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PRECAST PRESTRESSED CONCRETE

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Abstract

Prestressed concrete has been the latest great revolution in the building industry; it provided solutions to problems that could not be resolved with simply reinforced concrete or any of the other materials available at the time. It was soon related to long-line mass-production operations for economical reasons and the benefits of a controlled production environment. It provided engineers and architects the chance to design lighter and more resistant structures, allowing for higher quality materials to be used and being these more efficiently applied.

This report lays down the principles for the design, production and reception of precast-prestressed products for the building industry. From the history of its development, mainly by the work of the brilliant French engineer Eugene Freyssinet, to the relevance given to prestressed-prefabricated products by the latest national and European standards.

Acknowledgements

With this final dissertation I complete my degree in Architectural Technology for which I must thank my parents in the first place, for their support and motivation through all these years. I must also thank my brother and sister for their patience through the many hours spent on this report, and my boyfriend for his encouragement and continuous support.

Also my gratitude goes towards my dissertation tutors Carlos José Parra Costa and Alfonso Martínez Martínez, for their knowledgeable advice and guidance through the writing of this report.

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1 Introduction

1.1 Objectives

With the purpose of obtaining the degree in Architectural Technology, Sofía Lorenzo Romero has completed this dissertation titled **“Prestressed Precast Concrete”**. This paper has been guided by Professor Carlos José Parra Costa and Professor Alfonso Martínez Martínez from the Polytechnic University of Cartagena.

It is intended as an introduction to prestressed concrete as a precast structural material given the tendency towards a more industrialized building construction and the relevance given to prefabrication by the latest national and European standards. This dissertation lays down the principles for the production, reception and design of precast prestressed products.

1.2 Introduction

Reinforced concrete’s tensile strength is limited, while its compressive strength is extensive. Consequently prestressing becomes a tool to fully utilize that compressive strength and to eliminate or control cracking and deflection. This active combination of concrete and steel, along with the quality in design and production, makes precast prestressed units extremely structurally efficient. Eurocode 2 and the latest EHE standard make provision for reduced partial safety factors and special conditions for precast units, in acknowledgement of the controlled production environment.

There is also an increasing demand for construction options that will contribute to achieving sustainable development, giving importance to factory-made prestressed concrete for its excellent resource efficiency for materials, labor, energy and processes. Also the trend is towards lower material costs and higher labor costs, despite crisis that may temporarily affect that tendency. For all these reasons, precast-prestressed structural members will become increasingly relevant in building construction.

1.2 Chapter summary

In this dissertation the basic principles for prestressed precast structural products will be summarized.

In Chapter 2, a brief introduction of the concept behind prestressing concrete and the history of its development are described. The technique behind pretensioning concrete is further detailed in Chapter 3, from materials to production processes, and how it compares to simply reinforced concrete. Chapter 4 shows usual precast prestressed units produced by the methods described in the previous chapter and how CE marking and officially recognized quality marks affect the production and reception of these products. Finally, Chapter 5 covers a design example of a usual precast prestressed structural member in flexion: an uncracked, simply supported, double-t beam; while Chapter 6 reveals the final conclusions of this dissertation.

2 History of Prestressed Concrete

2.1 Introduction

Prestressed concrete is one of the leading materials in modern construction of the 21st century, yet the concept of prestressing has been employed long before we could figure out a way to apply it to structures. Traditional examples include a cartwheel, a barrel or simply the act of carrying a horizontal stack of books between our hands. These have been used in text books since the middle of last century to explain the concept behind prestressed concrete.

The system of the cartwheel is formed by a wood wheel which is tightened by an iron rim of slightly smaller radius that has been previously expanded by heat. As the steel band cools it is now in tension while the wood wheel is being compressed and is transferring the compression to the spokes. The load of the vehicle (F) is applied on the hub (Fig 2.1) increasing the compression of the lower spokes (B-B') which are shortened. Simultaneously the upper spoke (A-A') is extended but due to the previous compressive stress applied it remains compressed. Without this previous compression applied to the wheel rim the system would not work since the wood spokes could not withstand the tensile stress.

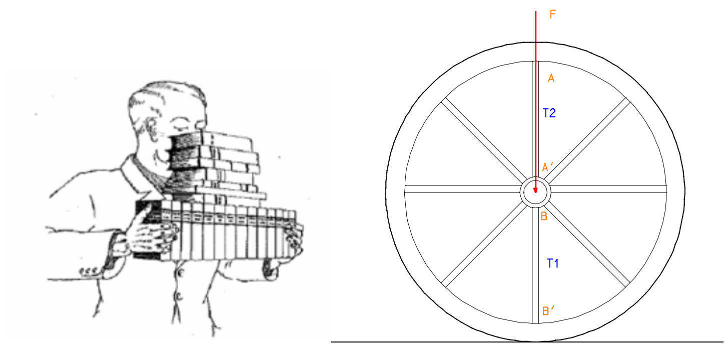


Figure 2.1 Practical Prestressing examples

This simplified system of a tensioned member (tendon) and a compressed member is essentially the same applied in modern prestressed concrete structures.

Even though the weakness of concrete in tension had been known ever since the Romans started using it, a long time and experience was needed to overcome this issue, first with reinforcement and later with prestressing. The Romans tried to reinforce concrete by using bronze bars with little success due to the different coefficients of expansion. Although the intuition of the potential benefits of combining concrete with other materials was already there, after the collapse of the Roman Empire the knowledge of how to make and use concrete as a primary structural material was lost for many centuries.

It was not until the nineteenth century that the reinforcement of concrete using iron or steel was discovered, noticing that these were more or less complementary materials. Joseph Monier developed the idea of reinforced concrete and received a patent for it in 1849. This meant a step further towards prestressed concrete as it can be seen in this flowchart about the development of the different building materials.

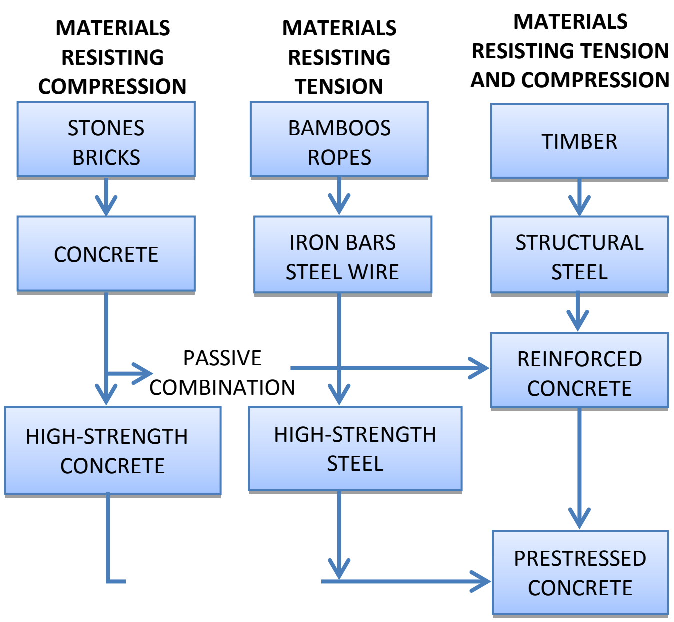


Figure 2.2 Development of building materials .Transcript from (Kramer,2005)

Steel and concrete where first combined in a passive manner and prestressed concrete would later result in an active combination.

With this combination the steel can provide the tensile strength and probably some of the shear strength while the concrete, strong in compression, protects the steel giving it durability and fire protection. Since the coefficients of thermal expansion for steel and for concrete are of the order of $10x10^{-6}$ per ºC and $7\text{-}12x10^{-6}$ per ºC respectively, it is assumed that there is a perfect bond. Therefore the strain in the reinforcement is identical to the strain in the adjacent concrete, known as 'compatibility of strains' across the cross section of the member.

The following figure illustrates the behaviour of a simply supported beam subjected to bending and shows the position of steel reinforcement to resist the tensile forces, while the compression forces on top of the beam are carried by concrete. In this case, wherever tension occurs it is likely that cracking of the concrete will take place. Even though this cracking does not detract from the safety of the structure provided the bonding stops the cracks from opening, the next natural step was to figure out a way in which concrete had only to resist compressions which lead to the invention of prestressed concrete. One of the fundamental approaches of prestressed concrete is to limit tensile stresses, and hence flexural cracking, in the concrete under working conditions.

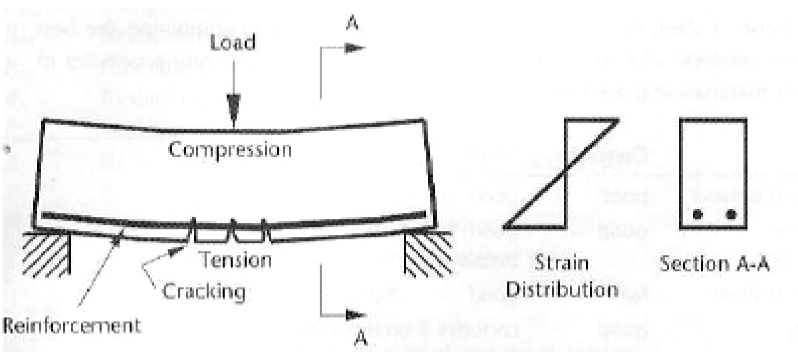


Figure 2.3 Reinforced simply supported beam: strain distribution

Prestressing concrete is providing a longitudinal compressive force by tensioned steel wires or strands which are anchored against the concrete. With the creation of stresses in the member before loading we can use concrete moreefficiently by maintaining it in a state of compression throughout. Detailed information on the process of prestressing and its comparison to simply reinforced concrete will be given in following chapters.

At the beginning of the 20th century prestressed concrete became the most significant new direction in structural engineering first and architecture at last (Billington, 2004) .It gave architects a new realm of reinforced concrete design pushing both the structural and architectural limits of design to a level that neither concrete nor structural steel could achieve.

In search of a system that allowed for prestress to be efficiently applied to the concrete member many were unsuccessful. The first attempt at prestressed concrete that is documented is a patent for a concrete pavement taken in 1888 by P.H. Jackson of the United States. Several patents were taken out for prestressing schemes using low-strength steel where the long term effects of creep and shrinkage of the concrete reduced the prestress force so much that any advantage was lost. It was the French engineer Eugene Freyssinet who developed the theory and systems that allowed for prestressed concrete to be practicable.

2.2 Freyssinet's discovery of creep.

The underlying idea of prestressing concrete had been around for a long time but it was Eugene Freyssinet who came up with the invention that allowed the idea to be put into practice. Early attempts worked with the beams being less likely to crack in tension, but after a few months the cracks reopened.

The answer to this problem was found when it was realised that creep occurred. It was then recognised that the initial prestressing force was reduced appreciably by losses, and hence high-strength steel (and therefore high initial tensional stress) were essential (Abeles, Bardham-Roy, 1981).

It was Freyssinet who recognised that high strength concrete and high steel pre-strains were needed to leave some prestress after creep had taken place. In 1932 when he was asked by the *Science etIndustrie* journal to write about his progress in prestressing, he outlined the following conditions for practical use of prestressing (Billington,2004):

- Using metals with a very high elastic limit.
- Submitting the steel to very strong initial tensions, much greater than 500 N/mm^2
- Associating the metals with concretes of a very low, constant and well-known rate of deformability, which offer the additional advantage of very high and regular strengths of resistance.
- High strength steel and high strength concrete to reduce the loss of prestress to a minimum.

In the mid 1920's Freyssinet had built three similar bridges over the River Allier, near Vichy, in France. They were built using reinforced concrete arches with open spandrels (figure 2.4). He installed jacks between the two halves of each arch span in order to avoid the problems of the use of wedges over the falsework that supports the arches while being built. When the wedges are knocked out it drops the falsework away and transfers the deadweight to the arch making the operation rather dangerous. By jacking the two arches against each other they lifted slightly away from the falsework, which could then be safely removed. The gap between the arches was then filled with in-situ concrete.

He was able to reinstall the jacks when he realised months later that the parapet over the bridge was no longer straight and was dipping at midspan. He concluded that the arch must have shortened and led him to realise that concrete creeps under load. By reinstalling the jacks he was able to push the arches apart again and make good the structure.

Figure 2.4 Boutiron Bridge. The only of the three similar built that survived World War II.

Until then it had been assumed that concrete had a Young's modulus which remained fixed, but he realised that the deferred strains due to creep explained why the prestress was not effective in the early trials. In order to minimise the total amount of creep he reasoned that high quality concrete should be used, as well as high tensile steel so that some prestress would remain after the creep had occurred.

Before using the jacks in the bridges Freyssinet tested the technique in a 50 m span arch built in Moulins in 1908. The arch was tightened by a lower tie which was stressed thus controlling its reactions. This is the ancestor of all the prestressed concrete works.

The discovery of creep was the first step that allowed Freyssinet to invent prestressed concrete and even precast segmental construction.

2.3 Invention of Prestressed Concrete

There were three inventions that Freyssinet patented in order to apply compression to concrete. The first, in 1928, was a process of applying compression by “pre-tension and bonded wires”, known as the birth of prestressing. After this he invented the flat jack (Fig. 2.5), in 1938 and in 1939 he patented the concrete anchorage shaped by a parallel wire system of cables tensioned and locked off by anchor cones (Xercavins, 2008).

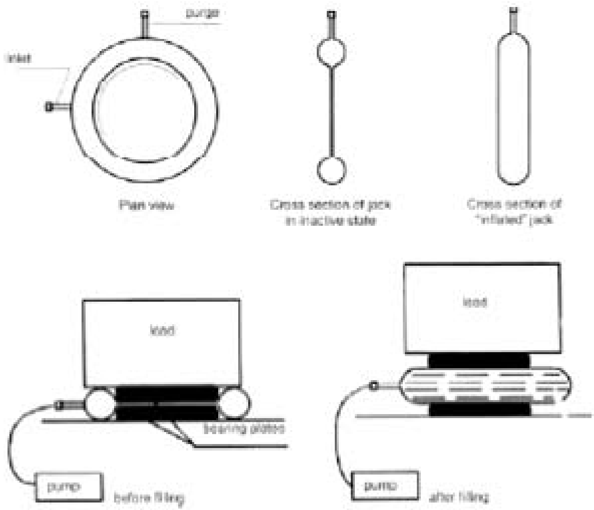


Figure 2.5 Flat jack invented by Freyssinet.

These three inventions were the results of the great efforts made by Freyssinet in order to transform the idea or prestressing into an industrial reality. This was necessary since at the time of the first patent the scientific community did not believe in prestressing.

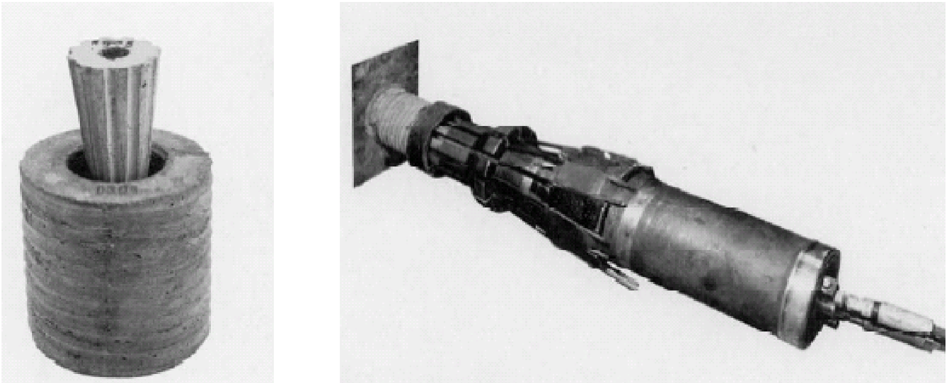


Figure 2.6 Concrete anchorage patented in 1939 by Freyssinet

The 1928 patent came after 25 years of work and laboratory tests (Xercavins, Demarthe, Shushkewich, 2008) .On the 2nd of October 1928, Freyssinet and Jean Séailles applied for the patent which described the process to manufacture precast beams, sleepers, poles, pipes, etc. The theory of permanent precompression in concretes or other materials was precisely described. At that time no one believed the process could be commercially exploited and all Freyssinet was able to produce were prestressed concrete pilons for electric power lines which was technically a success due to the improvement on industrial precasting techniques but it also was a commercial failure due to the worldwide depression of 1929 (Marrey, Grote, 2003).

Some of the advances he made in the area of precast concrete were the invention of steam curing, which accelerated the concrete hardening and rate of production, and the perfection of the grinding fineness of cement and the industrial process for precast concrete elements. So Freyssinet set the guidelines to allow for the construction of all the large structures being built nowadays, where both prestressing and precasting are techniques widely used.

The first practical application of prestressing devices in a building was the strengthening of the Maritime Station in Le Havre in 1934. It was sinking 25 mm per month into a deep layer of clay that was beneath the foundations. Although the solution proposed by Freyssinet was considered very bold at that time, as he said "imminent collapse seemed to be inevitable and this was the only possible hope of avoiding disaster" (Xercavins, Demarthe, Shushkewich, 2008).

He designed an integrated system: completing the existing footings with new ones in the spaces between and making them as monolithic as possible by tightening prestressed cables around them (Fig. 2.7) and also increasing the bearing capacity of the foundation by adding piles that reached sufficient resisting layers of soil.

These piles were cast inside the building in 2 m sections using the techniques of steam curing and vibration of the concrete developed by Freyssinet in order to improve the rate of casting and the quality of concrete.

The intervention was a great success and it earned Freyssinet at last a worldwide reputation and also the chance to work with the constructor EdmeCampenon. The Campenon Bernard Group had a range of sites around the world and this meant the proliferation of the prestressing techniques that the French engineer had developed. The early history of prestressed concrete is an interesting subject since it was much related to rearmament and war effort. Germany wished to save steel for World War II and so the company Wayss& Freytag obtained a license from Freyssinet to use his system for the construction of bunkers.

Figure 2.7 Horizontal prestressing on footings in Le Havre

Eventually the chief negotiator of the company (Professor Karl W. Mautner) ended up in a concentration camp (he was a Jew by birth) and was rescued by a

British Engineer's firm along with the help of the secret services of several governments. Since he had records of prestressing trials carried out in France and Germany, this event led to the advance of the technique in Britain (Marrey, Grote, 2003). The war created a need to provide emergency structures and along with the shortage of steel this made prestressed concrete the best choice regarding the scale and speed needed in the construction process (Burgoyne, 2005).

It is also of great importance the work that the engineer Gustav Magnel did in putting in writing and communicating the technical concepts that Freyssinet developed. During WWII he was responsible for a laboratory in the University of Ghent (Belgium) which was equipped for testing reinforced concrete; this allowed him to do his own research and testing on prestressed concrete, exploring Freyssinet's ideas. By testing he found out that creep also existed in steel and therefore that prestressed wires were a more significant contributor to creep in prestressed concrete structures than the concrete itself. It was him who introduced prestressed concrete in the U.S.A. through his books and through his practice and teaching there, since he was fluent in English and an experienced college Professor. Freyssinet knew the concept and method of prestressed concrete thoroughly, as displayed through his brilliant bridge designs and his patented anchorage devices. He clearly could communicate his passion for prestressing through design and construction, but he could not put in writing his technical concepts (Billington, 2004).

It was soon noticed that prestressed concrete could benefit from the advantage of controlled mass production and so this new technique was soon adopted by the precast concrete industry.While developing prestressed concrete, Freyssinet also came up with new techniques and perfected others towards a more efficient process of fabrication of the pieces. Steam-curing and the vibration of concrete are probablytwo of the most important ones. These two techniques allowed for precast/prestressed concrete to be efficiently produced and used as a structural material worldwide.

2.4 History of precast/prestressed concrete in Spain

The early history of prestressed concrete in our country starts in 1943 when the civil engineer Francisco FernándezConde is granted the first patent for Freyssinet's prestressing systems.The production of prestressed-precast concrete solutions in Spain begins when the company PACADAR is created in 1944. The first prestressed joist was cast on February the 15th in 1945, it had a span of 3.20 m and was prestressed by eight wires of 2 mm. These prestressed-precast joists were manufactured in Madrid under the name 'Freyssi joists'. This was the first industrial installation for the production of prestressed products in Spain (figure 2.8).

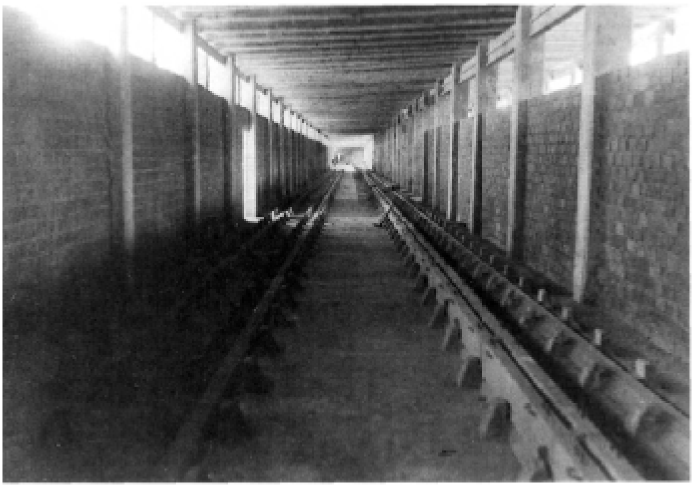


Figure 2.8First casting bed for precast/prestressed members

By 1950 prestressed joists were a common solution for floor construction in all sorts of arrangements, from slabs completed with poured on site concrete to peaked roof structures. Flat elements such as prestressed slabs and double-t beams are developed to avoid the need for hollow-core blocks, also larger precast members are produced for industrial installations. In 1955 the first continuous prestressed slab is produced by PACADAR.

During the 1960 prefabricated construction is promoted through government's programs. One of the milestones in precast concrete in that decade was the construction of the Central LecheraAsturiana factory, were state-of-the-art precast technology was used.

Another example of a great industrial factory that was built during the 1970's, was a 250.000 m^2 warehouse for El Corte Inglés which was probably the largest warehouse built with precast members in the world. The rate of production, transport and assembly was of 1000^2 per day(Burón, Fernández-Ordóñez, 1997).

During this decade considerable advances were made in the production of prestressed members such as floor slabs and double-t beams for larger loads and spans (figure 2.9)

Figure 2.9Double-t beams slab in the 1970's

It is during these years that prefabricated construction is the chosen technology to build hundreds of schools in Spain. One example is a contract award for the construction of 191 prefabricated schoolsin 1977 in 23 Spanish provinces (Pons, 2010). The concrete member mostly used for this purpose was hollow-core slabs in combination with concrete pillars and panels. The economic crisis in the late 70s and the transfer of authority on educational issues to the communities practically interrupted the construction of standard prefabricated schools during the following two decades. Even so the first high-rise prefabricated building was constructed in Madrid in the 1980s, with a total height of 110 m and great technical accuracy in its assembly. Also a new type of prestressed beams for pedestrian crossings over motorways was developed along with a system for connections (Burón, Fernández-Ordóñez, 1997).During the following decade the use of totally precast structures becomes more popular for sorting out various types of buildings such as shopping centres, industrial factories and even football stadiums. One remarkable project from these years is a building for the regional government of Extremadura, in Mérida, where a totally precast structure shaped by 25 m long prestressed beams was constructed to preserve archeological remains underneath (figure 2.10).

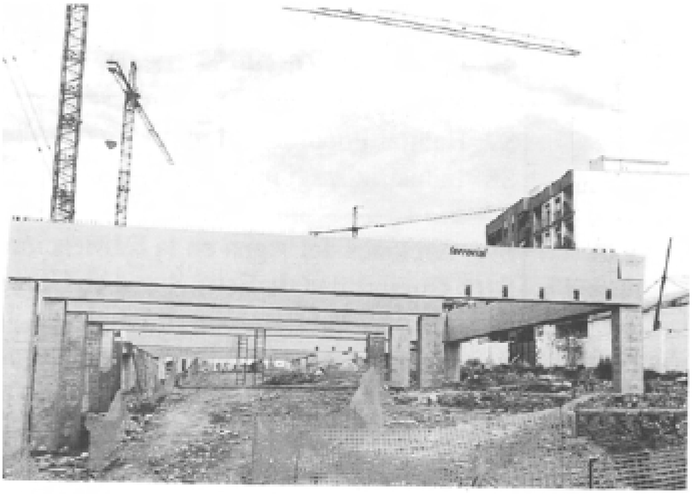


Figure 2.10 Prestressed precast beams. Architect:Juan Navarro Baldeweg

During the last years the importance of precast-prestressed structures has become increasingly relevant. The projects designed nowadays are more flexible than those from the 70s, where standard projects were usually repeated at each location. These days precast-prestressed construction has evolved to become an adaptable technology, which can provide special solutions for each particular case. The tendency is towards creative solutions instead of the reproduction of standard shapes. The use of high performance concrete, especially high strength and lightweight mixes, has become frequent and this has been regarded in the latest edition of the national standard EHE (2008). With special articles in the standard regarding the design, production, quality control and reception of precast-prestressed members it is clear that this technology has gained relevance in the building industry in Spain and will continue to do so in the future.

3 Prestressing materials and systems

We already described the origins and first attempts at prestressed concrete and now we will focus on the technique, how it compares to traditional reinforced concrete and its relation with the precast industry.

According to the Eurocode 2 the definition of prestressing for concrete structures is as follows:

"The process of prestressing consists in applying forces to the concrete structure by stressing tendons relative to the concrete member. Prestress is used globally to name all the permanent effects of the prestressing process which comprise internal forces in the sections and deformations of the structures. Other means of prestressing are not considered in this standard."

The Spanish national standard EHE (2008), article 20.1.1, defines prestressing as:

Prestressing shall be understood to be the controlled application of a stress to the concrete by means of the tensioning of steel tendons. The tendons shall be made of high strength steel and may consist of wires, strands or bars. No other forms of prestressing may be considered in this Code.

This means the structure is prepared to receive a load by applying a pre-emptive countervailing load. The basic concepts of prestressing can be represented by considering a simply supported rectangular beam as in the following figure (figure 3.1) (Nawy, 2010), assuming it is homogeneous and elastic.

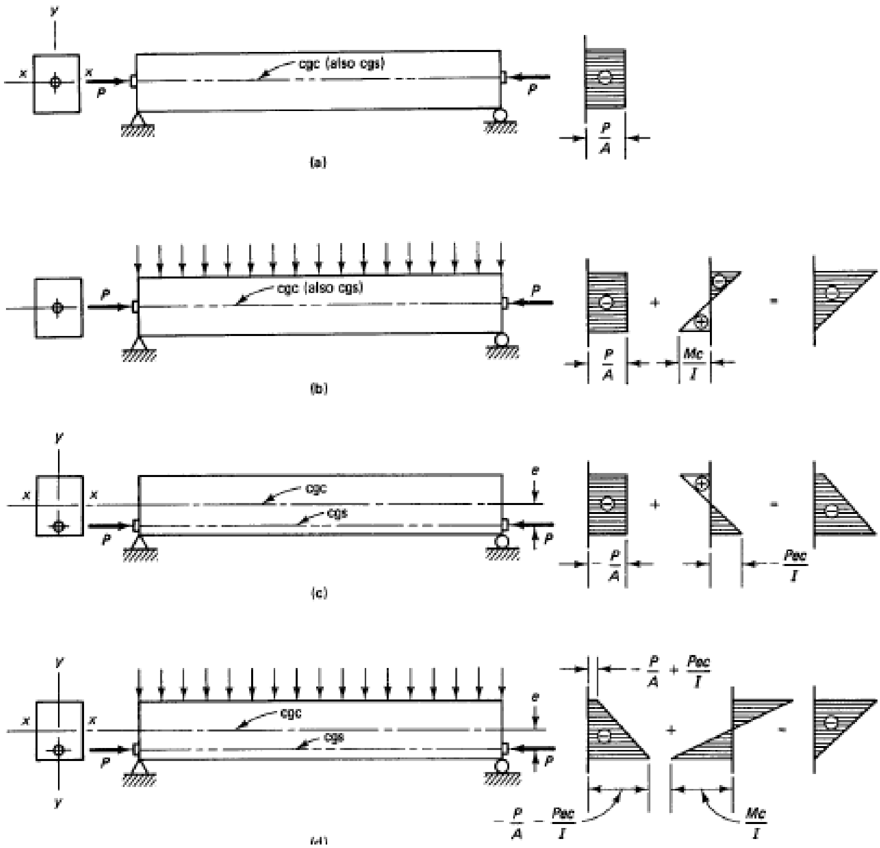


Figure 3.1 Concrete fiber stress distribution in a beam with straight tendon. (a)Concentric tendon, stress only.(b)Concentric tendon plus self-weight. (c) Eccentric tendon, prestress only.(d) Eccentric tendon plus self-weight.

In (a) the compressive stress on the beam cross section is uniform and equal to $-\dfrac{P}{A_c}$. When an external transverse load is applied to the beam, causing a maximum bending moment at midspan, the resulting stress becomes:

$$f_t = -\frac{P}{A_c} - \frac{M_c}{I_g}$$, at the top fibres.

$$f_b = -\frac{P}{A_c} + \frac{M_c}{I_g}$$, at the bottom fibres.

As seen in 3.1(b) the application of the loading stress $-M_c/I$ increases the compressive stresses at the top fibres of the beam, and so reducing the compressive stress capacity of the member. In order to avoid this, the prestressing tendon is placed below the neutral axis at midspan to induce tensile stresses at the top fibres due to prestressing (figure 3.1(c)). The tendon placed at eccentricity *e* from the center of gravity of the concrete, creates a moment *Pe*, and so the stresses at midspan become:

$$f_t = -\frac{P}{A_c} + \frac{P_{ec}}{I_g} - \frac{M_c}{I_g}$$

$$f_b = -\frac{P}{A_c} - \frac{P_{ec}}{I_g} + \frac{M_c}{I_g}$$

In the design of prestressed beams it is important not to overlook the minimum moment condition, especially when employing straight tendons, as stresses near the ends of beams where moments are small may often exceed those at sections nearer mid-span(Mosley, Bungey, Hulse, 2007).

Whether this prestress load is applied before or after the concrete has hardened will determine the two main categories of the technique**: pre-tensioning** or **post-tensioning**.

There are several ways to classify prestressed concrete based on different aspects of the technique:

- Pretensioning/post-tensioning: based on the sequence of casting the concrete and applying tension to the tendons
- External/internal prestressing: depending on the location of the prestressing tendon with respect to the concrete section.
- Full/partial prestressing: based on the amount of prestressing force and the allowance for tension or cracks to be developed.

- Uniaxial/biaxial or multiaxial prestressing: according to the directions of prestressing a member.
- Linear/circular prestressing: based on the shape of the prestressed member.

The main category for prestressing concrete methods is that for ***pretensioning and post-tensioning***so it will be the one detailed here.

3.1 Types

3.1.1 Pretensioning

In pretensioning, first the steel is tensioned between end-anchorages and then the concrete is placed in moulds around it. Once the concrete has achieved sufficient compressive strength, the steel is released, transferring the force to the concrete through the bond between both materials (figure 3.2). Because the force is transferred by bond, as large an area of contact as possible is desirable, and therefore these members will have a large number of wires or strands.

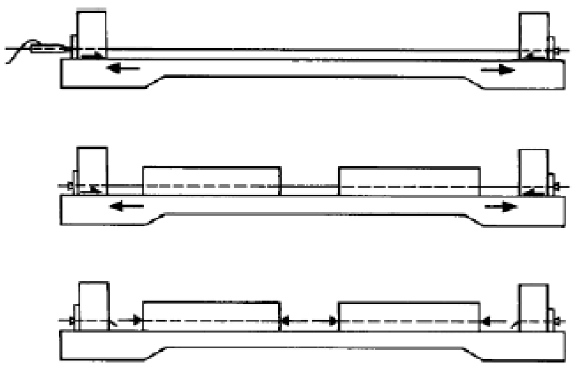


Figure 3.2 Prestressed concrete factory production schemes

This method is ideally suited for factory production since permanent stressing beds (figure 3.3) and large anchorages are required and this equipment can be amortized producing large numbers of identical units. It will be the method detailed in this dissertation since it is focused on precast members. The most effective method is long-line production where a number of similar units are produced at the same time.

Figure 3.3Long-line production of pre-tensioned members (Prefabricados Castelo).

In prestressed concrete construction, a rapid initial gain in strength is usually desirable in order to apply the prestress as early as possible. This is particularly so in the case of precast, pretensioned production. Steam curing and high early strength cement are often used to this end(Gilbert, Mickreborough, 1990).The majority of pre-tensioned units produced have straight tendons that are continuously bonded to the concrete although in larger units the prestress force can be more efficiently used by deflecting the tendons or debonding some of them by sheathing to prevent adherence. This is economically viable in case of large flexural members of constant section although the cost of operating and maintaining the mechanisms required for deflecting the tendons has deterred manufacturers. This technique is mainly used in bridge beams (Mosley, Bungey, Hulse, 2007) (figure 3.4).The different processes for producing prestressed concrete will be discussed in detail in the following sections.

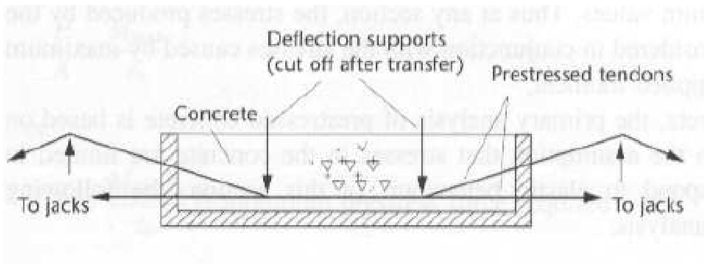


Figure 3.4 Tendon deflections in prestressed concrete

3.1.2 Post-tensioning

In post-tensioning the concrete is first cast and allowed to harden and then the prestress force is applied. The tendons are threaded through ducts cast into the concrete, which can be arranged in a curved shape (Fig. 3.5), and so the prestress force can be distributed more efficiently according to the bending moment at each section. These ducts can be shaped by preformed circular metal ducting or by using removable solid or inflatable rubber formers.

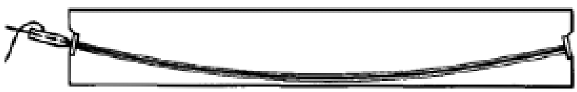


Figure 3.5 Post-tensioned concrete

Special built-in anchorages at the ends of the ducts are fixed to the mould and transfer the prestress to the concrete once the tendons have been stressed to their full force. The prestress force in these members is usually provided by several wires or strands grouped into large tendons in one anchorage. These anchorages, that are permanent in the structure, are quite expensive and so its cost outweighs the saving of steel (compared with pretensioning) in short pieces.Post-tensioned concrete is most commonly used in large building projects such as high-rises, and is especially relevant in bridge construction, being mainly an in-situ technology. Post tensioning in the precast industry is mostly used in the production of pieces for segmental construction that are assembled and stressed on site.

The steel in the tendons is protected from corrosion by either grease-based coating or grout, which are injected in the sheathing. The tendons would be unbonded or bonded depending on the material injected.

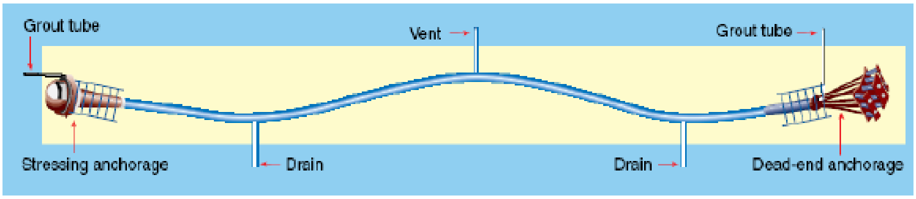


Figure 3.6 Post-tensioning duct (VSL International Ltd.)

One advantage of post-tensioning over pretensioning is that the tensioning can be carried out in stages for all tendons or for some of them.

Post-tensioning in buildings is most usually seen in floor slabs although there are many other areas where it can be an interesting option such as: foundations, post-tensioned masonry, transfer beams and plates, and as a means of combination between precast elements and cast-in-place concrete. However post-tensioning in buildings is not usual in Europe and is mostly a technology carried on-site, and since this report is centered on precast-prestressed concrete in the building industry we will focus on pretensioned/precast concrete.

3.2 Prestressed Concrete vs. Reinforced Concrete

There are several advantages of using prestressed concrete over simply reinforced, although these will only become relevant regarding the circumstances and needs of the project. It is in a fully prestressed concrete member (which is subjected to compression during service life) where we can see how prestressing rectifies several deficiencies of simply reinforced concrete. Some of these advantages are the following:

- **For a given span and loading a smaller prestressed concrete member is required.**

  This is especially relevant for large structures such as bridges or high-rise buildings, where the reduction in self-weight saves a considerable amount of concrete also in vertical members and foundations. In high-rise buildings this smaller depth of the slabs and saving of concrete may allow for extra floors to be built. Also slender sections provide a more aesthetic appeal.

- **High elastic limit steel can be used (≥ 1500 N/mm^2)**

  The use of this type of steel in simply reinforced concrete would render inadmissible values of deflections and cracking.

- **Section remains uncracked under service loads**

  This means a reduction of steel corrosion and therefore and increase in durability. This characteristic makes prestressed concrete suitable for liquid retaining structures and also improves the performance under dynamic and fatigue loading.

- **High span-to-depth ratios**

  The typical values for slabs are 28:1 for non-prestressed concrete and 45:1 for prestressed concrete

- **It offers a means of controlling deflections**

  Since eccentric prestressing causes a vertical deflection usually in the opposite direction to that caused by the applied load, deflections can be precisely controlled to protect partitions, finishes and associated structures.

- **Suitable for precast construction**

  Pretensioned concrete is mostly carried out in industrial installations where the cost of casting beds, anchorages and equipment makes it economically viable. Industrialization yields a higher quality of these members, a safer work environment on site and reduces the building process and the safety coefficients to be applied.

As we can see there are several advantages when using prestressed concrete over reinforced although several factors, including structural and economic reasons, will determine the choice of each technique.Nevertheless there are also some disadvantages that must be carefully considered when choosing prestressed concrete over simply reinforced.

One of them is the fact that most, if not all, of the concrete cross section is in compression under all load conditions so that any inherent problems due to long term creep movements are increased. Also connections must be carefully designed and considered in order to achieve an acceptable degree of monolithism. Even though a prestressed concrete member requires less concrete and about 20-30% the amount of reinforcement, many times this saving in material costs is balanced by the higher cost of the higher quality materials and professionals of prestressed concrete (Nawy, 2010).The difference between the initial costs of reinforced and prestressed concrete is reduced when a large enough number of precast units are manufactured, and also indirect long term savings are quite substantial since less maintenance is needed.

3.3Materials

3.3.1 Concrete

The development of prestressed concrete has been closely related to that of high-strength concrete, since strength and endurance are qualities that are particularly important in prestressing. In pretensioned concrete a high-strength concrete is desirable since it will improve bond and allow for an earlier transfer of the prestress force, in post-tensioning the strength of the concrete is especially relevant in the anchorage zone where local tensions are important. The use of high-strength and high-performance concretes is usual in the precast industry where the optimization of the production process justifies the higher cost of materials.

According to the Spanish standard EHE the types of cement that may be used in prestressed concrete are: CEM I, CEMII/A-D, CEM II/A-V and CEM II/A-P. Where CEM I refers to common or general purpose cement (with up to 5% of minor additional constituents), CEM II/A-D is silica fume cement (6-10%), CEM II/A-V is fly ash cement (6-20%), and CEM II/A-P is pozzolana cement with 6-20% of pozzolanic materials. For prestressed concrete the addition of silica fume and fly ash is limited to 10% and 20% respectively regarding the cement weight. Both of these additions increase the concrete's density and ultimate strengthand therefore the bond between concrete and steel. For this same reason air-entraining admixtures are generally not recommended for pretensioned concrete structures (which depend on adherence for bond), since the air bubbles reduce the bonding properties of concrete. The Spanish standard specifies that this type of admixtures may only be considered in a certain type of precast production for slabs where the final results in bonding can be assessed through testing of the member.

The minimum strength of concrete for prestressing according to the national standard is 25 N/mm^2, although usually the range employed goes from 35 to 50 N/mm^2. Since prestressing is performed prior to the concrete's achieving its 28 days' strength, it is important to determine the concrete compressive strength at the prestressing stage. This is generally referred as the 'initial' or 'transfer condition'. The Eurocode 2 also mentions that it may be required to specify the compressive stage at a number of stages such as demoulding, transfer of prestress, etc. This is one of the differences between simply reinforced and prestressed concrete. An early development of the concrete's strength is desirable to optimize the production process, and some techniques, such as steam-curing, make possible that strengths attained after 28 days might be reached after a few hours.

For early stages of the concrete, such as that of transfer in pretensioning, the evolution of the concrete's compressive strength and the modulus of elasticity must be determined from expressions that give an approximate value for an age different than the 28 day value.

The modulus of elasticity and compressive strength of concrete at a certain age (and under certain curing conditions, 20°C,95% humidity)according to the Eurocode 2 and the EHE standard is:

$$f_{cm}(t) = \beta_{cc}(t) f_{cm,28}$$

$$E_{cm}(t) = \beta_E(t) E_{cm,28}$$

$$\beta_E(t) = [\beta_{cc}(t)]^{0.3}$$

$$\beta_{cc}(t) = e^{s\left[1-\left(\frac{28}{t}\right)^{1/2}\right]}$$

$$E_{cm}(t) = \left(\frac{f_{cm}(t)}{f_{cm,28}} \right)^{0.3} E_{cm,28}$$

Where $f_{cm}(t)$ is the concrete's compressive strength at a given age t (in days), $f_{cm,28}$ is the value of resistance inup to 28 days (that can be estimated as $f_{ck}+8$), $E_{cm,28}$ is the modulus of elasticity at 28 days of age, and s is a coefficient that depends on the type of cement:

S=0.20 Class R cements (rapid)

S=0.35 Class N cements (normal)

S=0.38 Class S cements (slow)

Creep of concrete (the inelastic deformation due to sustained load) is another factor that is especially relevant when designing prestressed concrete members, since it reduces the prestress in the tendon with time and so it must be carefully considered when calculating the prestress losses.

Since prestressed concrete is mostly done in the precast industry, usually higher-strength and performance mixes are used compared with poured on site concrete members. The special quality control of the concrete production in the precast industry is relevant in many stages of the design, minimum concrete covers and material coefficients are reduced for that situation.

Minimum concrete covers in the EC2 standard are determined, among other factors, on account of the exposure class and the structural class, which is generally taken as 4 and then can be reduced under some circumstances which apply to the precast industry. Also the value for deviation Δc_{dev} may be reduced from the minimum 10 mm value to zero. The minimum cover for a pre-tensioned member would be the maximum satisfying the requirements for both bond and environmental conditions.

c_{min}= max {$c_{min,b}$; $c_{min,dur}$; 10 mm}

The minimum cover for bond in a pre-tensioned tendon is: two times the diameter of strand or plain wire and three times the diameter of indented wire.

The minimum cover for durability is related to the exposure and structural classes as detailed in the following tables from EC2:

Structural Class							
Criterion	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1) 2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Table 3.1 Structural classification according to EC2

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	15	20	25	30	35	40
S2	10	15	25	30	35	40	45
S3	10	20	30	35	40	45	50
S4	10	25	35	40	45	50	55
S5	15	30	40	45	50	55	60
S6	20	35	45	50	55	60	65

Table 3.2 Minimum cover requirements with regard to durability for prestressing steel.

As we can see, the higher concrete strengths and quality control procedures used in the precast industry allow for a reduction in the minimum concrete covers. The new 2008 version of the Spanish standard EHE has included special requirements for precast elements in the design, production and reception stages as well as for products in possession of a quality mark.

The general partial factors for materials for the ultimate limit states, γ_c and γ_s, according to the National Annex for the Eurocode 2 are:

Design situations	γ_c for concrete	γ_s for reinforcing/prestressing steel
Persistent and transient	1.5	1.15
Accidental	1.3	1.0

Table 3.3 Partial factors for materials for ultimate limit states

These values can be reduced according to the circumstances described in sections 15.3.1 and 15.3.2 of the EHE 2008 standard which are copied below:

15.3.1 Modification of the partial safety factor for steel

The partial safety factor for steel may be reduced to 1.10 when at least two of the following conditions are met:

a) That the construction of the structure is closely controlled pursuant to the provisions of Chapter XVII and that the attachment tolerances of the reinforcement comply with those explicitly laid down in the design, which must be at least as demanding as those indicated in paragraph 6 of Annex No 11 to this Code.

b) thatthe passive or active reinforcements, dependingon the case, bear an officially recognized quality mark with a guarantee level compliant with Section 8 of Annex 19 to this Code, or which form part of a precast element bearing an officially recognized quality mark with a guarantee level compliant with the aforementioned Section.

c) That the steel for the passive reinforcements bears an officially recognized quality mark.

15.3.2 Modification of the partial safety factor for concrete

The partial safety factor for concrete may be reduced to 1.40 in general and to 1.35 for precast elements, when the following conditions are met simultaneously:

a) that the construction of the structure is closely controlled pursuant to the provisions of Chapter XVII and that deviations in the geometry of the cross-section in relation to the nominal cross-sections of the design comply with those explicitly laid down in the design, which must be at least as demanding as those indicated in Section 6 of Annex No 11 to this Code, and

b) that the concrete bears an officially recognized quality mark with a guarantee level compliant with Section 8 of Annex 19 to this Code, or which form part of a precast element bearing an officially recognized quality mark with a guarantee levelcompliant with the aforementioned Section.

These circumstances are common in the precast industry and so it is one of the advantages when designing with precast members.

3.3.2 Steel

As we described in previous sections, because of high creep and shrinkage losses in concrete, only high-tensile steel may be used for prestressing reinforcement, and it is usually in the form of cold-drawn wires of bars. There are 3 types of steel products for prestressed reinforcement:

- Wires: vary in diameter from 3 to 7 mm, supplied in mill-coils.
- Bars: hot-rolled alloy-steel bars, vary from 20-40 mm diameter
- Strands: produced by spinning several wires around a central core wire. Can be made up of 2,3 or 7 wires. Usually 7 wires with an overall diameter of 8 to 18 mm.

According to EC2, prestressing tendons (which can be made of wires, strands and bars) shall be classified on account of:

- Strength: giving the value of the 0.1 proof stress ($f_{p0.1k}$) and the value of the ratio of tensile stress to proof stress ($f_{pk}/f_{p0.1k}$) and elongation at maximum load (ε_{uk}).
- Class: indicating the relaxation behavior
- Size
- Surface characteristics: wires and bars may have indentations.

Prestressing wire can be found with the following characteristics:

Designation	Nominal diameter (mm)	Nominal tensile strength f_{pk}
Y 1570 C	9.4-10	1570
Y 1670 C	7.0-7.5-8.0	1670
Y 1770 C	3.0-4.0-5.0-6.0	1770
Y 1860 C	4.0-5.0	1860

Table 3.4 Prestressing wire sections

According to the EHE-2008, the geometric characteristics will match those established by the UNE 36094:1997 standard for steel wire and strand for prestressed concrete reinforcement. Wire is the product used in pretensioned-prestressed concrete. The designation of prestressing wire will be according to this standard as follows:

- Reference to the product standard (UNE 36094:1997)
- Steel designation including:
    - Letter *Y* for prestressing steel
    - Nominal tensile strength in MPa
    - Letter *C* for cold-drawn wires
    - Nominal diameter
    - Letter *I* for indented wires followed by 1 or 2 depending on the type of indentation, in case of smooth wires this would be left blank.

For strands, the same designation applies, except the letter *C* is replaced by *S* followed by the number of wires shaping the strand (2, 3 or 7). The nominal tensile strength for strands according to the EHE-2008 should match the following tables:

Designation	Nominal diameter (mm)	Nominal tensile strength f_{max}
Y 1770 S2	5.6-6.0	1770
Y 1860 S3	6.5-6.8-7.5	1860
Y 1960 S3	5.2	1960
Y 2060 S3	5.2	2060

Table 3.5Prestressing strands: 2 and 3 wires

Designation	Nominal diameter (mm)	Nominal tensile strength f_{max}
Y 1770 S7	16.0	1770
Y 1860 S7	9.3-13.0-15.2-16	1860

Table 3.6Prestressing strands: 2 and 3 wires

Seven-wire strands are helically wound round a central straight wire, after this it can be drawn through a die and compacted, these are known as 'drawn' strands. The two cross sections are shown in the following image:

Figure 3.7 seven wire strands: wound and drawn.

The yield point for high strength steels is not as well-defined as that for mild steel, and so the *proof* stress is defined as the stress at which, when the load is removed, there is a given permanent deformation of 0.1 elongation.

The maximum stressing force, measured as the force applied to a tendon at the active end, should not exceed:

$P_{max}= A_p \cdot \sigma_{p,\ max}$

Being A_p the sectional area of the tendon and $\sigma_{p,\ max}$ the maximum stress applied to the tendon. This value will be the minimum of:

$k_1 \cdot f_{pk}$

$k_2 \cdot f_{p\ 0.1\ k}$

The Spanish Annex to EC2 (which is yet to be approved) provides a general value for those coefficients of k_1=0.70and k_2=0.85. These can be increased to 0.75

and 0.90 when the steel and the precaster possess a mark certificate. Temporary overtensioning is allowed if the force in the jack can be measured to an accuracy of ±5% of the final value of the prestressing force. In this case the maximum prestressing force may be increased to $k_3 \cdot f_{p0.1k}$, where k_3=0.90 or may be increased to 0.95 when both the product and process is mark certified. A practical example on the design of a pretensioned member and an estimation of the prestress losses is detailed in chapter 5. One of the factors contributing to the loss of prestress is the relaxation of steel.

The Eurocode defines 3 classes of relaxation:

- Class 1: wire or strand-ordinary relaxation
- Class 2: wire or strand-low relaxation
- Class 3: hot rolled and processed bars

Relaxation of steel defines the decrease in stress with time under constant strain. It reduces the prestress in the tendon with time, and so the study of relaxation is important in order to calculate the loss in prestress. Initial prestress force and the temperature are also relevant factors besides the type of steel. The relaxation loss may be obtained from the manufacturers' test certificates or taken from the estimated values given in standards. These values refer to a percentage ratio of the initial stress, at 1000 hours after tensioning and at a mean temperature of 20°C.In EHE-2008 (article 38.9) the required value is that of relaxation for an initial tensile stress equal to 70% of the characteristic tensile stress, for which the relaxation value should be 2.0. Eurocode 2, in section 3.3.2., provides expressions for the determination of the relaxation loss for the different steel classes, in absence of the value given by the manufacturer.

3.4 Production of precast-prestressed concrete

3.4.1 Stressing beds

Prestressing of beams and slabs is most economically applied in long-line precasting mass-production operations, where a higher efficiency allows for reduction in labor, formwork and hardware costs.

The simplest procedure is pretensioning with individual molds, where the tendons are anchored directly to the steel mold in which concrete is cast (figure 3.8). The molds must be designed to withstand the additional forces induced by the tendons. These are known as self-stressing forms and this solution is economical when the location of the prestressing force is not excessively high or eccentric.

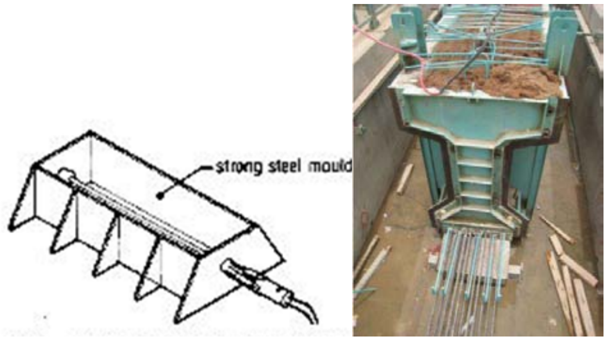


Figure 3.8 Mold method pretensioning

The system that is frequently used in the precast industry is the long line method (or Hoyer system), where end anchor blocks are kept a sufficient distance apart and several members are cast in a single line. Initially prestressing in beds was carried out in abutment beds where an anchor block was simply cast in the ground. Since back then the size of the members and the steel density was smaller, these abutments where sufficiently stiff to resist the stresses caused by the tendons. When large prestressing forces are required this method becomes very expensive because of the necessity of stiff and strong foundations for these anchor blocks.This is solved by connecting the two abutments by a full length concrete slab which is substantially thickened at each end to provide foundations for the support of these abutments (figure 3.9). The steel anchor blocks were initially cast in the concrete, and these were heavy members designed for the highest steel density across the bed which was a costly solution (Calavera, 2010). Today the common practice is the use of slots or trenches cast into the foundations so that the anchor blocks can be efficiently distributed across the prestressing bed. This provides a larger degree of flexibility.

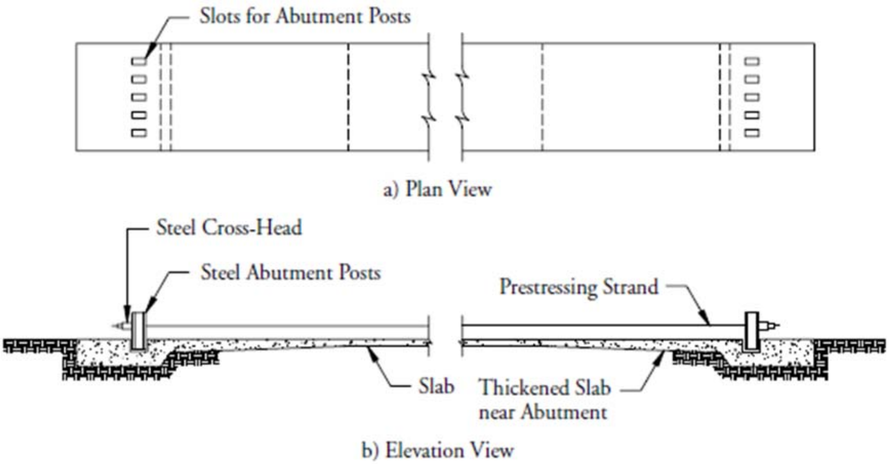


Figure 3.9 Prestressing bed with fixed abutments (PCI Bridge manual)

Another solution to avoid the need of stiff and strong foundations in prestressing beds is the adoption of a self-equilibrating system using a tension frame. This is known as a strutted bedin which thebed is designed to act as a strut without deformation when tensioning forces are applied.Steel shapes or prestressed concrete members running from end-to-end act as independent compression struts (figure 3.10) (PCI Bridge design manual).

In order to limit the amount of wasted strand when casting members that are shorter than the total length of the bed, another method includes slots or trenches between the ends. The anchor blocks can then be placed in intermediate positions limiting the amount of strands that need to be cut and later spliced to be used. These type of prestressing beds allow for the production of very different members (Calavera, 2010)

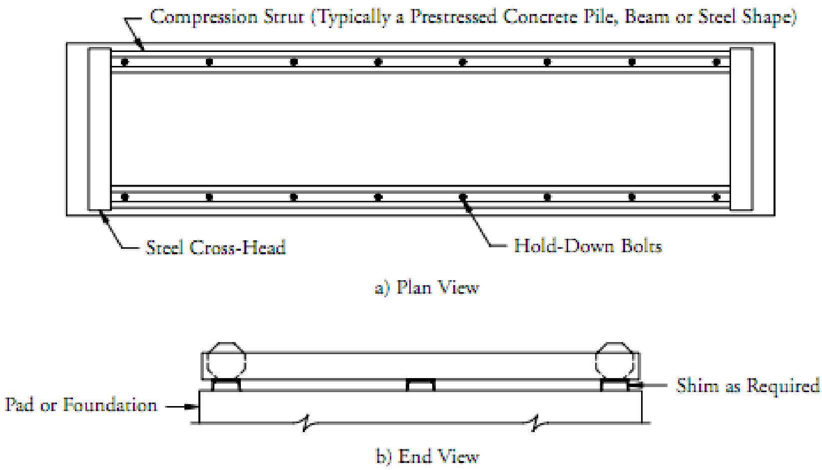


Figure 3.10 "Strutted" prestressing bed.

There are other accessories such as distributions or templates that are independent of the stressing hardware and employed to arrange strands from the configuration of the holes in the standard stressing hardware to the specific configuration required by the precast member. This must be done with a maximum deflection of 20° in order to avoid greater prestress losses(Calavera, 2010).

Other systems that are used mostly for the production of hollow-core slabs are the dry-cast or extrusion systemand slip-forming. In the extrusion system a very low slump concrete is forced through a machine in which tubes shape the cores of the slab.Due to the intense high frequency vibrations and pressure within the machine the concrete mix is 'plasticized' during the short time that it is passed through the extruder which allows it to be molded and formed into the required section.The slip-forming system uses a higher slump concrete. The sides are created by stationary forms (or by slip forming) with forms attached to the machine. Cores are typically created by pneumatic tubes attached to the form or by slip forming with long tubes attached to the casting machine, which travels along the prestressing bed (figure 3.11).

Figure 3.11 Top to bottom: extruding machine (Elematic) and slipformer (Echo engineering)

3.4.2 Strand profile

Straight pretensioning strands are the simplest solution and the most common arrangement in the production of members for the building industry, such as slabs. It is appropriate for moderate span simply supported beams. These members are wide and relatively shallow, and necessarily the eccentricity of the prestressing force is small. Otherwise the member's dead load moment at the ends is not enough to offset the excessive tensile and compressive stresses developed in that area.In large units, where the self-weight is considerable, it can be advantageous to increase the eccentricity of the tendons within the central portion of the span. The approach to control end stresses with straight tendons is to debond some strands at the end. This method can also be used to allow the casting of members that have different numbers of strand in the same bed and to prevent concrete bonding to strands placed for handling and shipping purposes.

Debonding is also known as shielding since it is usually done by placing a length of plastic tube or shield over the strand to prevent it from bonding to the concrete. Some recommended guidelines are: not to debond all strands in the bottom row and not to debond more than 50% of the strands below a dapped end or adjacent strands.

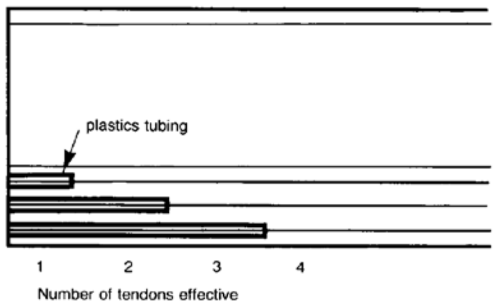


Figure 3.12 Debonded tendons (Allen, 1999)

When casting relatively deep members, such as I-beams, harped strands can provide a better correlation to the moment envelope. Hold-down devices are placed in the beds; they can be attached on the forms or directly to the concrete floor depending on the system used. Some examples of harping devices and the disposition of a pretensioning bed with deviation devices are given in figure 3.13 and 3.14 respectively.

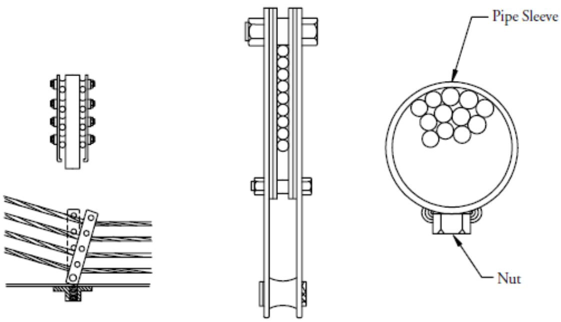


Figure 3.13 Harping devices (reference 19)

When using harped tendons the vertical component of the prestressing nonstraight tendon has a balancing effect when the prestress beam has to carry heavy loads. Since the required prestressing force for a harped tendon is smaller at midspan than the force required for a straight tendon, for the same stress level, a smaller number of strands are needed in the case of harped tendons. Sometimes even a smaller concrete section can be used, increasing the efficiency in the design.

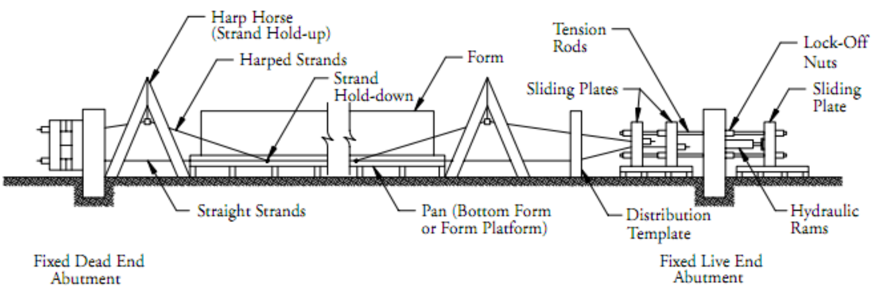


Figure 3.14 Typical pretensioning bed profile

The typical pretensioning set-up shown in figure 3.14 consists of a “live” end, from which the strands are jacked,and an opposite end called the “dead” end. When not using a bed with multiple lengths dispositions, the member is positioned in the line as close as possible to the dead end in order to reduce the amount of strand that is cut off and wasted. If the strands are going to be deflected a minimum dimension is needed to maintain a shallow slope (not greater than 20°) on the strands from the abutments to the endplate in the precast member. Since this type of disposition still leaves a considerable amount of free strand at the live end, some producers use “lead” strands that are spliced to the production strands and reused each day.

The temporary grips that hold the wires or strand during and after tensioning, usually consist of a barrel and wedge (figure3.15), which is the commonest system. This is generally shaped by two or three pieces and the wedge has grooves on the surface in contact with the tendon to improve grip. These grips are forced onto unstressed tendons close to the anchor plate and at the live end, where the prestressing is applied, these grips are simply placed. The jack is then positioned on the tendon and stressing begins, pulling the tendon through the grip. Once the required load and extension has been reached, the wedge is forced onto the tendon, the jack is released and the wedge is forced onto itself and firmly gripped as the tendon tries to pull through it. The strands can be tensioned individually or as a group.

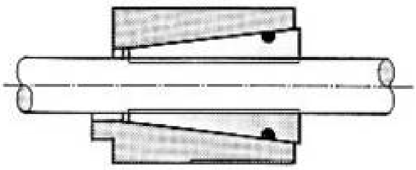


Figure 3.15 Grip assembly for pretensioning

When the concrete in the member has reached sufficient strength for transfer (after curing), a prefabricated “stool” is inserted between the anchor block and the jack(PCI bridge design manual). The tendon is jacked to its original force, allowing the barrel-and-wedge anchorage to be removed once relieved of its pressure. Then the jack pressure is released and the prestressing force is transferred to the concrete members along the prestressing bed. When using harped tendons, the strands are tensioned in the original straight profiles and then deflected and locked by a holding-down device. Otherwise the friction between the tendons and the holding-down devices must be taken into account.

3.4.3 Accelerated curing

In a precast plant it is desirable to achieve a daily production cycle for the installations to have a greater efficiency. Accelerated curing is usually the only way to achieve the necessary concrete's strength for prestressing release in the available curing period. The following example of a production cycle, which can be repeated on a daily basis, is given by Calavera:

- Prestressing bed cleaning — 0.5 h
- Setting and alignment of forms
- Release agent application to forms
- Placing of steel reinforcement — 3.0 h
- Stressing and anchoring
- Concrete casting and compaction — 1.5 h
- Initial set period — 2.0 h
- Steam curing — 6.0
- Release of prestress/strand cutting — 0.5 h
- Transport of members to yard storage — 2.0 h

TOTAL — 15.5 hours

Figure 3.16 Production cycle example (Calavera,2010)

Curing of the concrete takes place usually by covering to prevent loss of moisture and sometimes by the application of radiant heat or steam. This last procedure is the commonest since it is very flexible and has the advantage of providing a water-saturated environment. Steam curing allows the concrete to reach a strength that would take a week with regular curing.

The system consists of a steam generator and a network of pipes that go along the prestressing bed, releasing steam so that a uniform temperature is obtained. The materials used to "tent" the forms are polyethylene sheet or insulated tarp (PCI Bridge Manual). The treatment involves a preset or initial set period since the concrete is cast and then a period during which the temperature is increased at a rate of 20/30° C per hour. Once the maximum curing temperature is reached, this temperature is maintainedand then progressively decreased (Calavera, 2010). Before the concrete reaches room temperature the prestressing force is transferred to it, to compress the concrete and avoid cracking.

This heating process causes additional prestress force losses by steel relaxation and thermal expansion of the reinforcement, that have to be taken into account through the modification of the concrete's load age as described in the EHE 2008 standard, article 20.2.3. It provides expressions for a fictitious age adjusted to temperature and for the additional loss due to thermal expansion.

3.4.4 Removing products from forms

The procedure followed to remove the member from the form is referred to as "stripping" the products or "stripping the beds". The necessary lifting devices for each member, to keep stresses within allowable limits, are carefully designed and arranged during production. The Spanish standard EHE in its latest edition (article 59) covers the need for these temporary situations (such as the stripping of members, handling, transport and assembly) to be considered in the action's analysis and limit state verifications "*given the evolutionary nature of their construction, when designing precast structures and members*".

Usually the same devices are used for stripping the members off the forms and for erection. Orientation of components during storage, shipping and final in-service position is critical in determining stripping requirements.

3.4.4.1 Lifting devices

Lifting devices in precast/prestressed concrete members usually consist of strand lift loops, bolts or proprietary metal inserts. Strand lift loops are made of the same strand used in the production of the prestressed members, making it an economical option since it takes advantage of "waste" material. Some typical lift loop configurations are shown in figure 3.17. The surrounding concrete where the lift-loop is inserted should be reinforced to prevent splitting and loss of bond.Cast-in rope wires can be cut-off once the member is in its final position.

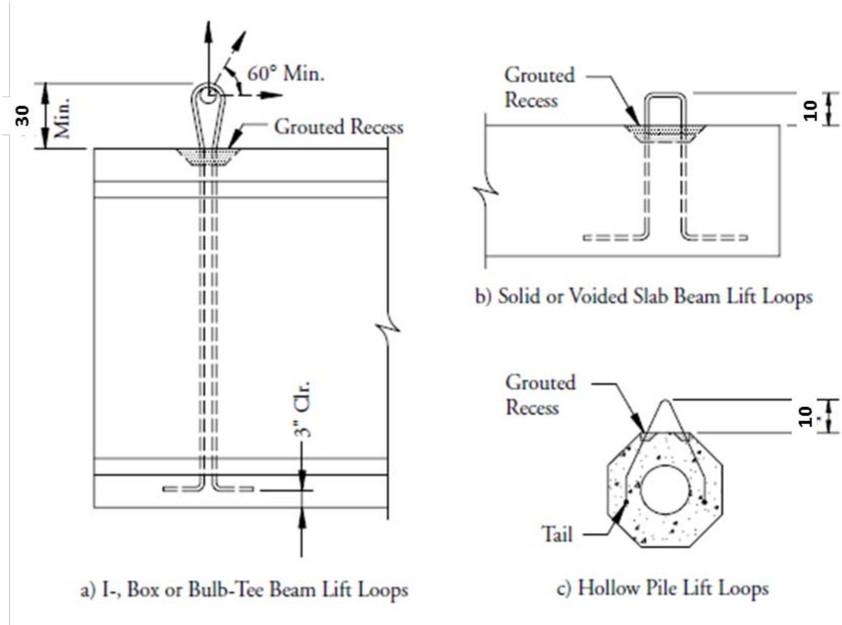


Figure 3.17 Typical lift-loop configurations (PCI Bridge design manual)

Other type of lifting devices involve casting a threaded insert in the concrete member, securely anchored, in which different lifting devices can be later attached. For example, swivel lifting eyes (figure 3.18) are installed that way. This type of device is more suitable for repeated use and is specially designed to allow angled lifts.

The stresses that the members have to bear during stripping, lifting, transportation and erection are carefully considered by manufacturers and usually factored in the member design. It is also very important to determine the force per anchor in order to provide the necessary lifting points. In this calculation both manufacturing plant and site conditions are taken into account. An example for a slab unit with two anchors is provided by Halfen (figure 3.19).

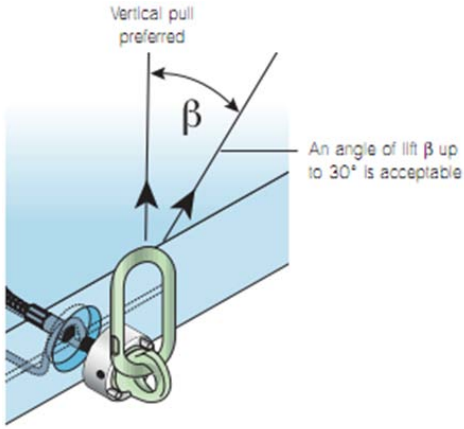


Figure 3.18 Swivel lifting eye device (Halfen)

Example: slab unit
Lifting, transporting in the plant and on site

	Manufacturing plant		Site
	Demould	Transport	Erect
G Mass	10,000 kg		10,000 kg
A Mould area	$20 m^2$	–	–
q Adhesion to mould	200 kg	–	–
f Crane factor	1.1	1.3	1.5
z Angle of pull factor	1.04 (β = 15°)		1.16 (β = 30°)
Concrete strength	$15 N/mm^2$		$35 N/mm^2$

With two anchors, the force **F** per anchor is as follows:

Demould at the manufacturing plant: $F = \frac{G + (q \times A) \times f \times z}{n}$ $= 10{,}000\ kg + (200\ kg/m^2 \times 20\ m^2) \times 1.1 \times 1.04/2$ $= 8008\ kg$

Transporting at the manufacturing plant: $F = \frac{G \times f \times z}{n}$ $= 10{,}000\ kg \times 1.3 \times 1.04/2$ $= 6760\ kg$

Erecting on the construction site: $F = \frac{G \times f \times z}{n}$ $= 10{,}000\ kg \times 1.5 \times 1.16/2$ $= 8700\ kg$

Figure 3.19 Anchor calculations for manufacturing and construction site (Halfen)

In larger members, to avoid excessive sling angles in lifting lines, spreader beams are used to allow for a better static weight distribution.This technique, as well as rolling blocks and lifting trusses, are necessary when multiple lifting points are used. These lifting systems are known as rigging arrangements. When using a spreader beam (figure 3.20), the component acts as a continuous beam with multiple supports; therefore consideration of lifting-hook locations, hook heights, and sling angles are critical to ensure even lifting of the component. To minimize concrete stresses due to the eccentricity of prestress, pretensioned flexural members are handled with lifting devices as close as possible to the location where the member will be supported in the structure.

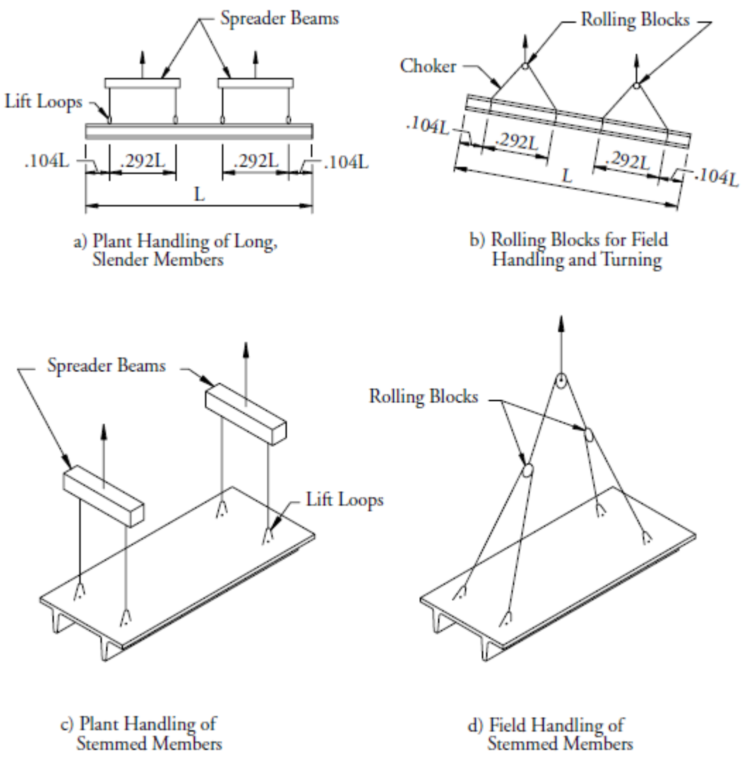


Figure 3.20 Rigging for multiple point lifting (PCI Bridge design manual)

3.4.4.2 Form suction

The most critical time in handling a precast member in the plant is when it is initially lifted from the form, since the concrete strength is lower and the prestressing force is higher than at any other time in the life of the member. Also careful considerationhas to be given to the amount of suction expected from the specific form. This effect has to be considered when choosing the crane, rigging and lifting device as well as in the determination of concrete stresses. It is common practice to apply a multiplierto the component weight and treat the resulting force as an equivalent static load. This multiplier is based on the experience and specific formwork of the individual precast concrete producers (PCI Design handbook). On pretensioned members whose sides have been removed suction is normally minimal since the elastic shortening and camber will usually break the bond between the remaining forms and concrete.

3.4.5 In-plant transport and storage

Usually products are moved to a designated finishing area after being stripped from the casting bed. A variety of equipment can be used for moving the products around the yard; the choice depends on each manufacturer. However the common cranes used to lift and transport the products in plant are usually overhead travelling cranes (bridge cranes)and gantry cranes (figure 3.21)

Figure 3.21 Gantry crane lifting a panel

Precast plants are normally designed in linear fashion to facilitate the most efficient movement of products from the casting bed to yard storage. This storage is usually located at the end of the production line, where access for trucks and moving cranes is readily available. The storage area should be provided of a sufficient sized foundation to resist crushing or excessive settlement.Wherever possible, a component should be stored on points of support located at or near to those used for stripping and handling. The importance in the design of storage relies mostly on the control of permanent concrete deformations rather than control of concrete stresses. Supports which cause no apparent initial damage can result in undesirable permanent deformations caused by creep of the concrete, while cracking and spalling are easily noticed.

Storing techniques depend also on whether the members are eccentrically prestressed or concentrically prestressed. In eccentrically prestressed flexural members an undesirable camber growth can result from storing them on supports a significant distance from the ends. Concentrically prestressed members, such as piles, have to be supported during storage at relatively short intervals along their length. These members carry a high level of prestress, and being long and slender, this can result in permanent deformations when stored with relatively large spaces between supports. When using multiple supports, care must be given to provide uniform support to the member, since differential settlements can have a substantial effect on both concrete stresses and permanent deformations. In two-point supports, differential settlements have no detrimental effect on concrete stresses as illustrated in figure 3.22.

Since yard storage is limited, precast members are stacked whenever possible, which is the case for shallow members. Deep flexural members such as I-beams are simply placed close to one another. Where units are stacked the timber spacing blocks used as support should be positioned vertically above one another. This is important because pre-tensioned single-span members cannot act as cantilevers, which they will try to do if the packing pieces do not line up.

The height between supports must be sufficient in order not to damage any projecting steel, such as stirrups, or lifting devices which also have to remain accessible.

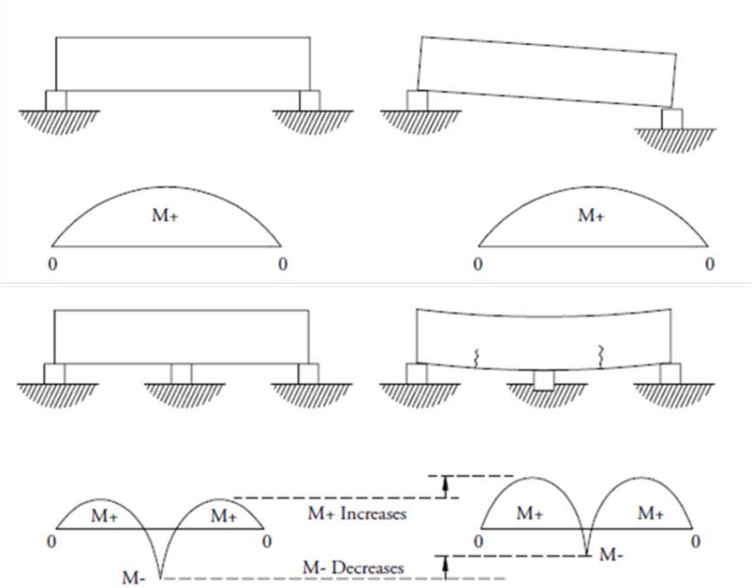


Figure 3.22 Product storage points, moment redistribution

3.4.6 Transportation

The most common system to transport precast/prestressed members to site is by truck:crane trucks of flat-bed trailers. Crane trucks are used for standard units, whenever possible. Trailer dimensions and hauling capacity will depend on the type of precast members to ship, but preferably a center of gravity as low as possible and the possibility to center the units is recommended. Steerable trailers are required when transporting large and long-pan members. The driver should be aware beforehand of any limitations regarding overheadhigh-voltage lines and the size restrictions for transport and special permits if required for overloads. The precaster must ensure that the elements are loaded in a sequencecompatible with the required unloadingsequence on site.

The supports on the truck should be located as close as possible to the lifting devices. However impact during transportation is a factor that must be considered by the manufacturer when checking concrete stresses.

Once on site there should be a designated area were the ground has been compacted in order to facilitate truck circulation. Precast members must be handled following the manufacturer's instructions, using the adequate lifting devices and support arrangement for storage/stacking when necessary as it was done during production.

The reception of precast products on sitewill be detailed in the following sections.

4 Precast/prestressed structures

We will now detail the specific building process and reception of units for precast/prestressed structures. There are many structural options for the available precast prestressed units, which can be combined in several ways with many different materials and structural systems, but since this report is based on precast/prestressed concrete members we will focus on the usual systems where these are more frequently used.

4.1 Usual sections of precast/prestressed products.

There use of prestressed concrete members to provide structural solutions allows producers to develop very different systems that cannot possibly be described in this report. Also even though precast and prestressed concrete components can be manufactured in a variety of customized sizes and shapes, maximum economy is achieved by using the common products that have evolved in the industry. We will focus on the most usual sections and structural systems available.

Some typical prestressed units are shown in figure 4.1. Among the most prevalent of these products are prestressed hollow-core slabs and joists. Prestressed beams and purlins are a frequent solution for industrial warehouse construction where totally precast structures have been traditionally used.The ability of prestressed concrete to span long distances with shallow depths and carry heavy loads is particularly suitable for warehouses and industrial buildings

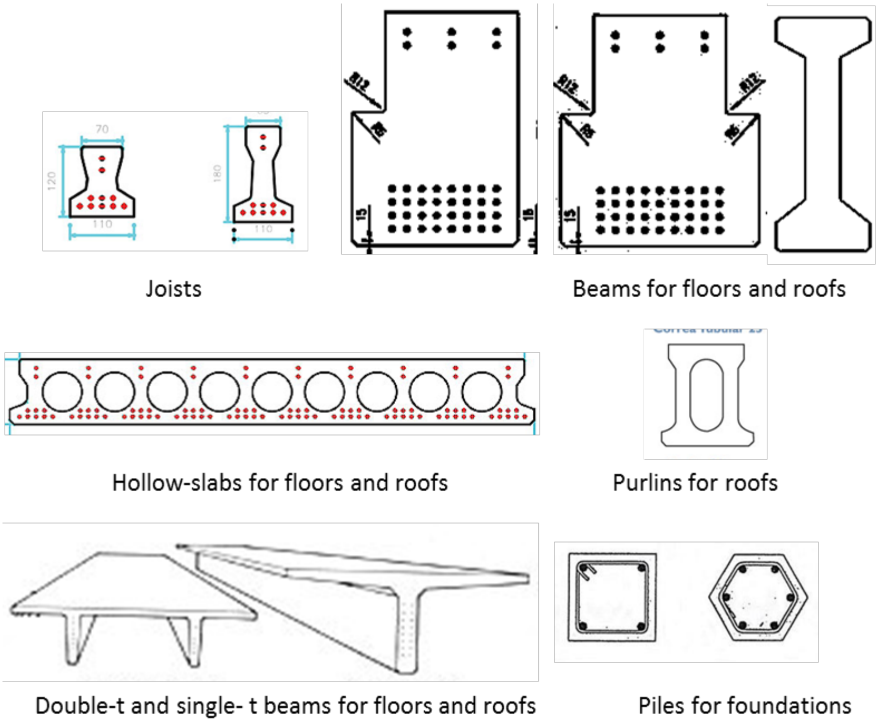


Figure 4.1 Typical precast/prestressed sections

4.2 Advantages of precast construction

As we mentioned previously the use of precast products is regarded in the construction standards as an advantage towards reducing some of the coefficients used in design, but there are other specific advantages of precasting that are easily noticed on site. Some of these specific advantages are (Gerwick, 1993):

- **Control of shrinkage**: the use of lower water to cement ratios in precast concrete and steam curing, as well as proper aging and drying before erection, may reduce shrinkage and allow for it to take place before being part of the building.

- **Reduction of creep**: again, higher strengths concretes and proper curing may produce concrete of reduced creep characteristics.
- **Quality control**: the factory production of precast concrete elements is inherently manufactured under the best conditions for forming, placement of reinforcement and concrete, vibration and curing. The accuracy and uniformity of concrete cover makes it possible to assure greater durability. It also permits the application of special manufacturing techniques that cannot be applied to cast-in-place concrete. Prestressing steel and inserts may accurately be positioned and held during concreting. Dimensional tolerance control is facilitated and more easily corrected.
- **Timely availability**: most plants are able to produce to order at the desired rate and even meet accelerated schedules. Many standardized, mass produced elements can be furnished to a construction size on very short order.
- **Reduction in site construction time**: actual erection time and site construction time may be efficiently reduced. In favorable circumstances, the floor of a building may be erected and jointed in two days.
- **Economy**: mass production of standardized elements reduces the cost of forms and manufacturing labor and site labor can be kept to a minimum. Since higher strengths and thinner sections are used, deadweight may be reduced, resulting in overall savings in concrete and steel or the possibility to add extra floors.
- **Suitability for composite construction**: precast members may frequently be combined with cast-in-place concrete to improve the monolithic behavior of the structure. The units may serve as partial forms or support for forms for the cast-in-place concrete.
- **Reduced maintenance**: the higher quality materials and control used in precast concrete increase durability and reduce the need for maintenance.

One of the greatest advantages when using precast products is that usually these products are in possession of officially recognized quality marks. For these marks the EHE-2008 standard regards special considerations that may be applied by the project management. Also CE marking is mandatory for precast concrete structural products and this guarantees that the minimum requirements laid by standards are covered, meaning a quality assurance and a reduced need for control checks during the building process. We will now describe the situation regarding CE marking (mandatory) and officially recognized quality marks (voluntary).

4.3 Quality assurance of precast products: marks

4.3.1 CE Marking

According to the Guidance paper D on CE Marking under the Construction Products Directive (CPD, 89/106/EEC) by the European Commission:

"[…] In the case of the CPD, theCE marking indicates that the product complies with the relevant national standards transposing the harmonized standards, or a European technicalapproval, or one of the national technical specifications referred to in Article 4 (2.c), ***and*** *that the system of attestation of conformity laid down in the Commission Decision relating to the product has been applied."*

It is not a mark of origin or a *quality mark*, it symbolizes that the construction products have been assessed for characteristics which have an influence on the satisfaction for the essential requirements for the works and are in compliance with the harmonised part of European standards. Each standard contains an annex ZA, detailing several tasks for the manufacturer and for the Notified Body in order to carry out CE marking.CE marks can be based on either a compliance with a harmonized European Standard (hEN) or a European Technical Approval (ETA).

The CPD aims to break down technical barriers to trade in construction products between Member States in the European Economic Area (EEA). To achieve this, the CPD provides for the following four main elements:

- a system of harmonized technical specifications
- an agreed system of attestation of conformity for each product family
- a framework of notified bodies

- The CE marking of products.

Any product that will be permanently incorporated in construction works must assess for certain characteristics which have an influence on the satisfaction of the following essential requirements for the works:

1. Mechanical resistance and stability
2. Safety in case of fire.
3. Hygiene, health and the environment.
4. Safety in use.
5. Protection against noise.
6. Energy economy and heat retention.

These essential requirements (aimed towards not endangering the safety of persons, domestic animals, property and/or the environment) are performance based, since no technical provisions regarding specific design or construction tips are given. According to this Directive, any product that may affect at least one of those essential requirements is subject to CE marking. This guarantees that the product fulfils the specific technical criteria agreed by the European countries for that purpose.

There is an agreed system of attestation of conformity for each product family that relates to the degree of involvement of third parties. There are six systems of attestation under the CPD going from the least onerous 4 of manufacturers' tasks only, to system 1+ of product conformity certification with audit testing. Structural precast products are certified with system 2+ that involves factory production control certification with continuous surveillance. A notified body is involved apart from the manufacturer's tasks.Notified attestation bodies are the product conformity certification bodies, factory production control (FPC) certification bodies, and test laboratories that are competent to carry out the attestation tasks to establish product conformity. There is an external body certifying the conformity with the CE mark and a manufacturer's declaration of conformity.Therefore it is safe to assume that the products leave the factory satisfying all the requirements that the specific standard requires. The attestation tasks for system 2+ are given in table 4.1.

System	Tasks for the manufacturer	Tasks for the notified body	Documents
2+	Initial type testing Factory production control (FPC) Further testing of samples taken at factory	Certification of FPC on account of: -Initial inspection of factory and FPC -Surveillance of FPC	CE marking label Declaration of conformity to CE mark from the manufacturer FPC certificate by the notified body

Table 4.1 Tasks for system of attestation 2+

The label for the CE mark must be in compliance with directive 93/68/CE and contain the following information:

- Identification number of the certification body
- Name or identifying mark and registered address of the manufacturer
- Last two digits of the year in which the marking was affixed
- Number of the FPC certificate
- Reference to the European standard
- Product description: generic name and intended use
- Information on regulated characteristics required in Z Annex of the corresponding European Standard.

This information must be provided by the manufacturer in all cases, but a simplified label which contains basic information may be used during the supply.

The EHE 2008 standard allows the precaster of a product with a CE mark to use a material coefficient in the design of 1.70 for concrete and 1.15 for steel. The next guarantee level (for products with CE mark) that the manufacturer may apply optionally is to produce concrete according to the requirements given by the EHE standard in Article 86.9 and certify it by an authorized third-party. In this case the concrete's coefficient may be reduced to 1.50 (for steel 1.15).

The documents that the producer of precast/prestressed structural products must provide to the project management/client, before the supply, are:

- CE Label
- Technical information (product catalogue): optional and according to the particular standard.
- Declaration of conformity to CE mark from the manufacturer (optional)
- Copy of the FPC certificate by the notified body.
- FPC certificate that proves compliance with EHE 2008 standard regarding the production of concrete: this is optional for CE marked products, it is certified by an authorized body and reduces the concrete's coefficient to 1.50.

During the supply, each batch of products must be provided with a document containing information such as the manufacturer's details, CE mark certificate number, date and time of delivery, product description (identification, materials and amount supplied) and place of supply.

According to Annex 19 of the EHE standard:

"The Manufacturer of any product, the Person responsible for any process or the Constructor may, voluntarily, opt for a quality mark that ensures a guarantee level that exceeds the minimum requirement laid down by this Code. In the case of products with a CE marking, said quality marks must bring added value with regard to characteristics not covered by CE marking."

Officially recognized quality marks are frequent in the precast/prestressed structural members' industry, since the advantages to both producers and clients are a powerful marketing tool.

4.3.2 Quality marks.

The conditions that allow differentiation when there is an additional guarantee level to the minimum required by regulation are laid down in Annex 19 of the EHE 2008 standard. These are general requirements in order for a quality mark to be officially recognized. There are specific requirements for precast elements (point 5.3) that deal with many aspects, including the traceability of the products, installation and factory production control requirements and quality surveillance in general.Most of precast/prestressed products are manufactured under a quality mark and this allows the Project Management to apply special considerations.

Some of the requirements that a manufacturer has to fulfill to be awarded a quality mark are:

- The production installation shall have implemented a quality system audited by an authorized certificatory body according to the standard UNE-EN ISO 9001, where applicable.
- The continuous control of production and product through a laboratory, being own or contracted.
- Have defined and implemented a continuous production control in factory, whose data from six months before the concession has to be available.

And particularly for precast products, some of the requirements are:

- Guarantee the requirements laid down in this Annex for the installations for producing the component elements (concrete, passive and active reinforcement, etc.)
- Guarantee that the prefabricator has a fixed concreting installation and a workshop for passive reinforcements capable of producing all the materials necessary for the manufacture of the precast elements
- Having a data management system of the concrete plant in order to supervise in real time the production of concrete.
- Statistic control of the concrete’s strength: more determinations than the minimum required and external control of the values of strength obtained.
- Use appropriate systems so that traceability is available for both the materials used and the precast elements themselves.
- For precast units for one-way floor slabs: provide a specification sheet and the corresponding Memory of Calculation of floor slab

systems in which the products might be used. This specification sheet shall include, among other, the following information:

- All the geometric and mechanical characteristics of the constituting systems and elements considered by the producer as useful in order to check their compliance with the EHE standard.
- Geometric characteristics and weight (per meter or square meter depending on the element) and of its constitutive elements if they are not included in the CE mark.
- The designation of materials used, giving the design strength, elastic limit or maximum unit load for each, if necessary according to this Code.
- The mechanical characteristics of the resistant elements considered as isolated, indicating the maximum resistant moments on secondary supports and at midspan. In case of prestressed elements: the bottom resistant module, the stress due to prestressing in the top and bottom fiber, and the value of the prestressing force multiplied by the eccentricity of the equivalent tendon relative to the section's gravity center, will also be indicated.
- The mechanical characteristics of the different types of floor slabs defined in the specification sheet, in both negative and positive bending, indicating the ultimate bending and cracking moments, gross and cracked stiffness, the limit moments in service according to different exposure classes and ultimate shear condition. The values for stiffness and cracking moment will be calculated at an age of 28 days indicating the coefficients to obtain these values at a different age.

One of the greatest advantages of having a quality mark is the possibility of reducing the safety factors for materials, as the following table shows:

Precast concrete products	Safety factors	
	CONCRETE	STEEL
	γ_C	γ_S
With CE marking	1.70	1.15
Concrete control according to EHE-08, attestation of conformity by a certified body: Voluntary for CE marked products Mandatory for products without CE mark	1.50	1.15
With a quality mark according to Annex 19 and when the construction of the structure is closely controlled (according to chapter XVII)[1]: Concrete: deviations in the geometry of the cross-section in relation to the nominal cross-sections comply with those mentioned in design (also as required by Annex 11) Steel: Attachment tolerances for the reinforcement comply with those laid down in design (and also Annex 11) and/or That the steel for passive reinforcement bears a quality mark	1.35	1.10
[1]According to the commentaries in Article 15.3 for the modification of the partial safety factors to the lowest values 1.35 and 1.10, the manufacturer of precast members should provide instructions for the assembly of the units so that the construction of the structure can be closely controlled.		

Table 4.2 Safety partial factors for different guarantee levels

Another advantage regarding the use of materials is that the tensioning force in active reinforcements can be a 5% increased. The stress provided by the tensioning force $P_{0,}$ at any point, must be the lowest of the following values:

Permanent situations: 0.70 f_{pmaxk}; 0.85 f_{pk}

Temporary situations: 0.80 f_{pmaxk}; 0.90 f_{pk}

When the precaster has a quality mark for the product:

Permanent situations: 0.75 f_{pmaxk}; 0.90 f_{pk}

Temporary situations: 0.85f_{pmaxk}; 0.95 f_{pk}

Being $f_{pmaxk}$the characteristic maximum unitary load, and f_{pk} the characteristic yield strength.

During the building process, the conformity control of the precast units prior-to and during supply is simplified. The checking of installations of the precast elements required by article 91.4.2 (EHE-08) can be dismissed by the Project Manager since it is already covered by the Certifying Body for the quality mark. Also the Project Management can exempt the precast units from some of the document checks (prior to supply) such as the weld type approval certificate and certificates guaranteeing the appropriate training of the staff carrying out non-resistant welding of passive reinforcement. Among the tests for checking the conformity of the materials used, in case of precast elements with a quality mark, the Project Manager is exempted from checking samples taken in the precasting installations.

4.4 Precast/prestressed structures

4.4.1 One way slab and floor systems.

Prestressed concrete is a technology related to members in flexion and in case of pretensioned units, these are usually simply supported at the ends. Therefore the most common use for precast/prestressed units in the building industry is in the construction of floor and roof structures working in a one-way span.

4.4.1.1 Joist and filler block floor systems.

This floor system is probably the most familiar use of precast/prestressed products in Spain. It is the one traditionally used for the construction of residential buildings and therefore the most suitable option in order to easily find trained professionals. It does not require any special teams or skills and that is part of its popularity as a floor and roof system.It is shaped by joists that span between girders, hollowcore filler blocks, and extra reinforcement (for negative moments and load distribution) and concrete cast on site. Annex 12 of the EHE 2008 standard provides specific design and construction criteria for this type of slabs.

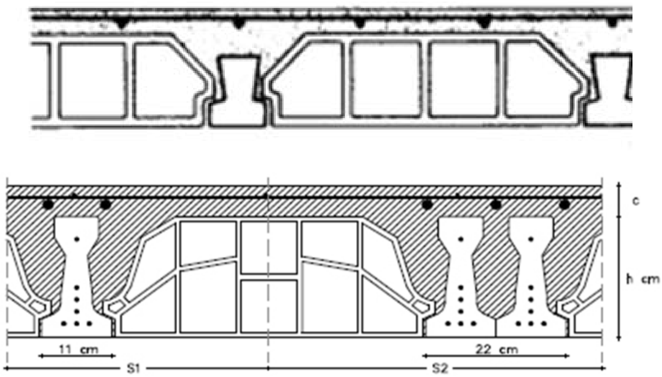


Figure 4.2 Top to bottom: one way slabs with semi-resistant joists and self-bearing joists

There are two types of prestressed joists employed, depending on whether the floor is totally precast or partially precast (figure 4.2). In partially precast floors, precast elements provide a partial resistance that has to be completed with cast-on-site concrete so that the slab can bear the loads it is designed to resist, during all the construction stages. The joists used in this system are known as *semi-resistant joists*, these prestressed/precast units have an inverted t shape and a limited flexural strength that requires propping (typically every 2.5 m) in order to resist loads during construction and as long as the system has not been completed with the concrete poured on site.

When the floor is totally precast, it means that the precast joists used are able to withstand the loads that the slab has to resist during construction, without the need to be propped (for up to 5 m spans). These prestressed joists have a double-t shape and are known as *self-bearing joists*. This system is mostly used in the construction of insulating suspended ground floors, where propping/forming is not possible due to the limited space. In “regular” floor and roof slabs the partially precast system with semi-resistant joists is the usual choice, due to tradition and since it provides a higher degree of monolithism on its own. Also the safety criteria require forming the whole surface of the slab or installing nets in this type of floors so the advantage of the self-bearing properties of joists is sometimes irrelevant (figure 4.3) if forming is going to be arranged.

Figure 4.3 Semi-resistant joists slab with safety net and propping

The filler blocks that are placed between prestressed joists to work as forming for a topping concrete slab can be made of concrete, ceramic or styrofoam. They can either be resistant or not, and the design criteria for calculation is given in Article 36 of the EHE 2008 standard. The shape of these hollow-core blocks varies according to manufacturers and also the system they are employed in. For precast/prestressed joists the walls of filler blocks in the ribs usually have an open profile so it does not reduce the shear capacity. Also in case of self-bearing joists, the shape of the infill blocks must allow the double-t shape of the joists to fit in the ribs (figure 4.3). The minimum concrete topping over joists and resistant filler blocks can be reduced to 40 mm or 50 mm when blocks are not resistant (Styrofoam) or when the building is in a zone with a seismic acceleration > 0.16 g (Calavera,2008).

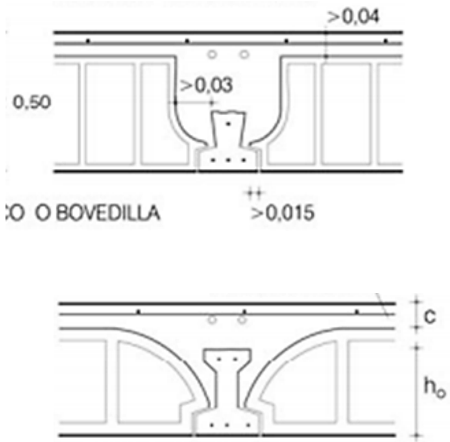


Figure 4.4 Filler block cross sections and rib dispositions (FECEA)

The connection between joists and the supports are classified as direct or indirect, depending on whether the shape and depth of the supporting member allows for the introduction of the joists or not. The minimum anchoring lengths for l_1 and l_2 (see figure 4.4), for prestressed joists, should be **l_1> 100 mm** for single supports and **l_2> 60 mm** for supports receiving two opposite joists. In case of beams that have the same depth as the floor slab another solution is to add extra reinforcement that can efficiently connect the precast joist with the cast on site concrete of the beam to complete the anchorage lengths above. Some usual support layouts given in the EHE standard are given in figure 3.5. These anchorage lengths are measured from the beam's concrete face in direct supports and from the links in case of indirect supports.

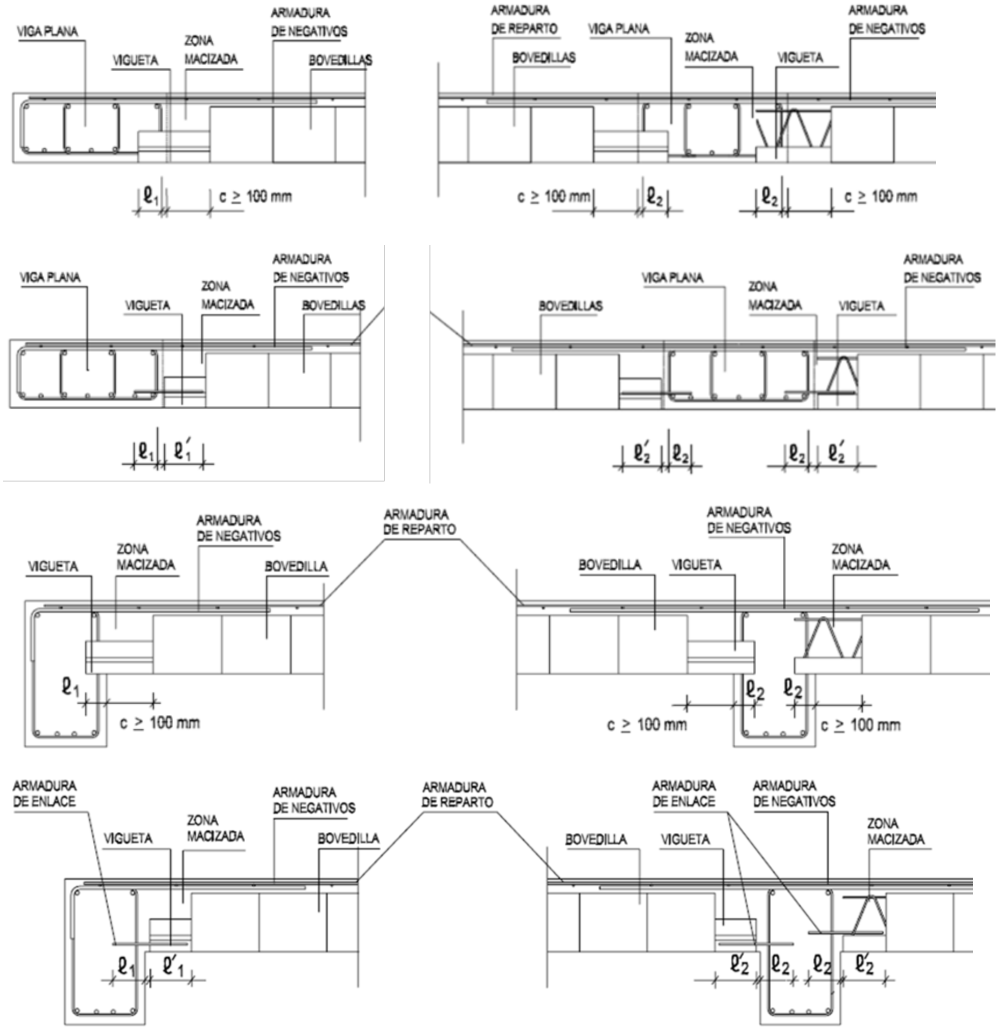


Figure 4.5 Support layouts given in the EHE standard for precast-prestressed joists

In the alignment of ribs a deviation smaller than the straight distance between faces in inner supports and 5 cm in cantilevers is allowed. In case of perpendicularly opposed floor slabs such as in cantilevers, the top reinforcement should be anchored in a length not smaller than the length of the cantilever or twice the joist inter-axis, as in the following layouts:

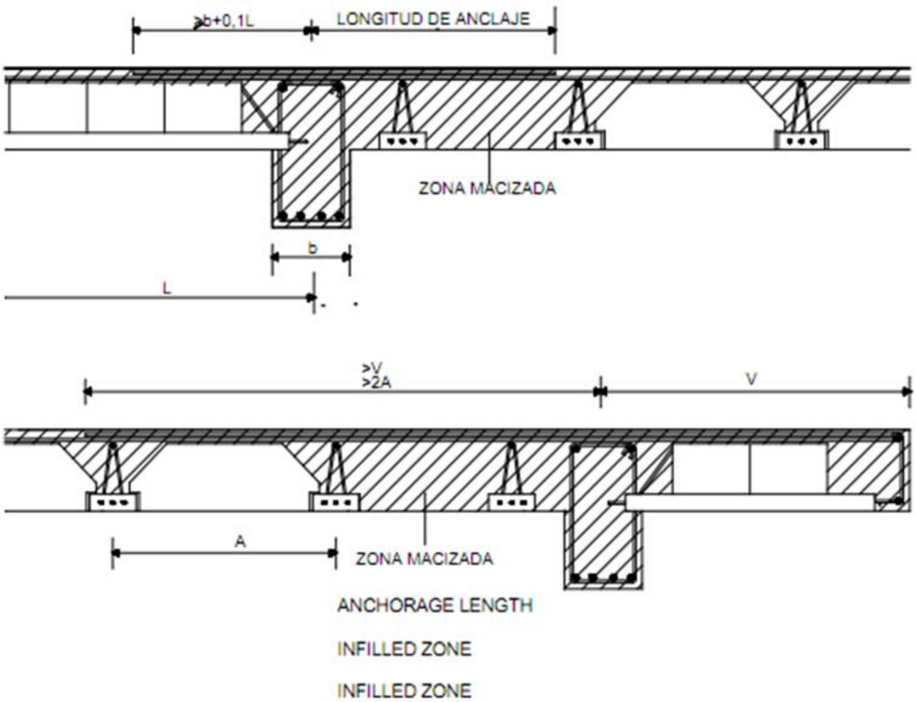


Figure 4.6 Opposed ribs and cantilevers.

3.4.1.2 Prestressed hollowcore slabs

These types of totally precast floor solutionsare usually employed in office buildings, shopping centers or wherever the advantages of a quick erection and no need for propping the floor are desirable. The voids in these precast units can be used for installations and even to regulate the internal temperatures of the building as part of a system for air circulation (such as Termodeck).The webs of the slab are punctured at various positions to create one continuous tube snaking along the floor slab.

It may be used along with a layer of concrete cast on site, to resist compressive stresses and increase the monolithic properties of the floor, or simply filling the joints between slabs (figure 4.7), depending on the requirements and loads for the structure. The in-situ cast concrete may be eliminated when compliance with the Ultimate and Serviceability Limit States is evidenced, in which case in order to ensure transverse transmission of loads and combined working of the slabs, a tie must be fitted in the zone where the slabs are connected to the main beams or walls acting as supports (EHE, Article 59.2.9). Perimeter reinforcement and grouted joints allow the hollow-core slab to work as a diaphragm.

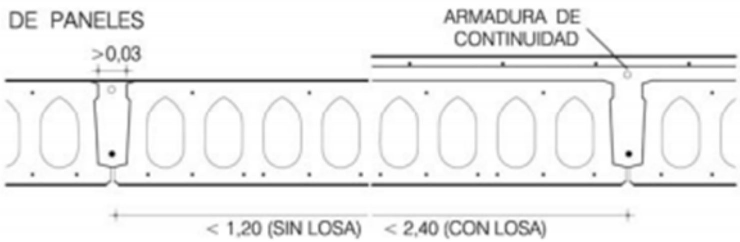


Figure 4.7 Hollow-core slabs with joint filling and concrete topping.

The maximum widths of 2.40 m and 1.20 depend on whether there is a concrete topping or not, and also these dimensions are related to the fact that 2.40 m is the maximum width a truck can transport without special arrangements or permits. The depth/span ratio is usually between 1/25 and 1/30. The extra reinforcement in joints between slabs is also provided to fulfill the fire protection requirements in the CTE standard. When there is no concrete topping, the top reinforcement required to resist negative moments at supports and for continuity, is cast in the joints, therefore using more slabs instead of wider slabs allows more space for this extra reinforcement. These joints also transmit the shear force between slabs, and the dimensions required by article 59.2.1 e) of the EHE standard are:

- A minimum top width of 30 mm
- In the area of the reinforcing bar, the width should be the greater of ϕ +20mm and ϕ+ 2D.

When the reinforcement in joints is not sufficient, the bars can also be concreted into the voids through openings cut on site or provided during production of the hollow-core slabs (figure 4.8).

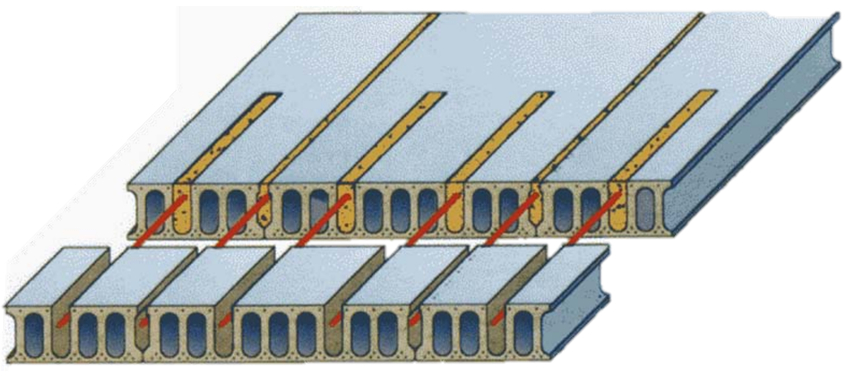


Figure 4.8 Reinforcement in joints and voids for slabs without concrete topping (AIDEPLA)

The use of these structural members is more efficient as a totally precast floor, since it barely requires any other materials on site except for the concrete poured in the joints between slabs. It is a faster process for completing a floor or roof since a single unit covers a larger surface in less time than joist and block systems. Transport, unit reception and storage are also simpler since a single element is shaping the floor, unlike joist and filler block floors where several precast units and on site materials are required (*Boletín*AIDEPLA, 2006). The single source of responsibility in this type of floors, usually combined with other precast concrete elements, makes project management and construction of these buildings much easier and eliminates the cost and schedule risk created by having many different material suppliers all working on the site at one time.

The supports for prestressed hollow-core slabs are detailed in Annex 12 just like in joist floors and can also be divided in direct supports and indirect supports. In case of direct supports there are two cases for the minimum bearing length l_1 (measured from the edge of the slab to the inner edge of the support). According to the following criteria:

a) If all the following conditions are simultaneously met:

- the design loads are distributed and there are no significant point loads or major horizontal loads, including seismic loads,
- the overload is equal to or less than 4 kN/m^2
- the depth of the hollow-core slab is equal to or less than 30 cm, and
- the design shear V_d is less than half that withstood by the prestressed hollowcore slab V_{u2}according to Article 44.2.3.2

$$V_d \leq V_{u2}/2$$

The nominal minimum bearing **l_1 will be 50 mm**, on which a tolerance of -10 mm is permitted so that the actual bearing in situ will never be less than 40 mm.

If any of these conditions are not met, passive reinforcement can be provided in the joints between adjacent slabs or in the voids as previously described.

Indirect supports depend on whether shoring is required for the execution of the connection (if the slabs rest on the supporting element or just the connecting reinforcement reach the support). The following details are provided in the Annex:

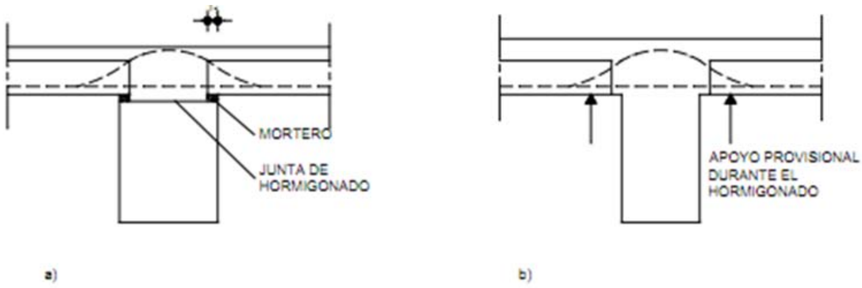


Figure 4.9 Indirect supports with connecting reinforcement in joints or voids: a) without shoring, b) with shoring

In case a) without shoring, the minimum value of l_1 will be 40 mm with a tolerance of ±10 mm. These indirect supports must be designed and checked according to other aspects detailed in the EHE standard, and it is generally recommended to tie opposite slabs with passive reinforcement that crosses the support and is properly anchored on each slab, whether it is placed in the site-cast concrete slab or the joints/voids of the hollow-core slab. If the hollow-cores are going to be infilled, at least two voids must be concreted with reinforcement if the slab is wider than 60 cm.

The connection of slabs with supporting members will be detailed in the project, and during the design process care should be given to avoid unwanted constraints in hollow-core slabs as detailed in point 9 of Annex 12.

Further examples of connection of hollow-core slabs with different supports are given in figure 4.10. The criteria for connection of precast concrete members are detailed in article 59 of the EHE standard.

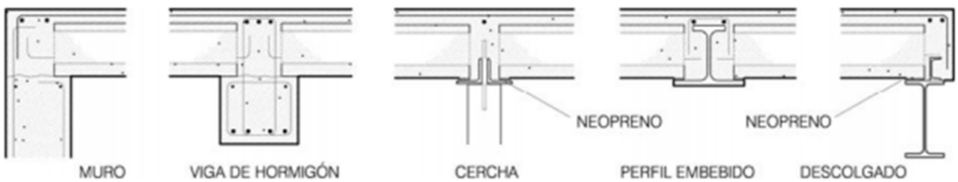


Figure 4.10 Hollow-core slabs supported in different structural elements.

4.4.1.3 Double-t slabs

These precast-prestressed concrete members can be used for most applications requiring a long span floor or roof system and/or additional load carrying capability, such as parking garages, office buildings, commercial buildings or industrial buildings. The use in residential buildings is not very common in Spain, although it is a good totally-precast solution for peaked roofs were its self-bearing properties and the possibility to install roofing tiles without any concrete poured on site can be an advantage. It is a simple solution for simply supported members where continuity is not required. The primary tension and shear steel reinforcement consists of prestressing strand, placed longitudinally in each stem, and mild reinforcement.

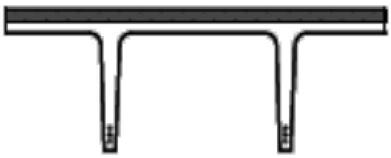


Figure 4.11 Prestressed double tee section with topping.

As it happened with hollow-core slabs, these units can be installed with or without a layer of concrete poured on site. Also the manufacturer can provide pre-topped slabs. As a totally precast floor and roof solution it is frequently used in parking structures as part of a precast concrete column-beam-deck framing that is ideal for the requirements of these structures (figure 4.12). The ribbed finishing of these units may be a disadvantage compared to the flat surface on hollow-core slabs, but the space between ribs can be used to suspend installations.

Figure 4.11 Typical precast concrete parking structure with double tees.

As a simply supported element these members can be connected to concrete beams (precast or cast-on-site), concrete or brick walls and steel beams. The usual practice is to provide a direct support of 7 cm (product brochure, Extremadura2000 Estructuras), and it is a floor system most usually combined with precast concrete beams and columns.In order to adjust the width of the finished slab, double tees are sometimes produced with dapped ends so that the beam end can sit flush on the bearing seat.

These slabs are also used for sloping roofs in industrial buildings in combination with other precast concrete members. They are manufactured according to the EN 13224 standard regarding the required CE mark.

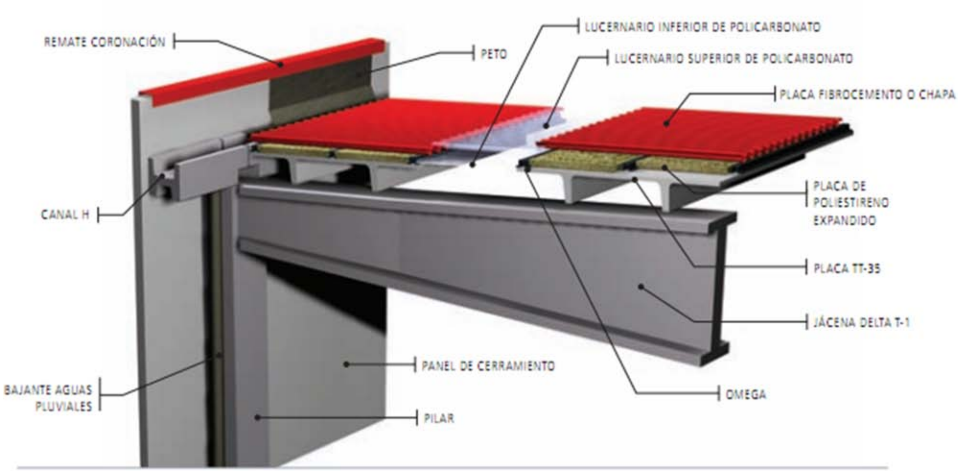


Figure 4.12 Double tee slabs for sloped roof construction (Trumes)

4.4.1.4 Prestressed concrete slabs

Another group of prestressed concrete members to produce one-way slabs are shaped by precast-prestressed slabs with an inverted-t shape, of prestressed ribs and in-fill materials such as clay or polystyrene blocks, or voids to be filled up by concrete cast on site. These are also self-bearing elements that may not require propping, for up to a certain span, and allow a fast assembly on site. The system is completed with cast on site concrete and passive reinforcement with the advantage that there’s a possibility to build in-situ beams by removing some of those blocks and placing the required reinforcement without the need of extra forming. It may be considered as a resistant forming system for a light-weight cast on site slab.

As a partially precast floor system, these units are lighter than other solutions such as hollow-core slabs and also provide greater monolithic properties since they are completed with cast-on-site concrete. The flat bottom surface and concrete topping allow for easier supporting conditions and continuity of members when required.

Figure 4.13 Prestressed slab (HERMO S.L.)

4.4.2 Prestressed beam and column framings

Totally precast framings are a typical structural system in industrial buildings and other large projects such as schools and commercial buildings. Prestressed members such as beams, girders and purlins are in this category.

Prestressed concrete girders can be totally precast, for simply supported solutions, or be manufactured with top reinforcement to be completed with concrete poured on site to allow for the connection with different precast or site built floors. Prestressed girders have a wide variety of sections, depending on the manufacturer or framework that they will be part of. Some usual sections are in the following image.

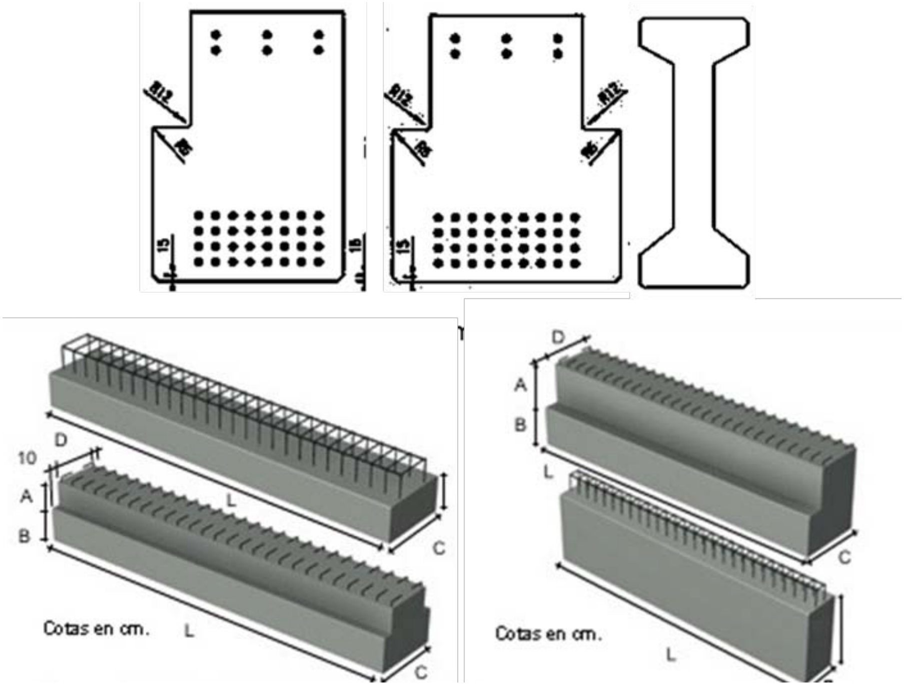


Figure 4.14 Prestressed girder sections.

The harmonized European standard for these elements is UNE EN 13225:2005, and as all other precast structural members they require CE marking.

The most frequent use in Spain of prestressed beams combined with precast columns is probably in industrial buildings such as warehouse construction that are usually sorted out with totally precast structures. So these framings are usually combined with other precast-prestressed floor units such as hollow-core slabs and double tees. The columns to support prestressed beams in totally precast structures are provided with corbels to provide a seat for the beams, as in the following image.

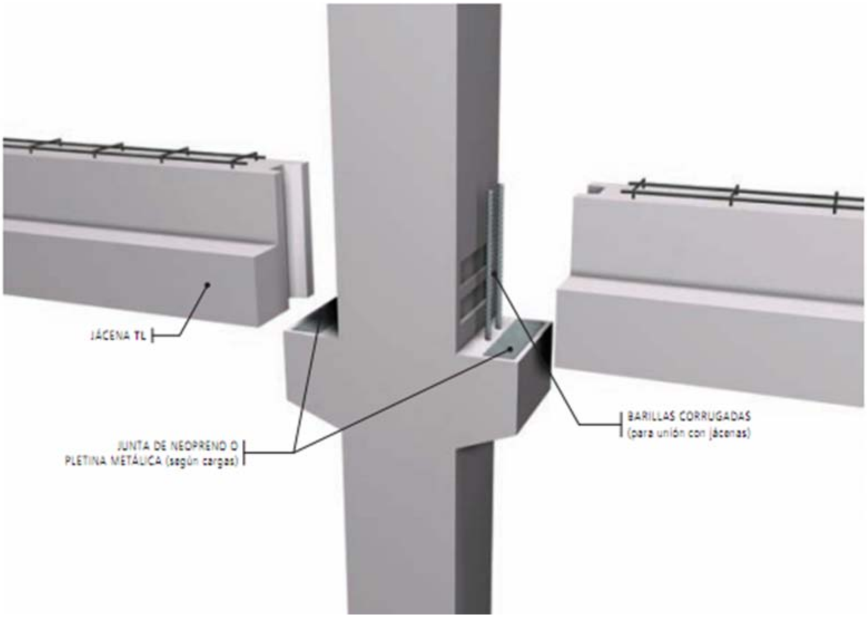


Figure 4.15 Precast concrete column with corbel for girder connection (with neoprene bearing pads and steel bars for connection)

In figure 4.15 the columns are continuous and the movement between girder and columns is limited by two vertical steel rods shaping an isostatic connection. Prestressed purlins combined with precast girders and columns are also a common solution in the construction of roofs for industrial buildings (figure 4.16).

Figure 4.16 Prestressed purlins in combination with precast beams for a peaked roof.

In residential building structures where continuity is necessary and rigid connections are required, the solution to use precast columns and girders requires extra reinforcement and concrete poured in the joints that can be sorted out as in the following arrangement.

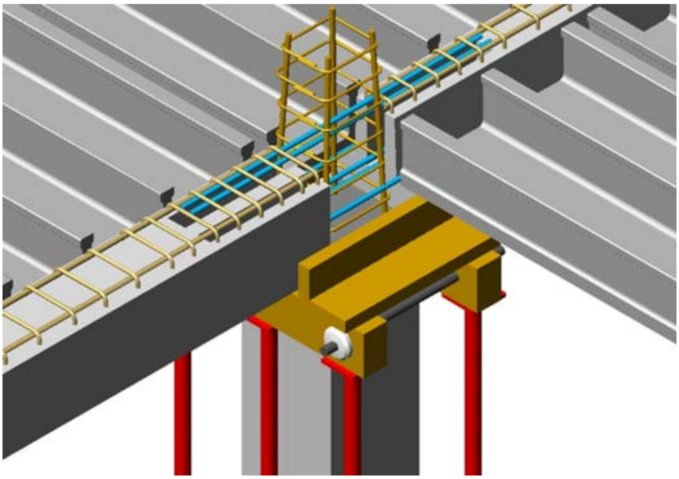


Figure 4.17 Column support for prestressed beams with rigid connections (PREFACINCA)

4.4.3 Prestressed piles

Precast-prestressed piles usually have a square or hexagonal section (figure 4.18). Due to the prestressing steel these elements are able to resist tension, flexion and shear stresses. These elements are widely used in marine structures and deep building foundations with special requirements. Their advantages are: their durability in adverse environments, their strength as a beam for handling and, as an unsupported column, their ability to resist tension. Their ability to withstand hard driving and their good soil-pile interaction behavior has led to rapidly increasing use of prestressed in this application. They can be as small in cross section as 200x200 mm and as large as 3.5 meters in diameter.

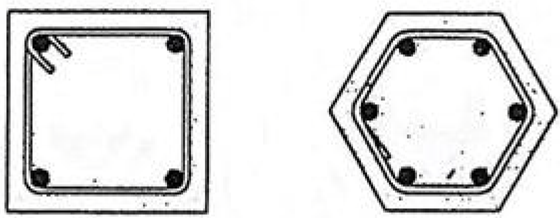


Figure 4.18 Precast-prestressed pile sections

Without joints, these piles are able to reach between 5-14 meters. In case larger depths of piling are required; several piles can be connected through joints that must be specially designed and carefully executed to resist larger stresses than the pile itself. The joints must guarantee protection from the soil as in the following detail.

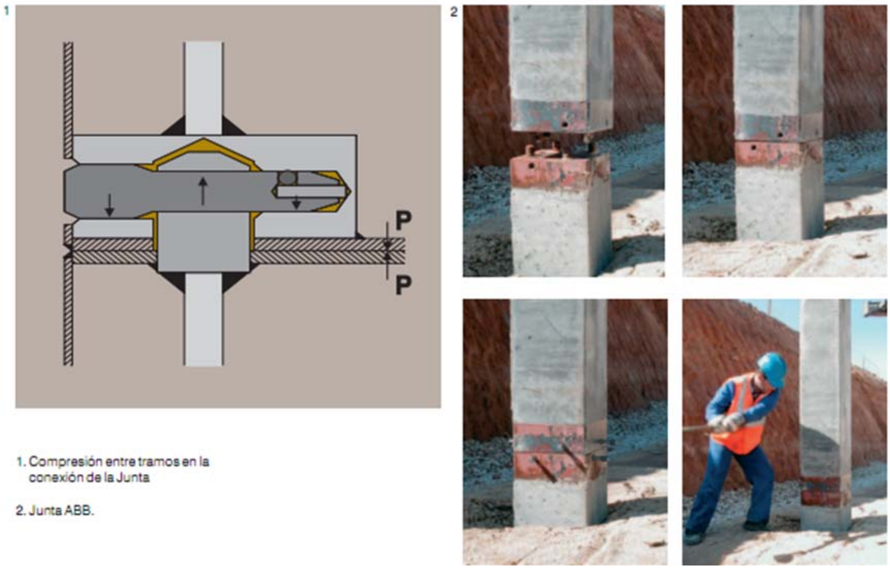


Figure 4.19 Prestressed pile connection details (Terratest)

In residential buildings the use of prestressed piles is advantageous for high-rise buildings or when in high seismic risk zones or when the phreatic stratum is above the ground or basement floor level.

5 Design example: uncracked member.

The code of practice followed for this example is Eurocode 2: Design of concrete structures, Part 1 *General rules and rules for buildings*, which will be referred to below as EC2. Design is based initially on the requirements of the serviceability limit state, since a fundamental aim of prestressed concrete is to limit tensile stresses, and hence flexural cracking. Subsequently considered are ultimate limit state criteria for bending and shear.

Consider simply supported pretensioned double-T beams that are required to span 15 m, for a parking structure, which bears an imposed load of 4kN/m^2. In addition to self-weight the beams carry a load of a concrete (screed) layer of 1 cm.

Exposure class

We will assume exposure class 2a: interior of buildings with high humidity, exterior components, and components in non-aggressive soil.

The criterion for the limit state of crack width according to table 5.1 (for members with bonded tendons only) for this exposure class is: *decompression.* This means that all of the tendons lie at least 25 mm within the compression zone.

	Design crack width under frequent load combination	
Exposure class	*Post-tensioned*	*Pretensioned*
1	0.2 mm	0.2 mm
2	0.2 mm	Decompression
3 & 4	Decompression or coating of the tendons and w_k=0.2 mm	Decompression

Table 5.1. Criteria for limit state of crack width.

The minimum concrete cover for durability (including a minimum construction tolerance of 5 mm) according to the exposure class (table 5.2) is:

Prestressing: 40 mm
Reinforcement: 35 mm

Exposure class	*Prestressing*	*Reinforcement*
1	25	20
2a	40	35
2b	40	35
3	45	40
4a	45	40
4b	45	40
5a	40	35
5b	40	35
5c	50	45

Table 5.2. Minimum concrete cover for durability

Loading

Screeding: 0.23 kN/m

Imposed load: 4.00 kN/m

Material Properties

Concrete: f_{ck}= 35 N/mm^2 (28 days)

f'_{ck}= 25 N/mm^2 (7 days)

Steel: f_{pk}= 1860 N/mm^2 (tendons)

f_{yk}= 250 N/mm^2 (links)

f_{yk}= 460 N/mm^2 (reinforcement)

Allowable stresses

The allowable tensile stresses vary according to the grade of concrete, which will have to be taken into account at transfer.

	Concrete grade						
	C20	C25	C30	C35	C40	C45	C50
f_{ctm}	2.2	2.6	2.9	3.2	3.5	3.8	4.1

Table 5.3 Allowable concrete stresses (N/mm2)

At transfer: f'_{min}= -2.6 N/mm^2

During service: f_{min}= -3.2 N/mm^2

The maximum compressive stresses in prestressed concrete are limited to 0.6 f_{ck} at transfer and under the rare load combination and 0.45 f_{ck} under the quasi-permanent loads. The last one is limited in order to minimize creep deformations to those predicted by other parts of EC2.

At transfer: f'_{max}= 0.6×f'_{ck}= 15 N/mm^2

In service:

Under quasi-permanent loads: $f_{max,qp}$=0.45×f_{ck}= 15.75 N/mm^2

Under rare-load combination: $f_{max,ra}$=0.60×f_{ck}= 21N/mm^2

Initial estimate of beam depth

Usual span/effective depth ratios for simply supported prestressed beams

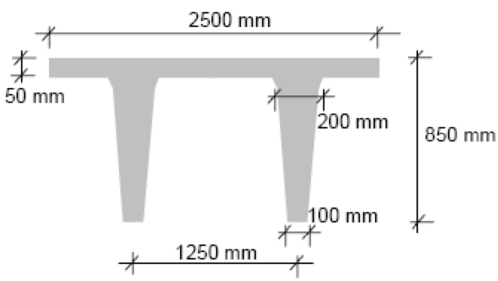
Section type	Variable/Imposed load (kN/m²)	Span/depth ratio
	< permanent load	40
	2,4 4,8	40-50 32-42
	2,4 4,8	20-30 18-28
	2,4 4,8	23-32 19-24
	< permanent load	20
	< permanent load	30
	road girders	18

Table 5.4 Usual span/effective depth ratios

According to table 5.4 we will use a length/height ratio of 18 to estimate the depth of the beam. Since the span of the double-t beam is 15 m:

H=15/18=0.83 m

We will choose a commercial section of **850** mm.



Beam section properties

According to beam section tables:

Beam width: 2500 mm I_c= 2.43 × 10^{10} mm^4

Beam depth: 850 mm y_b= 575 mm

Rib width: 100 mm Z_t= 8.813 × 10^7 mm^3

A_c = 365000 mm^2 Z_b= 4.22 × 10^7 mm^3

Minimum section size

Loading type	*Quasi-permanent*	*Frequent*
Dwellings	0.2	0.4
Offices and stores	0.3	0.6
Parking areas	0.6	0.7
Snow and wind	0	0.2

Table 5.5 Factors γ_f for quasi-permanent and frequent load combinations.

The coefficients used will be those stated in EC1 for building structures, particularly for parking areas (table 5.5).

Self weight:

$w_{o=}$ 25×3.65×10^5/10^6= 9.1 kN/m

M_o=(9.1×15^2)/8=256.6kN m

Quasi-permanent uniform load:

w_{qp}=9.1+2.5(0.23 + (0.6×4)) = 15.7 kN/m

M_{qp}= (15.7×15^2)/8= 441.6kN m

Frequent uniform load:

w_{fr}=9.1+2.5(0.23 + (0.7×4)) =16.7 kN/m

M_{fr}= (16.7×15^2)/8=469.7kN m

Rare uniform load:

w_{ra}= 9.1+2.5(0.23+4) =19.7 kN/m

M_{ra}= (19.7×15^2)/8=554.1kN m

At this point we will estimate the prestress force losses with the coefficientsα=0.9 and β=0.75 which represent a short-term loss of 10% and a long-term loss of 25%.We will calculate the elastic section moduli for the top and bottom fibresZtand Zb.

Equation (5.1)

$$Z_t \geq \frac{(\alpha M_{qp} - \beta M_o)}{\left[\alpha (f_{\max})_{qp} - \beta f'_{\min}\right]} = \frac{(0.9 \times 441.6 - 0.75 \times 256.6) \times 10^6}{[0.9 \times 15.75 - 0.75 \times (-2.6)]} = 1.27 \times 10^7 \; mm^3$$

Equation (5.2)

$$Z_t \geq \frac{(\alpha M_{ra} - \beta M_o)}{\left[\alpha (f_{\max})_{ra} - \beta f'_{\min}\right]} = \frac{(0.9 \times 554.1 - 0.75 \times 256.6) \times 10^6}{[0.9 \times 21 - 0.75 \times (-2.6)]} = 1.47 \times 10^7 \; mm^3$$

Equation (5.3)

$$Z_b \geq \frac{(\alpha M_{ra} - \beta M_o)}{\left[\beta (f'_{\max})_{ra} - \alpha f_{\min}\right]} = \frac{(0.9 \times 554.1 - 0.75 \times 256.6) \times 10^6}{[0.75 \times 15 - 0.9 \times (-3.2)]} = 2.29 \times 10^7 \; mm^3$$

Since the section properties were:

$$Z_t = 8.813 \times 10^7 \; mm^3$$

$$Z_b = 4.22 \times 10^7 \; mm^3$$

Section size is adequate.

Prestress force

Assuming a link diameter of 6 mm we will calculate the effective depth of tendons, and eccentricity.

Effective depth of tendons=850-(35+6+ (1.5×12.9) +20) =770 mm

e= y_b-(h-770) =575-(850-770) =495 mm

The next step is to find the prestress force based on the maximum eccentricity obtained from the section properties.

Equation (5.4)

$$P_0 \le \frac{\left(Z_t f'_{\min} - M_o\right)}{\alpha\left(Z_t / A_c + e\right)} = \frac{8.813\times10^7 \times(-2.6) - 256.6\times10^6}{0.9\times1000\left(8.813\times10^7 / 3.65\times10^5 + 495\right)} = 2131.7kN(*)$$

Equation (5.5)

$$P_0 \le \frac{\left(Z_b f'_{\max} + M_o\right)}{\alpha\left(Z_b / A_c + e\right)} = \frac{4.22\times10^7 \times 15 + 256.6\times10^6}{0.9\times1000\left(4.22\times10^7 / 3.65\times10^5 + 495\right)} = 1619.8kN$$

Equation (5.6)

$$P_0 \ge \frac{\left(Z_t (f_{\max})_{qp} - M_{qp}\right)}{\beta\left(Z_t / A_c - e\right)} = \frac{8.813\times10^7 \times 15.75 - 441.6\times10^6}{0.75\times1000\left(8.813\times10^7 / 3.65\times10^5 - 495\right)} = -4984.2kN(*)$$

Equation (5.7)

$$P_0 \ge \frac{\left(Z_t (f_{\max})_{ra} - M_{ra}\right)}{\beta\left(Z_t / A_c - e\right)} = \frac{8.813\times10^7 \times 21 - 554.1\times10^6}{0.75\times1000\left(8.813\times10^7 / 3.65\times10^5 - 495\right)} = -6828.2kN(*)$$

Equation (5.8)

$$P_0 \ge \frac{\left(Z_b f_{\min} + M_{ra}\right)}{\beta\left(Z_b / A_c + e\right)} = \frac{4.22\times10^7 \times(-3.2) + 554.1\times10^6}{0.75\times1000\left(4.22\times10^7 / 3.65\times10^5 + 495\right)} = 915.5kN$$

(*)Since the denominators are negative, the original inequality has been multiplied by a negative number and so the sense of the inequality must be reversed.

The limits to the prestress force would be **1619.8≥P_0≥915.5**.

We have chosen to use 12.9 mm diameter super-strands which have a steel area of 100 mm^2. Since the cost of prestressing steel is a significant proportion of the total cost of prestressed concrete structures, the minimum value within these bounds will be required. We will try with 8 tendons first.

$$A_p = 8 \times 100 = 800 mm^2$$

$$P_0 = 0.7 \times 1860 \times \frac{800}{1000} = 1041.6 kN$$

Check for decompression

It is now necessary to check if the tendons lie at least 25 mm within the compression zone, by determining whether or not the concrete stress at 25 mm below the tendons is compressive under the frequent load combination.Using 0.9 as the partial factor of safety:

$$Z_{b,25} = \frac{2.43 \times 10^{10}}{(575 - 35 + 25 - 6 - 12.9/2)} = 4.39 \times 10^7 \, mm^3$$

$$\sigma_{b,25} = 0.9 \times \beta \times P_0 \times 1000 \times \left(\frac{1}{A_c} + \frac{e}{Z_{b,25}} \right) - \frac{M_{fr} \times 10^6}{Z_{b,25}} = -0.85 N/mm^2$$

It is not a compressive stress as required (>0), therefore we have to try with a higher prestress force to satisfy the decompression requirement.

If we choose 10 tendons, they will provide a section of steel and prestress force of:

$$A_p = 10 \times 100 = 1000 mm^2$$

$$P_0 = 0.7 \times 1860 \times \frac{1000}{1000} = 1302 kN$$

Checking for decompression again yields:

$$\sigma_{b,25} = 0.9 \times \beta \times P_0 \times 1000 \times \left(\frac{1}{A_c} + \frac{e}{Z_{b,25}} \right) - \frac{M_{fr} \times 10^6}{Z_{b,25}} = 1.61 N/mm^2$$

This is now a compressive stress as required.Therefore we will use **10 tendons** of 12.9 mm diameter super-strand.We must now check that the section is uncracked at the soffit under the rare load combination, as assumed.

$$\sigma_{b,ra} = 0.9 \times \beta \times 1000 \times \left(\frac{1}{A_c} + \frac{e}{Z_t} \right) - \frac{M_{ra}}{Z_t} = 1.05 N/mm^2$$

$\sigma_{b,ra} > f_{\min} = -3.2 N/mm^2$, so the section is uncracked.

Estimate of losses

1. Elasticshortening

$$\sigma_{p0} = \frac{P_0 \times 1000}{A_p} = \frac{1302 \times 1000}{1000} = 1302 N/mm^2$$

$$m = \frac{E_s}{E_{cm}} = \frac{200}{32} = 6.3$$

$$r^2 = \frac{I_c}{A_c} \quad \text{(Radius of gyration)}$$

$$r = \sqrt{\frac{I_c}{A_c}} = \left(\sqrt{\frac{2.43 \times 10^{10}}{3.65 \times 10^5}}\right) = 258mm$$

At midspan

$$\sigma_{cg} = \frac{\sigma_{po}}{\left[m + \frac{A_c}{A_p\left(1 + e^2/r^2\right)}\right]} - \frac{M_0 e}{I_c} = 10.2N/mm^2$$

At supports, assuming that the number of tendons to be debonded is 2.

$$A_p = \frac{1000(10-2)}{10} = 800mm^2$$

$$\sigma_{cg} = 12.6N/mm^2$$

The average between both values would be:

$$\sigma_{cg} = 11.4N/mm^2$$

$$\Delta\sigma_p = m\sigma_{cg}A_p/1000 = 6.3 \times 11.4 \times 1000/1000 = 71.2kN$$

$$\text{Midspan } \alpha = (1302 - 71.2)/1302 = 0.94$$

2. Long term losses

According to table 5.6 and the notional size $2A_c/u = 87.95$; being u the perimeter (8300), the creep coefficient would be 1.5.Considering an outside environment, with a notional size≤150, the shrinkage strain is 330×10^{-6} (table 4.7).This is an approximate value according to EC2, since shrinkage is based in many factors

	Notional size $2A_c/u$ (mm)					
	50	150	600	50	150	600
Age at transfer (days) days	Dry atmospheric conditions (inside) (relative humidity 50%)			Humid atmospheric conditions (outside) (relative humidity 80%)		
1	5.5	4.6	3.7	3.6	3.2	2.9
7	3.9	3.1	2.6	2.6	2.3	2.0
28	3.0	2.5	2.0	1.9	1.7	1.5
90	2.4	2.0	1.6	1.5	1.4	1.2
365	1.8	1.5	1.2	1.1	1.0	1.0

Shrinkage strain ($\times 10^{-6}$)	Typical relative humidity %	Notional size ($2A_c/u$) (mm)	
		≤150	600
Inside	50	600	500
Outside	80	330	280

Tables5.6 and 5.7 Creep coefficients and shrinkage strains according to EC2

In absence of the 1000 hour relaxation test value given by the tendon manufacturer, it is specified in EC2 by the maximum relaxation value of 2.5% for class 2 steel (at 70% of breaking load) multiplied by the factors in table 5.8.

$$\Delta\sigma_{pr} = 0.025 \times 1.2 \times 0.7 \times 1860 = 39.1 N/mm^2$$

	Class 1 (wire and strand)	Class 2 (wire and strand)	Class 3 (bars)
Pretensioning	1.5	1.2	–
Post-tensioning	2.0	1.5	2.0

Table 5.8 Relaxation factors

At midspan:

$$\sigma_{cg} = P_0 \times \alpha \times 1000\left(\frac{1}{A_c} + \frac{e^2}{I_c}\right) - \frac{M_{qp}e}{I_c}$$

$$\sigma_{cg} = 1302 \times 0.95 \times 1000 \times \left(\frac{1}{3.65 \times 10^5} + \frac{495^2}{2.43 \times 10^{10}} \right) - \frac{441.6 \times 10^6 \times 495^2}{2.43 \times 10^{10}} = 6.78 N/mm^2$$

$$\sigma_{cpo} = 1302 \times 0.95 \times 1000 \times \left(\frac{1}{3.65 \times 10^5} + \frac{495^2}{2.43 \times 10^{10}} \right) = 15.79 N/mm^2$$

The following expression is given in EC2 for the long term losses due to concrete shrinkage and creep and to steel relaxation:

$$\sigma_{p,c+s+r} = \frac{\varepsilon_{sh} E_s + \Delta\sigma_{pr} + m\phi(\sigma_{cg} + \sigma_{cpo})}{1 + m\, A_p / A_c \left[\left(1 + A_c e^2 / I_c\right)(1 + 0.8\phi) \right]}$$

Where ε_{sh} is the estimated shrinkage strain from table 4.7; m is the modular ratio E_s/E_m; Φ is the creep coefficient from table 4.6; $\Delta\sigma_{pr}$is the variation of stress in the tendons due to steel relaxation; σ_{cg} is the stress in the concrete at level of tendons due to self weight and any other permanent loads; σ_{cpo} is the initial stress in the concrete adjacent to the tendons due to prestress.

$$\sigma_{p,c+s+r} = \frac{3.30 \times 10^{-4} \times 200 \times 10^3 + 39.1 + 6.3 \times 1.5(6.78 + 15.79)}{1 + 6.3 \dfrac{10^3}{3.65 \times 10^5} \left[\left(1 + 3.65 \times 10^5 \times 495^2 / 2.43 \times 10^{10}\right)(1 + 0.8 \times 1.5) \right]} = 269.2 N/mm^2$$

At the supports:

$$\sigma_{cpo} = \sigma_{cg} = \frac{15.79(10 - 2)}{10} = 12.63 N/mm^2$$

$$\sigma_{p,c+s+r} = \frac{3.30 \times 10^{-4} \times 200 \times 10^3 + 39.1 + 6.3 \times 1.5 \times 2 \times 12.63}{1 + 6.3 \dfrac{10^3}{3.65 \times 10^5} \left[\left(1 + 3.65 \times 10^5 \times 495^2 / 2.43 \times 10^{10}\right)(1 + 0.8 \times 1.5) \right]} = 290.6 N/mm^2$$

Average long-term losses:

$$\sigma_{p,c+s+r} = \frac{269.2 + 290.6}{2} = 279.9 N/mm^2$$

$$\Delta\sigma_{po} = \frac{279.9 \times 1000}{1000} = 279.9kN$$

Midspan $\beta = (1302 - 71.2 - 279.9)/1302 = 0.72$

Cable Zone

After choosing the prestress force, based on the most critical section,we can find the limits of the eccentricity along the member.The resultant of all the individual tendons is referred to as *cable*;and as long as it lies within the zone defined, the stresses will not exceed the allowable values at the different loading stages. Rearranging previous inequalities we have:

$$e \leq \frac{Z_t}{A_c} + \frac{1}{\alpha P_o}\left(M_o - Z_t f'_{\min}\right) \qquad (5.9)$$

$$e \leq \frac{1}{\alpha P_o}\left(M_o + Z_b f'_{\max}\right) - \frac{Z_b}{A_c} \qquad (5.10)$$

$$e \geq \frac{Z_t}{A_c} + \frac{1}{\beta P_o}\left[M_{qp} - Z_t\left(f_{\max}\right)_{qp}\right] \qquad (5.11)$$

$$e \geq \frac{Z_t}{A_c} + \frac{1}{\beta P_o}\left[M_{ra} - Z_t\left(f_{\max}\right)_{ra}\right] \qquad (5.12)$$

$$e \geq \frac{1}{\beta P_o}\left(M_{ra} + Z_b f_{\min}\right) - \frac{Z_b}{A_c} \qquad (5.13)$$

Since the values are symmetrical about the centre line, only half of the beam is shown.These inequalities and the values of M_o, $M_{qp}$and M_{ra} along the length of the beam are shown below:

			Position			
			0.0	**2.5**	**5**	**7.5**
		M_o	0.0	142.6	228.1	256.6
		M_{qp}	0.0	245.3	392.5	441.6
		M_{ra}	0.0	307.8	492.5	554.1
Equation	*2.9*	**e≤**	427.6	543.5	613.0	636.1
	2.10	**e≤**	398.7	514.5	584.0	607.2
	2.11	**e≥**	-1218.2	-960.3	-805.5	-753.9
	2.12	**e≥**	-1704.8	-1381	-1187	-1122
	2.13	**e≥**	-257.6	66.1	260.3	325.0

In simply supported beams with straight strands the prestressing force may cause excessive end release stresses, in order to control cracking we choose to debond some strands at the end of the member to reduce the prestress force in that area. Other methods to control this would be deflecting the tendons, to reduce the eccentricity at the supports, or adding untensioned reinforcement in that area.

In order to find out the debonding length, using equation 5.9

$$\frac{2\left(\left(e-\frac{Z_t}{A_c}\right)\alpha P_o + Z_t f'_{\min}\right)}{w_o} = \frac{2\left(\left(495-\frac{8.81\times10^7}{3.65\times10^5}\right)0.95\times1302\times1000+\left(8.81\times10^7\times(-2.6)\right)\right)}{9.1\times10^6}$$

$$=18.60$$

Distance to theoretical debonding point would be:

$$\frac{15-\sqrt{15^2-(4\times18.60)}}{2}=1.32m$$

The number of tendons to be debonded is two, which leaves a remaining prestress force in that area of:

$$P_o = \frac{(10-2)\times 1302}{10} = 1041.6kN$$

Limits in debonded zone according to the remaining prestress force are then given by:

		Position			
		0.0	**0.44**	**0.88**	**1.32**
	$\mathbf{M_o}$	0.0	29.2	56.7	82.4
	$\mathbf{M_{ra}}$	0.0	63.1	122.4	177.9
2.9	**e≤**	474	503.8	531.7	557.8
2.13	**e≥**	-293.1	-210.2	-132.2	-59.3

In pretensioned prestressed members there is no prestress force at the end of the tendonsonce cut, since there is no anchorage as in post-tensioned members. Therefore a certain length for the prestress force to develop by bond between the concrete and steel is necessary. The length up to a point where the prestress force developed equals the initial prestress force is known as *transmission length.* The distance to the theoretical debonding point does not take this factor into account so the transmission length has to be considered in order to stablish the real debonding length. The formula provided in EC2 for transmission length is:

$$l_{bp} = \beta_b \phi$$

Equation 5.14

Where β is a coefficient taken from table 5.9 and ϕ is the diameter of the strands or wires.

	Concrete grade at transfer					
	C25	C30	C35	C40	C45	C50
Strands and smooth or indented wires	75	70	65	60	55	50
Ribbed wires	55	50	45	40	35	30

Table 5.9 Transmission length factors β_b

The previous table is valid for strands with an area of up to 100 mm^2 and for indented wires with a diameter of 8 mm or less.

Therefore using formula 5.14:

$$l_{bp} = \frac{65 \times 12.9}{1000} = 0.84m$$

To find the debonding length we have to allow for $1.2l_{bp}$ after the point where the prestress force can theoretically be reduced. Therefore, the actual debonding length would be:

$1.32 - (0.84 \times 1.2) = 0.32m$ from support.

Ultimate strength

Assuming that the neutral axis lies within the flange and using the simplified stress block given in EC2 (figure 5.1)

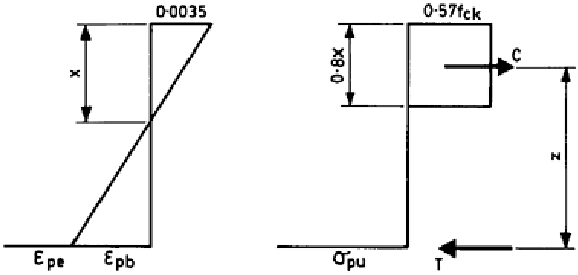


Figure 5.1 Simplified rectangular stress block

$$0.57 f_{ck} \times b \times 0.8x = 0.78 \times f_{yk} \times A_p$$

$$x = \frac{0.78 \times 1860 \times 1302}{0.57 \times 0.8 \times 35 \times 2500} = 36.4mm$$

$$\varepsilon_p = \frac{P_o \beta}{A_p E_s} + \frac{(770 - x)\varepsilon_{cu}}{x} = \frac{1302 \times 0.73}{1000 \times 200} + \frac{(770 - 36.4)}{36.4} = 0.0753$$

The tendons are initially stressed to 0.7 $f_{pk}$and the modulus of elasticity for steel is 200×103 N/mm^2 according to EC2, which gives this value for all types of prestressing steel for design purposes.

$$\varepsilon_{yk} = \frac{1860 \times 0.7}{200 \times 10^3} = 0.00651$$

0.0753>0.00651, so the **steel has yielded** (ductile failure).

$$M_u = 0.78 \times 1860 \times 1000(770 - 0.4 \times 36.4)/10^6 = 1095.5kNm$$

The values of γ_f for the ultimate limit state according with EC2 for building structures, considering the effect as adverse are

$$w_{ult} = 1.35\left(\frac{9.1 + 0.23 \times 2500}{1000}\right) + \frac{1.5 \times 4 \times 2500}{1000} = 28.1 kN/m$$

$$M_{ult} = \frac{28.1 \times 15^2}{8} = 790.17 kNm$$

Therefore the ultimate strength is adequate.

Shear

For beams with distributed loading, since there is enhanced shear resistance of members close to a support, it is only necessary to check shear resistance at a distance from the face of a support equal to the effective depth of the tendons.

The effective depth *d* to tendons at supports, being the lowest 2 tendons debonded:

$$d = 770 - (20 + 12.9) = 737 mm$$

The applied shear force at a distance *d* away from support is:

$$V_{sd} = w_{ult}\left(\frac{L}{2} - \frac{d}{1000}\right) = 28.1\left(\frac{15}{2} - \frac{737}{1000}\right) = 190.01 kN$$

$$M_{sd} = \frac{28.1 \times 737(15 - 737/1000)}{2 \times 1000} = 147.6 kNm$$

Shear resistance is calculated using a truss analogy, with the concrete being the compression elements and the reinforcement being the tension elements, as in figure 5.2.

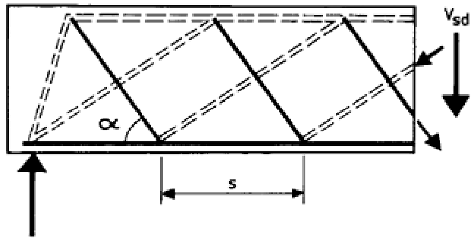


Figure 5.2 Truss model

The following formula is given in EC2 for the shear resistance, V_{Rd1}, of a given section of a prestressed concrete member with no shear reinforcement:

$$V_{Rd1}=[\tau_{Rd}k\,(1.2+40\ \rho_1)+0.15\ \sigma_{cp}]\ b_w d$$

Equation 5.15

Whereτ_{Rd} is the basic concrete shear strength (table 5.10); k is a factor related to the section depth: k= (1.6-d); b_w is the minimum width of the section; ρ_1 is the tension reinforcement ratio defined as $(A_p+A_s)/(b_w d)$, with a maximum value of 0.02; and σ_{cp} is the axial stress resulting from the prestressing force and any applied axial load.

$$\rho_1=\frac{(10-2)\times 1000}{2\times 737\times 100\times 10}=0.54\%\ \ k=1.6-737/1000=0.86$$

Concrete grade					
C25	C30	C35	C40	C45	C50
0.30	0.34	0.37	0.41	0.44	0.48

Table 5.10 Basic concrete shear strength τ_{Rd} (N/mm^2)

Therefore equation 5.15 is:

$$V_{Rd1} = \left[0.37 \times 0.87(1.2 \times 40 \times 0.55) + 0.15 \times 1302 \times 0.70 \times {}^{1000}\!/_{365000}\right] 2 \times 100 \times 737/1000 = 124.3kN$$

Since the applied shear force at the section, V_{sd}, exceeds the shear resistance, V_{Rd1}, then shear reinforcement must be provided.If V_{sd}exceeds the maximum shear resistance of the section, V_{Rd2}, then the section size should be increased. In this case:

$$V_{Rd2} = 0.3\nu f_{ck} b_w d(1 + \cot g\alpha)$$

$\nu = 0.7 - f_{ck}/200 = 0.7 - 35/200 = 0.525$, ν>0.5

For vertical links cotgα= 0, therefore:

$$V_{Rd2} = 0.3 \times 0.525 \times 35 \times 2 \times 100 \times 737 = 812.27kN$$

In caseσ_{cp}>0.27f_{ck}, the value should be further reduced to by the expression: 1.67 (1−1.5 σ_{cp}/f_{ck}) V_{Rd2}, to allow for the effect of axial compression in the section.In this example σcp≈0.8f_{ck} so the value does not have to be reduced.

Since the applied shear force is smaller than V_{Rd2}, the section size is adequate.

The following formula for the required shear reinforcement is given in EC2 as part of the 'standard' method:

$$A_{sw}/s = (V_{sd} - V_{Rd1})/\left[0.78 d f_{yk} (1 + \cot g\alpha)\sin\alpha\right]$$

WhereA_{sw} is the area of shear reinforcement over a length *s*, and f_{yk}=250N/mm^2.Once again, since we will be using vertical links cotgα=0.

$$A_{sw}/s = (190.01 - 124.3)1000/(0.78 \times 250 \times 737) = 0.46$$

The separation of the links can be taken as:

$$s = \frac{6^2 \times \Pi}{2 \times 0.46} = 123.59mm$$

Using single stirrups, for **6mm links at 100 mm centres**, A_{sw}=0.56.

The nominal shear reinforcement to be provided at the point where $V_s<V_{Rd1}$ is obtained from the following equation:

$$A_{sw}/s = \rho_w b_w \sin\alpha$$

Where ρ_W is a coefficient relating concrete grade and the steel type, from table 5.11.

	Steel type	
Concrete grade	Mild	High tensile
C25-C35	0.0024	0.0013
C40-C50	0.0030	0.0016

Table 5.11 Minimum values for ρ_w

$$A_{sw}/s = 0.0024 \times 100 = 0.24$$

This is covered by **6mm links at 200 mm centres**.At 3.5m from the support Vs=112.4kN which is less than V_{Rd1}, and so only the nominal shear reinforcement would be required.

The next step is to check the force in the longitudinal steel, which generally is only necessary near supports since at midspan it is sufficient to check that the ultimate limit state of collapse is satisfied. The additional longitudinal tensile force is given by:

$$T_d = M_{sd}/z + V_{sd}(1+\cot g\alpha)/2 = 147.6/(0.9 \times 737) + 190.01/2 = 95.23kN$$

Being *z* the lever arm which can be taken as 0.9d.

$$f_{yk}=460N/mm^2;\ A_s=\frac{95.2\times1000}{0.87\times460}=238mm^2$$

This additional longitudinal steel can be provided by **six untensioned 8 mm bars** placed at the bottom of the beam's cross section and fully anchored past the point required.

The type and arrangement of the shear reinforcement would be as follows:

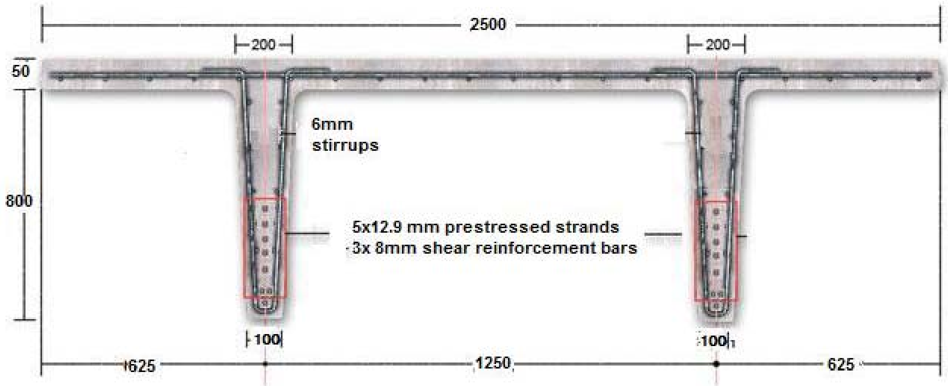


Figure 5.3 Beam section.Shear reinforcement

Deflection

Initial camber

Since the prestressed member is uncracked, deflections can be estimated by applying ordinary strength-of-materials methods. In simply supported prestressed concrete members with straight tendonsthe moment at any section due to the prestress force is equal to *Pe.*We will use a simplified method of finding the maximum deflection of concrete members which assumes that the distribution of curvature is similar to the shape of the bending moment diagram.

$$M_p = P_0 e$$

This bending moment results in an upwards deflection, known as camber, which value can be found by the following expression of the curve:

$$y'' = -\frac{P_0 e}{E_{cm} I_0}$$

The integration yields:

$$y' = -\frac{P_0 e}{E_{cm}} x + C_1$$

For the value at midspan (x=L/2) and y'=0:

$$C_1 = \frac{P_0 e}{2E_{cm} I}$$

$$\therefore y' = -\frac{P_0 e}{E_{cm} I} + \frac{P_0 e L}{2E_{cm} I}$$

Integrating the expression again we obtain:

$$y = -\frac{P_0 e}{2E_{cm} \ I} x^2 + \frac{P_0 e}{E_{cm} I} \frac{L}{2} x + C_2$$

Being *y=0* for *x=0*,then $C_2=0$, and so the camber at midspan results:

$$y = -\frac{P_0 e L^2}{8E_{cm} I}$$

Also, the midspan deflection due to a uniformly distributed load *w* over a span L, for a simply supported member, is given by:

$$y = \frac{5}{384} \frac{wL^4}{EI}$$

With the previous expressions we can now estimate the deflections, starting withshort-term deflections at transfer. The initial camber will be the sum of the deflection caused by prestress (upwards) and the one caused by the distributed load due to self-weight.The value of E_{cm}used will be that for the seventh day resistance of concrete,at transfer (f_{ck}*=25 N/*mm)^2.

Concrete grade	**Mean value (10^3N/mm^2)**
C20	29.0
C25	30.5
C30	32.0
C35	33.5
C40	35.0
C45	36.0
C50	37.0

Table 5.12 Modulus of elasticity of concrete.

E_{cm}=30.5 kN/mm^2

$$\delta_0 = -\frac{15^2 \times 1302 \times 0.94 \times 495 \times 10^6}{8 \times 30.5 \times 2.43 \times 10^{10}} + \frac{5 \times 9.1 \times 15^4 \times 10^9}{384 \times 30.5 \times 2.43 \times 10^{10}} = -15.0mm$$

Quasi-permanent load

Long-term creep movements will cause deflections in concrete members to increase with time. This effect can be estimated by using the expression given in EC2 for an effective modulus of elasticity $E_{c.eff}$:

$$E_{c.eff} = E_{cm} / (1 + \varphi)$$

Where ϕ is the creep coefficient taken from table 4.6. In this case ϕ=1.5 therefore $E_{c.eff}$ is:

$$E_{c.eff} = 30.5/1+1.5 = 12.20kN/mm^2$$

We can estimate the deflection with the same expressions used for the deflection at transfer, using the long-term coefficient for the prestress loss and the quasi-permanent loads as the distributed load:

$$\delta_0 = -\frac{15^2 \times 1302 \times 0.72 \times 495 \times 10^6}{8 \times 12.20 \times 2.43 \times 10^{10}} + \frac{5 \times 15.7 \times 15^4 \times 10^9}{384 \times 12.20 \times 2.43 \times 10^{10}} = -15.0mm$$

Which is also an upwards deflection.

The usual requirement to be satisfied in respect of deflections is that under the action of the quasi-permanent load the value for deflection should remain below the value of span/250. Since in our case the deflection is of 15.0 mm upwards, the value is satisfactory.

After checking deflections the design of the prestressed concrete beam is now complete.

6 Final Conclusions

6.1 Introduction

In this report we have learned the basic principles of prestressed concrete as a precast structural material in the building industry, and so its main goal has been accomplished.

It has been intended to provide a complete glance on the subject: from the history of its development as a technique, its production process and product reception on site, to a simple design example of a typical prestressed-precast unit.

6.2 Conclusions

Concrete has been in use as a primary building material since the Roman times. In the mid-nineteenth century, it was discovered that iron and later steel bars could be embedded in the concrete, effectively giving it tensile strength. This allowed it to be used in beams and slabs, where it worked in bending. However this beams and slabs still deflected significantly under load, requiring stocky sections to provide adequate stiffness, and cracks created by this deflection left the reinforcement bars vulnerable to corrosion.

In the 1930's Eugene Freyssinet invented prestressed concrete, where high tensile steel cables compressed the concrete, ridding it of its cracks, and improving both its appearance and its resistance to deteriorations. This allowed much more slender structures to be built, also making them quicker to build and less labor intensive. It was soon realized that the higher quality materials and techniques for prestressing concrete could be more efficiently applied in an industrial environment, where a large enough number of products could be produced. The development of prestressed concrete became closely related to the development of precast industry and materials, providing new solutions to existing problems.

Since then the precast industry is geared towards achieving materials savings and reducing weight, and thus the cost of transport and assembly. As prestressed members require less concrete and about 20 to 35% of the amount of the reinforcement, this technique becomes an excellent option to achieve materials savings. Prestressing of beams and slabs is most economically applied in long-line precasting mass-production operations. Because of the inherent efficiencies of mass-production techniques, large reductions in labor, formwork, and hardware costs can be realized.

The latest editions of national and European standards have also regarded the importance of precast products in residential building construction, in terms of higher quality materials, durability, and sustainable development. They provide specific conditions that benefit the use of products and processes manufactured under a controlled production environment. It is safe to assume that factory produced prestressed concrete will continue to gain relevance in residential building construction in Spain and across Europe, as it has happened in countries with a traditional use of precast concrete products such as Japan or the U.S.A.

6.3 Future reports

This dissertation is based on pretensioning since it is the prestressing technique that traditionally has been more related to the building industry and is widely used in Spain and across Europe for architectural concrete. Also since this report is centered on precast products, post-tensioning being mostly an in-situ technique has been out of its scope. Nevertheless post-tensioning has become increasingly relevant in buildings, and is frequently used in slabs of high-rise or special buildings in other countries such as the U.S.A., when continuous beam solutions are required in long span structures, and even in foundations.

Therefore many interesting reports centered on post-tensioning applied to residential, commercial or industrial construction could follow this one. One proposal would be a guide on post-tensioned slab design and production. Another proposal would be to compare simply reinforced, pretensioned and post-tensioned members as a structural solution for a building, comparing the three techniques regarding issues such as economic cost, material saving, site impact and aesthetics.

Finally a dissertation could be completed comparing the actual standards affecting pretensioned and post-tensioned concrete in Spain, Europe, the U.S.A. or Japan.

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