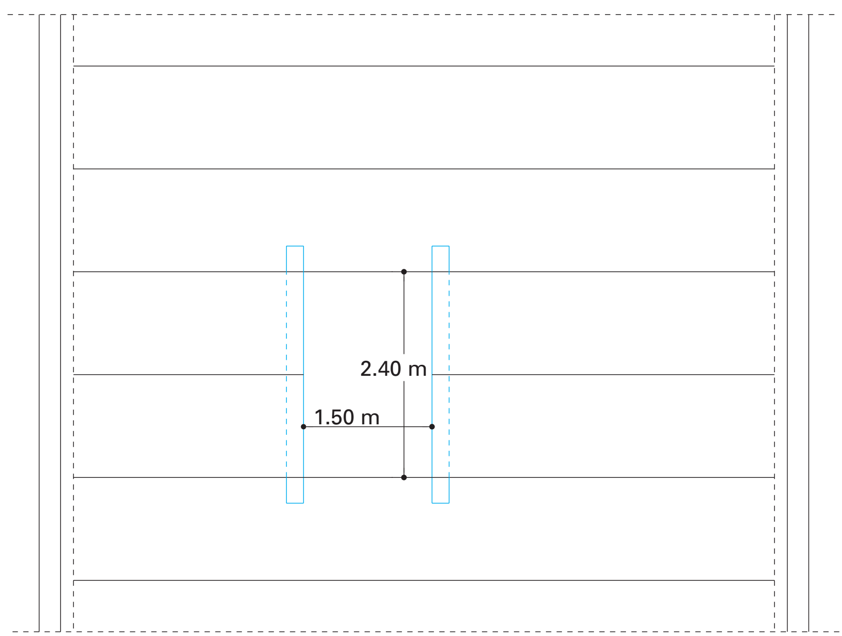


Fig. 4.42 Example of a large hole in a hollow core floor.

Such large holes may be of such a size as to involve the entire width of a slab or that of slabs on either side of it to allow the inclusion of skylights, large vertical equipment, stairs or internal passageways between floors.

In these cases, exemplified in Fig. 4.42, it is essential to design a suitable support for the slabs cut to make room for the hole.

Among the different kinds of support possible, here we focus on two systems applied with a certain frequency. These are shown in Figs. 4.43 and 4.44.

The first system (see Fig. 4.43) consists of a small steel beam, frequently galvanized, verified for transferring to adjacent hollow core slabs the dead weight of the shorter slab or slabs and the overloads that they must bear.

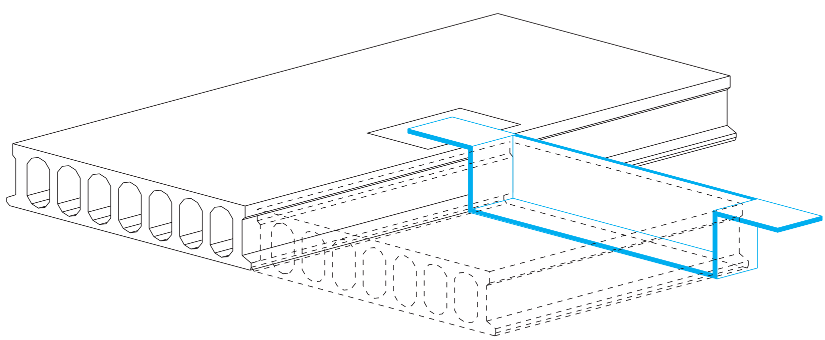


Fig. 4.43 Example of a small steel beam having a span of 120 - 240 cm to support hollow core slabs in the presence of large openings.

Generally speaking, the length of the steel beam is between 120 and 240 cm. The two concentrated loads on the supports of the small beam must obviously be considered in calculations to verify the two hollow core slabs that bear them.

It is best to make sure that the supports of the beam are locked to the supporting hollow core slabs to avoid all possible rotation or accidental shifting of the beam that may compromise the support of the hollow core slab lying on it.

The second system (see Fig. 4.44) is less simple but allows support of more than two slabs in the presence of very large openings.

Essentially, it consists of creating a reinforced concrete beam immediately facing the ends of the hollow core slabs facing the opening.

The longer slabs, to which the beam cast in situ is bound, must be capable of bearing the concentrated load on them.

As can be seen in Fig. 4.44, for the realization of a good connection between the reinforced concrete beam and the hollow core slabs supported as a clear span it is necessary to take into account the ways of suspending discussed in paragraphs 4.4.2, 4.4.3 and 4.4.4.

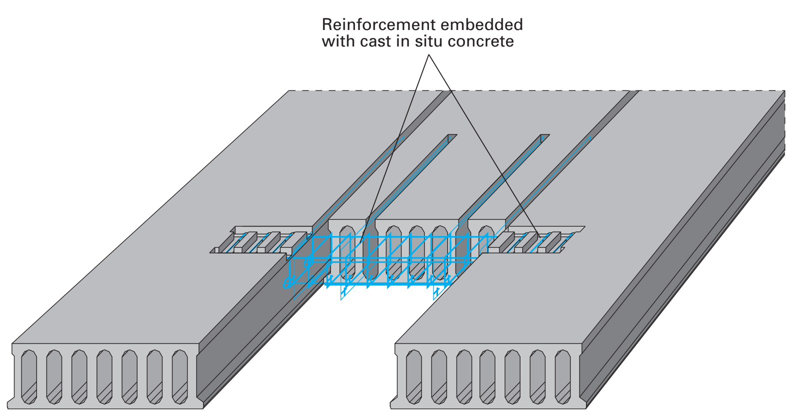


Fig. 4.44 Example of a reinforced concrete beam cast in situ to support hollow core slabs involved in a large opening.

Chapter 5
DESIGN PRINCIPLES

5.1. General considerations

As is the case for all structures, the design of hollow core floors is an operation that takes place in five separate and equally important phases:

a) considerations on materials to be used;
b) analysis of the overall structure and the static and flexural peculiarities of the hollow core floor;
c) preliminary dimensioning and design taking into account all static, flexural and deforming behaviours of the slab, including long-term ones;
d) graphic representation of the structures;
e) calculation with verification of flexure, shear and various stresses.

In this chapter we examine the materials, the criteria for pre-dimensioning and design concerning hollow core floors.

Much space is devoted to the analysis of states causing deformation of slab since it has been found that the importance of this issue is often underestimated by many designers and manufacturers of hollow core slabs. The latter, in order to reduce the weight and cost of slabs tend to supply very thin slabs which are consequently strongly prestressed on the lower edge and subject to concrete tensile stresses on the upper edge. Thus these slabs inevitably present excessive camber, which in time, and varying from case to case, tends to decrease or increase and will create serious problems for the building owner.

Special attention is devoted to slab deflection at the time of final inspection to respond to frequently-expressed perplexities of inspectors on finding elastic sag which is in all cases inexplicably less than expected.

Specific methods for calculating and verifying hollow core floors will be described in a forthcoming ASSAP publication in which theory and practice will be discussed in great detail.

5.2. Properties of materials and partial safety factors

In design and verification calculations at U.L.S. (Ultimate Limit States) it is operationally expedient to supply a synthesis of the properties of materials, such as strengths, and of their partial safety factors. The latter, generically defined with γ_m, become γ_c, γ_s for concrete and steel respectively.

Strength R_d, in investigating the stressing effects of actions, will thus be based on characteristic strengths divided by an appropriate $\gamma_m > 1$ to take into account possible differences between results obtained on test specimens and those obtained with the real material.

In general, therefore, a value $f_d = f_k/\gamma_m$ will represent the calculated strength to be used in the static analysis.

5.2.1. Properties of concrete

In Eurocode 2 ENV 1992-1-1 and pr EN 1992-1, all resistive properties of concrete are correlated to characteristic strength at cylinder compression f_{ck}. The subdivision into strength classes, for example C 20/25 N/mm^2, is therefore on the basis of cylinder/cube strengths. Lacking more accurate direct determinations, Eurocode 2 and the Italian NAD establish the following correlations between concrete resistive properties

f_{ck}	=		characteristic cylinder strength of concrete
$f_{ck\ cube}$	=		characteristic cube strength of concrete
f_{ck}	=	$0.83\ f_{ck\ cube}$	ratio between cylinder and cube characteristic resistance to compression of the concrete

$f_{ctm} = 0.30\ f_{ck}^{2/3}$ mean value of axial tensile strength

$f_{ctk0.05} = 0.7\ f_{ctm}$ lower characteristic axial tensile strength (5% fractile)

$f_{ctk0.95} = 1.3\ f_{ctm}$ upper characteristic axial tensile strength (95% fractile)

$f_{cfm} = 1.2\ f_{ctm}$ mean value of flexural tensile strenght

$E_{cm} = 9500\ (f_{ck} + 8)^{1/3}$ secant modulus of elasticity (ENV 1992-1-1, par. 3.1.2.5.2.)

Conversion for evaluating calculated strengths thus takes place on the basis of the following ratios

$$f_{cd} = 0.83\ \frac{f_{ck\ cube}}{\gamma_c}$$ design value of cylinder compressive strenght

$$f_{ctd} = \frac{f_{ctk\ 0.05}}{\gamma_c}$$ design value of axial tensile strenght

$$f_{cfd} = \frac{f_{cfm}}{\gamma_c}$$ design value of flexural tensile strenght

$$\tau_{Rd} = 0.25\ \frac{f_{ctk\ 0.05}}{\gamma_c}$$ design value of shear strenght

Table 5.1.

Values in N/mm²	γ_c = 1.6 In situ concrete for joins and topping				γ_c = 1.5 Prestressed hollow core slab concrete					γ_c = 1.42 Prestressed hollow core slab concrete in "Controlled Mass-Production"				
Concrete Class	C 16/20	C 20/25	C 25/30	C 30/37	C 30/37	C 35/45	C 40/50	C 45/55	C 50/60	C 30/37	C 35/45	C 40/50	C 45/55	C 50/60
$f_{ck\ cube}$	20	25	31	37	37	43	49	55	61	37	43	49	55	61
f_{ck}	16	20	25	30	30	35	40	45	50	30	35	40	45	50
f_{ctm}	1.90	2.21	2.56	2.90	2.90	3.21	3.51	3.80	4.07	2.90	3.21	3.51	3.80	4.07
$f_{ctk\ 0.05}$	1.33	1.55	1.80	2.03	2.03	2.25	2.46	2.66	2.85	2.03	2.25	2.46	2.66	2.85
$f_{ctk\ 0.95}$	2.48	2.87	3.33	3.77	3.77	4.17	4.56	4.93	5.29	3.77	4.17	4.56	4.93	5.29
f_{cfm}	2.29	2.65	3.08	3.48	3.48	3.85	4.21	4.55	4.89	3.48	3.85	4.21	4.55	4.89
f_{cd}	10.00	12.50	15.63	18.75	20.00	23.33	26.67	30.00	33.33	21.13	24.65	28.17	31.69	35.21
f_{ctd}	0.83	0.97	1.12	1.27	1.35	1.50	1.64	1.77	1.90	1.43	1.58	1.73	1.87	2.01
f_{cfd}	1.43	1.66	1.92	2.17	2.32	2.57	2.81	3.04	3.26	2.45	2.71	2.97	3.21	3.44
τ_{Rd}	0.21	0.24	0.28	0.32	0.34	0.37	0.41	0.44	0.48	0.36	0.40	0.43	0.47	0.50
E_{cm}	27400	29000	30500	32000	32000	33300	34500	35700	36800	32000	33300	34500	35700	36800

The following safety factor values are typic for the Italian Building Standard:

γ_c = 1.6 for concrete with normal reinforcement

γ_c = 1.5 for normal concrete produced in "Controlled Mass-Production" (Italian Prefab Regulations)

γ_c = 1.5 for prestressed concrete

γ_c = 1.42 for prestressed concrete produced in "Controlled Mass-Production”

As concerns prestressed hollow core slab floors and the Italian Building Standard, Table 5.1 can be considered a basic reference.

5.2.2. Steel properties.

For design purposes the following mechanical properties are relevant:

Steel for ordinary reinforcement

- characteristic tensile strength f_{tk}
- characteristic yield strenght f_{yk}
- ductility parameters

$$\text{high} \left\{ \begin{array}{l} \varepsilon_{uk} > 5\,\% \\ (f_t/f_y)_k > 1.08 \end{array} \right.$$

$$\text{normal}\quad\begin{cases}\varepsilon_{uk} > 2.5\% \\ (f_t/f_y)_k > 1.05\end{cases}$$

For design at the Ultimate Limit State (U.L.S.) of cross sections (which in the case of hollow core floors are prevalently those subject to negative moment) according to ENV 1992-1-1 (4.2.2.3.2.) we can assume a stress-deformation diagram of the type illustrated in Fig. 5.2 constructed for a FeB 44 K steel with $\gamma_s = 1.15$.

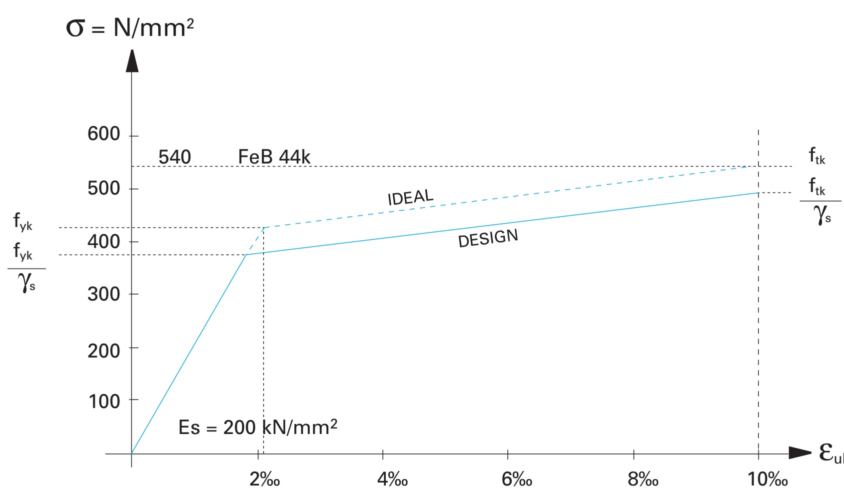


Fig. 5.2 *Design stress-elongation diagram for normal reinforcement steel*

Prestressing steels

- $f_{p0.1k}$ characteristic value of proof-stress at 0.1% of residual deformation

- ε_{uk} characteristic value of uniform elongation corresponding to maximum load

- f_{pk} characteristic tensile strength

The properties discussed above, which can be deduced in terms of value from EN 10138 or ENV 1992-1-1 - 4.2.3.3.3., must be certified by means of documents indicating technical approval.
According to ENV 1992-1-1 (4.2.3.3.3.), for calculation at U.L.S. a stress-deformation diagram of the type illustrated in fig. 5.2 can be assumed for prestressing reinforcement with the upper branch inclined and with steel deformation limited to 10 per thousand beyond decompression.

Concerning the partial safety coefficient for steel, $\gamma_s = 1.15$ was assumed in constructing the diagram.

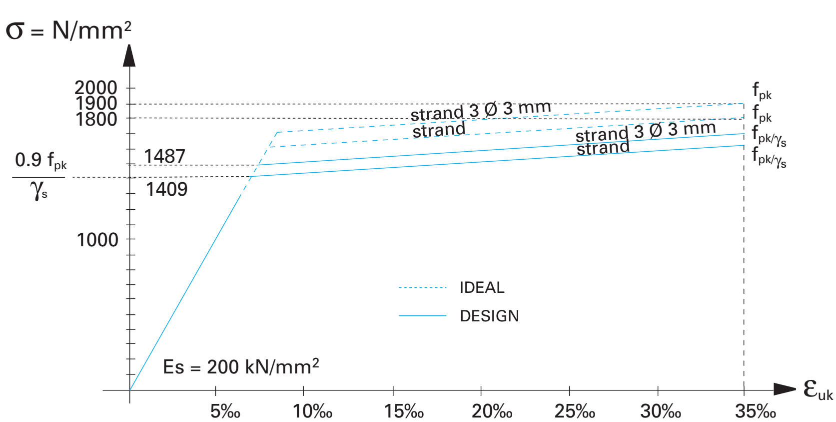


Fig. 5.2 *Stress-elongation diagram of prestressing steel*

5.3. Static and geometric preliminary dimensioning

By preliminary dimensioning is meant the proper, prior determination of floor depth, reinforcement and structural connections as a function of effective design requirements.

These consist of the conservation of long-term structural functionality, the maintaining of the aesthetics of the structure and the safeguarding of the integrity of elements connected to the structure, such as rigid partitions, windows, frames, pavements and so on.

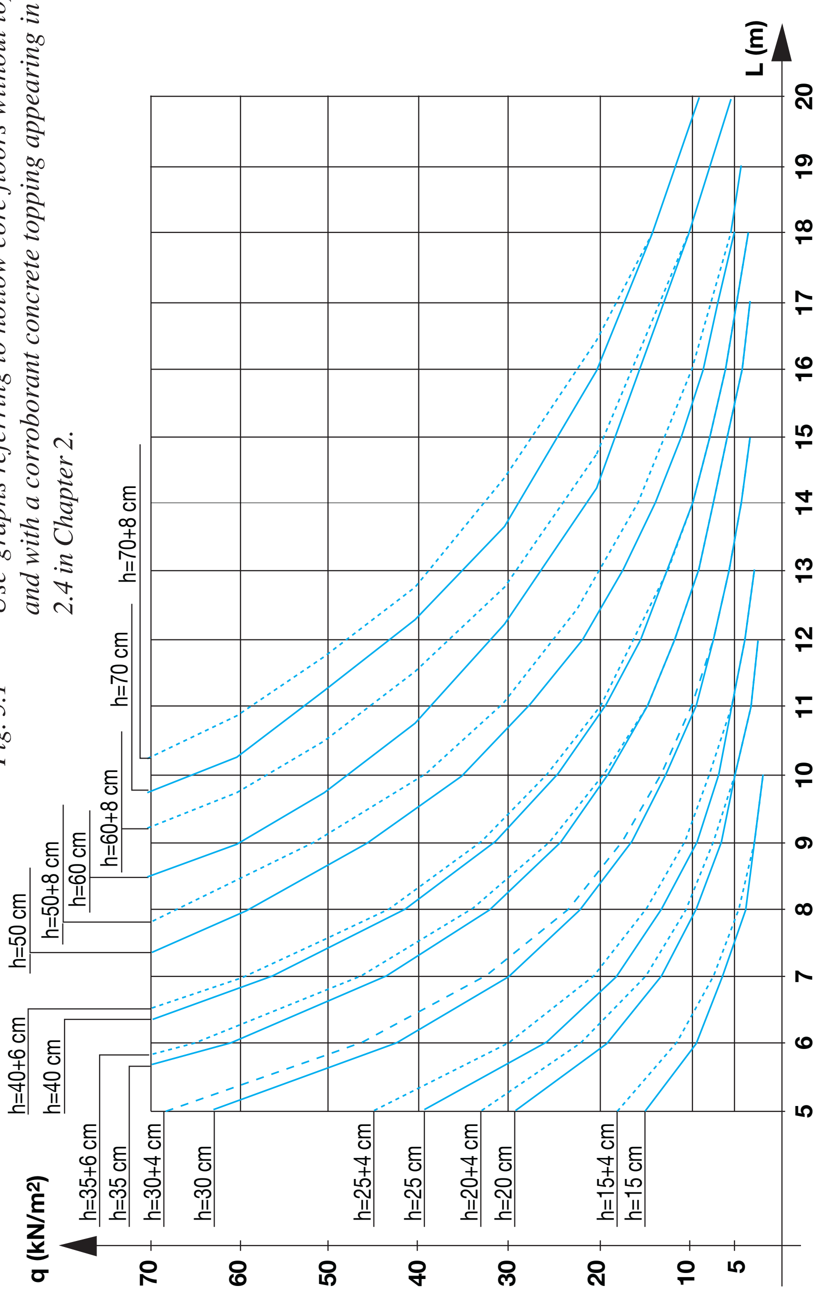
The minimum depth of the floor and concrete topping, when required, is determined as a function of span and restraint conditions (simple support or structural continuity). But the designer must also be careful in evaluating the type of structure, dead weights and overloads, the use to which it is to be put and the limits of deformation as well as environmental considerations and use (fire-resistance), that is, the presence of exceptional overloads and the extent of flexural and shear actions acting on the structures.

Decisions concerning prestressing reinforcement and its concrete cover are directly consequent to these design choices.
Specific design aspects may require different sizing with respect to the preliminary dimensioning rules given below.
Structural detailed calculation, which is required in all cases, shall be carried out to justify the final design dimensioning.

5.3.1. Use-graphs

All manufacturers of hollow core floors advertise the types of slabs they produce through technical literature in which the use-graph is almost always presented.
These graphs show the maximum performance offered by each kind of prestressed slab with the maximum applicable reinforcement in conformity with limits established by the codes.

Fig. 5.1 *Use-graphs referring to hollow core floors without topping and with a corroborant concrete topping appearing in Table 2.4 in Chapter 2.*



Every point on the graph expresses essentially the maximum rated positive bending moment that remains constant as a function of the calculated span and the useful overload.

The use-graphs shown in Fig. 5.1 do not take into account the limits imposed by shear stresses in play nor other factors such as slenderness limits imposed by the codes or by the specific requirements for the connection of the floor without direct support on the beam.

5.3.2. Limits of slenderness

It was observed in paragraph 3.2 above that Art. 7.3.2 of the Italian Building Standard as well as paragraph 2.2.1 of the CNR 10025/98 Instructions fix the criteria for use in determining the minimum depth of hollow core floors.

That is to say, it must be:

- for hollow core floors without a corrobant concrete topping Art. 7.3.2 of Italian Building Standard
  floors with simple support : $\ell_c/h \leq 35$
  floors restrained or in continuity : $\ell_c/h \leq 42$

- for hollow core floors with a corroborant concrete topping with depth **s** (Paragraph 2.2.1 of the CNR 10025/98 Instructions):
  floors with simple support : $\dfrac{\ell_c}{h + s/2} \leq 35$

  floors restrained or in continuity : $\dfrac{\ell_c}{h + s/2} \leq 42$

These slenderness limits are to be found neither in the Eurocode nor in the updated pr EN 1168, but it is advisable to take them into consideration in all cases as good design practice so as to establish the best depth of the floor envisaged by means of the use-graphs and to meet the deflection limits normally imposed.
When theese limits are not met, the calculation of elastic and long-term deformations is recommended.

5.3.3. Analytical method for preliminary dimensioning

What was stated above, taken from Italian Standards, is a sufficiently good approximation of parameters for controlling elastic deformation, represented by the restraint condition, but it does not account for the amount of loads which also quantitatively influence elastic deflections.
Therefore Art. 7.3.2. of Italian Building Standard also proposes verification of the fact that under permanent and variable loads the instantaneous elastic deflection is

$$\upsilon_{is} \leq \ell_c/1000$$

If this expression of instantaneous deflection is connected with the general expression of elastic deflection in the different restraint conditions and with uniformly distributed load:

$$\upsilon_{is} = K \frac{(G_k + Q_{ik})\,\ell_c^4}{E\,I}$$

we obtain an analytical design approach from which we can arrive at inertial moment I of the hollow core floor with reference to all factors in play:

$$\frac{\ell_c}{1000} \geq K \frac{(G_k + Q_{ik})\,\ell_c^4}{E\,I}$$

from which:

$$I \geq \lambda\,(G_k + Q_{ik})\,\ell_c^3$$

since

I $[cm^4]$ = moment of inertia for the width of 1 m of slab floor, which assures compliance with the condition $\upsilon_e \leq \ell_c/1000$ for the loads considered.

K = function of the restraints of the structure equal to

K =	0.0130	for simple support	(5.0/384)
K =	0.0099	for reduced restraint	(3.8/384)
K =	0.0078	for partial fixed end	(3.0/384)

C 45/55 = cylinder and cubic strength of the concrete

E_{cm} = 36 kN/mm^2 (ENV 1992-1-1, par. 3.1.2.5.2.)

λ = multiplying coefficient = K $10^3/E_{cm}$. It equals:

λ =	0.325 for simple support	[when $M = 1/8\ (G_k + Q_{ik})\ \ell_c^2$]
λ =	0.247 for reduced restraint	[when $M = 1/10\ (G_k + Q_{ik})\ \ell_c^2$]
λ =	0.195 for partial fixed end	[when $M = 1/12\ (G_k + Q_{ik})\ \ell_c^2$]

ℓ_c =	calculated span	[m]
G_k =	permanent actions	[daN/m^2]
Q_{ik} =	variable actions	[daN/m^2]

Having found the unitary moment of inertia, which is valid for slab width b = 100 cm, this is compared with the next highest unitary moment tabulated in the literature supplied by the manufacturer, taking into account the real width of the slab to which it refers.
Together with the latter value we will read the depth of the corresponding hollow core floor.

Examples of calculation

The floor is dimensioned with the following data in the examples that follow:

calculated span	ℓ_c	=	12 m
permanent overloads	G_k	=	250 daN/m^2
variable overloads	Q_{ik}	=	400 daN/m^2

Example 5.1:

For simple support we have $\lambda = 0.325$

$I \geq 0.325\ (250 + 400)\ 12^3 = 365{,}040\ cm^4/m$

For hollow core slabs with b = 1.20 m we have

$I' = 365{,}040 \times 1.20 = 438{,}048\ cm^4$

$I^* = 465{,}000\ cm^4$ It is the next highest manufacturer's tabulated value for b = 1.20 m (see Table 2.4 in Chapter 2) to which corresponds the depth of the floor h = 40 cm without topping.

Example 5.2:

For partial fixed end (continuity) we have $\lambda = 0.195$

$I \geq 0.195\ (250 + 400)\ 12^3 = 219{,}024\ cm^4/m$

For a hollow core slab with = 1.20 m we have

$I' = 219{,}024 \times 1.20 = 262{,}829\ cm^4$

$I^* = 315{,}000\ cm^4$ It is the next highest manufacturer's tabulated value for b = 1.20 m (see Table 2.4 in Chapter 2) to which corresponds the depth of the floor h = 35 cm without topping.

5.3.4. Design rules for floors laid in continuity or with fixed ends

Static and geometric pre-dimensioning of a hollow core floor with fixed ends or laid in continuity is greatly facilitated by keeping in mind the suggestions contained in Table 5.2 below. It appears in F.I.B. Bulletin no. 6 and has the advantage of proposing values that are appropriate both in the case of a hollow core slab supported by the bearing structure and those of clear span and thus indirectly supported by the bearing structure.

The values recommended in Table 5.2 are valid for floors with uniformly distributed loads on the order of $4.0 \div 8.0\ kN/m^2$ overall, including both permanent and variable loads, and thus for most common floors.

The ordinary reinforcement for use at the fixed ends on the supports is dimensioned on the basis of the values of shear V [kN] and negative moment M' [kNm] at S.L.S. (Serviceability Limit State).

Table 5.2.

PREDIMENSIONING OF A HOLLOW CORE FLOOR WITH A CONTINUITY RESTRAINT OR FIXED ENDS	FLOOR SUPPORTED BY THE BEARING STRUCTURE	CLEAR SPAN FLOOR WITHOUT DIRECT SUPPORT BY THE BEARING STRUCTURE
– **Slenderness of floor:** span/depth ratio ℓ_c/h	35÷40	30÷35
– **Minimum total thickness of vertical webs** for a slab 120 cm wide Σb_w	No limitation	380÷420 mm
– **Maximum number of 0.5" strands** for each web no.	2-3	1
– **Number of open cores** at slab ends including the longitudinal join, for insertion of continuity reinforcement. for $\ell_c \leq 6.0$ m no. for $6.0 < \ell_c \leq 10.0$ m no. for $10.0 < \ell_c$ no.	 2 - 3 3 3- 4	 3 3 - 4 4
– **Length of upper reinforcement** ℓ_s for negative moment for $\ell_c \leq 6.0$ m for $\ell_c > 6.0$ m	1 rod in the join $\ell_s \approx 1.4$ m other rods in the open cores $\ell_s \approx 1.0$ m. 1 rod in the join $\ell_s = 0.20 \div 0.25\, \ell_c$ other rods in the open cores $\ell_s \approx 1.2 \div 1.5$ m	1 rod in the join $\ell_s \approx 1.4$ m other rods in the open cores $\ell_s \approx 1.0$ m. 1 rod in the join $\ell_s = 0.20 \div 0.25\, \ell_c$ other rods in the open cores $\ell_s \approx 1.2 \div 1.5$ m
– **Upper reinforcement for negative moment** Total section per slab A_s Maximum diameter $Ø_{max}$	 $A_s = 5000\, M'/h$ [mm²] $Ø_{max} \approx 6 + h/25$ [mm]	 $A_s = 5000\, M'/h$ [mm²] $Ø_{max} \approx 6 + h/25$ [mm]
– **Lower reinforcement** Total section per slab A_s Maximum diameter $Ø_{max}$	Normally none (the Italian Building Standard Chapter 7 – prescribes in all cases lower reinforcement suitably anchored capable of supporting tension equal to shear)	 $A_s = 5\, V$ [mm²] $Ø_{max} \approx 4 + h/25$ [mm]
– **Number & length** of cores at slab ends, without reinforcement, to be filled in situ ℓ_n	Normally none	All the rest $\ell_n = 300\text{-}400$ mm

5.3.5. Design of the concrete topping

The reinforced concrete topping is usually not necessary with a hollow core floor in residential buildings and in service buildings having normal loads on the order of 4.0÷8.0 kN/m^2.

In fact, the usual peripheral tie rods and their connections to each single slab give the floor adequate diaphragm behaviour even without the presence of the topping cast in situ.
Moreover, the closed box structure of the slab section and the cylindrical hinge conformation of the longitudinal join between slab and slab give the floor a noteworthy capacity to distribute concentrated loads even without topping, as illustrated in paragraph 5.4.

In any case, as pointed out in paragraph 3.4 above, the concrete topping improves the overall rigidity of the floor, its load-bearing capacity and, if properly reinforced, its capacity for transverse distribution of concentrated loads.

It also perceptibly increases the plate or diaphragm behaviour of the floors and the division of the buildings into compartments in case of fire. Furthermore, an increase in the capacity to oppose horizontal transverse forces on floors transmitted by earth-retaining walls in the case of underground buildings has been found.

For this reason, a reinforced corroborant topping having the thicknesses indicated in Table 5.3 may be recommended, necessary or even made obligatory in the situations detailed in the same table.

The concrete class with which the topping is cast will be decided on the basis of the degree of exposure to aggressive environments as indicated in Tables 2.2 and 2.3 in Chapter 2.

When a corroborant topping is prescribed, it is necessary to indicate the chosen thickness clearly in project drawings. It shall be the measurable minimum, usually at midpoint of the floor (where camber due to prestressing is greatest). It must be kept in mind that in the zones of support the thickness of the topping will be greater because it will be equal to the prescribed minimum thickness increased by the amount of camber (see Fig. 5.14 below).

When a reinforced corroborant topping is prescribed, the peripheral tie rods mentioned in paragraph 4.1 can easily be positioned in correspondence to the perimeter of the floor and in the thickness of the topping, even when edge beams are lacking, as shown in Figures 4.3, 4.4 and 4.5 in Chapter 4.

Table 5.3.

THE CORROBORANT TOPPING ON HOLLOW CORE SLABS WITH DEPTH UP TO 40 cm (1)	CONCRETE TOPPING	STANDARD THICKNESS OF TOPPING	STANDARD REINFORCEMENT
Floor with variable overload of 8.0 ÷ 12 kN/m^2 or concentrated > 8 kN	**recommended**	5 ÷ 8 cm	wire reinforcement Ø 5 mesh 15×15 cm
Floor with variable overload of 12 ÷ 20 kN/m^2 or concentrated > 12 kN	**necessary**	6 ÷ 10 cm	wire reinforcement Ø 6 mesh 20×20 cm
Floor with road overload of 1° and 2° Categories	**obligatory**	12 ÷ 20 cm	wire reinforcement Ø 8 mesh 20 cm upper + Ø 12 mesh 20 cm lower
Floor in seismic zone	**necessary**	4 ÷ 5 cm	wire reinforcement Ø 5 mesh 15×15 cm
Floor with fire resistance (2) REI 60 ÷ 120 minutes	**recommended**	4 ÷ 5 cm (2)	wire reinforcement Ø 5 mesh 15×15 cm
Floor with fire resistance (2) REI 180 ÷ 240 minutes	**necessary**	5 ÷ 8 cm (2)	wire reinforcement Ø 5 mesh 15×15 cm
Floor acting as a diaphragm in a prefabricated structure	**recommended**	5 ÷ 8 cm	wire reinforcement Ø 5 mesh 15×15 cm

Note (1) When the hollow core slabs are 50 ÷ 80 cm high, the standard thicknesses of the topping must be increased by a minimum of 3 ÷ 4 cm.

Note (2) According to Italian Regulation UNI 9502, integrity E is assured in case of fire when a topping reinforced with at least a wire Ø 5 mm, 20 × 20 cm mesh resistance welded having a minimum thickness of s = 4 cm: up to 60 minutes and s= 5 cm: for 90 minutes and more.

5.4. Transverse load distribution

In Chapter 3 (paragraph 3.3) we discussed the importance of the longitudinal join between slab and slab for the good static functioning of the floor and especially for the lateral transfer of vertical forces.

The characteristic shape of this longitudinal join can function as a cylindrical hinge and thus favour the transverse distribution of concentrated loads, but only when lateral shifting between slab and slab is avoided.

If not, there can be no load redistribution and each slab must be dimensioned to support 100% of the load applied.

It is thus necessary to be particularly careful in designing floor perimeter tie rods or the consistency of the structure surrounding the floor.

Another reason for strengthening the tie rods is the fact that the pure cylindrical hinge can be theorized only in presence of very thin hollow core slabs.

In reality, the significant depth of hollow core slabs normally used today causes all transverse deflection upwards or downwards of a longitudinal join subjected to the vertical action of a concentrated load to be strongly opposed by the horizontal reactions generated between the upper or lower edges of the two adjacent slabs.

By opposing one another, the longitudinal edges of the slabs hinder the free deflection of the join loaded vertically, especially when lateral shifting is inhibited.

Such transverse deflections almost inhibited by the large depth of the slabs explain the typical plate-like behaviour of hollow core floors in service, which always comes as a pleasant surprise to inspectors when examining the effective deformations of the floor under test loads.

In all cases they find these deformations halved compared to preliminary theoretical calculations.

This favourable plate effect is still little-known since it is not easy to verify through laboratory tests, while the study by means of calculated modelling at finited elements is less problematic.

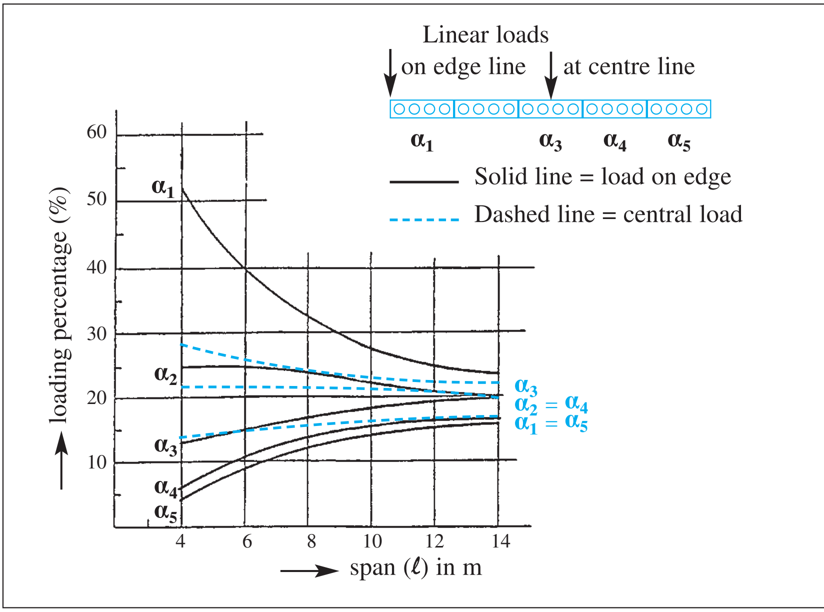


Fig. 5.4 Transverse distribution factors for linear loads

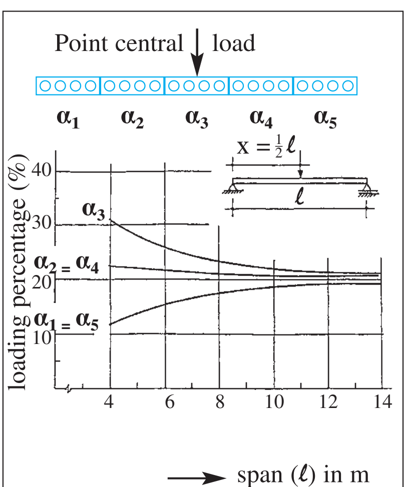


Fig. 5.5. Transverse distribution factors of point loads "at central line"

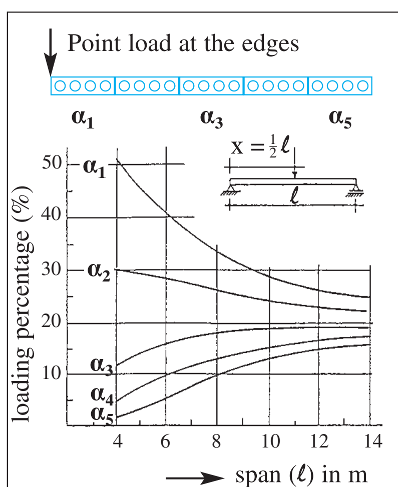


Fig. 5.6. Transverse distribution factors of point loads "at the edges"

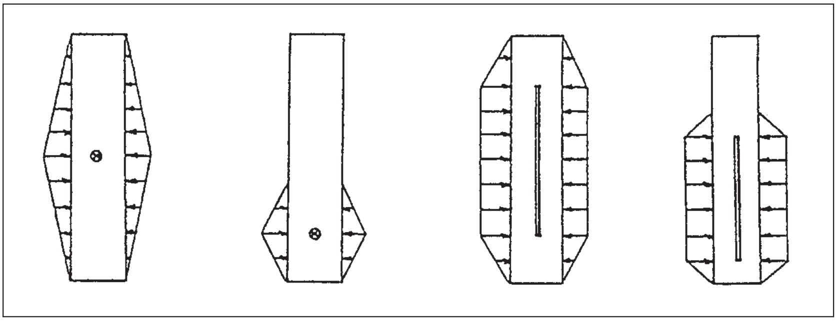


Fig. 5.7. Distribution of shear forces in the longitudinal join

a) Point load at the midpoint of the slab *b) Point load between support and midpoint* *c) Linear load at the midpoint of the slab* *d) Linear load not at the midpoint of the slab*

The European Standard EN 1168, as well as CNR 10025/98 Instructions, supply the graphs shown above (see Figs. 5.4, 5.5, 5.6 and 5.7) by means of which it is possible to evaluate the percentages of load that are transversely distributed, taking into account the following explanations.

a) Figs 5.4, 5.5 and 5.6 show the percentages of load on each hollow core slab in presence of concentrated loads on the floor in the "central" or "edge" position and are valid for hollow core slabs having width b = 1200 mm. A load is considered "central" when it is at least 3 m from the free edge. In intermediate cases it is necessary to interpolate between central and edge load values.

b) The linear loads considered in Fig. 5.4 must be longer than $\ell_c/2$. If they are shorter, they are to be considered linear loads if the centre of the load is at the midpoint of the floor span. Otherwise they are to be considered concentrated loads at the centre of the load if the centre is not at the midpoint of the floor span.

c) In Figs 5.5 and 5.6 we find the percentage of load for "central" and "edge" concentrated loads weighing on the midpoint of the floor ($\ell_c/x = 2$). For loads near the support ($\ell_c/x \geq 20$) the percentage relative to the slab under the load is 100%, while it is 0% for the one adjacent to it. For intermediate ℓ_c/x values between 2 and 20, the percentage can be found by linear interpolation.

d) At the ULS of floors without reinforced concrete topping the percentage determined using the graphs concerning the slab under the load is to be multiplied by the coefficient $\gamma_M = 1.25$. The sum of the percentages supported by the adjacent slabs may be reduced by the same amount, in proportion to the percentages.

e) The shear forces acting within the longitudinal join can be found from the load percentages and are to be considered distributed linearly. For concentrated loads and linear loads to be considered as concentrated, in accordance with point **b)** above, the effective length of the join to be considered for the transmission of shear forces is equal to twice the distance to the centre of the load from the nearest support (see Fig. 5.7).

f) On the basis of the load percentage found in the graphs, it is possible to determine the torsional moments for each slab. When lateral shifting of slabs is inhibited, the calculated torsional moments can be reduced by 50% for reasons given concerning the plate effect.

g) In any case, linear loads (partition walls for example) parallel to the floor frame and not amounting to more than 5 kN/m can be calculated as loads distributed uniformly over a width equal to one fourth of the span measured on both sides of the load, or distributed over the available strip if the load is less than $1/4\,\ell_c$ from the free edge.

5.5. Design of fire resistance

5.5.1. General considerations and calculations

Fire resistance **REI** XY of a floor defines its capacity, at the Ultimate Limit State and for a normalized fire, to maintain for time XY load **"R"** in its own weight + permanent loads + a given amount for variable loads, integrity **"E"** to prevent burning gas escape to upper floor, thermal insulation **"I"** so that the fire does not spread to the unexposed face, which must not reach a temperature of more than 140°C above the initial temperature.

Structural fire resistance **R**, integrity **E** and thermal insulation **I** of a hollow core slab can be determined experimentally in official tests performed by placing a pair of hollow core slabs under loads causing deflecting moments and shear stresses greater than or equal to design values in special ovens complying with regulations.

The sizes of such ovens are in any case too small to reliably represent the real behaviour of a floor under a load and exposed to fire.

These ovens are, however, quite useful in the experimental and certified determination of temperatures recorded progressively at all critical points of one or more hollow core slabs submitted to fire for the necessary time.
This recording procedure is known as **thermal mapping**.

Besides the experimental method described above, there are also tables for the determination of fire resistance **R** and thermal insulation **I** by means of limit values for the depth of the floor and the concrete cover of prestressing steels.

Such limit values tend to ensure the resistance class of a floor in a rather simplified manner.

Smoke seal **E** is also more simply guaranteed by adopting certain building techniques such as the casting of a reinforced corroborant topping or the proper sealing of longitudinal joins.

When the tabulated method worked out for solid slabs is applied to the specific morphology of the hollow core floor to determine fire resistance **R**, it is not considered officially acceptable, either in Italy or in the rest of Europe. Thus for this determination it is necessary to resort to different, more or less simplified, methods of analytical calculation.

Such methods of calculation normally follow those applied in verifying the cold section at the Ultimate Limit State (ULS) adopting reduced sections or reduced materials characteristics on the basis of temperatures reached under the fire load.

Thermal mapping of the particular hollow core slab being examined is the ground for all methods of analytical calculation. When a detailed experimental thermal map is unavailable, it must be constructed analytically with the aid of software that elaborates the specific formulations dictated by regulations.

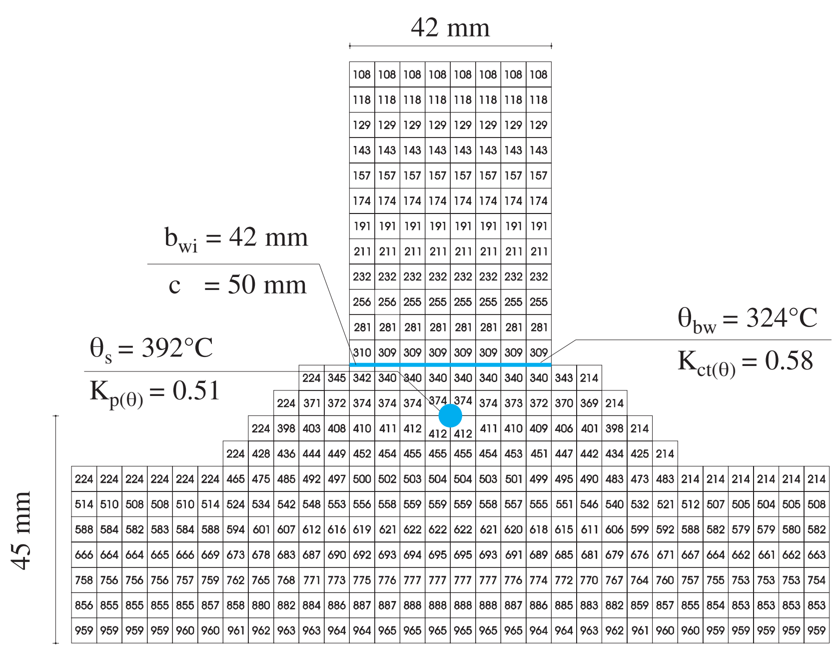


Fig. 5.8. Example of thermal mapping at 120 minutes of a hollow core slab h = 240 mm with strand 45 mm from the intrados and with 2% humidity

In Italy, for the analytical determination of fire resistance **R**, only the UNI 9502 standard dated May 2001, which is closely correlated with the European standard pr EN 1992-1-2, is accepted.

For verification of the ULS for fire, in accordance with the UNI 9205 standard, the partial safety coefficients are:

for permanent loads	$\gamma_G = 1.0$	
for variable loads	$\psi = 0.5$	(wind, snow, homes, offices, attics)
	$\psi = 0.7$	(shops, crowded halls, car parks)
	$\psi = 0.8$	(balconies, stairs, chimneys)
	$\psi = 0.9$	(archives, libraries, warehouses)
for concrete	$\gamma_c = 1.2$	
for steel	$\gamma_s = 1.0$	

The characteristics of materials at high temperatures at the Fire Limit State are given in the standards mentioned above; as concerns steel tensile strength and concrete tensile and compression strength, calculated values decrease with an increase in temperature, as can be seen in Figs. 5.9 and 5.10.

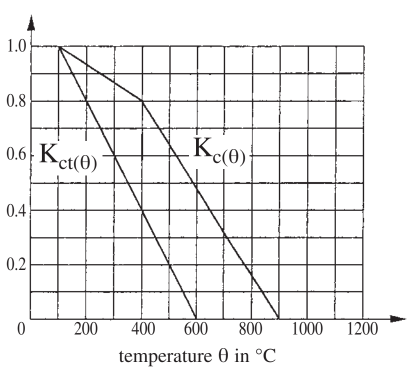


Fig. 5.9. Variation in the characteristic strength of concrete with variations in temperature

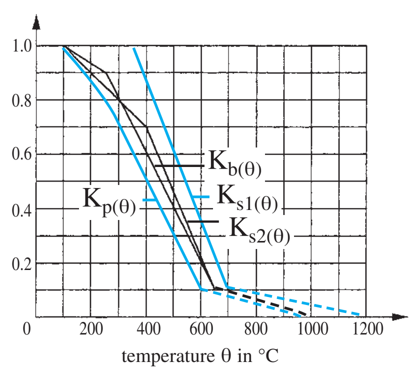


Fig. 5.10 Variation in the characteristic strength of prestressing steel with increases in temperature

5.5.2. The "tabulated data" method

The provisory predimensioning of a hollow core floor for a certain resistance to fire can be defined by using the following table taken from the European Product Standard pr EN 1168, in which the minimum depths of the hollow core slab and the cover of prestressing reinforcement are prescribed.

Since the table is taken from a standard that is not yet official, it is clear that Table 5.4 below must be used exclusively for provisory predimensioning, leaving to analytical verification the task of better defining the design of the floor's fire resistance.

Table 5.4.

HOLLOW CORE FLOOR FIRE RESISTANCE						
Guaranteed fire resistance	[minutes]	R30	R60	R90	R120	R180
Minimum depth of floor	[mm]	100	120	140	160	200
Nominal distance of strand axis from surface exposed to fire	[mm]	20	30	40	50	65

Mortar, plaster, insulating coats and other lining materials can be taken into consideration in addition to concrete covering as effective protection of the reinforcement, according to the indications given in regulations.

5.5.3. Analytical methods

On the basis of specific thermal mapping of the sections of the hollow core floor, whether found experimentally or elaborated with appropriate programs with the introduction of the parameters and characteristics of materials in accordance with reference standards UNI 9502 and pr. EN 1992-1-2, it is possible to calculate the U.L.S. of the section for shear and moment on the basis of the effective strengths of the concrete and steel, using the simplified method of reduced sections, and introducing the partial safety coefficients called for in the regulations.

For determination of the thermal map under a fire load, Standard UNI 9502 hypothesizes a humidity content of 50 kg/m^3 of concrete (2% by weight). Figure 5.11 shows the temperature trend as a function of distance from the intrados for different exposure times.

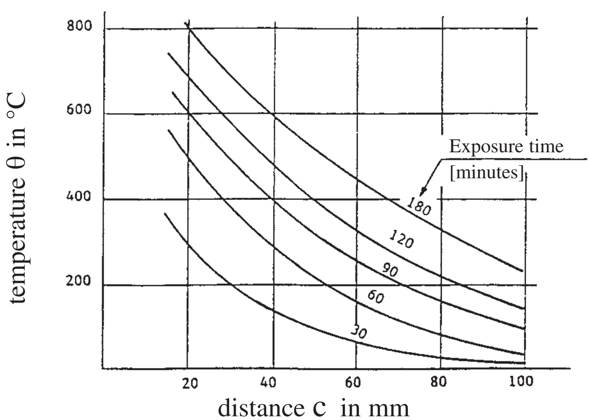


Fig. 5.11 Mean temperature values in the webs of hollow core slabs $h = 200 \div 400$ mm as a function of distance from the intrados

By introducing the appropriate reduced sections of concrete and steel, on the basis of the expected temperature for the class of design exposure and the distance from the surface exposed to fire, it is possible to proceed with calculation of the ultimate resistance capacity for moment M_{Rd} and shear V_{Rd} and compare it with acting stresses M_{Sd} and V_{Sd}, as in the "cold" calculation.

In calculating resistance to shear it is necessary to take into account the reduced value $(k_{ct(\theta)} \times f_{ctk})$ of tensile strength of the prefabricated concrete, at temperature $\boldsymbol{\theta}$ of the web in the zone of attachment to the lower surface so that the value $\Sigma b_w \times k_{ct(\theta)}$ is minimum.

It is necessary to pay attention that distance **d** between reinforcing bar and compressed flange corresponds to the distance between the extrados flange and the lower shear reinforcement in the case of simply supported floors, or to the distance between reinforcement for negative moment and the intrados flange in the case of floors with fixed end at the support.

In the case of floors in continuity, or even with partial fixed end at the support, it is necessary to check the section at support for negative moment at the U.L.S. and for the corresponding shear value at the U.L.S., taking into account the tension bar at the extrados and the compressed flange at the intrados, reduced due to exposure to fire and also subject to prestressing.

It is well to keep in mind that high negative moments in presence of strong prestressing may sharply reduce the resistance to fire of hollow core floors, thus leading to premature failure due to compression of the concrete of the intrados exposed to fire.

Example of calculation 5.3

Verification of a hollow core floor, h = 240 mm, for a car park, with a reinforced concrete corroborant topping having thickness **s** = 6 cm with surface finishing of 2 cm for the formation of the pavement.

Calculated span	8.40 m	(with simple support)
Dead load (h = 240 + 60 mm)	5.00 kN / m^2	
Permanent overloads	0.50 kN / m^2	
Variable overloads	3.50 kN / m^2	
Fire resistance	REI 120’	

Figure 5.12 shows the characteristics of the section and the reinforcement of the hollow core slab:

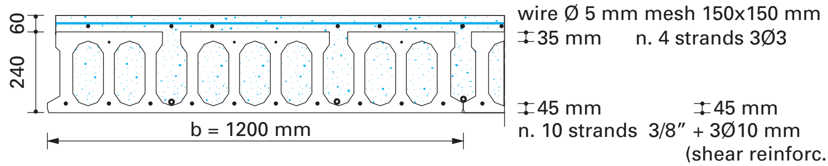


Fig. 5.12 Cross section at the support of hollow core floor h = 240 mm with concrete topping 60 mm thick

For fire verification up to 120 minutes, reference is made to the map in Fig. 5.8 and the safety coefficients for verification under fire are in compliance with Standard 9502/2001.

strand temperature	$\theta_p = 392°C$	from which	$K_p = 0.51$
web temperature	$\theta_{bw} = 324°C$	from which	$K_{ct} = 0.58$
$\psi_{acc} = 0.7$ (car park)	$\gamma_g = 1$	$\gamma_c = 1.2$	$\gamma_s = 1.0$

Therefore, to verify maximum positive moment at the centre line and shear at the support at U.L.S. for fire, the following design values are calculated:

$$M_{sd\ 120'} = (\Sigma G_i\ \gamma_G + Q\ \psi) \times 1.2 \times \ell_c^2 / 8 = 84.14 \text{ kNm (per slab)}$$

$$V_{sd\ 120'} = (\Sigma G_i\ \gamma_G + Q\ \psi) \times 1.2 \times \ell_c / 2 = 37.55 \text{ kN (per slab)}$$

$A_p = 10 \times 52 \text{ mm}^2$ (corresponding to 10 strands 3/8")

Since $K_p = 0.51$ is the area of efficacy of the lower prestressing reinforcement, in case of fire we have:

$10 \times 52 \times K_p = 265.2 \text{ mm}^2$ (corresponding to reinforcement reduced from 10 to 5.1 strands 3/8").

Going back to the usual calculation of failure with this reduced reinforcement, the moment of resistance is calculated:

$M_{rd\ 120'} = 104.34$ kNm

Since

$M_{rd\ 120'} = 104.34 > M_{sd\ 120'} = 84.14$ kNm

the floor is verified for the positive moment after 120 minutes in case of fire.

For verification of shear at the support after 120', the formula must be applied at the Limit States

$$V_{rd} = \left[0.25 \frac{f_{ctk,\theta}}{\gamma_c} k\,(1.2 + 40\,\rho) + 0.15\ \sigma_{cpm}\right] \Sigma b_w\ d$$

since

$f_{ctk,\theta} = f_{ctk0.05} \times k_{ct(\theta)} = 2.66 \times 0.58 = 1.54$ N/mm^2

$\gamma_c = 1.2$

$d = 300 - 45 = 255$ mm (distance between extrados flange and lower shear reinforcement bar)

$k = 1.6 - d\,[m] = 1.6 - 0.255 = 1.345$

$\Sigma_{bw} = 10 \times 42 = 420$ mm (in this example exceptionally we may neglect the thicknesses of the two open cores and the join at the support)

$\sigma_{cpm} = 0$ (in favour of safety)

$\theta_s = 392°C$ (temperature of the lower shear reinforcement bar)

$K_{s\,(\theta)} = 0.89$ (efficacy factor for reinforcing bar at 392°C)

$A_s = (3\ Ø\ 10\ mm) \times K_{s\,(\theta)} = 236 \times 0.89 = 210$ mm^2

$\rho = A_s / \Sigma_{bw} \times d = 210 / 420 \times 255 = 0.00196$

$V_{rd,120'} = 59.52 > V_{sd,120'} = 37.55$ kN

The floor is also checked for shear and is therefore verified for fire resistance R 120 minutes.

Since a reinforced topping 6 cm thick is called for, verification of integrity criterion **E** can be considered satisfied (see UNI 9502, point 7.2.2.).

Based on thermal mapping, the temperature of the extrados is found to be even lower than 50°C and therefore verification of the insulation criterion **I** can be considered satisfied (also on the basis of Table 5 of UNI 9502, point 7.2.1.).
Therefore the manufacturer can declare that the floor in question has been verified for fire resistance **REI 120’**.

5.6. Diaphragm behaviour

As is required for all kinds of floor system, floors composed of hollow core slabs can also be called upon to assume diaphragm behaviour for the transfer of horizontal forces (wind, earthquakes and so on) to the vertical windbracing elements present in the structure.
This kind of behaviour is guaranteed when the floor possesses sufficient rigidity in its own plane to conserve intact its original geometry up to the U.L.S..

This requires verification of precise applicative conditions:

a) Verification of the entire floor is performed taking into account realistic hypotheses concerning the deformability of windbraces, prefabricated elements and ties.

b) Each slab possesses connections and the entire floor is provided with a tying system such as to guarantee the transmission of horizontal forces performing as an arch-tie mechanism.

c) The complex of ties is capable of supporting all traction forces generated by the in plane actions (flexures, shear and traction). As concerns the amount of such ties, reference is made to the caption of Fig. 4.1 in paragraph 4.1.

5.6.1. Model for diaphragm calculation

The fundamental model for diaphragm calculation is that of the deep beam illustrated in Fig. 5.13, comparable to an arch and tie structure for which it is admitted that the compression trajectories tend to shift towards the lower supports, while those of traction concentrate in the lower part.
The diffusion of transverse compressions along the longitudinal joins improves their capacity to resist longitudinal shear forces.

For a simplified evaluation of the amount of stresses in play, we can place (with reference to Fig. 5.13):

$$M_{max} = q_w \ell^2/8$$
$$V_{max} = q_w \ell/2$$
$$N_{max} = M/H$$

having placed $H = 0.8\,\ell_2$ as the inner arm of the forces N and T.

The assembly of hollow core slabs by means of the casting of joins means that the shear parallel to the longitudinal joins is transmitted from one element to another through the concrete cast in situ.
As concerns the trends of the shear diaphragm, the join immediately adjacent to the edge of the floor is the one most subject to shear stresses.

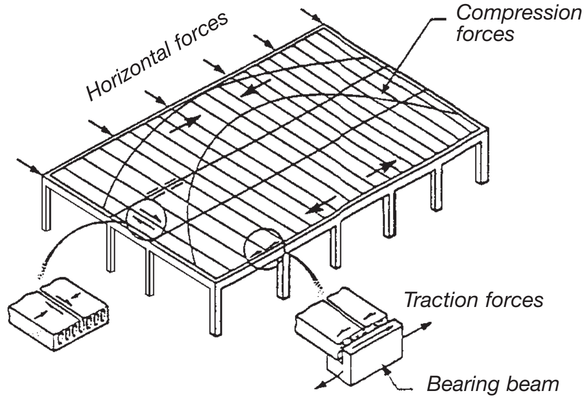
$$V'_{max} = (1 - 2\,x/\ell_2)\; V_{max}$$

Translating the equation into the design shear force we have

$$V_{sd} = \gamma_F\; V'_{max}$$

and the following condition must be satisfied:

$$V_{sd} \leq V_{rd}$$



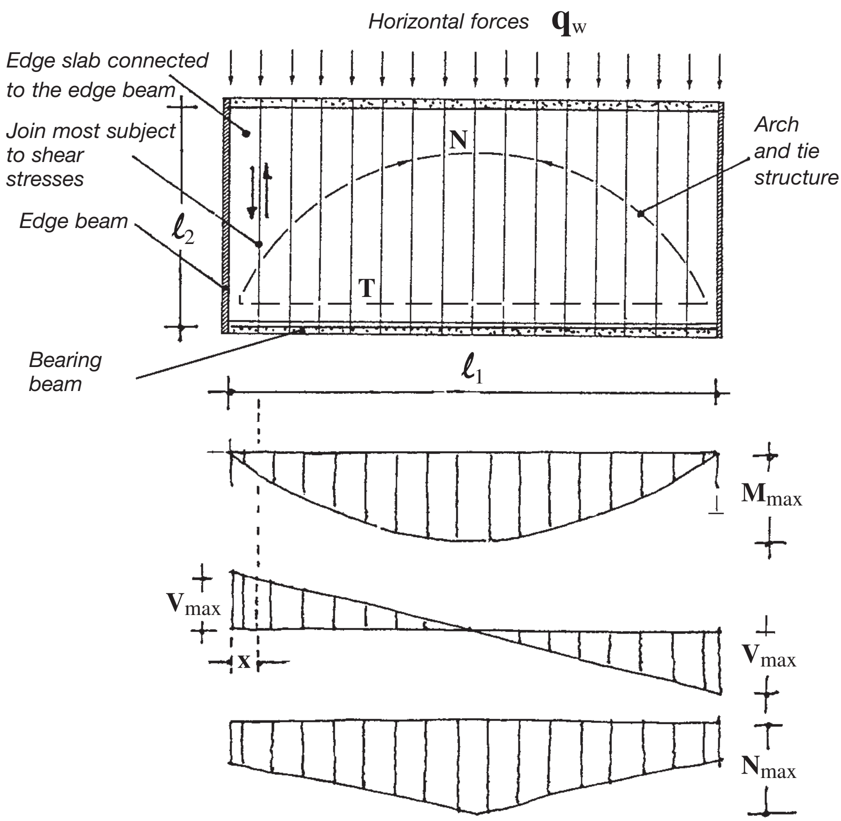


Fig. 5.13 A diaphragm behaviour of the hollow core floor with an illustration of verification methods

The resistance of longitudinal joins on the plane of shear forces for surfaces obtained with vibrofinishing machines or by extrusion without special treatment is limited to

$$V_{Rd} = \tau_{Rdj} \quad A_{cj}$$

where:

τ_{Rdj} = mean resistance to horizontal shear at the ULS which, in longitudinal joins filled with concrete, must be limited to 0.1 N/mm^2 lacking vertical indentation on the two lateral profiles of the slab.

A_{cj} = area of efficacy of concrete in transmitting shear stress.

The diaphragm behaviour of the floor is essential for the stability of a multistorey building subject to horizontal forces, not only in seismic zones but also for wind or other occasional actions or vertical misalignment.

The prescription of a corroborant concrete topping suitably reinforced with a resistance welded net and perimetric tie rods in most cases solves all problems connected with verification of structural stability as for all types of floors.

5.7. Calculation of deformations

Prestressing with the strand barycentre almost always below the kern of inertia generates in hollow core slabs a more or less accentuated upward deflection known as camber.
This initial rise, which is typical of the installed hollow core floor, varies in time because of long-term deformations on the basis of some well-defined parameters, such as floor span and static restraints, under the action of dead weight, permanent loads, long-lasting overloads and prestressing characteristics.

Such long-term deformations are a function of rheological and environmental parameters which by their nature vary with time and are definable only with a certain approximation.
The fluage of the different concretes making up the floor, the modulus of elasticity that varies, the shrinking, relative humidity and ambient temperature, as well as the loss of prestressing are the main factors causing long-term deformations.
The predetermination of initial camber is rather risky in that it depends on many environmental factors, the technologies and the manufacturing process, which all influence the rheological characteristics and the value of the modulus of elasticity. Camber is influenced by the concrete hardening development, which may be natural and thus dependent on environment temperature and humidity.
If hardening is accelerated by heat treatment, camber depends on the thermal cycle which also influences the modulus of elasticity and the shrinkage and viscosity coefficients.
The phase during which the slab is stored in the open prior to installation, with widely varying atmospheric conditions such as sunlight, shade, heat, cold, dryness and dampness has a strong influence on the camber value (and also on the modulus of elasticity and the coefficients of shrinkage and viscosity, and thus on the degree of long-term deformations).
It is therefore inevitable that the camber value, when required and supplied, will have to be considered an average and theoretical value which may admit tolerances of $\pm 0.1 \div 0.2\%$ of the length of the slab. In any case, it is to be considered normal when $5 \div 10\%$ of the slabs present camber values outside the tolerances indicated above.
Any correction of defects in camber between adjacent hollow core slabs in a floor can be made prior to casting in situ by following the instructions given in paragraph 2.3.8.
Elastic deflection during testing by service loads could instead be calculated with greater accuracy since it is a function of the real conditions of restraint and transverse distribution of concentrated loads.
However, this test normally produces values lower than theoretical ones since the plate behaviour almost always gives to the hollow core floor real stiffness greater than theoretical one (see paragraph 5.4).

Only testing a single isostatic slab leads to real elastic deflection values close to theoretical ones, since the error is univocally connected to the effective value of the modulus of elasticity of the concrete, the uncertainty of which is usually plus or minus 10%.

5.7.1. Applications and practical references

Although with the necessary approximations, it is sometimes important to be able to determine the value of initial camber when it is necessary to define the minimum and maximum thicknesses of the topping to be cast in situ on the hollow core slabs.
In fact, the topping increases in thickness starting from the centre line of the floor and going towards the supports. Thus the amount of concrete needed for the in situ casting increases (see Fig. 5.14).
In these cases it is necessary to predetermine the camber so as to be able to define with a certain accuracy the heights between floors when the total height of the building is strictly limited.

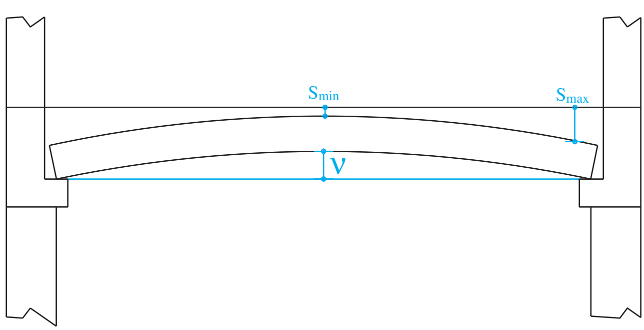


Fig. 5.14 The minimum height between floors must be measured at the supports of the hollow core floor.

Quite often, in order to obtain good flatness of the intrados of a hollow core floor it is indispensable to know beforehand the degree of camber so as to be able to arrive at a correct determination of the levels at which to install rather short hollow core slabs adjacent to much longer ones (see Fig. 5.15).

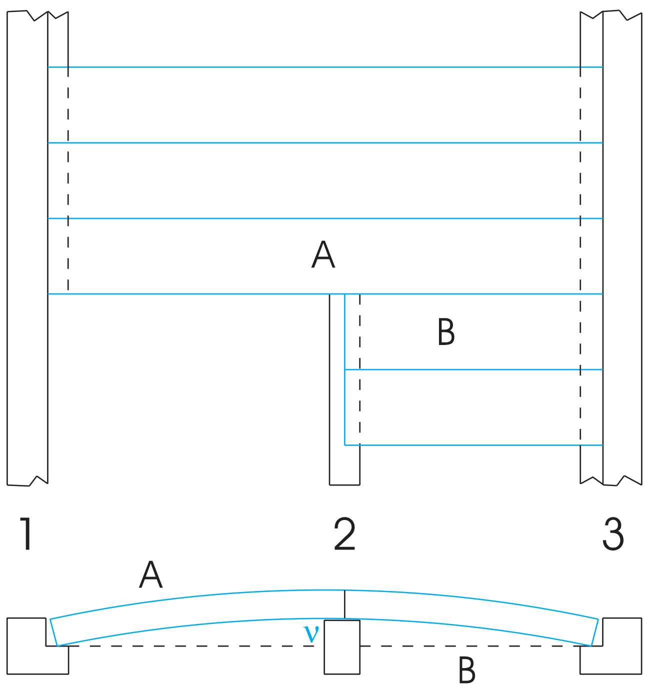


Fig. 5.15 To avoid ugly differences in flatness between adjacent slabs A and B, it is necessary to superelevate the intermediate support - 2 with respect to the supports of extremities - 1 - and - 3 -.

It may be important to evaluate long-term deformations as well, since they modify the initial camber of the hollow core floor subjected to "almost permanent" loads (dead load + permanent loads + long-term variable loads).

Indeed, long-term deformations may lead to serious problems of detachment of pavements and possibly even of the finish on the intrados.

Table 5.5.

TIMES & CHARACTE-RISTICS	DESCRIPTION	TYPICAL MEAN VALUES
t_0	Time elapsed between casting and release of prestressing tendons (that is, the cutting of slabs)	t_0 = 12÷16 h from casting (with accelerated hardening) t_0 = 24÷48 h from casting (with natural hardening)
t_1	Time elapsed between casting of slab and final castings following installation	t_1 = 0,5÷2 months approx.
t_2	Time elapsed between casting of slabs and application of permanent and variable loads	t_2 = 3÷6 months on average
t_∞	Time required for evaluation of long-term phenomena	t_∞ = 5÷10 years (after which viscous phenomena can be considered finished)
v_0	Initial camber at time of storing	from 0 to ℓ/200
v_1	Camber immediately after installation	from 0 to ℓ/300
v_∞	Long-term variation in camber due to the quasi - permanent combination of loads	$\leq \ell_2$/500
v_e	Instantaneous elastic sag due to the rare combination of loads	$\leq \ell_2$/1000
ℓ	Length of slab	
ℓ_p	Span for calculating the effects of prestressing	$\ell_p \approx \ell$ – 1000 [mm]
ℓ_0	Span between supports during storage	$\ell_0 \approx \ell_p$
ℓ_1	Design span during installation (1st phase)	Net span of slab during installation
ℓ_2	Design span after installation (2nd phase)	Design span of the floor after in situ castings
n	Coefficient of static restraint for calculation of deformations	n = 5 for simple support n = 2.5 ÷ 3 for the 1st edge floor n = 1 ÷ 2 for intermediate floors
I, I_1	Moment of inertia of single precast slab, and with corroborant in situ castings after installation	See project data
e, e_1	Prestressing eccentricity in precast slabs, and with corroborant in situ castings after installation	See project data
G, G_1	Dead load of precast slab, and of the floor in presence of corroborant in situ castings	See project data
Q_{perm}	Permanent overloads	See project data
Q_{var}	Variable overloads	See project data
Q_∞	Variable long-term overloads	See project data

continuation of **Table 5.5.**

TIMES & CHARACTE-RISTICS	DESCRIPTION	TYPICAL MEAN VALUES
A_p P_0 P_1 P_∞	Area of prestressing reinforcement Tension of prestressing reinforcement - at the moment of release - at time t_1 - at time t_∞	See design data $P_0 = 1200 \div 1250$ MPa $P_1 = 1150 \div 1200$ MPa $P_\infty = 1050 \div 1150$ MPa
$E_{0\ lower}$ $E_{0\ upper}$	Modulus of elasticity at lower flange of slab at the moment of release Modulus of elasticity at upper flange at the moment of release They depend on the manufacturing system and the hardening process. Temperatures in the slab during heat treatment are approximately 65 ÷ 70°C at the lower flange while they go down by approximately 10 ÷15°C at the upper flange, with a consequent higher value for the modulus of elasticity	Vibrofinishing with natural hardening. $E_{0\ low/up}$ = 20÷22000/19÷21000 MPa Vibrofinishing with accelerated hardening. $E_{0\ low/up}$ = 17÷19000 / 18÷20000 MPa Extrusion with natural hardening. $E_{0\ lower} = E_{0\ upper}$ = 21÷23000 MPa Extrusion with accelerated hardening. $E_{0\ low/up}$ = 18÷20000 / 19÷21000 MPa
E_1, E_2	Modulus of elasticity at time t_1, t_2	$E_1 = E_2 = 30000 \div 33000$ MPa
α (t)	Coefficient of development of viscous effects in time.	$\alpha(t_0) \approx 0.1$ α (15 days) ≈ 0.3 α (1 month) ≈ 0.4 α (2 months) ≈ 0.5 α (3 months) ≈ 0. 6 α (6 months) ≈ 0.7 α (1 year) ≈ 0.8 α (3 years) ≈ 0.9 α (10 years) ≈ 0.95 $\alpha(t_\infty) = 1$
$\varphi(t, t_0)$	Concrete viscosity coefficient between time t and time t_0 $\varphi(t, t_0) = \varphi(t_\infty, t_0)\ [\alpha(t) - \alpha(t_0)]$	$\varphi(t_\infty, t_0) \approx 2.5$ (between 2.2 and 3.0) $\varphi(t_\infty, t_{2\ =\ 6\ months}) \approx 0.3 \times 2.5 \approx 0.75$ $\varphi(t_\infty, t_{1\ =\ 1\ month}) \approx 0.6 \times 2.5 \approx 1.50$ $\varphi(t_{1\ =\ 1\ month}, t_0) \approx 0.3 \times 2.5 \approx 0.75$
$\varepsilon(t, t_0)$	Shrinkage coefficient at time **t** compared to time t = 0 of casting	$\varepsilon(t_\infty, {}_0) \approx 0.0003$ $\varepsilon(t_0, {}_0) \approx 0.5\ \varepsilon(t_\infty, 0)$ $\varepsilon(t_\infty, t_0) \approx 0.5\ \varepsilon(t_\infty, 0)$
ρ	Prefabricated concrete aging coefficient to take into account the increase in modulus of elasticity at infinity.	$\rho = 0.8$

They may modify the slopes in flat covers (see Fig. 5.16) with serious damage caused by the runoff of water or may compromise the stability of internal partition walls.

They may also cause malfunctioning of outside and inside doors and window frames.

The evaluation of such deformations is all the more important the greater the span and slenderness and the ratio between permanent and variable loads. To minimize these problems, the designer should abide by the quite valid criteria shown in paragraph 3.2 when deciding on the depth of the floor.

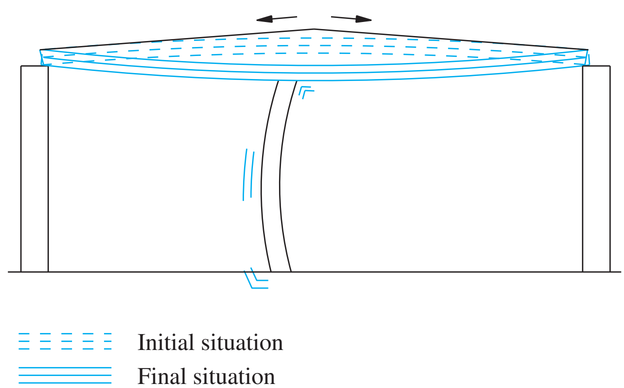


Fig. 5.16 The slopes of the cover must be assured even after the deformations at infinite time. The latter may even compromise the stability of partition walls.

In predetermining short- and long-term deformations as correctly as possible, it is necessary to know and evaluate as accurately as possible the structural, static and rheological characteristics as well as the times of application of loads.

Table 5.5 above shows the mean values of these characteristics to be found for the different typologies and production methods, and which manufacturers can determine as a function of the characteristics of their own production cycles by means of simple experimental findings.

5.7.2. Initial camber ν_0 at time t_0

Having determined the geometric characteristics of the slab, its prestressing and concrete rheological parameters, it is possible to calculate the initial camber on storage through the formula:

$\nu_0 = \nu_{0,p} + \nu_{0,G}$ (see Fig. 5.17)

with:

$$\nu_{0,p} = \frac{A_p P_0 \; e \; \ell_p^2}{8 \, E_{0,\,inf} \times I}$$ the initial camber caused by prestressing

and

$$\nu_{0,G} = \frac{5}{384} \; \frac{G \, \ell^4}{E_{0,\,sup} \times I}$$ sagging caused by the dead load of the slab supported at the extremities

where (see Table 5.5):

A_p	=	area of prestressing reinforcement
P_o	=	stress in prestressing reinforcement at time of release.
e	=	eccentricity of reinforcement compared to the neutral axis of the hollow core slab
ℓ_p	=	span for calculating the effects of prestressing
ℓ	=	length of slab
$E_{o,lower}$	=	modulus of elasticity of concrete at the lower flange of the slab at the time of release
$E_{o,upper}$	=	modulus of elasticity of concrete in the at the upper flange of the slab at the time of release
I	=	moment of inertia of the slab
G	=	dead load

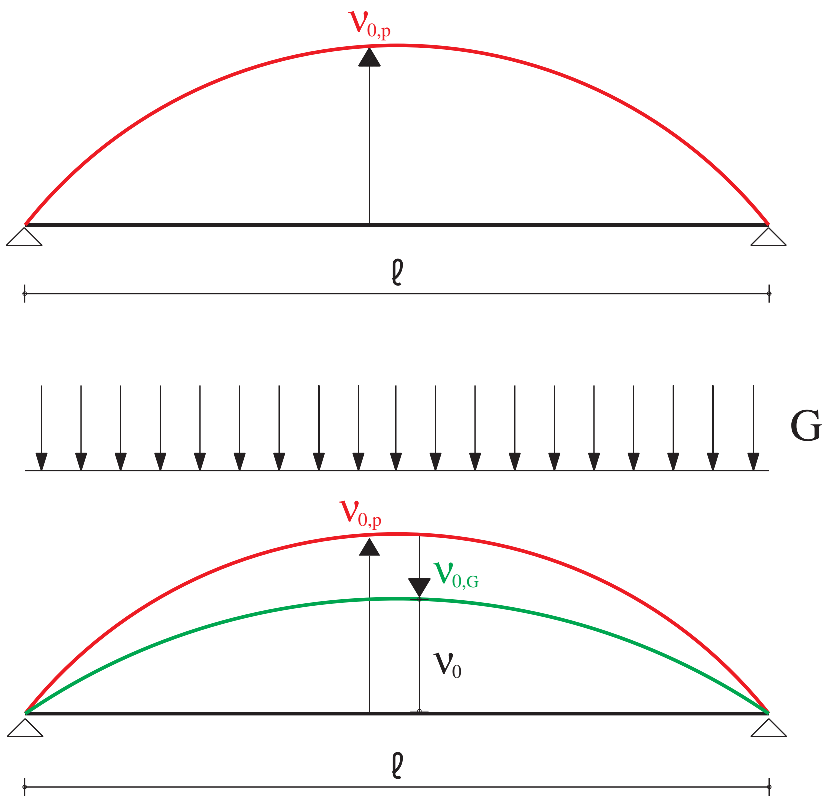


Fig. 5.17 Initial camber ν_0 *on removal from casting bed*

The most usual convention for the sign to adopt for camber attributes a negative value to it when it is above the horizontal.

Therefore the sign of the eccentricity of reinforcement is negative when the barycentre is below the neutral axis of the section and determines an upward deflection.

Every single producer, on the basis of simple statistical findings during production, can experimentally verify the values of $E_{o,lower}$ and $E_{o,upper}$, which vary slightly between summer and winter, check the effective losses of prestressing during the initial phase and can thus find tension P_o with a certain accuracy.

5.7.3. Camber ν_1 after installation at time t_1

At time t_1 the hollow core slab is considered installed, simply supported and finished with additional casts as indicated in plans.
During the storage period initial camber ν_o has changed due to the effect of viscous phenomena depending on the time elapsed between removal from bed and installation, on the environmental conditions to which the slab has been exposed, such as temperature and relative humidity, and on the greater amount of sunlight on one side of the slab compared to the other, on prestressing, dead load and the static conditions of storage.
Camber ν_1 of the installed slab is obtained by adding rise $\nu_{1,p}$ due to prestressing, deflection $\nu_{1,G+G1}$ due to dead load G and the finishing castings G_1 and viscous effects $\nu_{1,\varphi G}$ due to the action of dead load during storage between t_0 and moment t_1 (see Fig. 5.18).

$$\nu_1 = \nu_{1,p} + \nu_{1,G+G1} + \nu_{1,\varphi G}$$

with:

$\nu_{1,p}$ = initial camber due to prestressing $\nu_{0,p}$ modified by viscous effects and reduced by steel relaxation at time t_1

$$\nu_{1,p} = \nu_{o,p}\left[1 + \frac{E_{o,inf}}{(E_{o,inf} + E_1)/2}\varphi(t_1, t_o)\right] + \frac{(P_o - P_1)A_p\, e\, \ell_p^2}{8E_1I}$$

$\nu_{1,G+G1}$ = sag after installation due to dead load and in situ castings

$$\nu_{1,G+G1} = \frac{5}{384}\frac{(G+G_1)\ell_1^{\,4}}{E_1I}$$

$\nu_{1,\varphi G}$ = variation of initial sag due to dead load under viscous effects during storage up to time t_1

$$\nu_{1,\varphi G} = \nu_{0,G}\left[\frac{E_{0\,sup}}{(E_{o\,sup} + E_1)/2}\varphi(t_1, t_o)\right]$$

where A_p, I, G and G_1 refer to the modular width of the hollow core slab

and where the geometric characteristics to be considered refer to the prefabricated slab only, since the additional castings are not yet corroborant in this phase.
The symbols used have the same meanings as in Table 5.5.

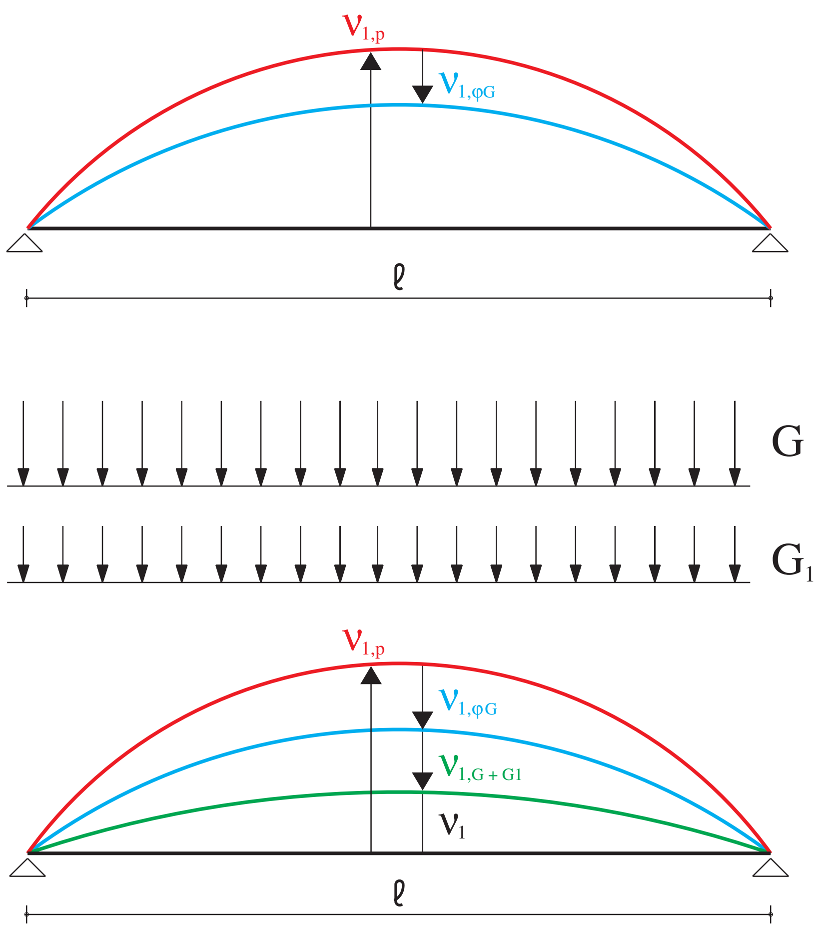


Fig. 5.18 Camber after installation ν_1 *at time* t_1.

5.7.4. In-service and long-term deformations

The typical deformation of the floor in service is the instantaneous elastic deflection ν_e under the action of the rare combination of loads.

This elastic sag ν_e is cited in Italian regulations, which limit its value to a maximum of 1/1000 of the span and is the one that is verifiable during final testing in service.

Elastic sag ν_e caused by the contemporary action of permanent and variable loads (corresponding to the rare combination of loads), can be found in the following way:

$$\nu_e = \frac{n}{384} \frac{Q_{perm} + Q_{var}}{E_1 I_1} \ell_2^{\,4}$$

In the expressions above it is necessary to consider that:

– I_1, Q_{perm} and Q_{var} refer to the modular width of the slab,

– the geometric characteristics refer to the prefabricated slab completed with the corroborant castings,

– coefficient $n = 1 \div 5$ depends on the restraint hypothesized at the ends of the slab,

with the symbols meaning what they mean in Table 5.5.

Long-term deformation ν_∞ corresponds to the incremental sag at infinity of the floor compared to the initial situation in service found previously with camber ν_1.
The variation of deformation is originated by the viscous effects of prestressing between time t_1 and time t_∞, by the viscous effects of the dead load and the additional castings between time t_1 and time t_2 of the application of loads, permanent and long-term variable overloads and by the viscous effects of all loads acting between time t_2 and time t_∞.

Deformation at infinity, evaluated on the basis of the characteristics of the floor with the additional corroborant cast in situ concrete, is also closely correlated with the degree of restraint that is created between slabs and bearing structures or adjacent elements (simple support, half-joint, full joint).

When the limits of the ratio between span and depth of the slab imposed by the Italian Code are deviated from, it is the value of ν_∞ that must be contained within 1/500 of ℓ..

The values of ν_∞ calculated with the formula proposed herein are to be considered mean theoretical values to which it is opportune to apply a tolerance of at least $\pm 0.1 \div 0.2\%$ of ℓ.

In the next Figure 5.19 following notations are to be considerated:

ℓ = span of the installed simply supported hollow core slab

ℓ_2 = calculated span in the 2nd phase as a function of effective restraints created

ν_1 = camber in service at time t_1

$\nu_{\infty,\varphi p}$ = viscous and relaxation effects of prestressing from time t_1 to infinity

$\nu_{2,\varphi G+G_1}$ = viscous effects of slab weight + casts from time t_1 to time t_2 of application of loads.

$\nu_{2,Qperm} + Q_\infty$ = elastic sag for permanent and long-term variable loads

$\nu_{\infty,\varphi G+Q}$ = viscous effects of permanent and long-term variable loads from time t_2 to infinity

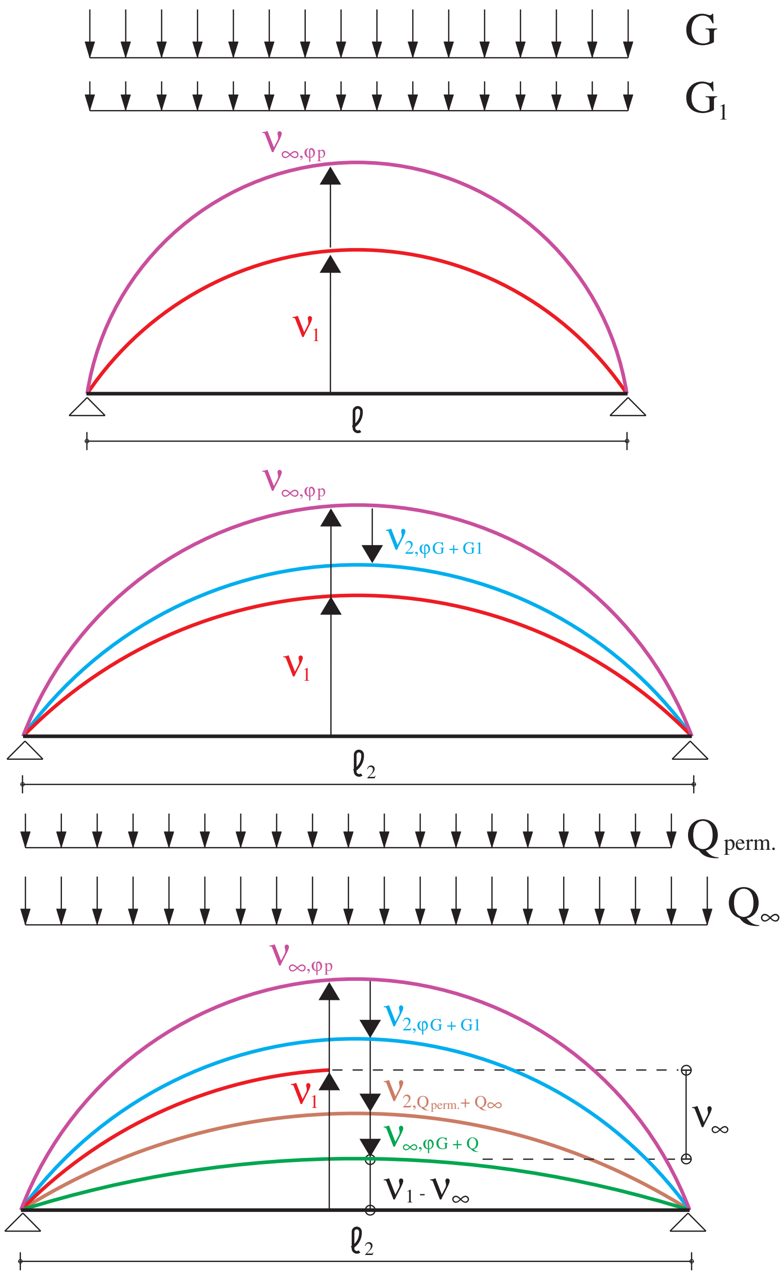


Fig. 5.19 *Deformation at infinite time* ν_∞ *of a hollow core floor of span* ℓ, *compared to the initial camber* ν_1 *once installed, in presence of all permanent and long-term variable loads (quasi - permanent load condition), with the real conditions of restraint in service and the corresponding calculated span* ℓ_2 .

The variation of camber at infinite time ν_∞, compared to initial condition ν_1 (camber in situ with castings at time t_1) is due to the sum of the following terms (see Fig. 5.19):

$$\nu_\infty = \nu_{\infty,\varphi p} + \nu_{2,\varphi G+G1} + \nu_{2,Q_{perm}+Q_\infty} + \nu_{\infty,\varphi G+Q}$$

where

$\nu_{\infty,\varphi p}$ = viscous effects and relaxation of prestressing between time t_1 of installation and final time t_∞

$$\nu_{\infty,\varphi p} = \left[\rho\varphi(t_\infty,t_1) - \frac{(P_1 - P_\infty)}{P_1}\right] \frac{A_p\, P_1\, e_1\, \ell_p^2}{8\, E_1 I_1} \; \frac{n}{5}$$

$\nu_{2,\varphi G+G1}$ = viscous effects of dead load and corroborant casts from time t_1 of installation up to time t_2 of application of live loads.

$$\nu_{2,\varphi G+G1} = \frac{n}{384} \frac{(G + G_1)\,\ell_2^{\,4}}{E_1 I_1} \varphi(t_2,t_1)$$

$\nu_{2,Q_{perm}+Q_\infty}$ = elastic sag for permanent and long-term variable loads

$$\nu_{2,Q_{perm}+Q_\infty} = \frac{n}{384} \frac{(Q_{perm} + Q_\infty)\,\ell_2^{\,4}}{E_1 I_1}$$

$\nu_{\infty,\varphi G+Q}$ = viscous effects of all permanent loads, including dead load and additional castings and long-term variable loads from time t_2 of application of live loads up to final infinite time t_∞

$$\nu_{\infty,\varphi G+Q} = \frac{n}{384} \frac{(G + G1 + Q_{perm} + Q_\infty)\,\ell_2^{\,4}}{E_1 I_1} \rho\varphi(t_\infty,t_2)$$

In the expressions above it is necessary to consider that:

– A_p, I, G, G_1, Q_{perm} and Q_∞ refer to the modular width of the slab,

– the geometric characteristics refer to the prefabricated element completed with the corroborant in situ concrete,

– coefficient $n = 1 \div 5$ depends on the restraint value hypothesized at the ends of the slab.

The meaning of symbols is the same as in Table 5.5 above.

Example of calculation of deformation

Example 5.4

Calculation of initial camber in service and deformation at infinite time of the hollow core floor illustrated in Example 4.1 (paragraph 4.4.4. Chapter 4) that is a hollow core floor $h = 300$ mm in continuity on two bays, with net span of 9.60 m and a final calculated span of 10.00 m.

Dead load of slabs	G	=	3.7 kN/m^2
Weight of in situ casts	G_1	=	0.3 kN/m^2
Permanent overload	Q_{perm}	=	3.0 kN/m^2
Variable overload	Q_{var}	=	5.0 kN/m^2
Long-term variable load	Q_∞	=	$0.3 \times 5.0 = 1.5$ kN/m^2
Hypothesizing an office load	ψ_2	=	0.3
Coefficient of static restraint	n	=	2.5 (taking into account the restraint resources at the extremities which are hardly ever merely simple supports)

The characteristics of the section of reinforcement are those indicated in Fig. 5.20.

$I = 206{,}100 \text{ cm}^4 \quad (2061 \times 10^6 \text{mm}^4)$

$I_1 = 212{,}000 \text{ cm}^4 \quad (2120 \times 10^6 \text{mm}^4)$

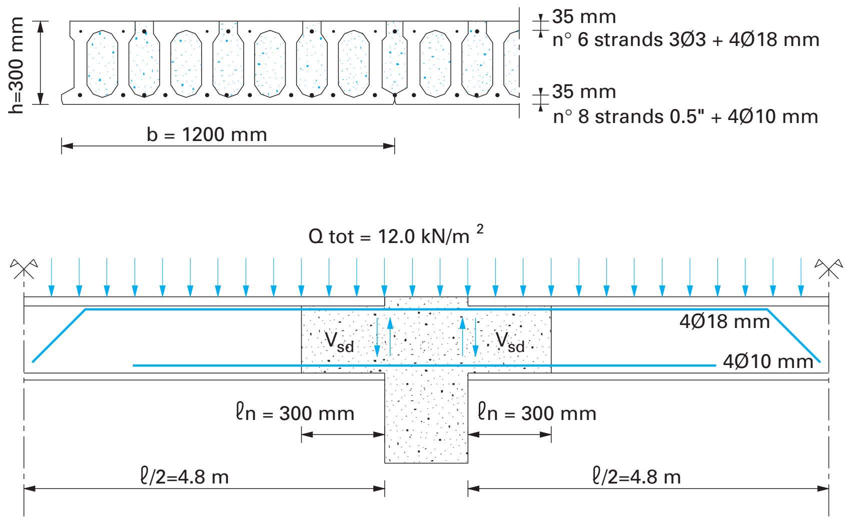


Fig. 5.20 Calculation of deformations of a hollow core floor with h = 300 mm

Let us consider a slab manufactured with vibrofinishing and accelerated hardening, installed in situ after one month of storage on provisory supports placed 30 cm from the ends.

ℓ = 9.60 m $\qquad$ ℓ_0 = 9.0

ℓ_1 = 9.60 m $\qquad$ ℓ_2 = 10.0 m.

Since t_1 = 1 month; t_2 = 4 months,

we can assume: $\alpha(t_1) = 0.4$ $\qquad$ $\alpha(t_2) = 0.65$

The hollow core slab is prestressed with eight 0.5” strands placed at 3.5 cm from the intrados and six 3 ∅ 3 stranded wires placed at 3.5 cm from the extrados.

$$A_p = 8 \times 93\ \text{mm}^2 + 6 \times 21.2\ \text{mm}^2 = 871.2\ \text{mm}^2$$

$$e = e_1 = -79\ \text{mm}$$

From the calculation of the prestressing relaxation we find:

$$P_0 = 1250\ \text{N/mm}^2 \quad P_1 = 1150\ \text{N/mm}^2 \quad P_\infty = 1100\ \text{N/mm}^2$$

Slab span, for calculating the effect of prestressing, takes into account the 70 diameters of the transmission length. In this case interpolation is made between eight 0.5” strands and six 3 ∅ 3 stranded wires and is assumed:

$$\ell_p = 9600 - (70 \times 8.1) = 9033 \text{ mm}$$

The following rheological characteristics are assumed:

$\varphi(\infty,o) = 2.5$

$\varphi(t_1, t_0) = 0.3 \times 2.5 = 0.75$ $\varphi(t_2, t_1) = 0.25 \times 2.5 = 0.625$

$\varphi(\infty, t_1) = 0.6 \times 2.5 = 1.5$ $\varphi(\infty, t_1) = 0.35 \times 2.5 = 0.875$

$E_{0,\,lower} = 18.000$ MPa $E_{0,\,upper} = 19.000$ MPa

$E_1 = E_2 = 30.000$ MPa $\rho = 0.8$

Initial camber ν_0
(mean theoretical value with tolerance $\pm 0.1 \div 0.2$ % of ℓ)

Initial camber on storage due to prestressing alone is:

$$\nu_{o,p} = \frac{A_p P_o e \ell_p^2}{8 E_{o,inf} I} = -\frac{871 \times 1250 \times 79 \times 9033^2}{8 \times 18000 \times 2061 \times 10^2} = -23.6 \text{ mm}$$

Initial sag on storage due to dead load is:

$$\nu_{o,G} = \frac{5}{384} \frac{G \ell_o^4}{E_{o,sup} I} = \frac{5}{384} \times 3.7 \times \frac{1.2 \times 900^4}{19000 \times 2061 \times 10^6} = 9.7 \text{ mm}$$

The resulting initial camber is:

$$\nu_0 = \nu_{0,\,p} + \nu_{0,\,G} = -23.6 + 9.7 = -13.9 \text{ mm}$$

(with tolerance $\pm 10 \div 19$ mm)

Camber after installation ν_1 in presence of additional in situ concrete
(mean theoretical value with tolerance $\pm 0.1 \div 0.2\%$ of ℓ)

The relative viscous effects at time t_1 (1 month) and relaxation of prestressing combine with initial camber due to prestressing only:

$$\nu_{1,p} = \nu_{o,p}\left[1 + \frac{E_{o,inf}}{(E_{o,inf} + E_1)/2}\varphi(t_1, t_o)\right] + \frac{(P_o - P_1)A_p\, e\, \ell_p^2}{8 E_1 I}$$

$$\nu_{1,p} = -23.6\left[1 + \frac{18000}{(18000 + 30000)/2} 0.75\right] + \frac{(1250 - 1150)871.2 \times 79 \times 9033^2}{8 \times 30000 \times 2061 \times 10^6}$$

$$= -23.6\,(1 + 0.562) + 1.13 = -35.7 \text{ mm}$$

– Viscous effects due to dead load during one month of storage:

$$\nu_{1,\varphi G} = \nu_{o,G} \quad \frac{E_{o,sup}}{(E_{o,sup} + E_1)/2}\varphi(t_1, t_o)$$

$$\nu_{1,\varphi G} = 9.7 \quad \frac{19000}{(19000 + 30000)/2} \quad 0.75 = 9.7 \times 0.581 = 5.6 \text{ mm}$$

– Sag after installation due to dead load and that of additional castings:

$$\nu_{1,G+G1} = \frac{5}{384}\frac{(G + G_1)\ell^4}{E_1\ I} = \frac{5}{384} 1.2(3.7 + 0.3)\frac{9600^4}{30000 \times 2061 \times 10^6} = 8.6 \text{ mm}$$

Camber after installation at time t_1 is thus found to be:

$\nu_1 = \nu_{1,p} + \nu_{1,\varphi G} + \nu_{1,G+G1} = -35.7 + 5.6 + 8.6 = -21.5$ mm

(with tolerance $\pm$ 10÷19 mm).

The relationship between camber after installation ν_1 and span ℓ_1 is found to be 1/465 (< 1/300).

Variation of camber at infinite time ν_∞

(mean theoretical value with tolerance $\pm 0.1 \div 0.2\%$ of ℓ)

– Viscous effects due to prestressing in aged concrete, added to relaxation of reinforcement:

$$\nu_{\infty,\varphi_p} = \left[\rho\,\varphi(t_\infty,t_1) - \frac{(P_1 - P_\infty)}{P_1}\right] \frac{A_p\,P_1\,e_1\,\ell^2_p}{8\,E_1 I_1}\;\frac{n}{5}$$

$$\nu_{\infty,\varphi p} = -\left[0.8 \times 1.5 - \frac{(1150 - 1100)}{1150}\right] \frac{1150 \times 871.2 \times 79 \times 9033^2}{8 \times 30000 \times 2120 \times 10^6}\;\frac{2.5}{5} =$$

$$= -(1.2 - 0.068)\;\; 13 \times 2.5/5 \;=\; -7.3 \text{ mm}$$

– Viscous effects due to dead load and additional castings up to time t_2 of application of live loads (4 months from casting)

$$\nu_{2,\,\varphi G+G1} = \frac{n}{384}\,\frac{(G + G_1)\,\ell_2^4\,\varphi(t_2,t_1)}{E_1\;I_1}$$

$$\nu_{2,\,\varphi G+G1} = \frac{2.5}{384}\;\frac{1.2\,(3.7 + 0.3)\,10000^4 \times 0.625}{30000 \times 2120 \times 10^6} \;=\; 3.1\text{mm}$$

– Elastic sag caused by permanent and long-term variable loads

$$\nu_{2,\,Q_{perm}+Q_\infty} = \frac{n}{384}\,\frac{(Q_{perm} + Q_\infty)\,\ell_2^{\,4}}{E_1 I_1}$$

$$\nu_{2,\,Q_{perm}+Q_\infty} = \frac{2.5}{384}\;\frac{1.2\,(3.0 + 1.5)\,10000^4}{30000 \times 2120 \times 10^6} \;=\; 5.5 \text{ mm}$$

– Viscous effects from time of application of live loads t_2 to infinite time due to dead load, weight of additional casting, permanent and long-term variable loads:

$$\nu_{\infty},\varphi_{G+Q} = \frac{n}{384}\frac{(G+G1+Q_{perm}+Q_{\infty})\,\ell_2^{\,4}}{E_1J_1}\ \rho\ \varphi(\infty,t_2)$$

$$\nu_{\infty},\varphi_{G+Q} = \frac{2.5}{384}\ \frac{1.2\,(3.7+0.3+3.0+1.5)\,10000^4}{30000\times2120\times10^6}\times0.8\times0.875 = 7{,}3\ \text{mm}$$

Variation in camber at infinite time is thus:

$$\boldsymbol{\nu}_{\infty} = \boldsymbol{\nu}_{\infty},\varphi_p + \boldsymbol{\nu}_2,\varphi_{G+G1} + \boldsymbol{\nu}_{2,Qperm+Q_{\infty}} + \boldsymbol{\nu}_{\infty},\varphi_{G+Q}$$

$$\boldsymbol{\nu}_{\infty} = -7.3 + 3.1 + 5.5 + 7.3 = 8.6\ \text{mm} \quad (\text{with tollerance } \pm 10\div19\ \text{mm})$$

The relationship between camber variation at infinite time and the in-service span ℓ_2 is 1/1160 (<1/500)

The theoretical instantaneous elastic sag $\boldsymbol{\nu}_e$
for the "rare" load combination (that is, corresponding to the maximum value of permanent and long-term variable overloads)

$$\nu_e = \frac{n}{384}\frac{Q_{perm}+Q_V}{E_1I_1}\,\ell_2^{\,4} = \frac{2.5}{384}\ \frac{1.2\,(3.0+5.0)\,10000^4}{30000\times2120\times10^6} = 9{,}8\ \text{mm}$$

The relationship between instantaneous elastic sag and the calculated span ℓ_2 is 1/1020 (<1/1000).

5.7.5. Elastic sag at the time of final testing

Theoretical sag under test loads, to be compared with the actual measured sagging, can be predetermined with good precision as a function of the effective capacity for transversal distribution of the floor and as a function of restraints at supports.

Normally a strip of floor 2.40 m wide, corresponding to two hollow core slabs, is uniformly loaded, leaving at the sides at least two or three unloaded slabs before the edge that is free or joined to the edge beam.
In this situation, and taking into account the capacity for distribution seen in point 5.4 above, to test the floor as if it were loaded with an overload equal to the uniformly distributed load in service, in practice it is necessary to double such a load on the two slabs (see Fig. 5.21).
In fact, if $\mathbf{q_c}$ is the overload applied to the two adjacent slabs the theoretical sag ν_e at the centre line is:

$$\nu_e = \frac{n}{384} \frac{1.2\, q_c (\alpha_3 + \alpha_2) \ell_c^{\,4}}{E_1 \; I_1}$$

$\alpha_2,\ \alpha_3$ = % of load on the slab and on the adjacent one

ℓ_c = calculated span [mm]

n = 5 for an isostatic restraint

n = 3 in the case of the first edge bay

n = $1 \div 2$ in the case of central bays

E_1 = $30.000 \div 33.000$

q_c = test load [N/mm^2]

I_1 = moment of inertia for 2nd phase [mm^4]

Percentages of load distribution refer to paragraph 5.4 (Fig. 5.4) above.

In the case of central slabs and spans of $6 \div 10$ m we may find:

$\alpha_3 = 24 \div 26\%$ and $\alpha_2 = \alpha_4 \approx 22\%$

$q(C) = q(D) = 0.46 \div 0.48\ q_c$

Thus to have a test load equivalent to the in-service load

$q(C) = q(D) \approx q_s$

It must be

$$q_c = \frac{q_s}{0.45 \div 0.48} \approx 2\, q_s$$

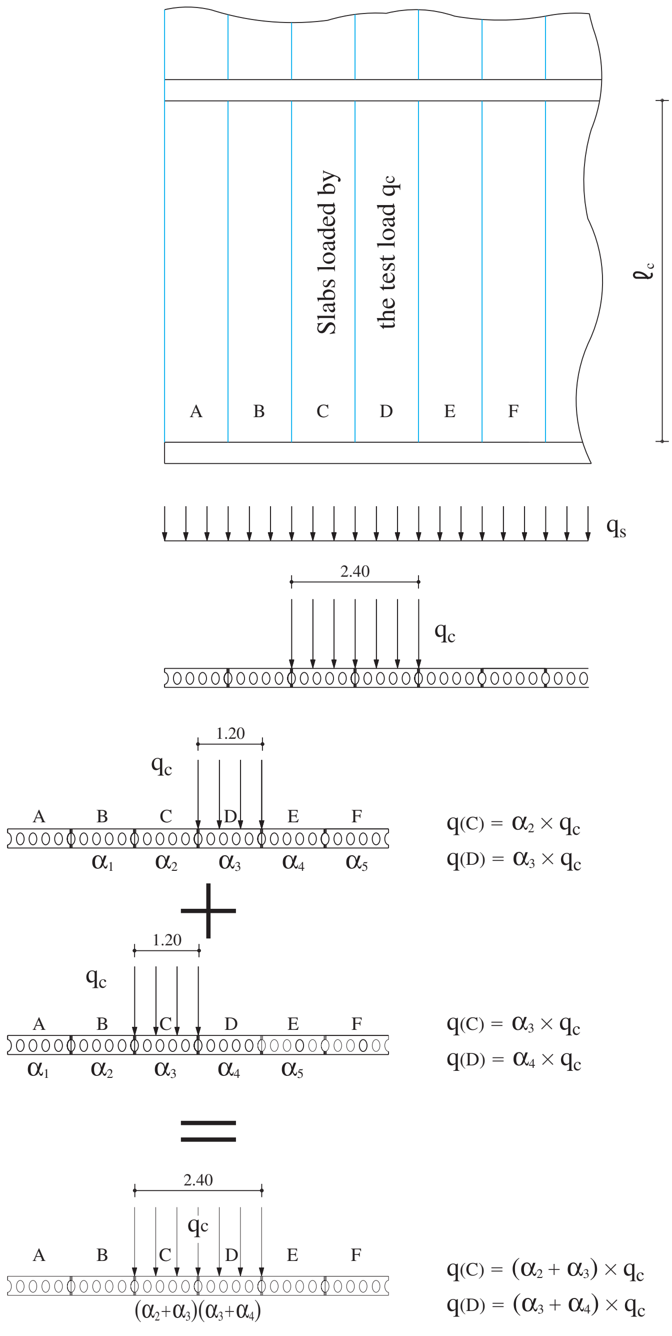


Fig. 5.21 Test overload q_c and in-service load q_s
Calculation of elastic sag for the final inspection test of a hollow core floor.

It must be kept in mind that structural continuity, in the case of testing up to in-service loads, is present even when a simple concrete topping without specific reinforcement against a negative moment on the bays of the floor with simple support has been prescribed.

The tensile strength of the concrete used in the topping (which can be evaluated at 100÷150 kN/m for a 4÷5 cm cover) is in fact sufficient, in normal testing conditions, to maintain structural continuity on the intermediate support, even when specific reinforcing is lacking.

Taking these structural resources into account, it is possible to obtain restraining conditions increased even by 20 ÷ 30% and transversal distribution capacities increased even by 15% compared to theoretical ones.
The real test sag may thus be:

$$\mathbf{v}_{e,\,real} = \mathbf{v}_{e,\,theor} \times \frac{1}{1 \div 1.15} \times \frac{1}{1 \div 1.3} = 0.6 \div 1.0\ \mathbf{v}_{e,\,theor}$$

Example of calculation of test sag
Example 5.5

Calculation of theoretical test sag of the hollow core floor with h = 300 mm described in Example 5.4 above on loading two adjacent slabs of the first bay with a uniformly distributed load of 16.0 kN/m², corresponding to twice the in-service load, equivalent to 8.0 kN/m².

ℓ_c = 10.0 m = 10,000 mm
E_1 = 30000 N/mm²
I_1 = 2120×10^6 mm⁴
q_c = $16.0 \times 10^3 / 10^6$ N/mm² (linear load = 1.2×16.0 N/mm)
n = 3 (degree of theoretical restraint for the edge bay in continuity)
α_2 = 0.22
α_3 = 0.24

The theoretical elastic sag is:

$$\mathbf{v}_{e,\,theor} = \frac{3}{384} \; \frac{1.2\times16.0\,(0.22 + 0.24) \times 10000^4}{30000\times2120\times10^6} = 10.8 \text{ mm}$$

The theoretical test sag with respect to the calculated span is approximately $\ell/1000$. In practice, since transversal distribution is usually greater, sometimes by even more than 15%, than theoretical values due to the plate effect, and since even on the external support there is the formation of a partial joint which means n = 2.0 instead of n = 3.0, the test may give lower sag values on the order of:

$$\mathbf{v}_{e,\,real} = 10.8 \times \frac{2}{3} \times \frac{1}{1.15} = 6.3 \text{ mm}$$

5.8. Graphic representations

In accordance with European Standard ISO 9001 it is required that the graphic representation of a structure such as a hollow core floor shall be accompanied by the following details to complete the executive plans:

- indication of the dead weights of the structure,
- indication of permanent and variable overloads,
- restraints,
- characteristics of the concretes to be employed,
- characteristics of the types of steel to be employed,
- report on calculations and verifications of cross sections at supports and at midpoint,
- detailed instructions concerning handling, lifting and assembly of prefabricated slabs with sizes of cables and equipment required,
- sequence of assembly with special emphasis on safety instructions,
- environmental destination of the structure to be erected,
- indications concerning protections used:
  fire resistance, resistance to aggressive actions, etc.;
- indications of manufacturing and assembly tolerances,
- rules concerning temporary timbering,
- rules concerning support devices,
- detailed prescriptions for finishing operations to be performed in situ,
- detailed instructions concerning the forming of unions and joints,
- specification of maintenance operations to be performed over the years.

Accuracy and scrupulousness are recommended in preparing the details listed above so as to remain a Qualified Designer and Supplier of hollow core floors.

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