POST-TENSIONED SLABS

Fundamentals of the design process

Ultimate limit state

Serviceability limit state

Detailed design aspects

Construction Procedures

Preliminary Design

Execution of the calculations

Completed structures

4.2

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Foreword

With the publication of this technical report, VSL INTERNATIONAL LTD is pleased to make a contribution to the development of Civil Engineering.

The research work carried out throughout the world in the field of post-tensioned slab structures and the associated practical experience have been reviewed and analysed in order to etablish the recommendations and guidelines set out in this report. The document is intended primarily for design engineers, but we shall be very pleased if it is also of use to contractors and clients. Through our representatives we offer to interested parties throughout the world our assistance end support in the planning, design and construction of posttensioned buildings in general and post-tensioned slabs in particular.

I would like to thank the authors and all those who in some way have made a contribution to the realization of this report for their excellent work. My special thanks are due to Professor Dr B. Thürlimann of the Swiss Federal Institute of Technology (ETH) Zürich and his colleagues, who were good enough to reed through and critically appraise the manuscript.

Berne, January 1985

Hans Georg Elsaesser
Chairman of the Board and President
If VSLINTERNATIONALLTD

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1. Introduction

1.1. General

Post-tensioned construction has for many years occupied a very important position, especially in the construction of bridges and storage tanks. The reason for this lies in its decisive technical and economical advantages.

The most important advantages offered by post-tensioning may be briefly recalled here:

- By comparison with reinforced concrete, a considerable saving in concrete and steel since, due to the working of the entire concrete cross-section more slender designs are possible.
- Smaller deflections than with steel and reinforced concrete.
- Good crack behaviour and therefore permanent protection of the steel against corrosion.
- Almost unchanged serviceability even after considerable overload, since temporary cracks close again after the overload has disappeared.
- High fatigue strength, since the amplitude of the stress changes in the prestressing steel under alternating loads are quite small.

For the above reasons post-tensioned construction has also come to be used in many situations in buildings (see Fig 1).

The objective of the present report is to summarize the experience available today in the field of post-tensioning in building construction and in particular to discuss the design and construction of post-tensioned slab structures, especially post-tensioned flat slabs*. A detailed explanation will be given of the checksto be carried out, the aspects to be considered in the design and the construction procedures and sequences of a post-tensioned slab. The execution of the design will be explained with reference to an example. In addition, already built structures will be described. In all the chapters, both bonded and unbundled post-tensicmng will be dealt with.

In addition to the already mentioned general features of post-tensioned construction, the following advantages of post-tensioned slabs over reinforced concrete slabs may be listed:

- More economical structures resulting from the use of prestressing steels with a very high tensile strength instead of normal reinforcing steels.
- larger spans and greater slenderness (see Fig. 2). The latter results in reduced dead load, which also has a beneficial effect upon the columns and foundations and reduces the overall height of buildings or enables additional floors to be incorporated in buildings of a given height.
- Under permanent load, very good behavior in respect of deflectons and crackIng.
- Higher punching shear strength obtainable by appropriate layout of tendons
- Considerable reduction In construction time as a result of earlier striking of formwork real slabs.

* For definitions and symbols refer to appendix 1.

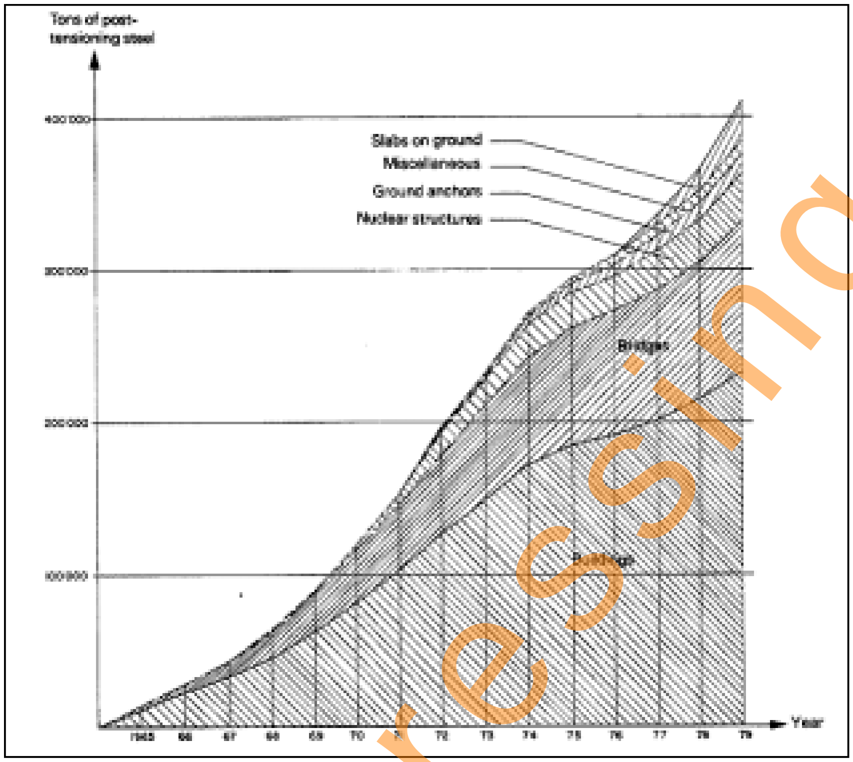


Figure 1. Consumption of prestressing steel in the USA (cumulative curves)

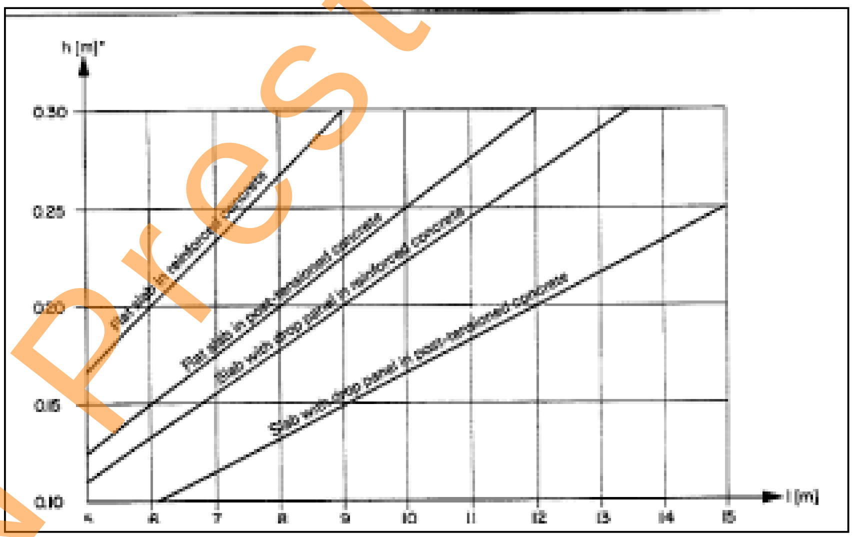


Figure 2: Slab thicknesses as a function of span lengths (recommended limis slendernesses)

1.2. Historical review

Although some post-tensioned slab structures had been constructed in Europe quite early on, the real development took place in the USA and Australia. The first post-tensioned slabs were erected in the USA In 1955, already using unbonded post-tensioning. In the succeeding years numerous post-tensioned slabs were designed and constructed in connection with the lift slab method. Post-tensionmg enabled the lifting weight to be reduced and the deflection and cracking performance to be improved. Attempts were made to improve knowledge In depth by theoretical studies and experiments on post-tensioned plates (see Chapter 2.2). Joint efforts by researchers, design engineers and prestressing firms resulted in corresponding standards and recommendations and assisted in promoting the widespread use of this form of construction in the USA and Australia. To date, in the USA alone, more than 50 million m^2 of slabs have been post tensioned.

In Europe. renewed interest in this form of construction was again exhibited in the early seventies Some constructions were completed at that time in Great Britain, the Netherlands and Switzerland.

Intensive research work, especially in Switzerland, the Netherlands and Denmark and more recently also in the Federal Republic of Germany have expanded the knowledge available on the behaviour of such structures These studies form the basis for standards, now in existence or in preparation in some countries. From purely empirical beginnings, a technically reliable and economical form of constructon has arisen over the years as a result of the efforts of many participants. Thus the method is now also fully recognized in Europe and has already found considerable spreading various countries (in the Netherlands, in Great Britain and in Switzerland for example).

1.3. Post-tensioning with or without bonding of tendons

1.3.1. Bonded post-tensioning

As is well-known, in this method of post-tensioning the prestressing steel is placed In ducts, and after stressing is bonded to the surrounding concrete by grouting with cement suspension. Round corrugated ducts are normally used. For the relatively thin floor slabs of buildings, the reduction in the possible eccentricity of the prestressing steel with this arrangement is, however, too large, in particular at cross-over points, and for this reason flat ducts have become common (see also Fig. 6). They normally contain tendons comprising four strands of nominal diameter 13 mm (0.5"), which have proved to be logical for constructional reasons.

1.32. Unbonded post-tensioning

In the early stages of development of post-tensioned concrete in Europe, post-tensioning without bond was also used to some extent (for example in 1936/37 in a bridge constructed in Aue/Saxony [D] according to the Dischinger patent or in 1948 for the Meuse, Bridge at Sclayn [B] designed by Magnel). After a period without any substantial applications, some important structures have again been built with unbonded post-tensioning in recent years.
In the first applications in building work in the USA, the prestressing steel was grassed and wrapped in wrapping paper, to facilitate its longitudinal movement during stressing During the last few years, howeverthe method described below for producing the sheathing has generally become common.
The strand is first given a continuous film of permanent corrosion preventing grease in a continuous operation, either at the manufacturer's works or at the prestressing firm. A plastics tube of polyethylene or polypropylene of at least 1 mm wall thickness is then extruded over this (Fig. 3 and 4). The plastics tube forms the primary and the grease the secondary corrosion protection.

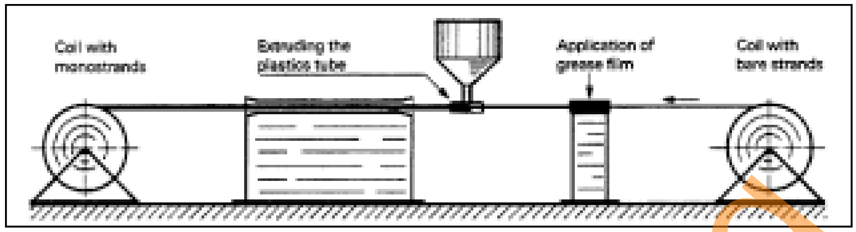


Figure 3: Diagrammatic illustration of the extrusion process

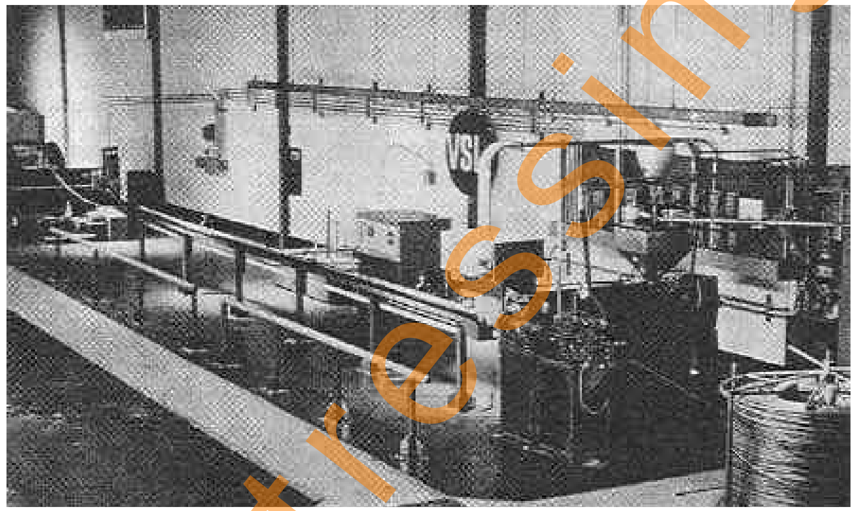


Figure 4: Extrusion plant

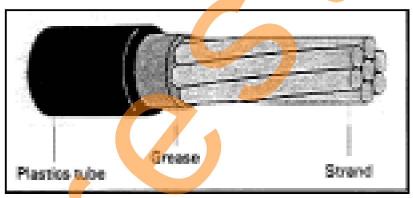


Figure 5: Structure of a plastics-sheathed, greased strand (monostrantd)

Strands sheathed in this manner are known as monostrands (Fig. 5). The nominal diameter of the strands used is 13 mm (0.5") and 15 mm (0.6"); the latter have come to be used more often in recent years.

1.3.3. Bonded or unbonded?

This question was and still is frequently the subject of serious discussions. The subject will not be discussed in detail here, but instead only the most important arguments far and against will be listed:

Arguments in favour of post-tensioning **without** bonding:

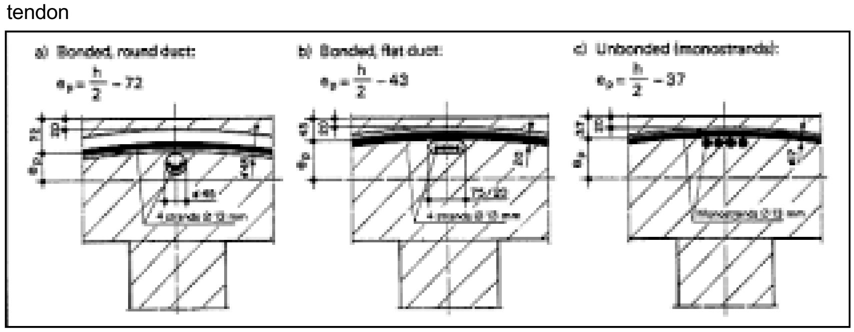
- Maximum possible tendon eccentricities, since tendon diameters are minimal; of special importance in thin slabs (see Fig 6).
- Prestressing steel protected against corrosion ex works.
- Simple and rapid placing of tendons.
- Very low losses of prestressing force due to friction.
- Grouting operation is eliminated.
- In general more economical.

Arguments for post-tensioning **with** bonding:

- Larger ultimate moment.
- Local failure of a tendon (due to fire, explosion, earthquakes etc.) has only limited effects

Whereas in the USA post-tensioning without bonding is used almost exclusively, bonding is deliberately employed in Australia.

Figure 6 Comparison between the eccentricities that can be attained with various types of tendon



Among the arguments for bonded post-tensioning, the better performance of the slabs in the failure condition is frequently emphasized. It has, however, been demonstrated that equally good structures can be achieved in unbonded post-tensioning by suitable design and detailing.
It is not the intention of the present report to express a preference for one type of post-tensioning or the other. Il is always possible that local circumstances or limiting engineering conditions (such as standards) may become the decisive factor in the choice. Since, however, there are reasons for assuming that the reader will be less familiar with undonded post-tensioning, this form of construction is dealt with somewhat more thoroughly below.

1.4. Typical applications of post-tensioned slabs

As already mentioned, this report is concerned exclusively with post-tensioned slab structures. Nevertheless, it may be pointed out here that post-tensioning can also be of economic interest in the following components of a multi-storey building:

- Foundation slabs (Fig 7).
- Cantilevered structures, such as overhanging buildings (Fig 8).
- Facade elements of large area; here light post-tensioning is a simple method of preventing cracks (Fig. 9).
- Main beams in the form of girders, lattice girders or north-light roofs (Fig. 10 and 11).

Typical applications for post-tensioned slabs may be found in the frames or skeletons for office buildings, mule-storey car parks, schools, warehouses etc. and also in multi-storey flats where, for reasons of internal space, frame construction has been selected (Fig. 12 to 15).
What are the types of slab system used?

- For spans of 7 to 12 m, and live loads up to approx. 5 kN/m^2, flat slabs (Fig. 16) or slabs with shallow main beams running in one direction (Fig. 17) without column head drops or flares are usually selected.
- For larger spans and live loads, flat slabs with column head drops or flares (Fig 18), slabs with main beams in both directions (Fig 19) or waffle slabs (Fig 20) are used.

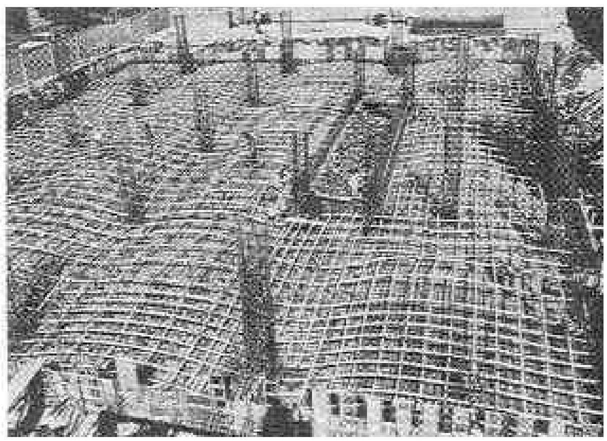


Figure 7: Post-tensioned foundation slab

Figure 9: Post-tensioned facade elements

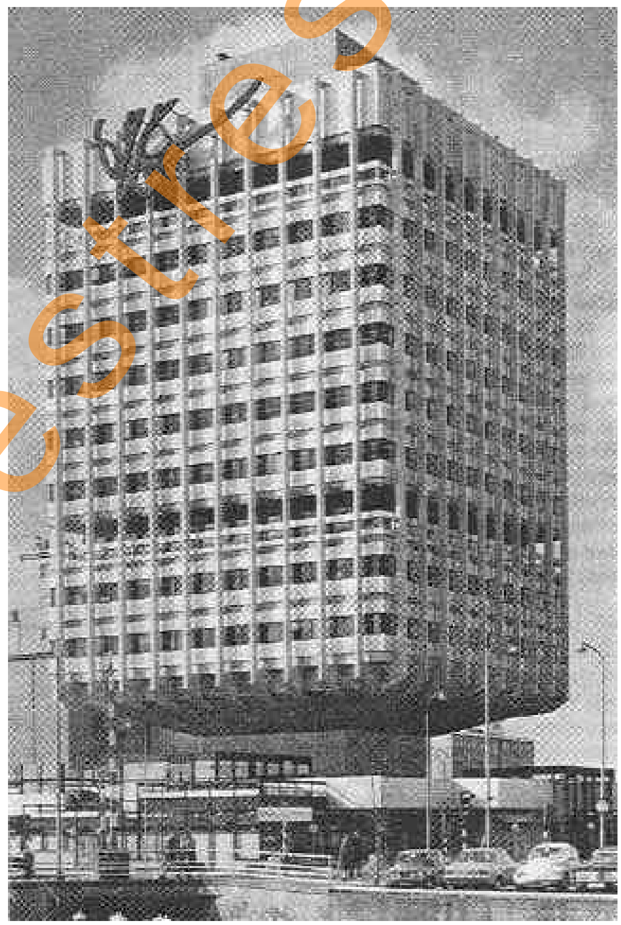


Figure 8: Post-tensioned cantilevered building

Figure 10: Post-tensioned main beams

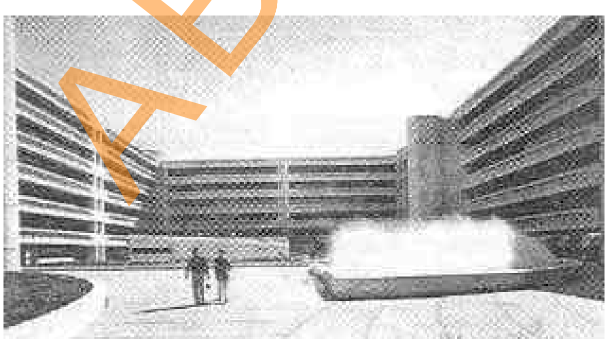
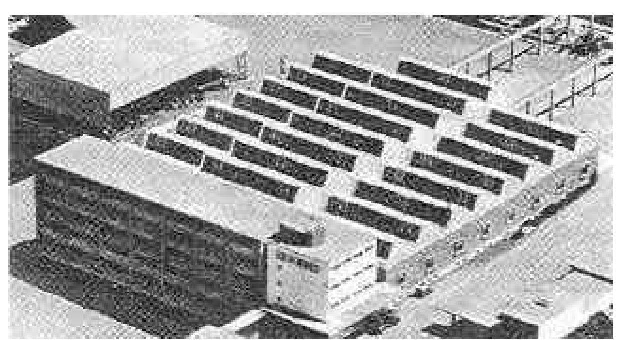


Figure 11: Post-tensioned north-light roofs



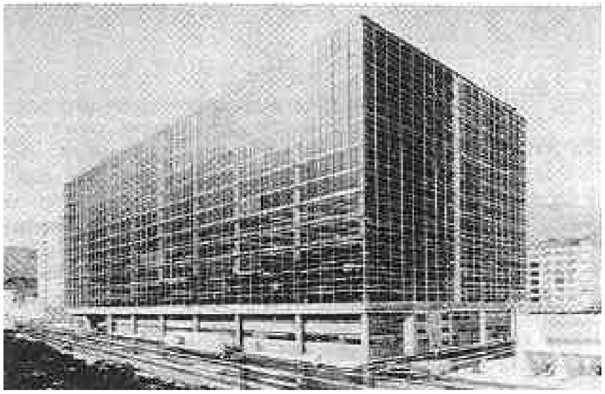


Figure 12: Office and factory building


Figure 13: Multi-storey car park


Figure 14: School

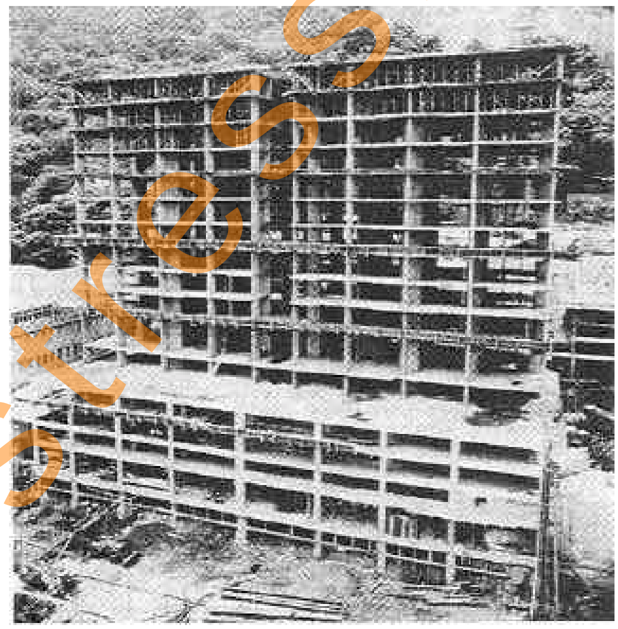


Figure 15: Multi-storey flats

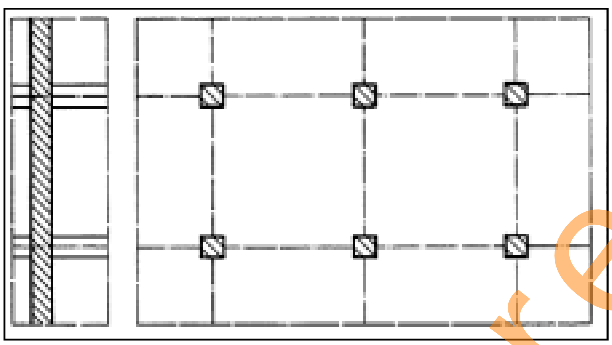


Figure 16: Flat Slab

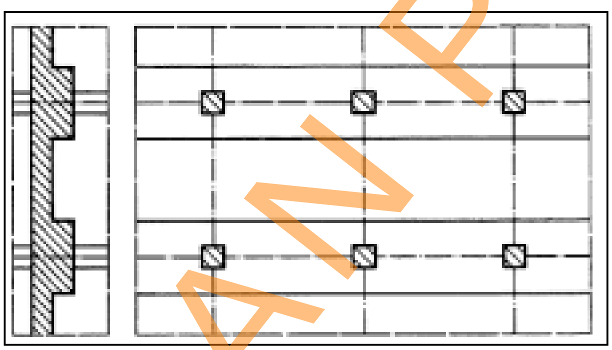


Figure 17: Slab with main beams in one direction

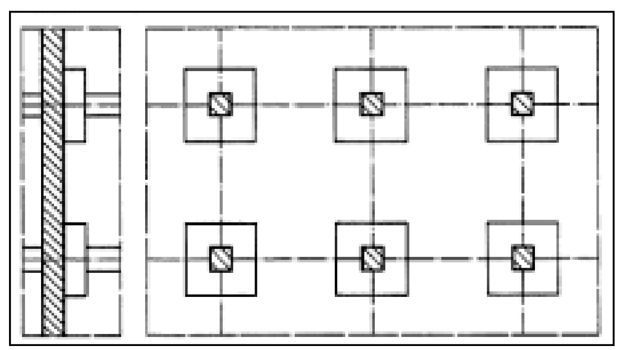


Figure 18: Flat slab with column head drops

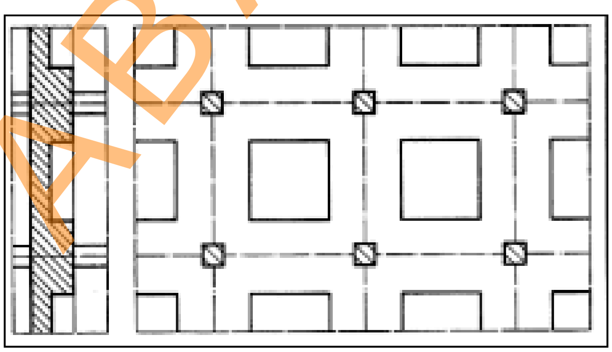


Figure 19: Slab with main beams in both directions

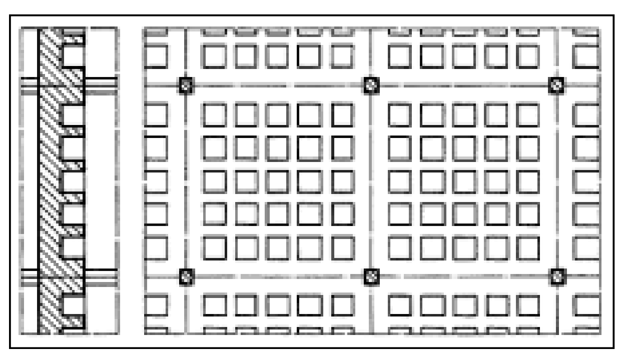


Figure 20: Waffle slab

2. Fundamentals of the design process

2.1. General

The objective of calculations and detailed design is to dimension a structure so that it will satisfactorily undertake the function for which it is intended in the service state, will possess the required safety against failure, and will be economical to construct and maintain. Recent specifications therefore demand a design for the «ultimate» and «serviceability» limit states.

Ultimate limit state: This occurs when the ultimate load is reached; this load may be limited by yielding of the steel, compression failure of the concrete, instability of the structure or material fatigue The ultimate load should be determined by calculation as accurately as possible, since the ultimate limit state is usually the determining criterion

Serviceability limit state: Here rules must be complied with, which limit cracking, deflections and vibrations so that the normal use of a structure Is assured. The rules should also result in satisfactory fatigue strength.

The calculation guidelines given in the following chapters are based upon this concept They can be used for flat slabs with or without column head drops or flares. They can be converted appropriately also for slabs with main beams, waffle slabs etc.

2.2. Research

The use of post-tensioned concrete and thus also its theoretical and experimental development goes back to the last century. From the start, both post-tensioned beam and slab structures were investigated. No independent research has therefore been carried out for slabs with bonded pos-tensioning. Slabs with unbonded post-tensioning, on the other hand, have been thoroughly researched, especially since the introduction of monostrands.

The first experiments on unhonded post-tensioned single-span and multi-span flat slabs were carried out in the fifties [1], [2]. They were followed, after the introduction of monostrands, by systematic investigations into the load-bearing performance of slabs with unbonded post-tensioning [3], [4], [5], [6], [7], [8], [9], [10] The results of these investigations were to some extent embodied in the American, British, Swiss and German, standard [11], [12], [13], [14], [15] and in the FIP recommendations [16].

Various investigations into beam structures are also worthy of mention in regard to the development of unbonded post-tensioning [17], [18], [19], [20],[21], [22], [23].

The majority of the publications listed are concerned predominantly with bending behaviour. Shear behaviour and in particular punching shear in flat slabs has also been thoroughly researched A summary of punching shear investigations into normally reinforced slabs will be found in [24]. The influence of post-tensioning on punching shear behaviour has in recent years been the subject of various experimental and theoretical investigations [7], [25], [26], [27]. Other research work relates to the fire resistance of post-tensioned structures, including bonded and unbonded post-tensioned slabs Information on this field will be found, for example, in [28] and [29].

In slabs with unbonded post-tensioning, the protection of the tendons against corrosion is of extreme importance. Extensive research has therefore also been carried out in this field [30].

2.3. Standards

Bonded post-tensioned slabs can be designed with regard to the specifications on post-tensioned concrete structures that exist in almost all countries.

For unbonded post-tensioned slabs, on the other hand, only very few specifications and recommendations at present exist [12], [13], [15]. Appropriate regulations are in course of preparation in various countries. Where no corresponding national standards are in existence yet, the FIP recommendations [16] may be applied. Appendix 2 gives a summary of some important specifications, either already in existence or in preparation, on slabs with unbonded post-tensioning.

3. Ultimate limit state

3.1. Flexure

3.1.1. General principles of calculation

Bonded and unbonded post-tensioned slabs can be designed according to the known methods of the theories of elasticity and plasticity in an analogous manner to ordinarily reinforced slabs [31], [32], [33].

A distinction Is made between the following methods:

A. Calculation of moments and shear forces according to the theory of elastimry; the sections are designed for ultimate load.

B. Calculation and design according to the theory of plasticity.

Method A

In this method, still frequently chosen today, moments and shear forces resulting from applied loads are calculated according to the elastic theory for thin plates by the method of equivalent frames, by the beam method or by numerical methods (finite differences,finite elements).

The prestress should not be considered as an applied load. It should intentionally be taken into account only in the determination of the ultimate strength. No moments and shear forces due to prestress and therefore also no secondary moments should be calculated.

The moments and shear forces due to applied loads multiplied by the load factor must be smaller at every section than the ultimate strength divided by the cross-section factor.

The ultimate limit state condition to be met may therefore be expressed as follows [34]:

$$S \cdot \gamma_f \leq \frac{R}{\gamma_m} \qquad (3.1.)$$

This apparently simple and frequently encoutered procedure is not without its problems. Care should be taken to ensure that both flexure and torsion are allowed for at all sections (and not only the section of maximum loading). It carefully applied this method, which is similar to the **static method** of the theory of plasticity, gives an ultimate load which lies on the sate side.

In certain countries, the forces resulting from the curvature of prestressing tendons (transverse components) are also treated as applied loads. This is not advisable for the ultimate load calculation, since in slabs the determining of the secondary moment and therefore a correct ultimate load calculation is difficult.

The consideration of transverse components does however illustrate very well the effect of prestressing in service state. It is therefore highly suitable in the form of the load balancing method proposed by T.Y. Lin [35] for calculating the deflections (see Chapter 4.2).

Method B

In practice, the theory of plasticity, is being increasingly used for calculation and design The following explanations show how its application to flat slabs leads to a stole ultimate load calculation which will be easily understood by the reader.

The condition to be fulfilled at failure here is:

$$\frac{(g+q)_u}{g+q} \geq \gamma \qquad (3.2.)$$

where $\gamma = \gamma_f \cdot \gamma_m$

The ultimate design loading $(g+q)_u$ divided by the service loading $(g+q)$ must correspond to a value at least equal to the safety factor y.

The simplest way of determining the ultimate design loading $(g+q)_u$ is by the **kinematic method,** which provides an **upper boundary for the ultimate load.** The mechanism to be chosen is that which leads to the lowest load.

Fig. 21 and 22 illustrate mechanisms for an internal span. In flat slabs with usual column dimensions (ξ>0.06) the ultimate load can be determined to a high degree of accuracy by the line mechanisms ① or ② (yield lines 1-1 or 2-2 respectively). Contrary to Fig. 21, the negative yield line is assumed for purposes of approximation to coincide with the line connecting the column axes (Fig. 23), although this is kinematically incompatible. In the region of the column, a portion of the internal work is thereby neglected, which leads to the result that the load calculated in this way lies very close to the ultimate load or below it.

On the assumption of uniformly distributed top and bottom reinforcement, the ultimate design loads of the various mechanisms are compared in Fig. 24.

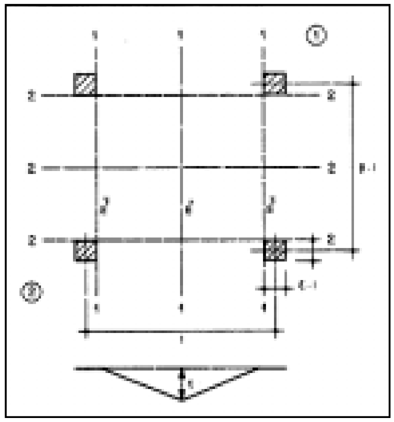


Figure 21: Line mecanisms

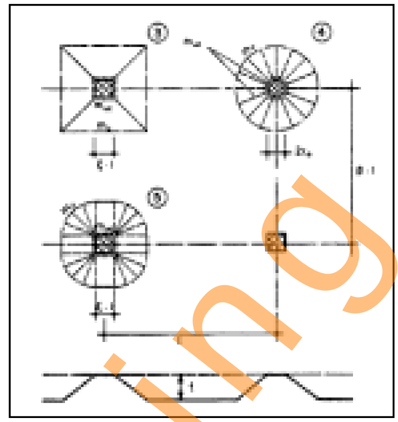


Figure 22: Fan mecanisms

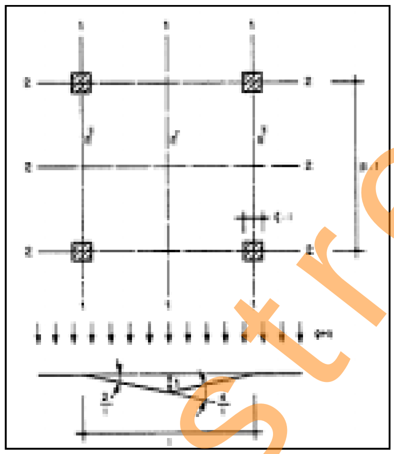


Figure 23: Line mecanisms (proposed approximation)

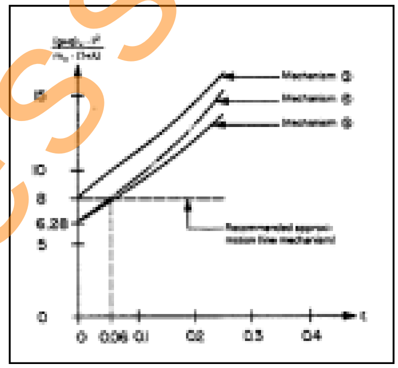
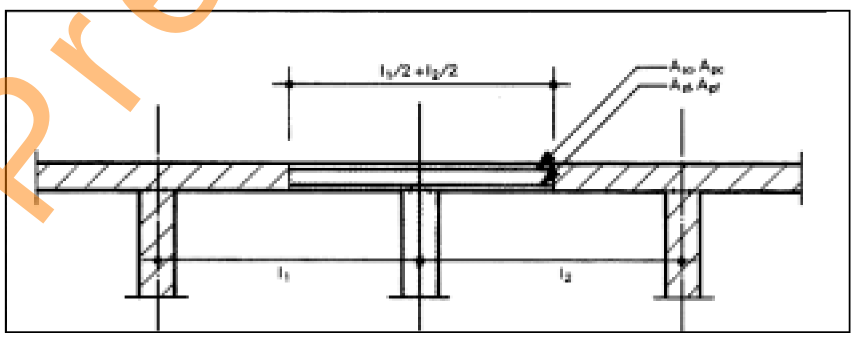


Figure 24: Ultimate design load of the various mecanisms as function of column diemnsions

Figure 25: Assumed distribution of the reinforcement in the approximation method



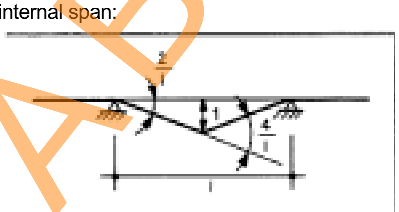
In post-tensioned flat slabs, the prestressing and the ordinary reinforcement are not uniformly distributed. In the approximation, however, both are assumed as uniformly distributed over the width $l_1/2 + l_2/2$ (Fig. 25). The ultimate load calculation can then be carried out for a strip of unit width 1. The actual distribution of the tendons will be in accordance with chapter 5.1. The top layer ordinary reinforcement should be concentrated over the columns in accordance with Fig. 35.

The load corresponding to the individual mechanisms can be obtained by the principle of virtual work. This principle states that, for a virtual displacement, the sum of the work W_e performed by the applied forces and of the dissipation work W_i performed by the internal forces must be equal to zero.

$$W_e + W_i = 0 \qquad (3.3.)$$

If this principle is applied to mechanism ① (yield lines 1-1; Fig. 23), then for a strip of width $l_1/2 + l_2/2$ the ultimate design load $(g+q)_u$ is obtained.

internal span:



$$W_e = \frac{1}{2} \cdot (g+q)_u \cdot l \cdot \frac{(l_1+l_2)}{2} \qquad (3.4.)$$

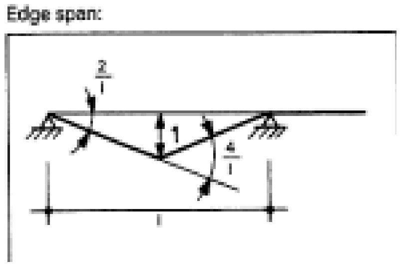
$$W_i = -\frac{4}{l} \cdot m_u \cdot (1+\lambda) \cdot \frac{(l_1+l_2)}{2} \qquad (3.5.)$$

With $W_e + W_i = 0$ the ultimate design load becomes

$$(g+q)_u = \frac{8}{l^2} \cdot m_u \cdot (1+\lambda) \qquad (3.6.)$$

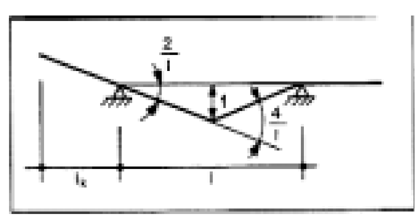
where $\lambda = \frac{m_{uc}}{m_u}$ (considering Fig. 25)

Edge span:



$$(g+q)_u = \frac{8}{l^2} \cdot m_u \cdot \frac{(1+\lambda)}{2} \qquad (3.7.)$$

Edge span with cantilever:



$$(g+q)_u = \frac{8}{l^2} \cdot \left[m_u \cdot \left(1 + \frac{\lambda}{2}\right) + \frac{1}{2} \cdot m^k_{min} \right] \qquad (3.8.)$$

where $m^k_{min} = \frac{g l_k^2}{2}$

For complicated structural systems, the determining mechanisms have to be found. Descriptions of such mechanisms are available in the relevant literature, e.g. [31], [36].
In special cases with irregular plan shape, recesses etc., simple equilibrium considerations (static method) very often prove to be a suitable procedure. This leads in the simplest case to the carrying of the load by means of beams (beam method). The moment distribution according to the theory of elasticity may also be calculated with the help of computer programmes and internal stress states may be superimposed upon these moments. The design has then to be done according to Method A.

3.12. Ultimate stength of a cross-section

For given dimensions and concrete qualities, the ultimate strength of a cross-section is dependent upon the following variables:
- Ordinary reinforcement
- Prestressing steel, bonded or unbonded
- Membrane effect

The membrane effect is usually neglected when determining the ultimate strength. In many cases this simplification constitutes a considerable safety reserve [8], [10]. The ultimate strength due to ordinary reinforcement and bonded post-tensioning can be calculated on the assumption, which in slabs is almost always valid, that the steel yields, This is usually true also for cross-sections over intermediate columns, where the tendons are highly concentrated.
In bonded post- tensioning, the prestressing force in cracks is transferred to the concrete by bond stresses on either side of the crack . Around the column mainly radial cracks open and a tangentially acting concrete compressive zone is formed. Thus the so-called effective width is considerably increased [27]. In unbonded post-tensioning, the prestressing force is transferred to the concrete by the end anchorages and, by approximation, is therefore uniformly distributed over the entire width at the columns.

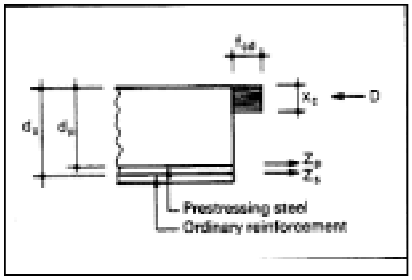


Figure 26: ultimate strenght of a cross-section (plastic moment)

For unbonded post-tensioning steel, the question of the steel stress that acts in the ultimate limit state arises. If this steel stress is known (see Chapter 3.1.3.), the ultimate strength of a cross-section (plastic moment) can be determined in the usual way (Fig. 26):

$$m_u = z_s \cdot \left(d_s - \frac{x_c}{2}\right) + z_p \cdot \left(d_p - \frac{x_c}{2}\right) \qquad (3.9)$$

where

$$z_s = A_s \cdot f_{sy} \qquad (3.10.)$$

$$z_p = A_p \cdot (\sigma_{p\infty} + \Delta\sigma_p) \qquad (3.11.)$$

$$x_c = \frac{z_s + z_p}{b \cdot f_{cd}} \qquad (3.12.)$$

3.1.3. Stress increase in unbonded post-tensioned steel

Hitherto, the stress increase in the unbonded post-tensioned steel has either been neglected [34] or introduced as a constant value [37] or as a function of the reinforcement content and the concrete compressive strength [38].
A differentiated investigation [10] shows that this increase in stress is dependent both upon the geometry and upon the deformation of the entire system. There is a substantial difference depending upon whether a slab is laterally restrained or not. In a slab system, the internal spans may be regarded as slabs with lateral restraint, while the edge spans in the direction perpendicular to the free edge or the cantilever, and also the corner spans are regarded as slabs without lateral restraint.
In recent publications [14], [15], [16], the stress increase in the unbonded post-tensioned steel at a nominal failure state is estimated and is incorporated into the calculation together with the effective stress present (after losses due to friction, shrinkage, creep and relaxation). The nominal failure state is established from a limit deflection a_u.
With this deflection, the extensions of the prestressed tendons in a span can be determined from geometrical considerations.
Where no lateral restraint is present (edge spans in the direction perpendicular to the free edge or the cantilever, and corner spans) the relationship between tendon extension and the span l is given by:

$$\frac{\Delta l}{l} = 4 \cdot \frac{a_u}{l} \cdot \frac{y_p}{l} = 3 \cdot \frac{a_u}{l} \cdot \frac{d_p}{l} \qquad (3.13.)$$

whereby a triangular deflection diagram and an internal lever arm of $y = 0.75 \cdot d$, is assumed The tendon extension may easily be determined from Fig. 27.
For a rigid lateral restraint (internal spans) the relationship for the tendon extension can be calculated approximately as

$$\frac{\Delta l}{l} = 2 \cdot \left(\frac{a_u}{l}\right)^2 + 4 \cdot \frac{a_u}{l} \cdot \frac{h_p}{l} \qquad (3.14.)$$

Fig. 28 enables the graphic evaluation of equation (3.14.), for the deviation of which we refer to [10]
The stress increase is obtained from the actual stress-strain diagram for the steel and from the elongation of the tendon Δl uniformly distributed over the free length L of the tendon between the anchorages. In the elastic range and with a modulus of elasticity E_p for the prestressing steel, the increase in steel stress is found to be

$$\Delta\sigma_p = \frac{\Delta l}{l} \cdot \frac{l}{L} \cdot E_p = \frac{\Delta l}{L} \cdot E_p \qquad (3.15)$$

The steel stress, plus the stress increase $\Delta\sigma_p$ must, of course, not exceed the yeld strength of the steel.
In the ultimate load calculation, care must be taken to ensure that the stress increase is established from the determining mechanism. This is illustaced diagrammatically

Figure 27: Tendon extension without lateral restraint

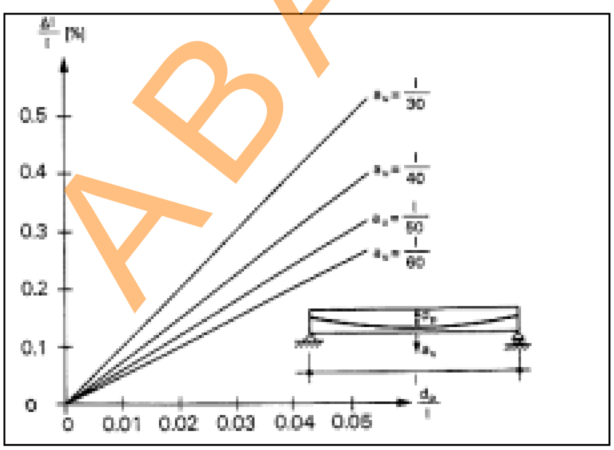
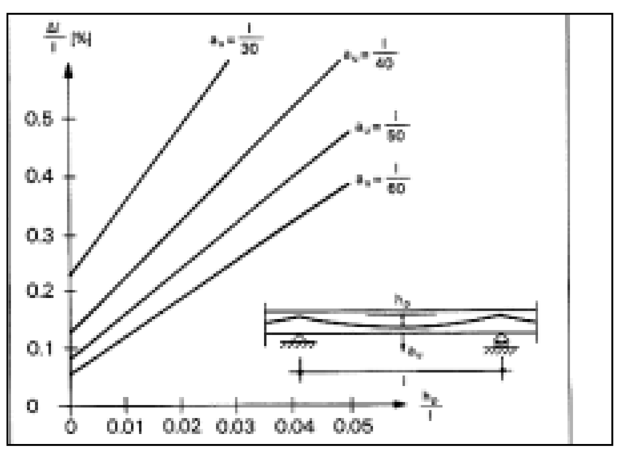


Figure 28: Tendon extension with rigid lateral restraint



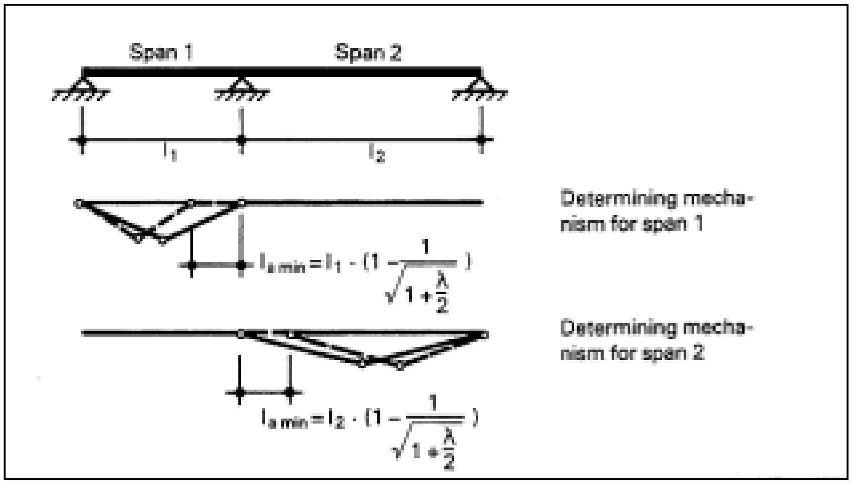


Figure 29: Determining failure mechanisms for two-span beam

in Fig 29 with reference to a two-span beam. It has been assumed here that the top layer column head reinforcement is protruding beyond the column by at least

$$l_{a\,min} \geq l \cdot (1 - \frac{1}{\sqrt{1+\frac{\lambda}{2}}}) \qquad (3.16)$$

in an edge span and by at least

$$l_{a\,min} \geq \frac{1}{2} \cdot (1 - \frac{1}{\sqrt{1+\lambda}}) \qquad (3.17)$$

in an internal span. It must be noted that $l_{a\,min}$ does not include the anchoring length of the reinforcement.
In particular, it must be noted that, if $l_1 = l_2$, the plastic moment over the internal column will be different depending upon whether span 1 or span 2 is investigated.

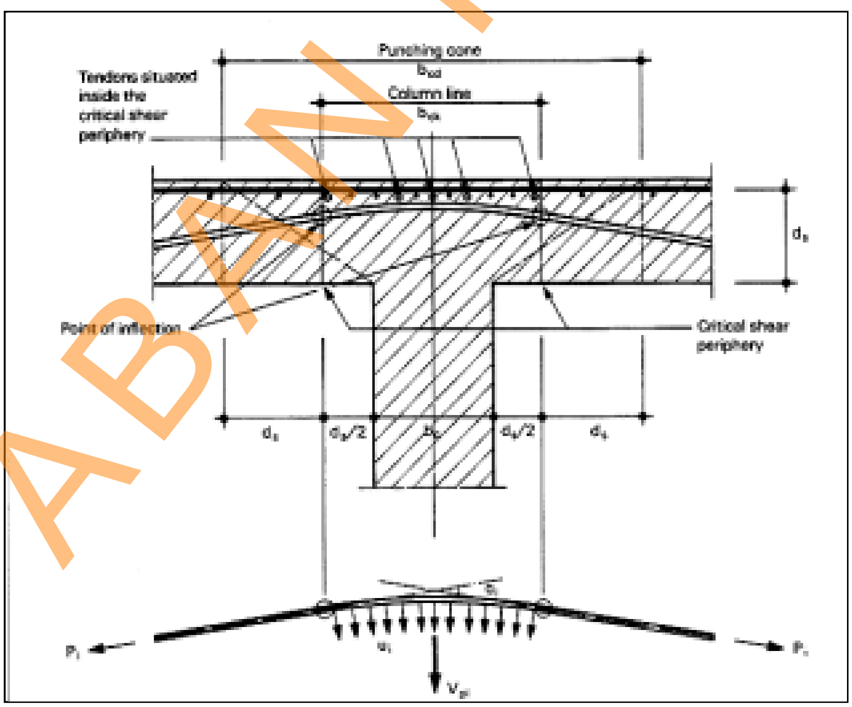
Example of the calculation of a tendon extension:
According to [14], which is substantially in line with the above considerations, the nominal failure state is reached when with a determining mechanism a deflection a_u of 1/40th of the relevant span l is present.
Therefore equations (3.13) and (3.14) for the tendon extension can be simplified as follows:
Without lateral restraint, e.g. for edge spans of flat slabs:

$$\Delta l = 0.075 \cdot d_p \qquad (3.18.)$$

With a rigid lateral restraint, e g. for internal spans of flat slabs:

$$\Delta l = 0.05 \cdot (0.025 \cdot 1 + 2 \cdot h_p) \qquad (319.)$$

Figure 30: Portion of slab in column area; transverse components due to prestress in critical shear contrary



3.2. Punching shear

32.1. General

Punching shear has a position of special importance in the design of flat slabs. Slabs, which are practically always under-reinforced against flexure, exhibit pronounced ductile bending failure. In beams, due to the usually present shear reinforcement, a ductile failure is usually assured in shear also. Since slabs, by contrast, are provided with punching shear reinforcement only in very exceptional cases,because such reinforcement is avoided if at all possible for practical reasons, punching shear is associated with a brittle failure of the concrete.
This report cannot attempt to provide generally valid solutions for the punching problem. Instead, one possibile solution will be illustrated. In particular we shall discuss how the prestress can be taken into account in the existing design specifications, which have usually been developed for ordinarily reinforced flat slabs.
In the last twenty years, numerous design formulae have been developed, which were obtained from empirical investigations and, in a few practical cases, by model represtation. The calculation methods and specifications in most common use today limit the nominal shear stress in a critical section around the column in relation to a design value as follows [9]:

$$\tau_{Sd} \leq \tau_{ud}(f_c, \rho_m, \rho_v, \frac{h}{l}, \zeta, \ldots) \qquad (3.20.)$$

The design shear stress value τ_{ud} is established from shear tests carried out on portions of slabs. It is dependent upon the concrete strength f_c, the bending reinforcement content ρ_m, the shear reinforcement content ρ_v,the slab slenderness ratio h/l, the ratio of column dimension to slab thickness ζ, bond properties and others. In the various specifications and standards, only some of these influences are taken into account.

3.2.2. Influence of post tensioning

Post-tensioning can substantially alleviate the punching shear problem in flat slabs if the tendon layout is correct.
A portion of the load is transferred by the transverse components resulting from prestressing directly to the column. The tendons located inside the critical shear periphery (Fig. 30) can still carry loads in the form of a cable system even after the concrete compressive zone has failed and can thus prevent the collapse of the slab. The zone in which the prestress has a loadrelieving effect is here intentionally assumed to be smaller than the punching cone. Recent tests [27] have demonstrated that, after the shear cracks have appeared, the tendons located outside the crlncal shear periphery rupture the concrete vertically unless heavy ordinary reinforcement is present, and they can therefore no longer provide a load-bearing function.
If for constructional reasons it is not possible to arrange the tendons over the column within the critical shear periphery or column strip b_{ck} defined in Fig. 30 then the transfer of the transverse components resulting

from tendons passing near the column should be investigated with the help of a space frame model. The distance between the outermost tendons to be taken into account for direct load transfer and the edge of the column should not exceed d_s on either side of the column.

The favourable effect of the prestress can be taken account of as follows:

1 The transverse component $V_{p\infty}$ resulting from the effectively present prestressing force and exerted directly in the region of the critical shear periphery can be subtracted from the column load resulting from the applied loads. In the tendons, the prestressing force after deduction of all losses and without the stress increase should be assumed. The transverse component V_P is calculated from Fig. 30 as

$$V_P = \Sigma\, P_i \,.\, a_i = P.\, a \qquad (3.21.)$$

Here, all the tendons situated within the critical shear periphery should be considered, and the angle of deviation within this shear periphery should be used for the individual tendons.

2 The bending reinforcement is sometimes taken into account when establishing the permissible shear stress [37], [38], [39]. The prestress can be taken into account by an equivalent portion [15], [16]. However, as the presence of concentric compression due to prestress in the column area is not always guaranteed (rigid walls etc.) it is recommended that this portion should be ignored.

3.2.3. Carrying out the calculation

A possible design procedure is shown in [14]; this proof, which is to be demonstrated in the ultimate limit state, is as follows:

$$\frac{R_d}{1.3} \geq 1.4 \cdot V_{g+q} - \frac{V_P}{1.3} \qquad (3.22.)$$

The design value for ultimate strength for **concentric punching of columns through slabs of constant thickness without punching shear reinforcement** should be assumed as follows:

$$R_d = u_c \cdot d_s \cdot 1.5 \cdot T_{ud} \qquad (3.23.)$$

U_c is limited to 16 $\cdot$ d_s, at maximum and the ratio of the sides of the rectangle surrounding the column must not exceed 2:1.

T_{ud} can be taken from Table I.

If punching shear reinforcement must be incorporated, it should be designed by means of a space frame model with a concrete compressive zone in the failure state inclined at 45° to the plane of the slab, for the column force 1.8 V_{g+q}-V_P. Here, the following condition must be complied with.

$$2 \cdot R_d \geq 1.8 \cdot V_{g+q} - V_p \qquad (3.24.)$$

For punching shear reinforcement, vertical stirrups are recommended; these must pass around the top and bottom slab reinforcement. The stirrups nearest to the edge of the column must be at a distance from this column not exceeding 0.5 • d_s. Also, the spacing between stirrups in the radial direction must not exceed 0.5 • d_s (Fig.31). Slab connections to edge columns and corner columns should be designed according to the considerations of the **beam theory.** In particular, both ordinary reinforcement and post-tensioned tendons should be continued over the column and properly anchored at the free edge (Fig. 32).

Table I – τ_{ud} as a function of f_c^{28}

Cube strength of concrete f_c^{28}	τ_{ud} in N/mm²
30 N/mm²	1.0
40 N/mm²	1.2
> 50 N/mm²	1.4

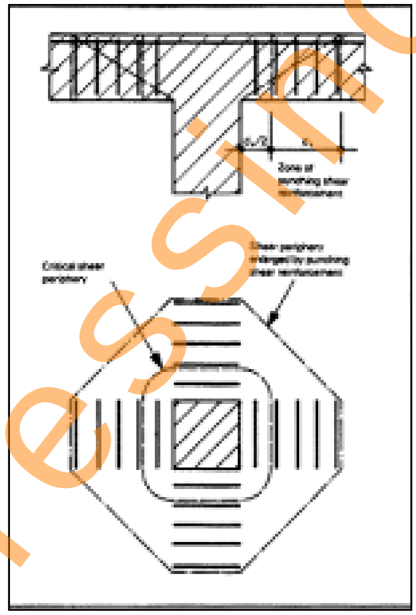


Figure 31: Punching shear reinforcement

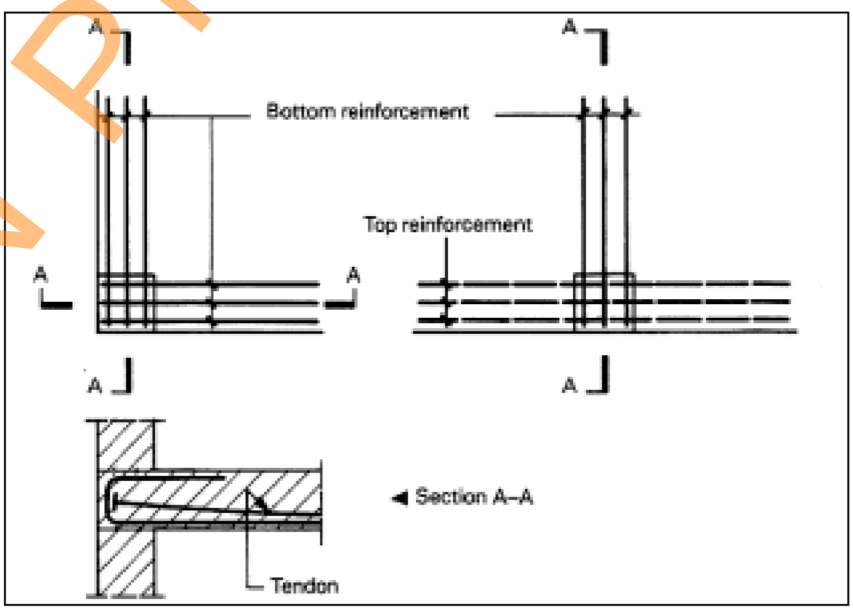


Figure 32: Arrangement of reinforcement at corner and edge columns

4. Serviceability limit state

4.1. Crack limitation

4.1.1. General

In slabs with ordinary reinforcement or bonded post-tensioning, the development of cracks is dependent essentially upon the bond characteristics between steel and concrete. The tensile force at a crack is almost completely concentrated in the steel. This force is gradually transferred from the steel to the concrete by bond stresses. As soon as the concrete tensile strength or the tensile resistance of the concrete tensile zone is exceeded at another section, a new crack forms.

The influence of unbonded post-tensioning upon the crack behaviour cannot be investigated by means of bond laws. Only very small frictional forces develop between the unbonded stressing steel and the concrete. Thus the tensile force acting in the steel is transferred to the concrete almost exclusively as a compressive force at the anchorages.

Theoretical [10] and experimental [8] investigations have shown that normal forces arising from post-tensioning or lateral membrane forces influence the crack behaviour in a similar manner to ordinary reinforcement.

In [10], the ordinary reinforcement content p* required for crack distribution is given as a function of the normal force arising from prestressing and from the lateral membrane force n.

Fig. 33 gives p* as a function of p*, where

$$p^* = p_p - \frac{n}{dp \cdot \sigma_{po}} \qquad (4.1.)$$

If n is a compressive force, it is to be provided with a negative sign.

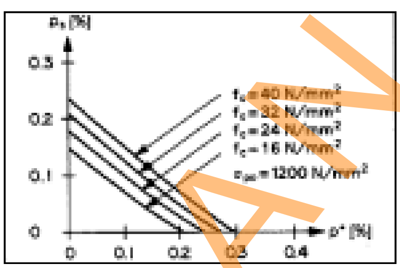


Figure 33: Reinforcement content required to ensure distribution of cracks

Various methods are set out in different specifications for the assessment and control of crack behaviour:

- Limitation of the stresses in the ordinary reinforcement calculated in the cracked state [40].
- Limitation of the concrete tensile stresses calculated for the homogeneous cross-section [12].
- Determination of the minimum quantity of reinforcement that will ensure crack distribution [14].
- Checking for cracks by theoretically or empirically obtained crack formulae [15].

4.12. Required ordinary reinforcement

The design principles given below are in accordance with [14]. For determining the ordinary reinforcement required, a distinction must be made between edge spans, internal spans and column zones.

Edge spans:
Required ordinary reinforcement (Fig. 34):
$p_s \geq 0.15 - 0.50 \cdot p_p$ (4.2)
Lower limit: $p_s \geq 0.05\%$

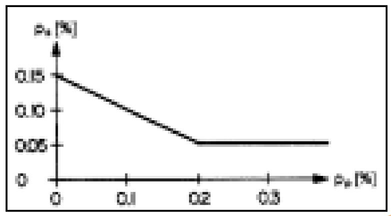


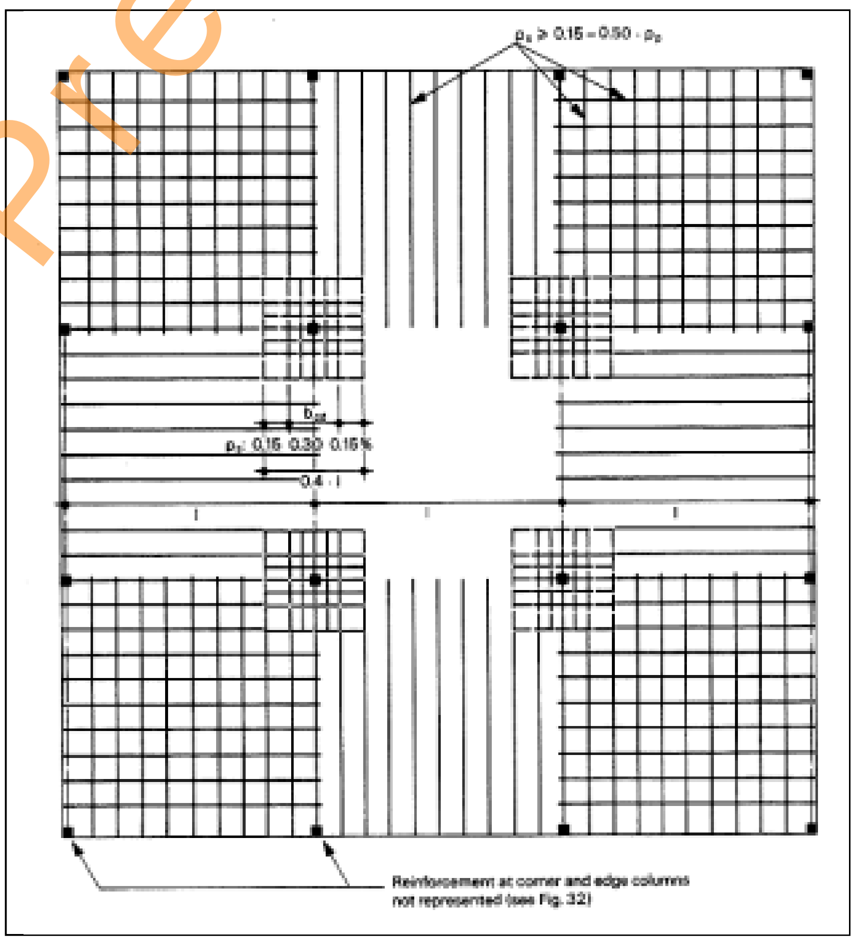
Figure 34: Minimum ordinary reinforcement required as a function of the post-tensioned reinforcement for edge spans

Internal spans:
For internal spans, adequate crack distribution is in general assured by the post-tensioning and the lateral membrane compressive forces that develop with even quite small deflections. In general, therefore, it is not necessary to check for minimum reinforcement. The quantity of normal reinforcement required for the ultimate limit state must still be provided.

Column zone:
In the column zone of flat slabs, considerable additional ordinary reinforcement must **always** be provided. The proposal of DIN 4227 may be taken as a guideline, according to which in the zone $b_{cd} = b_c + 3 \cdot d_s$ (Fig. 30) at least 0.3% reinforcement must be provided and, within the rest of the column strip ($b_g = 0.4 \cdot l$) at least 0.15% must be provided (Fig. 35). The length of this reinforcement including anchor length should be $0.4 \cdot l$. Care should be taken to ensure that the bar diameters are not too large.

The arrangement of the necessary minimum reinforcement is shown diagrammatically in Fig.35. Reinforcement in both directions is generally also provided everywhere in the edge spans. In internal spans it may be necessary for design reasons, such as point loads, dynamic loads (spalling of concrete) etc. to provide limited ordinary reinforcement.

Figure 35: Diagrammatic arrangement of minimum reinforcement



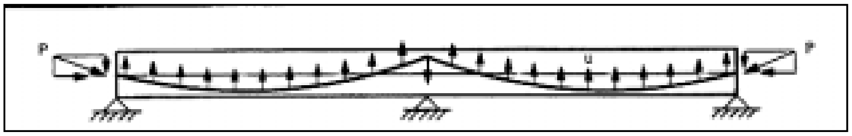


Figure 36: Transverse components and panel forces resulting from post-tensioning

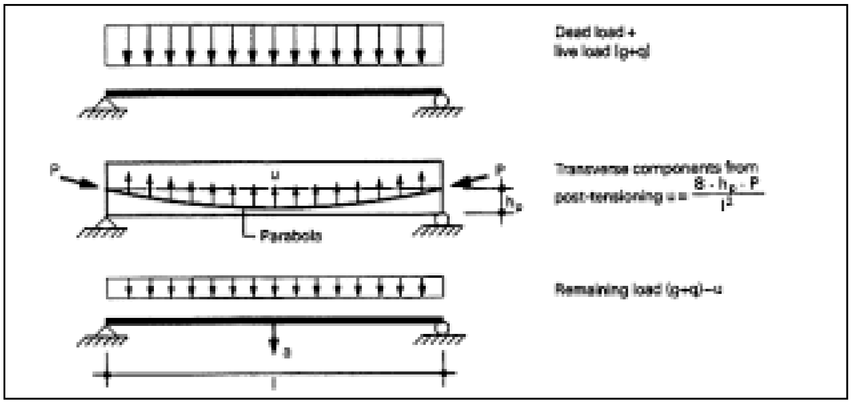


Figure 37: Principle of the load-balancing method

4.2. Deflections

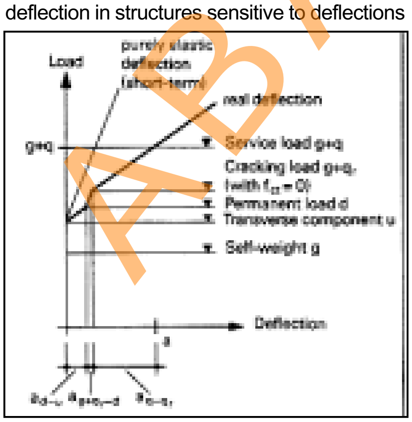
Post-tensioning has a favourable influence upon the deflections of slabs under service loads. Since, however, post-tensioning also makes possible thinner slabs, a portion of this advantage is lost.

As already mentioned in Chapter 3.1.1., the load-balancing method is very suitable for calculating deflections. Fig. 36 and 37 illustrate the procedure diagrammatically.

Under permanent loads, which may with advantage be largely compensated by the transverse components from post-tensioning, the deflections can be determined on the assumption of uncracked concrete.

Under live loads, however, the stiffness is reduced by the formation of cracks. In slabs with bonded post-tensioning, the maximum loss of stiffness can be estimated from the normal reinforced concrete theory. In slabs with unbonded post-tensioning, the reduction in stiffness, which is very large in a simple beam reinforced by unbonded post-tensioning, is kept within limits in edge spans by the ordinary reinforcement necessary for crack distribution,

Figure 38: Diagram showing components of deflection in structures sensitive to deflections



and in internal spans by the effect of the lateral restraint.

In the existing specifications, the deflections are frequently limited by specifying an upper limit to the slenderness ratio (see Appendix 2).

In structures that are sensitive to deflection, the deflections to be expected can be estimated as follows (Fig. 38):

$$a = a_{d-u} + a_{g+qr-d} + a_{q-qr} \qquad (4.3.)$$

The deflection a_{d-u}, should be calculated for the homogeneous system making an allowance for creep. Up to the cracking load $g+q_r$, which for reasons of prudence should be calculated ignoring the tensile strength of the concrete, the deflection a_{g+q_r-d} should be established for the homogeneous system under short-term loading. Under the remaining live loading, the deflection a_{q-qr} should be determined by using the stiffness of the cracked crosssection. For this purpose, the reinforcement content from ordinary reinforcement and prestressing can be assumed as approximately equivalent, i.e. $p=p_s+p_p$ is used.

In many cases, a sufficiently accurate estimate of deflections can be obtained if they are determined under the remaining load (g+q-u) for the homogeneous system and the creep is allowed for by reduction of the elastic modulus of the concrete to

$$E_c^I = \frac{E_c}{1+\varphi} \qquad (4.4.)$$

On the assumption of an average creep factor φ = 2 [41] the elastic modulus of the concrete should be reduced to

$$E_c^I = \frac{E_c}{3} \qquad (4.5.)$$

4.3. Post-tensioning force in the tendon

4.3.1. Losses due to friction

For monostrands, the frictional losses are very small. Various experiments have demonstrated that the coefficients of friction μ= 0.06 and k = 0.0005/m can be assumed. It is therefore adequate for the design to adopt a lump sum figure of 2.5% prestressing force loss per 10 m length of strand. A constant force over the entire length becomes established in the course of time.

For bonded cables, the frictional coefficients are higher and the force does not become uniformly distributed over the entire length. The calculation of the frictional losses is carried out by means of the well-known formula $P_x = P_o \cdot e^{-(\mu\alpha+kx)}$. For the coefficients of friction the average values of Table II can be assumed.

The force loss resulting from wedge drawin when the strands are locked off in the anchorage, can usually be compensated by overstressing. It is only in relatively short cables that the loss must be directly allowed for. The way in which this is done is explained in the calculation example (Chapter 8.2.).

4.32. Long-term losses

The long-term losses in slabs amount to about 10 to 12% of the initial stress in the prestressing steel. They are made up of the following components:

Creep losses:

Since the slabs are normally post-tensioned for dead load, there is a constant compressive stress distribution over the cross-section. The compressive stress generally is between 1.0 and 2.5 N/mm^2 and thus produces only small losses due to creep. A simplified estimate of the loss of stress can be obtained with the final value for the creep deformation:

$$\Delta\sigma_{pc} = \varepsilon_{cc} \cdot E_p = \varphi_n \cdot \frac{\sigma_c}{E_c} \cdot E_p \qquad (4.6.)$$

Although the final creep coefficient φ_n due to early post-tensioning is high, creep losses exceeding 2 to 4% of the initial stress in the prestressing steel do not in general occur.

Shrinkage losses:

The stress losses due to shrinkage are given by the final shrinkage factor scs as:

$$\Delta\sigma_{ps} = \varepsilon_{cs} \cdot E_p \qquad (4.7.)$$

The shrinkage loss is approximately 5% of the initial stress in the prestressing steel.

Table II - Average values of friction for bonded cables

Duct	μ	k
flat	0.20	0.0030/m
round	0.19	0.0008/m

Relaxation losses:
The stress losses due to relaxation of the post-tensioning steel depend upon the type of steel and the initial stress. They can be determined from graphs (see [42] for example). With the very low relaxation prestressing steels commonly used today, for an initial stress of 0.7 f_{pu} and ambient temperature of 20°C, the final stress loss due to relaxation is approximately 3%.

Losses due to elastic shortening of the concrete:
For the low centric compression due to prestressing that exists, the average stress loss is only approximately 0.5% and can therefore be neglected.

4.4. Vibrations

For dynamically loaded structures, special vibration investigations should be carried out. For a coarse assessment of the dynamic behaviour, the inherent frequency of the slab can be calculated on the assumption of homogeneous action.

4.5. Fire resistance

In a fire, post-tensioned slabs, like ordinarily reinforced slabs, are at risk principally on account of two phenomena: spalling of the concrete and rise of temperature in the steel. Therefore, above all, adequate concrete cover is specified for the steel (see Chapter 5.1.4.).

The fire resistance of post-tensioned slabs is virtually equivalent to that of ordinarily reinforced slabs, as demonstrated by corresponding tests. The strength of the prestressing steel does indeed decrease more rapidly than that of ordinary reinforcement as the temperature rises, but on the other hand in post-tensioned slabs better protection is provided for the steel as a consequence of the uncracked cross-section.
The behaviour of slabs with unbonded post-tensioning is hardly any different from that of slabs with bonded post-tensioning, if the appropriate design specifications are followed. The failure of individual unbonded tendons can, however, jeopardize several spans. This circumstance can be allowed for by the provision of intermediate anchorages.
From the static design aspect, continuous systems and spans of slabs with lateral constraints exhibit better fire resistance.
An analysis of the fire resistance of posttensioned slabs can be carried out, for example, according to [43].

4.6. Corrosion protection

4.6.1. Bonded post-tensioning

The corrosion protection of grouted tendons is assured by the cement suspension injected after stressing. If the grouting operations are carefully carried out no problems arise in regard to protection.
The anchorage block-outs are filled with low-shrinkage mortar.

4.62. Unbonded post-tensioning

The corrosion protection of monostrands described in Chapter 1.3.2. must satisfy the following conditions:

- Freedom from cracking and no embrittlement or liquefaction in the temperature range -20° to +70 °C
- Chemical stability for the life of the structure
- No reaction with the surrounding materials
- Not corrosive or corrosion-promoting
- Watertight

A combination of protective grease coating and plastics sheathing will satisfy these requirements.
Experiments in Japan and Germany have demonstrated that both polyethylene and polypropylene ducts satisfy all the above conditions.
As grease, products on a mineral oil base are used; with such greases the specified requirements are also complied with.
The corrosion protection in the anchorage zone can be satisfactorily provided by appropriate constructive detailing (Fig. 39), in such a manner that the prestressing steel is continuously protected over its entire length.
The anchorage block-out is filled with lowshrinkage mortar.

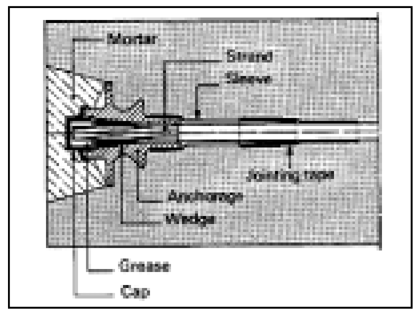


Figure 39: Corrosion protection in the anchorage zone

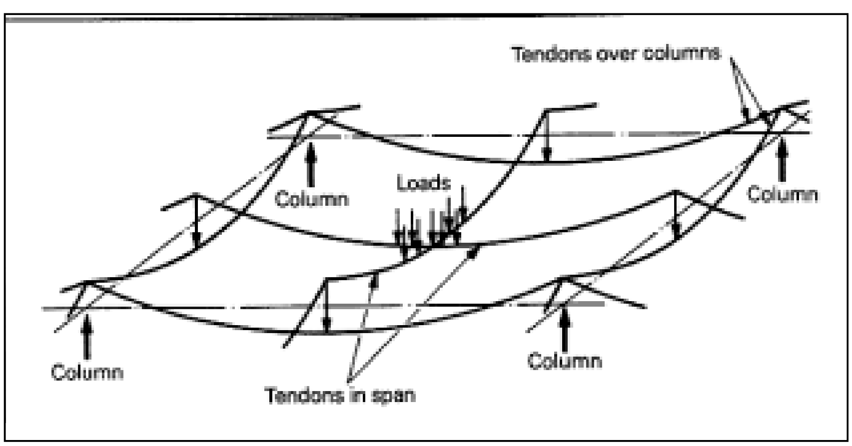
5. Detail design aspects

5.1. Arrangement of tendons

5.1.1. General

The transference of loads from the interior of a span of a flat slab to the columns by transverse components resulting from prestressing is illustrated diagrammatically in Fig. 40.
In Fig. 41, four different possible tendon arrangements are illustrated: tendons only over the colums in one direction (a) or in two directions (b), the spans being ordinarily reinforced (column strip prestressing); tendons distributed in the span and concentrated along the column lines (c and d). The tendons over the colums (for column zone see Fig. 30) act as concealed main beams.
When selecting the tendon layout, attention should be paid to flexure and punching and also to practical construction aspects (placing of tendons). If the transverse component is made equal to the dead load,then under dead load and prestress a complete load balance is achieved in respect of flexure and shear if 50 % of the tendons are uniformly distributed in the span and 50 % are concentrated over the columns.

Figure 40: Diagrammatic illustration of load transference by post-tensioning



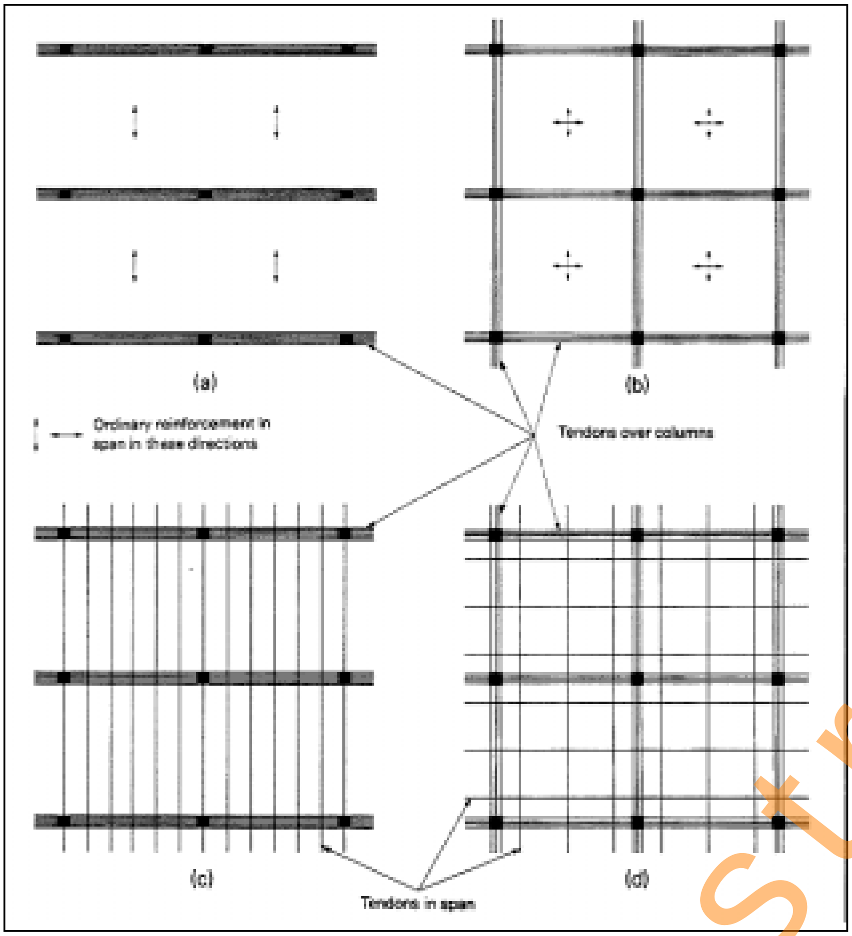


Figure 41: Possible tendon arrangements

Table III - Required cover of prestressing steel by concrete (in mm) as a function of conditions of exposure and concrete grade

Conditions of exposure	Concrete grade		
	C 20	C 25–C 35	C 40–C 50
Mild 1)	15	15	15
Moderate 2)	25	20	15
Severe 3)	35	30	25

1) for example, completely protected against weather, or aggressive conditions, except for brief period of exposure to normal weather conditions during construction.
2) for example, sheltered from severe rain or against freezing while saturated with water, buried concrete and concrete continuously under water.
3) for example, exposed to driving rain, alternate wetting and drying and to freezing while wet, subject to heavy condensation or corrosive fumes.

Under this loading case, the slab is stressed only by centric compressive stress. In regard to punching shear, it may be advantageous to position more than 50 % of the tendons over the columns.

In the most commonly encountered cases, the tendon arrangement illustrated in Fig. 41 (d), with half the tendons in each direction uniformly distributed in the span and half concentrated over the columns, provides the optimum solution in respect of both design and economy.

5.1.2. Spacings

The spacing of the tendons in the span should not exceed 6h, to ensure transmission of point loads. Over the column, the clear spacing between tendons or strand bundles should be large enough to ensure proper compaction of the concrete and allow sufficient room for the top ordinary reinforcement. Directly above the column, the spacing of the tendons should be adapted to the distribution of the reinforcement.

In the region of the anchorages, the spacing between tendons or strand bundles must be chosen in accordance with the dimensions of the anchorages. For this reason also, the strand bundles themselves are splayed out, and the monostrands individually anchored.

5.1.3. Radii of curvature

For the load-relieving effect of the vertical component of the prestressing forces over the column to be fully utilized, the point of inflection of the tendons or bundles should be at a distance $d_s/2$ from the column edge (see Fig. 30). This may require that the minimum admissible radius of curvature be used in the column region. The extreme fibre stresses in the prestressing steel must remain below the yield strength under these conditions. By considering the natural stiffness of the strands and the admissible extreme fibre stresses, this gives a minimum radius of curvature for practical use of **r = 2.50 m.** This value is valid for strands of nominal diameter 13 mm (0.5") and 15 mm (0.6").

5.1.4. Concrete cover

To ensure long-term performance, the prestressing steel must have adequate concrete cover. Appropriate values are usually laid down by the relevant national standards. For those cases where such information does not exist, the requirements of the CEB/FI P model code [39] are given in Table I I I.

The minimum concrete cover can also be influenced by the requirements of fire resistance. Knowledge obtained from investigations of fire resistance has led to recommendations on minimum concrete cover for the post-tensioning steel, as can be seen from Table IV. The values stated should be regarded as guidelines, which can vary according to the standards of the various countries.

For grouted tendons with round ducts the cover can be calculated to the lowest or highest strand respectively.

5.2. Joints

The use of post-tensioned concrete and, in particular, of concrete with unbonded tendons necessitates a rethinking of some long accepted design principles. A question that very often arises in building design is the arrangement of joints in the slabs, in the walls and between slabs and walls. Unfortunately, no general answer can be given to this question since there are certain factors in favour of and certain factors against joints. Two aspects have to be considered here:

Table IV - Minimum concrete cover for the post-tensioning steel (in mm) in respect of the fire resistance period required

Support	Concrete	Fire resistance period				
		F 60	F 90	F 120	F 180	F 240
Continuous or prevented from straining	Normal	20	20	20	25	35
	Light-weight	20	20	20	20	25
Freely supported and not prevented from straining	Normal	20	35	40	55	–
	Light-weight	20	25	35	40	–

- Ultimate limit state (safety)
- Horizontal displacements (serviceability limit state)

5.2.1. Influence upon the ultimate limit state behaviour

If the failure behaviour alone is considered, it is generally better not to provide any joints. Every joint is a cut through a load-bearing element and reduces the ultimate load strength of the structure.
For a slab with unbonded post-tensioning, the membrane action is favourably influenced by a monolithic construction. This results in a considerable increase in the ultimate load (Fig. 42).

5.2.2. Influence upon the serviceability limit state

In long buildings without joints, inadmissible cracks in the load-bearing structure and damage to non load-bearing constructional elements can occur as a result of horizontal displacements. These displacements result from the following influences:
- Shrinkage
- Temperature
- Elastic shortening due to prestress
- Creep due to prestress

The average material properties given in Table V enable one to see how such damage occurs.
In a concrete structure, the following average shortenings and elongations can be expected:

Shrinkage Δl_{cs} = -0.25 mm/m
Temperature Δl_{ct} = -0.25 mm/m to+0.15 mm/m
Elastic shortening
(for an average centric prestress of 1.5 N/mmz and E_c= 30 kN/mm2) Δl_{cel} = -0.05 mm/m
Creep Δl_{cc} = - 0.15 mm/m

These values should be adjusted for the particular local conditions.
When the possible joint free length of a structure is being assessed, the admissible total displacements of the slabs and walls or columns and the admissible relative displacements between slabs and walls or columns should be taken into account. Attention should, of course, also be paid to the foundation conditions.
The horizontal displacements can be partly reduced or prevented during the construction stage by suitable constructional measures (such as temporary gaps etc.) without damage occurring.

Shrinkage:
Concrete always shrinks, the degree of shrinkage being highly dependent upon the water-cement ratio in the concrete, the cross-sectional dimensions, the type of curing and the atmospheric humidity. Shortening due to shrinkage can be reduced by up to about one-half by means of temporary shrinkage joints.

Temperature:
In temperature effects, it is the temperature difference between the individual structural components and the differing coefficients of thermal expansion of the materials that are of greatest importance.

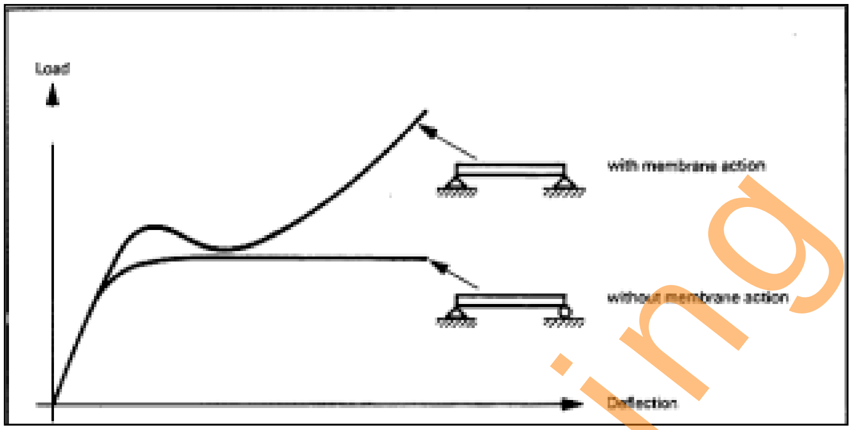


Figure 42: Influence of membrane action upon load-bearing capacity

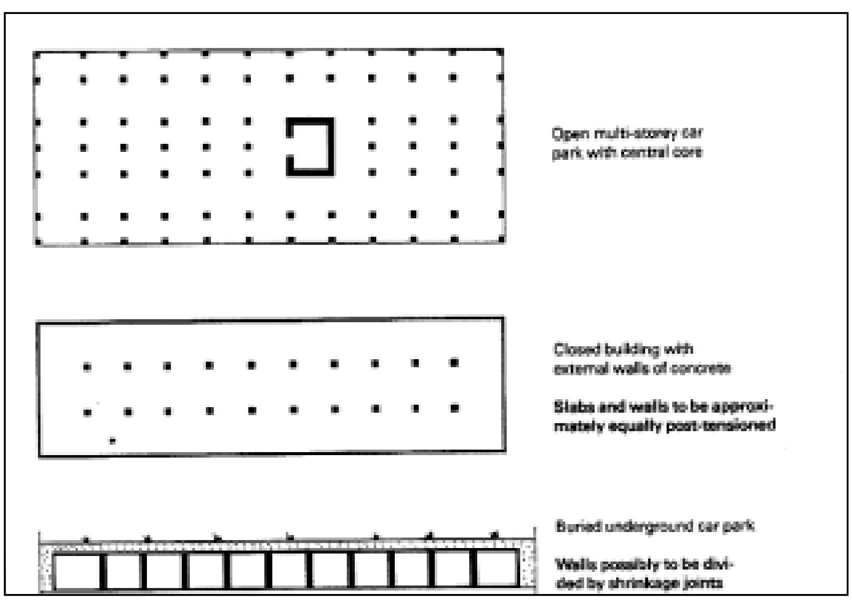
Table V -Average material properties of various construction materials

Construction material	Linear coefficient of thermal expansion ε_t Degree^{-1}	Final shrinkage coefficient ε_s	Final creep coefficient φ_n	Elastic modulus E kN/mm^2
Reinforced concrete	$12 \cdot 10^{-6}$	$2.5 \cdot 10^{-3}$	3.0	20 to 50
Brick walling	$6 \cdot 10^{-6}$	~ 0	~ 0	5 to 15
Block walling	$8 \cdot 10^{-6}$	~ 0	~ 0	7 to 18

In closed buildings, slabs and walls in the internal rooms are subject to low temperature fluctuations. External walls and unprotected roof slabs undergo large temperature fluctuations. In open buildings, the relative temperature difference is small. Particular considerations arise for the connection to the foundation and where different types of construction materials are used.

Elastic shortening and creep due to prestress:
Elastic shortening is relatively small. By subdividing the slab into separate concreting stages, which are separately post-tensioned, the shortening of the complete slab is reduced.
Creep, on the other hand, acts upon the entire length of the slab. A certain reduction occurs due to transfer of the prestress to the longitudinal walls.
Shortening due to prestress should be kept within limits particularly by the centric prestress not being made too high. It is recommended that an average centric prestress of σ_{cpm} = 1.5 N/mm^2 should be selected and the value of 2.5 N/mm^2 should not be exceeded. In concrete walls, the relative shortening between slabs and walls can be reduced by approximately uniform prestress in the slabs and walls.

Figure 43: Examples of jointless structures of 60 to 80 m length



5.2.3. Practical conclusions

In slabs of more than 30 m length, a uniform, «homogeneous» deformation behaviour of the slabs and walls in the longitudinal direction should be aimed at. In open buildings with concrete walls or columns, this requirement is satisfied in regard to temperature effects and, provided the age difference between individual components is not too great, is also satisfied for shrinkage and creep.

In closed buildings with concrete walls or columns, a homogeneous behaviour for shrinkage and creep should be achieved. In respect of temperature, however, the concreted external walls behave differently form the internal structure. If cooling down occurs, tensile stresses develop in the wall. Distribution of the cracks can be ensured by longitudinal reinforcement. The tensile stresses may also be compensated for by post-tensioning the wall.

If, in spite of detail design measures, the absolute or relative longitudinal deformations exceed the admissible values, the building must be subdivided by joints.

Fig. 43 and 44 show, respectively, some examples in which joints can be dispensed with and some in which joints are necessary.

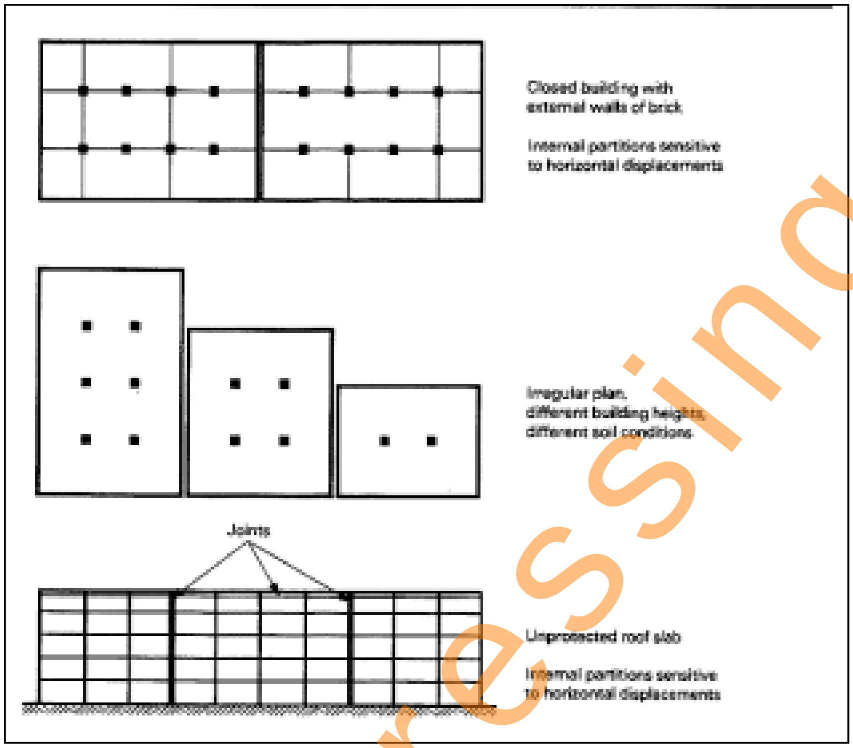


Figure 44: Examples of structures that must be subdivided by joints into sections of 30 to 40 m length

6. Construction procedures

6.1. General

The construction of a post-tensioned slab is broadly similar to that for an ordinarily reinforced slab. Differences arise in the placing of the reinforcement, the stressing of the tendons and in respect of the rate of construction.

The placing work consists of three phases: first, the bottom ordinary reinforcement of the slab and the edge reinforcement are placed. The ducts or tendons must then be positioned, fitted with supports and fixed in place. This is followed by the placing of the top ordinary reinforcement. The stressing of the tendons and, in the case of bonded tendons the grouting also, represent additional construction operations as compared with a normally reinforced slab. Since, however, these operations are usually carried out by the prestressing firm, the main contractor can continue his work without interruption.

A feature of great importance is the short stripping times that can be achieved with post-tensioned slabs. The minimum period between concreting and stripping of formwork is 48 to 72 hours, depending upon concrete quality and ambient temperature. When the required concrete strength is reached, the full prestressing force can usually be applied and the formwork stripped immediately afterwards. Depending upon the total size, the construction of the slabs is carried out in a number of sections.

The divisions are a question of the geometry of the structure, the dimensions, the planning, the construction procedure, the utilization of formwork material etc. The construction joints that do occur, are subseqently subjected to permanent compression by the prestressing, so that the behaviour of the entire slab finally is the same throughout.

The weight of a newly concreted slab must be transmitted through the formwork to slabs beneath it. Since this weight is usually less than that of a corresponding reinforced concrete slab, the cost of the supporting structure is also less.

6.2. Fabrication of the tendons

6.2.1. Bonded post-tensioning

There are two possible methods of fabricating cables:

- Fabrication at the works of the prestressing firm
- Fabrication by the prestressing firm on the site

The method chosen will depend upon the local conditions. At works, the strands are cut to the desired length, placed in the duct and, if appropriate, equipped with dead-end anchorages. The finished cables are then coiled up and transported to the site.

In fabrication on the site, the cables can either be fabricated in exactly the same manner as at works, or they can be assembled by pushing through. In the latter method, the ducts are initially placed empty and the strands are pushed through them subsequently. If the cables have stressing anchorages at both ends, this operation can even be carried out after concreting (except for the cables with flat ducts).

6.22. Unbonded post-tensioning

The fabrication of monostrand tendons is usually carried out at the works of the prestressing firm but can, if required, also be carried out on site. The monostrands are cut to length and, if necessary, fitted with the dead-end anchorages. They are then coiled up and transported to site. The stressing anchorages are fixed to the formwork. During placing, the monostrands are then threaded through the anchorages.

6.3. Construction procedure for bonded post-tensioning

In slabs with bonded post-tensioning, the operations are normally carried out as follows:

1. Erection of slab supporting formwork

2. Fitting of end formwork; placing of stressing anchorages
3. Placing of bottom and edge reinforcement
4. Placing of tendons or, if applicable, empty ducts* according to placing drawing
5. Supporting of tendons or empty ducts* with supporting chairs according to support drawing
6. Placing of top reinforcement
7. Concreting of the section of the slab
8. Removal of end formwork and forms for the stressing block-outs
9. Stressing of cables according to stressing programme
10. Stripping of slab supporting formwork
11. Grouting of cables and concreting of block-outs

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* In this case, the stressing steel is pushed through either before item 5 or before item 9.

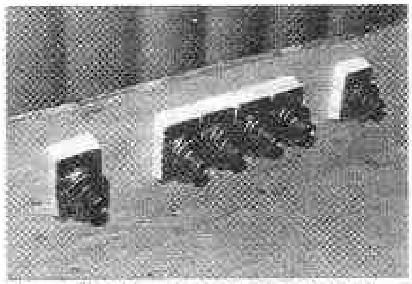


Figure 45: Stressing anchorages at end formwork

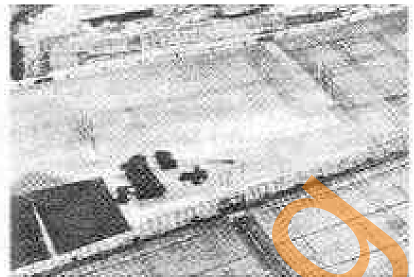


Figure 46: Placing of bottom and edge reinforcement

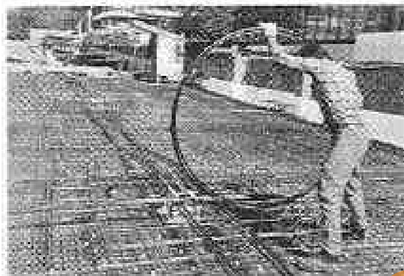


Figure 47: Uncoiling of tendons

Figure 48: Supporting of tendons

Figure 49: Placing of top reinforcement

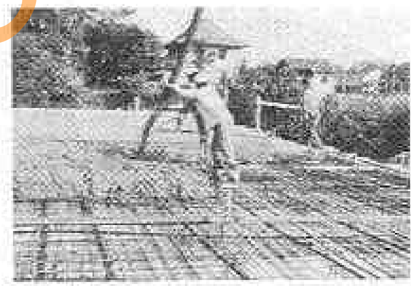


Figure 50: Concreting

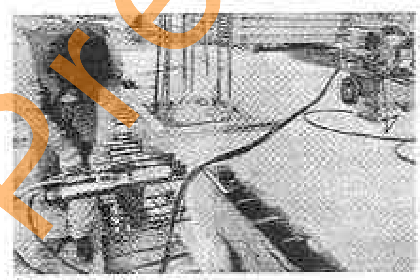


Figure 51: Stressing

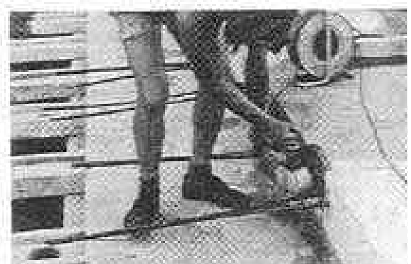


Figure 52: Cutting off protruding strand lengths

6.4. Construction procedure for unbonded post-tensioning

If unbonded tendons are used, the construction procedure set out in Chapter 6.3. is modified only by the omission of grouting (item 11).

The most important operations are illustrated in Figs. 45 to 52. The time sequence is illustrated by the construction programme (Fig. 53).

All activities that follow one another directly can partly overlap; at the commencement of activity (i+1), however, phase (i-1) must be completed. Experience has shown that those activities that are specific to prestressing (items 4, 5 and 9 in Chapter 6.3.) are with advantage carried out by the prestressing firm, bearing in mind the following aspects:

6.4.1. Placing and supporting of tendons

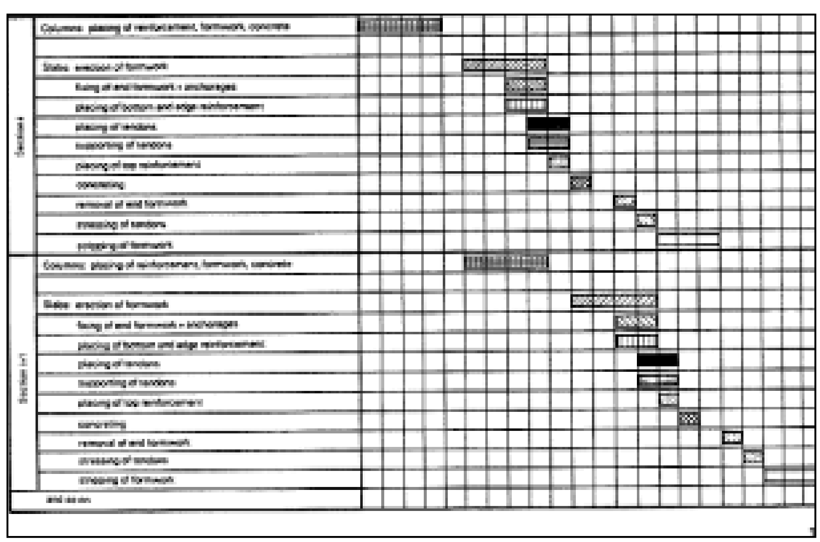
The placing sequence and the supporting of the tendons is carried out in accordance with the placing and support drawings (Figs. 54 and 55). In contrast to a normally reinforced slab, therefore, for a post-tensioned slab two drawings for the prestressing must be prepared in addition to the reinforcement drawings. The drawings for both, ordinary reinforcement and posttensioning are, however, comparatively simple and the number of items for tendons and reinforcing bars is small.

The sequence in which the tendons are to be placed must be carefully considered, so that the operation can take place smoothly. Normally a sequence allowing the tendons

Table VI-Achievable accuracies in placing

Direction	column strip	Remaining area
Vertical	± 5mm	± 5mm
Horizontal	± 20 mm	± 50 mm

Figure 53: Construction programme



to be placed without «threading» or «weaving» can be found without any difficulty. The achievable accuracies are given in Table VI.
To assure the stated tolerances, good coordination is required between all the installation contractors (electrical, heating, plumbing etc.) and the organization responsible for the tendon layout. Corresponding care is also necessary in concreting.

6.4.2. Stressing of tendons

For stressing the tendons, a properly secured scaffolding 0.50 m wide and of 2 kN/m^2 load-bearing capacity is required at the edge of the slab. For the jacks used there is a space requirement behind the anchorage of 1 m along the axis and 120 mm radius about it. All stressing operations are recorded for each tendon. The primary objective is to stress to the required load; the extension is measured for checking purposes and is compared with the calculated value.

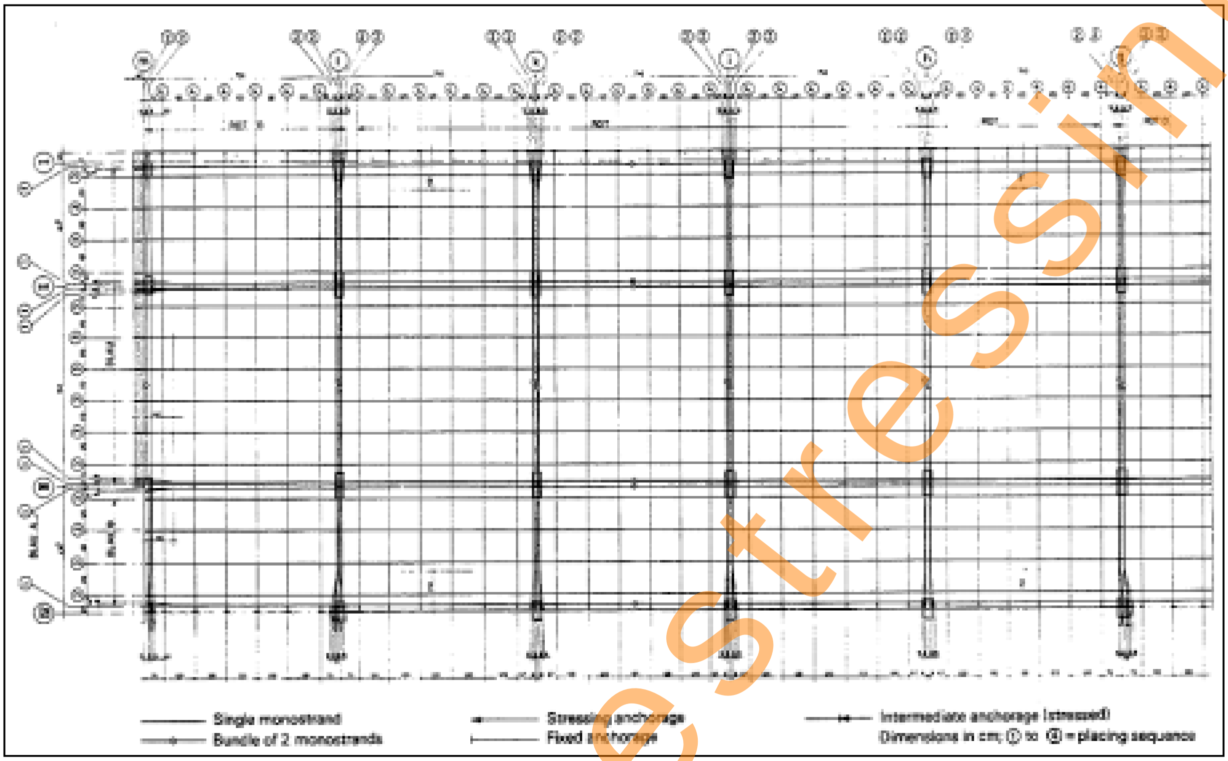
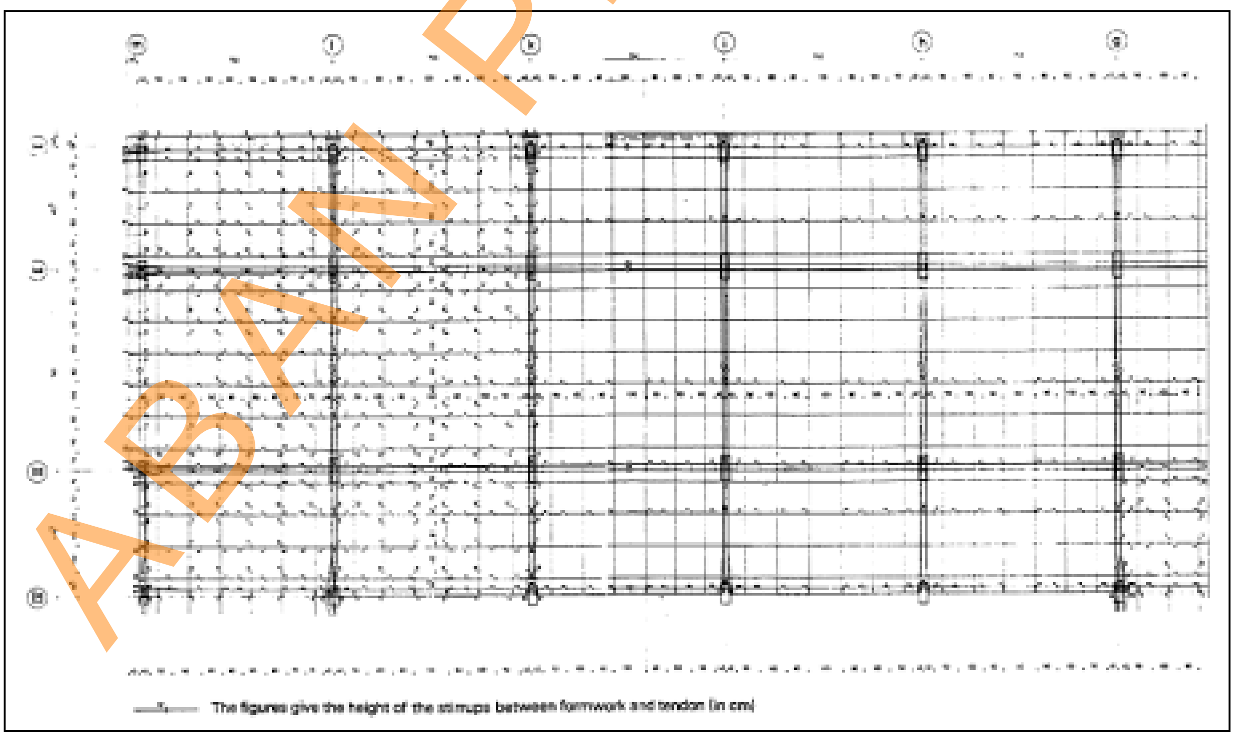


Figure 54: Placing drawing

Figure 55: Support drawing



7. Preliminary design

In the design of a structure, both the structural design requirements and the type of use should be taken into account. The following points need to be carefully clarified before a design is carried out:

- Type of structure: car park, warehouse, commercial building, residential building, industrial building, school, etc.
- Shape in plan, dimensions of spans, column dimensions; the possiblility of strengthening the column heads of a flat slab by drop panels
- Use: live load (type: permanent loads, moving loads, dynamic loads), sensitivity to deflection (e.g. slabs with rigid structures supported on them), appearance (cracks), vibrations, fire resistance class, corrosive environment, installations (openings in slabs).

For the example of a square internal span of a flat slab (Fig. 56) a rapid preliminary design will be made possible for the design engineer with the assistance of two diagrams, in which guidance values for the slab thickness and the size of the prestress are stated.

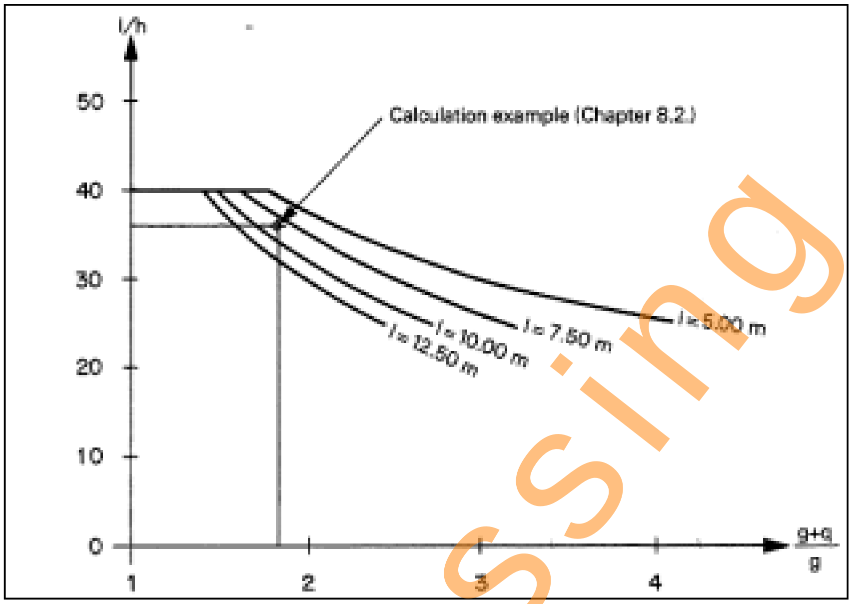


Figure 57: Recommended ratio of span to slab thickness as a function of service load to self-weight (internal span of a flat slab)

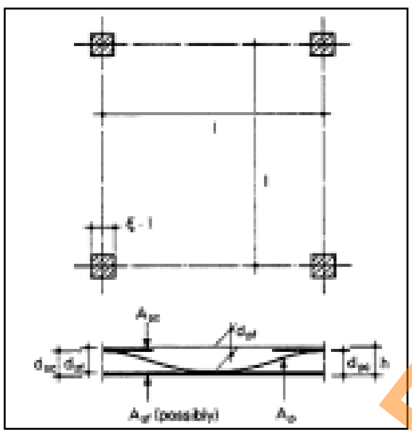


Figure 56: Internal span of a fla slab

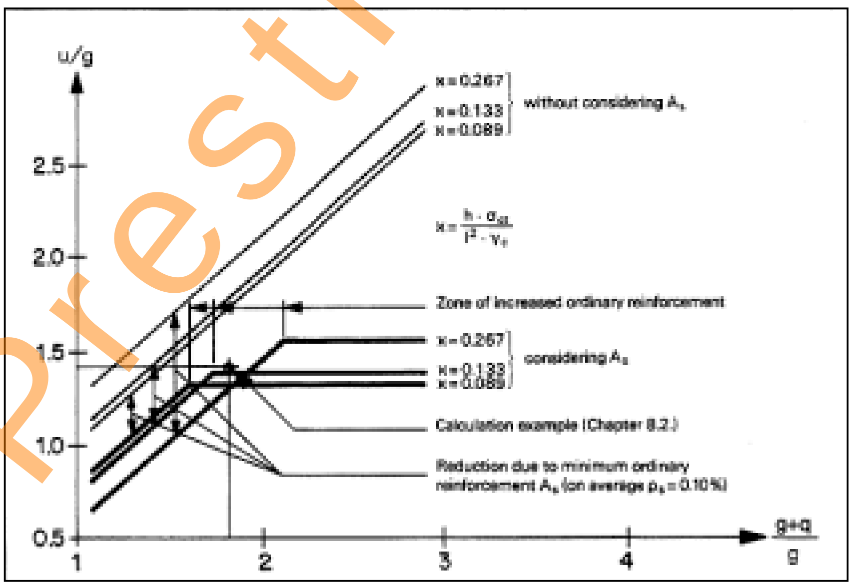


Figure 58: Ratio of transverse component a from prestress to self-weight g as a function of service

The design charts (Figs. 57 and 58) are based upon the following conditions:

1. A factor of safety of y = 1.8 is to be maintained under service load.
2. Under self-weight and initial prestress the tensile stress 6c;t for a concrete for which f_c^{28} = 30 N/mm^2 shall not exceed 1.0 N/mm^2.
3. The ultimate moment shall be capable of being resisted by the specified minimum ordinary reinforcement or, in the case of large live loads, by increased ordinary reinforcement, together with the corresponding post-tensioning steel.

The post-tensioning steel (tendons in the span and over the columns) and the ordinary reinforcement are assumed as uniformly distributed across the entire span. The tendons are to be arranged according to Chapter 5.1. and the ordinary reinforcement according to Fig. 35.

From conditon 1, the necessary values are obtained for the prestress and ordinary reinforcement as a function of the slab thickness and span. Conditon 2 limits the maximum admissible prestress. In flat slabs, the lower face in the column region is usually the determining feature. In special cases, ordinary reinforcement can be placed there. The concrete tensile stress oct (condition 2) should then be limited to σ_d 2.0 N/mm^2.

With condition 3, a guidance value is obtained for economic slab thickness (Fig.57). It is recommended that the ratio l/h shall be chosen not greater than 40. In buildings the slab thickness should normally not be less than 160 mm.

Fig. 57 and 58 can be used correspondingly for edge and corner spans.

Procedure in the preliminary design of a flat slab:

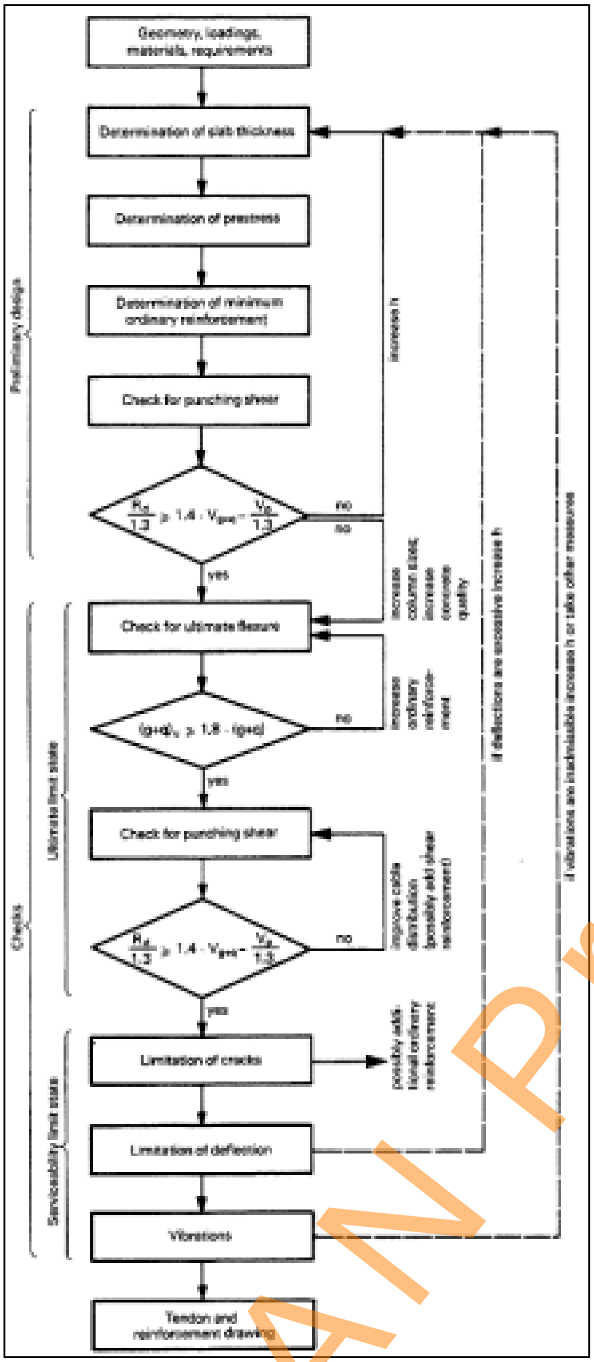
Given: span l, column dimensions, live load q

1. Estimation of the ratio l/h → self-weight g.
2. With ratio of service load (g+q) to selfweight g and span l, determine slab thickness h from Fig. 57; if necessary correct g.
3. With l, h and (g+q)/g; determine transverse component from Fig. 58 and from this prestress; estimate approximate quantity of ordinary reinforcement.
4. Check for punching; if necessary flare out column head or choose higher concrete quality or increase h.

The practical execution of a preliminary design will be found in the calculation example (Chapter 8.2.).

8. Execution of the calculations

8.1. Flow diagram



8.2. Calculation example

8.2.1. Bases

- Type of structure: commercial building

- Geometry: see Fig. 59

- Loadings:

• Live load	p	=	2.5 kN/m²
• Floor finishes	g_B	=	1.0kN/m²
• Walls	g_w	=	1.5 kN/m²
	q	=	5.0 kN/m²

- Material properties:
  - Concrete: $f_c^{28} = 35$ N/mm²; $f_{cd} = 0.6 \cdot f_c^{28} = 21$ N/mm²
  - Prestressing steel: Monostrands ∅ 15 mm (0.6")
    - A_p = 146 mm²
    - f_{py} = 1570 N/mm²
    - f_{pu} = 1770 N/mm²
    - $E_p = 1.95 \cdot 10^5$ N/mm²
    - very low relaxation (3%)
    - Admissible stresses:
      - at stressing: $0.75\ f_{pu}$
      - after wedge draw-in: max. $0.70\ f_{pu}$
    - Friction coefficients: μ=0.06, k = 0.0005/m
  - Reinforcing steel: $f_{sy} = 460$ N/mm²
- Concrete cover:
  - Prestressing steel: c_p = 30 mm
  - Reinforcing steel: c_s = 15 mm
- Long-term losses (incl. relaxation): assumed to be 10% (see Chapter 4.3.2.)

8.2.2. Preliminary design

— Determination of slab thickness:
Assumption: l/h = 35

$$\rightarrow h = \frac{8.40}{35} = 0.24\ \text{m}$$

$g = 0.24 \cdot 25 = 6$ kN/m²
$q =$ 5 kN/m²
11 kN/m²

$$\frac{g+q}{g} = \frac{11}{6} = 1.83;\ \text{hence from Fig. 57}$$

$\rightarrow$ l/h = 36

$$h = \frac{8.40}{36} = 0.233\ \text{m}$$

chosen: **h=0.24 m**

— Determination of prestress:
a) Longitudinal direction:

$$\frac{g+q}{g} = 1.83; \kappa = \frac{0.24 \cdot 1000}{8.40^2 \cdot 25} = 0.136;$$

hence from Fig. 58

$$\rightarrow \frac{u}{g} = 1.39;\ u = 1.39\ .\ 6 = 8.34\ \text{kN/m}^2$$

$$P = \frac{u\ .\ l^2}{8 \cdot h_p}$$

$$h_p = 0.144\ .\ \frac{4.20^2}{3.78^2} = 0.178\ \text{m (Fig. 60)}$$

$$P = \frac{8.34 \cdot 8.40^2}{8 \cdot 0.178} = 413\ \text{kN/m}$$

on 7.80 m width: P = 7.80 - 413 = 3221 kN
per strand: $P_L = 146\ .1770\ .\ 0.7\ .\ 10^{-3} = 181$ kN

$$\text{Number of strands:} n_p = \frac{3221}{181} = 17.8$$

→ **18 monostrands ∅ 15 mm on 7.80 m width**

$$\text{on 7.40 m width: } n_p = \frac{7.40}{7.80} \cdot 17.8 = 16.9$$

→ **17 monostrands ∅ 15 mm on 7.40 m width**

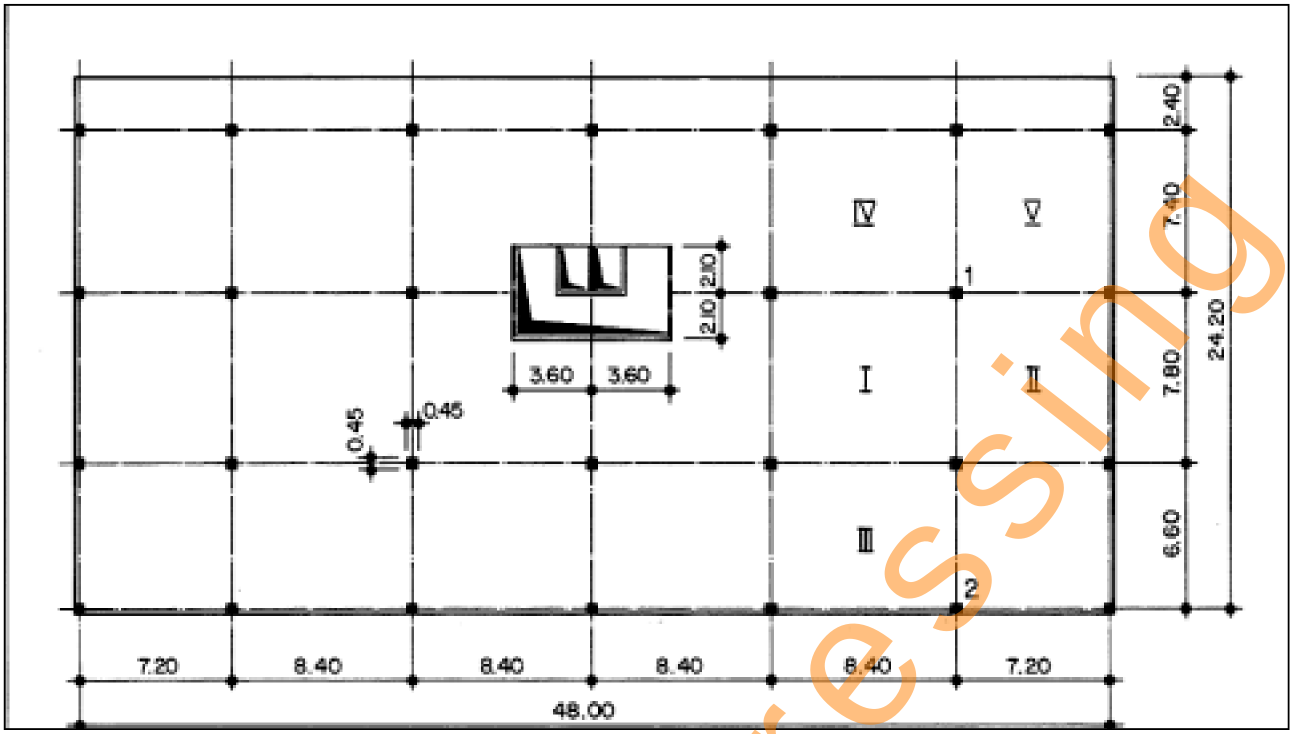


Figure 59: Plan showing dimensions

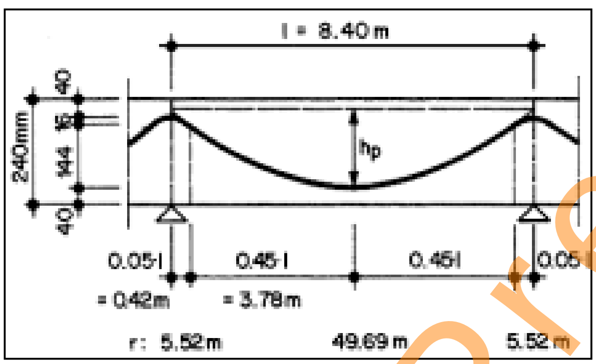


Figure 60: Tendon profile in longitudinal direction (internal span)

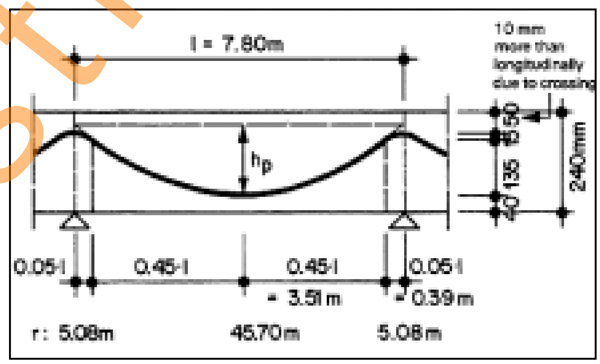


Figure 61: Tendon profile in transverse direction (internal span)

on 6.60 m width: $n_p = \frac{6.60}{7.80} \cdot 17.8 = 15.1$

→ **16 monostrands 0 15 mm on 6.60 m width**

on 2.40 m width: $n_p = \frac{2.40}{7.80} \cdot 17.8 = 5.5$

→ **6 monostrands ∅ 15 mm on 2.40 m width**

b) Transverse direction:

$\frac{g+q}{g} = 1.83; \kappa = \frac{0.24 \cdot 1000}{7.80^2 \cdot 25} = 0.158$

hence from Fig. 58

$\rightarrow \frac{u}{g} = 1.41; u = 1.41 \cdot 6 = 8.46 \mathrm{kN/m}^2$

$h_p = 0.135 \cdot \frac{7.80^2}{3.51^2} = 0.167$ m (Fig. 61)

$P = \frac{8.46 \cdot 7.80^2}{8 \cdot 0.167} = 385$ kN/m

on 8.40 m width: P=8.40 · 385=3234 kN

Number of strands: $n_p = \frac{3234}{181} = 17.9$

→**18 monostrands 0 15 mm on 8.40 m width**

on 7.20 m width: $n_p = \frac{7.20}{8{,}40} \cdot 17.9 = 15.3$

→**16 monostrands 0 15 mm on 7.20 m width**

- Determination of ordinary reinforcement:

a) Top reinforcement:
In the region of the punching cone:
$p_s = 0.3\%$ (Fig. 35)
Average of effective depth of reinforcement in both directions:
$d_{sc} = 240 - 15 - 15 = 210$ mm (approx. value)
Width b_{cd} (Fig. 30):
$b_{cd} = b_c + 3d_{sc} = 450 + 3 \cdot 210 = 1080$ mm
$\rightarrow A_{SS} = 0.003 \cdot 210 \cdot 1080 = 680$ mm²
chosen: 7 ∅12 mm (A_{ss}= 791 mm²)

- In column strip:

$p_s = 0.15\%$ (Fig. 35)
longitudinally:
$b_g = 0.4 \cdot 7800 - 1080 = 2040$ mm
$A_{sg} = 0.0015 \cdot 210 \cdot 2040 = 643$ mm²

chosen: **6 ∅12 mm** (Asg=678 mm²)

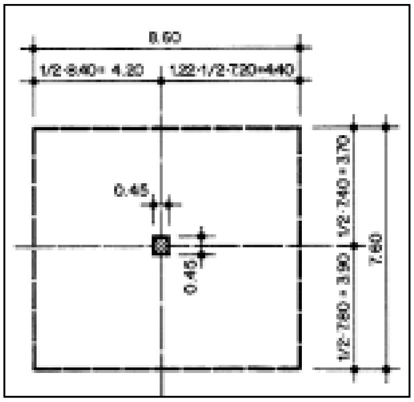


Figure 62: Influence zone column 1

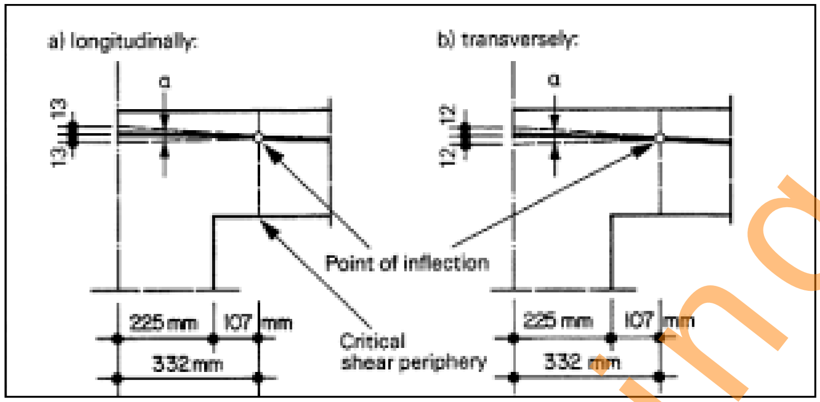


Figure 63: Tendon profile in critical shear periphery

transversely:
$b_s = 0.4 \cdot 8400 - 1080 = 2280$ mm
$A_{sg} = 0.0015 \cdot 210 \cdot 2280 = 718 \text{ mm}^2$
chosen: **4+4 ∅ 12 mm** (A_{sg}= 904 mm^2)

b) Bottom reinforcement:
- Internal spans: none
- Edge spans: $p_s \geq 0.15 - 0.50 \cdot p_p$ (Formula 4.2.)

longitudinally:

$$p_p = \frac{np \cdot A_p}{dp \cdot b} = \frac{18 \cdot 146}{200 \cdot 7800} = 0.17\%$$

$\rightarrow p_s \geq 0.15 - 0.50 \cdot 0.17 = 0.065\%$
$\rightarrow A_s \geq 0.065 \cdot 220 \cdot 10 = 143 \text{ mm}^2/\text{m}$
chosen: **∅ 6 mm, spacing 175 mm**

transversely:

$$p_p \geq = \frac{18 \cdot 146}{200 \cdot 8400} = 0.16\%$$

$\rightarrow p_s \geq 0.07\%$
$\rightarrow A_s \geq 0.07 \cdot 220 \cdot 10 = 154 \text{ mm}^2/\text{m}$
chosen: **∅ 6 mm, spacing 175 mm**

Check for punching:
Determining column 1 (Fig. 62):
$g+q = 11 \text{ kN/m}^2$
$V_{g+q} = 11 \cdot 7.60 \cdot 8.60 = 719$ kN
Prestress:
50% within the critical shear periphery, i.e. 9 monostrands in each direction

Point of inflection:
According to Fig. 30 the point of inflection ideally lies at a distance $d_s/2$ from the column edge. In Figs. 60 and 61 it is assumed that the dimensions of the column are not yet known and the point of inflection is adopted at a distance 0.05l from the column axis (value from experience). In Fig. 62 the dimensions of the column have been established. Thus the real position of the point of inflection is known. The values given in Figs. 60 and 61 change accordingly (Fig. 63).

longitudinally :
$$tg\alpha = \frac{2 \cdot 13}{332} = 0.078 = \sin\alpha \text{ (Fig. 63a)}$$
$V_p = 2 \cdot 0.078 \cdot 181 \cdot 9 \cdot 09 = 229$ kN
(Factor 0.9=10% long-term losses)

transversely :
$$tg\alpha = \frac{2 \cdot 12}{332} = 0.072 = \sin\alpha \text{ (Fig. 63b)}$$
$V_p = 2 \cdot 0.072 \cdot 181 \cdot 9 \cdot 0.9 = 211$ kN
$\Sigma V_p = 440$ kN

Ultimate design strength:
$R_d = u_c \cdot d_s \cdot 1.5 \cdot \tau_{sd}$ (Formula 3.23.)
$u_c = 4 \cdot b_c + d_{sc} \cdot \pi$
$d_{sc} = 240 - 15 - 12 = 213$ mm (real average)
$u_c = 4 \cdot 450 + 213 \cdot \pi = 2469$ mm
$16 \cdot d_{sc} = 16 \cdot 213 = 3408$ mm
$\rightarrow$ condition $u_c < 16 \cdot d_s$ satisfied
$\tau_{sd} = 1.1 \text{ N/mm}^2$ (interpolated from Table I)
$R_d = 2469 \cdot 213 \cdot 1.5 \cdot 1.1 \cdot 10^{-3} = 868$ kN

$$\frac{R_d}{1.3} \geq 1.4\, V_{g+q} - \frac{V_p}{1.3} \quad \text{(Formula 3.22.)}$$

$$\frac{868}{1.3} \geq 1.4 \cdot 719 - \frac{440}{1.3}$$

$\rightarrow 668 \geq 1006 - 338 = 668$
Condition 3.22. satisfied.

8.2.3. Checks
– **Flexural failure:**
a) Increase in prestressing force:
- **Longitudinal direction:**
Edge span: Cable position:

$$h_p = \frac{7.20^2}{8.40^2} \cdot 147 \cdot \frac{4.20^2}{3.87^2} = 127 \text{ mm (Fig. 64)}$$

$$d_p = 240 - [(120 + 213) \cdot \frac{1}{2} - 127)] = 200 \text{ mm}$$

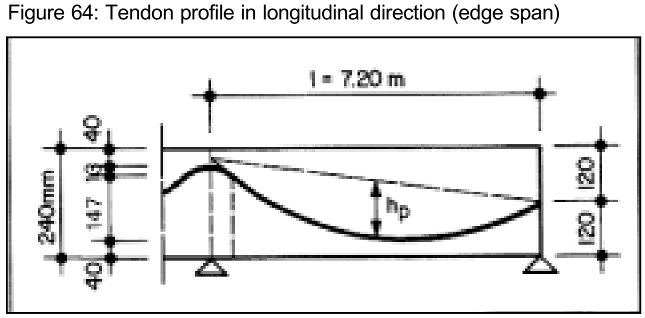
$\Delta l = 0.075 \cdot d_p$ (Formula 3.18.)
$= 0.075 \cdot 0.20 = 0.015$ m

$$\Delta\sigma_p = \frac{\Delta l}{L} \cdot E_p = \frac{0.015}{48.00} \cdot 1.95 \cdot 10^5 = 61 \text{ N/mm}^2$$

Internal span:
$\Delta l = 0.05 \cdot (0.025 \cdot l + 2 \cdot h_p)$ (Formula 3.19.)
$= 0.05 \cdot (0.025 \cdot 8.40 + 2 \cdot 0.173)$
$= 0.0278$ m

$$\Delta\sigma_p = \frac{0.0278}{48.00} \cdot 1.95 \cdot 10^5 = 113 \text{ N/mm}^2$$

Figure 64: Tendon profile in longitudinal direction (edge span)



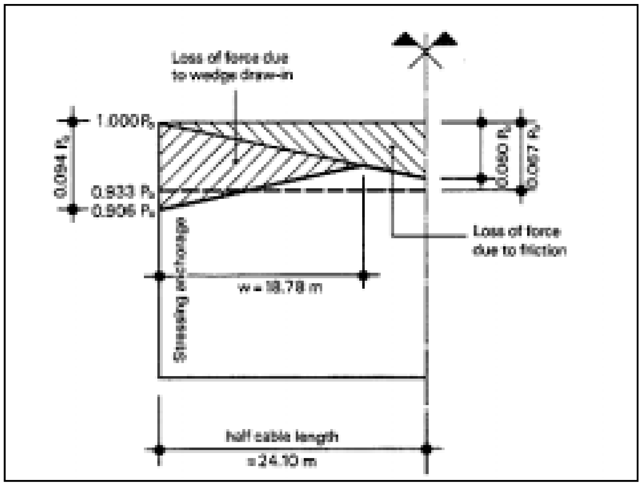


Figure 65: influence of wedge draw-in

- **Transverse direction:**
  Edge span: Cable position:

$$h_p = \frac{6.60^2}{7.80^2} \cdot 138 \cdot \frac{3.90^2}{3.67^2} = 118\,\text{mm}$$

$$d_p = 240 - [(120+204) \cdot \frac{1}{2} - 118] = 196\,\text{mm}$$

$$\Delta\sigma_p = \frac{0.0147}{24.20} \cdot 1.95 \cdot 10^5 = \mathbf{118\ N/mm^2}$$

Internal span:

$$\Delta l = 0.05 \cdot (0.025 \cdot 7.80 + 2 \cdot 0.164) = 0.0262\,\text{m}$$

$$\Delta\sigma_p = \frac{0.0262}{24.20} \cdot 1.95 \cdot 10^5 = \mathbf{211\ N/mm^2}$$

b) Check in longitudinal direction:
Cable equipped with stressing anchorages at each end and stressed to 0.75 f_{pu} at each end.

Influence of wedge draw-in of 6 mm (Fig. 65):

$$w = \sqrt{\frac{\Delta l_c \cdot E_p \cdot A_p}{\Delta p}}$$

$\Delta l_c = 0.006$ m

$E_p = 1.95 \cdot 10^8$ kN/m²

Losses due to friction: 2.5% per 10 m (Chapter 4.1.3.)

$$\frac{\Delta p}{A_p} = 0.0025 \cdot 0.75 \cdot f_{pu}$$

$$\rightarrow w = \sqrt{\frac{0.006 \cdot 1.95 \cdot 10^8}{0.75 \cdot 1770 \cdot 10^3 \cdot 0.0025}} = 18.78\,\text{m}$$

Losses:
x = 0 m $\quad \Delta P = 2w \cdot 0.0025\ P_o = 0.094\ P_o$
x = 18.78 m $\quad \Delta P = w \cdot 0.0025\ P_o = 0.047\ P_o$
x = 24.10 m $\quad \Delta P = 24.10 \cdot 0.0025\ P_o = 0.060\ P_o$

$$\text{Average}\quad \Delta P = \frac{\frac{1}{2} \cdot 18.78 \cdot 0.094 + \frac{1}{2} \cdot 24.10 \cdot 0.060}{24.10} \cdot P_o = 0.067\ P_o$$

Span I (Internal span):
Ultimate strengths:
Span:
no ordinary reinforcement

$\sigma_{pm} = 1770 \cdot 0.75 \cdot 0.933 \cdot 0.9 \quad = 1115$ N/mm²
(Factor 0.9 ≙ 10% long-term losses)
$\Delta\sigma_p = \quad = 113$ N/mm²
$\sigma_{pu} = \sigma_{pm} + \Delta\sigma_p \quad = 1228$ N/mm²
(≙ 0.694 f_{pu})

$$Z_p = \frac{18 \cdot 1228 \cdot 146 \cdot 10^{-3}}{7.80} = 414\ \text{kN/m}$$

$$x_c = \frac{Z_p}{f_{cd}} = \frac{414}{21 \cdot 10^3} = 0.020\,\text{m}$$

$$\mathbf{m_u = 414 \cdot (0.200 - 0.010) = 78.7\ kNm}$$

Column:
ordinary reinforcement 7 ∅ 12 + 6 ∅ 12
→ 791 + 678 = 1469 mm²

$$A_s = \frac{1469}{7.80} = 188\ \text{mm}^2/\text{m}$$

$$Z_s = A_s \cdot f_{sy} = 188 \cdot 460 \cdot 10^{-3} = 86.5\ \text{kN/m}$$

$$x_c = \frac{Z_s + Z_p}{f_{cd}} = \frac{86.5 + 414}{21 \cdot 10^3} = 0.0238\,\text{m}$$

$$m_{uc} = Z_s \cdot (d_s - \frac{x_c}{2}) + Z_p \cdot (d_p - \frac{x_c}{2})$$

$$= 86.5 \cdot (0.207 - 0.012) + 414 \cdot (0.200 - 0.012)$$

$$\mathbf{m_{uc} = 94.7\ kNm}$$

Ultimate load:

$$(g+q)_u = \frac{8}{l^2} \cdot m_u \cdot (1 + \lambda) \quad \text{(Formula 3.8.)}$$

$$\lambda = \frac{94.7}{78.7} = 1.20$$

$$(g+q)_u = \frac{8}{8.40^2} \cdot 78.7 \cdot (1 + 1.2) = 19.6\ \text{kN/m}$$

$$\gamma = \frac{19.6}{11} = 1.78 < 1.80$$

→ Increase of ordinary reinforcement:
7 ∅ 12 + 8 ∅ 12 → 791 + 904 = 1695 mm²

$$A_s = \frac{1695}{7.80} = 217\ \text{mm}^2/\text{m}$$

$$Z_s = 217 \cdot 460 \cdot 10^{-3} = 99.8\ \text{kN/m}$$

$$x_c = \frac{99.8 + 414}{21 \cdot 10^3} = 0.0244\,\text{m}$$

$$m_{uc} = 99.8 \cdot (0.207 - 0.012) + 414 \cdot (0.200 - 0.012)$$

$$\mathbf{m_{uc} = 97.3\ kNm}$$

$$\lambda = \frac{97.3}{78.7} = 1.24 \rightarrow \gamma = \frac{2.24}{2.20} \cdot 1.78 = 1.81 > 1.80$$

Required safety thus achieved.

Span II (Edge span):
Ultimate strengths:
Span:
ordinary reinforcement $A_s = 162$ mm²/m
$Z_s = 162 \cdot 460 \cdot 10^{-3} = 75$ kN/m
$\sigma_{pm} = 1115$ N/mm²
$\Delta\sigma_p = \quad 61$ N/mm²
$\sigma_{pu} = 1176$ N/mm²

$$Z_p = \frac{18 \cdot 1176 \cdot 146 \cdot 10^{-3}}{7.80} = 396\ \text{kN/m}$$

$$x_c = \frac{75 + 396}{21 \cdot 10^3} = 0.0224\,\text{m}$$

$$m_u = 75 \cdot (0.220 - 0.011) + 396 \cdot (0.200 - 0.011)$$

$$\mathbf{m_u = 90.5\ kNm}$$

Column:
ordinary reinforcement $A_s = 217$ mm²/m
$Z_s = 99.8$ kN/m

$Z_p = 396$ kN/m
$x_c = 0.0236$ m
$m_{uc} = 99.8 \cdot (0.207 - 0.012) + 396 \cdot (0.200 - 0.012)$
$\mathbf{m_{uc} = 93.9}$ **kNm**

Ultimate load:

$$(g+q)_u = \frac{8}{l^2} \cdot m_u \cdot (1 + \frac{\lambda}{2}) \quad \text{(Formula 3.7.)}$$

$$\lambda = \frac{93.9}{90.5} = 1.04$$

$$(g+q)_u = \frac{8 \cdot 90.5}{7.20^2} \cdot (1 + 0.52) = 21.2 \text{ kN/m}$$

$$\gamma = \frac{21.2}{11} = 1.93 > 1.80$$

Required safety thus achieved.

c) Check in transverse direction:
Cables equipped at one end with stressing anchorages and stressed there to $0.75 f_{pu}$. Average loss due to friction and wedge draw-in same as in longitudinal direction, since the transverse cables are of practically half the length of the longitudinal cables.

Span I:
Ultimate strengths:
Span:
no ordinary reinforcement

$\sigma_{p\infty} = 1115$ N/mm²
$\underline{\Delta\sigma_p = \;\; 211 \text{ N/mm}^2}$
$\sigma_{pu} = 1326$ N/mm² ($\hat{=} 0.749 f_{pu}$)

$$Z_p = \frac{18 \cdot 1326 \cdot 146 \cdot 10^{-3}}{8.40} = 415 \text{ kN/m}$$

$$x_c = \frac{415}{21 \cdot 10^3} = 0.0198 \text{ m}$$

$\mathbf{m_u = 415 \cdot (0.200 - 0.010) = 78.9}$ **kNm**

Column:
ordinary reinforcement 7 Ø 12 + 8 Ø 12
→ 791 + 904 = 1695 mm²

$$A_s = \frac{1695}{8.40} = 202 \text{ mm}^2/\text{m}$$

$Z_s = 202 \cdot 460 \cdot 10^{-3} = 92.9$ kN/m
$Z_p = 415$ kN/m

$$x_c = \frac{92.9 + 415}{21 \cdot 10^3} = 0.0242 \text{ m}$$

$m_{uc} = 92.9 \cdot (0.219 - 0.012) + 415 \cdot (0.190 - 0.012)$
$\mathbf{m_{uc} = 93.1}$ **kNm**

Ultimate load:

$$\lambda = \frac{93.1}{78.9} = 1.18$$

$$(g+q)_u = \frac{8 \cdot 78.9}{7.80^2} \cdot (1 + 1.18) = 22.6 \text{ kN/m}$$

$$\gamma = \frac{22.6}{11} = 2.05 > 1.80$$

Required safety achieved.

Span III (Edge span):
Ultimate strengths:
Span:
ordinary reinforcement $A_s = 162$ mm²/m
$Z_s = 75$ kN/m

$\sigma_{p\infty} = 1115$ N/mm²
$\underline{\Delta\sigma_p = \;\; 118 \text{ N/mm}^2}$
$\sigma_{pu} = 1233$ N/mm²

$$Z_p = \frac{18 \cdot 1233 \cdot 146 \cdot 10^{-3}}{8.40} = 386 \text{ kN/m}$$

$$x_c = \frac{75 + 386}{21 \cdot 10^3} = 0.0220 \text{ m}$$

$m_u = 75 \cdot (0.220 - 0.011) + 386 \cdot (0.200 - 0.011)$
$\mathbf{m_u = 88.6}$ **kNm**

Column:
ordinary reinforcement $A_s = 202$ mm²/m
$Z_s = 92.9$ kN/m
$Z_p = 386$ kN/m

$$x_c = \frac{92.9 + 386}{21 \cdot 10^3} = 0.0228 \text{ m}$$

$m_{uc} = 92.9 \cdot (0.219 - 0.012) + 386 \cdot (0.190 - 0.012)$
$\mathbf{m_{uc} = 87.9}$ **kNm**

Ultimate load:

$$\lambda = \frac{87.9}{88.6} = 0.99$$

$$(g+q)_u = \frac{8 \cdot 88.6}{6.60^2} \cdot (1 + 0.49) = 24.2 \text{ kN/m}$$

$$\gamma = \frac{24.2}{11} = 2.20 > 1.80$$

Required safety achieved.

Span IV + Cantilever:
Cable position: The eccentricity e_k results from equating the transverse components in the cantilever and in the internal span (Fig. 66):

$$e_k = \frac{l_k^2 \cdot (4 \cdot e_f + 2 \cdot e_c)}{l^2 - 2 \cdot l_k^2}$$

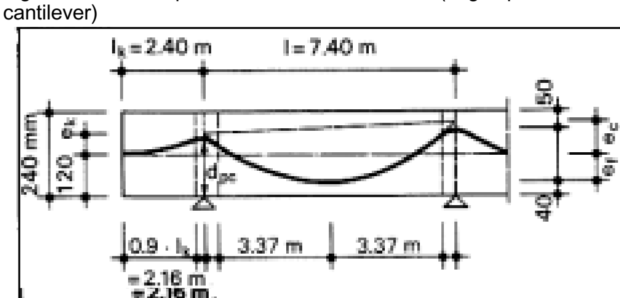
$e_f = 0.08$ m
$e_c = 0.204 - 0.120 = 0.084$ m
$l = 7.40$ m
$l_k = 2.40$ m

$$e_k = \frac{2.40^2 \cdot (4 \cdot 0.08 + 2 \cdot 0.084)}{7.40^2 - 2 \cdot 2.40^2} = 0.065 \text{ m}$$

$$d_{pc} = \frac{0.065 \cdot 2.16^2}{2.40^2} \cdot \frac{1}{0.9} + 0.12 = 0.178 \text{ m}.$$

Ultimate strength:
Span:
ordinary reinforcement $A_s = 162$ mm²/m
Z_s = 75 kN/m
Z_p = 386 kN/m
m_u = 88.6 kNm
internal column:
m_{uc} = 87.9 kNm

Figure 66: Tendon profile in transverse direction (edge span and cantilever)



Ultimate load:

$$\lambda = \frac{87.9}{88.6} = 0.99$$

$$(g+q)_u = \frac{8}{l^2} \cdot \left[m_u \cdot (1 + \frac{\lambda}{2}) + \frac{1}{2} \cdot m^k_{min} \right] \quad \text{(Formula 3.8.)}$$

$$m^k_{min} = \frac{(g+g_B) \cdot l^2_k}{2} = \frac{(6+1) \cdot 2.40^2}{2} = 20.2 \text{ kNm}$$

$$(g+q)_u = \frac{8}{7.40^2} \cdot \left[88.6 \cdot 1.49 + \frac{1}{2} \cdot 20.2 \right] = 20.8 \text{ kNm}$$

$$\lambda = \frac{20.8}{11} = 1.89 > 1.80$$

Required safety achieved.

- Punching shear:
  Column 1:
  The check performed in the preliminary design (Chapter 8.2.2.) is still valid.

  Column 2 (Edge column):
  Bottom reinforcement perpendicular to the edge for tension force $Z_a = 1.8 \cdot V_{g+q} - P \cdot \sin\alpha$

$$V_{g+q} = \frac{8.40 + 7.20}{2} \cdot \frac{1}{4} \cdot 6.60 \cdot 11 = 142 \text{ kN}$$

$P = 9 \cdot 181 \cdot 0.9 = 1466$ kN; $\sin\alpha = 0.060$

$P \cdot \sin\alpha = 88$ kN

$$A_s = \frac{Z_a}{f_{sy}} = \frac{1.8 \cdot 142 - 88}{460 \cdot 10^{-3}} = 365 \text{ mm}^2$$

Chosen: 4 ∅ 12 mm ($A_s = 452$ mm²)

- Limitation of cracks:
  Internal spans:
  assured by lateral membrane compressive forces

  Edge spans:

  $\rho_s \geq 0.15 - 0.50 \cdot \rho_p$ (Formula 4.2.)

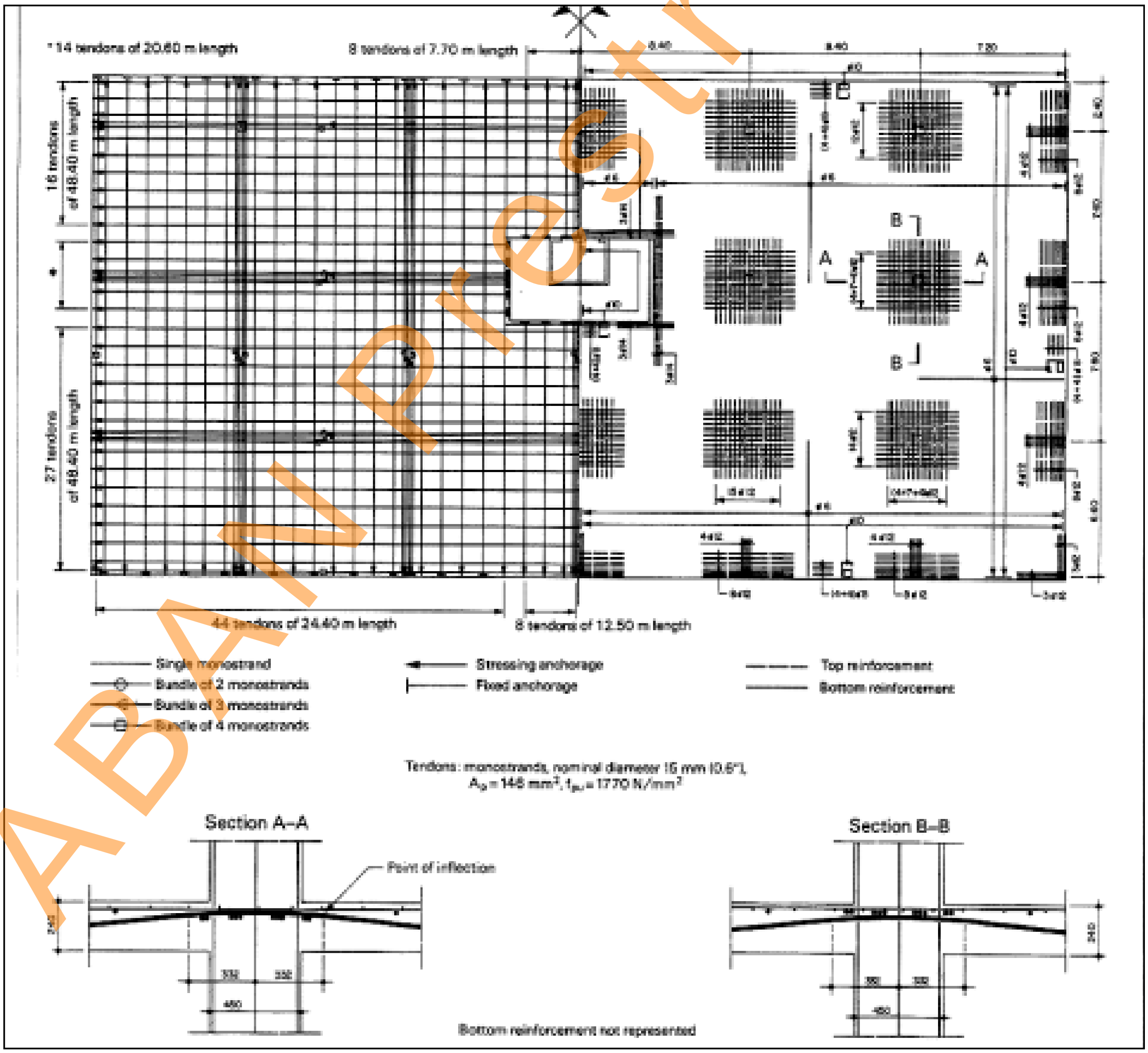
$$\rho_p = \frac{18 \cdot 146}{200 \cdot 8400} = 0.156\%$$

$\rho_s \geq 0.15 - 0.50 \cdot 0.156 = 0.072\%$

$\rho_s = 0.074\% > 0.072\%$

Adopted reinforcement sufficient.

Figure 67: Tendon and reinforcement layout drawing



– Deflections:
Determining span V

$E_c^l = \frac{E_c}{3}$ (Formula 4.5.)

$E_c = 19000 \cdot \sqrt{350} \cdot 10^{-1} = 35546$ N/mm² (according to [40])
$E_c^l = 11849$ N/mm²

Remaining load (g+q) –u:
longitudinally: $u = 0.9 \cdot 8.34 = 7.51$ kN/m²
transversely: $u = 0.9 \cdot 8.46 = 7.61$ kN/m²
average: $u = 7.56$ kN/m²
$\rightarrow$ (g+q) –u = 11.0 – 7.56 = 3.44 kN/m²

$$a = \frac{[(g+q) - u] \cdot (l_1^4 + l_2^4)}{184.6 \cdot E_c^l \cdot I}$$

$$= \frac{3.44 \cdot (7.20^4 + 7.40^4) \cdot 12}{184.6 \cdot 11849 \cdot 0.24^3 \cdot 10^3} = 0.0078 \text{ m}$$

$$\frac{l_{min}}{a} = \frac{7.20}{0.0078} = 923$$

Deflection is very small.

8.2.4. Steel quantities

– Prestressing steel:

43	monostrands of 48.40 m length =	2081.2 m
2×14	monostrands of 20.60 m length =	576.8 m
88	monostrands of 24.40 m length =	2147.2 m
16	monostrands of 12.50 m length =	200.0 m
16	monostrands of 7.70 m length =	123.2 m
		5128.4 m

5128.4 · 1.10 = 5641.2 kg
Slab area: 48.40 · 24.40 = 1181 m²

Content of prestressing steel: 4.8 kg/m²

– Ordinary reinforcement:
Top reinforcement above columns, Ø 12 mm:
Bar length: according to (3.16.) and (3.17.)
$= \Sigma l_{s\,min} + 2 \cdot 65 Ø$

longitudinally:
Bar length = 2.71 + 1.56
= 4.27 m → chosen: 4.30 m
[5 × 12 + 4 × 15 + 5 × 14 + 5 × 8] · 4.30 = 989 m

transversely:
Bar length = 2.44 + 1.56 = 4.00 m
[14 × 15 + 6 × 8] · 4.00 = 1032 m

989 + 1032 = 2021 m
2021 · 0.888 = 1795 kg

Bottom reinforcement over columns, perpendicular to the edge, Ø 12 mm:
Bar length: chosen to 3.50 m

11 × 4 × 3.50 m = 154.0 m
2 × 6 × 3.50 m = 42.0 m
196.0 m

196.0 · 0.888 = 174 kg

Bottom reinforcement in span, Ø 6 mm:

$\frac{48.40}{0.175} = 276.6 \rightarrow 277 \times 6.60$ m = 1828.2 m

$\frac{48.40-7.20}{0.175} = 235.4 \rightarrow 235 \times 7.40$ m = 1739.0 m

$\frac{7.20}{0.175} = 41.1 \rightarrow 42 \times 5.30$ m = 222.6 m

$\frac{24.40}{0.175} = 139.4 \rightarrow 140 \times 7.20$ m × 2 = 2016.0 m
5805.8 m

5805.8 · 0.222 = 1289 kg

Elevator core, Ø 14 mm, top and bottom:
3 × 10.20 m = 30.6 m
2 × 3 × 8.30 m = 49.8 m
2 × 3 × 2.50 m = 15.0 m
95.4 m

95.4 · 1.21 = 115 kg

Edge reinforcement, Ø 8 + Ø 10 mm:
Longitudinal bars Ø 8 mm:
[2 · (48.40 + 24.40) + 2 · (7.20 + 4.20)] × 8 = 1347.2 m
1347.2 · 1.15 · 0.395 = 612 kg

Stirrups Ø 10 mm: 570 pieces
570 × 1.25 m = 712.5 m
712.5 · 0.617 = 440 kg

Σ = 1795 + 174 + 1289 + 115 + 612 + 440 = 4425 kg

Content of ordinary reinforcement: 3.75 kg/m²
(support chairs and bars for fixing the tendons not included)

9. Completed structures

9.1. Introduction

In Chapters 9.2. to 9.11. ten projects are described in which post-tensioned slabs were used. They comprise structures covering a wide range of applications and geographical conditions. The post-tensioning in some of the slabs is bonded, in others unbonded. Thus a good overall view is obtained of the great variety of possible applications of post-tensioned slabs. In addition, the sequence of the descriptions is chronological, so that it is possible to follow the course of development over the last eight years. In Chapter 9.12. the main technical data of the ten structures are summarized in a table in order to enable an easy comparison.

9.2. Orchard Towers, Singapore

Client	Golden Bay Realty (Pte.) Ltd., Singapore
Architect	Chng Heng Tat & Associates, Singapore
Engineer	T.H. Chuah & Associates, Singapore
Contractor	Lian Hup Construction Co. Pte. Ltd., Singapore
Post-tensioning	VSL Systems Pte. Ltd., Singapore
Years of construction	1972-74

Introduction
This high-rise project consists of two similar building complexes. Each comprises a more or less flat, rectangular lower section and a central, 24-storey block virtually square in plan. The front block contains spaces for shops and offices. The seven lower storeys of the rear block contain car parking areas, with flats in the multi-storey building above (Fig. 68).

Structural arrangement
In the front block the colums are generally arranged in a grid of 6.85 x 6.40 m. The slabs are **flat**, 180 mm thick and post-tensioned in both directions. In the storeys containing

Figure 68: The Orchard Towers shortly before completion

shops the slabs cantilever out beyond the outermost columns. In this region it was possible to keep within the depth specified by the architect for the load-bearing structure thanks to the use of post-tensioning. In the upper storeys the slabs are strengthened with post-tensioned edge beams, which assist in supporting the heavy facade cladding.

The column spacing in the rear building in the central part of the low structure and in the storeys of the high-rise section is 8.25 m in both directions. In the low structure the most economical arrangement proved to be a combined floor structure, namely **low main beams** in the transverse direction **and thin flat slabs** in the longitudinal direction. The depth of the slabs is 150 mm and that of the beams 380 mm. By the use of this shallow structural depth it was possible, without changing the overall height of the building, to incorporate a complete additional storey for car parking.

The slabs of the rear high-rise building are **flat**. Their thickness ranges from 150 to 200 mm. They are post-tensioned in both directions. Like the slabs of the low level portion, some of them possess fairly large cantilevers.

The post-tensioning ensures the necessary limitation of deflections. As a result, problems such as those associated with service pipes etc. were largely eliminated. The advantages of post-tensioning in respect of watertightness of the concrete become evident in the roof slabs.

Construction

The slabs of the low buildings were each constructed in two sections, a system which favoured the construction program and the course of the other work. In the high-rise slabs, the construction program provided for the erection of one storey every fourteen days. After an initial phase, it was possible to reduce this cycle to 9 days. To permit early removal of formwork and thus a rapid resumption of work on the next slab, stressing was carried out in two stages and the formwork was transferred on the fourth or fifth day after concreting, i.e. at a concrete strength higher than 21 N/mm^2 (Fig. 69).

Post-tensioning

For all the slabs, bonded tendons were used. Each cable consists of four strands ∅ 13 mm (0.5"), lying in a flat duct and fitted with VSL anchorages. The service load per cable after deduction for all losses is 440 kN. The main beams in the rear low level building, which are 1.83 m wide, each contain 6 cables. In the slab, the tendons are almost uniformly

Figure 69: View during construction

Figure 70: Plan and cable distribution in low level portion of rear block

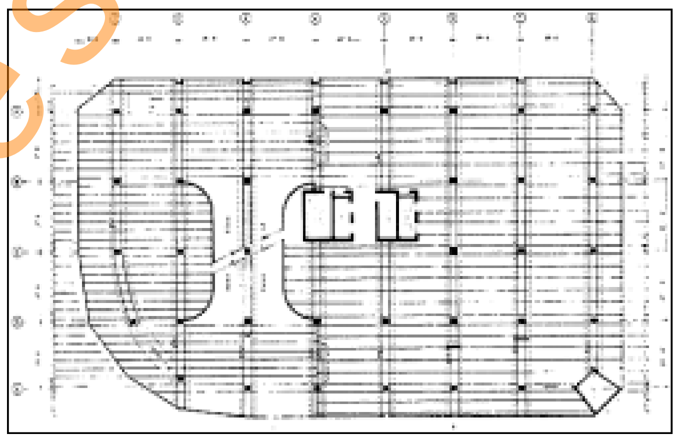


Figure 71: Plan and cable distribution in high-rise section of rear

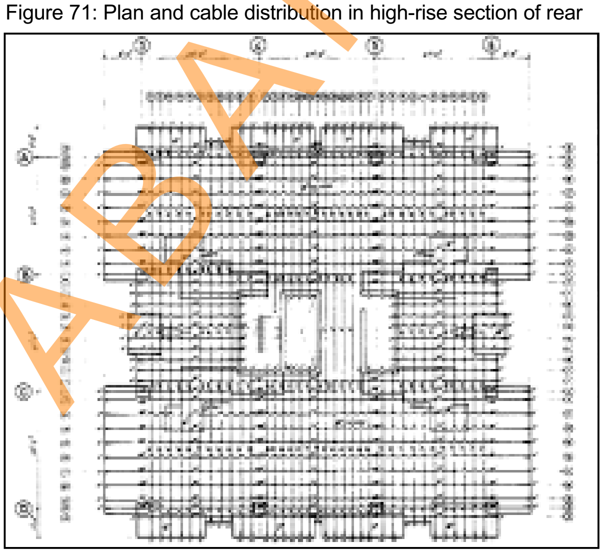
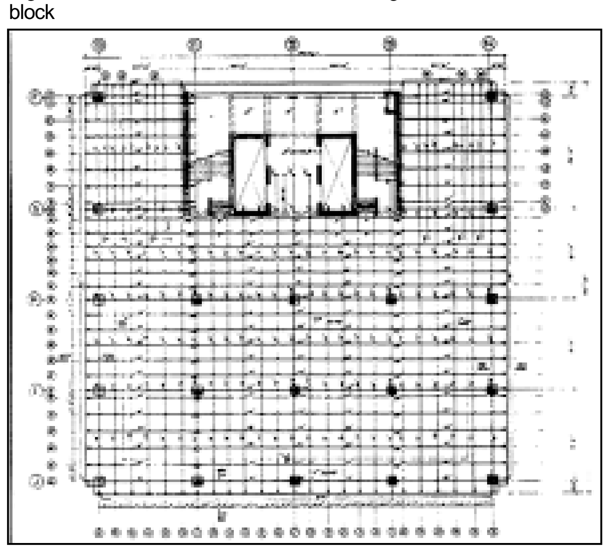


Figure 72: Plan and cable distribution in high-rise section of front block



distributed, the spacings ranging from 1.00 to 1.45 m (Fig. 70).
In the flat slabs of the high-rise building the cables are also at more or less uniform spacings in both directions (Figs. 71 and 72).
The total quantity of prestressing steel required for all the slabs was about 300 metric tons.

9.3. Headquarters of the Ilford Group, Basildon, Great Britain

Client	Ilford Films, Basildon, Essex
Architect	Farmer and Dark, London
Engineer	Farmer and Dark, London
Contractor	Th. Bates & Son Ltd., Romford
Post-tensioning	Losinger Systems Ltd., Thame
Years of construction	1974-75

Introduction

The Ilford Group has had a new Head Office building constructed at Basildon, to centralize its administration. The building comprises offices for 400 persons, a computer centre, a department for technical services (laboratories), conference rooms and a lecture hall. Building commenced in the middle of 1974. The work was completed only one year later (Fig. 73).

Structural arrangement

The building comprises three post-tensioned slabs with a total area of 7,480 m². The basement slab accounts for 1,340 m² and the two upper slabs for 3,070 m² each. The column spacing was fixed at 12 m in both directions; only the end spans are shorter (6.10 to 7.30 m). The slab over the ground floor cantilevers 0.40 m beyond the edge columns. All slabs are 300 mm thick. The internal columns are square. Their side dimension is 600 mm.
The lowest slab was designed for a live load (including partitions) of 8.5 kN/m², and the other two slabs for 5 kN/m². The detailed design was carried out on the basis of the technical report (then in draft) by the Concrete Society on «The design of post-tensioned flat slabs in buildings» (which, in the meantime, has been issued in a revised version [13]). The higher loading of the basement slab meant that it had to be strengthened at the column heads by flat drop panels of 2.60 m side dimension and 50 mm additional depth.
Post-tensioned **flat slabs** were chosen, because they proved to be **cheaper** than the originally intended, ordinarily reinforced waffle slabs of 525 mm depth. The difference in price for the slabs alone, i.e. without taking into account the effects on other parts of the structure, was more than 20% and was evident both in the concrete and in the reinforcement and formwork [44].

Construction

The slabs were divided into a total of seven sections. It was initially intended that these should be constructed at intervals of ten weeks each. By the use of sufficient formwork materials, however, the contractor was able to achieve an overlap of the cycles and thus more rapid progress. This was also necessary, because the construction programme was very tight, as Ilford had to leave their old offices by a specific date.
The concrete used had to reach a strength f_c^{28} of 41 N/mm² for the lower slab and of 30 N/mm² for the upper slabs.

Post-tensioning

The slabs were post-tensioned with monostrands ∅ **15 mm (0.6")**. The initial stressing force per strand was 173 kN, i.e. 0.70 f_{pu}. For the basement slab 70 strands were required per 12 m span and for the two upper slabs 60 strands. The strands were individually fitted with VSL anchorages; for practical reasons, however, they were combined into bundles of four.
The load balancing method [35] was used for determining the prestressing force. This force was selected so that the dead load and 10% of the live load were fully balanced by the transverse components from prestressing. Where the remainder of the live load led to tensile stresses, ordinary reinforcement was used. In the column region, stirrups were required to withstand the shear forces. This created some problems in the placing of the tendons.

Figure 73: The Headquarters of the Ilford Group

9.4. Centro Empresarial, São Paulo, Brazil

Client	LUBECA S.A. Administração e Leasing, São Paulo
Architect	Escritório Técnico J.C. de Figueiredo Ferraz, São Paulo
Engineer	Escritório Técnico J.C. de Figueiredo Ferraz, São Paulo
Contractor	Construtora Alfredo Mathias S.A., Sao Paulo
Post-tensioning	Sistemas VSL Engenharia S.A., Rio de Janeiro
Years of construction	1974-77

Introduction

The «Centro Empresarial» (the name means «Administrative Centre» is a type of office satellite town on the periphery of Sao Paulo. When completed it will comprise six multi-storey buildings, two underground car parks and a central building containing conference rooms, post office, bank branches, data processing plant and restaurants.
A start was made on the foundation work in September 1974. The first phase, i.e. approximately 2/3 of the centre, was completed at the beginning of 1977. There is at present no programme for the construction of the second stage.

Structural arrangement

The «Centro Empresarial» is divided structurally into three different parts: the multistorey office buildings, the underground car parks and the central block. Each of the high buildings comprises eleven storeys (two of which are below ground), each of 53.50 x 53.50 m area. To provide for maximum flexibility in use of the available building surfaces a **column spacing** of **15 m** was chosen. There are thus three spans of 15 m length in each direction in each slab, with a cantilever at each end of 4.25 m (Fig. 75).
The slabs had to be light, simple to construct and of minimum possible depth. For a live load of 5 kN/m², the best method of meeting these requirements was by using post-tensioning.
In order to find the most economic solution, a number of slab systems were compared: flat slab with hollow cores, one-way joisted beams, drop panel slab and **waffle slab**. The last-named type proved to be the most suitable for the multi-storey buildings. The slab depth was established at 400 mm, giving a slenderness ratio of 37.5. The slab itself is 60 mm thick, and the ribs which are spaced at 1.25 m between centres, are 170 mm wide. The main beams over columns are 2.50 m wide and give the structure great stiffness (Fig. 76).

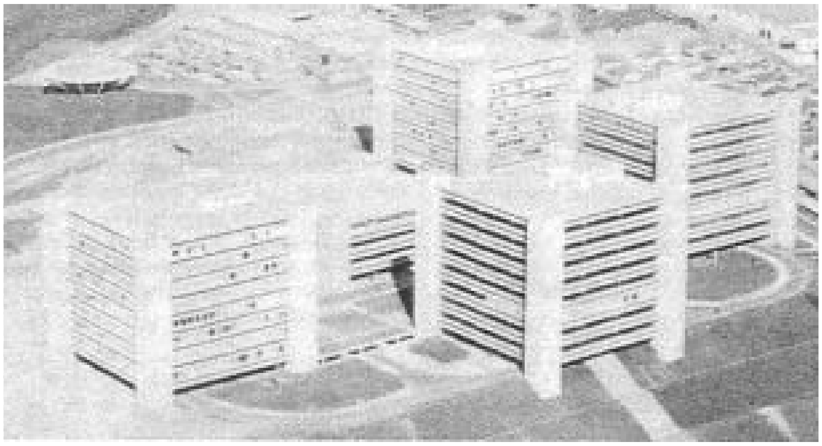


Figure 74: The Centro Empresarial (first phase)

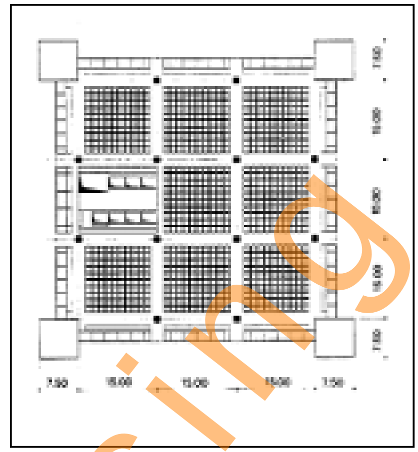


Figure 75: Plan of the multi-storey buildings

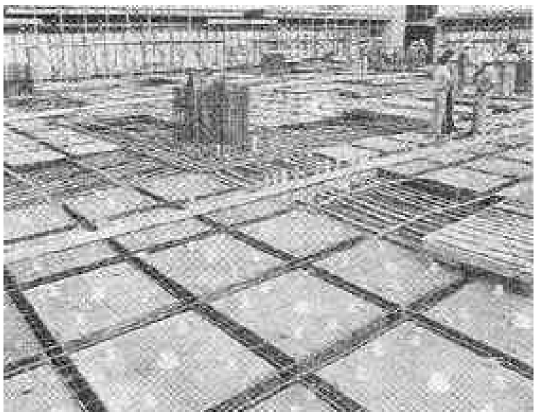


Figure 76: Waffe slab during construction

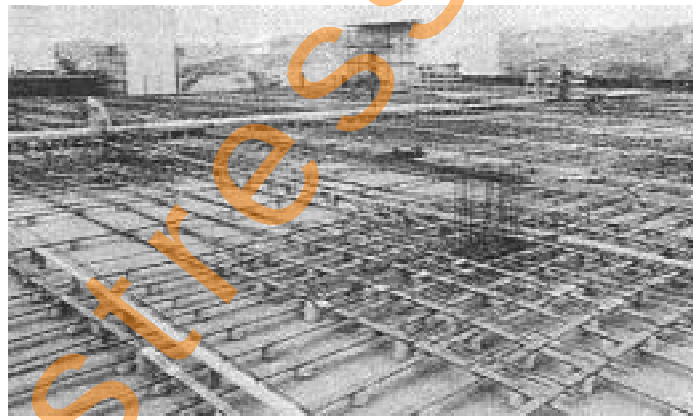


Figure 77: Flat slab during construction

The slabs of the two underground garages (four slabs each) are supported on a grid of 7.50 x 10.00 m. They are 180 mm thick **flat slabs** (Fig. 77). The uppermost slab of each garage, which has to carry a soil loading of 0.40 m, is 250 mm thick.

The building complex for the central services was designed as closely as possible along the same lines as the office towers. In the central block, ribbed slabs were adopted.

The design of the slabs was generally in accordance with the Brazilian prestressed concrete standard P-NB-116, in so far as it could be applied to post-tensioned slabs. The waffle slabs were designed on the method of equivalent frames. The flat slabs were designed by the load-balancing method.

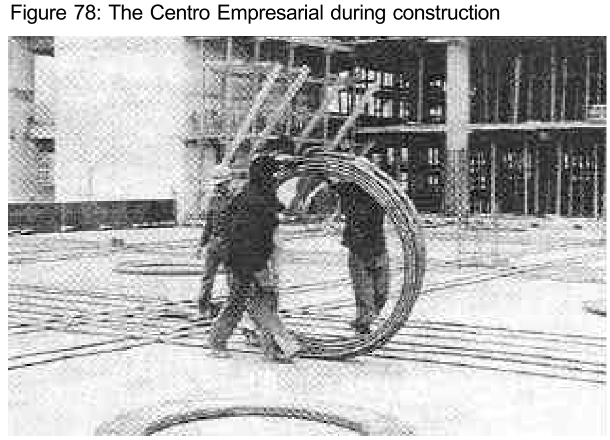
Post-tensioning

For the waffle slabs, VSL cables of type 5-4 in flat ducts (that is **bonded post-tensioning**) were used. One such cable was necessary in each rib; in each column-line beam, 22 cables of this type had to be incorporated, equivalent to approximately 70% of the total prestress. Due to this high concentration, the cables had to be placed in two layers. By the use of the flat ducts, it was possible for the maximum eccentricity over the columns to be achieved however.

The cables were prefabricated in a hall on the site. Two or three strands were simultaneously unreeled from the coil by means of a VSL push-through machine and pushed directly into the duct. The tendons then had to be stored for a shorter or longer period since the demand for cables to be built in often fluctuated appreciably. The «cable factory» supplied up to 330 metric tons of cables in the peak months. The cables were coiled up for transport from the assembly hall to the place of installation. On the formwork they were unrolled in a sequence that had previously been tested with a model (Fig. 79). It had originally been intended to apply half the prestressing force three days after concreting and the full force after seven days.

Figure 78: The Centro Empresarial during construction

Figure 78: The Centro Empresarial during construction



This procedure was later modified to full stressing of most of the tendons after six or seven days. After the stressing operation had been carried out at both ends of the cable, the protruding strands were cut off and the anchorage block-outs concreted in. The cables were then grouted.

The post-tensioning operations commenced in February 1975 and lasted 17 months. During this period, 1,670 metric tons of prestressing steel and approximately 140 metric tons of anchorages were built into the 143,500 m^2 of slabs.

9.5. Doubletree Inn, Monterey, California, USA

Client	Doubletree Inc., Phoenix, Arizona
Architect	Kivett Myers, AIA, Kansas City, Missouri
Engineer	VSL Corporation, Los Gatos, California
Contractor	Baugh Construction, Seattle, Washington
Post-tensioning	VSL Corporation, Los Gatos, California
Years of construction	1976-77

Introduction

The Doubletree Inn at Fisherman's Wharf is a hotel comprising 374 guest rooms, conference rooms, restaurants, shops and a parking structure for 420 private cars (Fig. 80).

The project almost failed to get built. The tender price for the original design, specified in reinforced concrete, was considerably above that which the client was prepared to pay. Other proposals also, including a variant involving prefabrication, were outside the stipulated limits.

The VSL Corporation was consequently commissioned to look for possible savings. It proposed that post-tensioned, in-situ **flat slabs** should be used and the earthquake forces transmitted via the walls. This resulted in a **cost reduction** of more than half a millions US $ or of 20% in terms of the cost of the concrete frame itself. The VSL Corporation was subsequently awarded the contract for developing the design in detail and for supervising the entire civil engineering construction for the hotel and car parking.

Structural arrangement

The hotel comprises 24,150 m^2 of posttensioned slabs, and the car parking 9,750 m^2. In the hotel the spans and the slab depths vary considerably. In general the ratio span/slab depth is 44 to 45. The car parking is a three-storey building of dimensions 39 x 86 m. The spans here are usually 8.28 m, and the slab depths 190 mm (Fig. 81).

The slabs were designed in accordance with the American standards UBC 1970 and ACI 318-71, in conjunction with [12]. The live load assumed in the hotel area was from 1.9 to 4.8 kN/m^2. For the car parking, a live load of 1.4 kN/m^2 was adopted.

Post-tensioning

As is general in the USA, **monostrands** ∅ 13 mm (0.5") were used for this project also. The tendons were cut to length at works and delivered to the site rolled up. They were placed either by pulling the strand from the roll or by rolling them out. Tendons with intermediate stressing anchorages at the construction joints were hung rolled up from scaffolds until they could be extended further (Fig. 82).

The service load per monostrand after deduction for all losses is 0.60 f_{pu}, i.e. 110 kN. To keep the frictional losses as low as possible, cables exceeding 30 m in length were stressed at both ends. The stressing operation was carried out at a concrete strength of 17.5 to 21.0 N/mm^2, that is at about 0.625 to 0.75 f~8.

9.6. Shopping Centre, Burwood, Australia

Client	Berbert Investment Co. Ltd., Sydney
Architect	Hely, Horne, Stuart & Perry, Milsons Point, N.S.W.
Engineer	Rankine -Hill Pty. Ltd., Sydney
Contractor	Concrete Constructions Pty. Ltd., Potts Point, N.S.W.
Post-tensioning	VSL Prestressing (Aust.) Pty. Ltd., Thornleigh, N.S.W.
Years of construction	1976-78

Introduction

Burwood is a suburb of Sydney. The shopping centre, built there between May 1976 and October 1978, predominantly serves a large department store, but also comprises 68 specialist shops and three storeys with car parking places (Fig. 83).

Figure 80: The Doubletree Inn

Figure 81: car parking of the Doubletree Inn

Figure 82: Scaffolds at construction joints with rolls of tendon

Figure 83: Main hall of the Burwood Shopping Centre

Structural arrangement

The building comprises five storeys in total. It is 103 m long and 74 m wide. All the slabs (total area 28,500 m²) are posttensioned **(bonded post-tensioning)**. The longitudinal column spacing is 4.04 - 12 · 7.90 - 4.04 m, the transverse spacing 4.65 - 8 · 8.40 m (Fig. 84). The slabs are 170 mm thick flat slabs, with **main beams** along the transverse column lines. The live load is generally 6 kN/m²).

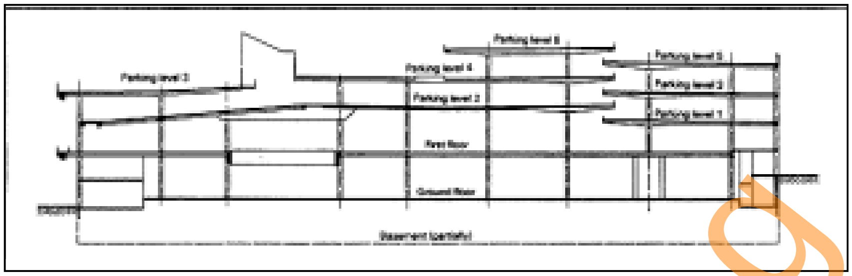


Figure 84: Cross-section

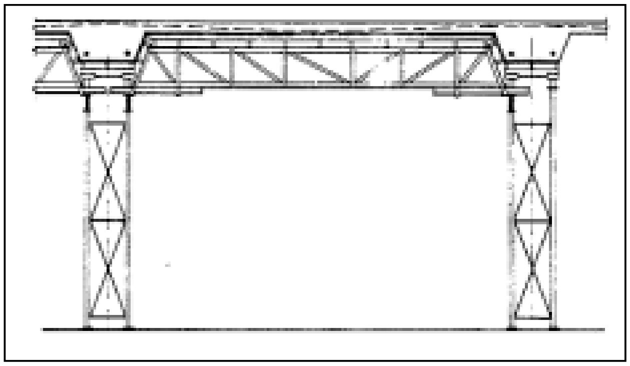


Figure 85: Formwork system

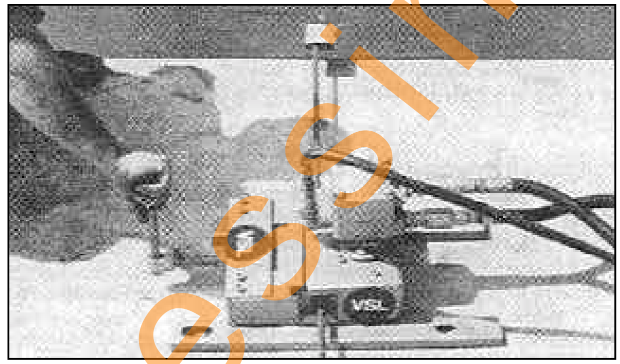


Figure 86: Pushing-through the strands

Figure 87: Positioned cables

Figure 88: Anchorage with stressed strands

Construction

Rapid speed of construction was of the utmost importance in this project. VSL Prestressing Ltd. had already been brought in at an early stage, co-operating not only in developing the design for the project but also in planning the construction sequence and programming. It was therefore possible to adapt the design to suit the posttensioning and the formwork system. For the type of slab referred to, VSL Prestressing Ltd. had developed a special formwork system, which is especially applicable to regular, flat structures. The formwork panels are so constructed that they can be easily adapted to a column grid of 8 to 12 m (Fig. 85).

The slabs for the shopping centre were constructed in a total of 28 stages. The two largest slabs, that over the basement and that over the ground floor, which both cover the entire building area, were each sub-divided into eight sections.

Post-tensioning

All the cables consist of four strands Ø13 mm (0.5") and have an ultimate strength of 736 kN. The strands were pushed into the flat ducts (Fig. 86). In each of the main beams there are four cables; in each of the spans between them there are three tendons. The post-tensioning of the slab transversely to the main beams consists of uniformly distributed cables at 1.20 m spacing (Fig. 87). The total requirement for post-tensioning steel was almost 160 metric tons.

24 hours after each concreting operation, a partial prestress was applied, i.e. one strand of each 4-strand cable was fully stressed. After 36 hours, a second strand was fully stressed in the longitudinal direction, to permit the formwork to be transferred. After 7 days all the remaining strands were stressed (Fig. 88). Grouting of the cables was carried out from one day to eight weeks after stressing.

9.7. Municipal Construction Office Building, Leiden, Netherlands

Client	Municipal Public Works of Leiden
Architect	FA. Temme, City Architect, Leiden
Engineer	Engineering Office van der Have, Rotterdam
Contractor	IBB-KondorB.V.,Leiden
Post-tensioning	Civielco B.V, Leiden
Years of construction	1977-78

Introduction

In order to centralize different services and thereby improve co-operation, the town of Leiden decided to erect a new administrative building. On May 17, 1977 the first pile was officially driven. Towards the end of 1978 the structure was completed and on February 12, 1979, i.e. exactly 50 years after the

Figure 89: The finished Municipal Construction Office Building

Figure 91: The building during construction

destruction of the historic town hall by fire, the official opening took place (Fig. 89).

Structural arrangement

The building forms a group around three sides of an inner courtyard. It comprises a basement for bicycles and four aboveground storeys (Fig. 90). Its shape in plan is quite complicated and is an expression of individualistic architecture.

The ground floor slab (area 2,000 m^2) consists of prefabricated elements supported on beams. The other slabs **(flat slabs)** were constructed of post-tensioned, in-situ concrete, **unbonded post-tensioning** being used. The column spacing in both directions is alternately 7.20 and 3.60 m. All slabs are 240 mm thick. They are strengthened in the column head regions. The live load varies, but on average is about 4 kN/m^2.

The design was based upon earlier projects involving post-tensioned slabs, since the first Dutch guidelines did not appear until early 1978. In addition, use was made of the specifications of the VB 1974 [45]. The calculations were carried out with the use of computer programmes, the finite element method being used.

Particular attention was given to the connection between slab and columns, especially at the corner columns where large stress concentrations occur. As mentioned, the slabs were therefore locally reinforced by using appropriate reinforcement.

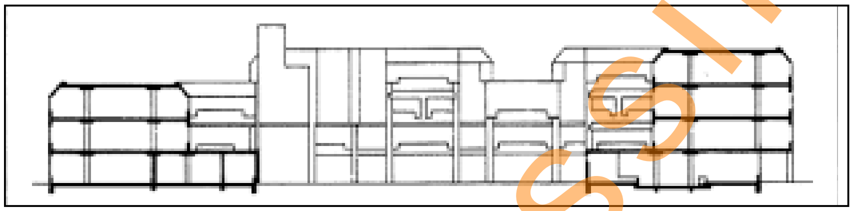


Figure 90: Section of the building

Construction

Each slab consists of three independent parts, separated by expansion joints. In construction (Fig. 91), the larger parts were sub-divided into sections of about 350 m^2 to permit rational use of the formwork. The influence of horizontal movements (stressing, creep and shrinkage) on the slabs was limited by forming the connection with the stiff cores subsequently.

The slabs were not designed for carrying the concrete weight of the slab above. This was therefore transferred in every case to two lower slabs. To achieve rapid construction, it was necessary to apply the prestress as quickly as possible. Partial stressing was carried out three days and full stressing about 14 days after concreting.

Post-tensioning

The prestressing consists of monostrands ∅ 13 mm (0.5") of 184 kN ultimate load. These were cut to length on site, fitted with anchorages and transferred in bundles by the crane onto the formwork. Placing of the monostrands and ordinary reinforcement of a 350 m^2 section required approximately three days. In total, some 6,000 m^2 of slab area was post-tensioned, requiring approximately 37 metric tons of prestressing steel.

9.8. Underground garage for ÖVA Brunswick, FR Germany

Client — Öffentliche Versicherungs-Anstalt, Brunswick

Architect — Laskowski and Schneidewind, Brunswick

Engineer — Office of Meinecke & Dr. Odewald, Brunswick

Contractor — Telge & Eppers, Brunswick

Post-tensioning — SUSPA Spannbeton GmbH, Langenfeld

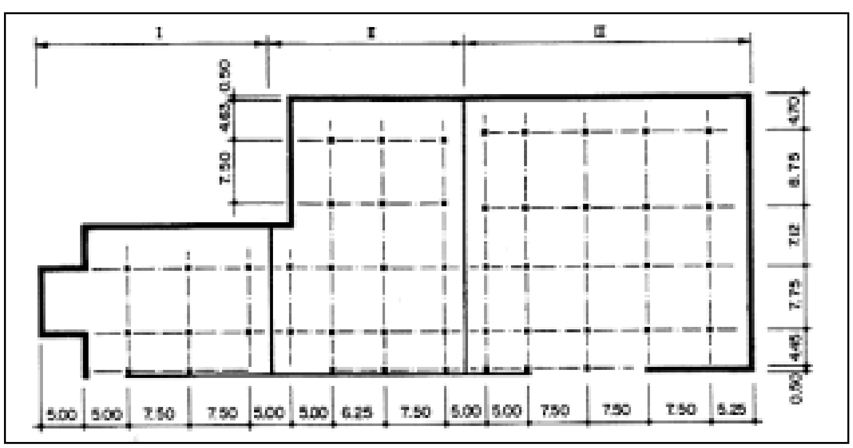
Year of construction — 1979

Introduction

In the course of extending its buildings, ÖVA Brunswick had a single-storey, underground car park for 99 private cars constructed inside already existing buildings. The roof of the structure, of area approximately 2,290 m^2, consists of a post-tensioned **flat slab**, for which **unbonded post-tensioning** was used. This was the **frist time** a partially post-tensioned flat slab with unbonded tendons was carried out **in the FR Germany** [46].

Construction of the slab took place during during the Summer of 1979. On account of the high groundwater level, the floor of the car park was to be kept as

Figure 92: Plan of the underground parking



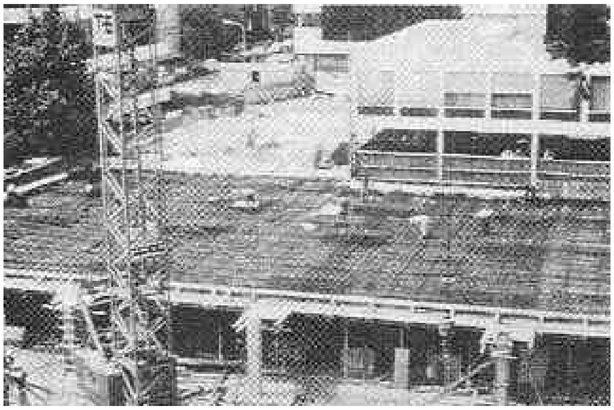


Figure 93: Section II during construction

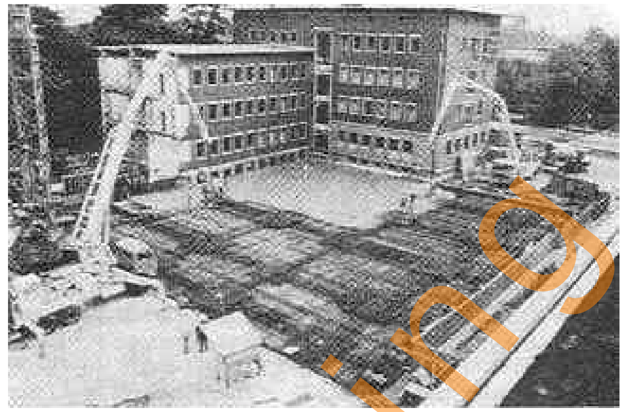


Figure 94: Section I II during concreting

high as possible, to avoid expensive watertight tanking and drainage during construction. Therefore, after an ordinarily reinforeced slab 500 mm thick had initially been designed, an alternative solution in post-tensioned concrete was developed, which provided a reduction in slab depth of 150 mm and the saving of two column axes accompanied by an increase in spans from 5.0 to 7.5 m. The post-tensioned slab was also satisfactory from the economic aspect.

Structural arrangement

The slab, of total length 86.50 m and width 18.20 and 33.50 m respectively, is divided by permanent joints into three sections. The spans of the internal bays vary longitudinally between 5.00 and 7.50 m, and transversely between 7.10 and 8.75 m. The edge spans range from 4.40 to 5.00 m. The slab depth is 350 mm (Fig. 92).

The slab was designed for a soil overburden of 0.40 m (7.5 kN/m^2) and a heavy goods vehicle of class SLW 30 (equivalent loading 11.8 kN/m^2), since the slab was located partly beneath a road. Account had to be taken of additional loads in individual spans. The design method adopted was not in accordance with DIN 4227, Part 1, the standard that was then in general use for post-tensioned structures, and it was also not yet possible to base the design upon Part 6 «Components with unbonded post-tensioning» [15], which was in preparation. A uniform balancing loading of 16.3 kN/m^2 was assumed, i.e. self-weight plus soil overburden including road pavement.

At that time there was also no general approval issued for the VSL monostrand tendon*. An agreement was therefore required for the particular case, and this was granted by the Responsible Authority for Construction of Lower Saxony, on the recommendation of the Institut für Bautechnik, Berlin.

Construction

Formwork erection, reinforcement placing and concreting were carried out for the three parts of the slab in succession (Figs. 93 and 94). The tendons were cut to length at works, fitted with the dead-end anchorage and rolled up. During placing they were unrolled, starting from the deadend anchorage, and were pushed through the stressing anchorages fixed to the formwork. Two, three or four strands were combined together into bundles.

Four or five days after concreting the strands were stressed in one stage to 0.75 f_{pu} and anchored at 0.70 f_{pu}. This steel stress exceeds the value of 0.55 f_{pu} until now generally the maximum allowed in post-tensioned concrete in the FR Germany. This value, however, is to be increased in the near future. In [15] the increased value was already adopted for unbonded post-tensioning as this method would have been at an economical disadvantage against bonded. post-tensioning [47]. After stressing the protruding ends of the strands were cut off, the stressing anchorages closed with a grease-filled plastics cover, and the block-outs filled with mortar.

Post-tensioning

The tendons used consist of monostrands ∅ 15 mm (0.6"), each of 140 mm^2 cross-sectional area and 247.8 kN ultimate load. Placing of the strands was carried out by three to four operatives. This work and the placing of the top reinforcement was carried out for an equivalent of 1,000 m^2 slab area in approximately 7 working days. 7.3 kg of prestressing steel and 18.3 kg of ordinary reinforcement were required per m^2 slab.

9.9. Shopping Centre, Oberes Murifeld / Wittigkofen, Berne, Switzerland

Client	Kleinert Geschaftshäuser AG, Berne
Architect	Joint Venture Thormann & Nussli AG, Berne / 0. Senn, Basle
Engineer	Engineering office Walder AG, Berne
Contractor	General contractor LOSAG AG, Berne Building contractor Losinger AG, Berne
Post-tensioning	VSL INTERNATIONAL LTD. (formerly Spannbeton AG)
Year of construction	1979

Introduction

The building complex serves as a shopping centre for the new development of Oberes Murifeld/Wittigkofen at the periphery of the city of Berne. It comprises various shops, a restaurant, several storage areas, a car parking hall and an office floor.

Structural arrangement

The building comprises three storeys, the two lower ones of reinforced/post-tensioned concrete and the upper in structural steel framing. The slabs over the basement and ground floor are **flat slabs with unbonded post-tensioning**. The column spacing longitudinally is 13 x 5.00 m and transversely 4.25 - 5 x 8.50 - 4.25 m. In axis 7, the slabs are divided in the transverse direction by expansion joints. In total, 4,657 m2 of slab were post-tensioned. Both concrete slabs are 240 mm thick. The slab over the basement can carry a live load of 5 kN/m^2 and that over the ground floor a live load of 3 kN/m^2. The connection between the slabs and the load-bearing walls and columns is monolithic (Figs. 95 and 96).

Construction

Each slab was constructed in three sections. Sections I and II were separated by a construction joint, sections I I and III by the expansion joint (Fig. 95). The sub-dividing made possible a rational use of formwork and **rapid construction progress.** This was of great importance, since the construction of the entire shopping centre was subject to a very tight construction schedule. 5 1/2 months after commencement of excavation the greater part of the building was to be handed over to the client. It was possible to achieve this date, thanks not least to the choice of post-tensioned flat slabs. Only 14 weeks were required for the construction of the slabs.

The average working times per slab section, after erection of formwork, were:

- 1 day for placing the bottom reinforcement, which was of mesh throughout and only required local additional reinforcement,
- 2 days for placing the tendons,
- 1 day for placing the top reinforcement.

* In the meantime this approval has been granted.

These operations were carried out with some overlap. Only 4 days after concreting full stressing could be applied. For this, a concrete strength of 22.6 N/mmz was required.
The tendons were cut to length at works, fitted with the dead-end anchorages and transported to site rolled up. Strands and bundles of the same length were identified by the same colour. At the intermediate anchorage locations, the tendons were temporarily stored resting on a 3.5 m wide formwork overhang (Fig. 97).

Post-tensioning

The choice of post-tensioning by areas gave not only the most economic solution but also that with the minimum slab thickness and minimum deflections. Monostrands Ø15 mm (0.6") of 146 mm^2 cross-section and 257.8 kN ultimate strength were used for the tendons. In total, 21 metric tons of prestressing steel, 596 stressing anchorages, 596 dead-end anchorages and 138 intermediate stressing anchorages were required. Along the colum lines, two and

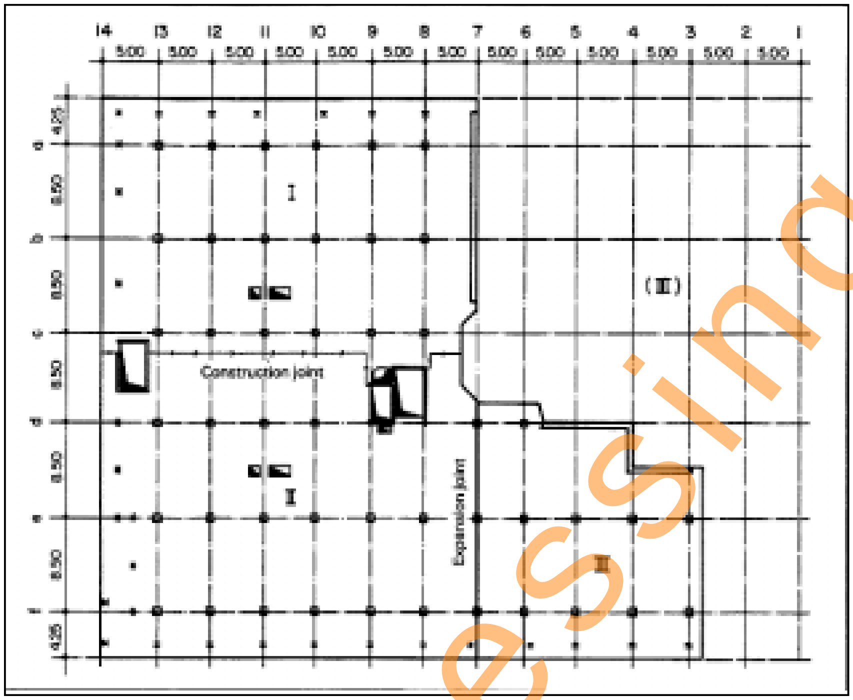


Figure 95: Plan of the slab above ground floor

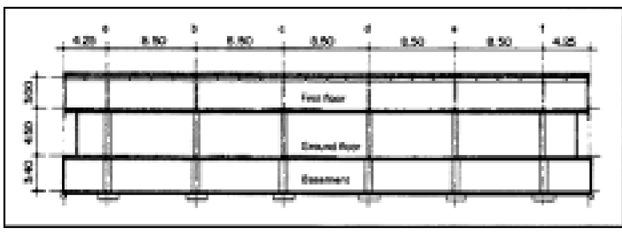


Figure 96: Cross-section (axis 10)

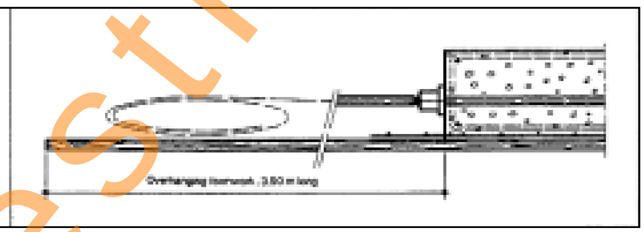


Figure 97: Storage of tendons at construction joint

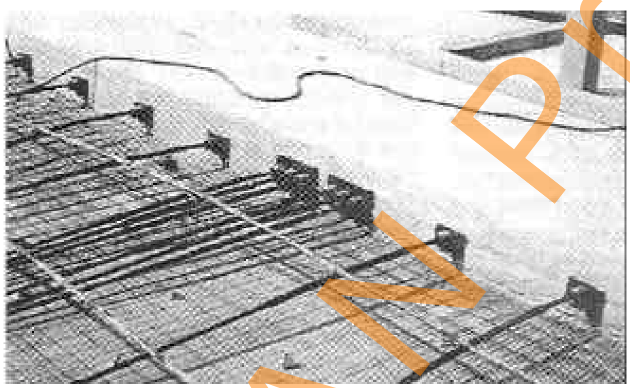


Figure 98: Monostrands with intermediate an chorages at construction joint

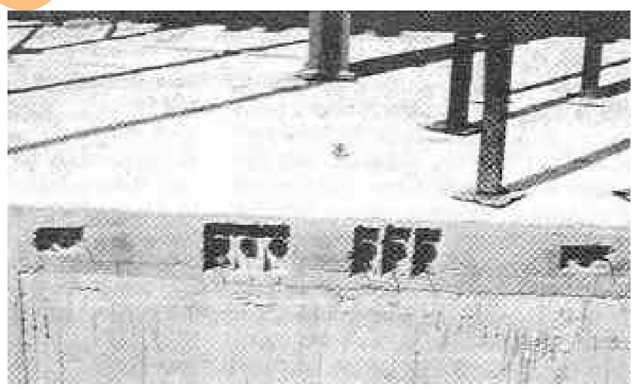


Figure 99: Anchorage block-outs before being filled whit mortar

Figure 100: Installed cables of the third section

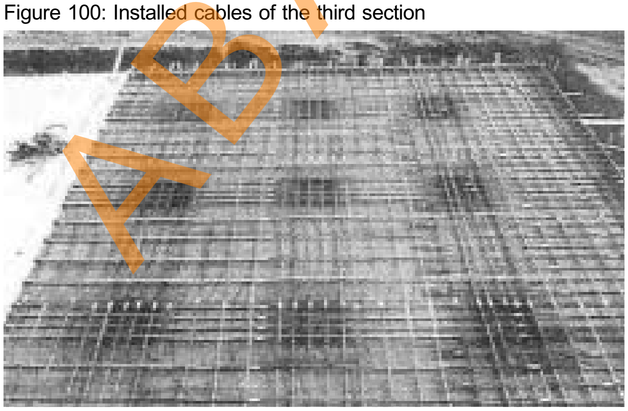
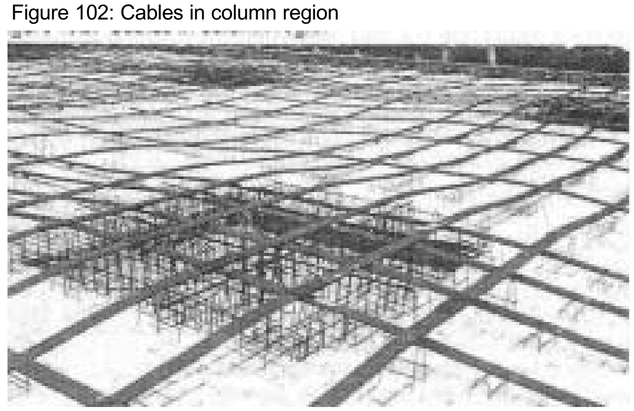


Figure 102: Cables in column region



three strands were combined into bundles. Between them, single strands are uniformly distributed (spacing approx. 0.60 and 1.00 m respectively). The requirement for prestressing steel in the lower slab was 4.7 kg/m2, in the upper 4.6 kg/m². The corresponding figures for the ordinary reinforcement are 9.5 and 7.9 kg/m² (Figs. 98 and 99).

9.10. Underground garage Oed XI I, Linz, Austria

Client Wohnungsaktiengesell-schaft, Linz
Architect Franz Reitzenstein, Salzburg
Engineer Hellmut Preisinger, Linz
Contractor Josef Pirkl & Georg Eysert, Linz
Post-tensioning Sonderbau GesmbH, Vienna
Years of construction 1979-80

Introduction

The single-storey, soil covered underground car park forms part of a development in a suburb of Linz. It provides places for approximately 110 private cars. A cost comparison prepared during optimization of the slab gave a price advantage for the post-tensioned solution over reinforced concrete. Construction was carried out between November 1979 and May 1980.

Structural arrangement

The slab is 75.30 m long and 33.90 m wide. It contains no permanent joints. Longitudinally, the spans are 7.65 - 8 x 7.50 - 7.65 m, and transversely 4.85 - 8.05 - 8.10 - 8.05 - 4.85 m. The slab is **flat**, 300 mm thick and is strengthend at each column with a square drop panel of 2.20 m side dimension and an additional 300 mm depth, since due to the high loading punching shear was the determining factor. The column dimensions are 0.25 x 0.60 m. The applied load is composed of 0.60 m backfill (10 kN/m²) and a live load of 5 kN/m². The slab and side walls are connected together monolithically.

Construction

The slab, which is **post-tensioned with bonded tendons,** was constructed in three sections. After placing of the bottom reinforcement, consisting of mesh, the ducts were made up from 10 m lengths, placed at the specified sequence and fixed to the anchorages. Pushing-through of the strand was then carried out. After they had been assembled, the tendons were equipped with the necessary supports beneath (concrete blocks and stirrups). The tendons were fixed laterally by attaching the supports to the bottom mesh reinforcement (Fig. 100). Finally, the top reinforcement above the columns was placed. Ten days after concreting the cables were stressed.

Post-tensioning

Since there was not sufficient time to go through an approval procedure for unbonded tendons, grouted VSL cables, type 5-4, of 699 kN ultimate strength each were used. The four strands of each tendon are laid in a flat duct. The transverse cables have stressing anchorages at both ends. The continuous cables in the longitudinal direction also have stressing anchorages at both ends, with couplers between them at each construction joint. In the two end spans, additional cables were necessary; these have buried dead-end anchorages type H (Figs. 101 and 102).

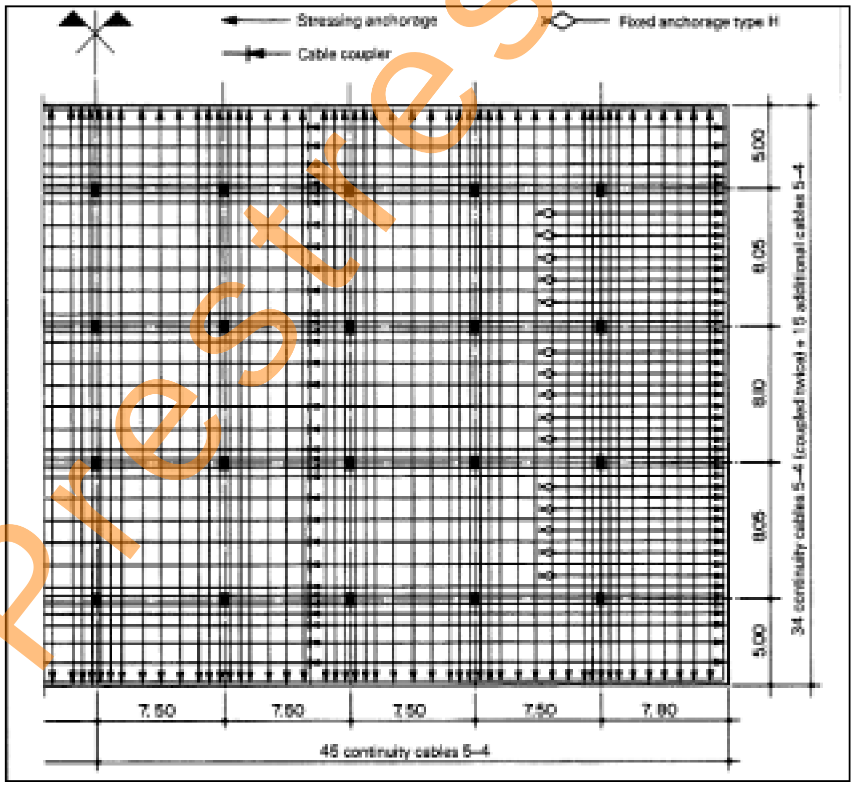


Figure 101: Cable layout drawing

9.11. Multi-storey car park, Saas-Fee, Switzerland

Client Community of Saas-Fee
Engineer Schneller+Schmidhalter+ Ritz, Brig
Contractor Anthamatten&Kaibermatten AG, Saas-Fee
Post-tensioning VSL INTERNATIONAL LTD. (formerly Spannbeton AG)
Years of construction 1979-80

Introduction

Saas-Fee, a well-known Summer and Winter resort in the Alps of the Valais, is about 1800 m above sealevel and can only be reached by road. The ever-increasing number of holiday-makers who bring their own cars and the shortage of parking places especially in winter led the local community authorities to construct an 8-storey car park containing approximately 950 parking spaces (Fig. 103). Construction commenced in October 1979. The construction period was about one year.

Two different supporting systems were specified in the invitation for tenders. One variant consisted of a completely prefabricated solution with in-situ concrete over the slab elements. The other proposal was based upon a monolithic in-situ concrete design with post-tensioned **flat slabs.** This proved to be economically and technically superior.

Structural arrangement

The car park is an open, unheated building with ventilation. It is 83.3 m long, 34.8 m wide and 25.5 m high. The column spacing is a uniform 7.50 m in the longitudinal direction. Transversely, the spacings are 4.50 – 7.60 – 2x4.765 – 7.60 – 4.50 m. The core containing the staircase and the lift shafts is located virtually at the centre. This core, together with cross-beams connecting the slabs to the end supporting walls, assures the horizontal stability of the structure (Figs. 104 to 106).

In the end spans and in the central span, the slabs are horizontal; in between, they have a 4.5% gradient in the longitudinal direction, thus serving also as ramps. This form of structure results in a separation over 2 x 4 spans along the longitudinal axis of each floor.

The seven lower slabs are 200 mm thick. They are designed for a live load of 2 kN/m^2. The loading of the roof slab is composed at a maximum of the snow load of 11.5 kN/mz and a roof garden of 4 kN/m^2. The thickness of the roof slab is therefore 250 to 400 mm. The high loading also necessitated strengthening at the column heads. It was possible to dispense with expansion joints in all the slabs.

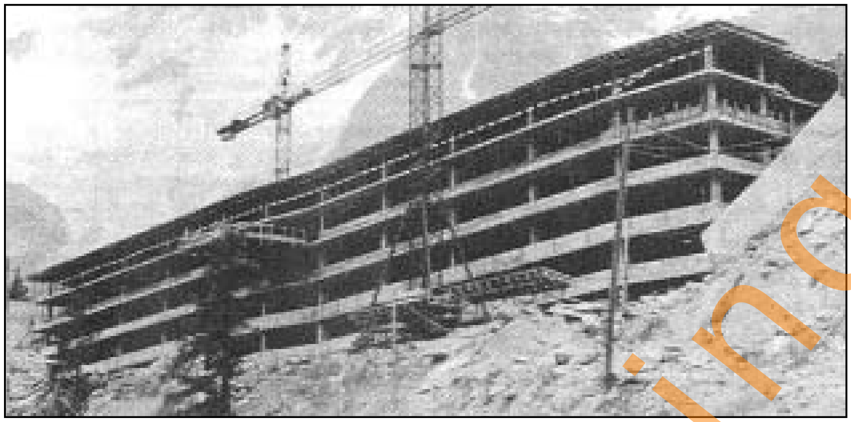


Figure 103: The car park during construction

Construction

Due to its high elevation, Saas-Fee has a long winter season in which no construction work is possible. In addition, sudden cold spells and falls of snow must be expected in spring and autumn. Only the period from the end of May to the middle of October 1980 was therefore available for building the main structure. This meant that on average one half slab (area 1,450 m^2) together with the associated columns and walls had to be erected each week. Furthermore, due to reasons associated with formwork and post-tensioning, the half slabs at the uphill side always had to be two storeys in advance (Fig. 107). Fig. 108 shows the construction programme of two half slabs situated on adjacent levels.

The required minimum concrete strength for stressing was reached normally three days after concreting. One half slab could therefore be fully stressed each Tuesday and the formwork then stripped from it.

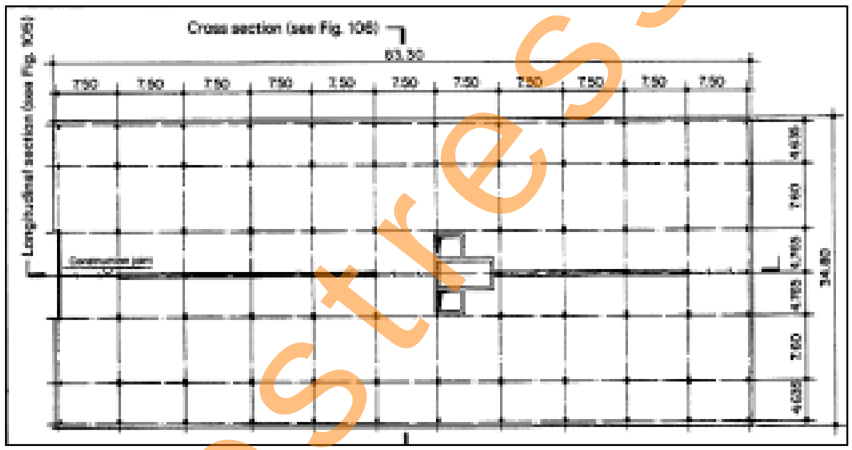


Figure 104: Plan

Post-tensioning

The slabs were designed on the principles for **unbonded post-tensioning** set out in this document. The monostrands used are of nominal diameter 15 mm (0.6"), have a cross-section of 146 mm^2 and an ultimate strength of 257.8 kN. 50% of the tendons are located in the column lines, 50% in the span. Some of the tendons over the columns are formed into bundles of two.

In the longitudinal direction, the strands were divided by a non-stressed inter-media

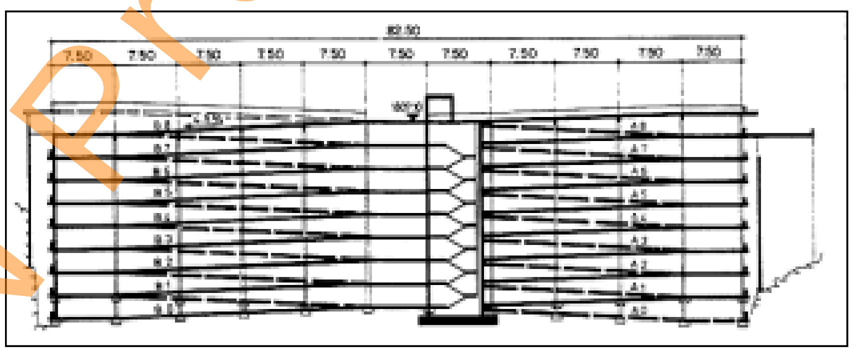


Figure 105: Longitudinal section

Figure 106: Cross-section

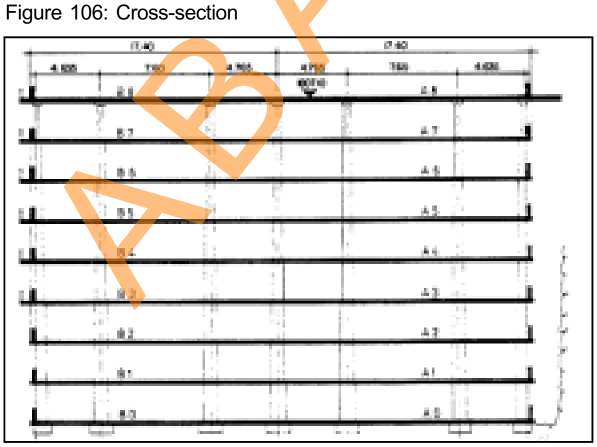
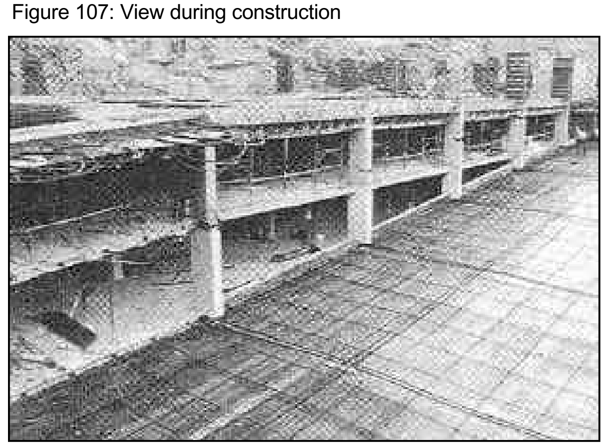


Figure 107: View during construction



anchorage into sections of 46.3 and 37 m length. This enabled a reduction in the free strand length to be achieved with a corresponding increase in the ultimate strength. The ends of all the monostrands in the longitudinal direction are fitted with VSL stressing anchorages. The strands in the transverse direction also utilize intermediate anchorages in the horizontal areas of the slabs. These anchorages are, however, located at the construction joints and therefore served as stressing anchorages (Fig. 109). The remaining transverse strands have a deadend anchorage at one end and stressing anchorages at the other.

In the seven lower slabs, the quantity of post-tensioning steel is 3.7 kg/m^2, and in the roof slab it is 6.0 kg/m^2. The quantities of ordinary reinforcement required were 6.4 kg/m^2 and 12 kg/m^2 respectively (including 1 kg/m^2 fixing steel for the tendons in each case). This low reinforcement content is explained by the fact that no bottom reinforcement was necessary in the internal spans.

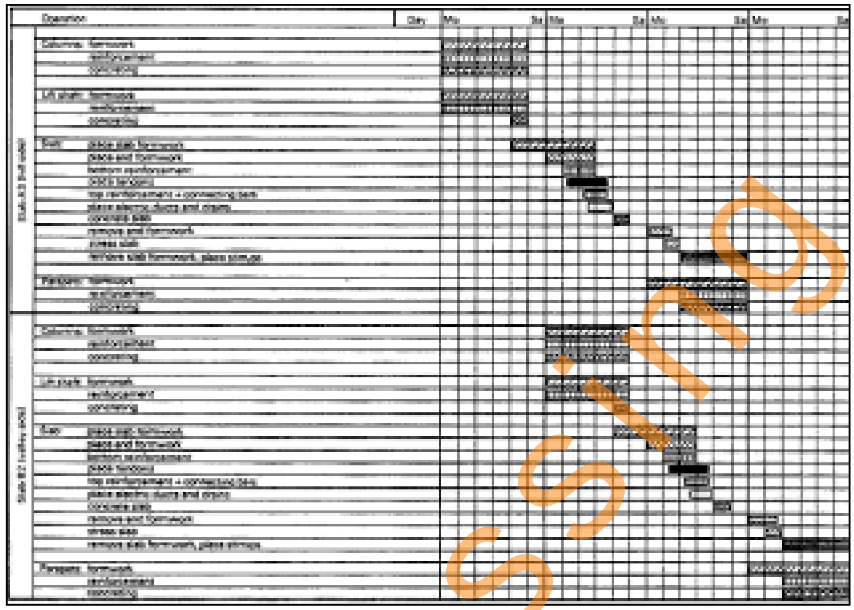


Figure 108: Extract from the construction programme

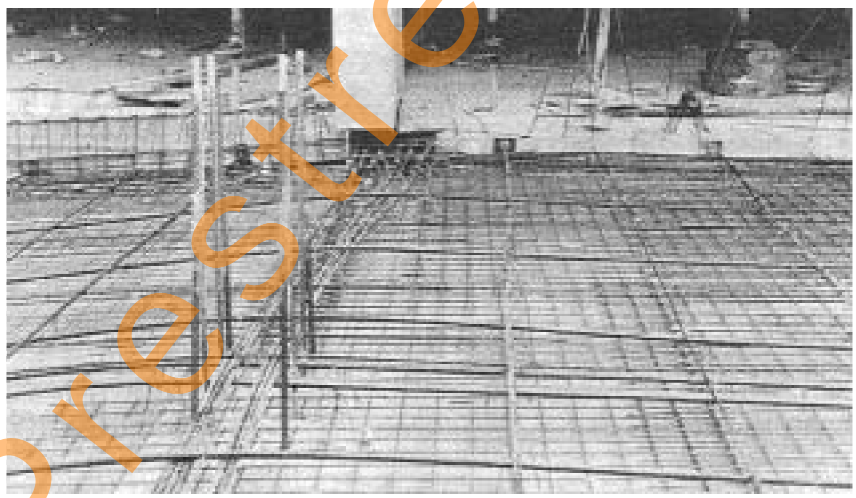


Figure 109: Construction joint with stressed intermediate anchorages

9.12. Summary

Some important data for the slabs described in Chapters 9.2. to 9.11. are summarized in Table VII. When a comparison is being made between the values, it must be remembered however that different standards were used for different projects and the design methods have progressively developed in the course of time.

Table VI I - Main data of the structures described in Chapters 9.2. to 9.1 1.

Chapter	9.2.	9.3.	9.4.	9.5.	9.6.	9.7.	9.8.	9.9.	9.10.	9.11.
Name of structure	Orchard Towers, Singapore	Headquarters of the Ilford Group, Basildon, Great Britain	Centro Empresarial, São Paulo, Brazil	Doubletree Inn, Monterey, California, USA	Shopping Centre Burwood, Australia	Municipal Construction Office Building Leiden, Netherlands	Underground garage for ÖVA Brunswick, FR Germany	Shopping Centre Oberes Murifeld/ Wittigkofen, Berne, Switzerland	Underground garage Ded XII, Linz, Austria	Multi-storey car park Saas-Fee, Switzerland
Years of construction	1972–74	1974–75	1974–77	1976–77	1976–78	1977–78	1979	1979	1979–80	1979–80
Type of structure	Commercial and residential building	Administrative and laboratory building	Administrative building	Hotel with car park	Shopping centre	Administrative building	Car park	Shopping centre	Car park	Car park
Bonded or unbonded	bonded	unbonded	bonded	unbonded	bonded	unbonded	unbonded	unbonded	bonded	unbonded
Type of slab	F B	F	W F	F	B	F	F	F	F	F
max. span l (m)	6.85 8.25	12.00	15.00 10.00	8.28	8.40	7.20	8.75	8.50	8.10	7.80
Slab thickness h (mm)	180 150/380	300	60/400 180	190	170/450	240	350	240	300	200 400
Live load q (kN/m^2)		5.0 8.5	5.0	1.4	6.0	~ 4.0	19.3	3.0 5.0	15.0	2.0 15.5
Prestressing steel content (kg/m^2)		5.5 6.4	~ 11.6	~ 3.0	~ 5.6	6.1	7.3	4.8 4.7	7.4	3.7 6.0

F=Flab slab B=Slab with main beams w=Waffle slab

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Appendix 1: Symbols/ Definitions/ Dimensional units/ Signs

Symbols

A_p	Cross-sectional area of post-tensioning steel
A_{pc}	Cross-sectional area of post-tensioning steel at column
A_{pf}	Cross-sectional area of post-tensioning steel in span
A_s	Cross-sectional area of ordinary reinforcement
A_{sc}	Cross-sectional area of ordinary reinforcement at column
A_{sf}	Cross-sectional area of ordinary reinforcement in span
A_{sg}	Cross-sectional area of ordinary reinforcement in column strip
A_{ss}	Cross-sectional area of ordinary reinforcement in column line
C (C 20)	Concrete grade (20=f_{cd})
D	Compressive force
E	Modulus of elasticity
E_{c_i}	Modulus of elasticity of concrete
E_c	Reduced modulus of elasticity of concrete
E_p	Modulus of elasticity of prestressing steel
F (F 60)	Fire resistance class (60 = fire resistance time in minutes)
I	Second moment of a plane area
L	Free length of tendon between two anchorages
M_g	Bending moment due to distributed permanent load
M_q	Bending moment due to distributed variable load
M_u	Ultimate resistance moment
P	Post-tensioning force
P_l	Post-tensioning force per strand
P_o	Post-tensioning force at stressing anchorage prior to wedge draw-in
P_x	Post-tensioning force at point x
R	Ultimate strength of the cross-section
R_d	Design value for ultimate strength of cross-section
S	Moments and shear forces due to applied loads
V_g	Column load due to dead load
V_{g+q}	Column load due to dead load plus live load
V_p	Transverse component from prestressing inside the critical shear periphery
$V_{p\infty}$	Transverse component from prestressing inside the critical shear periphery at time t = ∞ (after deduction of all losses)
V_q	Column load due to live load
V_u	Punching resistance force (failure)
W_e	Virtual work of applied forces
W_i	Virtual work of internal forces
Z_a	Tension force at support
Z_p	Tension force due to post-tensioned reinforcement
Z_s	Tension force due to ordinary reinforcement
a	Deflection
a_{d-u}	Deflection due to permanent load minus transverse component from prestressing
a_{g+qr-d}	Deflection due to cracking load minus permanent load
a_{q-qr}	Deflection due to live load minus portion of live load in cracking load
a_u	Limit deflection
b	Width
b_c	Column dimension (width, diameter)
b_{cd}	Width of punching cone
b_{ck}	Width of column line
b_g	Width of column strip
c_p	Concrete cover to post-tensioned reinforcement
c_s	Concrete cover to ordinary reinforcement
d	Permanent load
d_p	Effective depth of post-tensioned reinforcement
d_{pc}	Effective depth of post-tensioned reinforcement at column
d_{pf}	Effective depth of post-tensioned reinforcement in span
d_s	Effective depth of ordinary reinforcement
d_{sc}	Effective depth of ordinary reinforcement at column
d_{sf}	Effective depth of ordinary reinforcement in span
e	Base of Napierian logarithms
e_c	Eccentricity of the parabola of post-tensioned reinforcement at column
e_f	Eccentricity of the parabola of post-tensioned reinforcement at centre of span
e_k	Eccentricity of the parabola of post-tensioned reinforcement in cantilever
e_p	Average eccentricity of post-tensioned reinforcement (average of both directions)
f_c	Compressive strength of concrete (cube, prism or cylinder strength, depending upon country)
f_c^{28}	Compressive strength of concrete at 28 days
f_{cd}	Design value for compressive strength of concrete
f_{ct}	Tensile strength of concrete
f_{pu}	Characteristic strength of post-tensioning steel
f_{py}	Yield strength of post-tensioning steel
f_s	Characteristic strength of reinforcing steel
f_{sy}	Yield strength of reinforcing steel
g	Self-weight of slab ($y_c \cdot h$)
$(g+q)_u$	Ultimate design load
g_B	Distributed weight of slab surfacing
g_W	Distributed load due to weight of walls
h	Slab thickness
h_p	Sag of tendon parabola
k	Wobble factor
l	Length of span
l_1	Length of span 1
l_2	Length of span 2
$l_{a\,min}$	Minimum length of reinforcement (anchoring length not included)
l_k	Length of cantilever
l_l	Length of span in longitudinal direction
l_q	Length of span in transverse direction
m_{min}^k	Smallest negative moment over column with adjoining cantilever
m_u	Plastic moment (in span)
m_{uc}	Plastic moment at column
n	Lateral membrane force per unit width
n_p	Number of tendons
q	Distributed variable load
q_r	Proportion of distributed variable load in cracking load
r_o	Radius of curvature
r	Radius
t	Time
u	Transverse component from prestressing per length unit
u_c	Smallest convex envelope which is completely surrounding the column at a distance of d s/2
w	Influence length of wedge draw-in
x	Distance
x_c	Depth of compressed concrete zone
y_p	Internal lever arm (post-tensioning steel)
α	Angle of deviation of the tendons
β	Ratio, coefficient
γ	Safety factor

Y_c	Volumetric weight of concrete
Y_f	Load factor (partial safety factor)
Y_m	Cross-sectional factor (partial safety factor)
δ	Coefficient
ε_{cc}	Creep strain of concrete
ε_{cs}	Final shrinkage factor of concrete
ε_s	Final shrinkage factor
ε_t	Coefficient of thermal expansion
η	Coefficient
χ	Coefficient
λ	Ratio, coefficient
μ	Coefficient of friction
ξ	Ratio of column dimension to span length
ζ	Ratio of column dimension to slab depth
p	Reinforcement content
p^*	Reinforcement content (fictive figure)
P_m	Bending reinforcement content
p_p	Content of prestressing steel
P_s	Content of ordinary reinforcement
P_v	Content of shear reinforcement
σ_c	Concrete stress
σ_{cpm}	Average centric concrete stress due to prestress
σ_{ct}	Concrete tensile stress
σ_{po}	Stress in post-tensioning steel
$\sigma_{p\infty}$	Stress in post-tensioning steel at time $t = \infty$ in undeformed system after deduction of all losses
σ_{pu}	Stress in post-tensioning steel at failure (of load-bearing structure)
T_{Sd}	Nominal shear stress
T_{ud}	Design shear stress
φ	Creep coefficient
φ_n	Final creep coefficient
ΔP	Difference in post-tensioning force
Δl	Tendon elongation
Δl_c	Wedge draw-in
Δl_{cel}	Elastic strain of concrete
Δl_{cc}	Creep strain of concrete
Δl_{cs}	Shrinkage strain of concrete
Δl_{ct}	Concrete strain due to temperature
Δ_p	Loss of force in tendon due to friction
$\Delta\sigma_p$	Increase of stress in prestressing steel
$\Delta\sigma_{pc}$	Loss of stress in prestressing steel due to creep
$\Delta\sigma_{ps}$	Loss of stress in prestressing steel due to shrinkage
Σ	Sum
$\varnothing$	Diameter

Definitions

Slab	Plate in the form used in building construction as load-bearing element in every storey or as roof
Flab slab	Slab with parallel top and bottom faces
One-way foisted slab	Flat slab reinforced on its lower face at uniform intervals by ribs running in one direction only
Waffle slab	Flat slab reinforced on its lower face by ribs in orthogonal pattern
Main beams	Relatively broad beams of shallow depth which reinforce a slab along the column axes. They may be arranged in only one direction or orthogonally.
Plate	Flat, more or less horizontal panel which is supported at at least three points or two opposite lines
Tendon, cable	Prestressing cable consisting of one or more strands, which may be grouted or ungrouted
Monostrand	Prestressing cable consisting of one strand which is not grouted
Bundle	Several monostrands bundled together
Extruding	The process of applying the grease layer and plastics sheath onto a bare strand for producing monostrands
Design calculation	Determination by calculation of the stresses and loads of a structure
Detailed design	Determination of the dimensions of the load-bearing structure and its components on the basis of the design calculation
Ultimate load	The load at which failure of the structural element just takes place
Under-reinforced	Said of a cross-section, in which the proportion of reinforcement is sufficiently low for failure always to be initiated by yield of the steel
Precompressed tensile zone	Zone subjected to tensile stresses in service state but under compression immediately after stressing of tendons
Column head strengthening	Strengthening of a column directly below the slab to increase the resistance to punching. The strengthening consists either of a uniform thickening of the slab in the region of the column or of a mushroom-shaped flaring of the column at the top.
Column strip	Strip-shaped portion of a slab, the longitudinal axis of which coincides with the column axis
Column line	Strip-shaped portion of a slab, the longitudinal axis of which coincides with the column axis and the width of which is given by the critical shear periphery
Wedge draw-in	The movement of the wedges during the anchoring operation, in which the wedges press into the strands and consequently draw in through a small distance in the bore of the anchorage until they jam. The movement results in a corresponding loss of prestressing force.

Dimensional units

In this report units of the SI system are exclusively used (mm, m, N, kN, N/mm^2, kN/m^2). Weights are given in kilogram (kg) or metric tons (t). Formulae which were originally obtained in another system have been converted to the SI system.

Signs

The following sign rules are used:
- Compressive force, compressive stress: -
- Tension force, tensile stress: +

- Moments:
  - when the upper fibre of the cross-section is tensioned: -
  - when the lower fibre of the cross-section is tensioned: +

- Shortening: -
- Lengthening: +

Appendix 2: Summary of various standars for unbonded post-tensioning

Country	USA	Great Britain	Switzerland	FR Germany	Fédération Internationale de la Précontrainte (FIP)
Title of standard Year of issue	ACI-ASCE Committee 423: «Tentative Recommendations for Prestressed Concrete Flat Plates» [12] 1974	Concrete Society Technical Report No. 17: «Flat slabs in post-tensioned concrete with particular regard to the use of unbonded tendons – design recommendations» [13] 1979	SIA 162, Working party 5: «Ultimate load behaviour of slabs» (Draft) [14] 1979	DIN 4227, Teil 6: «Spannbeton; Bauteile mit Vorspannung ohne Verbund» (Entwurf) [15] 1980	FIP: «Recommendations for the design of flat slabs in post-tensioned concrete (using unbonded and bonded tendons)» [16] 1980
Required proofs and checks	– Admissible stresses (prestressing steel, concrete) – Ultimate limit state (flexure, punching) – Serviceability limit state (crack limitation, deflection) – Detail design aspects (ordinary reinforcement, slab thickness, centric prestress)	– Admissible stresses (prestressing steel, concrete) – Ultimate limit state (flexure, punching) – Serviceability limit state (crack limitation, deflection) – Detail design aspects (ordinary reinforcement, slab thickness, centric prestress)	– Admissible stresses (prestressing steel) – Ultimate limit state (flexure, punching) – Serviceability limit state (crack limitation, deflection) – Detail design aspects (ordinary reinforcement, slab thickness)	– Admissible stresses (prestressing steel) – Ultimate limit state (flexure) – Serviceability limit state (crack limitation, deflection) – Detail design aspects (ordinary reinforcement, slab thickness, centric prestress)	– Admissible stresses (prestressing steel, concrete) – Ultimate limit state (flexure, punching) – Serviceability limit state (crack limitation, deflection) – Detail design aspects (ordinary reinforcement, slab thickness, centric prestress)
Admissible stresses	– Prestressing steel: ● At stressing: $0.80\ f_{pu}$ or $0.94\ f_{py}$ ● Under service loads: $0.70\ f_{pu}$ – Concrete: Admissible tensile stress σ_{ct} Ordinary reinforcement / pos. moments / neg. moments without / $\sigma_{ct} = 0.17\sqrt{f_{cd}}$ / $\sigma_{ct} = 0$ with / $\sigma_{ct} = 0.50\sqrt{f_{cd}}$ / $\sigma_{ct} = 0.50\sqrt{f_{cd}}$	– Prestressing steel: At stressing: $0.70\ f_{pu}$ or $0.80\ f_{pu}$ to overcome frictional losses etc. – Concrete: Admissible tensile stress σ_{ct} Ordinary reinforcement / pos. moments / neg. moments without / $\sigma_{ct} = 0.15\sqrt{f_{cd}}$ / $\sigma_{ct} = 0$ with / $\sigma_{ct} = 0.45\sqrt{f_{cd}}$ / $\sigma_{ct} = 0.45\sqrt{f_{cd}}$	– Prestressing steel: ● At stressing: $0.75\ f_{pu}$ ● Under service loads: $0.70\ f_{pu}$ or $0.85\ f_{py}$	– Prestressing steel: ● At stressing: $0.75\ f_{pu}$ or $0.85\ f_{py}$ ● Under service loads: $0.70\ f_{pu}$ or $0.80\ f_{py}$	– Prestressing steel: At stressing: $0.70\ f_{pu}$ or $0.80\ f_{pu}$ to overcome frictional losses etc. – Concrete: Admissible tensile stress σ_{ct} Ordinary reinforcement / pos. moments / neg. moments without / $\sigma_{ct} = 0.15\sqrt{f_{cd}}$ / $\sigma_{ct} = 0$ with / $\sigma_{ct} = 0.45\sqrt{f_{cd}}$ / $\sigma_{ct} = 0.45\sqrt{f_{cd}}$
Ultimate limit state	– Flexure: according to ACI 318–77 [20] Increase of stress: $\Delta\sigma_p = 70 + \frac{f_{cd}}{100 \cdot \rho_p}$ $\Delta\sigma_p \leq 420\ N/mm^2$ – Punching: according to ACI 318–77 [20] ● Critical section: 0.5 d_v from edge of column ● Vertical component of prestress within critical section to be taken into account $\tau_{bu} = \frac{1.4\,V_g + 1.7\,V_q - V_p}{0.85 \cdot u_v \cdot d_v} \leq 0.29\sqrt{f_c} + 0.3\,\sigma_{pm}$	– Flexure: Sectional forces elastically or plastically Increase of stress: $\Delta\sigma_p = 0.28 \cdot \sigma_{pu} - 0.7\frac{\sigma_{pu}^2}{f_{cd}} \cdot \rho_p$ (derived formula) – Punching: ● Critical section: 1.5h from edge of column ● Vertical component of prestress within h/2 from column edge to be taken into account $V_u = \xi\left[\left(\frac{f_{cd}}{410}\rho_p + \rho_s\right) \cdot f_{cd}\right]^{1/3} \cdot u_c \cdot d_p$ $\geq (1.4 \cdot V_g + 1.6 \cdot V_q)/\delta$	– Flexure: Calculation by plastic method or sectional forces elastically; dimensioning of sections to ultimate Increase of stress: Edge span: $\Delta\sigma_p = \frac{0.075\,d_p}{L} \cdot E_p$ Internal span: $\Delta\sigma_p = \frac{0.0017(1 + 0.1\,h_s)}{L} \cdot E_p$ – Punching: ● Critical section: 0.5 d_v from edge of column ● Vertical component of prestress within $d_v/2$ from column edge to be taken into account $\frac{R_d}{1.2} \geq 1.4 \cdot V_{g+q} - \frac{V_p}{1.2}$ $R_d = u_v \cdot d_v \cdot 1.5 \cdot \tau_{cd}$	– Flexure: Sectional forces determined elastically, approximation method according to [49] Increase of stress: Span: $\Delta\sigma_p = \frac{\Delta l}{L} \cdot E$ Column: $\Delta\sigma_p = (\Delta l_1 + \Delta l_2) \cdot \frac{E}{L}$ $\quad \Delta l = \frac{d_p}{17}$ $\sigma_{pu} \leq f_{py}$	– Flexure: Sectional forces elastically or plastically Increase of stress: Span: $\Delta\sigma_p = \frac{\Delta l}{L} \cdot E$ Column: $\Delta\sigma_p = (\Delta l_1 + \Delta l_2) \cdot \frac{E}{L}$ $\quad \Delta l = \frac{d_p}{17}$ $\sigma_{pu} \leq f_{py}$ – Punching: ● Critical section 0.5 d_v from edge of column ● Vertical component of prestress within $d_v/2$ from column edge to be taken into account ● Increase of shear resistance due to bending reinforcement and equivalent portion of centric prestress
Serviceability limit state	– Crack limitation: by limiting concrete tensile stresses – Deflection: see Detail design aspects	– Crack limitation: by limiting concrete tensile stresses – Deflection: ● In general: $a \leq l/250$ ● Where building contains components sensitive to deflection: $a \leq l/350$ or 20 mm	– Crack limitation: by minimum reinforcement – Deflection: to be checked where necessary	– Crack limitation: If $\rho_s \geq 0.10\%$ no check needed If $\rho_s < 0.10\%$ consideration of a crack condition – Punching: ● Critical section: 0.5 d_v from edge of column ● Vertical component of prestress within critical section to be taken into account ● Increase of shear resistance due to bending reinforcement and equivalent portion of centric prestress	– Crack limitation: by limitation of concrete tensile stresses – Deflection: ● In general: $a \leq l/250$ ● Where building contains components sensitive to deflection: $a \leq l/350$
Detail design aspects	– Ordinary reinforcement: Column zone: ● $\rho_s \geq 0.15\%$ of the column strip ($b_v = 0.5\ l$) ● Concentrated over column ● Length = l/3; min. 4 bars Span: ● Reinforcement, if $\sigma_{ct} \geq 0.17\sqrt{f_{cd}}$ – Slab thickness: Slabs: l/h = 40 to 45 or 48* Roofs: l/h = 45 to 48 or 52* (* admissible, if deflections and vibrations are checked) – Centric prestress: For crack limitation ≥ 1.4 N/mm² recommended Lowest limit: 0.9 N/mm² Highest limit: 3.5 N/mm²	– Ordinary reinforcement: Column zone: ● $\rho_s \geq 0.075\%$ of $b \cdot (b_v + 4h)$ ● Concentrated over column ● Length ≥ l/3; Ø ≥ 12 mm Take account of tensile wedges. – Slab thickness: Slabs: l/h ≤ 42 or 48* Roofs: l/h ≤ 48 or 52* (* admissible, if deflections and vibrations are checked) – Centric prestress: ≥ 0.7 N/mm²	– Ordinary reinforcement: Column zone: heavy top reinforcement Edge and corner spans: $\rho_s \geq 0.15 - 0.50 \cdot \rho_p$ $\rho_s \geq 0.05\%$ Internal spans: $\rho_s \geq 0$ – Slab thickness: l/h ≤ 45	– Ordinary reinforcement: Column zone: ● Punching zone $\rho_s = 0.30\%$ ● Column strip $\rho_s = 0.15\%$ Span: depending upon concrete and steel quality $\rho_s = 0.04$ to 0.08% Internal spans: no ordinary reinforcement, if crack limitation demonstrated – Slab thickness: l/h ≤ 40 – Centric prestress: ≥ 1.0 N/mm²	– Ordinary reinforcement: Column zone: ● $\rho_s \geq 0.075\%$ of total cross-sectional area ● Concentrated over column ● Length l/3 Edge and corner spans: always with ordinary reinforcement Internal spans: no ordinary reinforcement prescribed – Slab thickness: Slabs: l/h ≤ 42 or 48* Roofs: l/h ≤ 48 or 52* Waffle slabs: l/h ≤ 35 (* admissible, if deflections and vibrations are checked) – Centric prestress: ≥ 1.0 N/mm²

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