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Post-tensioning in buildings

technical report

Post-tensioning in buildings

Technical report prepared by

Task Group 1.1

February 2005

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This report was drafted by Working Party 1.1.2, *Post-tensioning in buildings*, of ***fib*** Task Group 1.1:

João Almeida (Convener, Instituto Superior Tecnico, Portugal)
José Camara (Instituto Superior Tecnico, Portugal), Hugo Corres (ETS de Ingenieros de Caminos, Spain), Thomas Friedrich (Domostatik Ag, Switzerland), Manfred Miehlbradt (EPF Lausanne, Switzerland), Jean-Marc Voumard (VSL Switzerland), Bo Westerberg (Tyréns Byggkonsult, Sweden)

Task Group 1.1 members contributing to the report:
Stathis N. Bousias (University of Patras, Greece), Ludovit Fillo (Slovak Technical University, Slovakia), Stein Atle Haugerud (Olav Olsen a.s., Norway), Toshio Ichihashi (until March 2004, Taisei Corporation, Japan), Milan Kalny (Pontex s.r.o, Czech Republic), Santiago Perez-Fadon (Ferrovial – Agromán, Spain), Karl-Heinz Reineck (University of Stuttgart, Germany), Jouni Rissanen (Pontek Oy, Finland), Hiroshi Shiratani (from March 2004, Taisei Corporation, Japan)

Full address details of Task Group members may be found in the ***fib*** Directory or through the online services on ***fib's*** website, www.fib-international.org.

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Post address: Case Postale 88, CH-1015 Lausanne, Switzerland
Street address: Federal Institute of Technology Lausanne - EPFL, Section Génie Civil
Tel +41 21 693 2747, Fax +41 21 693 6245, E-mail fib@epfl.ch, web www.fib-international.org

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Preface

There is no other material than structural concrete that has been the major form-giving element to the architecture of our time. Although used ubiquitously, expediently, and routinely throughout the world, it has been rarely used expressively and rarer still beautifully at the hands of very few gifted individuals.

Its use goes back to the very beginning of civilisation, but it is only since the early part of the 20th Century that some few ingeniously creative people were able to give it expressive form. The French engineer, Freyssinet and the Swiss Maillart, whose uniquely memorable bridges span the vast gorges of that mountainous country, who stand out as the pioneers of concrete design. Their works have remained as valid structurally as they are visually significant to this day. Pier Luigi Nervi, the Italian giant of engineering inventiveness, not only in design of concrete, but of construction method, has not been, in my view, surpassed in giving poetically expressive form to concrete structures.

Nervi felt intensely what was "correct" structurally; it seemed to be in his blood to "feel" the rightness of a structural solution, to know the way stresses flowed, to give free reign and beautiful expression to the laws of nature - not what is "imagined" to be so by many structurally naive architects - but the unassailable physical truth of statics, based soundly on practical and ingeniously inventive constructional sense. This is what makes Nervi so unique in this age of lawlessness and indulgent, irresponsible excesses in architecture and construction. Not only was he an engineer with a rare gift of poetic expression in the language of structure, but he also was an experienced - no nonsense - practical contractor-builder.

He not only gave the problems beautiful form, but he also showed us ***how*** to build it.

Concrete structures today will vary in different socio-economic locations in the world. Where manual labour is economically available, concrete structures can be formed to take on complex geometries, something that is often economically unfeasible in high labour cost countries with available sophisticated technologies. The speed with which the building must be erected has a direct effect on the structure and method of construction chosen. The preference in high labour cost countries is to avoid external scaffolding, for low-rise structures to precast as far as practical and to prestress concrete, not only horizontally, but also vertically so as to resist lateral loads.

In most structures today, the increasing use of prestressing has given new freedom to any concept of forms previously considered uneconomic or unfeasible or unduly bulky to resist loads. Curvilinear forms have produced an entirely new vocabulary for architecture that in previous decades would have been thought of as impractical. The use of prestressing has also been found to be a way of achieving waterproof flat roof surfaces even without the application of normal bituminous waterproofing, since the concrete is kept in continual compression and resists cracking which can otherwise lead to water penetration in the long term.

The use of concrete in building is here to stay in our time. It is fireproof, virtually indestructible and challenges us to devise ever-improved ways of using it with greater economy in labour, and speed. By constantly absorbing advances in technology it continues to respond to the demands we place on our new buildings. Concrete helps us in defying gravity in a way that gives expressive form to the laws of nature.

Prof. Harry Seidler, Australia

Foreword

Working Party WP1.1.3 *Post-tensioning in Buildings* was established in 1999, integrating the activities of TG1.1 *Design Applications* and, more generally, of *fib* Commission 1 *Structures*.

The main objective of the Bulletin is to point out the benefits of using post-tensioning for the more common practical applications in concrete building construction: functional advantages and architectural freedom, economy, reduced construction time and element dimensions, improved structural behaviour and quality.

It was the wish of TG1.1 that the document should be addressed mainly to architects, contractors and owners. Therefore, basic design aspects and design criteria are only summarized, whereas conceptual design aspects are emphasized.

A set of practical examples, recently constructed buildings, most of them designed by the WP members, is presented, showing the adopted solutions and their advantages when meeting the requirements of specific problems. The selected examples were precisely not chosen because they are outstanding structures. As a matter of fact, post-tensioning principles and technology can be used in any structure, independently of its importance, covering a wide range of building structural applications, improving the structure quality and promoting concrete as a structural material.

The contribution of the Working Party members listed above should be specially acknowledged. The involvement of all Task Group 1.1 members at the technical discussions, particularly Milan Kalny, Santiago Pérez-Fadon and Karl-Heinz Reineck, is to be mentioned as well. Finally, WP members express their thanks to Mr. Miguel Lourenço and Mrs. Cristina Ventura for their important contribution to the final editing work.

February 2005

João F. Almeida
Convener of TG 1.1
Convener of WP *Post-tensioning in Buildings*

Jean-François Klein
Chairman of Commission 1

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1 Introduction

The development of prestressing technology has certainly constituted one of the more important improvements in the fields of structural engineering and construction. Referring particularly to post-tensioning applications, it is generally recognized how it opens the possibility to improve economy, structural behaviour and aesthetic aspects in concrete solutions.

In spite of the simplicity of its basic concepts and well known advantages, the application extent of post-tensioning solutions can not be considered harmonized in the different areas and structural applications. In fact, for various reasons, it appears that the potential offered by prestressing is far from being fully exploited, especially in building structures field. In many cases in which post-tensioning would provide a visibly superior solution, it happens after all that a more *conventional* non-prestressed solution is often selected.

The development of a new building project usually involves the owner, the design team, where the architect and the structural engineer are included, and, more and more frequently, the contractor. A survey of practical situations would show that post-tensioning solutions are frequently not adopted because some of those involved are not familiar with prestressing technology and its advantages.

It should be pointed out that post-tensioning allows more architectural freedom and can provide important functional advantages: longer spans providing more “flexible” solutions, transition structures solving the conflicts of vertical discontinuities in the building use, and slender column-free spaces for public areas, are good examples, where post-tensioning is the right solution.

Concerning economic aspects, the more significant cost in building structures corresponds to the floor structural system. The use of post-tensioning allows slender and lighter floor systems with gains on the total building height. Due to the reduction of permanent loads and seismic action effects, the floor weight influences the size of the vertical elements and foundations. It should be as well emphasized that the total construction time can in general be substantially reduced.

The use of prestressing steel can result in a substantial reduction in the total steel area. Therefore, structural details are improved together with easier placing and compacting of concrete.

Prestressing offers the possibility of introducing a favourable system of anchorage forces and tendon deviation forces on the concrete. The transverse components of the anchorage forces and the tendon deviation forces provide a load balancing system, counteracting the vertical applied loads, effectively controlling cracking and deformations. The in-plane anchorage forces precompress the concrete element, leading to higher cracking resistance, improved stiffness and water tightness, allowing the reduction of expansion joints.

As a conclusion, it can be stated that in many practical situations better technical and economical solutions can be achieved by using post-tensioning, improving the structure quality and promoting concrete.

The main objective of the present report is to show the benefits of using post-tensioning for the more common practical applications in concrete buildings. The document is mainly addressed to architects, contractors and owners. It is also drafted with the goal of motivating

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building designers to use post-tensioning: basic design aspects related to prestressing effects and design criteria are summarized and conceptual design aspects are emphasized.

The advantages of using post-tensioning, concerning structural behaviour, economy, detailing and constructive aspects, are illustrated by the presentation of several existing structures, most of them designed by Working Party members. General design calculations are not presented, but design results showing the improvement in structural behaviour illustrated.

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2 Post-tensioning in buildings

2.1 General

Application of prestress in building concrete structures poses no major difficulty in comparison with any other type of structure. Nevertheless, in some aspects it has its particularities concerning the type of geometry, loads and/or border constraints. It is the case of thin slabs, heavy concentrated loads on transition floors or rigid vertical elements that restrain the prestress effects.

In this chapter the basic structural concepts of prestress are reviewed, the main technology aspects of prestressing, specially for thin slabs, are presented and the design aspects are described in a brief way. In what concerns design criteria and calculations the text is oriented to call attention upon the major issues a structural engineer has to take into account. Simple calculations are exemplified in order to give the lecturer the order of magnitude of the different design verifications. Particular aspects of building structures are always referred.

2.2 Basic concepts of prestressing

2.2.1 Introduction

A structural element subjected to bending, e.g. a beam, will carry the load by means of internal compressive and tensile stresses, see figure 2.1.

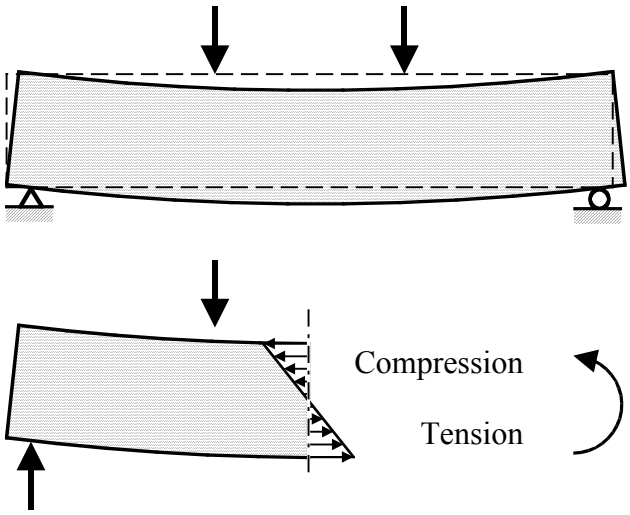

Figure 2.1: Compressive and tensile stresses in a structural element subjected to bending, e.g. a beam

If the material has a low tensile strength, which is the case e.g. for masonry and unreinforced concrete, the load bearing capacity will be correspondingly low. One way of compensating for this is to apply a compressive force to the element. This will increase the stresses on the compressive side and reduce, or even cancel, the tensile stresses, figure 2.2.

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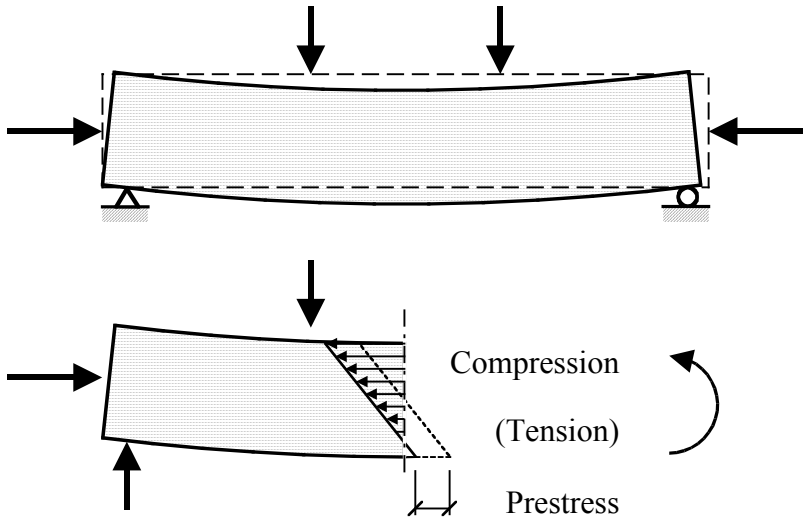

Figure 2.2: Compensating a low tensile strength by prestressing with longitudinal compression

In this way a "structure" with no tensile strength at all can act like a beam, e.g. a pack of books which can be lifted from a shelf by being pressed together, figure 2.3.

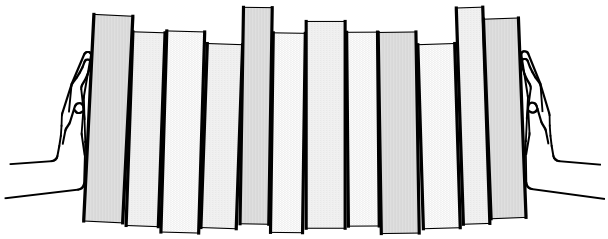

Figure 2.3: A pack of books, carrying its own weight like a beam due to the effect of "prestressing"

The technique of compensating a low tensile strength by means of compressive forces has been used for centuries in masonry structures carrying vertical and horizontal forces from arches. The horizontal forces cause bending in supporting columns. Since the tensile strength of masonry is low, it was sometimes necessary to add extra weight on the columns, to achieve sufficient compression for reducing or cancelling the tensile stresses. This is illustrated schematically in figure 2.4.

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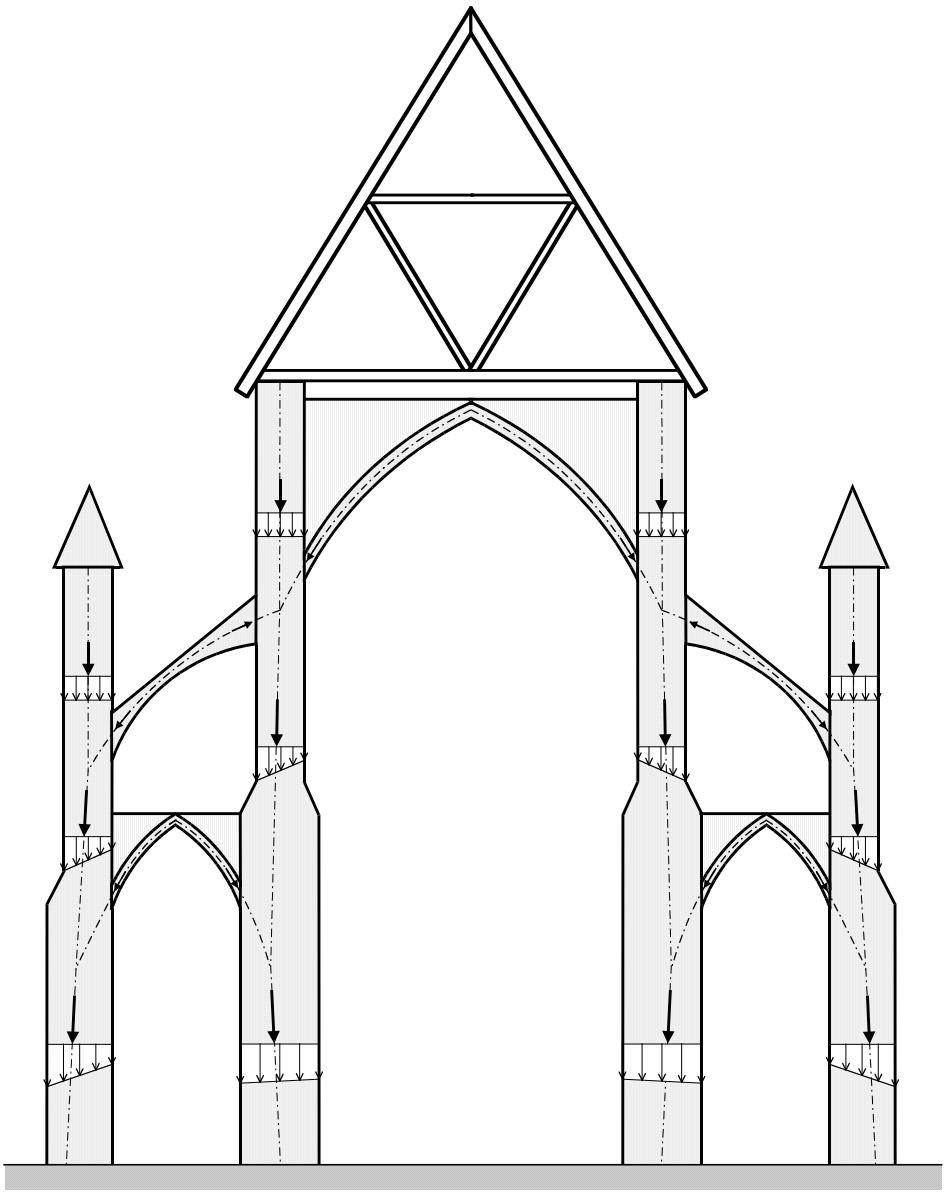

Figure 2.4: Example of "prestressed columns" in a structure with arches

In modern concrete structures, most tensile forces are taken by reinforcement. However, no significant stress can develop in the reinforcement until the concrete has cracked. This cracking can often be accepted, but for various reasons it is sometimes desirable to prevent it, or at least reduce it. Then, again, prestressing can be used. The prestressing is then normally achieved by means of steel tendons in the form of bars, strands or cables, stressed in tension and thereby producing a corresponding compression in the concrete. Modern prestressing techniques and the potential benefits of using them are dealt with in subsequent clauses.

2.2.2 Prestressing in concrete structures

Prestressing is a way of counteracting the effect of external loads on a structure by imposing a state of stresses contrary to the load effects. The most common way to achieve this is by means of tendons, which are stressed prior to final loading of the structure.

Prestressing with tendons has two main effects, *axial* and *transverse*, see figure 2.5. The axial effect gives compression in the concrete, caused by anchorage forces at the tendon ends –

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figure 2.5 (a, b and c). In case b), the eccentricity of the straight tendon causes bending in addition to the axial effect. Finally, the use of curved tendons (case c) introduces a transverse effect that can be designed to more or less counteract the external loads, with both axial, bending and shear effects.

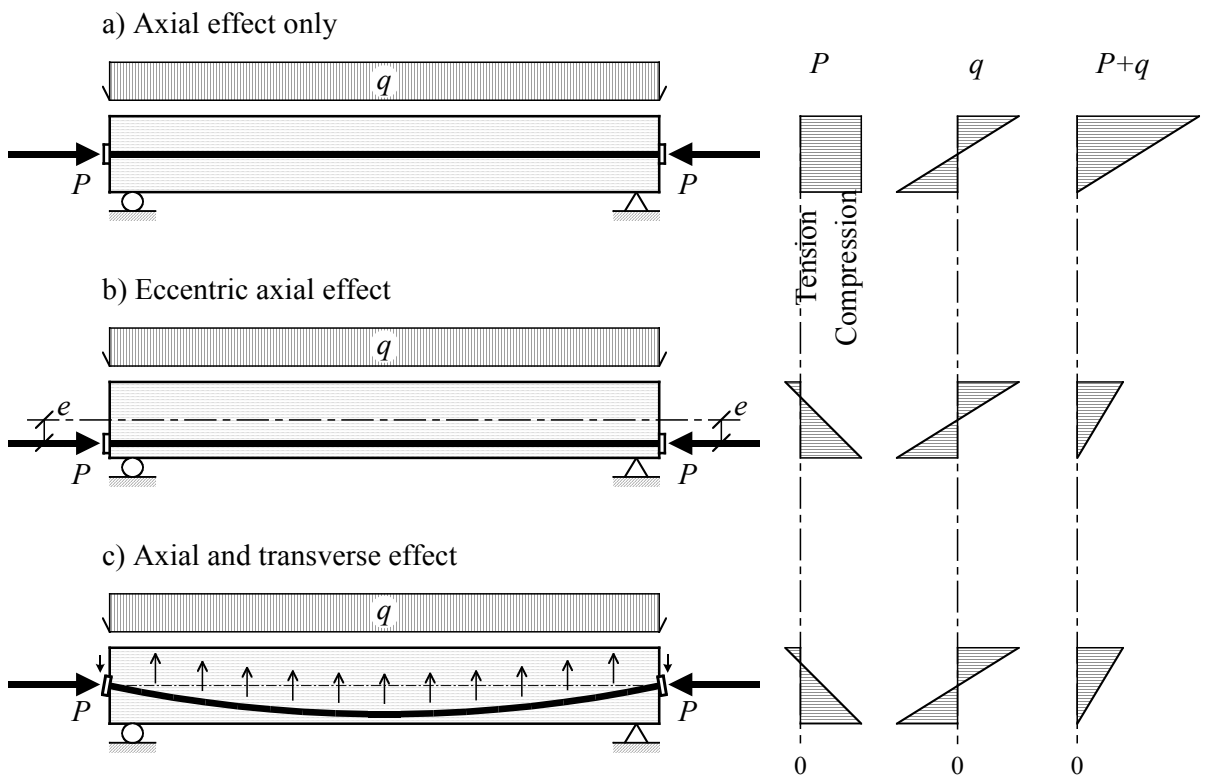

Figure 2.5: Illustration of the main effects of prestress. a) represents the pure axial effect. b) represents a typical pre-tensioned member with eccentric straight tendons, introducing an additional bending effect. c) represents a typical post-tensioned member with curved tendons, giving axial, bending and shear effects

The transverse effect of prestress will carry a certain part of the external load directly to the supports. For the remaining load, the structure will have an enhanced resistance to shear, punching and torsion due to compressive stresses from the axial effect. Prestress will also reduce deflections under service conditions, due to both the reduced effect of external load and the increased stiffness caused by delayed or eliminated cracking.

The fundamental and well-known advantages of prestressing can be summarized as the possibility to limit cracking and deformations in structural members with large spans, to reduce cross section dimensions for a given span and load and, finally, to increase the load capacity for given span and dimensions. This is further developed in clause 2.3.

2.2.3 Main systems for prestressing

2.2.3.1 Pre-tensioning and post-tensioning

The two main systems are *pre-tensioning* and *post-tensioning*. Although post-tensioning is the main topic here, the basic features of pre-tensioning will be mentioned for comparison.

With *pre-tensioning* the reinforcement is prestressed in the mould, *before* pouring the concrete. Fixed anchorages for the tendons are needed outside the moulds; therefore the method is primarily suitable for factory production of *precast elements*, where several units can be

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prestressed with the same tendons in a long line. After sufficient hardening of the concrete, the prestress is released from the anchorages, and is instead transferred to the elements by *bond*. For adequate bond anchorage, tendons have to be rather small; common types are strands ∅ 9 or 13 mm, or single wires. Tendons are usually straight and eccentrically placed, cf. figure 2.5 (b). It is possible to improve the structural efficiency by introducing a transverse effect from deviation forces, but this requires deviators fixed into the mould, and is usually avoided for ease of production.

With *post-tensioning* the reinforcement is stressed *after* hardening of the concrete. The anchorages are fixed into the concrete, and without need for external anchorages the method is suitable for in-situ construction. Special end anchorages transfer the prestressing forces to the concrete. This allows larger tendons than in pre-tensioning, since anchorage no longer depends on bond. Tendons usually consist of several wires or strands with a common anchorage, or of large diameter bars. Post-tensioning offers great freedom in the layout of tendons for optimum transverse effect, cf. figure 2.5 (c).

To be stressed after hardening of concrete, tendons must be freely movable in the concrete. The most common way to achieve this is to enclose them in metal or plastic ducts. After tensioning, the ducts are filled with cement grout by pressure injection. This has two important aims: *corrosion protection* and *bond* between tendons and surrounding concrete.

Bond is favourable for the structural behaviour, even if tendons are fully end anchored. However, post-tensioning *without bond* is also possible. Tendons are then enclosed in plastic sheaths, with a layer of grease in between, to give corrosion protection and reduce friction. Unbonded tendons usually consist of single strands. The lack of bond has effects on the structural behaviour, which must be considered in design.

In most countries, until recently, the most common use of prestressed concrete in buildings has been in the form of precast pre-tensioned elements, such as hollow core slabs, double-T slabs and beams of various shapes. Post-tensioning has been used mainly in bridges.

One reason for the limited application of post-tensioning in buildings is that the traditional systems for post-tensioning have been adapted to bridges, where the need for space, grouting and heavy stressing equipment is not a major problem. Another reason is that engineers involved in building design have been unfamiliar with post-tensioning and its potentials.

More recently, however, post-tensioning systems suitable for buildings have been developed, and post-tensioning is becoming more popular among design engineers. Typical applications are beams and slabs with large spans and/or heavy loads, particularly flat slabs where deformations are often a limiting factor.

2.2.3.2 Bonded and unbonded systems

In thin *slabs*, for obvious reasons, tendons and anchorages must have small dimensions. Post-tensioning with *unbonded* tendons then offers some special advantages:

- Single strands tightly enclosed in plastic sheaths need less space than multi-strand tendons in ducts with room for grouting. They can therefore be placed closer to the surface.

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- Large concrete covers are not needed for corrosion protection, since tendons have a built-in protection, and in principle no cover at all is needed for bond. However, conditions for fire protection are the same as for other reinforcement, and may sometimes govern the cover.

- Lighter stressing equipment and the absence of grouting simplifies execution.

Figure 2.6 illustrates the minimum edge distance for unbonded mono-strand tendons and two types of bonded tendons. For bonded tendons, it is assumed that the minimum cover is 30 mm and not less than the duct diameter. For unbonded tendons the cover can be reduced, e.g. to 20 mm (unless fire protection requires more).

For a slab with h = 280 mm the average eccentricity for the two directions will be for the cases illustrated in Figure 2.6:

a) $e = h/2 - t =$ 140 – 20 – 17 = 103 mm
b) $e = h/2 - t =$ 140 – 30 – 21 – 3 = 86 mm
c) $e = h/2 - t =$ 140 – 50 – 50 – 10 = 30 mm

The disadvantage of bonded tendons with circular ducts is striking. With flat ducts bonded tendons can be an alternative also in slabs, although they will still be less effective than unbonded tendons, due to their larger edge distance.

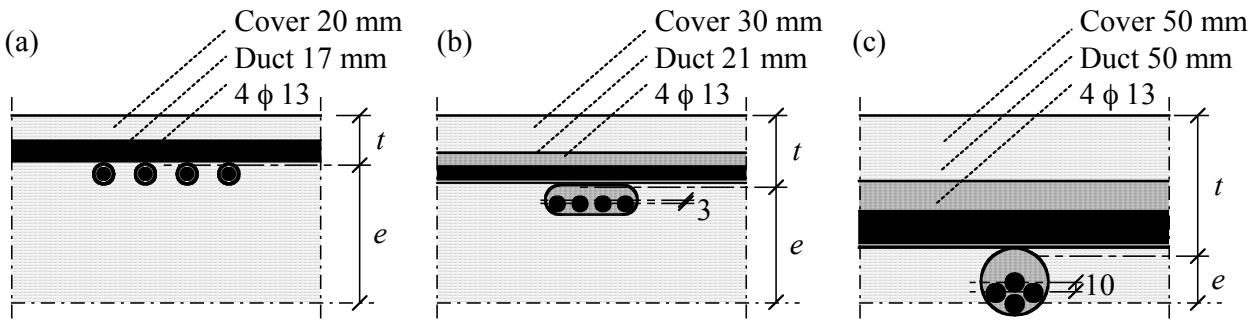

Figure 2.6: Example of (a) unbonded tendons compared to bonded tendons with (b) flat and (c) circular ducts respectively

For *beams*, the disadvantage of reduced eccentricity is normally less pronounced, due to the greater depth of cross section. Bonded tendons can then be more economical than unbonded ones, since one tendon then consists of many strands, giving a higher prestressing force per anchorage.

In the comparison between bonded and unbonded systems, one should also consider the structural importance of bond, particularly in ultimate limit states. The disadvantages of unbonded systems from this point of view must be taken into account in design, see clause 2.3.4.

2.3 Design aspects

This clause deals with general design considerations, such as tendon layout and structural effects of prestress, prestress losses, serviceability and ultimate limit states, anchorages and restraint from adjacent structural components, with focus on particular aspects for buildings.

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2.3.1 Structural effects and tendon profiles

As already outlined in 2.2.2, prestress has two main effects:

- The *axial* effect, causing compression in the concrete, with favourable effects on cracking and deflections and also contributing to shear, torsion and punching resistances.
- The *transverse* effect caused by deviation forces, directly carrying part of the external load to the supports.

With an appropriate tendon layout, the transverse forces can more or less balance part of the external load. A simple example is shown in figure 2.7. For best efficiency, the tendon curve should correspond to the bending moment diagram as far as possible. The transverse forces will then have the same distribution as the external load.

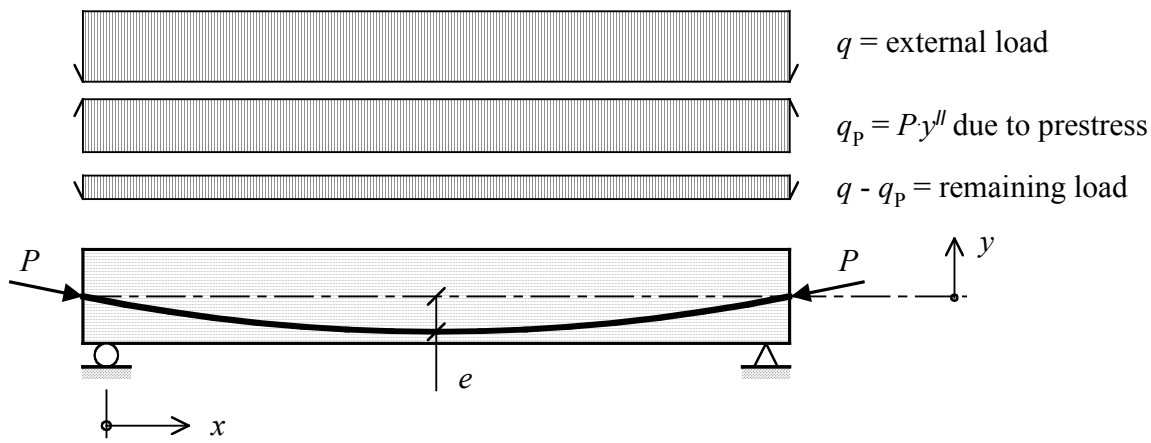

Figure 2.7: The transverse effect of prestress. P = prestressing force, $q_P = P \cdot y''$ = transverse load due to prestress, e = eccentricity of prestressing force, $y'' = d^2y/dx^2$ = tendon curvature (= constant if y is parabolic, as shown in the figure)

In continuous members, the minimum curvature required for the tendons will necessitate a certain distribution of the downward transverse effect over supports. This means that the external moment diagram cannot be completely followed, see figure 2.8. However, this is not a major problem as far as the global behaviour is concerned.

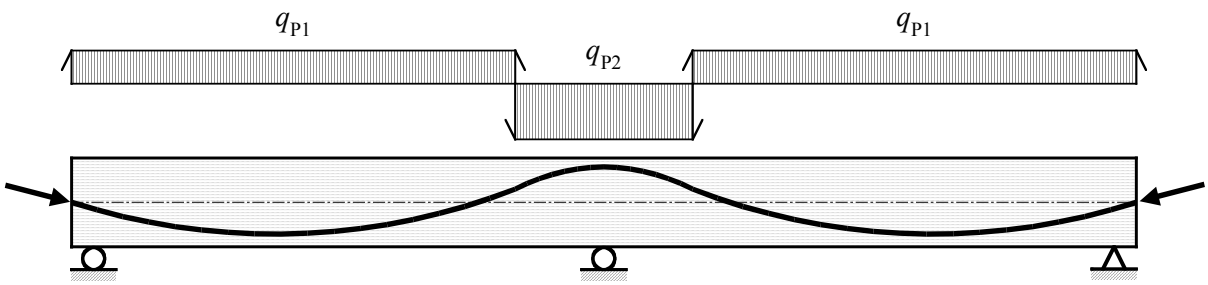

Figure 2.8: Example of tendon layout in a continuous member

Normally, tendons are located centrically at simply supported ends. If not, the eccentricities will give end moments $M_P = P \cdot e$ that should be added to the effect of the transverse

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load q_P, taking into account boundary conditions if the structure is hyperstatic. See figure 2.9. The effect of the eccentricities can also be seen as the "eccentric axial effect" mentioned in 2.2.2.

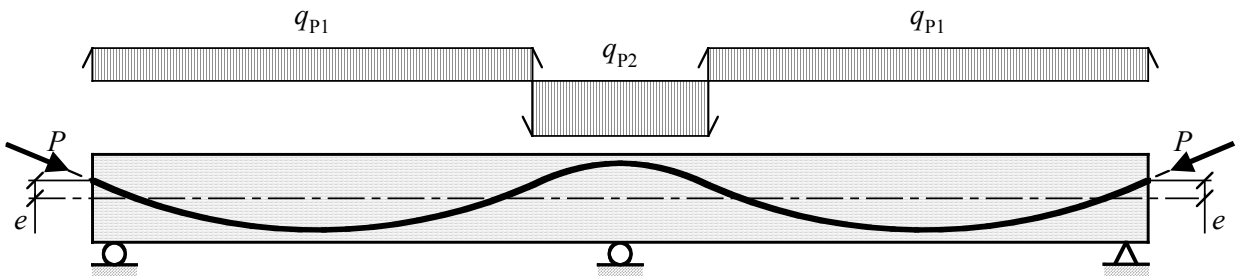

Figure 2.9: Effect of end eccentricities

End eccentricities can be used to enhance a certain effect of prestress. Thus, an upward end eccentricity as shown in figure 2.9 is favourable with regard to shear, whereas a downward end eccentricity will reduce deflections.

If there are large concentrated loads with fixed position, the tendons can be bent in a concentrated curvature at these positions, and be more or less straight in between. This gives concentrated lifting forces to directly balance (part of) the external loads, see figure 2.10.

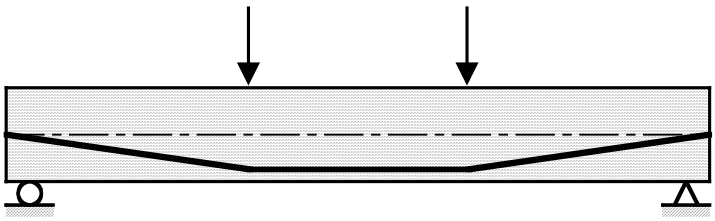

Figure 2.10: Tendon layout with straight parts, in this case to balance concentrated loads

A tendon layout with straight parts is sometimes used also for practical reasons, even if there are no concentrated loads. Parts of the tendons can then be supported directly on the bottom reinforcement, which simplifies execution, especially in slabs.

2.3.2 Prestressing force

2.3.2.1 Maximum prestress

The maximum prestress is limited to values, which are given in codes. As an example the following stress values can be mentioned:

Maximum stress during tensioning: $\sigma_p \leq 0{,}90 \cdot f_{yk}$ and $\leq 0{,}80 \cdot f_{uk}$
Maximum stress after tensioning and anchoring: $\sigma_p \leq 0{,}85 \cdot f_{yk}$ and $\leq 0{,}75 \cdot f_{uk}$

Here f_{yk} and f_{uk} are the characteristic values of yield and ultimate tensile strength respectively. For common types of strands, the maximum stresses will be around 1500 and 1400 MPa respectively.

The effect of the total prestressing *force* on the concrete may impose other limits on the maximum prestress.

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2.3.2.2 Losses of prestress

The prestressing force applied at the anchorage will decrease along the tendon length, and also with time. These so-called losses of prestress are of the following main types:

a) Losses due to elastic shortening of concrete
b) Losses due to friction along the tendon
c) Losses at the anchorage zone due to wedge draw-in
d) Time-dependent losses due to material properties of concrete and steel

The different types of losses will be described in the following, with special emphasis on friction and time-dependent losses.

a) Elastic shortening of concrete

During tensioning the concrete is subjected to compression and a corresponding shortening. If there are several tendons which can not be tensioned at the same time, the force in tendons already tensioned will decrease each time another tendon is tensioned.

The average loss can be related to half the total prestress. The concrete shortening ε_c and the corresponding loss of prestress $\Delta\sigma_{cp}$ is then:

$$\varepsilon_c \approx 0{,}5P / (A_c E_c) = 0{,}5\sigma_c/E_c \qquad \Delta\sigma_p = E_s \cdot \varepsilon_c = 0{,}5(E_s/E_c)\cdot\sigma_c \approx 3\sigma_c$$

(P = total prestressing force, A_c = concrete area, $\sigma_c = P/A_c$, E_c and E_s = E-modulus of concrete and steel respectively; normally, $E_s/E_c \approx 6$)

For post-tensioned slabs in buildings, where typically $\sigma_c \approx 1{,}5$ MPa, the loss will be about 5 MPa, which is quite negligible. For beams and certain types of bridges, prestress may be 5 times as high and the corresponding loss then about 25 MPa; this is still not very much.

b) Friction

The prestressing force decreases with increased distance from the active end due to friction. The variation of the force then follows from

$$P_x = P_0 \cdot e^{-\mu\,\Sigma\cdot(\theta + k\cdot x)}$$

where

P_0	prestressing force at the active end
μ	coefficient of friction
$\Sigma\theta$	sum of angular deviations over distance x (absolute value)
k	unintentional angular deviation per unit length
x	distance from the active end to the section considered

The distance x should in principle be measured along the tendon, but a straight length coordinate can normally be used. The reduction of the prestressing force over a distance x is

$$\Delta P = P_0 \cdot \left[1 - e^{-\mu\cdot(\Sigma\theta + k\cdot x)}\right] \approx P_0 \cdot \mu \cdot (\Sigma\theta + k\cdot x)$$

The values of μ can vary substantially, but there is a fundamental difference between unbonded and bonded tendons. The following values can be taken as indications:

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1) 0,05 for unbonded tendons
2) 0,15 for bonded tendons in plastic sheathing
3) 0,20 for bonded tendons in metal sheathing

In the simplest case, a simply supported beam or slab with parabolically varying eccentricity of tendons, the prestressing force will vary practically linearly from one end to the other, see figure 2.11.

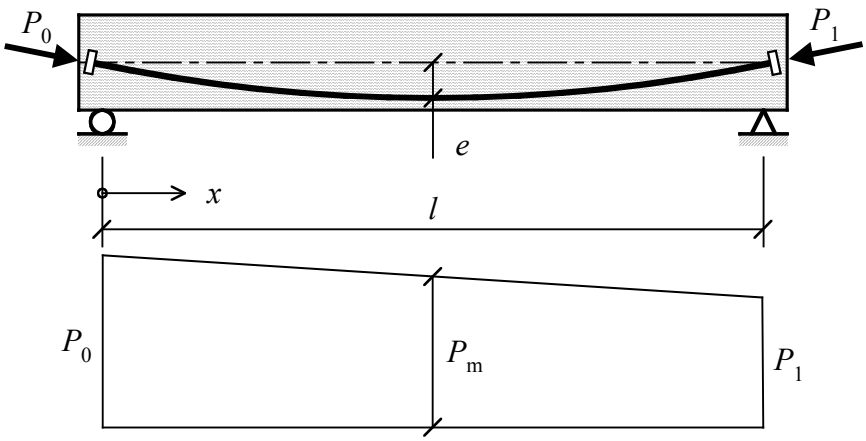

Figure 2.11: Example of variation of prestress due to friction

Example

Compare unbonded and bonded tendons with the friction values indicated above.

l = 10 m, e = 0,2 m. In the middle, $\theta = 4e/l = 4 \cdot 0{,}2/10 = 0{,}08$ rad. Assume $k = 0{,}01$.

1) $\mu = 0{,}05 \Rightarrow P_m = P_0 \cdot e^{-0{,}05 \cdot (0{,}08 + 0{,}01 \cdot 5)} = 0{,}994 \cdot P_0 \Rightarrow$ friction loss = 0,6 % of P_0
2) $\mu = 0{,}15 \Rightarrow P_m = P_0 \cdot e^{-0{,}15 \cdot (0{,}08 + 0{,}01 \cdot 5)} = 0{,}981 \cdot P_0 \Rightarrow$ friction loss = 1,9 % «
3) $\mu = 0{,}20 \Rightarrow P_m = P_0 \cdot e^{-0{,}20 \cdot (0{,}08 + 0{,}01 \cdot 5)} = 0{,}974 \cdot P_0 \Rightarrow$ friction loss = 2,6 % «

For cable layouts of continuous members these losses are naturally bigger and would, in normal situations, amount to values of the order of 5% to 20%.

c) Wedge draw-in

When tendons are locked in the anchorage, a certain displacement (draw-in) occurs before the wedges have full grip, see figure 2.12. This causes a reduction of the prestressing force, which can be calculated if the magnitude of the draw-in is known (this is usually stated in the technical specifications for a certain system).

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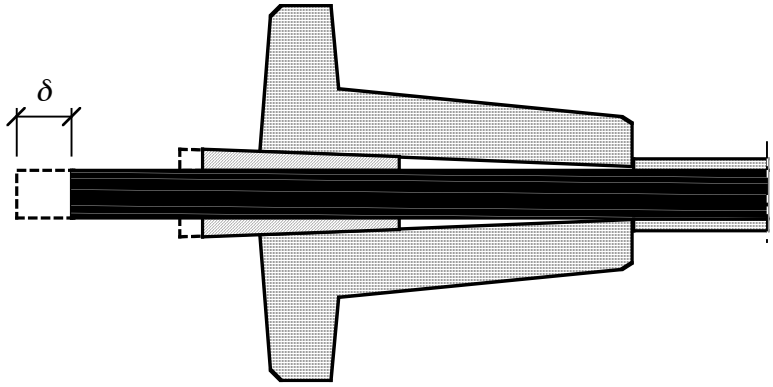

Figure 2.12: Wedge draw-in

Figure 2.13 shows the variation of the prestressing force before and after wedge draw-in (exaggerated), and for the cases of normal length and short tendons.

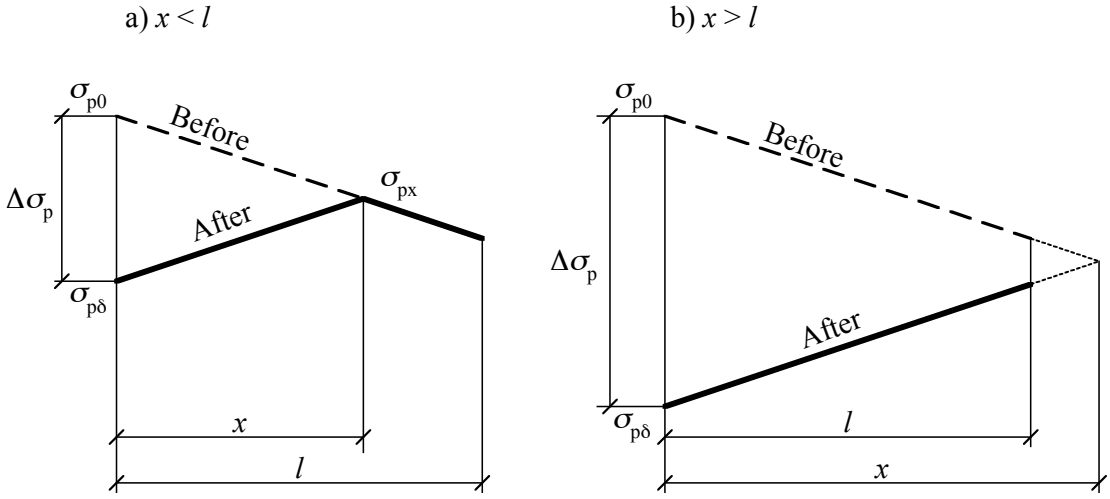

Figure 2.13: Variation of prestress before and after wedge draw-in for a normal length and a short tendon respectively

In case a) we have

$$\delta = \frac{\Delta\sigma_p}{E_s} \cdot \frac{x}{2}$$

δ = wedge draw-in

$$\Delta\sigma_p = \sigma_{p0} - \sigma_{p\delta} \approx 2 \cdot \sigma_{p0} \cdot \mu\,(\theta + kx) = 2 \cdot \sigma_{p0} \cdot \beta \cdot x$$

$\beta = \mu\,(\theta / x + k)$ = average relative friction loss per unit length

$$x = \sqrt{\delta / \beta\varepsilon_{p0}} \quad \text{and} \quad \Delta\sigma_p = \sigma_{p0} \cdot 2\sqrt{\beta\delta / \varepsilon_{p0}}$$

$\varepsilon_{p0} = \sigma_{p0}/E_s$ = initial prestrain

In case b), which occurs if the resulting x is greater than l, the following value of the loss at the active end can be derived:

$$\Delta\sigma_p = E_s \cdot \delta / l + \sigma_{p0} \cdot \beta \cdot l$$

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Example

l = 10 m, e = 0,2 m (parabolic), k = 0,01, δ = 6 mm, σ_{p0} = 1000 MPa $\Rightarrow$ ε_{p0} = 0,005

Parabola $\Rightarrow$ $\theta/x = 8e/l^2$ and $\beta = \mu\cdot(\theta/x + k) = 0{,}05\cdot(8\cdot 0{,}2/10^2 + 0{,}01) = 0{,}0013$

$x = \sqrt{0{,}006 / (0{,}0013 \cdot 0{,}005)}$ = 30 m.

Since $x > l$ we have case b):

$\Delta\sigma_p = E_s\cdot\delta/l + \sigma_{p0}\cdot\beta l = 200000\cdot 0{,}006/10 + 1000\cdot 0{,}0013\cdot 10 = 120 + 13 =$ <u>133 MPa</u>

d) Time-dependent losses

The prestress will decrease with time due to shrinkage and creep in the concrete, plus relaxation of the tendons. Different expressions for the time-dependent loss can be found in codes. However, the basic expression is always

$$\Delta\sigma_p = E_s\varepsilon_{cs} + E_s\varphi\frac{\sigma_{cp}}{E_c} + \chi\sigma_{sp}$$

where

E_s	E-modulus of steel
E_c	E-modulus of concrete
ε_{cs}	shrinkage of concrete
φ	creep coefficient of concrete (creep = strain increase under constant stress)
σ_{cp}	concrete compressive stress at level of tendons for quasi-permanent load
χ	relative relaxation loss (relaxation = stress decrease under constant strain)
σ_{sp}	stress in tendons

The physical meaning of the basic expression is simple: the first two terms express the stress decrease due to concrete shortening from shrinkage and creep respectively, the third term expresses the stress decrease due to prestress steel relaxation, given by the coefficient χ.

The concrete stress σ_{cp} should be evaluated for the quasi-permanent (long-term) load combination together with prestress.

Example

σ_{p0} = 1340, σ_{cp} = 3 MPa; E_p = 200, E_c = 30 GPa; ε_{cs} = 0,0003, φ = 2,5, χ = 0,03

Shrinkage: $\Delta\sigma_{ps} = E_s\cdot\varepsilon_{cs} = 200\cdot 10^3\cdot 0{,}30\cdot 10^{-3} = 60$ MPa

Creep: $\Delta\sigma_{pc} \approx (E_p/E_c)\cdot\varphi\cdot\sigma_{cp} = (200/30)\cdot 2{,}5\cdot 3 = 50$ MPa

Relaxation: $\Delta\sigma_{pr} \approx 0{,}03\cdot 1340 = 40$ MPa

$\Delta\sigma_p = 60 + 50 + 40 =$ <u>150 MPa</u>

$\Delta\sigma_p/\sigma_{p0} = 150/1340 = 0{,}11 =$ <u>11 %</u>

This is a typical value; time-dependent losses are generally between 10 and 15 %.

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2.3.3 Serviceability limit states (SLS)

The governing design criteria for prestressed structures are normally those relating to service conditions, the so-called *serviceability limit states* (SLS). The reason for choosing prestressed concrete is often a large span and/or a requirement on reduced depth, leading to a high span-depth ratio. In such cases, deflections often become critical.

If concrete is uncracked, the deflection (see figure 2.14) can be expressed in the following general way:

$$a = \frac{k_1 q l^4}{EI} = \frac{k_1 q l^4}{E \cdot k_2 b h^3}$$

$$\frac{a}{l} = k_3 \frac{q}{bE}\left(\frac{l}{h}\right)^3$$

where

q	distributed load
l	span length
E	concrete modulus of elasticity
I	moment of inertia of cross section
k_1	coefficient depending on load distribution and boundary conditions
k_2	coefficient depending on cross section geometry
k_3	k_1/k_2
b	width of cross section
h	depth of cross section

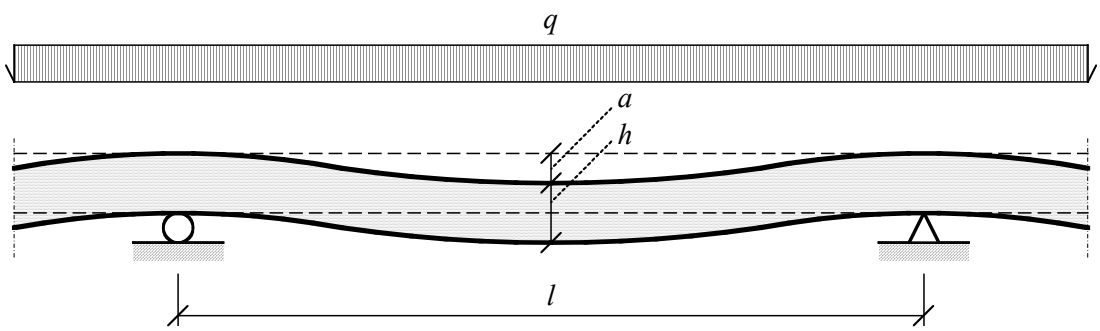

Figure 2.14: Illustration of deflection a, length l, depth h and load q

The above expressions shows that the absolute deflection a is proportional to l^4 and the relative deflection a/l is proportional to $(l/h)^3$. This illustrates the strong influence on deformability of the span length and the span-depth ratio.

Prestress can be designed so that the deformation under a certain load, e.g. permanent or quasi-permanent, is partially or totally balanced by the transverse effect. In this way deformations can be kept within acceptable limits.

However, there are limits to the possible slenderness with regard to economy and structural behaviour: excessive quantities of prestressing steel should be avoided and the structure should have sufficient stiffness for variable loads, sometimes also with regard to vibrations.

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In the final deformation calculation, concrete creep has to be taken into account. The *additional* deflection due to concrete creep is, provided the structural member is uncracked:

$$a_{\text{creep}} = \varphi \cdot (a_{\text{g0}}^{\downarrow} + a_{\text{P0}}^{\uparrow}) \approx 2{,}5(a_{\text{g0}}^{\downarrow} + a_{\text{P0}}^{\uparrow})$$

where

φ creep coefficient, see below

$a_{\text{g0}}^{\downarrow}$ downward deflection due to quasi-permanent load

$a_{\text{P0}}^{\uparrow}$ upward deflection due to transverse effect of prestress

The creep coefficient φ depends on concrete composition and quality, ambient humidity etc. Values 2 and 3 are typical for outdoor and indoor conditions respectively. Thus, due to creep the *total* deflection may become 3 to 4 times the *immediate* (elastic) deflection.

The limits for acceptable deflections depend on the type of structures. They are generally not stated in much detail in codes, instead they must often be specified by the client. For large spans and/or span-depth ratios, it is often impossible to fulfil the deformation requirements without prestress. In the specification of deflection limits, different criteria and different types of deflection must be considered. For example, additional deflections have implications with regard to damage to non-structural building components, total deflections have implications with regard to function and appearance, initial deflections can be compensated by pre-camber, etc. Therefore, a specification like “deflection not greater than span/500” or similar is too vague. It must also be stated whether a limit concerns immediate, additional or total deflections, and to which load combination it applies.

With regard to deflections and other serviceability criteria, the slenderness l/h of prestressed members can be up to 1,5 to 2 times that of reinforced concrete members. The prestressing is generally designed for a transverse effect corresponding to about 70 to 90% of permanent or quasi-permanent loads. With such prestress, crack control is generally not a major issue, since there will be compression or only small local tension under quasi-permanent load. Cracks as such are generally not a problem in normal buildings, but cracking may have a significant effect on deflections, and may therefore have to be avoided or limited in frequent or rare load combinations. Cracks may also be undesirable with regard to appearance.

With regard to the possibility of cracking under a high variable load, or due to imposed deformations (sometimes not foreseen in design), minimum reinforcement should in general be provided for crack control. Minimum amounts of reinforcement are generally well-defined in modern codes of practice.

2.3.4 Ultimate limit states

For load bearing structures it is not sufficient to limit deflections, cracks and stresses in SLS. To ensure a certain safety margin against failure or collapse, also the so-called *ultimate limit states* (ULS) have to be considered. All possible failure modes should be considered, e.g. bending, shear, punching, torsion, anchorage of reinforcement and prestressing tendons, etc.

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Design models and criteria for ULS verifications are generally treated in sufficient detail in codes. Therefore, in this document only some particular aspects will be treated, especially those related to the effects of prestress. These effects can be considered in different ways: as an imposed deformation, an equivalent load or a contribution to resistance. Apart from that, certain types of resistances are improved due to the axial effect of prestress: shear, torsion, punching. In design verifications it is important to consider these different effects in a consistent way, so that no single effect is taken into account more than once.

Concerning the *bending resistance*, a major advantage of prestressing is that steel with very high strength can be used. Without prestress, the utilisation of high steel stresses will lead to excessive deflections and cracking in SLS. With a high strength, 3 to 4 times that of ordinary reinforcement, the steel area necessary to achieve a certain bending resistance can be reduced proportionally. This, together with the possibility of grouping several strands in few tendons, clearly facilitates the detailing of the tension zone.

The ultimate bending resistance is

$$M_{Rd} = F_T \cdot z$$

where F_T is the total tensile force and z is the internal lever arm, see figure 2.15. The tensile force F_T is approximately given by

$$F_T = A_s f_{yd} + A_p f_{pd} \quad \text{for bonded tendons} \qquad (1)$$

$$F_T = A_s f_{yd} + P + A_p \Delta\sigma_p \quad \text{for unbonded tendons} \qquad (2)$$

where

f_{yd}	design strength of ordinary reinforcement
f_{pd}	design strength of prestressing tendons
$P = A_p \sigma_p$	prestress stress force
σ_p	prestress stress
$\Delta\sigma_p \approx 100$ MPa	stress increase above prestress for unbonded tendons

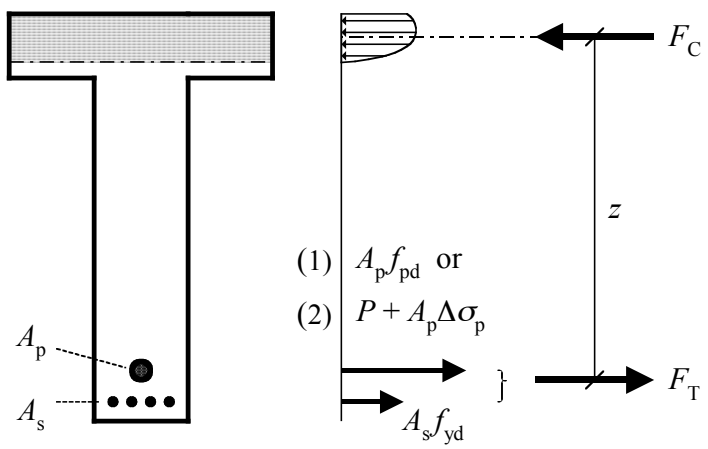

Figure 2.15: Examples of stresses and forces in a prestressed concrete section for the ultimate limit state of bending

Bonded tendons (1) are locally elongated at cracks. This gives a higher strain and a correspondingly high stress increase, whereby the total strength of the total steel can be utilized.

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Unbonded tendons (2) are instead elongated uniformly between the anchorages. This may involve several spans in a continuous beam or slab. As a consequence, the strain and the corresponding stress increase at failure is limited, often leaving the total stress below yielding, even with extensive cracking. The value 100 MPa is a rough estimate of the stress increase, which can be used in the absence of a detailed calculation. In such a calculation, the global structural deformation has to be considered, including possible restrictions with regard to plastic rotation capacities.[1]

Example

The ultimate bending capacities are compared for a cross section with (1) bonded and (2) unbonded tendons and the following data: A_s = 1256 mm^2 (4ϕ20); A_p = 2100 mm^2 (2 tendons, each with 7 strands at 150 mm^2); σ_p = 1050 MPa; f_{pd} = 1680/1,15 = 1460 MPa; f_{yd} = 500/1,15 = 435 MPa, z = 1,25m.

(1) $M_{Rd} = 10^{-3}\cdot[1256\cdot 435 + 2100\cdot 1460]\cdot 1{,}25 = (546 + 3066)\cdot 1{,}25 = 4515$ kNm

(2) $M_{Rd} = 10^{-3}\cdot[1256\cdot 435 + (1050+100)\cdot 2100]\cdot 1{,}25 = (546 + 2415)\cdot 1{,}25 = 3700$ kNm

Thus, in this case only 80 % of the potential capacity can be used if tendons are unbonded.

Shear resistance

With curved tendons, the main contribution of prestress to the *shear resistance* is normally given by the inclination of tendons, as illustrated in figure 2.16, i.e. the transverse effect of prestress. The contribution to the shear resistance is simply the transverse component of the prestressing force. Using a so-called truss model for the design of shear reinforcement, and with prestress considered on the "action side", the design criterion is:

$$V_{Rdsy} \geq V_{Sd} - P\tan\alpha$$

where $V_{Rdsy} = \dfrac{A_{sw}}{s} f_{yd}\cdot z\cdot\cot\theta$ resistance of shear reinforcement, see figure 2.16.

Alternatively, with prestress considered as a contribution to the resistance:

$$V_{Rd} = V_{Rdsy} + P\tan\alpha \geq V_{Sd}$$

Thus, the only difference is whether the contribution of inclined tendons is placed on the right or left hand side of the design equation.

[1] The lack of bond also has other implications. Thus, a local failure will have consequences along the whole length between anchorages, and in a continuous member several spans may suffer from loss of prestress. This should be considered particularly with regard to accidental actions and unintentional cutting of tendons for openings. (In the case of planned cuttings, it is possible to install new anchorages in existing tendons.)

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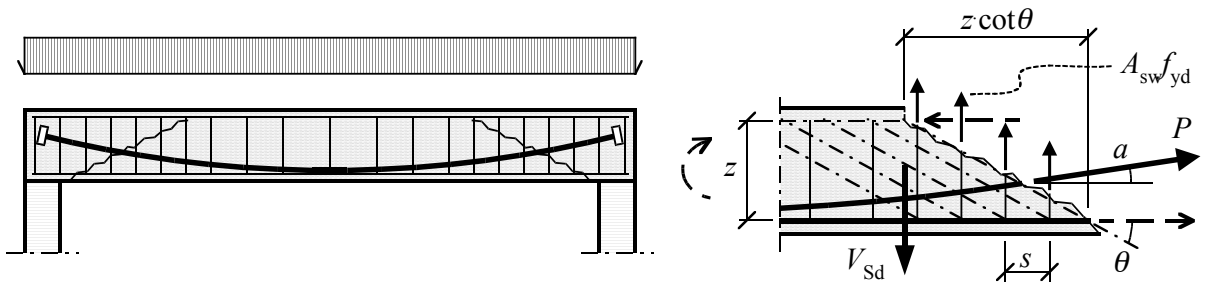

Figure 2.16: Shear ultimate limit state of a prestressed concrete element

Example

d = 1,4 m; $z \approx 0{,}9d$ = 1,26 m; A_{sw}/s = 1050 mm^2/m (stirrups ϕ10s150); A_p = 2100 mm^2; P=2200kN; tanα = 0,1; cotθ = 2 ($\theta \approx 26^o$ = inclination of compression struts in truss)

$V_{Rd} = 2200 \cdot 0{,}1 + 10^{-3} \cdot 1050 \cdot 435 \cdot 1{,}26 \cdot 2 = 220 + 1150 = 1370$ kN

The *axial effect* of prestress also has an effect on the shear resistance, particularly for members without shear reinforcement. For members with shear reinforcement, the contribution from the axial effect will be different depending on which design model is used. In the truss model, the axial effect of prestress is generally small, often zero. In other models (e.g. addition model or modified compression field theory) this effect can be more significant. However, the effect of inclination as shown above is the same for all models.

In *punching of slabs* the effect of prestress is similar, but only tendons close to the column can be taken into account. Figure 2.17 gives an indication of which tendons can be taken into account; the distances x and y can be found in some codes being x generally limited to $d/2$. The axial effect of prestress is favourable also in punching, but less than in shear. It also depends on the problem of prestress "disappearing" into adjacent members, see paragraph 2.3.6.

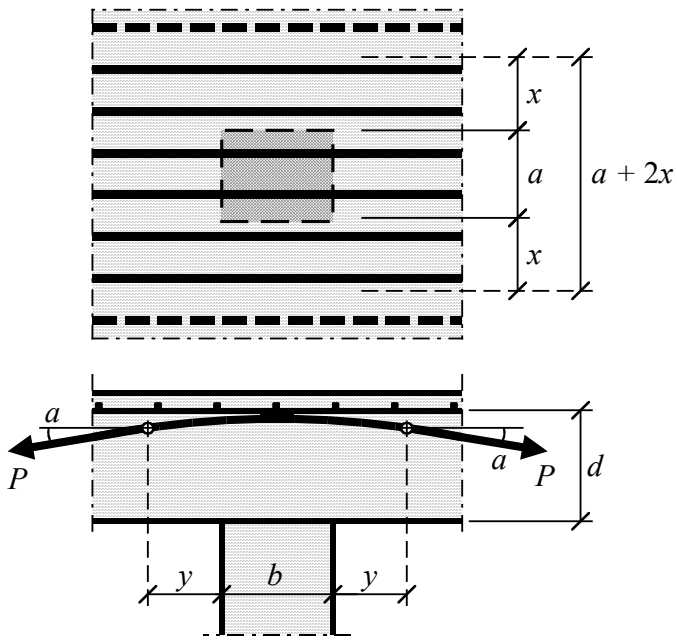

Figure 2.17: Tendons contributing to the punching resistance in slabs

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2.3.5 End anchorage and intermediate anchorages

The prestressing force is transferred to the concrete at the anchorages. Anchorages where stressing takes place are called *active* anchorages, whereas the others are called *passive*. Sometimes stressing is made from both ends of a tendon to reduce friction losses.

Where tendons cross a construction joint, *intermediate* anchorages can be placed. After hardening of the concrete in the first pour, the tendons are stressed and locked. The tendons in the next pour are connected to the ones already stressed, and stressed after hardening of this concrete. It is not necessary to have intermediate anchorages in all construction joints, or in all tendons in one joint; tensioning can also be postponed to subsequent joints.

At the anchorages, a complex state of stresses is generated. Two aspects deserve special attention: high local compressive stresses at the anchorages, and tensile stresses caused by the dispersion of the compressive stresses over a larger area. The compressive stresses must be limited, and the tensile stresses normally have to be equilibrated by reinforcement. Figure 2.18 illustrates these two aspects, particularly how to take care of the tension. The tension is taken by reinforcement, and the compression is limited with regard to the concrete strength at the time of tensioning.

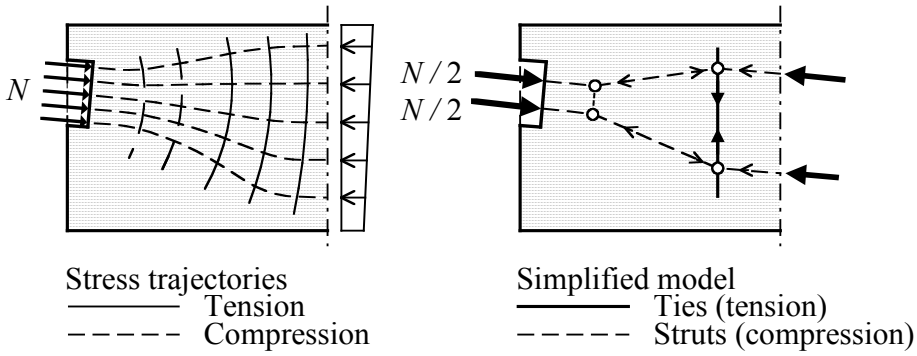

Figure 2.18: Distribution of stress at an anchorage zone and typical detailing

In the detailing of prestressing solutions in buildings, the geometry of anchorage zones may have to be designed in order not to interfere with architectural and functional needs.

2.3.6 Structural restraints

In long structures, joints may be needed to allow movements due to prestressing, shrinkage and temperature changes. The axial effect of prestress produces immediate elastic shortening of the concrete, later increased by creep. Shrinkage always occurs, whereas significant temperature changes mainly occur in outdoor structures. Examples of the magnitude of different types of movements are given below:

	Movement in mm/m			
	Nordic countries		Mediterranean countries	
	Outdoors	Indoors	Outdoors	Indoors
Prestress	0,15	0,20	0,20	0,20
Shrinkage	0,25	0,40	0,30	0,30
Temperature	0,40	0,00	0,30	0,10
Total	0,80	0,60	0,80	0,60

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The example is based on a prestress of 1,5 MPa in the concrete. The following has been assumed for both regions (outdoors / indoors respectively): creep coefficient 2 / 3, shrinkage 0,25 / 0,4 $^{o}/_{oo}$. The temperature decrease has been assumed to 40 / 0° for Nordic and 30 / 10° for Mediterranean countries. The values given are only indications, not general recommendations.

Due to the axial effect of prestress, the need for movement joints is more pronounced in a prestressed floor than in the same floor without prestress. The need is also generally more pronounced with post-tensioned cast in situ floors than with pre-tensioned precast elements. In these last elements, all the elastic shortening plus part of the creep and shrinkage has occurred before assembly, furthermore there is no prestress in the transverse direction and movements can be distributed between the many joints.

The need for movement joints depends not only on the dimensions of the building, the type of floor etc, but also on the layout of the *stabilizing system*. Some schematic examples are shown in figure 2.19 to illustrate this.

In examples a), b) and c) there will be no significant forces due to restraint, since the stabilizing walls have a low out-of-plane stiffness. On the other hand, the *unrestrained movements* may have effects on non-structural parts of the building, such as windows, partitions etc.

In examples d) and e) the restraint forces may become significant, particularly if the floor is post-tensioned in the longitudinal direction of the building. It may be necessary to arrange some possibility for movement, either between the floor and one of the stabilizing units, or in the floor itself. The second alternative may require additional stabilization, since the joint will interrupt the stiffness and bending capacity of the floor diaphragm. *Without joints*, various aspects of restraint have to be taken into account in design, among other things a reduction of the axial effect of prestress. This will be further discussed below.

In cases like d) and e), the effects of restraint should be analysed, taking into account the magnitude of unrestrained movements (cf. example above), the stiffness of floor and restraining structural components, favourable effects of concrete relaxation and cracking, etc.

The effects of shortening and restraint described above also have another aspect, namely that part of the prestress may ”disappear” into adjacent members, which are not primarily intended to be prestressed. This means that the axial effect is reduced in the prestressed member, and this must be taken into account in design. It may also have undesirable effects on the adjacent members concerned.

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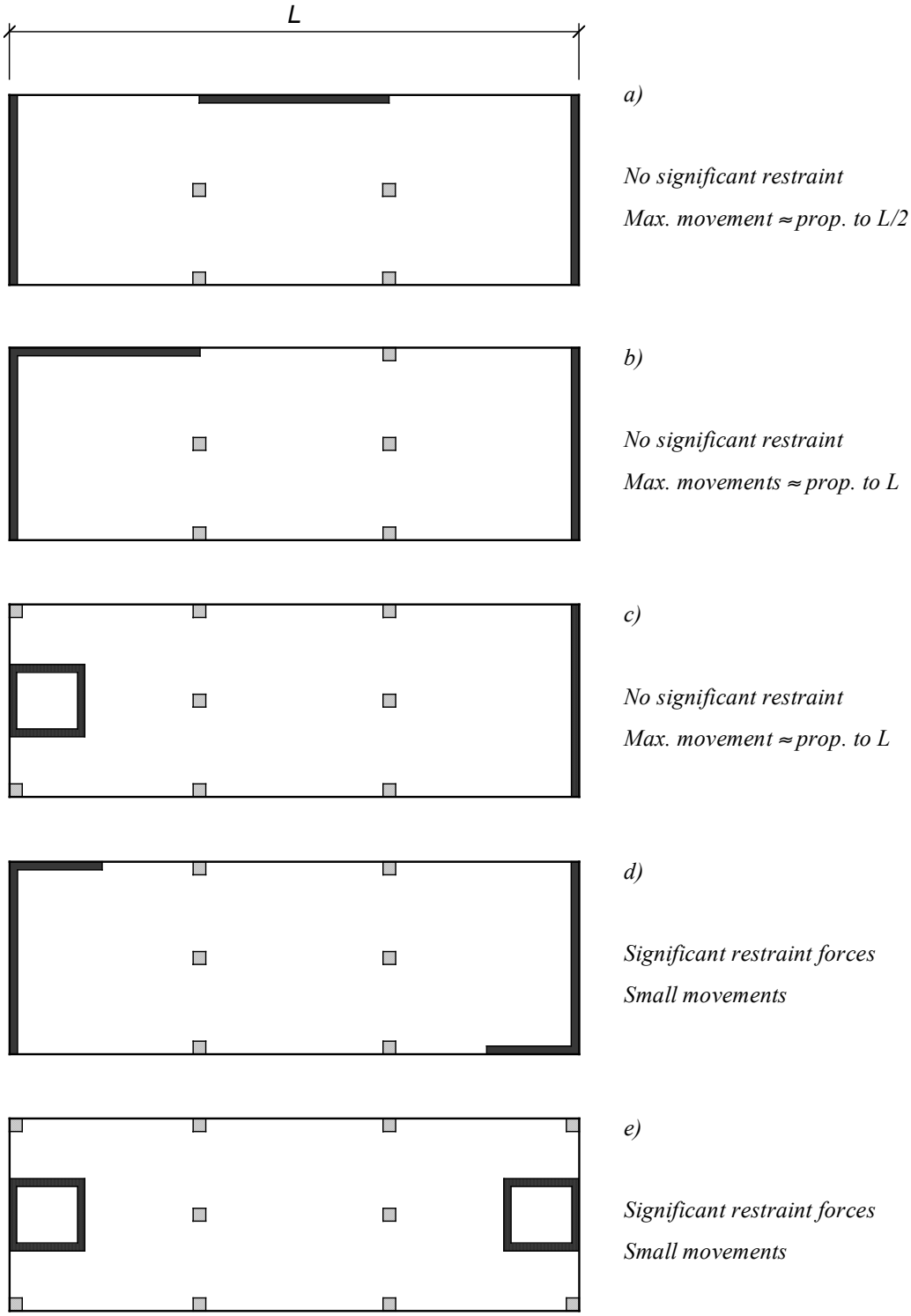

Figure 2.19: Examples of stabilizing systems and their effect with regard to restraint

A typical example is a floor supported on walls, as is often the case with underground and bottom floors, see figure 2.20. Cracks as indicated may occur in the floor due to restraint from the walls. The cracks are not necessarily a major problem as such (if appropriate minimum reinforcement is provided), but they may give an uneven distribution of the axial effect of prestress in the floor, due to arch action. In this particular example it would reduce the compressive stress on the central zone of the floor, where in this example it could be particularly useful with regard to the punching resistance.

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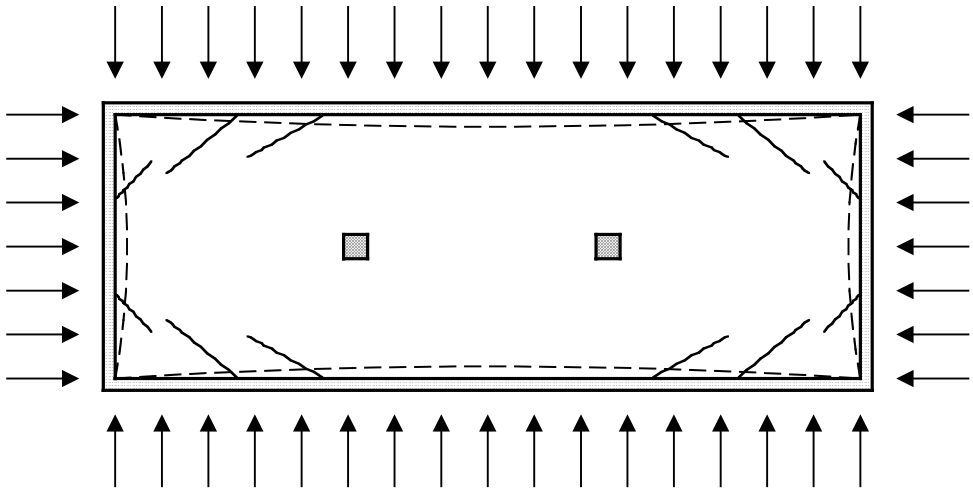

Figure 2.20: Shortening and cracks in floor due to prestress and restraint from walls

The problem can be avoided by reducing restraint through movement joints, or by prestressing also the walls. In the latter alternative the movements in the walls could be made similar to those of the floor, but the result could also be that the problem is moved to some other part of the building, e.g. the connection of the wall to a rigid foundation.

It should be kept in mind that, wherever the axial effects of the prestress end up, the *transverse effects will always act fully on the prestressed member*, and can be accounted for in every aspect of design.

2.4 Technology of prestressing in building

This section illustrates some examples of post-tensioning systems currently used in building construction.

2.4.1 The monostrand post-tensioning system with unbonded greased and sheathed strand

For thin construction elements such as slabs in building, the monostrand post-tensioning system was developed to suit efficient construction methods. These light, flexible monostrands can be easily and rapidly installed and as there is no grouting can lead to economical solutions. Each end of the strand is anchored in an individual anchorage device.

Components:

The following types of anchorage are available: stressing anchorage, dead end anchorage and coupler.

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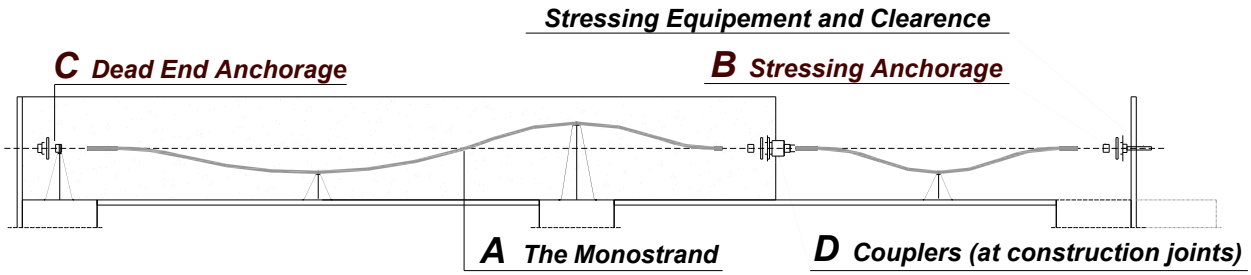

Figure 2.21: Schematic layout of the mono-strand post-tensioning system

A The monostrand

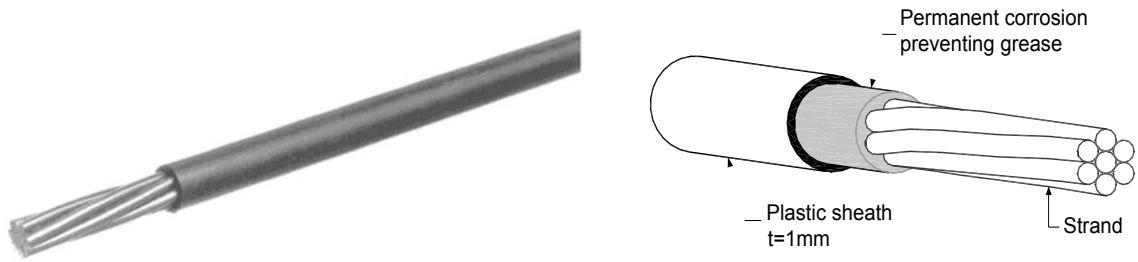

Figure 2.22: Structure of the monostrand

The monostrand is a 7-wire strand of patented cold-drawn twisted wires which have been stress-relieved or stabilised. In the factory or workshop, the strand is first given a continuous coating of permanent corrosion preventive grease and then a plastic sheath of either polyethylene or polypropylene is extruded or pushed over the greased strand.

The quality and dimensions of the materials vary from one country to another and therefore careful attention should be given to the criteria and codes (EN 10138-1, BS 5896 or ASTM A416). It is believed the EN standard, which combines the specifications of the materials certified in the member states of CEN (Comité Européen de Normalisation), will soon become the common standard.

Figure 2.23: Views of monostrand bundles at anchorage and over support. Monostrands can be supplied from the workshop already bundled and placed in this form. A monostrand bundle may comprise a group of 2, 3 or at most 4 strands

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Recommended design values:

- Spacing of tendon supports 0.6 to 1.5 m
- Minimal radius 2.5 m
- Friction coefficient $\mu = 0.06$
- Unintentional angular deviation per unit length $k=\mu*\Delta\alpha = 0.0005\ m^{-1}$

Corrosion protection

The corrosion protection should be in accordance with ***fib*** or PTI recommendations. The plastic sheathing (PE or PP) forms the primary protection of the monostrand and the corrosion preventive grease the secondary protection.

Special product

Internal Unbonded Tendons

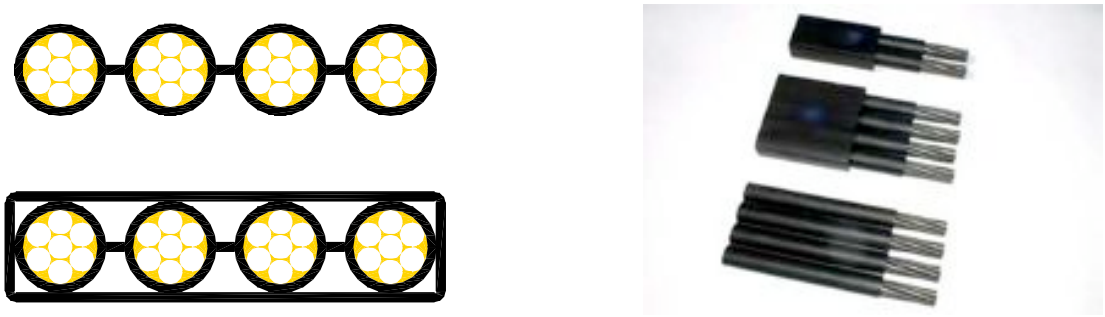

Figure 2.24: Tendons connected by webs to a flat band (VT-CMM-System)

B Stressing anchorage

The components of the stressing anchorages are the anchorage body of cast steel (sprayed at the workshop with a corrosion preventive oil) with wedges, a polyethylene sealing sleeve and the recess former. The fixation of the stressing anchorage is done by setting out and marking of cable axes on stop-end formwork, drilling a hole ∅ 30 - 35 mm for passage of the recess former fastener, then fastening the recess former to the stop end with the lock nut.

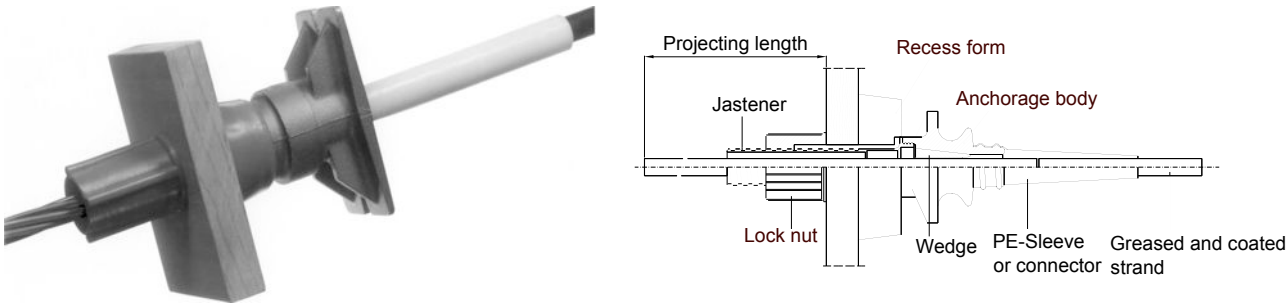

Figure 2.25: Anchorage elements

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Corrosion protection:

The corrosion protection of the strand portion and wedges in the anchorage body is critical. The internal cavity of the anchorage body is therefore injected under pressure with permanent corrosion protective grease and closed by a grease-filled PE-protective cap.

The plastic sealing sleeve, which is pushed or screwed on the transition pipe of the anchorage body, seals the transition zone between anchorage body and PE strand sheath. To protect the anchorage body from external influences, the block-out is afterwards carefully packed with mortar.

Figure 2.26: System with enhanced corrosion protective properties

C Dead end anchorage

The dead end anchorage is identical in appearance to the stressing anchorage. It is usually fitted onto the tendon at the workshop. The wedges are pressed in and secured against backwards movement.

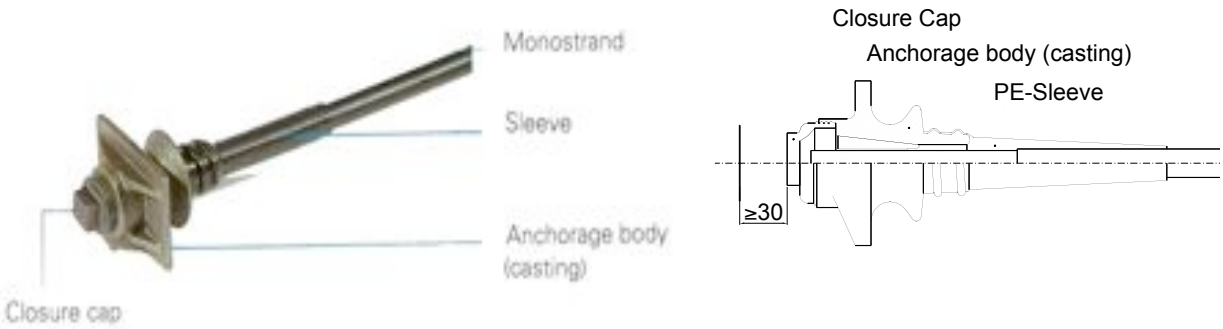

Figure 2.27: Anchorage elements

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Corrosion protection:

Once again, the cavities in the anchorage are injected with a corrosion preventive grease and the anchorage body is sealed with the closure plug. The plastic sealing sleeve seals the transition zone between anchorage body and PE strand sheath.

The dead end anchorage is fixed to the formwork in such a manner that, once concreted, there is sufficient concrete cover to protect the anchorage permanently against external influences. Other systems with a lower degree of protection consist in a dead end casting combined with compression fitting.

D Couplers

As extensive floor areas are subdivided into smaller manageable pouring stages and post-tensioned in sections, the cables at the construction point are connected with couplers to the cables that have been already stressed.

All the couplers are practically based on the same concept and consist of a coupling body with coupling head and threaded coupling.

The coupling head is screwed in the coupling body of the stressed cable. The strand is then inserted into the self-gripping locking device of the coupling head.

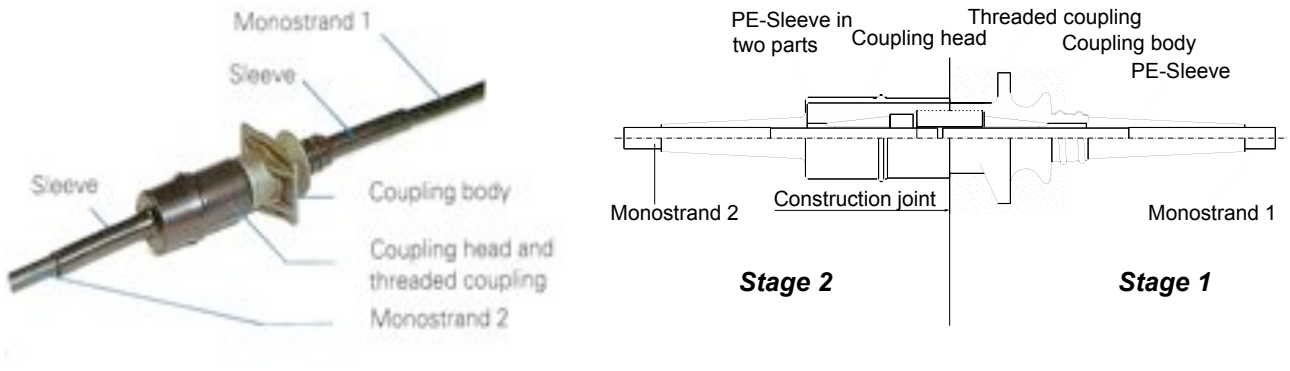

Figure 2.28: Coupler elements

Corrosion protection:

The corrosion protection is the same as for stressing anchorage with the coupling being injected with a permanent corrosion protection grease. Setting a sleeve coated with grease over the coupling head completes the corrosion protection. Aside from the usual method of protection, some systems with additional special protection in the sleeve between the end of the plastic sheath of the monostrand and the anchor head can be found.

2.4.2 The bonded slab post-tensioning system

As an alternative to the unbonded monostrand, the bonded post-tensioning system is also particularly suitable for thin construction elements in building and bridges such as transverse deck slab pre-stressing. Due to the flat profile of the duct, the static depth can be more efficiently utilised and the cable eccentricity improved in comparison to a round duct for the same number of strands.

Each strand end is anchored either in an individual anchorage device (monostrand) or more often in a common anchor for up to 5 strands contained in flat-shaped ducting and anchorages.

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Strands are individually stressed and gripped by the normal wedge action. After stressing the duct is injected with a cementicious grout, which bonds the strands to the surrounding concrete. As a result of the bonding, the stressed tendon has a higher capacity at ultimate design.

2.4.2.1 The monostrand system

In case of bonded monostrand tendons, a corrugated metal or plastic conduit is used, which is grouted after completion of the stressing operation.

A special transition piece (grout connector or grout pipe) allows for grouting.

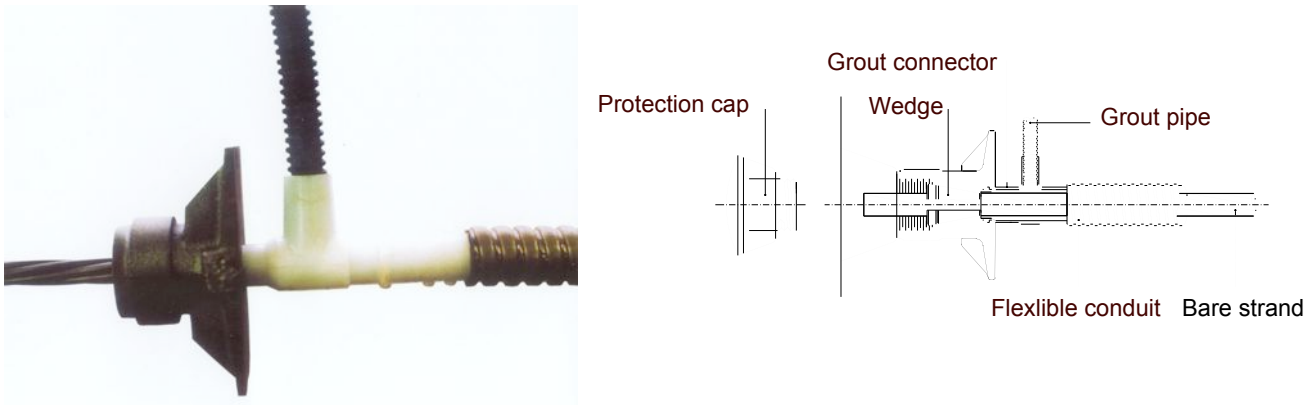

Figure 2.29: Elements of the bonded monostrand tendon

2.4.2.2 The multistrand system

Components:

The following types of anchorage are available: stressing anchorage, dead end anchorage and coupler.

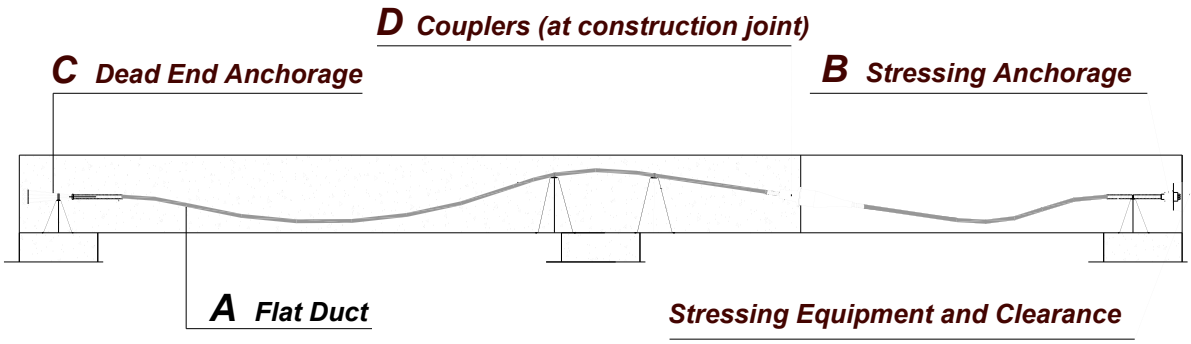

Figure 2.30: Schematic layout of the multistrand post-tensioning system

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A Flat duct

Tendons in standard corrugated steel flat ducts
Recommended design values:

Spacing of tendon support 0.8 to 1.0 m
Minimal radius 2.5 m (vertical)
6.0 m (horizontal)
Friction coefficient μ =0.2
Unintentional angular deviation per unit length $k = \mu * \Delta\alpha = 0.0008$

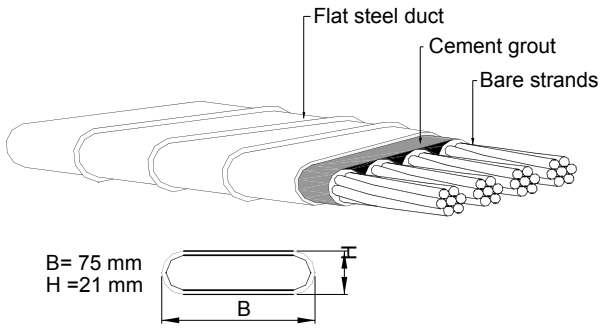

Figure 2.31: Steel flat ducts

Tendons in corrugated polyethylene or polypropylene ducts
Recommended design values:

Spacing of tendon support
Minimal radius 2.5 m (vertical)
6.0 m (horizontal)
Friction coefficient $\mu = 0.14$
Unintentional angular deviation per unit length $k = \mu * \Delta\alpha = 0.0010$

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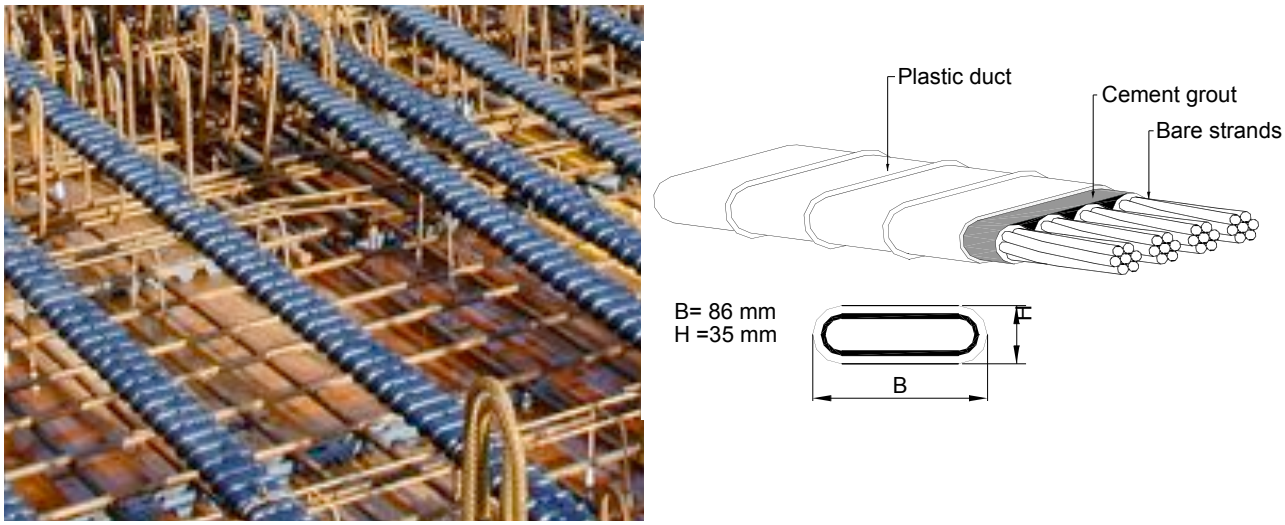

Figure 2.32: Polyethylene flat ducts(PT-Plus TM)

B Stressing anchorage

Anchorages for flat duct system can be differentiated in three groups:

One piece comprising of both bearing plate and anchor head, which is installed into a block-out after concreting; the four strands, which lie alongside one another in the flat tendon are individually threaded through the anchorage and stressed. After concreting, the reusable block-out form is removed with the end formwork.

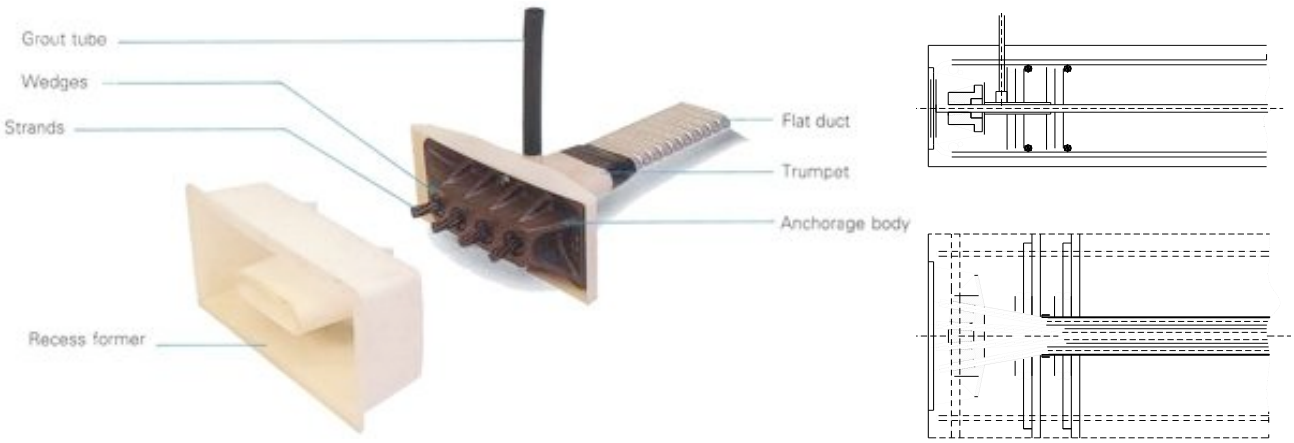

Figure 2.33: Components of a stressing anchorage of group 1

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Figure 2.34: Stressing anchorage of group 1

One piece comprising of both bearing casting and anchor head where the casting is installed in the same way as a casting for the multi-strand system

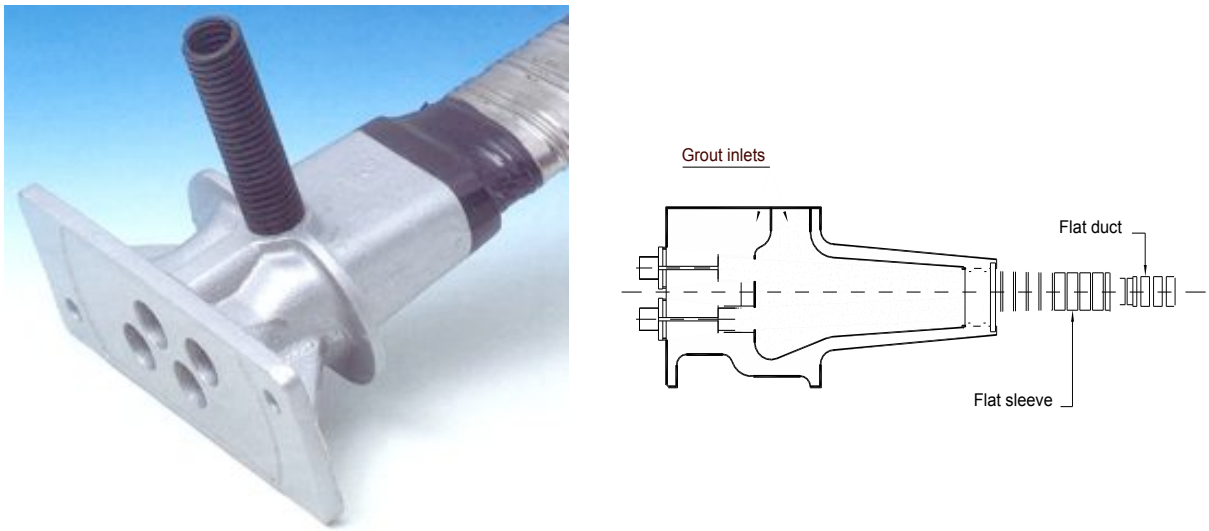

Figure 2.35: Components of a stressing anchorage of group 2

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Two pieces with a casting and an anchor block allowing the stressing of 4 or 5 strands.

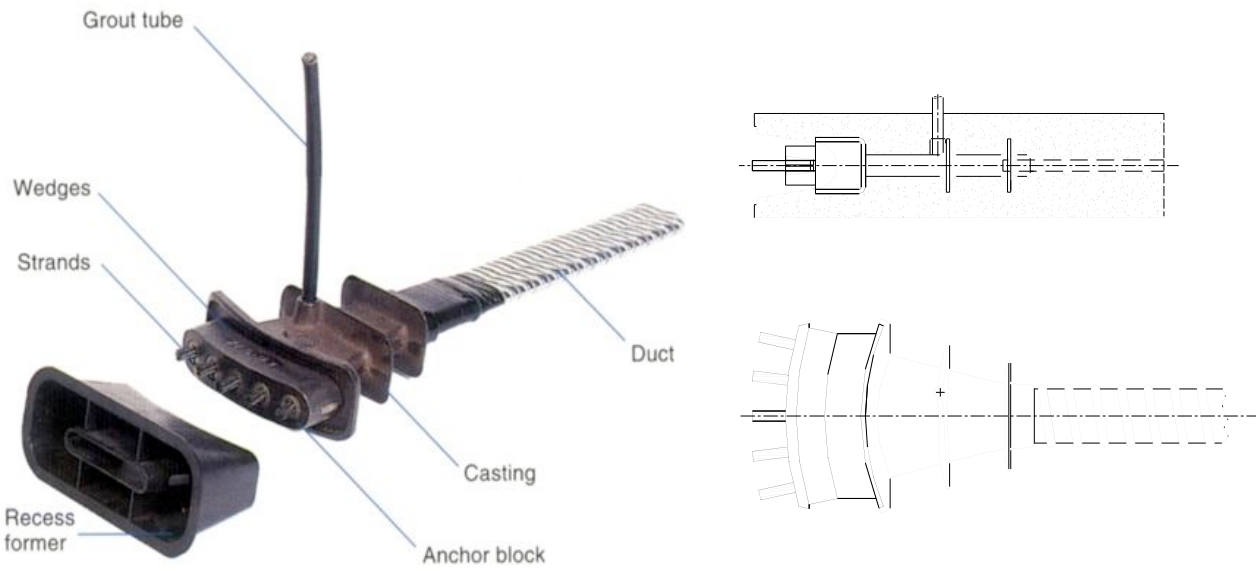

Figure 2.36: Components of a stressing anchorage of group 3

Corrosion protection

The demands for construction quality and durability have been increased leading to the requirement for systems with a higher performance level. Each of the strands is placed within a corrosion-resistant polypropylene duct. Positive duct-to-anchorage connections provide full strand encapsulation, leaving no partial strand lengths exposed to the surrounding concrete.

The strand encapsulation is maintained even at slab construction joints by the incorporation of improved system details.

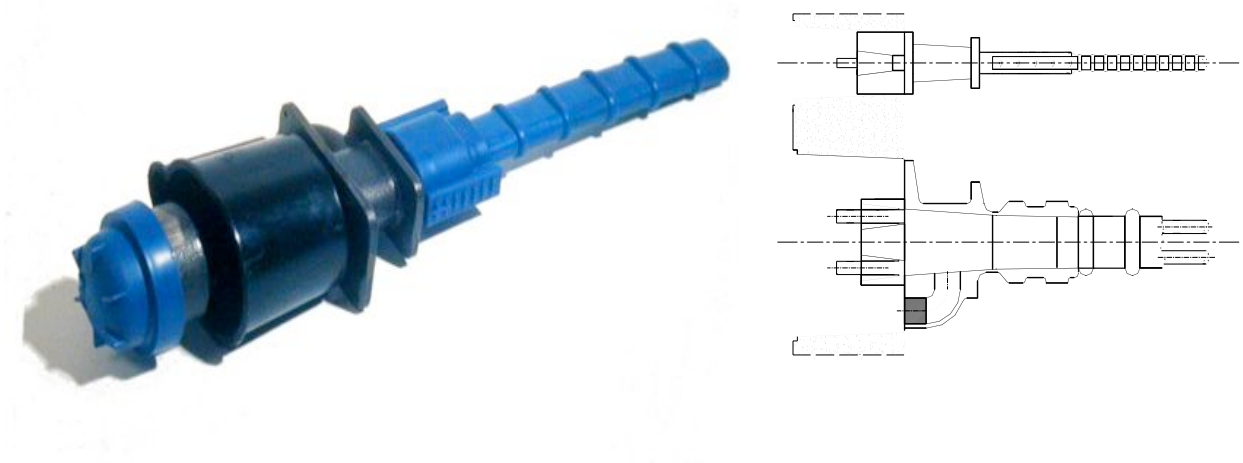

Figure 2.37: Stressing anchorage with enhanced corrosion protection (VSLAB TM)

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C Dead-end anchorage

Where high level of corrosion protection are required and a need for the pre-stressing force to be transferred as near as possible to the end of a structural component, the stressing anchorage can be used as dead end anchorage.

Where the pre-stressing force must only be transferred as near as possible to the end, an anchorage with retainer plates and compression fitting can be used.

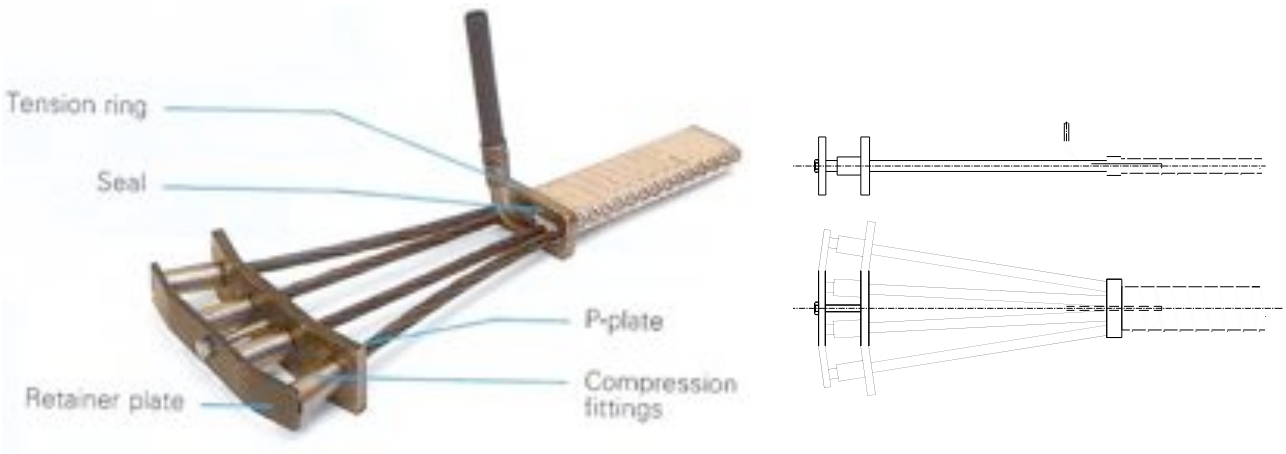

Figure 2.38: Elements for dead end anchorage with bearing plate

In other cases, the pre-stressing force can be transferred by the bonding of the bare strand and partly by direct bearing of the bulb at the end of the strands.

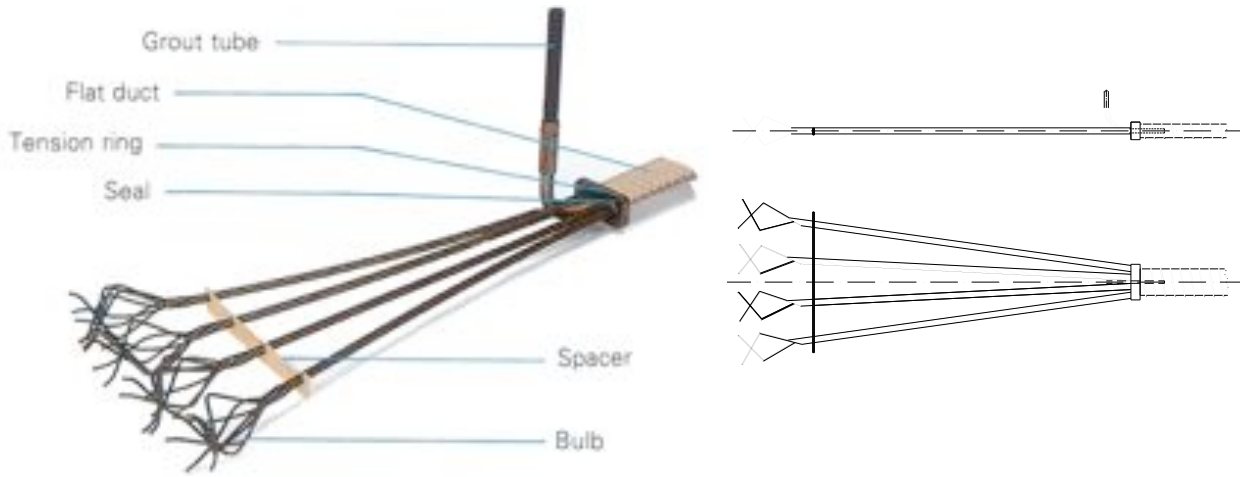

Figure 2.39: Elements for dead end anchorage by bond

A third not so usual dead end anchorage is the loop anchorage where the strands form a loop around a bent plate. The force is transferred through the bonding of the strands and the pressure onto the plate.

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D Couplers

The couplers enable a new cable to be connected onto a previously installed and stressed cable. Different degrees of corrosion protection can be achieved:

– high degree of corrosion protection with fully encapsulated strands

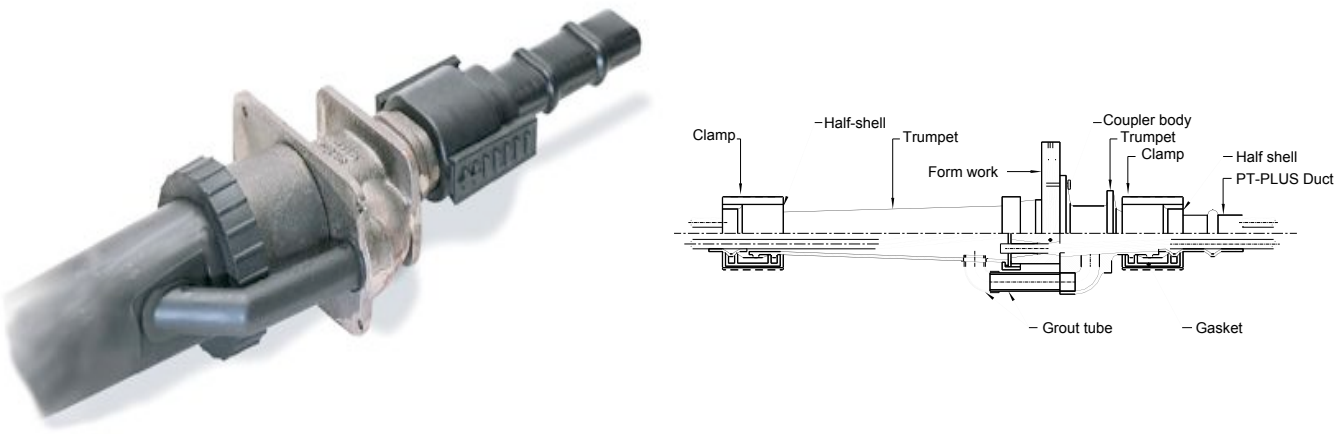

Figure 2.40: Components of a coupler with high degree of corrosion protection. (VSLAB TM)

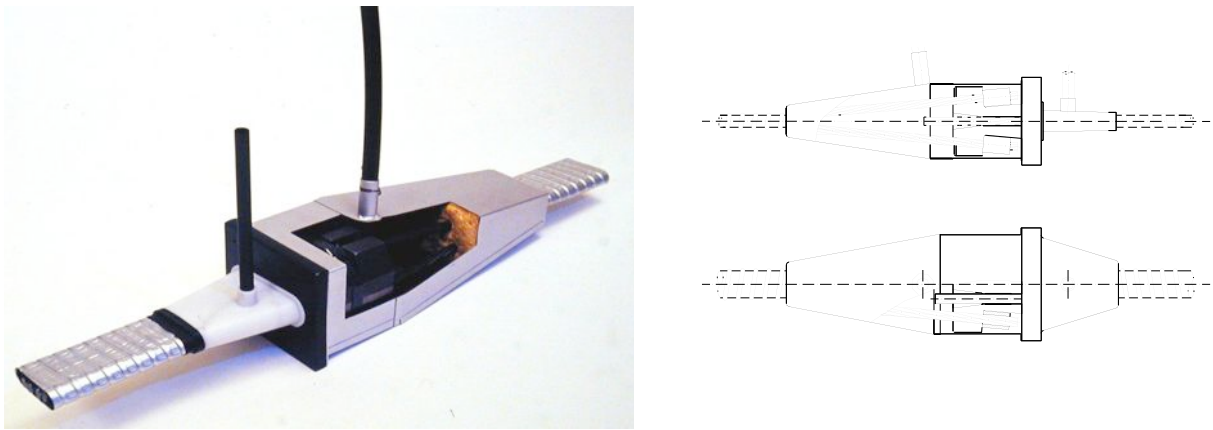

Figure 2.41: Other example of coupler

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– lower degree of corrosion protection where the strands are exposed to the surrounding concrete.

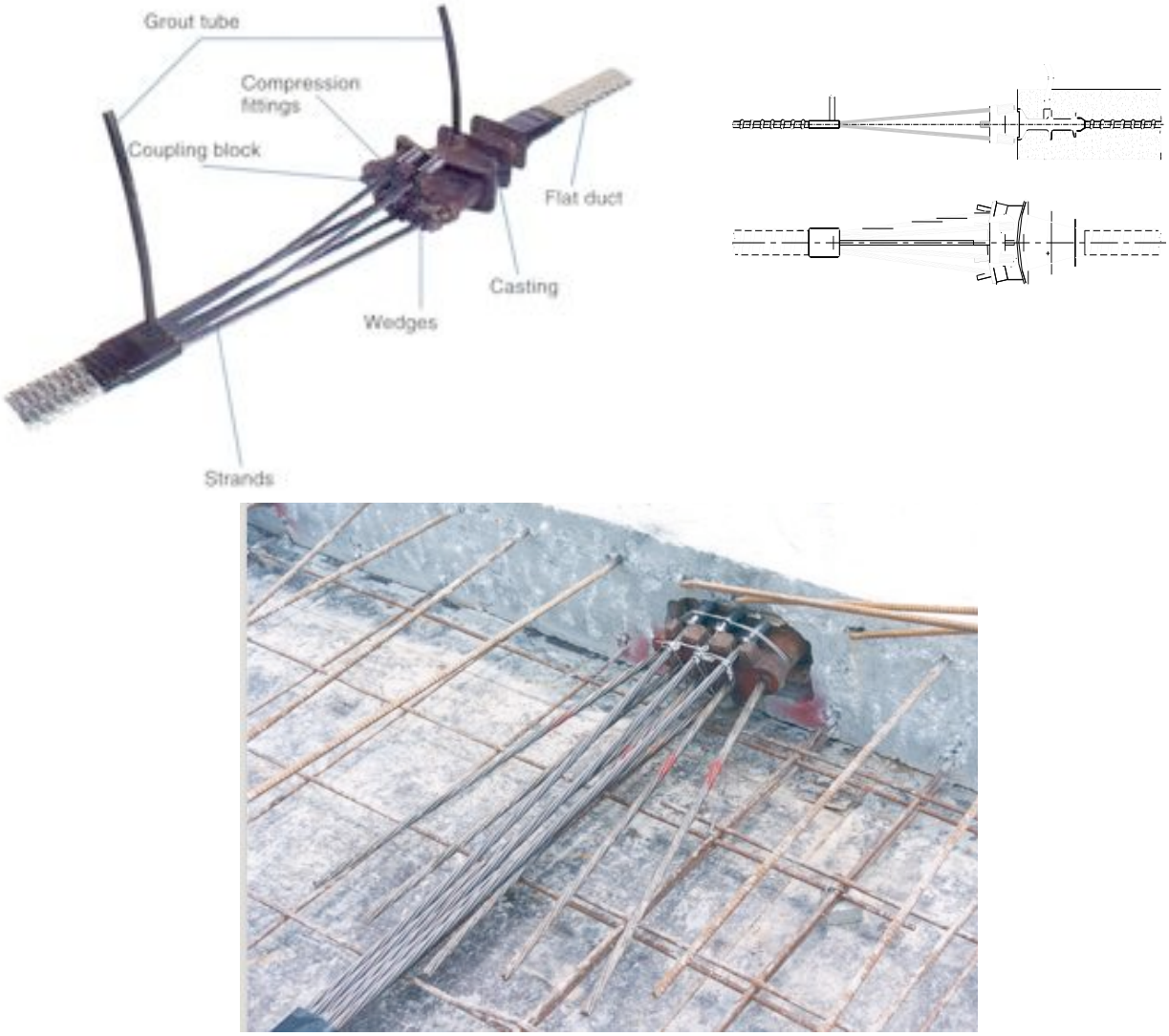

Figure 2.42: Coupler elements with lower degree of corrosion protection

2.4.3 Stressing equipment and clearance

In normal case, the stressing of the strands for post-tensioning in buildings is done with a front-gripping hollow piston jack with a stroke of 200 mm and a weight of approximately 20 kg.

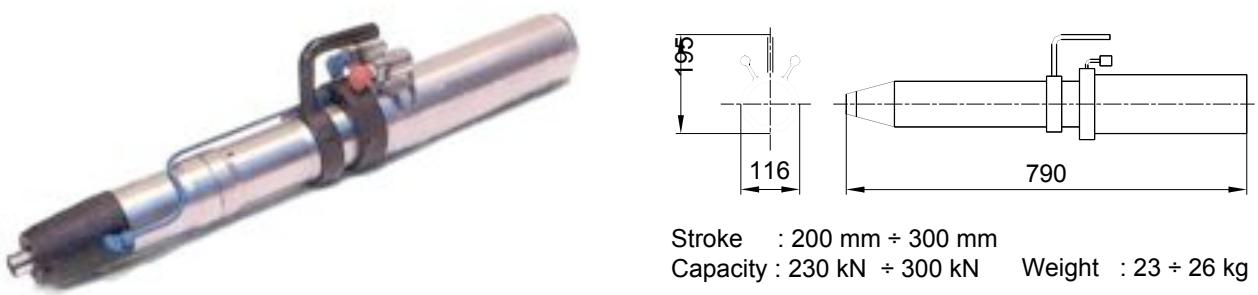

Figure 2.43: View of the front-gripping hollow piston jack and some characteristics

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In special cases such as short clearances (block-outs) or exceptionally for intermediate anchorage with continuous strands, the twin ram jack will be used with a special chair for stressing.

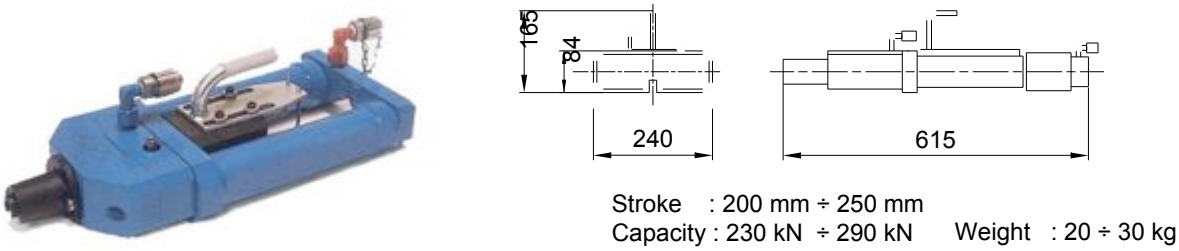


Figure 2.44: View of the twin ram jack and some characteristics

Jacking systems with two to four strands have been developed.

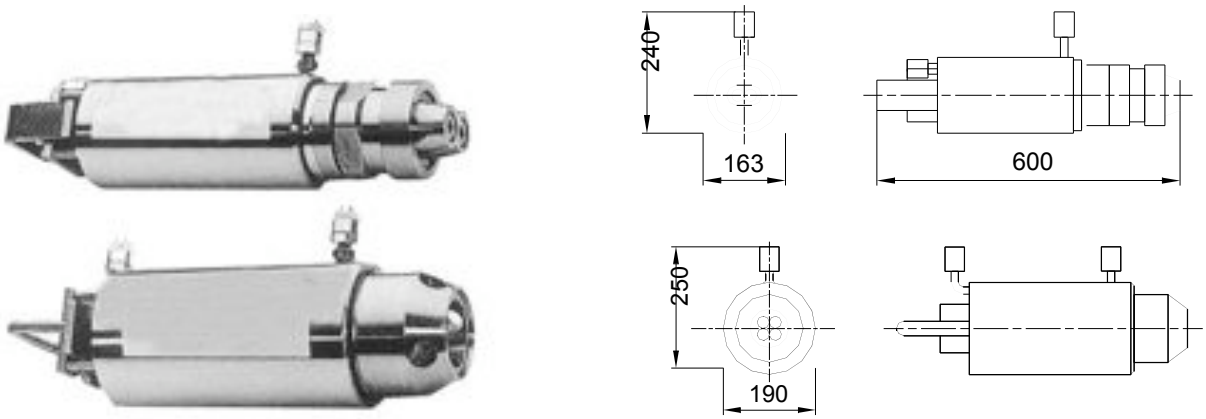
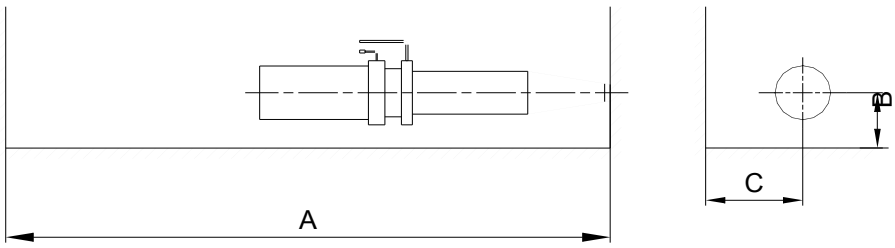


Figure 2.45: Systems with two and four strands

For further information, consult system catalogues. The jack clearance requirement can be assumed as follows:



		Centre hole jack	Twin ram jack
A [mm]		950 - 1100	700 - 1200
B [mm]	2 strand jack	70 - 90	60 - 80
	2 strand jack	110	-
	4 strand jack	130	-
C [mm]	rectangular anchor		
	4 strands	280	300
	5 strands	400	400
	square or circular anchor		
	1 strand	70	-
	2 strands	105	-
	4 strands	115	-

Table 2.1: Clearance for jacks

The dimensions of internal stressing pockets or recess depend on the above values and may vary for particular applications.

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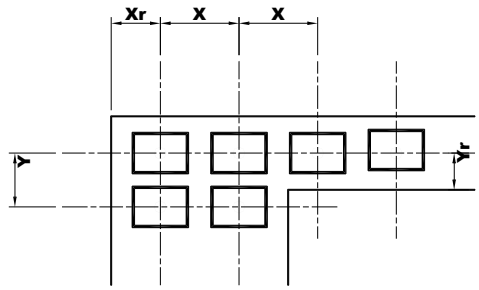
Space requirements

The average distances to concrete edges and adjacent anchorages are given in the following tables as a first approximation. For final value, refer to the agreements.

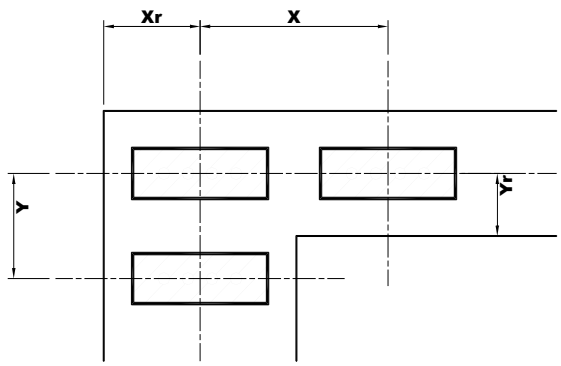
Monostrand post-tensioning



Strand type	X	Xr	Y	Yr	Anchor dim.
	[mm]				
13 mm	150	90	100	70	110x70
15 mm	160	100	110	75	105x75

Table 2.2: Space requirement for monostrand post-tensioning (average values)

Bonded slab post –tensioning



Strand type	X	Xr	Y	Yr	Anchor dim.
	[mm]				
13 mm	340	190	160	100	240x100
15 mm	390	210	200	120	265x130

Table 2.3: Space requirements for bonded slab post-tensioning (average values)

2.4.4 Installation

Unbonded monostrand system

Reeling chair with drive motor for coiling individual cables in the factory and transporting individual cables coils. Transport palette up to 2.5 to bundles of tendons.

Bonded slab post-tensioning system

For short cables, prefabrication in the factory and transport on dispenser 2 m x 5.5 m.

Typical, the ducts are transported to site in bundles or packed loose in special transport frames. Pushing through of individual strands by push-through machine (or by hand for short cables) before concreting.

For pushing through of the strands after concreting, special care must be taken on the flat duct to avoid deformation of the duct.

The support of cables is done either with single supports or with support cages. The distance between tendon supports should be between 0.60 and 1.50 m. The lower limiting value should be used especially at the high points of the tendon (small radius of curvature). The tendon are stabilised horizontally and orthogonal reinforcement of plain round bars (diameter 8 mm, mesh width 1.20 m), fixed directly to the tendon.

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Unbonded monostrand system

Bonded slab post-tensioning system

Figure 2.46: Transport palette loaded with bundles of monostrands and view of the placed ducts before pushing-through of the strands

2.4.5 Fire resistance

The minimum concrete cover maintains normally sufficient protection for the post-tensioning steel [1.3]. The values can vary according to the standards of the various countries.

2.4.6 Specifications

National and international codes provide a substantial support. Gaps of information can occur in some special cases. These can be covered with specifications based on different renowned institutions and technical commissions. The following annex includes an example of specifications.

References

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1.2. CEB-FIP Model Code 1990, Thomas Telford, 1993
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Annex: Specification example

1.0 Prestressing steel

Strands in accordance with BS 5896
Strands 0.6”

Low Relaxation 2.5 % at 70 % GUTS at 20 °C at 1000 hours

Nominal diameter	∅	=	15.2 mm/strands
Nominal area	A_P	=	140.0 mm^2/strands
Tensile Strength	f_{tk}	=	1860 N/mm^2
Yield Strength	f_y	=	1670 N/mm^2
Young’s modulus	E_P	=	195000 N/mm^2
Min. breaking load	P_N	=	260 kN/strand
Duct diameter	∅	=	90 x 20 mm or 70 x 19 mm for slab tendons
Coefficient of friction	μ	=	0.25
Unintentional angular deviation	k	=	0.0012 m^{-1}
Wedge draw-in	w	=	6 mm
Max. Stressing	P_0	=	70 % $f_{tk} * A_P$

2.0 Anchorages

2.1 Bearing plates shall be placed perpendicular to the tendon path and shall be shimmed as necessary.
2.2 Grout fittings shall be standard plastic pipe, black and galvanized steel or flexible plastic tubing at the placer’s option.
2.3 Additional reinforcement steel required for anchorage block - outs and bursting grids shall be grade 410 unless otherwise noted.

3.0 Tendon fabrication

3.1 Tendon shall be fabricated with sufficient length beyond the bearing plate to allow stressing. A minimum length of 1.0 m at both ends is required for multistrand stressing.
3.2 Tendon shall be cut to length at the job site from bulk coils. Excessively damaged duct length shall be removed and replaced completely, not repaired.
3.3 Use of a nylon sling is required to prevent damage to the materials during handling.
3.4 All prestressing coils shall be satisfactorily protected at the job side and when stored off the job site from corrosion and damage. Sufficient protection shall also be provided for exposed in - place prestressing steel to prevent excessive deterioration from corrosion.

4.0 Tendon placement

4.1 Strands, ducts and bearing plates according to the quantity and spacing shown on the placing drawings.
4.2 All vertical profiles shall be measured from the slab soffit to the underside of the tendon duct except at stressing and dead end anchorages where they measured to the centerline of the anchorage.
4.3 The general contractor shall provide sufficient end form bulk-heads for fastening anchors, attach bearing plates, and drill forms for extending strands though as required by contract documents. The general contractor shall provide all necessary shimming required insuring the bearing plates are placed perpendicular to tendon path.
4.4 Placement of mild steel reinforcement shall be co-ordinated with placement of post-tensioning tendon. Proper tendon placement has priority.
4.5 Sufficient support steel (size and spacing as indicated on placement drawings) shall be provided. Maximum spacing of support bars is 1000 mm and minimum ∅ = 16 mm. These bars are used to prevent lateral and vertical movement of the tendon during concrete placement.

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4.6 All support steel and post-tensioning tendons shall be firmly secured in forms to obtain dimension and locations as shown on placing drawings.
4.7 Concrete shall be placed in such a manner as to insure that alignment of post-tensioning tendons remains unchanged. Special provision shall be made to insure proper vibration of concrete around post-tensioning anchorages
4.8 All galvanized duct joints shall be taped mortar tight.

5.0 Stressing

5.1 The stressing operations must be under the immediate control of a person experienced in this type of work. He must maintain a close check and rigid control of all operations. Safety is paramount.
5.2 Adequate scaffolds, platforms and safety devices shall be provided by the general contractor prior to commencing any fabrication, installation or stressing procedures.
5.3 Take safety precautions as necessary. Do not permit anyone to stand behind, above or below rams while stressing.
5.4 All tendons shall be stressed by means of Hydraulic rams, equipped with calibrated hydraulic pressure gauges. A calibration chart shall accompany each ram. (Rams and gauges shall not be interchanged).
5.5 The strands may be fully stressed when concrete test cubes, cured under site conditions have been tested and indicate the concrete has reached a minimum strength of 28 N/mm^2 for multistrand system and prestressing bars or 25 N/mm^2 for slab tendons.
5.6 A partial stressing on each tendon may be required to reduce early cracking due to concrete volume changes. The partial stressing may begin 24 hours but not less than 3 days after casting concrete and the jacking force on each strand shall be proportional to the concrete strength. (Example:- if the required concrete strength = 28 N/mm^2 for a full stress and the partial stress concrete strength = 14 N/mm^2, a partial jacking of half the full jacking force may be applied).
5.7 The post-tensioning operation shall be so conducted that accurate elongation of the tendons can be recorded and compared with elongation shown on approved post-tensioning contractor drawings.
5.8 Records of all gauge pressure and elongation's shall be submitted promptly to the engineer of records for approval if measured elongation's vary more than ±7 % from calculated elongation's. The cause of discrepancy shall be determined and resolved by post-tensioning contractor and the engineer of record.
5.9 Calculated elongations shall be based upon friction calculations, assuming friction coefficient μ = 0.25 and k = 0.0012 m^{-1} for galvanized steel duct.
5.10 Proper alignment of anchor and jacking equipment is mandatory during all stressing operations.
5.11 Tendon stressed from one end only shall be indicated on the placing drawing. Tendon stressed from both ends shall not be stressed from both ends simultaneously. These tendons may have more elongation at one end than at the opposite end. Wedge seating will normally occur in a two end pull at the end opposite to that being stressed which must be accounted for in the total measured elongation.
5.12 Stressing procedure
a) Install anchor wedge. Insert wedges into each wedge cavity (do not remove oily film from wedge).
b) Inspect ram and pump for loose screws, fitting, electrical and hose connections and tighten if necessary. Check jack grippers to insure they are clean and aligned properly.
c) Stress initially to 20% of P_{jack} to remove slack and seat ram.
d) Measure the distance from the face of concrete to a pre-marked datum point on the tendon.
e) Stress to 100% of P_{jack} and measure the distance from the face of concrete to the pre-marked datum point on the tendon. This is the measurable elongation before seating. Record elongation and calculate backwards to 100%.
f) Retract ram and remove from tendon.
g) If a two-end stress is required. Place ram and stress to 20% P_{jack} to seat ram. Mark tendon and stress to 100% of P_{jack}. Record elongation and calculate backwards to 100%.
h) If a partial stress is required, the procedure shall be modified accordingly.
i) Promptly submit stressing records to engineer of record. Upon approval of the elongations, stressing tails may be removed by approved means within 20 to 30 mm of face of anchor head.
j) Install grout cap or fill recess to prepare for grouting.

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6.0 Sealing anchor recesses

6.1 Coat anchorage recesses with approved bonding agent and drypack or fill with concrete flush with face of concrete, non-shrink, non-corrosive, non-metallic grout or concrete shall be used for this purpose. This procedure shall be the responsibility of the general contractor.
6.2 Grout tube may be extended through the drypack material or tendon may be grouted prior to sealing anchorage recesses if a grout cap is used.

7.0 Grouting

7.1 A trial grout mix shall be batched in advance of grouting operations to verify the compatibility of the cement, water and admixtures.
7.2 The grouting operation may proceed when the air temperature and surrounding concrete temperature are a minimum of +2°C. The grout temperature shall be maintained between +5°C and 30°C before entering the duct. The grout temperature shall be maintained at a minimum of +2°C after entering the duct until a minimum compressive strength of 10 N/mm^2 is attained as determined by 5 cm cube tests.
7.3 Procedure
a) Mixing order: Water, Cement, admixture
b) Normal mixing time to achieve a uniform and thoroughly blended grout is approximately 2-3 min..
c) Grout temperature at time of pumping shall be between +5°C and 30°C. Grout shall be continuously agitated until it is pumped.
d) Pumping - grout is continuously pumped thought a tendon. When grout is continuously ejected from the outlet with no visible slugs of air or water, close the inlet valve prior to reliving the pressure at the pump.
e) Normal pumping range shall be 0.7 - 1.0 N/mm2.

8.0 Miscellaneous

8.1 All equipment and procedure used for handling and placing tendons shall not damage or cause deterioration to the prestressing steel, duct or components.
8.2 All concrete inserts must be cast-in-place. If additional inserts are required after the concrete is cast, the contractor must locate tendons at the surface before driving fasteners. Written approval must be obtained from the engineer of record and post-tensioning contractor prior to cut the concrete surface.
8.3 Burning and welding in the vicinity of the tendon is discouraged, (except for removal of stressing tails). Care must be taken that tendons are not subjected to high temperatures, welding sparks or ground.
8.4 The contractor shall check all plans and details shown on these drawings for conformance with the structural drawings. Any discrepancies shall be reported to the engineer of record for clarification or adjustment prior to proceeding with the work.

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3 Post-tensioned floors

3.1 Conceptual design

3.1.1 General

The choice between prestressed and non-prestressed construction is governed by span lengths, loads and structural system. Prestressing allows the use of reduced depth, longer span and/or higher load. Sometimes, e.g. in heavily loaded beams, the main advantage of using prestress may be a simplification of reinforcement detailing and execution.

A simple guide for slab thickness and span for different types of slabs is given in table 3.1, based on [3.1].

Type of slab	Imposed load, kPa	Depth mm	Maximum recommended span, m (0 2 4 6 8 10 12 14)
Simply supported	1,75	200	
		300	
	4,0	200	
		300	
Continuous, interior span	1,75	200	
		300	
	4,0	200	
		300	
Simply supported, span ratio ≈ 1:1	1,75	200	
		300	
	4,0	200	
		300	
Continuous, interior span, span ratio ≈ 1:1	1,75	175	
		250	
	4,0	175	
		250	
Span ratio 1:1	1,75	200	
		300	
	4,0	200	
		300	
Span ratio 1:1,5 (recommended *longer* span)	1,75	200	
		300	
	4,0	200	
		300	

Table 3.1: Examples of slab thickness and indication of maximum span for solid slabs with ordinary ( ) and prestressed ( ) reinforcement. Apart from the stated imposed load, there is also self-weight plus 0,5 kPa for partitions. Based on [3.1]

"Ordinary slabs" are supported on walls or stiff beams, see 3.1.2.1. "Flat slabs" are supported on columns, see 3.1.2.2. Slabs supported on flexible beams will behave between ordinary and flat slabs, depending on the stiffness of beams. The span lengths in table 3.1 should only be seen as indications, not as definite limits. For ordinary slabs the criterion has been that the slab should remain uncracked in SLS, in order to limit deflections. Without prestress,

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cracking will lead to a drastic increase of the deflection. With prestress, on the other hand, cracking will lead to a more gradual increase of deflections. Therefore, larger spans than those indicated in the table are often possible for prestressed slabs. For flat slabs, the criterion has instead been a deflection not greater than $L_D/400$, where L_D is the diagonal span length.

The values in table 3.1 are primarily applicable to solid slabs, i.e. slabs with uniform depth; see 3.1.2. For slabs with variable depth, larger spans are possible for a given total depth due to the reduced self-weight, but generally also the total depths are larger in this case; see 3.1.3.

3.1.2 Solid slabs

3.1.2.1 Ordinary slabs

Slabs supported on stiff beams or walls are here called ordinary slabs. They can be one-way or two-way spanning. The need for prestressing is greater for one-way slabs, but it is less than for flat slabs, see table 3.1. In ordinary slabs, compared to flat slabs, deflections are smaller and the punching problem disappears; thereby two important reasons for prestressing may also disappear.

In a slab supported on and connected to walls, part of the prestressing forces will go into the walls. The transverse effect of prestress will always act fully on the slab, but part of the axial effect may be lost. See 2.3.6.

The restraint from walls can be partly or fully eliminated if the supports are detailed to allow the slab to move in the direction of the support. The transfer of forces between slab and walls can then be limited to the direction perpendicular to the wall, as needed for horizontal stability and prevention of progressive collapse. There is also a link here to the problems of restraint and movements; see 2.3.6.

3.1.2.2 Flat slabs

The term “flat slab” is commonly used for slabs supported on columns. Three typical tendon layouts for a regular flat slab are shown in figure 3.1. Some concentration of tendons along the columns strips is recommended, at least in one direction, considering the concentration of forces and bending moments that takes place over the columns. For layout a), FIP [3.2] recommends at least 50% of the tendons to be concentrated on the column strips.

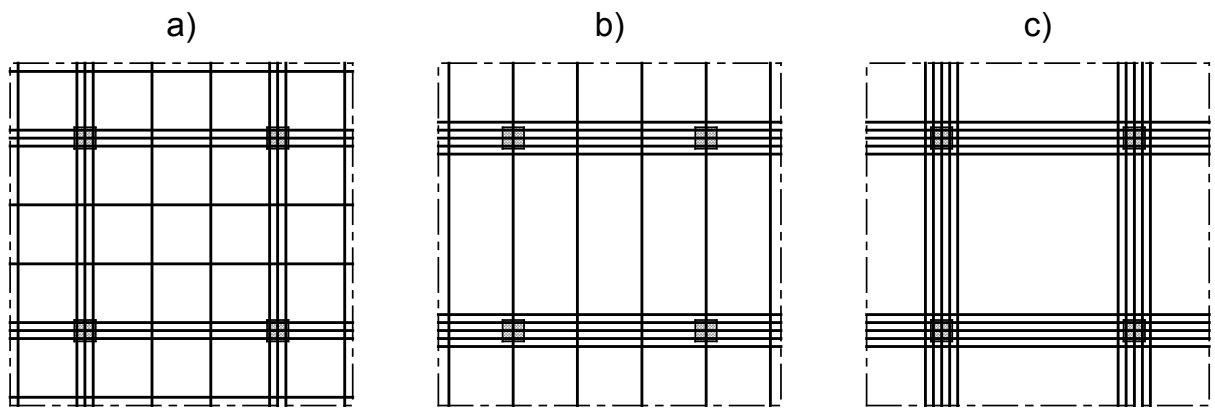

Figure 3.1: Typical tendon layouts in regular flat slab

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Figure 3.2 shows schematically the transverse effect of the span prestress cables in tendon layout a). The span loads are transferred to the columns strips by distributed tendons, and the concentrated tendons in the columns strips transfer the loads to the columns.

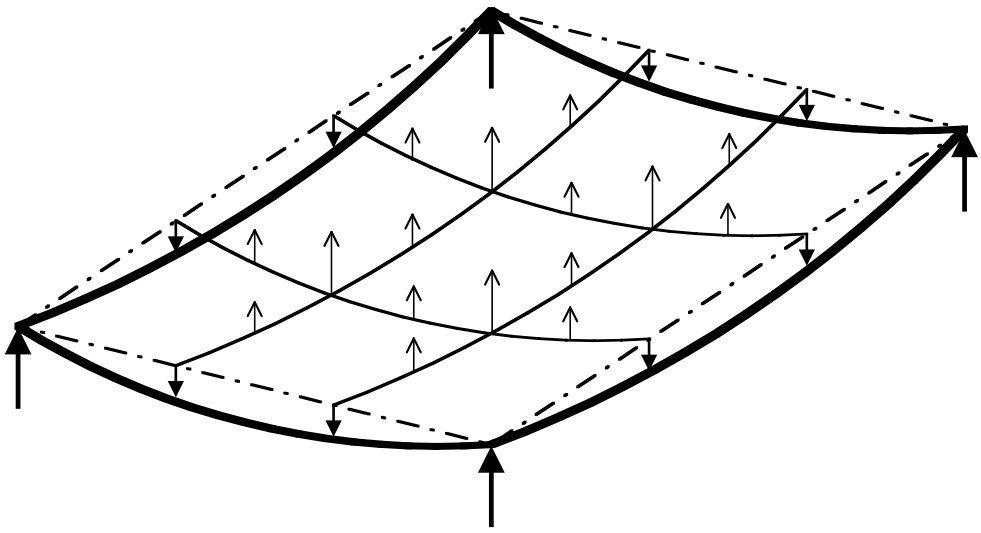

Figure 3.2: Illustration of transverse effect of prestress for the span cables of tendon layout a) in figure 3.1

Tendon layout b) produces a similar effect, but the span load is transferred by distributed tendons in only one direction and then to the supports by concentrated tendons in the perpendicular direction.

In layout c), the transverse effect goes directly to the supports. For a given amount of tendons, it is generally the most efficient layout with regard to both punching and deflections. This case can be seen as an ordinary slab supported on beams = the column strips. The "ordinary slab" may require more ordinary reinforcement than in the other alternatives, but the favourable axial effect of prestress can be fully utilized in the design of this reinforcement, at least for interior spans.

The axial effect depends mainly on the total amount of tendons and not so much on their distribution, except near the anchorages; the compressive stress can be assumed to be uniformly distributed already at a distance about 3/4 of the largest tendon spacing.

With regard to execution, concentration of tendons in the column strips in at least one direction is more practical, as in b) or c). Alternative a) is more complicated since distributed tendons in two directions may form an intricate wickerwork, at least if their number is large.

The tendon layout has implications for the possibility of making openings, either planned from the beginning, or unforeseen future ones. From this point of view c) is normally the less restrictive alternative, but if the band of tendons extends outside the column area there will be restrictions for openings near the column. From this point of view a) may sometimes be preferable, since it is often "popular" to place openings close to columns.

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3.1.3 Slabs with variable depth

By increasing the slab depth along certain lines, the efficiency of prestressing tendons can be enhanced. Beams can be located in the column strips in one or two directions. Ribs can also be distributed in one or two directions.

With increased depth only around the columns, so called "drop panels", the punching resistance is increased (to some extent, depending on the width of the drop panels, also the overall flexural capacity and stiffness is increased). Post-tensioning generally reduces or eliminates the need for drop panels, and it can often be a more attractive alternative to solve the punching problem. However, there may be extreme cases where both are used, therefore it is mentioned here for the sake of completeness.

Figure 3.3 shows beams and drop panels. Case a) with beams in one direction is suitable for tendon layout b) in figure 3.1 Analogously, case b) with beams in two directions is suitable for tendon layouts a) or c).

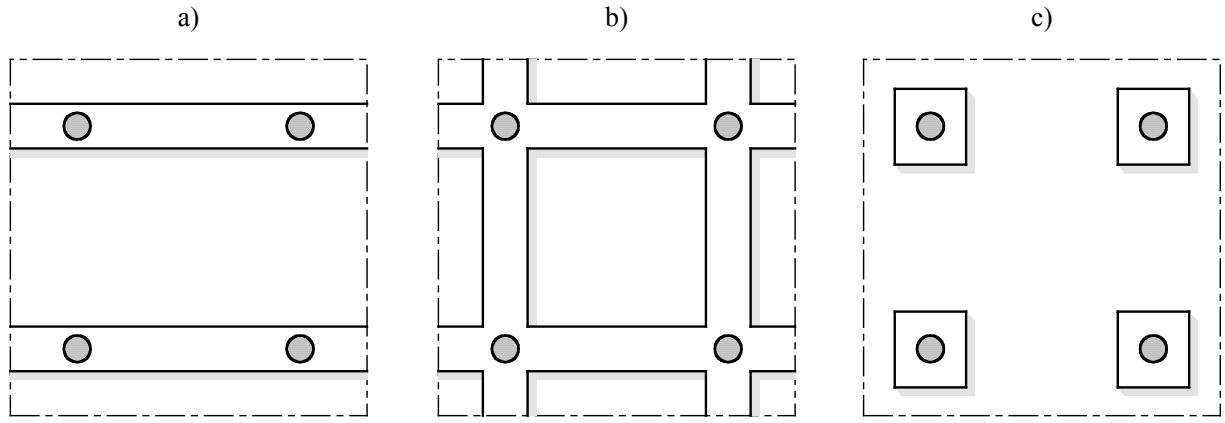

Figure 3.3: Examples of slabs with beams (a, b) and drop panels (c)

A ribbed floor can be seen either as a flat slab with increased depth along certain lines, or as one with a depth equal to that of the ribs, and with recesses to reduce weight. There can be beams in one direction and ribs in the other direction (a), beams and ribs in both directions (b) or ribs everywhere except in column areas (c); see figure 3.4. Ribbed floors can be prestressed in one or two directions, or not at all; the choice is governed by the same criteria as for massive slabs, i.e. spans, loads and economy. Table 3.1 may give some guidance, but usually both the total depth and the span will be beyond the values given in table 3.1.a. The criteria for layout of ribs and beams are similar to those for the layout of tendons in flat slabs; see paragraph 3.1.2.

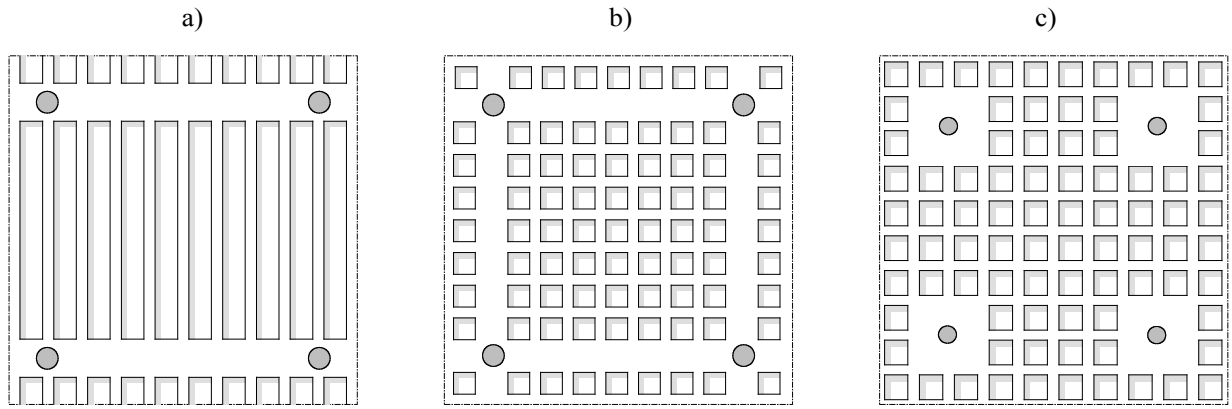

Figure 3.4: Examples of ribbed floors

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3.2 FIP Design of post-tensioned slabs and foundations, fib, Thomas Telford, May 1998.

3.2 Applications

3.2.1 Solid slabs with bonded tendons "Fuenlabrada Shopping Center"

- Designer: Fhecor Consulting Engineers, Spain, 1994
- Contractor: Ferrovial / Agroman, Spain, 1996

Figure 3.5: General view

The *Avenida de las Provincias* shopping centre is located at Fuenlabrada, close to Madrid. In plan, the building is shaped as a semi trapeze with mean lengths of 107.00 × 78.00 m. The building has five levels, two of which are located partially underground. The building is conceived as multifunctional. The first level corresponds to parking areas, as well as areas for loading and unloading of goods. The second and third levels correspond to shopping areas and the two upper levels are meant as sports and leisure areas. The total surface of the building is 35,000 m^2.

The structure of the floors is a post-tensioned concrete slab. The prestressing is bonded and uses flat ducts, in order to maximize the eccentricity of the prestressing. The slabs are supported on a mesh of columns with a distance of 12 × 12 m. The depth of the slabs is 0.32 m. In the area near the columns an abacus shaped as a pyramid is designed, thereby increasing the depth by an additional 0.23 m.

The prestressing tendons are formed by 4 strands of 0.6", grouped together in a single flat duct. The width of the duct is 80mm and its depth, only 25 mm.

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In plan the prestressing is distributed according to the geometric conditions of each level. In the lower levels, the prestressing is concentrated in the column strips. In the ceiling floor, the distribution is modified because the continuity disappears at this level in one direction, the structures being reduced to a single span of 12.00 m. For this geometry, the distribution of prestressing along the direction which is continuous remains the same as on the other levels (prestressing concentrated in the column strips), whereas in the perpendicular direction the prestressing cables are distributed uniformly.

In spite of its dimensions, the building has no expansion joints. The reason for this is that, in this type of building, it is always hard to solve the functional problems (such as double columns), structural problems (such as the length of the end span if the mesh of columns is left unaltered) as well as service problems (durability problems and possible leaks) which are caused by expansion joints. The absence of expansion joints brings therefore many benefits, even though it requires a slightly more complicated analysis to be carried out and additional minimum reinforcement rules to be followed.

The construction of the slabs in each level was carried out by building approximately 1300 m^2 at a time. This implies a division of each level in approximately 6 parts, except for the ceiling floor which could be divided into 3 parts due to its smaller surface. For each sector a one week cycle was planned. The structure was finally built in 3 months. The use of up to 3 working teams, allowed the construction of to 3000 m^2 per week.

Figure 3.6: View of an abacus

Figure 3.7: View of reinforcement and of the prestressing ducts previous to the concreting of level 1

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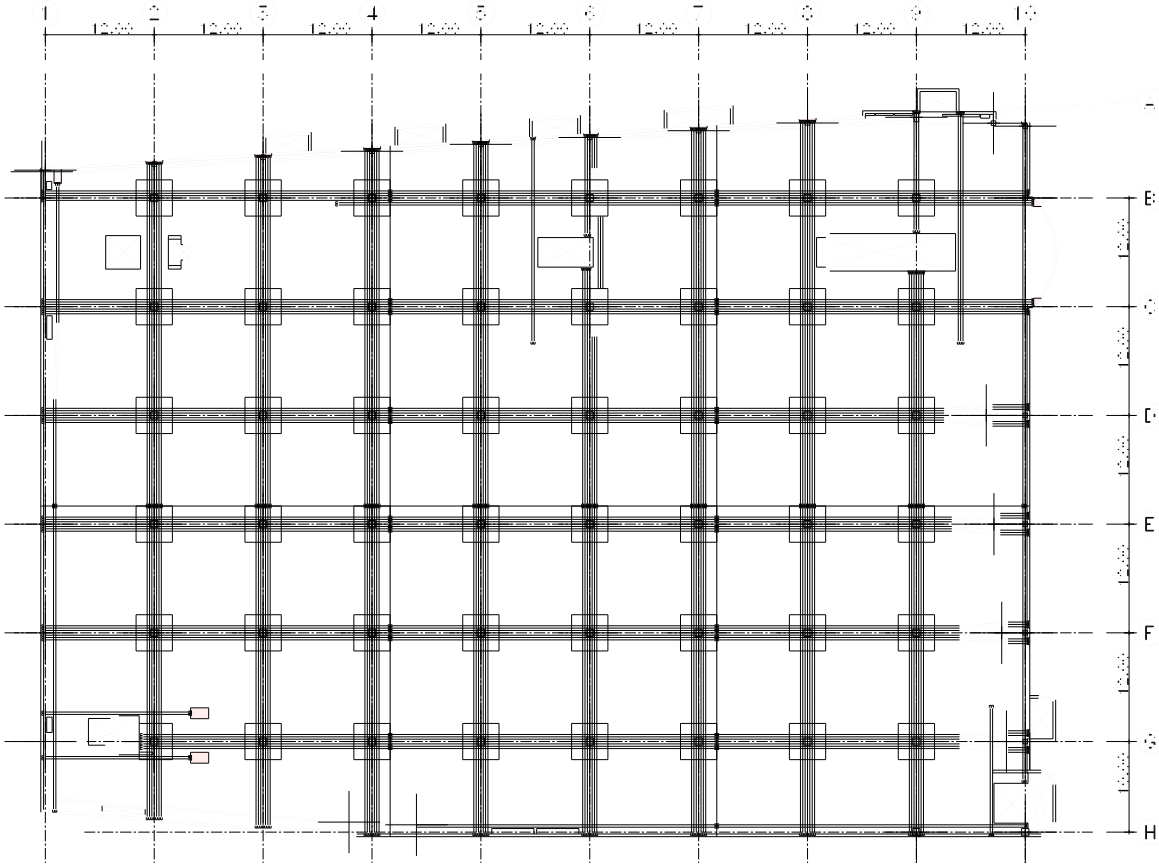

Figure 3.8: Distribution of prestressing tendons in plan in level 1

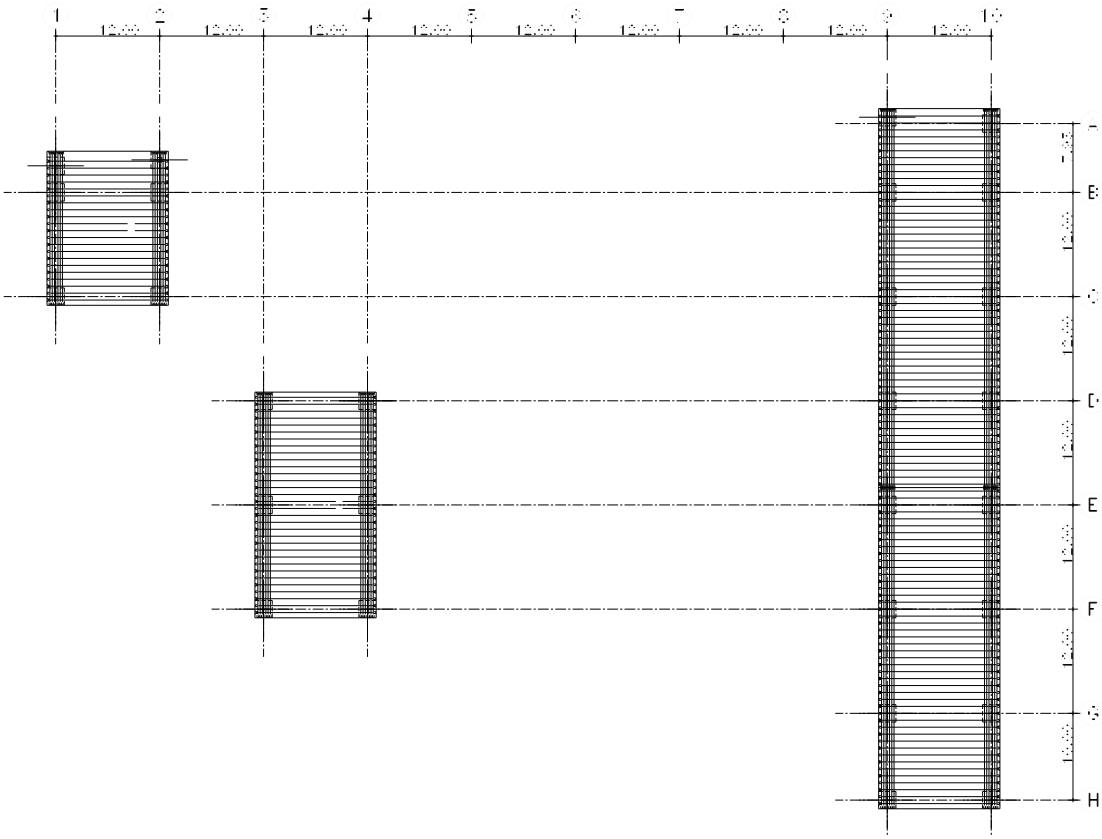

Figure 3.9: Distribution in plan of prestressing cables in the ceiling floor

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3.2.2 Waffle slab with unbond monostrands "Alicante OAMI Headquarters"

- Designer: Fhecor Consulting Engineers, Spain, 1997
- Contractor: Dragados Obras e Proyectos - ECISA, Spain, 1998

Figure 3.10: General view

This building, which is the definitive headquarters of the Office for Harmonization in the Internal Market, is located outside Alicante, on the site known as *Agua Amarga*, next to the sea shore.

The building has three underground floors, used for parking, which have a surface of 16000 m^2. Above the ground, the building is formed by a main module with 6 levels and two annex modules of only two stories, destined as space for offices and other activities. These take up 15500 m^2.

In this building, two different systems of prestressing have been used: unbonded single strand cables and bonded multi-strand cables, placed in circular ducts.

In plan, and above the ground, the main building is shaped as a trapeze with mean dimensions of 155.0 × 16.0 m^2. These floors are supported by two rows of columns located near the edges of the building parallel to the longer dimension. The mesh obtained by this support system is of 15.0 × 7.2 m^2. The floors are also supported by the concrete walls located at the shorter ends as well as by the walls corresponding to the vertical communication nuclei.

In order to maximize the transparency of the building, the structural solution is a waffle slab with a depth of 40 + 5 cm, prestressed with non bonded prestressing placed in the direction of the longer span and with conventional reinforcement in the perpendicular direction. The resulting slenderness is h/L = 1/34.

Due to the location of the columns, the slab bends mainly in only one direction. The solution adopted allows for sufficient depth in the transverse direction (reinforced concrete), and at the same time, minimizes self weight. The prestressing makes up for the greater bending moments which appear in the longer direction.

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The webs of the floor are separated 0.80 m. In each web two single strand cables of 0.6” are placed. The vertical layout of the cables is parabolic.

For the annex modules, with two levels over the ground and a square plan with a 26.00 m side, bonded prestressing, inside a circular duct was used. The buildings have a very transparent structure, the floor being supported on 8 columns located at the building perimeter.

The structural solution is a mesh of beams with a depth of 1.00 m and a width of 0.6 m. The main beams are supported on the columns and are prestressed. The other beams are located on the perimeter and are made of reinforced concrete. On these beams, a reinforced concrete slab of 0.30 m of depth is cast. The beams form a mesh with four important areas which are supported as cantilevers. The layout of the prestressing in the beam is parabolic. Each beam is prestressed with two cables of 22 strands of 0.6”.

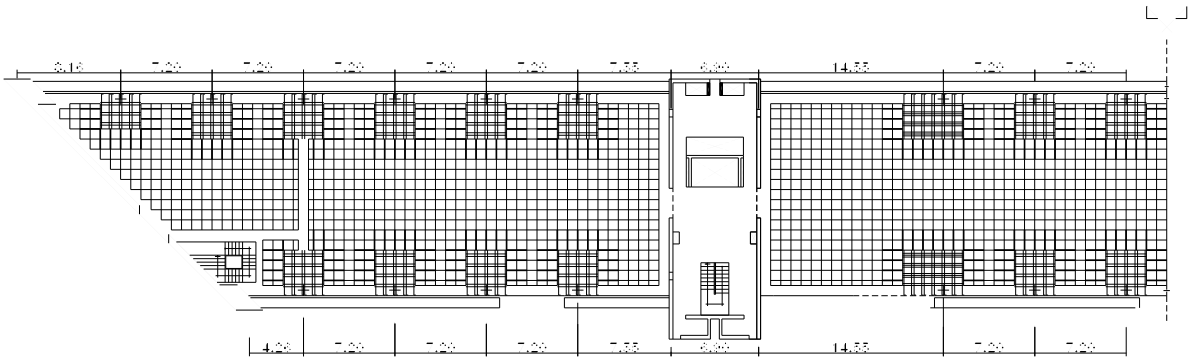

Figure 3.11: Typical plan layout of the main building, above the ground. Structural solution

Figure 3.12: View of the waffle slab with single strand cables

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Figure 3.13: General view of the building during construction

Figure 3.14: General view of one of the levels of the annex buildings during construction

3.2.3 Parking deck one-way banded solution "GAD Munsten"

- Designer: DOMOSTATIK AG, Switzerland

The newly built multi-storey car park serves the co-workers of a big computer centre. Approximately 800 parking spaces were accommodated on the seven floors with a total area of about 18,000 m^2.

The overall building dimensions are 120 m length and 33.80 m width. A parking floor consists of a traffic lane of width 6.0 m and on both sides parking spaces of length 5.0 m. The resulting width of 16.0 m is spanned without intermediate columns. Two parking floors are

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arranged beside each other and staggered in height by a half floor height. The staggered floors are connected by separate up and down ramps. In the middle of the building the ramps are arranged on the axis B, and overcome with a corresponding slope the half-height of the floor of 1.40 m.

A solution involving a slab with prefabricated downstand beams spaced along the axis at 5.0 m intervals was planned. The intermediate decks were originally to comprise prefabricated slab elements and a corresponding concrete top course. The task was to design a slender deck structure, which in execution demanded a minimum of effort and at the same time with a crack-free deck slab providing a durable structure.

The variant actually carried out also consists of a mixed constructional form with prestressed "*in situ*" concrete downstand beams spaced at 7.50 m and of an intermediate deck consisting of prefabricated slab elements. The "*in situ*" concrete downstand beams, thanks to the prestressing, could be made very slender (l/h = 28).

As static system a frame structure was chosen. The dimensions for the horizontal beams and the columns (b/h = 1.35/0.60 m) are the same. The downstand beams are fixed to the columns. Due to the prestressing of the beams a redistribution of the sectional forces of the beams to the columns is constrained. Since the columns are adequately overstressed due to the normal force, they are able to withstand the high bending moments.

The columns will be built beforehand in three stages. The anchorage and starter bars will be already installed in the column elements. The downstand beams are enclosed in formwork panels of large area. Because of the generous width of the downstand beam the stirrup reinforcement and the few longitudinal bars as well as the prestressing cables consisting of bundles, each with 4 single strands, could be inserted without any problems. The side formwork for the downstand beams also serve during the construction phase as supports for the prestressed slab elements. These were supplied with a uniform length (l = 7.50 - 1.35 = 6.15 m) and arranged between the formwork for the downstand beams. The prefabricated slab elements were augmented by longitudinal prestressing and the corresponding top reinforcement. The concreting work for the downstand beams and the top concrete course was carried out simultaneously.

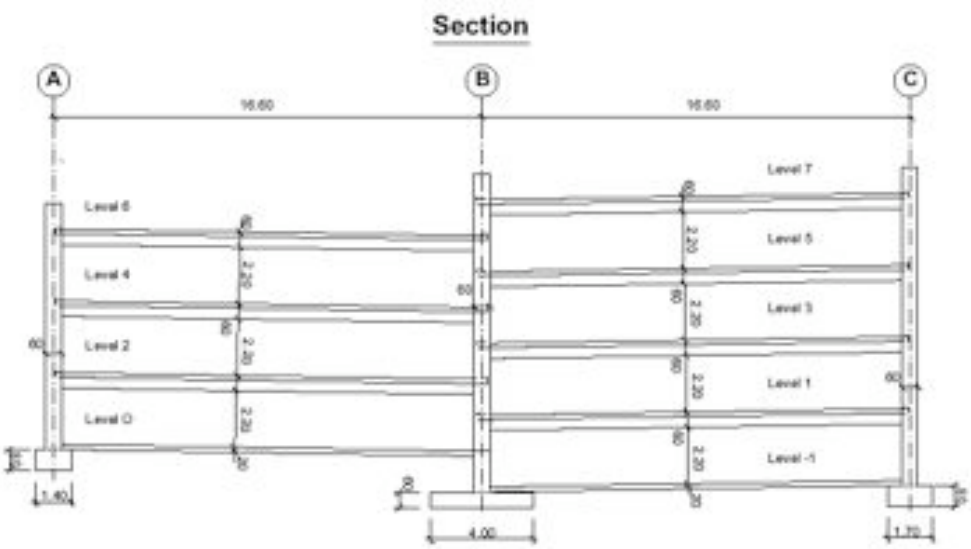

Figure 3.15: Building cross section

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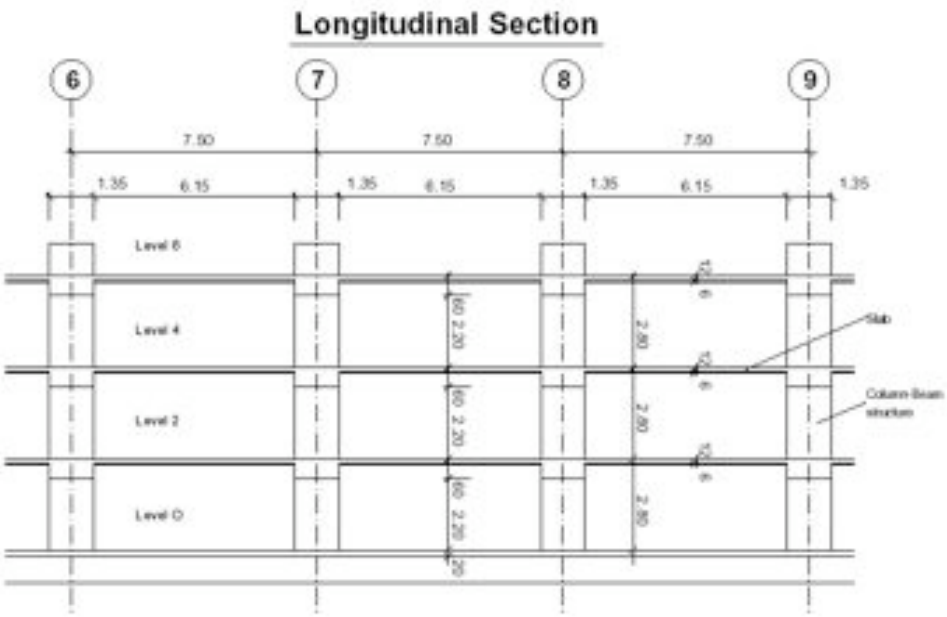

Figure 3.16: Construction in the longitudinal direction

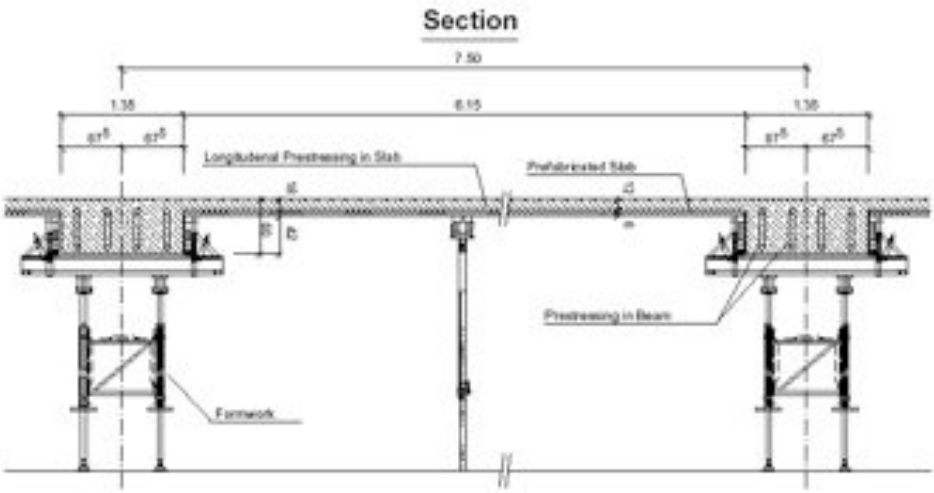

Figure 3.17: Construction method with in situ concrete downstand beams and prefabricated element slabs

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Figure 3.18: Downstand beams in the formwork

Figure 3.19: Formwork the the downstand beam serves simultaneously as support for the element floors

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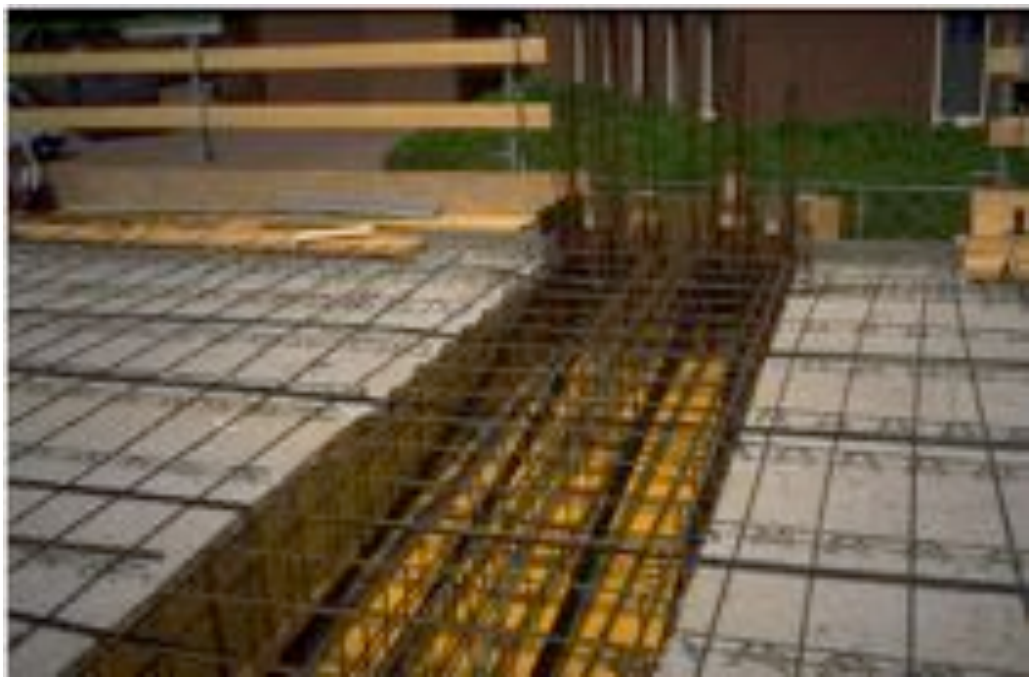

Figure 3.20: Reinforcement arrangement (prestressing and untensioned reinforcement) for the downstand beam and deck

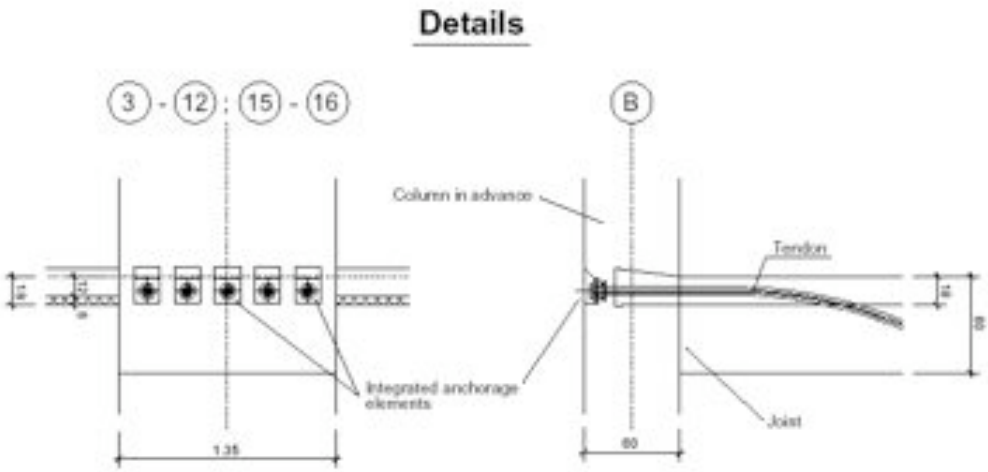

Figure 3.21: Starter bars for the horizontal beams to the columns

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Figure 3.22: Finished structure – underside view of floor slab

Figure 3.23: Finished structure – view of façade

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3.2.4 Particular applications

3.2.4.1 Solid slab "Funchal Crown Plaza Hotel"

- Designer: JSJ Consult, Portugal, 1998
- Contractor: Edifer Construções, SA, Portugal, 1999

The Crown Hotel in Funchal shown in Figure3.24 picture is a two structure building with 6 floors over the main entrance level and three below. The guest rooms, all facing the seashore on a regular basis, are about 3.5 m wide which permitted a basic 7 meters span structural layout with a 0.25 m thick reinforced concrete slab. This general solution was very convenient as it permitted an easy interaction with the installations of all the hotel mechanical equipment.

For functional and architectural reasons, in certain areas of the hotel spans from 10 to 12 meters were needed. In these cases it was thought better to maintain the general structural solution of a 0.25 m thick slab. In these circumstances, and to obtain the same level of characteristics for serviceability behavior, the slabs were prestressed in these regions.

In the figures presented, two of these cases are shown, the first one (Figure 3.25), as part of a ceiling on the 1st floor, and the 2nd one on the interior of the building over the hotel's deliveries area. The plan layout of the cables, particularly on the second case, in not a traditional one, but was found to be the most efficient taking into account the columns alignment and the existence of a beam along the lateral façade. The design criterion was to obtain a maximum deformation smaller or equivalent to the general reinforced concrete solution with approximately 7 meter spans.

Figure 3.24: General view of the Madeira Crown Plaza Hotel

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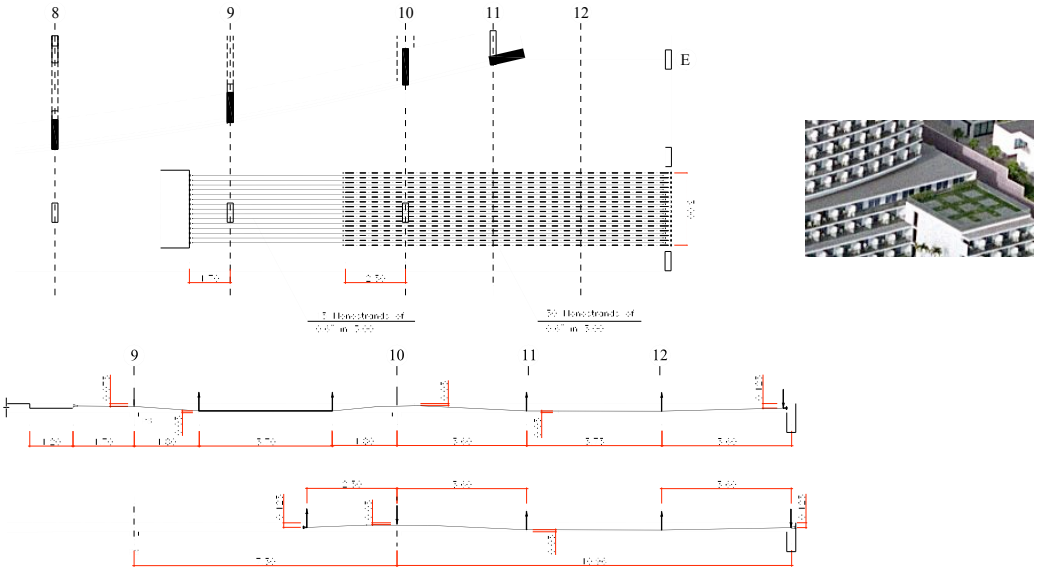

Figure 3.25: General cable layout over a 10.5m span for a 0.25m thick slab

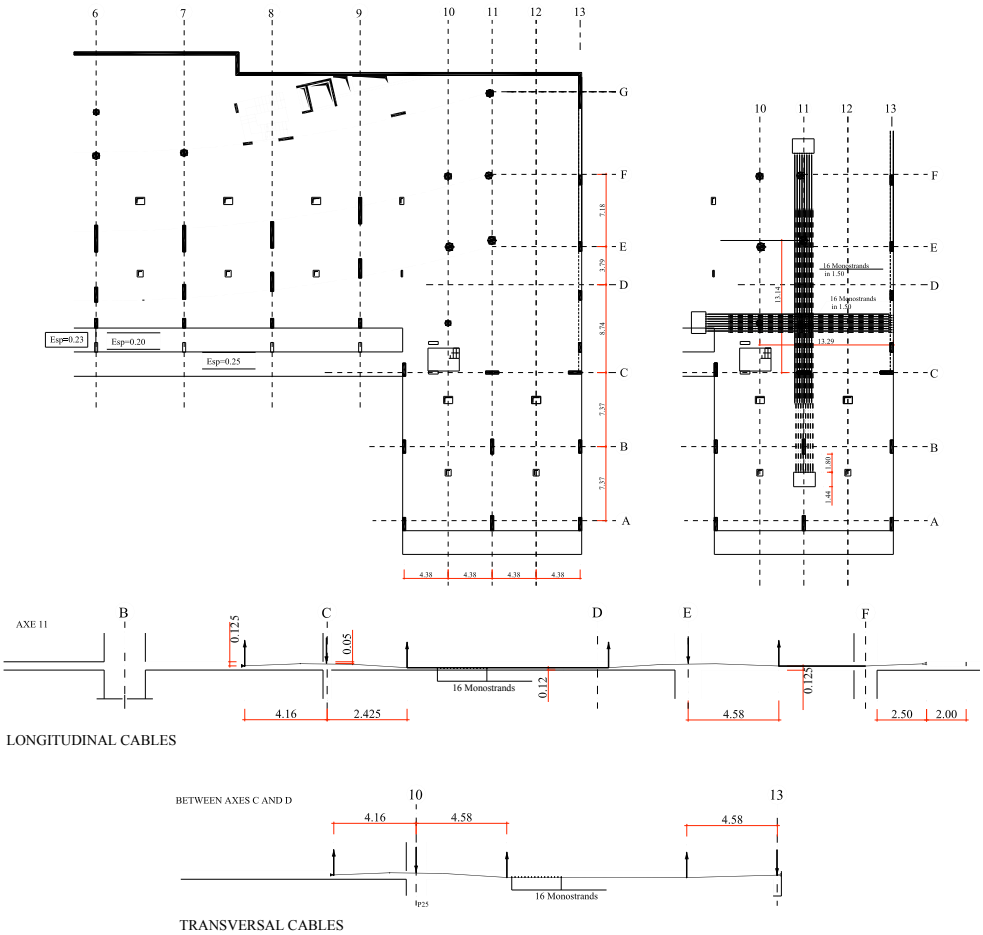

Figure 3.26: Particular prestress layout over the deliveries area in order to obtain bigger spans maintaining however the slab thickness

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3.2.4.2 One-way slab in bending and torsion "Oeiras House"

- Designer: JSJ Consult, Portugal, 2000
- Contractor: Teixeira Duarte SA, Portugal, 2001

Oeiras house is a family residence recently built at Oeiras, in the region of Lisbon. It is a typical situation where architectural concepts lead frequently to unusual and sometimes complex local structural solutions.

In the case hereby illustrated, the roof slab extends by a 2.60 m concrete corbel all around the house contour. In general the corbel is continuous to the interior, except at the south entrance where it is supported on a concrete one-way solid slab, 0.40 m thick, free spanning a distance of about 13.00 m. The solid band, 1.0 m width, has to equilibrate vertical loads as well as bending effects induced by the corbel.

Maximum elastic displacements of about 50 mm, further increased by cracking and time effects, make a traditional reinforced concrete solution impossible. Post-tensioning was used to equilibrate not only bending, but, due to the adopted cable layout, as well torsion effects in the slab. As illustrated in Figure 3.31, prestressing was designed in order to approximately balance deflections due to permanent loads, allowing concurrently a proper control of concrete stresses.

In the present case unbonded monostrands were adopted. In fact, the required prestressing forces were relatively unimportant and the small size of the system elements, strands and anchorages, fit very well with geometrical and dimensional constraints of the situation.

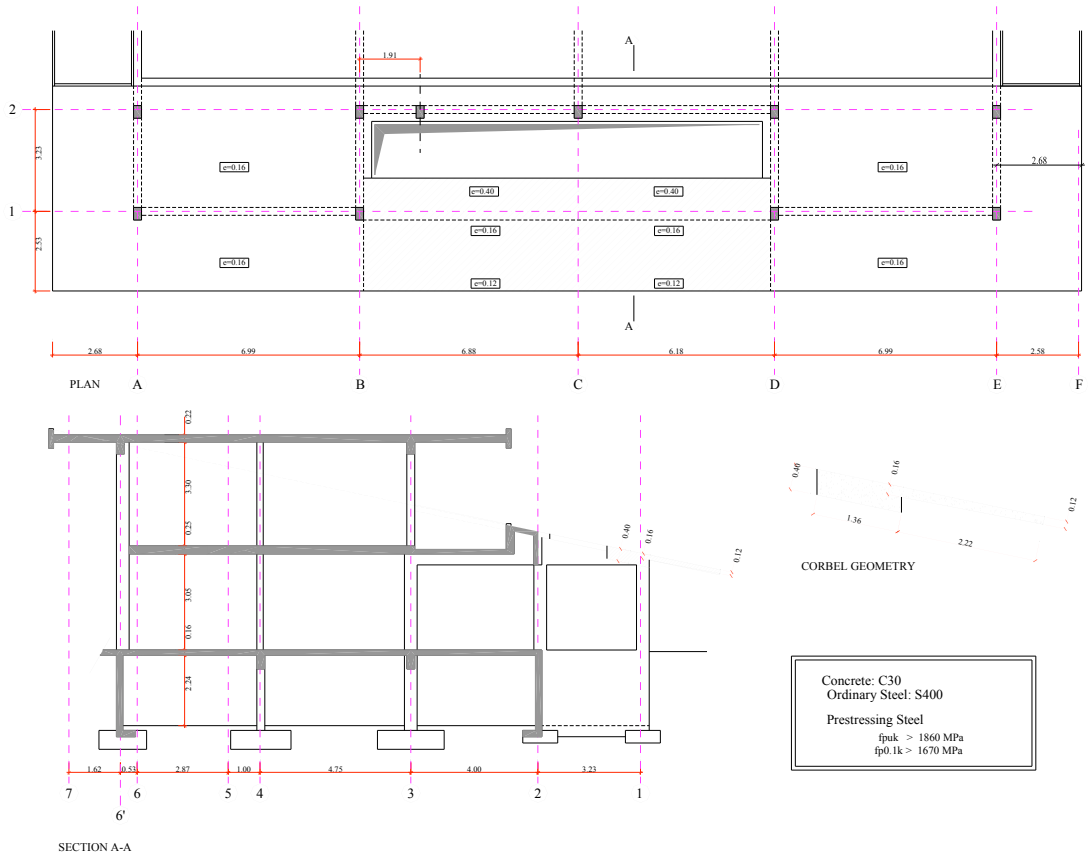

Figure 3.27: Corbel geometry

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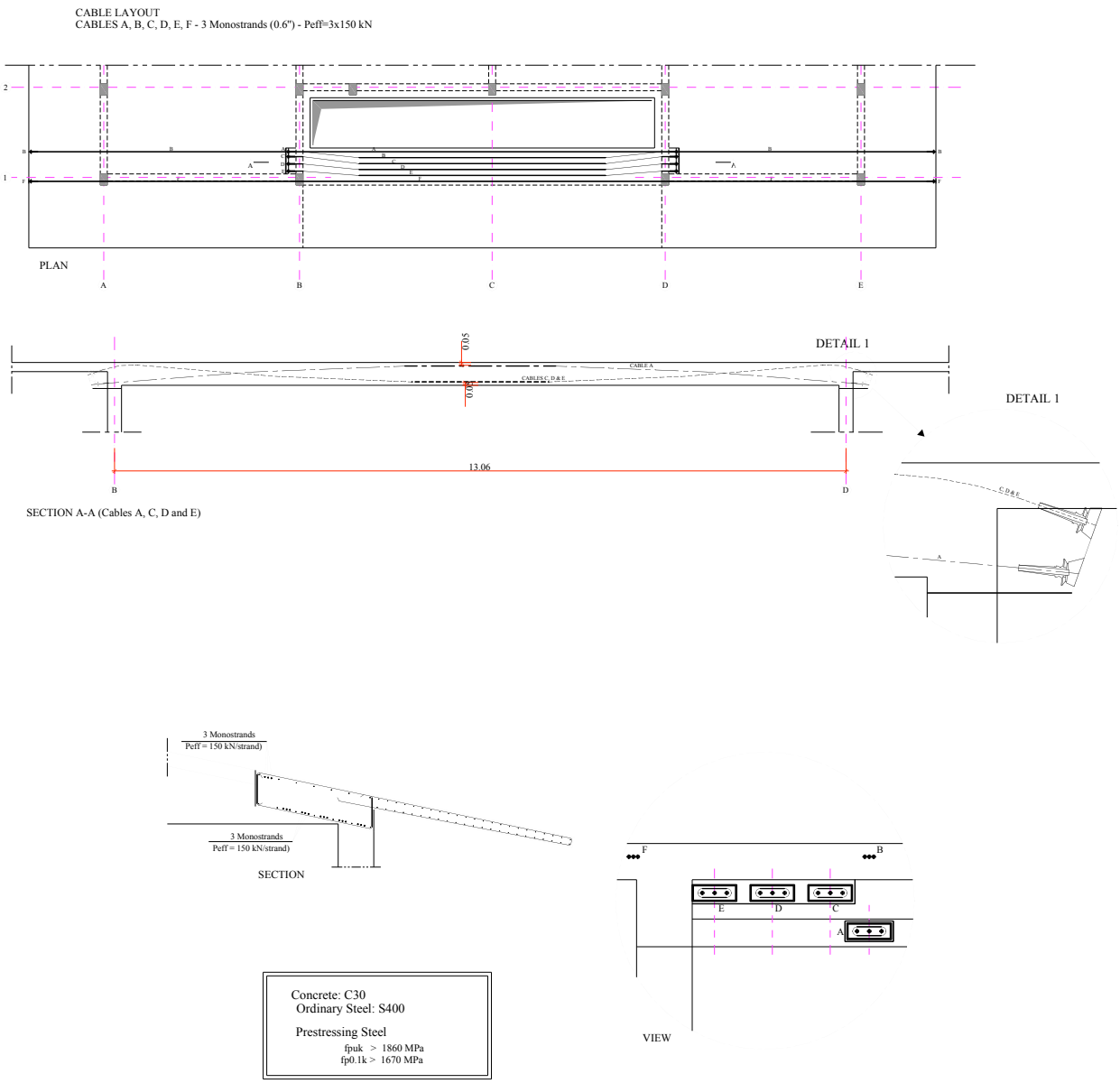

Figure 3.28: Prestressing layout

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Figure 3.29: Plan view

Figure 3.30: General views

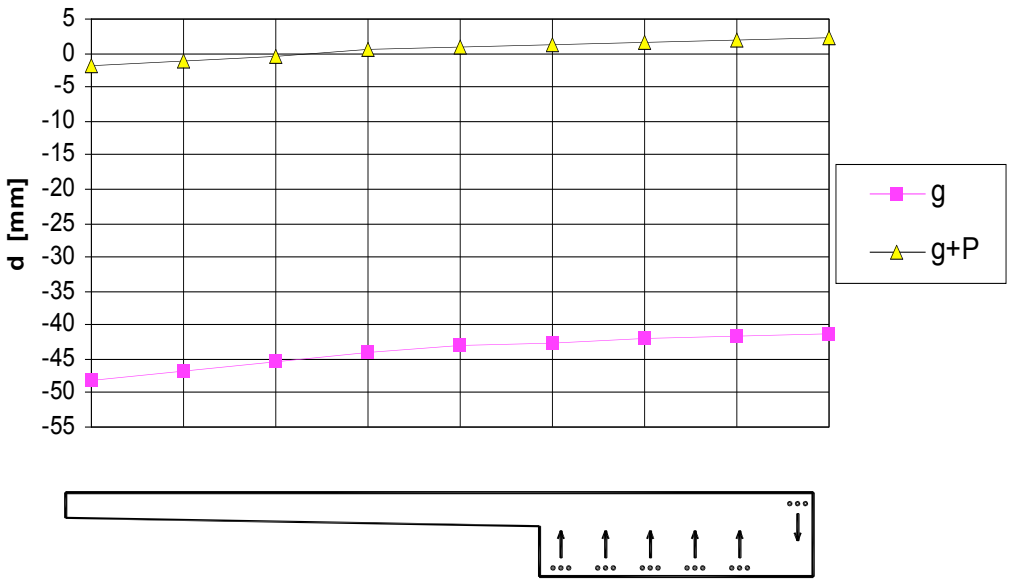

Figure 3.31: Mid span corbel deflections

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3.2.4.3 Cantilever slab "Sevens Düsseldorf"

- Designer: DOMOSTATIK AG, Switzerland

In the city centre of Düsseldorf, in the best pedestrian area, an old building was demolished and in its place a new reinforced concrete building was erected. The new building serves as a department store with sales areas in the upper floors and with underground parking facilities. The boundary of the new building is situated next to an existing building. In order to avoid expensive support and underpinning work for the neighboring building, the new building was constructed using diaphragm walls for all-round support against the earth pressures, constructing the floor slabs downwards as the excavation beneath the basement floor proceeded. The diaphragm walls were constructed in the immediate vicinity of the neighboring building. First the floor slab situated at ground level was constructed, with it resting on the diaphragm walls and acting like a lid. Then, as the structure is built above the ground floor the earth is excavated below it.

Due to the restricted light conditions caused by the nearness of the adjoining building a large opening was planned for the inside of the new building, which utilizes the incoming light through the roof in all the upper floors. In order to facilitate a free flow of persons in the area of the opening, it was planned to place no columns along the edge of the floor. This led to a big overhang of the thin floor slab, since the next series of supports was at a distance between 5.0 and 6.0 m from the free edge of the slab. This cantilever part is loaded by the usual live loads and also by concentrated loads due to the installations, e.g. escalators. The task was to reduce the large bending deflection of the slab's edges to an acceptable value.

For the deck a flat slab with square drop panels was planned. The floor slab has a thickness of d=0.30 m and the size of the drop panels is 3.0 x 3.0 m with a thickness of d=0.60 m. In order to minimize the bending deflections at the edges of the floor slab, normal to the edges, in a concentrated way, single strips with prestressing cables were arranged. In those places where the prestressing cables cannot transfer their load directly to the column heads additional transverse cables were planned. Various inserts, the irregular grid of columns and the inaccessibility of some of the edges of the slab resulted also in an irregular arrangement of prestressing cables.

The prestressing cables were installed with subsequent grouting in flat ducts. Post-tensioning with grouting was selected to keep the potential damage to a prestressing tendon due to unintentional cutting or drilling work to a minimum. Construction with flat sheaths allowed maximizing the rise of the curve of the cables.

With the selected prestressing, its execution as a pre-tensioned system and its unconventional arrangement in plan view, the bending deflection of the big overhanging edges could be reliably controlled.

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Figure 3.33: Plan view with the opening between the axes E and H

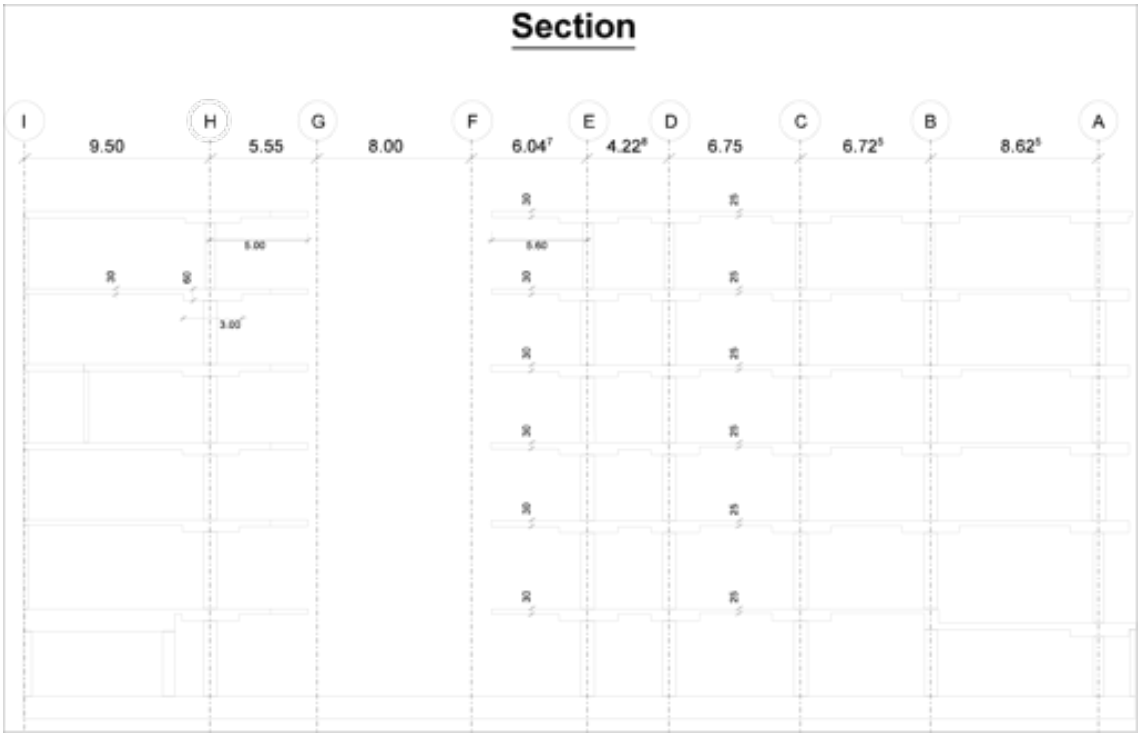

Figure 3.34: Cross section through building with the cantilever of the axes E and H, respectively H

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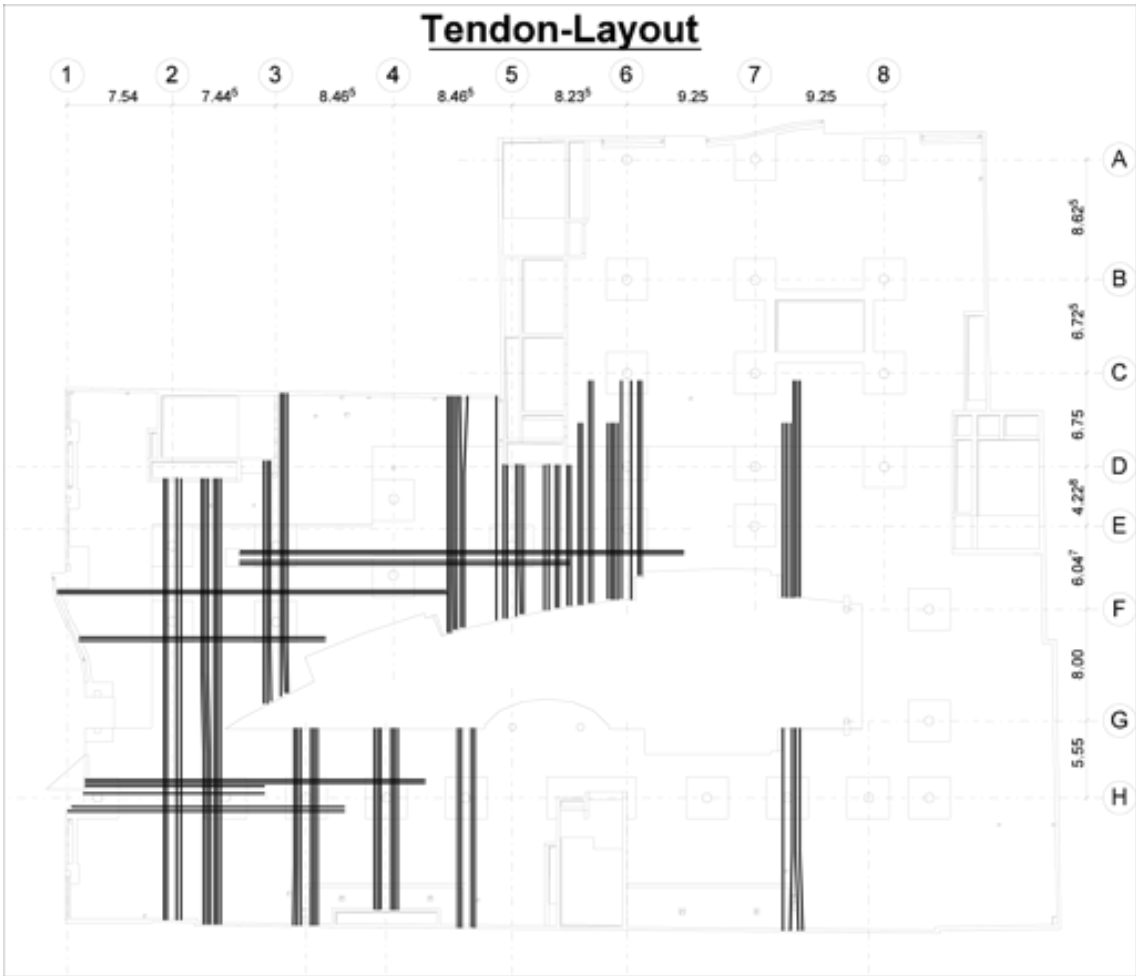

Figure 3.35: Arrangement of the prestressing cable bundles to reduce the effects of cantilevering

Figure 3.36: Floor slab reinforcement: prestressing cable and top reinforcement

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Figure 3.37: Prestressing cable over the columns with pre-concreted concrete plugs made of high strength concrete

Figure 3.38: Installation openings also serve as anchoring points for the prestressing cables

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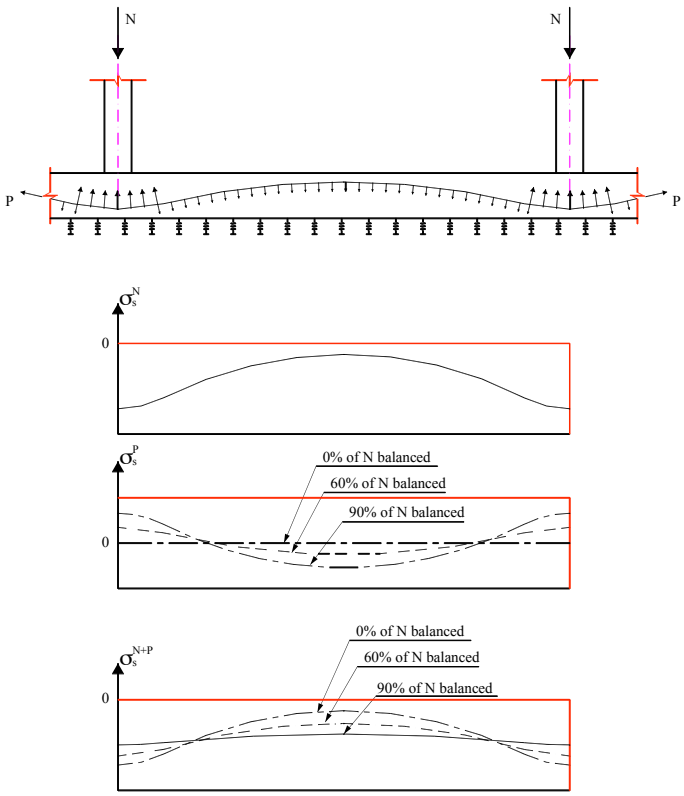
4 Post-tensioned foundations

4.1 Conceptual design

The general well known advantages of prestressed concrete can be applied to foundation concrete structures. Shallow foundations such as strip footings or raft slabs, and even concrete slabs directly on ground, are applications that can substantially benefit from the adoption of post-tensioning solutions.

The purpose of a foundation is to safely transfer applied loads from the superstructure to the soil. Additionally, specific needs for uniform soil stresses or settlements, requirements of water tightness and constructive advantages can be aspects in favour of a post-tensioned solution.

Prestressing offers the possibility of introducing a favourable system of anchorage forces and tendon deviation forces on the concrete. The transverse components of the anchorage forces and the tendon deviation forces (Figure 4.1) provide a load balancing force, resulting in a more uniform soil pressure distribution and a reduction in maximum soil pressure. The prestressing force and the tendon profile can be selected in order that, for a given load case, e.g. dead load only, a nearly uniform soil pressure distribution results.



(a) Prestressed continuous beam on elastic foundation

(b) Soil pressure due to column loads

(c) Soil pressure due to prestressing

(d) Soil pressure due to combined action of column loads and prestressing

Figure 4.1: Effect of prestressing on the soil stresses distribution [4.1]

The ULS bending and shear resistance is provided by the combination of prestressed and non-prestressed reinforcement. The use of prestressing steel can result in a substantial reduction in the total steel area. Thus, good structural details are achieved together with easier placing and compacting of the concrete. Additionally, as illustrated in Figure 4.2, the slab or foundation punching shear resistance is as well improved.

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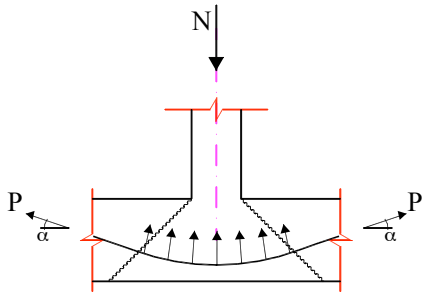

Figure 4.2: Effect of prestressing on shear-punching resistance [4.1]

The early application of prestress, in combination with an appropriate wet curing of the concrete surface, can substantially reduce the occurrence of early hydration shrinkage cracks. The in-plane anchorage forces precompress the concrete element, leading to higher cracking resistance, improved stiffness and water tightness, resulting in a better general quality of the structure.

SLS checks limit the stresses in the concrete and reinforcing steel in accordance with specified performance requirements. In general, the effects of post-tensioning and the influence of restraining elements and subgrade friction should be considered.

A check of concrete stresses at the foundation element can determine the need for stage stressing during construction. In such cases, special care has to be taken to enable accessibility of anchorage zones for stressing and injection works during construction. An inverse situation can occur if the accessibility to the anchorage zones can not be guaranteed during construction: it should be verified that prestressing can be applied without the specified column loads being present, giving particular attention to the design of the corresponding stirrups reinforcement in the columns region, in order to prevent the pull-out of the highly loaded prestressing cables.

4.1.1 Influence of stiff elements and subgrade friction

Superstructure elements can potentially restrain slab shortening and reduce compression due to prestress into the foundation. The level of restraint is directly proportional to the probable displacement of the slab at the location of the restraining point. It should be noted that, whereas restraint effects will potentially affect the precompression in the foundation element, this is not the case for the deviation forces, directly related with the prestressing force in the tendon, cf. 2.3.6.

Locating restraining items closer to the slab centre, or releasing their connection to the slab, minimizes their influence on the slab. A smooth transition at steps in the slab raft reduces subgrade interference. An optimised concrete placement schedule and prestressing sequence, in combination with distributed minimum reinforcing, are effective at controlling restraint problems.

A comprehensive treatment of subgrade friction effects is presented in [4.1], where it is shown that in ordinary cases they remain at a quite acceptable level. Tests [4.1] indicate that the slab can shorten up to Δ_s = 0.5 mm on a smooth surface (friction coefficient, $\mu \approx 0.5$), or up to Δ_s = 3.0 mm on a rough surface ($\mu \approx 1.0$), before sliding occurs at the interface. Displacements below these values are essentially unrestrained, being absorbed by deformation of

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the subgrade. For very long slabs such as pavements, two layers of polyethylene can be placed over the subgrade to facilitate slab movement [4.1,4.2].

The influence of subgrade friction on the concrete compression stresses is illustrated on the next figure [4.2].

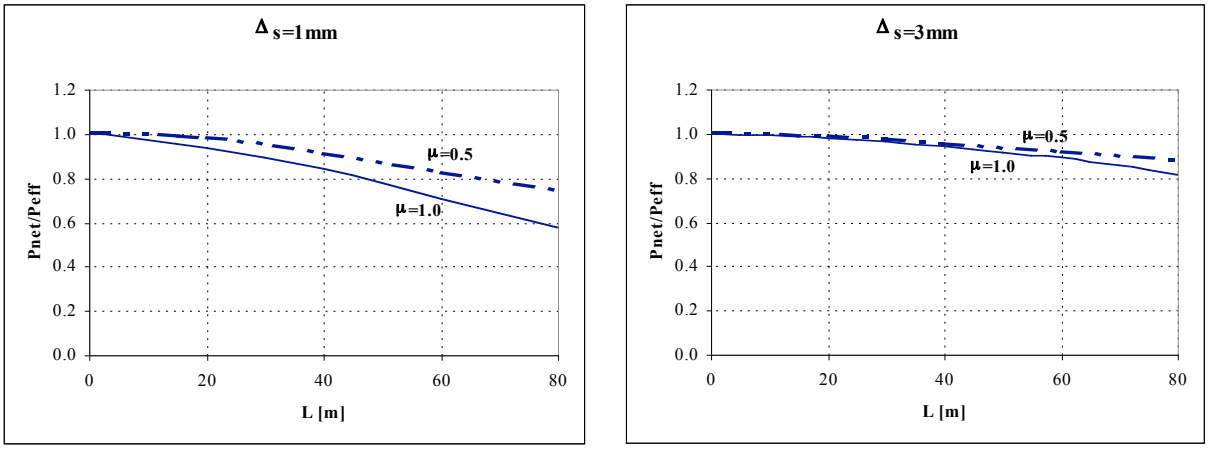

Figure 4.3: Effect of subgrade friction on precompression of the raft [4.2] (σ_{cPeff}=1 Mpa; P_{net}=effective prestressing force in centre of slab, L – total length of the slab)

In general, shrinkage, creep and temperature movements of the slab, and associated subgrade restraint effects, are less important. Creep of the subgrade typically exceeds that of the structural concrete. The surrounding earth dampens thermal fluctuations in the raft, and, moist earth and groundwater permeating into the concrete substantially reduces, or even suppresses, drying shrinkage. Surface cracking that may occur can be controlled by the minimum ordinary reinforcing.

On the other hand, Figure 4.3 assumes that the slab is prestressed while it is subjected to its self-weight only. Foundation elements often have to be prestressed in stages, the additional loads will be superimposed and, as a consequence, subgrade friction effects will be increased.

4.1.2 Raft foundations

The principle of a raft foundation is similar to that of a floor slab turned upside-down. The distributed soil pressure acts at the bottom surface and is held in equilibrium by the downward-acting concentrated forces from columns and walls. The tendons are arranged as in an elevated slab, but with an inverted layout such that the low points are under the columns and walls, and the high points are in the spans.

In setting the slab depth, primary consideration is given to shear resistance and allowable soil pressure. Common L/h values range from 10 to 12, depending upon the modulus of subgrade reaction, the load magnitude and arrangement, the concrete strength, the level of prestress, and the allowable differential deformation. The following figure [4.2] can support a preliminary selection of the raft thickness and prestressing requirements.

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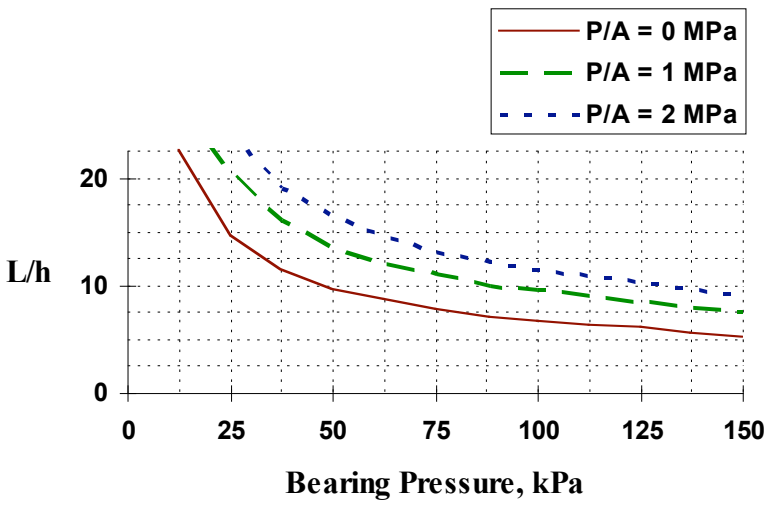

Chart notes:

1. Bearing pressure, $\sigma = N/L^2$, where:
   L= distance between column centre lines
   N= unfactored dead load on column

2. h= slab thickness
   a= radius of column contact area

3. Chart based on a/L = 0.05; larger a/L values produce slightly higher L/h ratios.

4. Chart based on C25 concrete; higher grades produce slightly higher L/h ratios

Figure 4.4: Design-aid for preliminary raft sizing [4.2] (σ_{cPeff}=P/A)

A simplified check of the punching shear resistance is generally performed. If the resistance is insufficient, the slab thickness can be increased overall, or just thickened under the columns. To improve the tendons contribution for shear resistance and local flexural effects in the columns region, most of the tendons in each direction should be located in the column strips.

The prestressing force is sized in general for precompression requirements and/or load balancing criteria. Practical experience recommends effective prestress values in the range from 0.75 to 1.5 MPa, balancing 60% to 100% of the permanent loads. The proposed values are generally appropriate to control slab deflections and improve punching shear resistance, being moderate enough to avoid excessive camber or slab shortening.

Early stressing of some of the tendons is advantageous for controlling hydration cracking. From the practical aspect it is desirable to stress the tendons once, instead of multiple stressing stages. Care must be exercised in order to not overstress the slab at transfer, when the gravity loads from the superstructure do not yet balance the tendons deviation forces. A check of the net bending stresses at critical locations of the slab determines the need for stage stressing during construction.

4.1.3 Post tensioned slab on ground

The post-tensioned industrial ground floors constitute the main application. The system allows thinner crack-free slabs without or with less expansion joints. The subgrade quality and the applied loads level govern the thickness of the pavement [4.3]. Practical experience shows that with a poor quality subgrade and higher loads the post-tensioning solution can offer an economic solution in regard to greater durability and minimized maintenance or repair costs.

Usually post-tensioning for pavements is designed to ensure that acting stresses, due to concentrated loads [4.4], wheel loads and temperature gradient, does not exceed the assumed tensile concrete strength. In particular cases fatigue strength under repetitive loadings should as

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well be considered. Practical experience recommends effective prestress values in the range from 1.0 to 2.0 MPa.

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4.2 Applications

4.2.1 Foundation raft "P&C Bergisch Gladbach"

- Designer: DOMOSTATIK AG, Switzerland

Due to the poor soil conditions in the city centre area, a pile-raft foundation was planned for the new building of a textile department store. The admissible soil pressure of the soil containing pockets of soft peaty material was approximately 100kN/m^2. Due to the restricted space and the very tight schedule an alternative solution was sought.

The concentrated and, in terms of magnitude, non-uniform column loads were to be transmitted to a stiff slab in such a way that this resulted everywhere in a uniform soil pressure. A "stiff" foundation slab can, apart from its thickness, also be achieved by adopting prestressing. By specifying the prestressing along the individual strips, the cable curvature below the columns provides the vertical component to support the concentrated loads directly, and distribute them uniformly in the bays. In this way the highly concentrated slab area is relieved and a uniform loading in the slab is achieved. Prestressing arranged only in the column strips can lead to the desired load relief. Due to the high bending and shear action in the area of the columns the slab was strengthened by thickening.

The prestressing had to be adjusted because of the non-uniformly distributed column forces. The number of prestressing cables and their arrangement in one or both directions leads to the desired stress relief. The inaccessibility of three of the four slab boundaries, in addition to the selected prestressing arrangement, required that only some cables were stressed at the construction joints, with the corresponding coupling. In combination with the untensioned reinforcement this unorthodox prestressing arrangement could be successfully executed.

Due to the confined space the accessibility of the anchor positions could not be guaranteed as construction progressed. As a result, immediately after the hardening of the slab the prestressing cables had to be tensioned, without the specified column load being present. The corresponding stirrup reinforcement in the area of the columns should be paid attention to, in order to prevent an uncontrolled pull-out of the highly loaded prestressing cables.

Grouted prestressing members with up to 9 strands in a duct were used. The highly concentrated load in the prestressing cables and the high bending resistance to be achieved by means of grouting governed the choice of the prestressing cables.

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Figure 4.5: Foundation slab with the individual construction stages

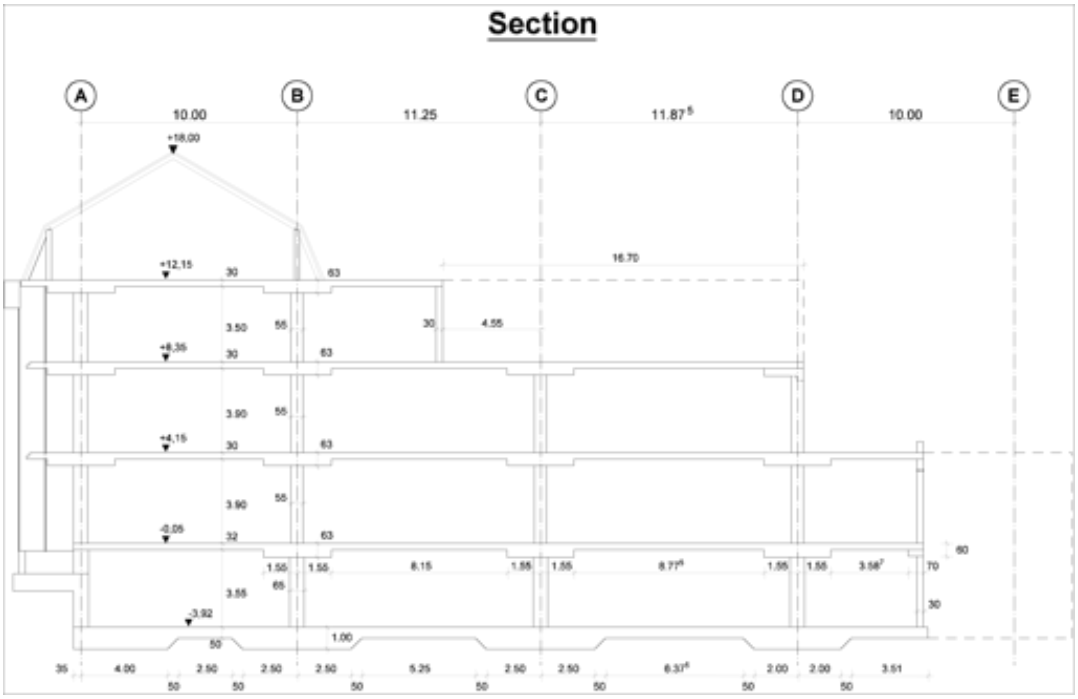

Figure 4.6: Section though building with details of raft foundation

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Figure 4.7: Plan of building with axes and the stiffening of the raft foundation

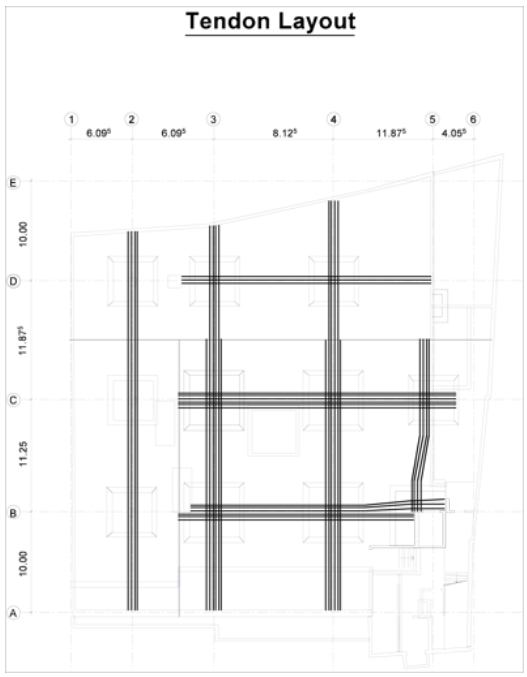

Figure 4.8: Arrangement of the prestressing cable bundles in the column strips

Figure 4.9: Placing the prestressing cables within the column strips

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Figure 4.10: Relieving the stresses by means of prestressing cables in the thickened parts of the raft foundation

Figure 4.11: Reinforced raft foundation construction stage

4.2.2 Slabs on ground

Widely used in Australia and America, by commercial, industrial and car park owner, the post-tensioned ground slab system has only partially been introduced into Europe.

The post-tensioning of industrial floors has provided both developers and builders with advantages in a wide range of applications. Projects such as storage and distribution centres, air-craft hangars, water tank bases and transport facilities have all adopted post-tensioned ground floor slabs.

Post-tensioning offers certain advantages over conventional constructions such as thinner crack-free slabs with less or no expansion joints. Being more flexible, the thinner slab enables the sub-grade to carry a larger share of wheel loads. As the thickness of a concrete pavement is governed by the sub-grade quality and the applied loads, poor quality sub-grades combined with high loads lead to a pre-stressing solution becoming economic with regard to greater durability and minimized maintenance and repair costs.

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Two case studies of where post-tensioning has been used in such a structure, the first is an internal 150 mm thick slab post-tensioned with an un-bonded mono-strand system for moderate static and mobile loads in Casa de Piedra, Chile and the second is a heavily loaded 300 mm thick external pavement pre-stressed with a bonded slab post-tensioning system in Melbourne, Australia. Each case study illustrates the application of the design methods previously described in Chapter 4.1.

4.2.2.1 Nestlé Distribution Centre, Casa de Piedra, Chile

- Designer: VSL LTD, Chile, 2000
- Contractor: Constuctora Marco SA , Chile, 2000

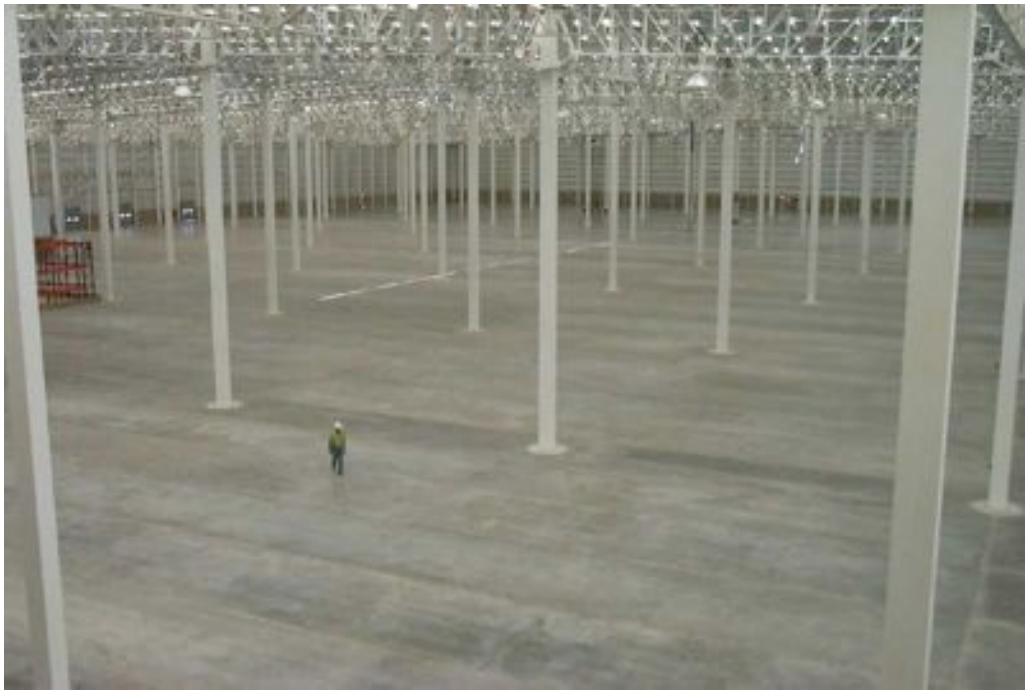

Figure 4.12: Post-tensioning eliminates joint problems for slab on grade floors

This Nestlé Distribution Center uses a 150 mm thick post-tensioned industrial ground slab which consists of bundles of four un-bonded 0.5“ mono-strands at 1.5 m centers in both directions. There is no un-stressed normal reinforcement in the slab itself other than at edges and adjacent to the anchorages.

The slab plan dimension of 150 m × 220 m, constructed over a two month period from November 2000 through January 2001, is monolithic without any expansion joints with the entire 27,000 m^2 of slab area being divided into 32 major pours, with a number of minor pours as shown on the concreting sequences plan in Figure 4.13.

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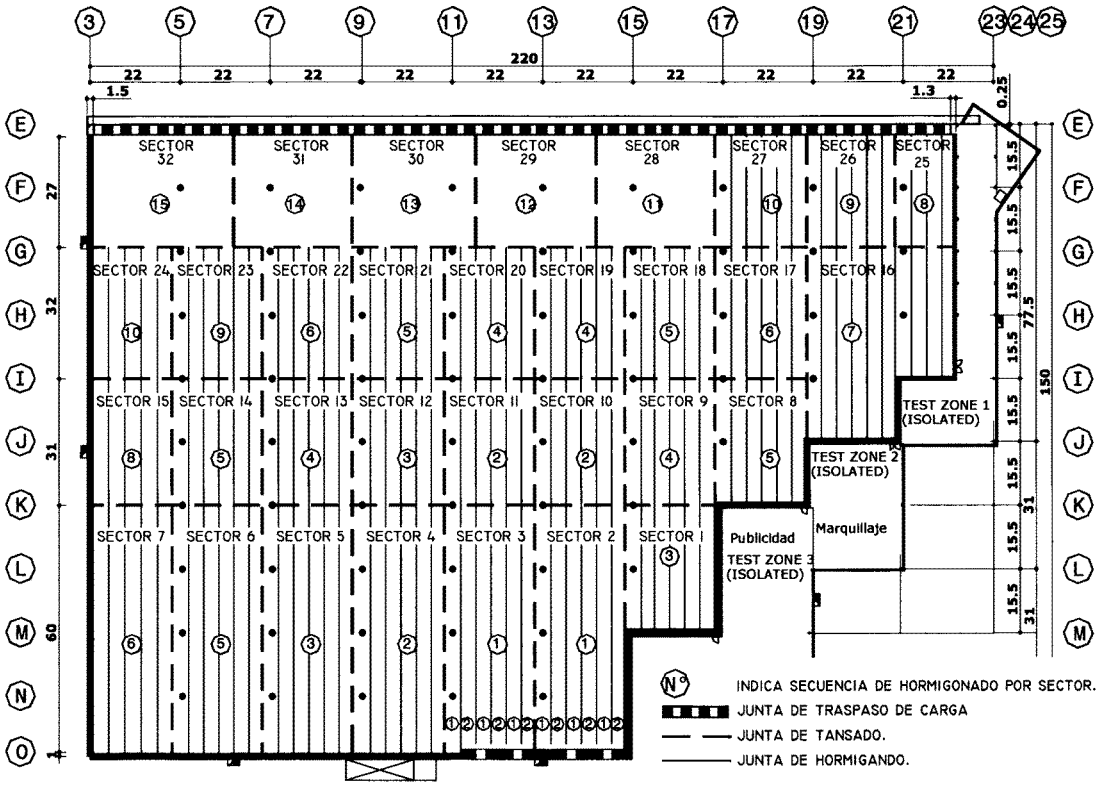

Figure 4.13: Floor plan showing the sector numbers (N°), the concreting joints and the prestressing sequences

The stressing joints are at 30 m along the 150 m length and 22.5 m along the 220 m length. The slab major pour were axially post-tensioned by the un-bonded 0.5" mono-strands once again arranged in bundle of 4 strands running in both directions and located at the centroid of the slab at 1.5m spacing. From the experience of similar projects, the resulting pre-stressing content of 4.5 kg/m^2 for the Nestlé Distribution Center is typical for such normal industrial floor structures. The early application of an initial stressing load corresponding to 50% of the jacking force at a concrete transfer strength of 20 MPa (typically after 1 day) controlled any early shrinkage cracking problems. The final stressing up to 100% of the jacking load was carried out upon the concrete achieving 35 MPa cylinder strength (typically after 3 days) which was higher than the design compressive strength of 28 MPa. In order to allow displacement between slab and sub-grade mainly during pre-stressing, a clear separation between the two was provided by a 25 to 50 mm layer of sand on top of the compacted sub-grade. To avoid the wet concrete from penetrating the sand layer, as well as helping to reduce the friction coefficient to the lowest possible value between 0.3 to 0.5, two layers of plastic sheet were placed over the sand layer.

The final pre-stress application was designed to ensure that the slab would remain permanently in a state of compression. Under the most adverse factored combination of the applied stacker loads (60 and 30 kN), crane load (36 kN) and temperature effect, the pre-stressed concrete design ensured that the maximum tensile stresses of the slab would not exceed the tension resisting capacity of the concrete section which is the total of pre-stress and the flexural tensile strength of he concrete, as shown on Figure 4.14. In cases of repetitive loading, a third dimensioning criteria is the stress ratio (*SR*) which is a measure of the net working tensile stress to the net cracking stress. The stress ratio calculated is compared to the value corresponding to the allowable number of repetitions. For *SR* = 0.5 or less, the repetitions are unlimited.

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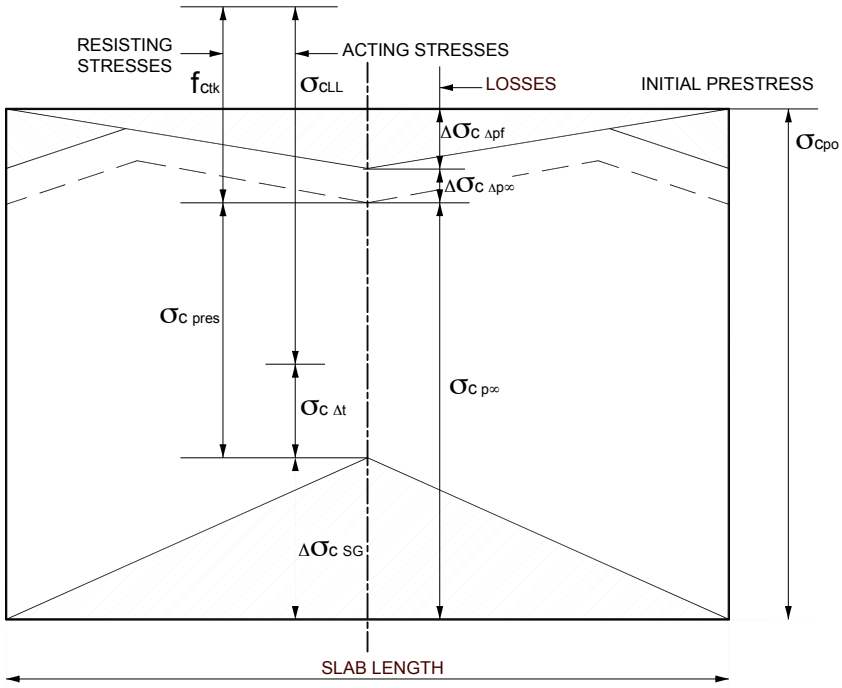
SCHEME OF STRESS DIAGRAM
OF PRESTRESSED SLABS ON GRADE



WHERE:

RESISTING STRESSES

σ_{Cpres} = PRESTRESS

σ_{Cpo} = INITIAL PRESTRESS

$\sigma_{Cp\infty}$ = PRESTRESS AFTER LOSSES

f_{ctk} = FLEXURAL TENSILE STRENGTH OF CONCRETE

γ_{ct} =PARTIAL SAFETY FACTOR FOR CONCRETE TENSITE STRENGTH

ACTING STRESSES

σ_{CLL} = MAX TENSILE STRESS DUE TO WHEEL LOAD

$\sigma_{C\Delta t}$ = STRESS DUE TO TEMPERATURE GRADIENT

LOSSES

$\Delta\sigma_{C\,\Delta pf}$ = FRICTION LOSSES DUE TO CABLE WOBBLE

$\Delta\sigma_{C\,\Delta p\infty}$ = PRESTRESS LOSSES DUE TO RELAXATION CREEP AND SHRINKAGE

$\Delta\sigma_{C\,SG}$ = LOSS OF PRESTRESS DUE TO SUBGRADE FRICTION

DIMENSIONING CRITERIA (CENTER OF SLAB)

CRITERIA 1

$$\underbrace{\sigma_{Cpres} + f_{ctk}/\gamma_{ct}}_{\text{RESISTING STRESSES}} \geqslant \underbrace{\sigma_{CLL} + \sigma_{C\,\Delta t}}_{\text{ACTING STRESSES}} \rightarrow \text{pres}$$

CRITERIA 2

$$\sigma_{Cpres} = 1 \text{ N/mm?}$$

DETERMINATION OF INITIAL PRESTRESS AT THE EDGE

$$\sigma_{Cpo} = \frac{\sigma_{Cpres} + \Delta\sigma_{C\Delta\,p\infty} + \Delta\sigma_{C\,SG}}{e^{-\frac{k_L}{2}}}$$

Figure 4.14: Scheme of stress diagram of prestressed slabs on grade

In order to satisfy these criteria, the following losses acting against the pre-stressing force at the anchorage were considered:

- Cable friction losses
- Long term losses
- Sub-grade friction or restraints.

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4.2.2.2 FCL Transport Melbourne

- Designer: VSL Australia LTD, Melbourne, 1998
- Contractor: Marral Constructions PTY LTD , Australia, 1998

Figure 4.15: 100 t container stacking area under construction

FCL, a major national transportation and container-handling group, needed to relocate their operations as a result of the Melbourne Citylink project. All floors and pavements of FCL's new facility, situated in the Melbourne port side area of Footscray, were designed as post-tensioned to accommodate a range of loading requirements including:

- Container stacking (4 boxes high, 100 tonne stacks)
- Operating container handling forklift (96 tonne axle load)
- Empty container storage and maintenance building
- Warehouse racking storage (6 tonne post loads)

The post-tensioned slabs were designed to suit the poor 'Coode Island Silt' sub-grade material present across the site with slab thicknesses ranging from 300 mm in the heavily loaded container stacking areas, to 160 mm in the more lightly loaded warehouse areas. A total area of 39,000 m^2 of slab was built with the largest single slab pour of 2,800 m^2 requiring 826 m^3 of concrete. The design of all pavements and footings needed to allow for possible long term differential settlements characteristic of the local sub-grade.

In this second example, we will limit our description to the heavily loaded container stacking area AB 13 and AB 15 shown in Figure 4.16.

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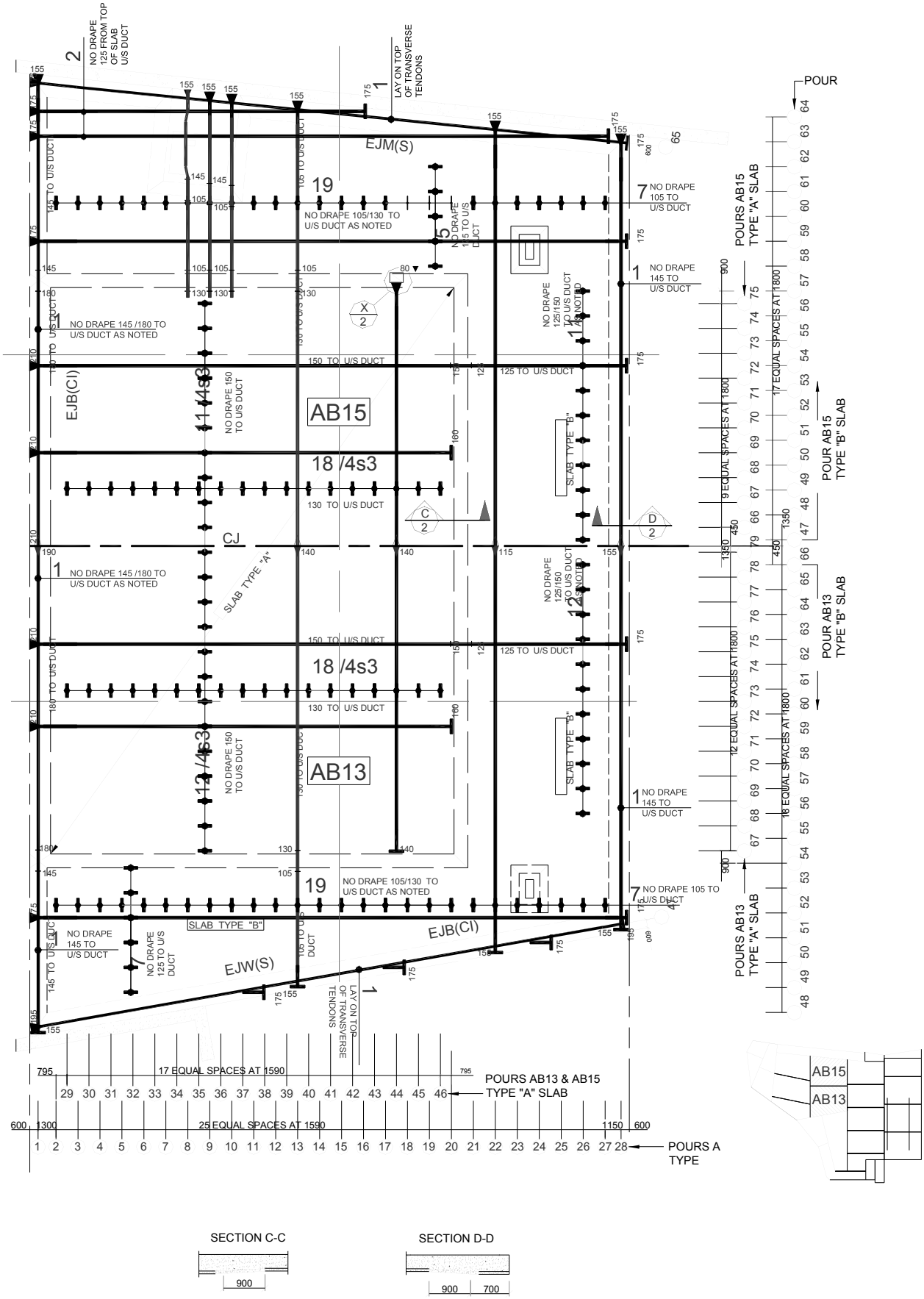

Figure 4.16: Tendon arrangement

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The 300 mm thick slab in the central areas and 420 mm at the edges used bonded slab tendons at 0.80 m through 0.9 m spacing.

The concrete with a compression cylinder strength of f_{ck} = 40 MPa was cast and cured so as to minimize any initial and long-term shrinkage. The stressing was carried out in two stages:

1. 25% of the jacking force (25% x 85% of the specified tensile strength) 24 hours after completion of pour
2. 100% of the jacking force (85% of the specified tensile strength) at a minimum concrete strength of f_{ci} = 25 MPa.

The concrete flexural tensile strength considered in the calculation is according AS 1012 $f_{ctk} = \gamma_{ct} \times 0.438\, f_{ck}^{2/3} = 4.8$ MPa with γ_{ct} = partial safety factor for concrete tensile strength. The critical loading for the container stack is given on the Figure 4.17.

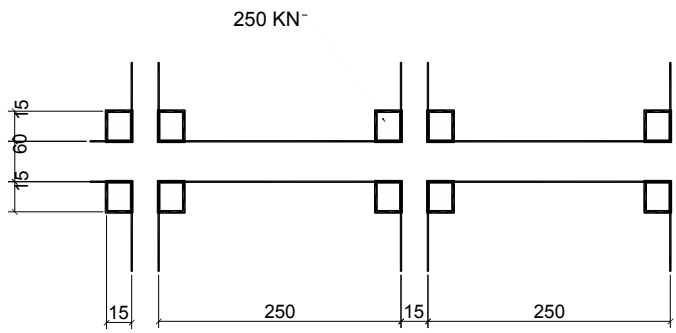

Figure 4.17: Container stack loading

A temperature gradient of 0.03° Celsius/mm corresponding to the outside conditions was adopted.

In contrast with the Nestlé Distribution Center, where a satisfactory modulus of sub-grade reaction of 76 kPa/mm was available, the site geotechnical report obtained from a CBR test revealed the Footscay site was only 5% on a soil consisting on fill (gravely sands and clays) extending to depths ranging from 1.1 through 2.5 m and clay silt of high plasticity extending to depths ranging from 5 through 7 m.

To increase the sub-grade modulus, it was decided that a bound sub-base layer 125 mm thick using cement stabilized crushed rock (CSCR), with 4% cement by weight, was placed over the natural ground or filling, then compacted to a minimum of 95% modified compaction and then subsequently trimmed to a tolerance of +10 mm / - 5 mm of the required level thus achieving a satisfactory modulus of sub-grade reaction of 80 KPa/mm, similar to the value of the Nestlé Distribution Centre, was achieved as shown on Figure 4.18.

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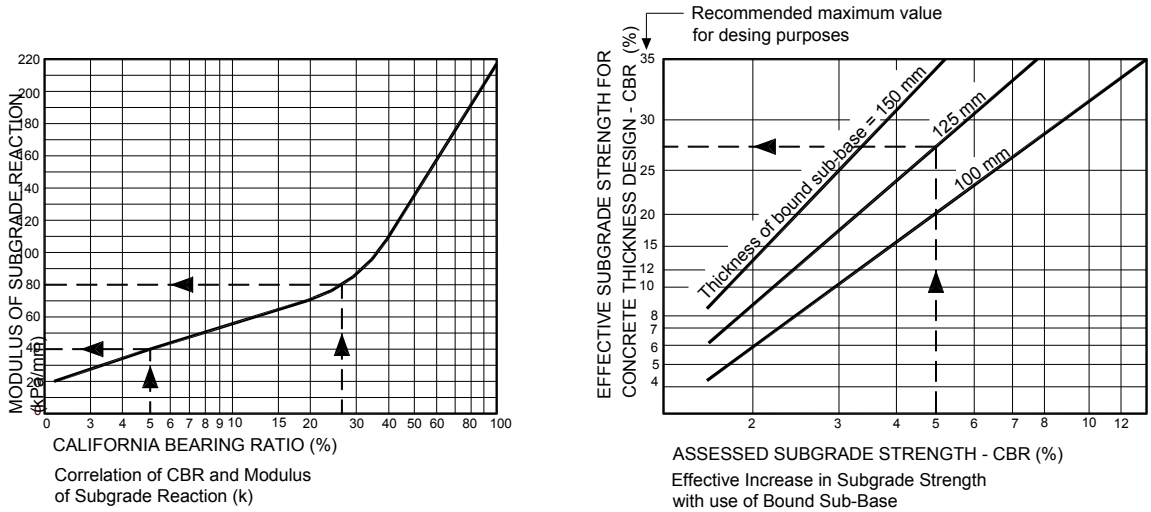

Figure 4.18: Subgrade strength and bound sub-base

Based on this value, the maximum tensile stress due to the factored point loads was calculated and the required initial prestress (σc_{po}) defined according the formula:

$$\sigma_{c_{po}} = \frac{\sigma_{c_{LL}} + \sigma_{c_{\Delta t}} + \Delta\sigma_{c_{SG}} - \frac{f_{ct}}{\gamma_{ct}} + \Delta\sigma_{c_{\Delta p\infty}}}{e^{-k.\frac{L}{2}}} \left[N/mm^2\right]$$

where:

$\sigma_{c_{LL}}$ = max. tensile stress due to wheel load

$\sigma_{c_{\Delta t}}$ = stress due to temperature gradient

$\Delta\sigma_{c_{SG}}$ = loss or prestress due to subgrade friction

$\Delta\sigma_{c_{\Delta p\infty}}$ = long term losses due to C+S

Figure 4.19: View of ground slabs before concreting

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4.2.3 Isolated footing "Lisbon St. Gabriel Tower"

- Designer: JSJ Consult/A2P Consult, Portugal, 1998
- Contractor: ENGIL SA, Portugal, 1998

St. Gabriel tower is a 25 elevated floors building, one of the twin towers recently built in the Nations Park, at the oriental part of Lisbon where the 98 World Exposition took place.

Composite floors, steel beams connected to a concrete slab, two important structural concrete cores and a mesh of steel columns, constitute the elevated structure. Due to architectural reasons, gravity loads carried by the columns are transmitted to an impressive post-tensioned concrete transfer structure, vessel shaped, at level 20.00. The transfer floor is just supported at the main cores, carrying down to the foundations the total vertical loads from the elevated floors.

Soil conditions allowed direct foundations with maximum contact service stresses of about 0.5 MPa. The adopted solution consists in an isolated footing, with in plane dimensions of 13.0 m × 34.0 m. Intensive technical and economical comparative studies lead to a post-tensioned concrete *flexible* solution with a variable depth between 1.30 m and 2.50 m.

Prestressing steel provides the major part of the total tensile force needed at Ultimate Limit State (about 21 000 kN/m), leading to simpler detailing and more economical construction procedures. In addition, as illustrated in Figure 4.23, the use of post-tensioning slightly improves the soil stress distribution, getting better quality and durability of the structure.

In order to not overstress the structure, prestressing was applied in three steps, the first one after the execution of the footing and the two remaining phases, following the introduction of loads, during the tower construction. Special care was assured so that accessibility of anchorage zones for stressing and injection works during construction was provided.

Figure 4.20: General view

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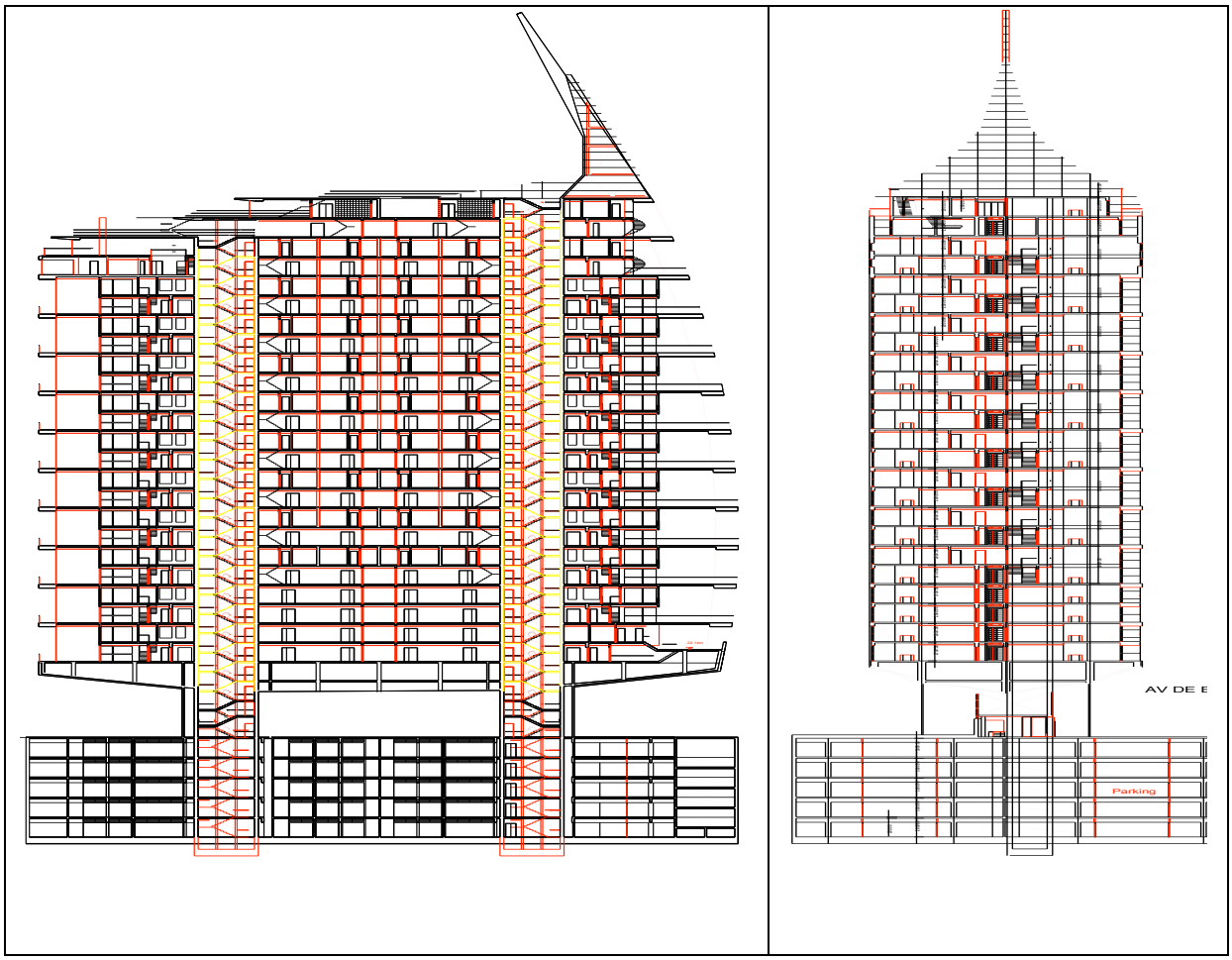

Figure 4.21: Longitudinal and transverse sections

Figure 4.22: General view during construction

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Figure 4.23: Foundation geometry

Figure 4.24: General view during construction

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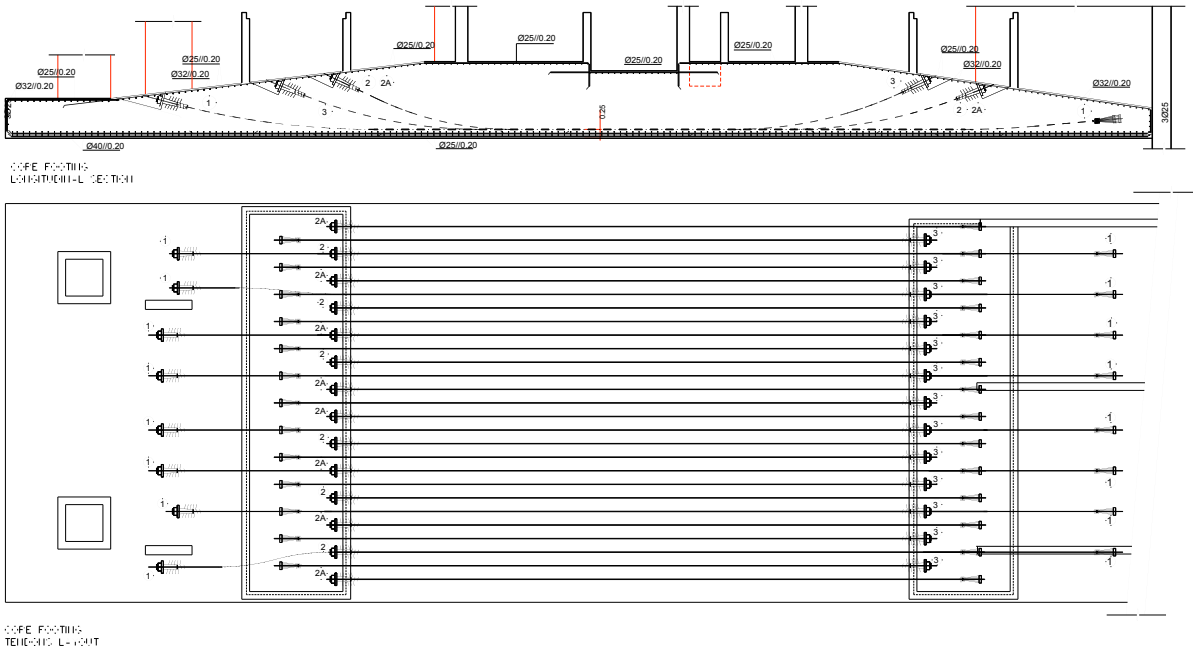

CABLES	PRESTRESSING PHASES		P_{eff}(KN)
1	AFTER EXECUTION OF THE FOOTING	8 X 22 STRANDS (0.6")	24 640
2A	AFTER EXECUTION OF LEVEL 9.00	8 X 31 STRANDS (0.6")	34 720
2 & 3	DURING THE TOWER CONSTRUCTION (TWO PHASES)	19 X 31 STRANDS (0.6")	82 460

Active Anchorage

Dead Anchorage

Figure 4.25: Prestressing layout

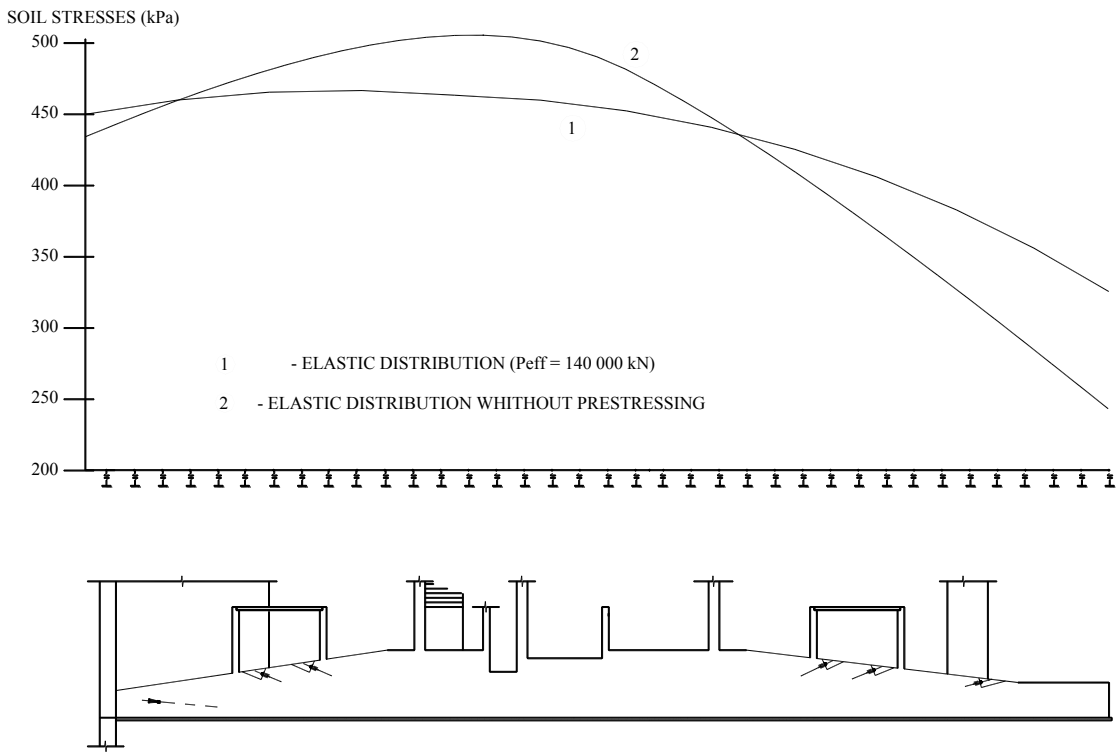

Figure 4.26: Soil pressure distribution

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5 Post-tensioned transfer slabs and beams

5.1 Conceptual design

In buildings it is sometimes needed to transfer, at a certain level, some of the vertical loads to other alignments in order to obtain bigger spans at the lower levels.

In this way, a more economic structure, with smaller column or wall spacings can be envisaged for the upper floors, while at the lower levels, in particular at the ground level, bigger inter-column distances can be adopted. This is often the case at office buildings or hotels where lobbies and other facilities require more open space.

The transition from the smallest grid structures to a larger column spacing is done by means of a transfer structure (with beams or slabs).

In order to transmit the high concentrated forces from the columns or wall, that have no continuity to the foundations, the transfer structures require considerable rigidity and strength. The design of the transfer system is dependant not only on the flexural behaviour (resistance and deformability), but, in general, is considerably influenced by shear or punching resistance.

Due to the considerably high forces involved, the transition structure usually requires large depths and great amounts of reinforcement. Post-tensioning is obviously, in these cases, a very effective way to reduce both depth and reinforcement content. Due to the high strength of the prestress steel (f_{pu} = 1770 to 1860N/mm^2) the amount of normal reinforcement needed for flexion design is drastically reduced, thus improving detailing. The arrangement is simplified, making it possible to have top, bottom and face reinforcements kept nearly as any standard case. Prestressing also provides a positive measure against early age shrinkage cracking in concrete.

The tendon layout may be based on draped tendons or on top and bottom layers of horizontal tendons. In the first case, apart from the favourable effects of load balancing, an important contribution for the shear or punching resistance is obtained by the inclined component of the cables near the supports. Of course that, even in the situation of horizontal cables layout, the compression induced on the concrete has some contribution to the shear or punching resistance. In this regard it has to be verified if this compression can be counted on, this is, if there is no important restrictions to the free introduction of the axial prestress effects.

Considering the usually small slendernesses of these structures and the type of loads applied (concentrated loads) the shear resistance should normally be modelled by a combination of concrete arches and struts with the tension ties provided by the post-tensioning and general reinforcement.

If draped tendons are used, it may be necessary to apply the prestress in steps, in order not to over-balance the vertical load present at different stages of construction.

When the transfer structures thickness is considerably high (≅ 2.5m to 5m) it is usual to adopt a multiple layer construction concept. In these cases the transfer plate is built in phases (2 or 3), each layer being prestressed then, usually with horizontal cables. In this way the load on the falsework is reduced (as the weight of the second level can be taken by the previous one) the concrete pours have more manageable sizes and all prestressing is normally applied all the time of the transfer level construction, thus simplifying building procedures.

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5.2 Dr. H.R. Ganz; A. Payne, 'Prestressing to Enhance Constructibility of Bridges and Buildings'.

5.2 Applications

Hong Kong is a city composed of commercial, residential and industrial highrise structures. A large percentage of these structures incorporate transfer structures of one form or another. Where the small column and wall grid of residential, office or hotel framing needs the large spacious column grid of commercial spaces or lobbies, it is common for a solid transfer slab or plate to be used. These typically vary between 1.5 m to 4.5 m thick depending on the respective upper and lower column grids and the imposed column and/or wall loads.

5.2.1 Pacific Place Hong Kong

- Designer: Wong and Ouyang Hong Kong Consulting Engineers / VSL International LTD., Bern, 1988
- Contractor: Dragage et Travaux Publics, Hong Kong, 1988

The first post-tensioned transfer plate constructed in Hong Kong was for the Pacific Place major commercial development in 1988 consisting of two 61 storey high-rise towers with an open podium/retail area. Each of the towers have a transfer level above the podium structure to accommodate the change in the vertical load paths. Tower A has a transfer structure comprising a 7.81 m deep concrete truss located around the perimeter of the tower. Both top and bottom chords of the truss were pre-stressed with 12 tendons in the top chord and 16 tendons in the bottom chord, each tendon containing 42 strands of 12.9 mm diameter. The top and bottom pre-stressed chords are separated by normally reinforced concrete inclined web members. The pre-stressing in the upper and lower chords controls the large tension forces and subsequently limiting the cracking of the concrete. Although it was not specifically used as such on this structure, the tendons can also be utilised for their ultimate strength allowing substantial reduction in normal reinforcement, as was the case for tower B transfer plate.

For Tower B, as there were a number of internal load bearing walls whose loads needed to be transferred to both the perimeter columns and central core, a different type of transfer element was required. The conforming solution was to provide a 4.5 m thick normally reinforced concrete transfer plate.

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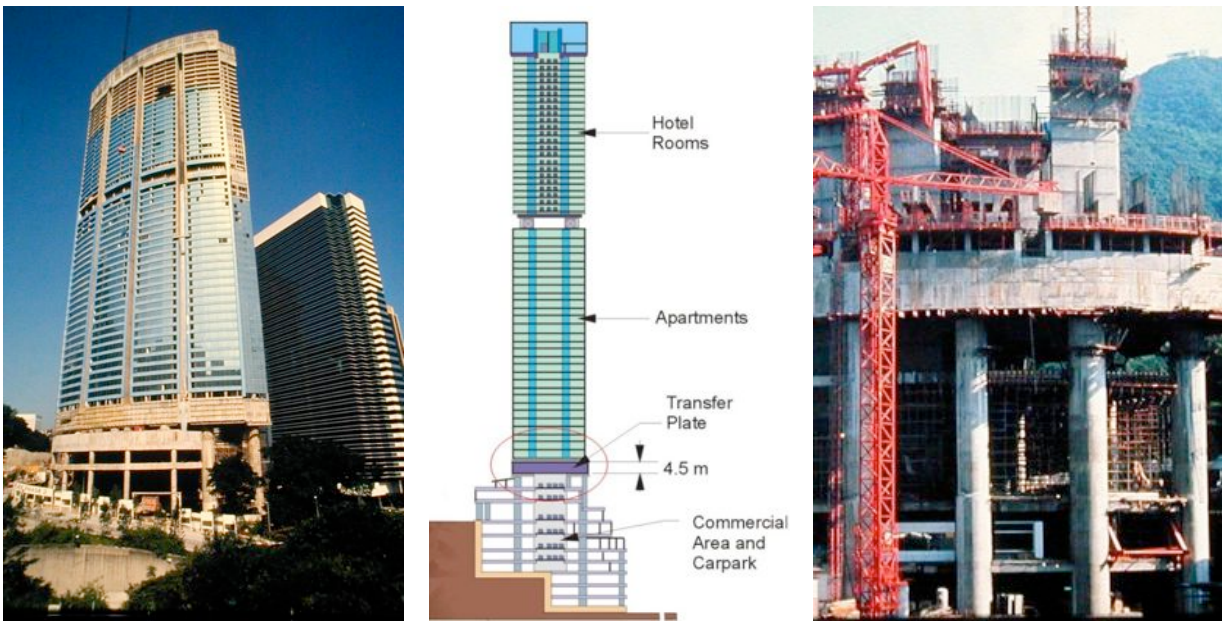

Figure 5.1: a) Pacific Place buildings after completion
b) Section showing location of transfer plate
c) View of the transfer plate during construction

An alternative design using a prestressed concrete plate was proposed and subsequently constructed, where the same overall depth (4.5 m) was maintained to satisfy the shear requirements however with the introduction of only 22 kg/m^3 of pre-stressed reinforcement the normal/unstressed reinforcement was significantly reduced from 500 kg/m^3 in the conforming scheme to 180 kg/m^3. The reduction in normal reinforcement led to both financial savings in materials as well as in a construction program where the simplified design and detailing allowed the construction of the 6,300 m^3 of concrete plate in only five weeks which included the fixing of 1,150 tonnes of un-stressed reinforcement and 135 tonnes of pre-stressed reinforcement. The plate was constructed in three layers each of 1.5 m thick layers with the first layer designed to support the latter two. The arrangement of the prestressing tendons and the simplification of the reinforcement can be seen in Figure 5.2 below.

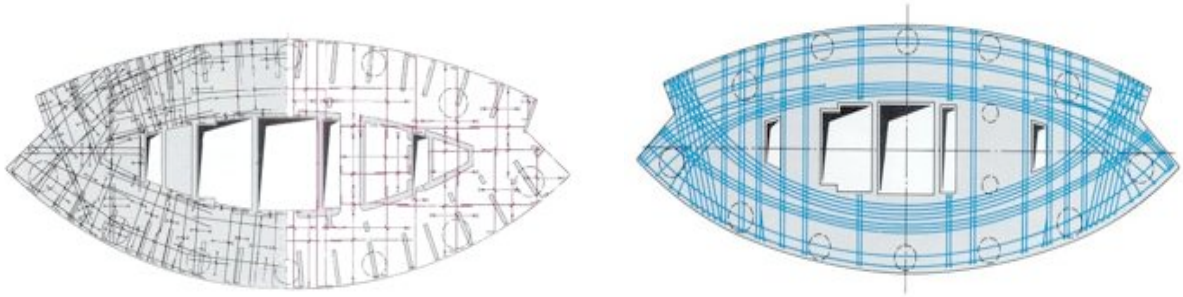

Figure 5.2: a) Typical arrangement of reinforcement, original / alternative
b) Typical tendon arrangement

The longitudinal tendons were stressed at each end whilst the transverse tendons were stressed at one end only. The majority of the tendons were placed in horizontal planes however, tendons which draped vertically were used in the vicinity of the three large columns at either end of the plate to reduce the substantial effect of transverse shear. The in-plane reinforcement, which consists of three layers of orthogonally placed bars, is bent at all free edges (as can be seen in Figure 5.3 to act as edge reinforcement with stirrups detailed as lapped hairpin. The

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transverse reinforcement requirement was sub-divided into 3 distinct groups; 0.15% (minimum); though 0.30%; and 0.60%.

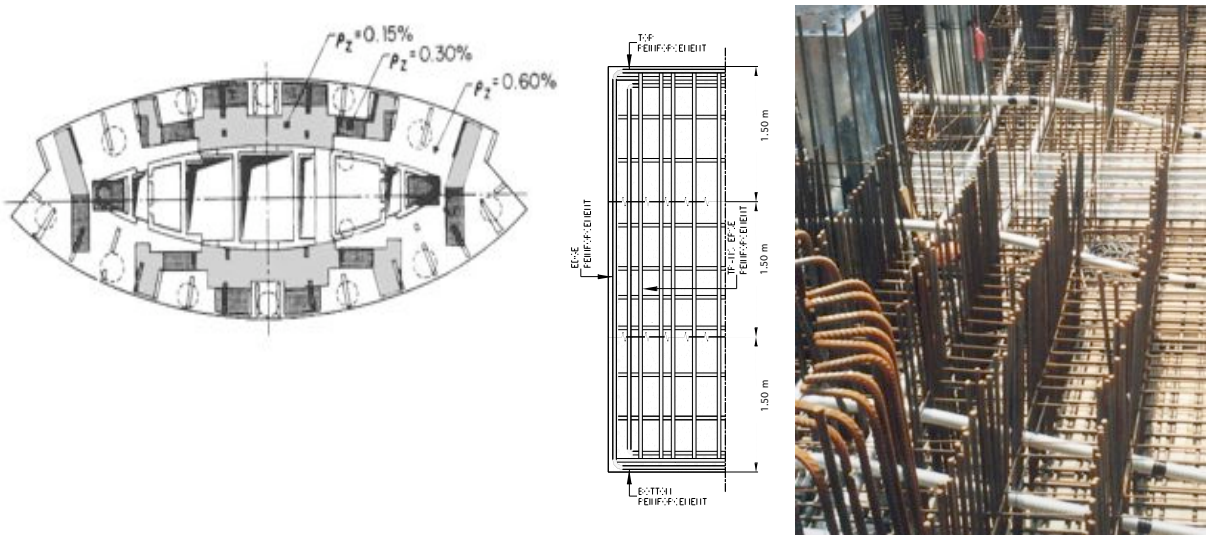

Figure 5.3: a) Transverse reinforcement ratios
b) Cross section showing non prestressed in-plane, transverse and edge reinforcements
c) Transverse reinforcement

5.2.2 Sandwich-Class Housing Development

- Designer: JMK Joseph Chow & Partners LTD. / VSL International LTD., Bern, 1996
- Contractor: Yau Lee Constructions CO LTD., Hong Kong, 1996

In early 1995, the Hong Kong Housing Board made enquiries into the feasibility of utilising pre-stressed transfer plates for the 'Sandwich-Class' housing development of 3 residential blocks in north Ap Lei Chau.

Figure 5.4. a) The three residential blocks facing Aberdeen Harbour, Hong Kong
b) View of the transfer plate during the construction

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Each block consists of three podium levels below a transfer plate, forty-two residential floors and a main roof. The transfer plate was required to once again redirect the loads from the closely spaced walls of the residential towers into the large column-free layouts demanded at the podium levels.

The objective of the study to use pre-stressing was to reduce the overall cost of the structure, minimise the thickness of the transfer plate and simplify the construction. A post-tensioned transfer plate solution was found to be significantly cheaper and thinner than the conforming normally reinforced concrete transfer plate, reducing the plate thickness from 4.2 m to 3.2 m.

To expedite the construction, the post-tensioned transfer plate was designed to permit construction of the plate in two layers. The first layer of 1.3 m was designed to support the weight of the second 1.3m layer. With this feature, the supporting falsework for the transfer plate was made lighter and subsequently erected in a shorter time. Furthermore, the post-tensioned solution led to the simplified detailing and reduced quantity of unstressed reinforcement (Figure 5.5).

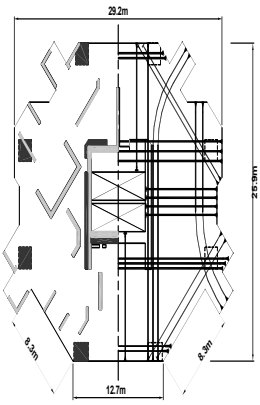

Figure 5.5: a) Wall and support lay-out and typical tendon arrangement

b) View of the in-plane and edge reinforcements and the anchorage locations

The transfer plates were constructed by concreting the first 1.3 m stage, then upon achieving the required concrete transfer strength of f_{cu} = 30 N/mm^2 the tendons within the first stage were stressed and subsequently grouted. The formwork and false-work were then removed from beneath the transfer plate, the upper 1.3 m of the transfer plate was then concreted and finally the remaining layers of tendons stressed and grouted before the execution of all upper floors. As a result of the simple construction method for the post-tensioned transfer plate, the main contractor's program of works was considerably shortened with the construction of all three plates beginning mid 1996 and completed by October of the same year.

Both plates were designed in accordance with the conceptual design principles outlined in chapter 5.1 based first on ultimate limit state considerations for both bending and shear: calculation of extreme values of the forces with a variety of factored load combinations

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including wind and seismic loads. For bending, the slab moments M_x, M_y and M_{xy} are combined together according to the rules of superposition to calculate the 'reinforcement-moments'. The four resulting reinforcement-moment combinations allow the design of the necessary longitudinal reinforcement in the two principle axis (X and Y) assuming linear plasticity. For shear, the design method is based upon British Code BS 8110 assuming a cracked section in flexure with a minimum shear resistance of the concrete (V_{crmin}) equal to 0.67 N/mm^2. As it is usually decisive in the determining of the transfer plate thickness, a check of the maximum shear stress at face of support is carried out. The shear forces at ultimate are represented graphically from the outputs of the FEM calculations which give the maximum factored shear force $V_{max} = \sqrt{V_x^2 + V_y^2}$.

Shear forces considered are those at a distance greater than $d/2$ from the face of support. The reinforcement designed for the forces at $d/2$ from the face of column were then continued to the support.

Based on the reinforcement section defined at ultimate limit state and the minimum in-plane un-stressed reinforcement between 0.13% and 0.15% per surface (upper and lower edge of transfer plate) in both X and Y directions, a check at serviceability limit state is provided considering surface crack widths limited to 0.2 mm. The unstressed reinforcement is increased if the crack width exceeds the allowable value. At Serviceability Limit States, the area of pre-stressed reinforcement is considered as un-stressed reinforcement with the residual normal force due to the pre-stressing taken into account at the centroid of the concrete section with the effect of the deviation forces and anchorage eccentricities included in the flexural moments.

Since Pacific Place development in 1988 over 60 pre-stressed transfer plates have been completed with a total area of 50,000 m^2. The dimensioning of the plates for the residential blocks has followed the same principles as the plate design of the Pacific Place development as outlined in [5.1] 'Design of Concrete Slabs for Transverse Shear' P. Marti, ACI Structural Journal, March-April 1996. This highly successful application shows that the design procedure is fully operational and ready to be used for similar plates and foundation slabs applications. As well as proving to be an economic solution, the introduction of pre-stressing in transfer plates has greatly simplified and thus improved the constructability of these traditionally heavily reinforced structures with a lean and clean pre-stressed solution leading to the obvious benefits for the contractor both in time and overall resource usage [5.2].

5.2.3 Transfer slab "Lisbon Eden Hotel"

- Designer: JSJ Consult, Portugal, 1994
- Contractor: Teixeira Duarte SA, Portugal, 1994

The Eden building situated in the town center of Lisbon recently suffered an important architectural and structural intervention. It was a well-known cinema and theatre where the interior part of the building was demolished so that a new apartment hotel and a commercial center could be built in place. A reinforced concrete structure was designed to preserve the main part of the façade and the old staircases between the entrance level and the 2nd floor (Figure 5.6 shows the building after the intervention).

The semicircular disposition of the hotel rooms over the 2nd floor level coupled with the needs for a regular structural grid on the lower levels, with columns spaced approximately 6.5 to 8.5 meters, and the obligation to preserve the above mentioned staircase, has lead to the design of a transfer slab 0.60 m thick. The prestressing solution was conceived mainly in order to

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guarantee a good flexion behavior for the 13.0 m span over the staircase and to give the necessary punching over the main columns of the lower levels. The prestress was applied in two phases, at the time of the slab construction and when the 5th floor was being built. The cable layout adopted is presented in Figure 5.9.

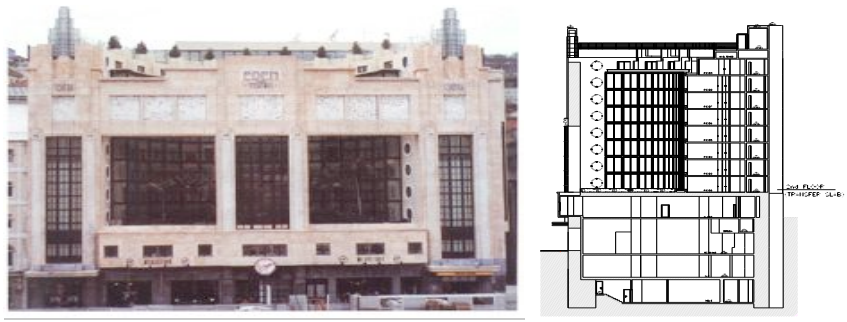

Figure 5.6: General view of the façade and building transversal section

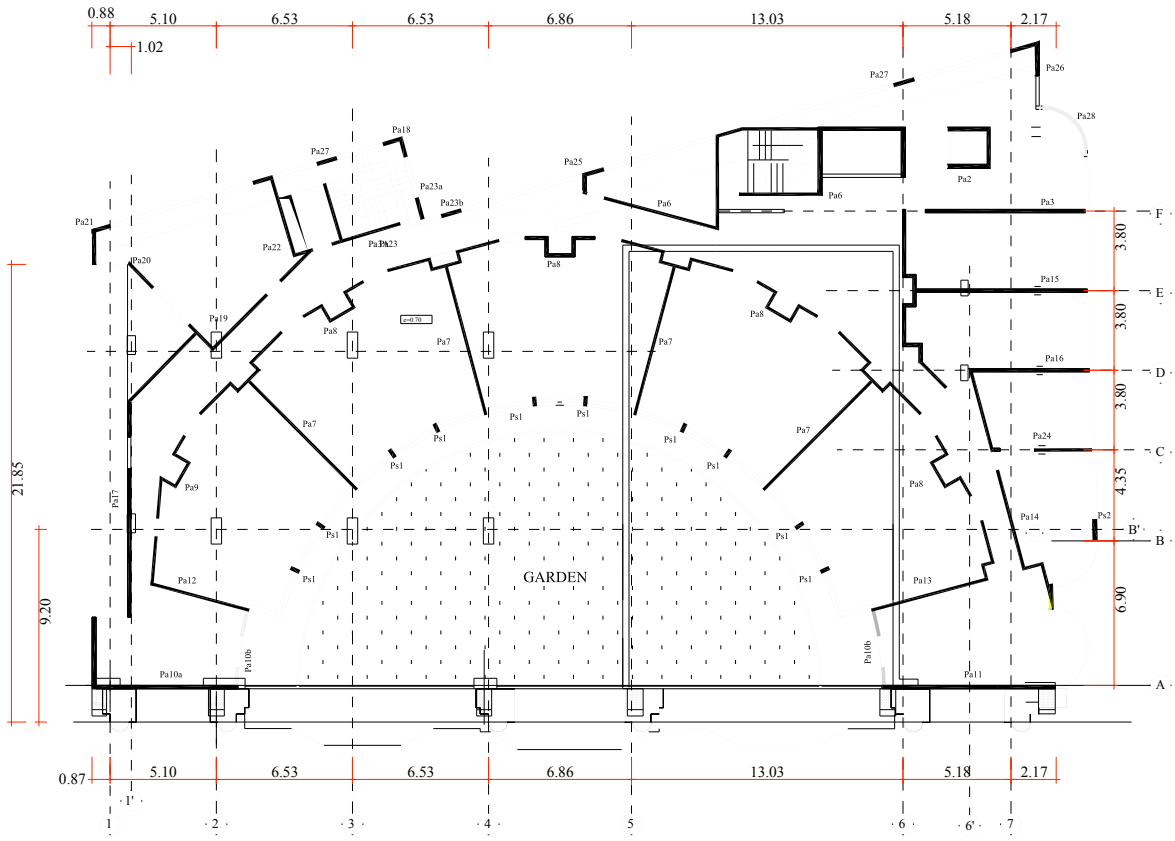

Figure 5.7: Transfer floor (2nd level)

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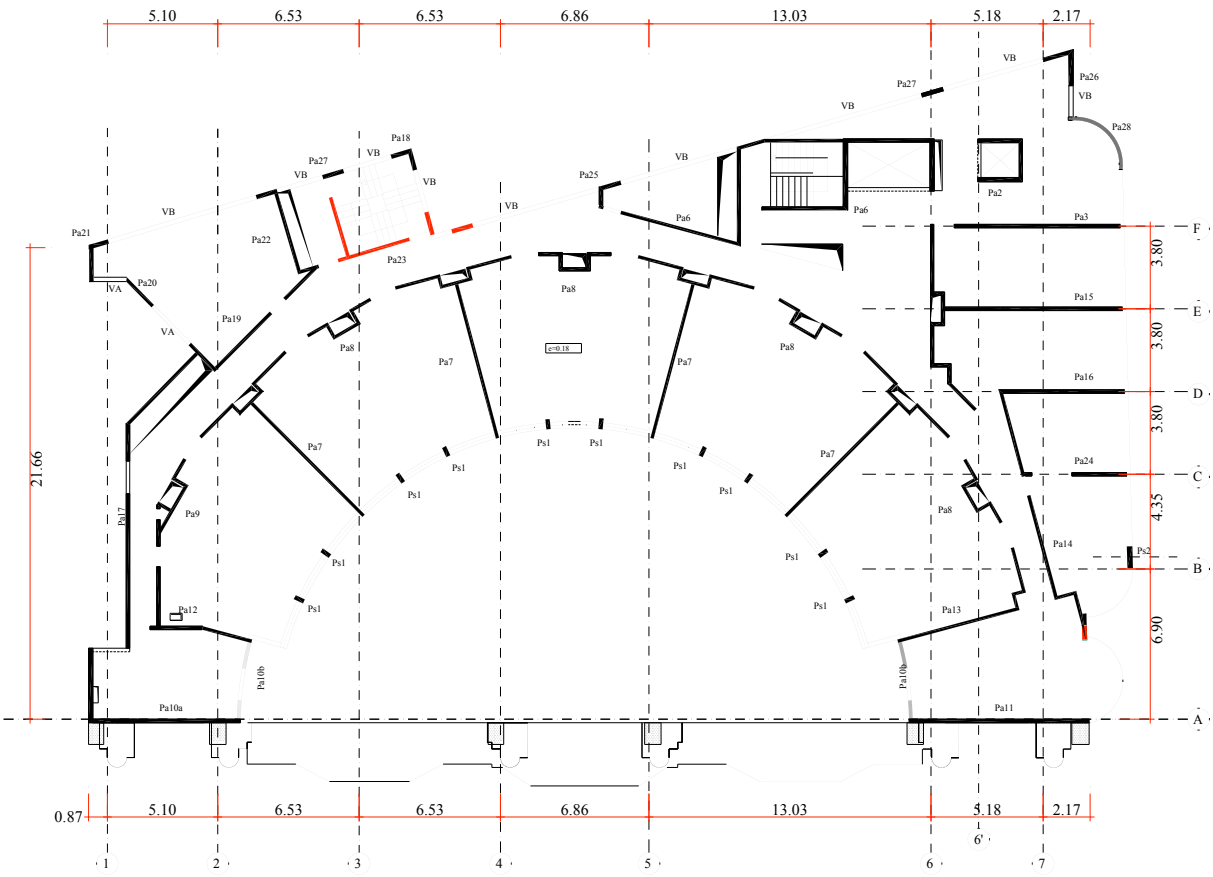
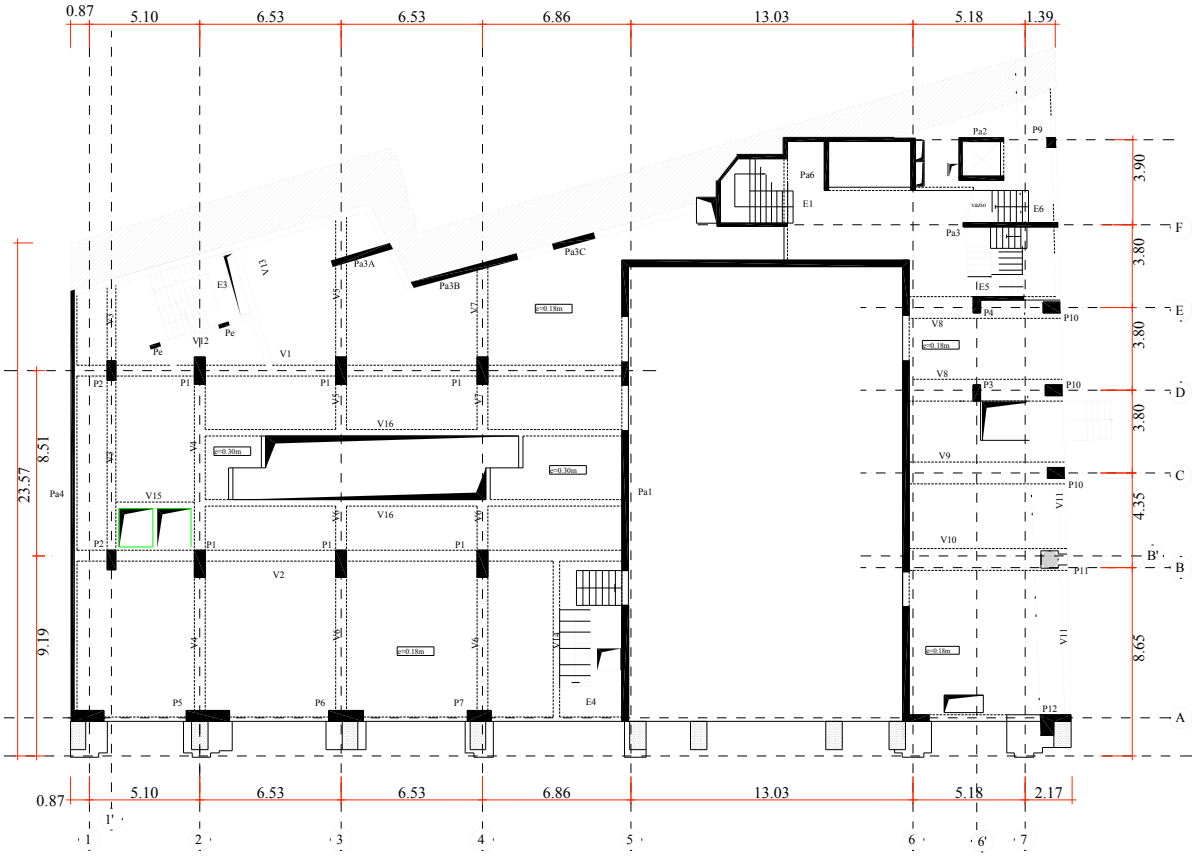


UPPER FLOORS (Hotel Floors)



ENTRANCE FLOOR (Virgin Megastore)

Figure 5.8: Floor Plans at the Hotel room levels and at the lower levels

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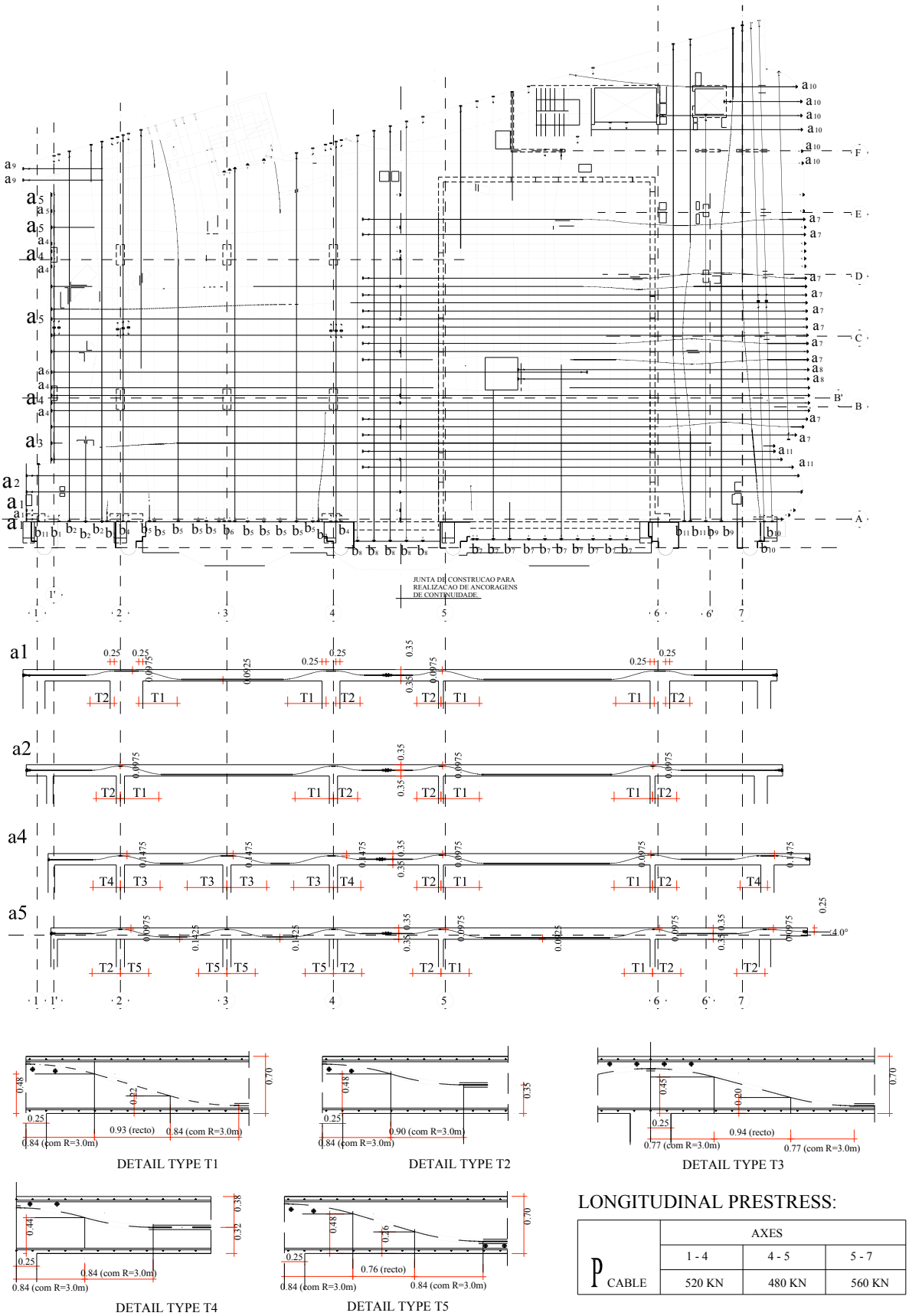

P CABLE	AXES		
	1 - 4	4 - 5	5 - 7
	520 KN	480 KN	560 KN

Figure 5.9: General prestress layout

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5.2.4 Transfer beams "Funchal Crown Plaza Hotel"

- Designer: JSJ Consult, Portugal, 1998
- Contractor: Edifer Construções SA, Portugal, 1999

The Crown Plaza Hotel in Funchal is a twin structure building with 6 floors over the main entrance level and three below as presented before at § 3.2.4.1. The guest rooms, all facing the seashore on a regular basis, are about 3.5 m wide which permitted a basic 7 meter span structural layout with a 0.25 m thick reinforced concrete slab.

For architectural reasons, the main entrance hall needed a clear span of about 18 m. Given the circumstances a system of prestress transfer beams was designed so as to support the loads transmitted from the 6 floors above.

Post-tensioning beams with continuity to the adjacent spans were designed in order to obtain a more efficient structure and adequate behavior under service and ultimate conditions.

Prestress was designed in order to, approximately, balance the permanent loads, in addition to longitudinal reinforcement, to ensure adequate resistance to bending at ultimate limit state. Four prestressing cables was adopted in each of the three beams.

Due to the short distance of 3.5 m between one of the upper column, cables A and C had a special layout. Cable A extends over the upper part of the beam to account for the tension due to the applied load near the support. Cable C, on the other hand, although not efficient over the support for flexion, induces an equivalent load that contributes to equilibrate the applied load.

Due to the variation of loads during construction, the beams were prestressed in two phases. The first phase corresponded to the application of prestress to cables type B after construction of the 1st floor. The 2nd phase corresponded to the application of the cables, type A and C, after the construction of the 4th floor.

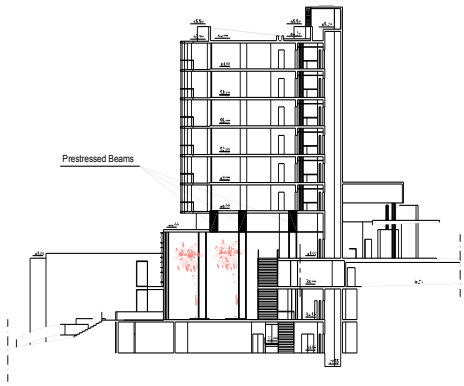

Figure 5.10: Hotel transversal section showing the prestress beams over the main entrace hall

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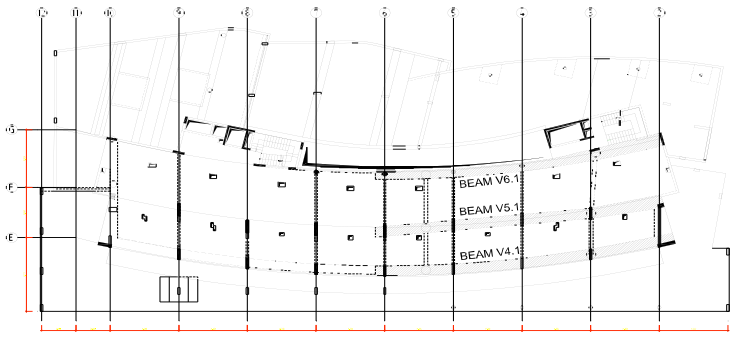
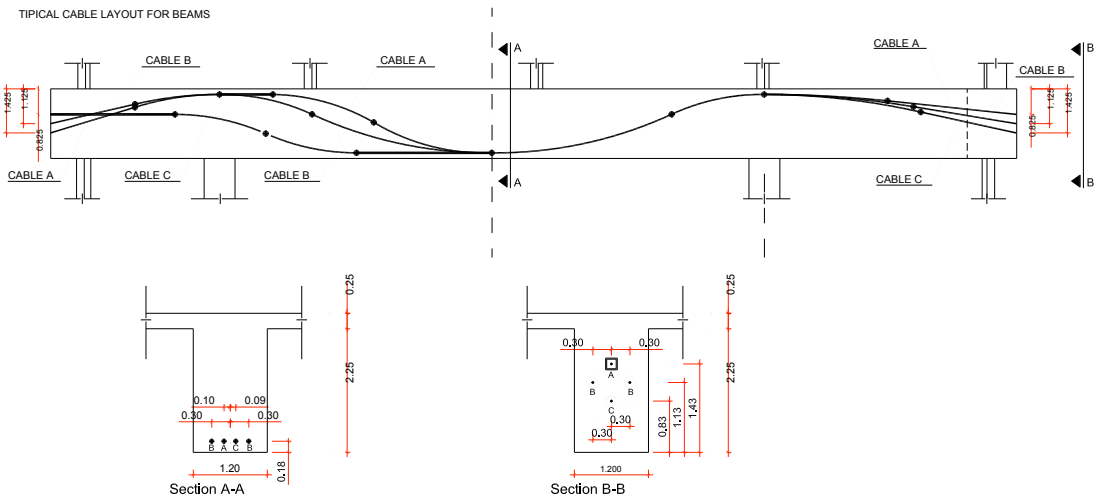


Figure 5.11: Prestress beams plant showing the circular columns that support them with a 18m main span



CABLE COMPOSITION:

		BEAMS		
		V4.1	V5.1	V6.1
CABLES	A	1 CABLE 7 Strands 0.6"	1 CABLE 10 Strands 0.6"	1 CABLE 7 Strands 0.6"
	B	2 CABLES 12 Strands 0.6"	2 CABLES 12 Strands 0.6"	2 CABLES 7 Strands 0.6"
	C	1 CABLE 10 Strands 0.6"	1 CABLE 12 Strands 0.6"	1 CABLE 7 Strands 0.6"

CONSTRUCTION PHASES:

	BEAMS		
PHASE	V4.1	V5.1	V6.1
1	CABLE B	CABLE B	CABLE B
2	CABLES A & C	CABLES A & C	CABLES A & C

Notes:

1st Phase - Prestress applied to cables B after construction of Floor 1
2nd Phase - Prestress applied to remaning cables after construction of Floor 4

Figure 5.12: Beams cable layout with construction phases indications

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5.2.5 Transfer grid "Lisbon St. Gabriel Tower"

- Designer: JSJ Consult/A2P Consult, Portugal, 1998
- Contractor: ENGIL SA/Contacto SA, Portugal, 1999/2000

St. Gabriel tower is a 25 elevated floors building, currently under construction in the Nations Park, at the oriental part of Lisbon where the 98 World Exposition took place. Composite floors, steel beams connected to a concrete slab, two important structural concrete cores and a mesh of steel columns, constitute the elevated structure.

For architectural reasons, steel columns are stopped at the intermediate level 20.00, where an impressive post-tensioned concrete transfer structure was adopted. The transfer floor is just supported at the main cores, with a central span of about 25 meters, extending approximately 15 meters in cantilever to one side of each core wall. Its geometry is ellipsoidal, vessel shaped, being composed by an orthogonal grid of post-tensioned concrete beams, directly supporting the gravity loads carried by the steel columns.

A maximum service load level of about $8 \times 25 = 200$ kN/m^2 is to be considered for the design of the transfer structure, imposing the adoption of a post-tensioned concrete solution as the only way to ensure adequate behaviour under service and ultimate conditions.

Post-tensioning was designed in order to approximately balance deflections due to permanent loads, allowing simultaneously a proper control of concrete stresses. In addition, prestressing steel provides the major part of the total tensile force needed at Ultimate Limit State, leading to simpler detailing and more economical construction procedures.

In order not to overstress the structure at intermediate stages, prestressing was applied in four steps, the first one after the execution of the grid beams, the second after the execution of the bottom and top slabs, and the two remaining phases, following the introduction of loads, during the elevated floors construction.

Figure 5.13: General view

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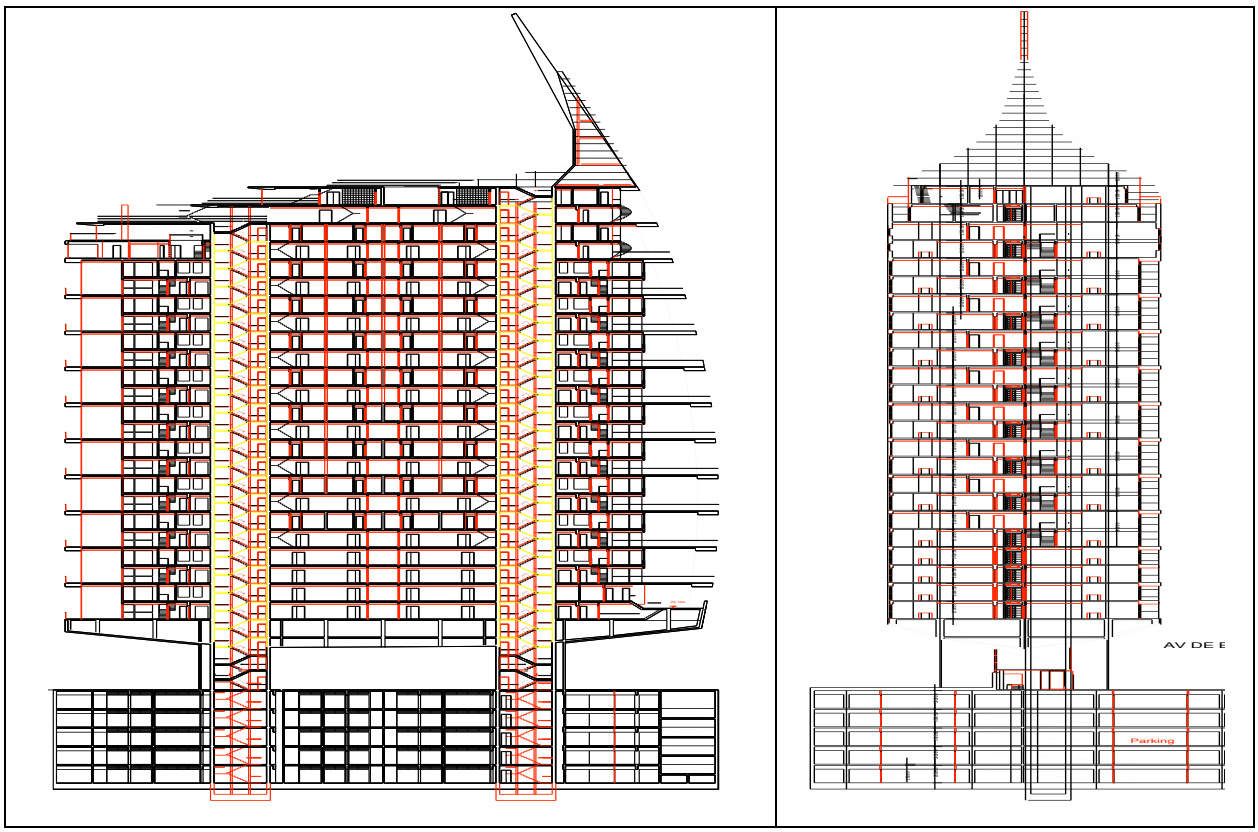

Figure 5.14: Longitudinal and transverse sections

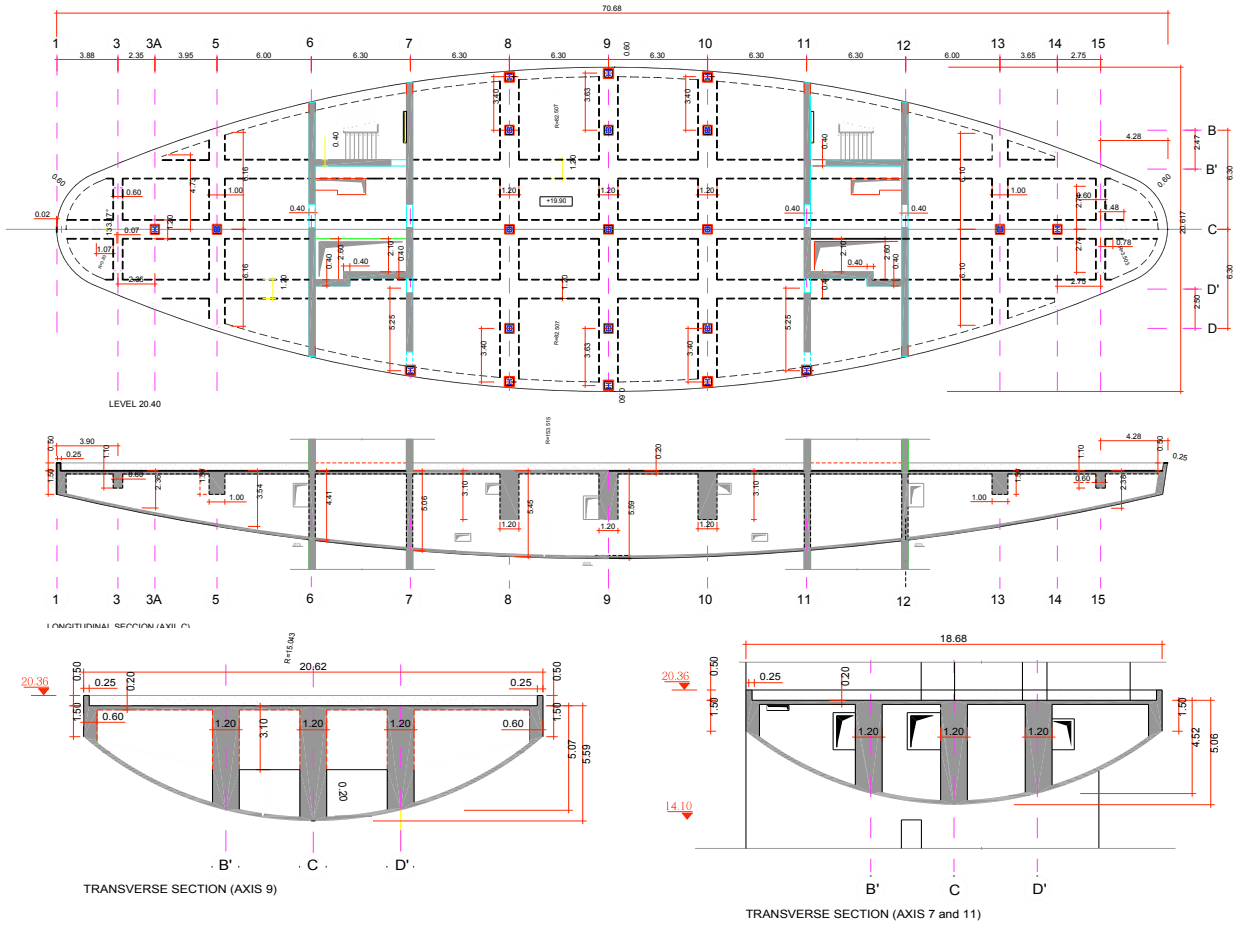

Figure 5.15: Transfer grid

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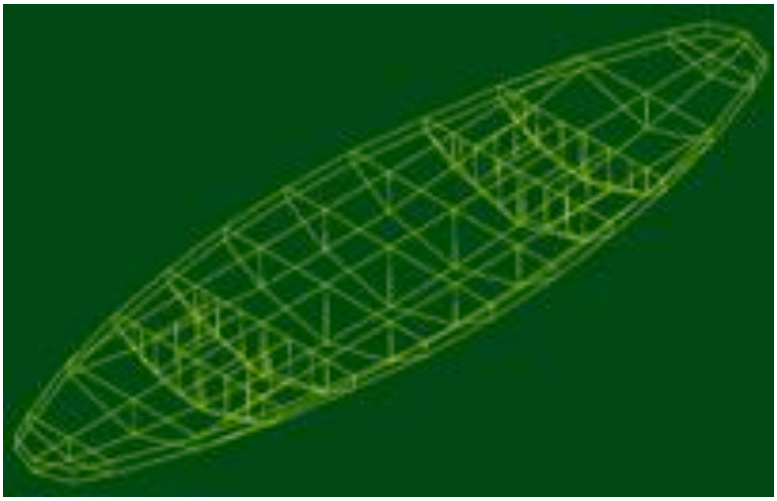
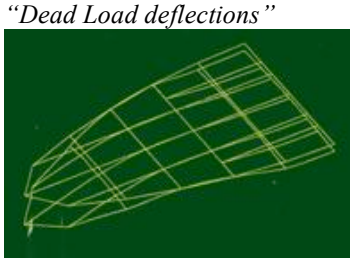
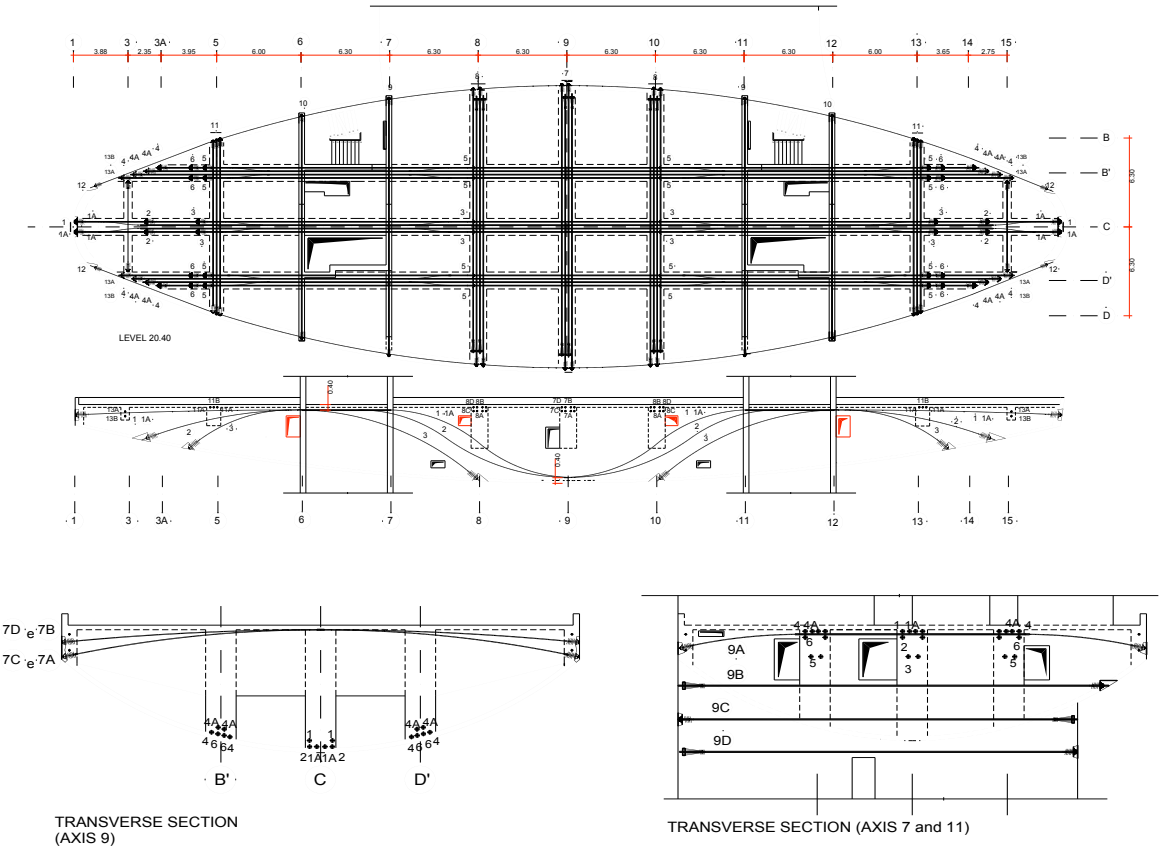


"Dead Load deflections"



"Prestress deflections"

Figure 5.16: Design model



PHASE	CABLE		Peff (kN)
I	1A	2 X 37 STRANDS (0.6")	10000
	4A	(2x) 2 X 37 STRANDS (0.6")	(2x) 10000
	11A	(2x) 2 X 22 STRANDS (0.6")	(2x) 6160
	8A	(2x) 2 X 19 STRANDS (0.6")	(2x) 5320
	7A	2 X 19 STRANDS (0.6")	5320
	12	(2x) 2 X 12 STRANDS (0.6")	(2x) 3360
	9	(2x) 4 X 12 STRANDS (0.6")	(2x) 6720
	10	(2x) 4 X 12 STRANDS (0.6")	(2x) 6720
	13A	(2x) 1 X 12 STRANDS (0.6")	(2x) 1680

PHASE	CABLE		Peff (kN)
II	2	2 X 31 STRANDS (0.6")	8380
	5	(4x) 2 X 31 STRANDS (0.6")	(4x) 8380
	11B	(2x) 1 X 22 STRANDS (0.6")	(2x) 3080
	8B	(2x) 2 X 12 STRANDS (0.6")	(2x) 3360
	7B	2 X 12 STRANDS (0.6")	3360
	13B	(2x) 1 X 12 STRANDS (0.6")	(2x) 1680
III	1	2 X 31 STRANDS (0.6")	8380
	4	(2x) 2 X 31 STRANDS (0.6")	(2x) 8380
	8C	(2x) 2 X 19 STRANDS (0.6")	(2x) 5320
	7C	2 X 19 STRANDS (0.6")	5320
IV	3	(2x) 2 X 37 STRANDS (0.6")	(2x) 10000
	6	(2x) 2 X 37 STRANDS (0.6")	(2x) 10000
	8D	(2x) 2 X 19 STRANDS (0.6")	(2x) 5320
	7D	2 X 19 STRANDS (0.6")	5320

PHASE I - AFTER EXECUTION OF TRANSFER GRID
PHASE II - AFTER EXECUTION OF LEVEL 4
PHASE III - AFTER EXECUTION OF LEVEL 10
PHASE IV - AFTER EXECUTION OF LEVEL 17

Figure 5.17: Prestressing layout

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Figure 5.18: General view during construction

Figure 5.19: General view of transfer floor

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5.2.6 Curved transfer beam "School Building-Auditorium Zug"

- Designer: DOMOSTATIK AG, Switzerland

As part of the construction of a new school, an auditorium with the library on top of it was to be built. Because of the layout in one direction of the whole school complex with the individual buildings, covered over connecting ways and the external facilities, wall sections with different orientations resulted. The individual walls form the exterior of the building. The task was to connect the non-parallel walls in a spatial context. For the auditorium, on the other hand, a circular arch section was planned. The curvature was adopted from the external walls for the further columns. The columns for the library situated over the auditorium are arranged along the curved axes. The loads of in total three floors had to be supported over the column-free auditorium. The spans over the auditorium were up to 16 m. Supporting the curved section, the supporting beams placed under the columns are also curved, and are visible for the audience in the room.

The task was to control the high combined loads in the beams, which involve bending, torsion and shear, with the choice of appropriate prestressing. The prestressing is aimed to help reduce the required ordinary reinforcement consisting of longitudinal bars and stirrups.

The curved beams are arranged as a single span system resting on the side walls. The deck slab is monolithically connected to the beams and serves to continuously resist the torsional moment. The bending moments in the beams are resisted by the slab-beam cross section. The tensile force is resisted by the concentrated prestressing reinforcement, which thanks to the anchorage of the slab is lead exactly up to the bearings. Because the deviation forces from the prestressing resist external loads, this reduces the shear and torsional effects. The two prestressing cables installed in the beams each with a prestressing force of about 2,400KN considerably reduce the actions in the beams, so that the necessary ordinary reinforcement can be easily incorporated.

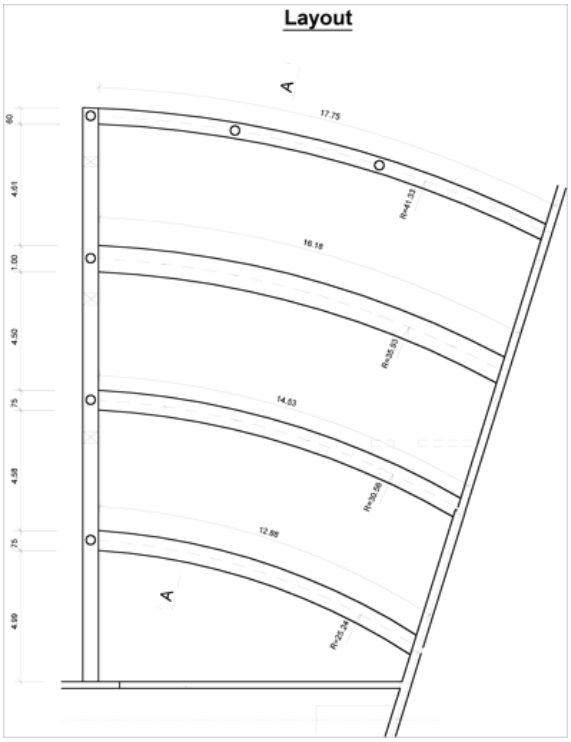

Figure 5.20: Plan outline with details of the curved beams which are monolithic with the slab

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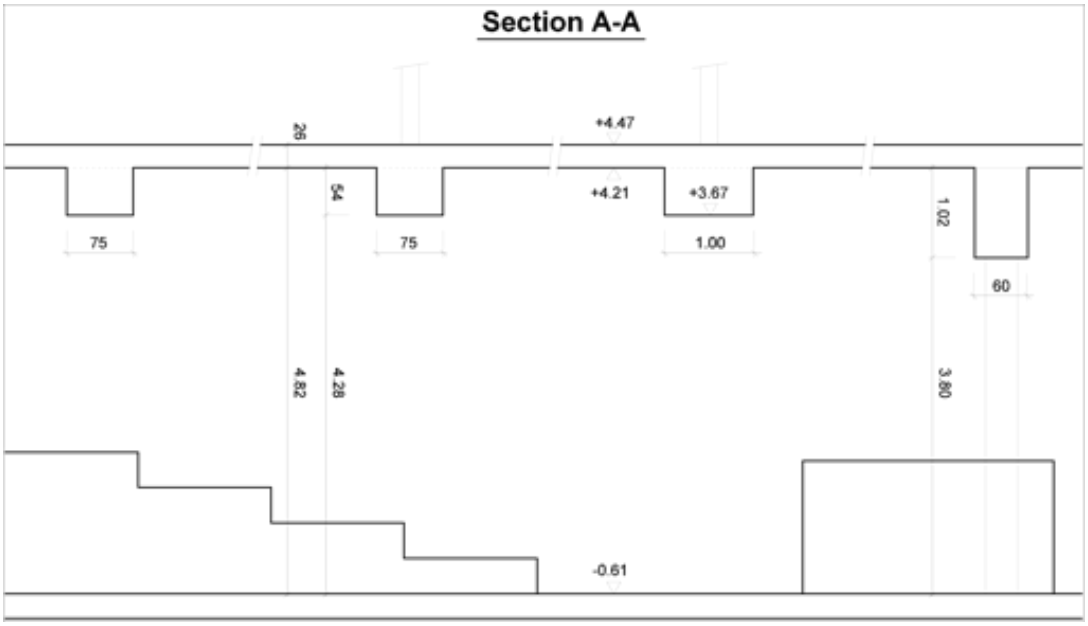

Figure 5.21: Section through the auditorium

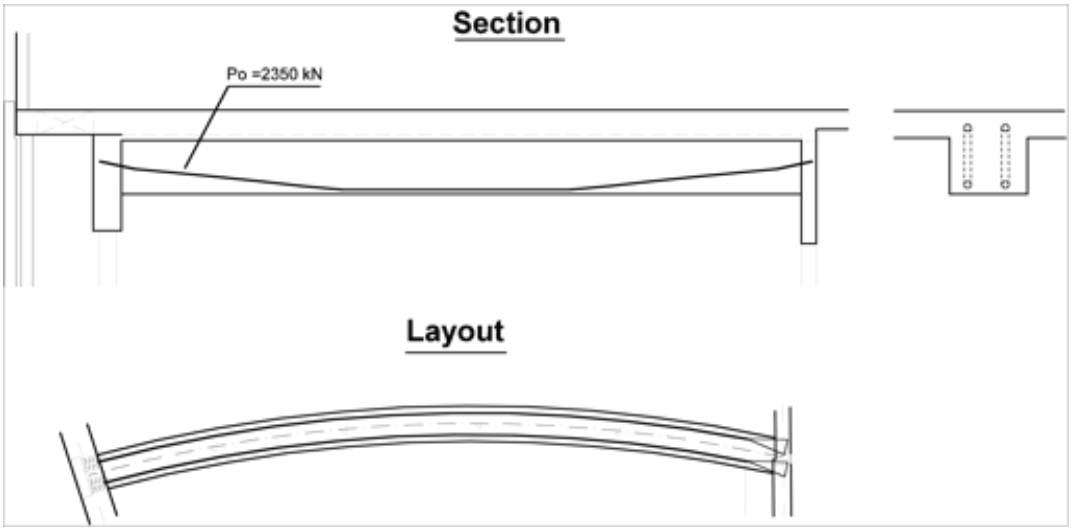

Figure 5.22: Arrangement of the prestressing cables in the beams

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Figure 5.23: Plan view of the beams during the construction phase

Figure 5.24: Installation of the reinforcement for the beam

Figure 5.25: Adjustment of the stirrup reinforcement for an opening in the side of the deck slab

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6 Prefabricated post-tensioned solutions

6.1 Conceptual design

The advantages of using both prefabrication and post-tensioning can, in some particular cases, result in very effective solutions.

This applies for example to situations where the structure has to be constructed at a considerable height above the ground level, and the falsework makes up a large part of the total construction cost. In such cases, a composite construction with prestressed prefabricated elements and *in-situ* concrete, possibly also post-tensioned, can considerably simplify the construction process, thus reducing the global cost. This aspect can be even more important if supports, definitive ones or temporary ones during construction, can only be placed at large distances.

The main concept is to use prefabricated elements as supports for the concreting of the remaining *in-situ* structural elements. During the erection works the prefabricated elements are in general stored at temporary areas, as close as possible to the in-situ construction site. To support the dead load and other actions during construction, prefabricated elements have to be at least partially pre-tensioned for the construction stage. The extra post-tensioning strands can be pushed in afterward into prearranged empty sheaths in the prefabricated elements.

6.2 Applications

6.2.1 Prefabricated beams used as scaffolding "Printers Bucher-Luzern"

- Designer: DOMOSTATIK AG, Switzerland

To extend the premises of an existing firm of printers, a roof was to be constructed over the delivery yard. The previous and future utilization resulted in various boundary conditions, which had to be considered in the design and construction work. The level of the upper surface was fixed by the need to connect the slab to the existing building. The level of the lower surface should be as high as possible, to allow an adequate height for vehicles. The previous and future delivery was to be made with an articulated lorry, whose manoeuvrability necessitates as few columns as possible. Further, the delivery work had to be maintained even during the construction period. From the above conditions a span of up to 28.0 m resulted. In order to keep the thickness of the slab as small as possible prestressed beams were required. Maintaining delivery work during the construction period was possible only by using prefabricated beams.

The construction area was divided into a grid with small spacing between beams, so that the loads carried by each beam were kept to a minimum. In some areas the concrete beams could be constructed in situ. Due to this constellation a structural system consisting of prefabricated beams with a cast-in-place concrete slab was worked out. The construction work was carried out in such a way that the in situ concrete area was scaffolded and completely reinforced, so that the prefabricated beams could be immediately connected in the in situ concrete area. The prefabricated beams were stored during the erection work on a prepared support and in a temporary storage area near to the in situ construction area. To support the dead load of the beams and the slab during the construction period the beams had to be partially prestressed. This was done by prestressing one of the two cables in the beams. This prestressing cable is coupled at the interface to the in situ concrete and stressed at the side of the in situ concrete beams. The

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second prestressing cable also passes through the in situ concrete area and is pushed into an empty sheath in the prefabricated beam. The prestressing is carried out after the beams and the slab have been concreted and have hardened. The cross section of the prefabricated beam is such that the bond is effective between it and the slab concreted on top of it. The lateral consoles at the base of the beam cross section serve to support the scaffolding elements for the cast-in-place concrete slab between the beams.

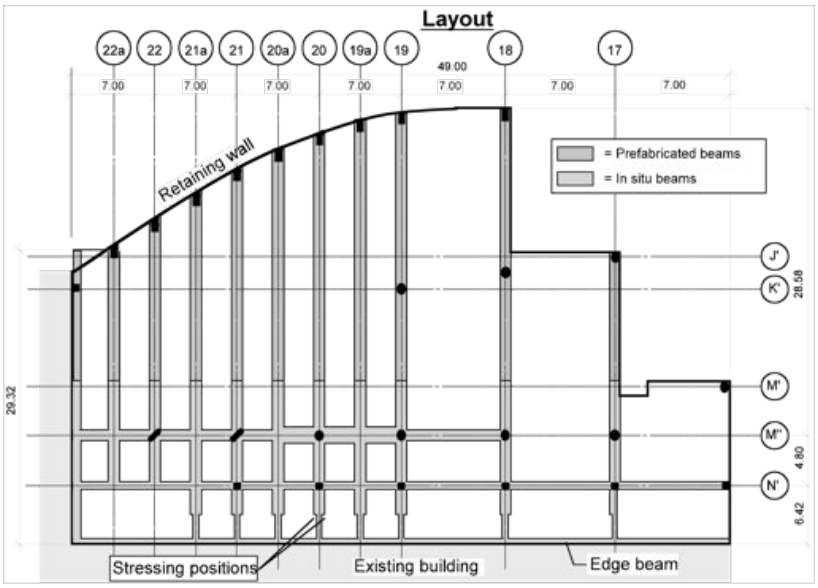

Figure 6.1: Plan outline with details of the beams for the deck system

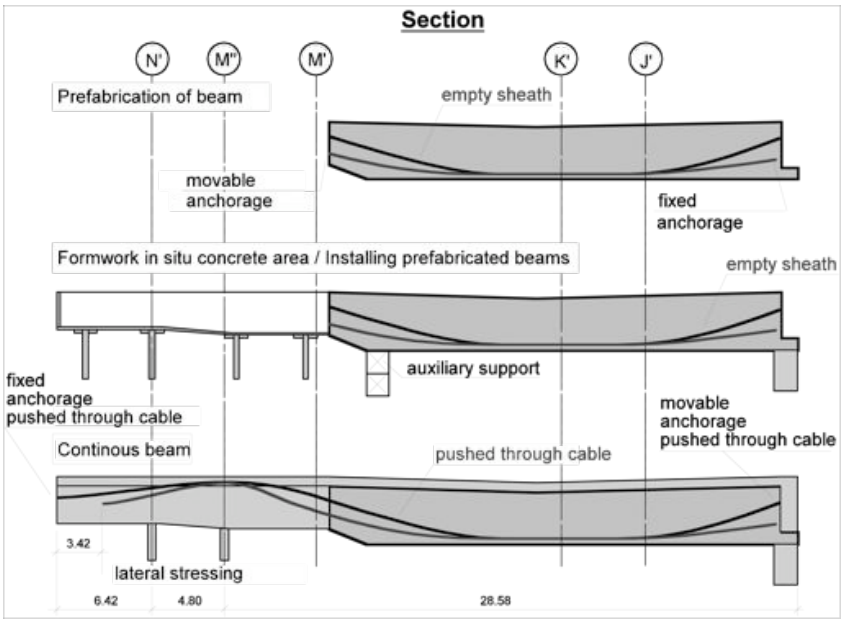

Figure 6.2: Erection procedure for the prefabricated beams and prestressing arrangement

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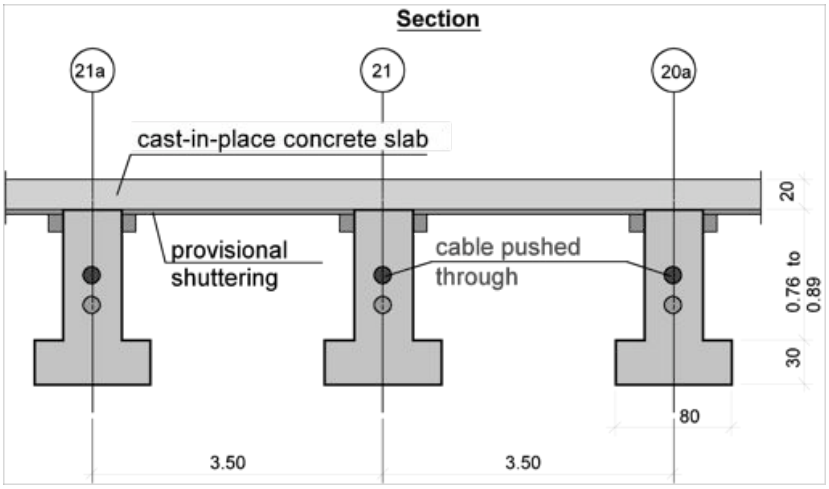

Figure 6.3: Prefabricated beams as support for the scaffolding for the deck formwork

Figure 6.4: Overview of the building site showing areas in situ concrete and the already erected prefabricated beams

Figure 6.5: Detail of prefabricated beams and prestressing cables during erection stage

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6.2.2 Platform for a Heliport "KHIB-Ibbenburen"

- Designer: DOMOSTATIK AG, Switzerland

A platform for a helicopter was to be built on the roof of an existing hospital. The reinforced concrete platform of diameter of approx. 26.0 m had to be constructed roughly 20 m above ground level. The width of the building at this point was around 15.0 m. As a result, the platform extends over the outside edge of the building on both sides by 5.50 m. A reinforced concrete grillage construction was first proposed, whose beams intersect at the axes of the existing structure. The lower part of the formwork for this complex cast-in-place concrete beam construction was constructed on falsework placed on the ground. The expenditure for the falsework made up a large part of the total costs. In order to simplify the construction process and minimize the costs, a composite construction of prestressed prefabricated and prestressed cast-in-place concrete was proposed.

In this way the dimensions of the individual structural components can be reduced.

Dispensing with the proposed grillage system, prefabricated beams in one direction only – cantilevering from the edge of the building – were planned. The prefabricated beams serve as supports for the prefabricated slab elements of the full slab in between the beams. In addition, the slender slab construction was prestressed at right angles to the prefabricated beams, in order to control the cantilevering in the area of the axes 26, 27 and 32, 33. With this solution the complicated falsework up to a height of about 20m could be avoided. Only the light protection and erection scaffolding was needed.

The prefabricated beams were fitted with two tendons each with 4 strands. To withstand the dead weight of the beam during erection and of the slab, one tendon had to be tensioned at the factory. All six beams were positioned within 4 hours using a truck crane and fixed to the columns by means of the starter bars. The vertical duct in the beam to receive the starter bars was grouted after erection to fix the beam. Then the erection of the prefabricated slab elements between the beams was carried out. Some areas had to be augmented locally with conventional formwork. Some single strands were placed over the slab elements at right angles to the beams. After completing the concrete topping the remainder of the tendons in the beams and the slab were tensioned. The edge beam running around the perimeter was concreted in situ. It serves to support the guard rail and at the same time closes off all prestressing recesses in the slab and the beam.

Figure 6.6: Over view of the whole construction

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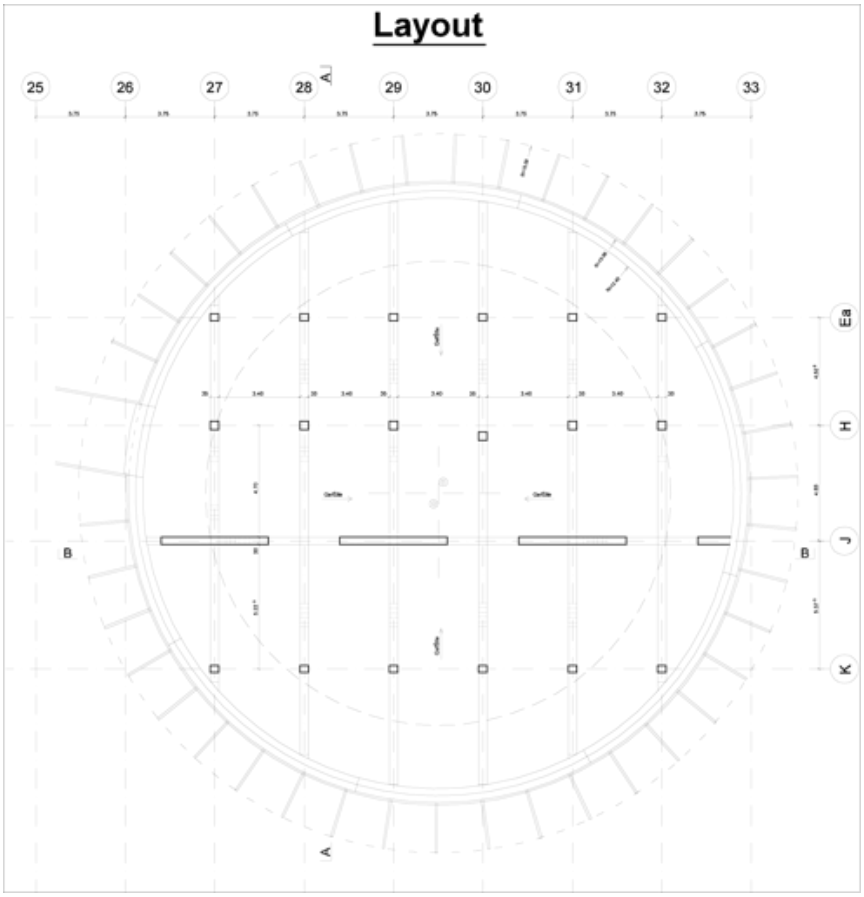

Figure 6.7: Plan outline of the platform with the prefabricated beams

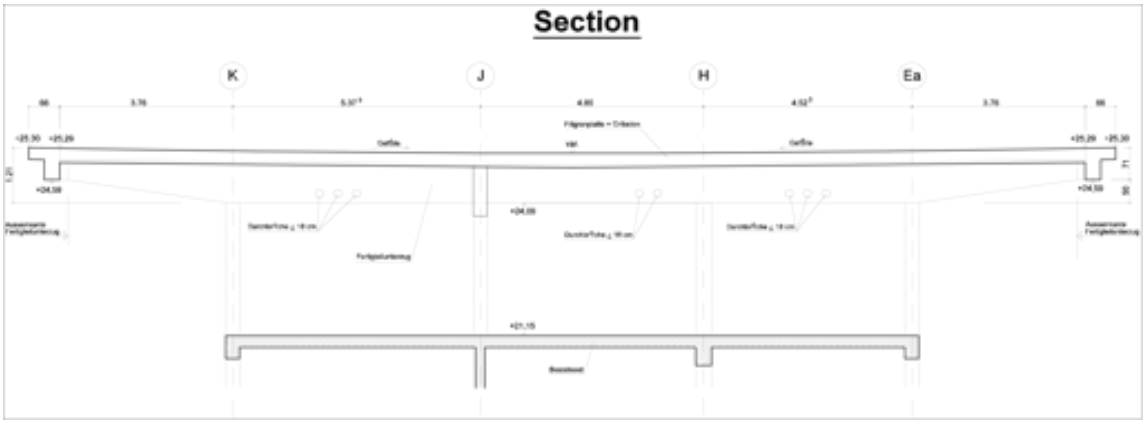

Figure 6.8: Prefabricated beam with cantilever part

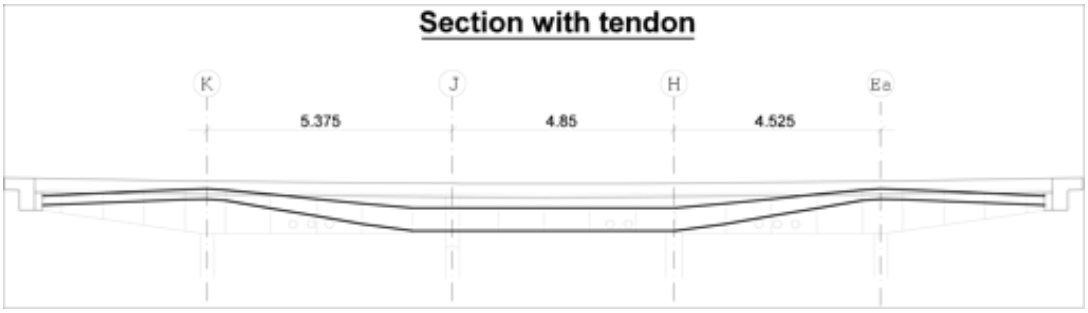

Figure 6.9: Arrangement of the prestressing in the beams

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Figure 6.10: Construction stage prefabricated beams placed and fixed

Figure 6.11: Cantilevering prefabricated beams during construction

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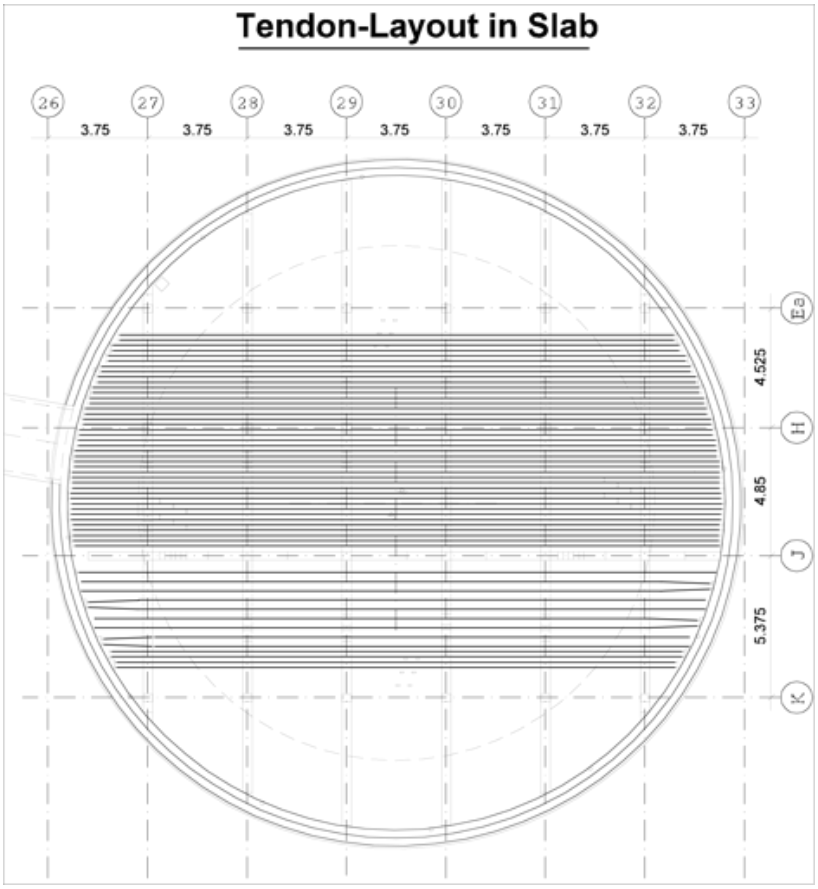

Figure 6.12: Layout of the tendon for the slab between the beams

Figure 6.13: Completed structure: view from underneath

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