VSL

POST- TENSIONED CONCRETE
IN BUILDING CONSTRUCTION

POST- TENSIONED FOUNDATIONS

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POST-TENSIONED CONCRETE IN BUILDING CONSTRUCTION - POST-TENSIONED FOUNDATIONS

1. Introduction

While the use of prestressing in superstructures is well established, foundations have only rarely been prestressed. Apart from certain technical considerations, this may be due to some conservatism inherent in foundation engineering. The purpose of this report is to contribute to an improved understanding and a more widespread application of post-tensioning in foundation engineering. It is shown that subgrade friction and similar restraining actions influence the effectiveness of the prestressing less than is frequently assumed. In fact, with proper application, the well-known advantages of prestressing can also be exploited in foundation engineering.
This report addresses the design and detailing of shallow foundations, such as those shown in Fig. 1. Similar considerations apply to the design of pile caps and thick transfer plates in buildings. Several typical applications are illustrated in Fig. 2. For the use of prestressing in pavements and in residential, commercial and industrial floors, the reader is referred to the literature [2], [3], [4].
The main purpose of foundations is safely to transfer applied gravity and lateral loads from the superstructure to the soil without undue deformation of either the foundation or the superstructure and without the bearing capacity of the soil being exceeded. Apart from this basic requirement, other considerations such as the need for watertightness may govern the design of certain types of foundations.
Any foundation design is complicated by the dual interaction with the superstructure above and the soil below the foundation. However, while the static system and the material properties of the superstructure are well known, the knowledge about the governing soil parameters is typically crude and limited. Simplifying engineering approaches are therefore indispensable. Rather than performing sophisticated analyses, the designer should direct his efforts towards a reasonable design concept and appropriate detailing, for the proper transfer of the load from the superstructure via the foundation to the soil. In this way the designer avoids being forced into a passive role. Instead, he assumes an active attitude and bases his decisions on relatively simple considerations. Apart from an appropriate selection of the structural layout, the possibility of introducing a favourable system of active forces by prestressing is one of the primary means of reaching these objectives.

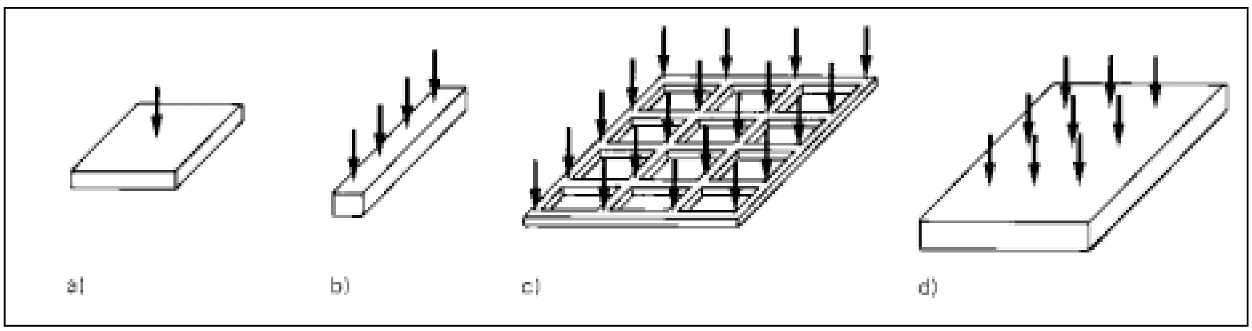


Figure 1 : Typical shallow foundations: (a) Footing; (b) Strip foundation; (c) Grid foundation; (d) Mat foundation

2. General Considerations

2.1. Soil-structure interaction

Fig. 3 illustrates the basic problem of the interaction between a building structure and the underlying soil. Applied gravity and lateral loads acting on the superstructure result in significant stresses and deformations within a certain region of the soil in the neighbourhood of the foundation. At a certain distance from the foundation, soil deformations will be negligibly small. The part of the soil beyond this distance will not appreciably influence the overall structural behaviour and hence this part can be modelled as rigid.

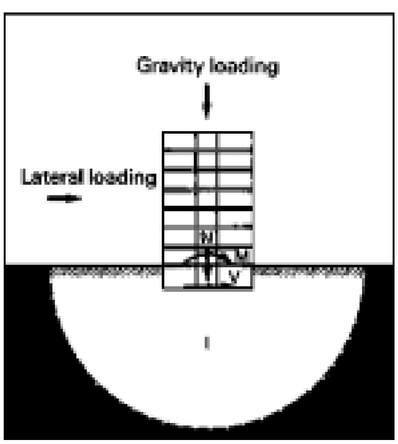


Figure 3: Basic problem of soil-structure interaction

In Fig. 3, the deformable region of the soil beneath the foundation and the adjoining region which may be idealized as rigid are labelled I and II respectively. The boundary between the two regions would have to be determined on the basis of a sensitivity analysis and Region I could be modelled as a part of the entire structural system. By applying state-of-the-art techniques, a comprehensive analysis of the soil-structure interaction could be made and the dimensioning and detailing of both the foundation and the superstructure could proceed with the use of conventional methods.While such considerations are helpful in principle, practical difficulties prevent their appli cation in general. In particular, the actual non linear and time-dependent behaviour substantially add to complexity of the problem. Furthermore, the knowledge of soil data generally is rather crude and limited. Soil properties inherently display considerable scatter. However, any soil investigation must be limited in scope and extent and the results of such an investigation are not easy to interpret. Finally, residual stresses in the soil are largely unknown and therefore their influence on the overall structural behaviour cannot be assessed. To a lesser extent a similar problem occurs also in regard to the influence of the construction history.
In view of these difficulties it is unrealistic to believe that any analysis would ever lead to an accurate representation of the stresses and deformations throughout the entire structural system. Therefore, simplifying engineering approaches must be adopted. Essentially, such approaches address the problem of how the known stress resultants M, N and V at the base of the superstructure can be transmitted via the foundation to the soil, see Fig. 3. The various approaches lead to different soil pressure distributions at the base of the foundation. Typically, transmission of the shear

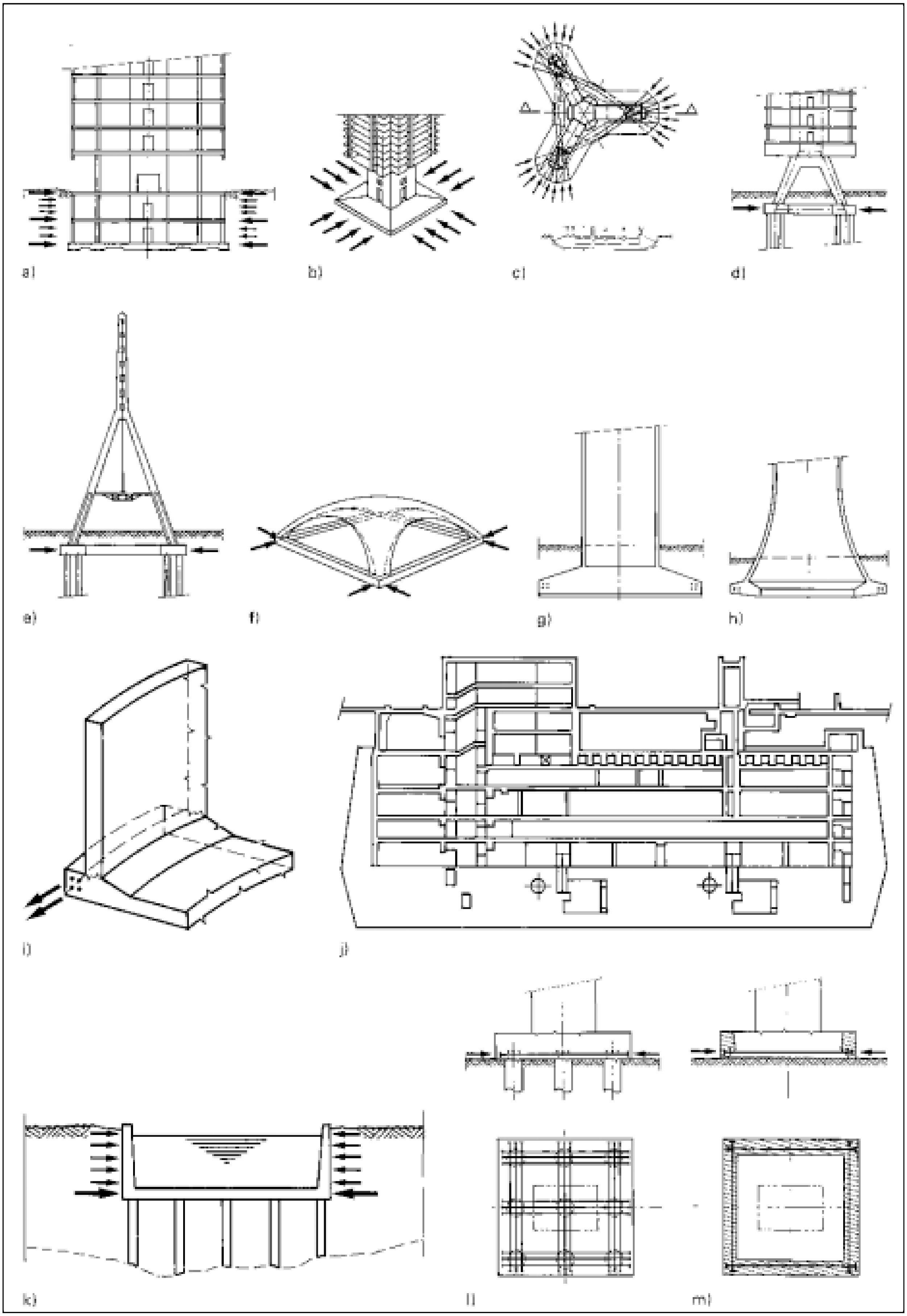


Figure 2: Applications of post-tensioning: (a) Box foundation of building structure; (b) Mat foundation of building structure; (c) Foundation of CN Tower in Toronto, Canada; (d), (e) and (f) Tie beams; (g) Circular mat foundation of tower; (h) Ring foundation of cooling tower; (i) Ring foundation of storage tank; (j) Massive foundation of power station [1]; (k) Rectangular liquid container; (1) Pile cap; (m) Strengthening of footing

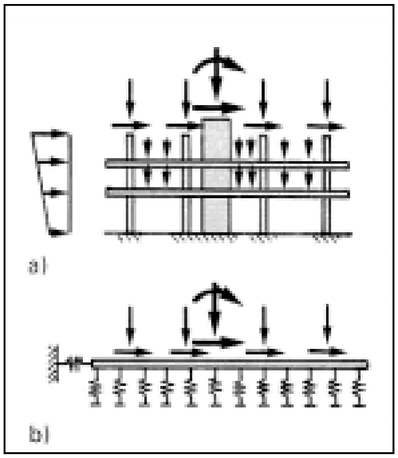


Figure 4: Conventional approach: (a) Analysis of superstructure assuming rigid foundation; (b) Reactions from superstructure analysis applied to flexible foundation

forces, V, can be treated as a secondary problem in comparison with the primary problem of transferring normal forces, N, and moments, M.

A conventional approach is illustrated in Fig. 4. In a first step, the superstructure is analysed by assuming a rigid foundation. In a second step, the reactions determined at the base of the superstructure are applied as loads acting on the foundation and the resulting forces and moments are taken as a basis for the dimensioning of the foundation. Depending on the manner of determining the soil pressure distribution, several methods can be distinguished. These include the linear soil pressure distribution method, simple and advanced elastic foundation methods and more sophisticated methods.

For relatively small and/or stiff foundations, the linear soil pressure distribution method illustrated in Fig. 5a is a suitable simplification. The soil pressure distribution follows from equilibrium with the overall stress resultants. For relatively large and/or flexible foundations, the elastic foundation method illustrated in Fig. 5b, is more appropriate [5], [6], [7], [8] This method assumes that the soil pressures are proportional to the settlements of the foundation. The proportionality constant, k_s, is called the modulus of subgrade reaction. The value of k_s should be determined from a settlement analysis. In general, k_s would vary along the foundation. However, for preliminary calculations, an average constant value of k_s is normally adopted. Fig. 5c gives typical orders of magnitude of ks for various soils.

Fig. 6 illustrates the relationship between settlements and subgrade reaction depending upon the type of soil and the stiffness of the foundation. For a granular soil and a flexible foundation, the settlements will be non-uniform, with maximum values occurring near the edges of the foundation. For a non-granular soil and a flexible foundation, the settlements will also be non-uniform but the maximum settlement will occur at the centre of the foundation. In any case, however, the soil pressure under a flexible foundation will tend to be more or less uniform. On the contrary, for a stiff foundation, the settlements will be more or less uniform whereas the soil pressure distribution will be non-uniform, with maximum values occurring near the centre of the foundation for granular soils and near the

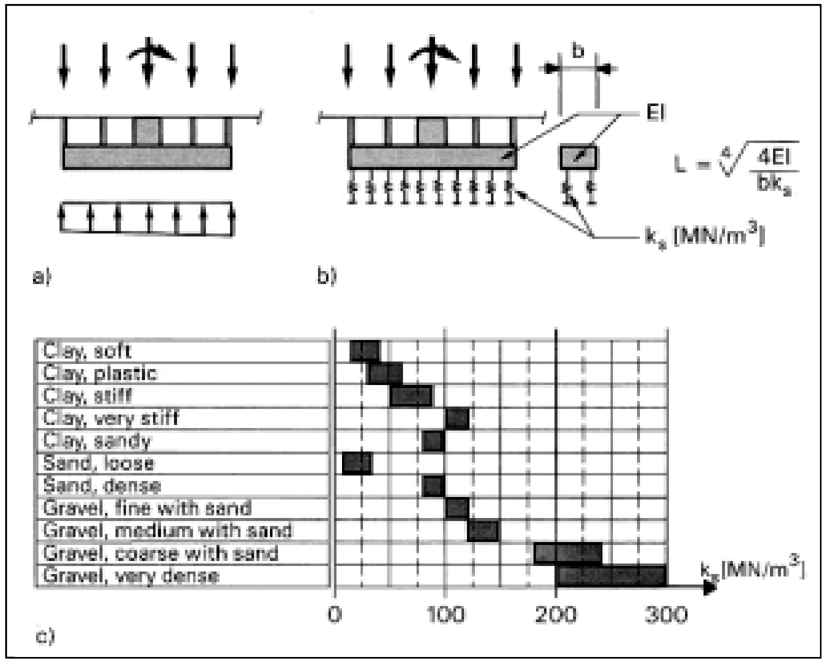


Figure 5 : Simplified approaches: (a) Linear soil pressure distribution method; (b) Elastic foundation method; (c) Typical moduli of subgrade reaction

edges for non-granular soils. This discussion reveals that for a uniformly loaded foundation the modulus of subgrade reaction, k_s, would vary roughly in a similar manner to the soil pressure under a stiff foundation, i.e. maximum values of k_s would occur near the centre of a foundation on a granular soil and near the edge of a foundation on a non-granular soil. In any case, the distribution of k_s would be nonuniform in general. The so-called simple elastic foundation method uses an average constant value of k_s. On the other hand, various advanced elastic foundation methods take the variation of the modulus of subgrade reaction along the foundation into account.

The characteristic length, L, given in Fig. 5b is a meausre for the size of the influence region of a load acting on the foundation. This will be further discussed in Section 3.2. below. Since L is proportional to the fourth root of the ratio of the stiffnesses of the foundation and the soil, changes by factors of 2 or 10 in the stiffness ratio will affect L by factors of only 1.19 or 1.78, respectively. More sophisticated methods of determining the soil pressure distribution consider the compatibility between the settlements and the deformations of the superstructure. In fact, by assuming a rigid foundation as discussed in connection with Fig. 4, the conventional approach ignores this compatibility

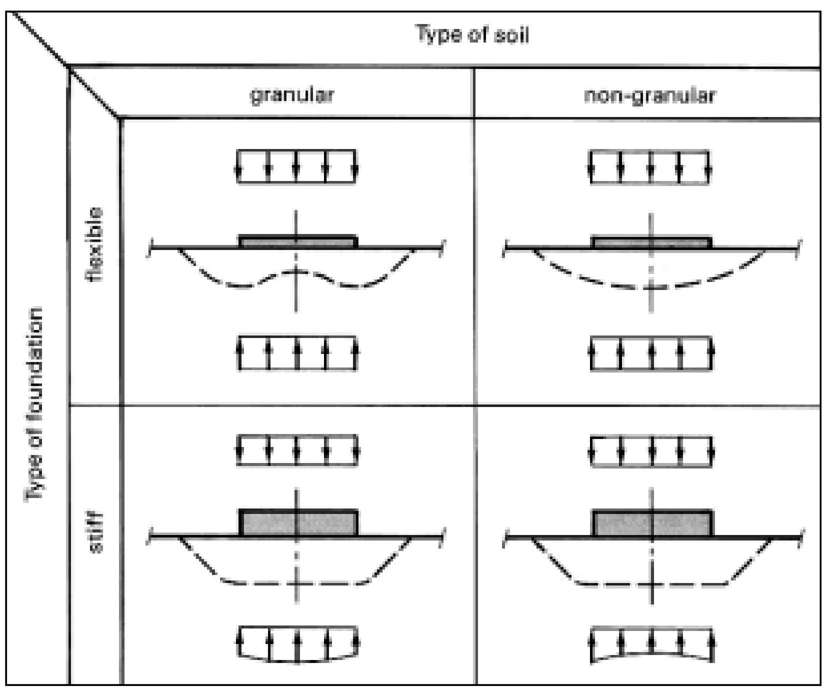


Figure 6: Subgrade reaction and settlement as a function of soil and foundation stiffness

quirement. The deformations of the foundation should-be considered as restraining actions on the superstructure. As a consequence, the reactions at the base of the superstructure would change, leading to a further change of the deformations of the foundation. With suitable iterative techniques the ensuing small changes of reactions and deformations could be pursued and compatibility could be approximated to any desired degree of accuracy.

The designer must consider not only the transfer of gravity loads, but also the transfer of lateral loads. Normally, the transfer of shear forces at the base of a foundation is made possible by subgrade friction. Typically, the ratio V/N of the forces indicated in Fig. 3 is of the order of 0.1 or less and hence the transfer of lateral loads does not in general create difficulties. However, where as for load transfer or strength high subgrade friction values are desired, low subgrade friction values will reduce restraining actions caused by volumetric changes of the foundation. Such volumetric changes are caused primarily by shrinkage and temperature effects and to a much lesser degree by prestressing and creep of the concrete.

The problem of subgrade friction is illustrated in Fig. 7. Fig. 7a shows a unit slab element subjected to a constant normal pressure, σ, and a shear stress, τ. These applied stresses are transferred from the slab through the interface at the base of the element to the underlying soil. By increasing τ a displacement Δ will occur. For low values of τ there is no relative displacement at the interface, i.e. the whole displacement Δ is due to a deformation of the soil. Under a certain, higher value of τ, relative displacements at the interface will set in, until at the ultimate the entire increase of Δ will be due to pure sliding at the interface.

Figures 7b and 7c represent selected sheardisplacement curves obtained from tests in which the normal pressure σ was held constant [2], [9].

Fig. 7b shows the influence of a repeated loading under a very low normal pressure. The friction coefficient, μ_1, obtained from the peak load of the first test is considerably greater than the friction coefficients obtained from subsequent tests.

Fig. 7c shows a similar test curve determined from a direct shear test on a loose sand under a very high normal pressure.

The friction coefficients, μ, depend upon the nature of the interface. Fig. 7d summarizes typical ranges of friction coefficients for various materials. For slabs cast directly on the subgrade, friction coefficients will range from about 0.7 to 2.0. By using double layers of polyethylene sheeting, w may be reduced to about 0.5.

For practical purposes, the non-linear behaviour exemplified by Figures 7b and 7c can be idealized by either one of the two sheardisplacement relationships shown in Fig. 7e. Curve 1, which corresponds to an assumed rigid-plastic behaviour, is often used because of its simplicity in application. For relatively long foundations and low friction coefficients, this idealization will provide reasonable approximations. Curve 2, which represents an elastic-plastic idealization, provides a better approximation of the actual shear-displacement relationship. The elastic and plastic branches of this curve correspond to a pure deformation of the soil and a pure sliding at the

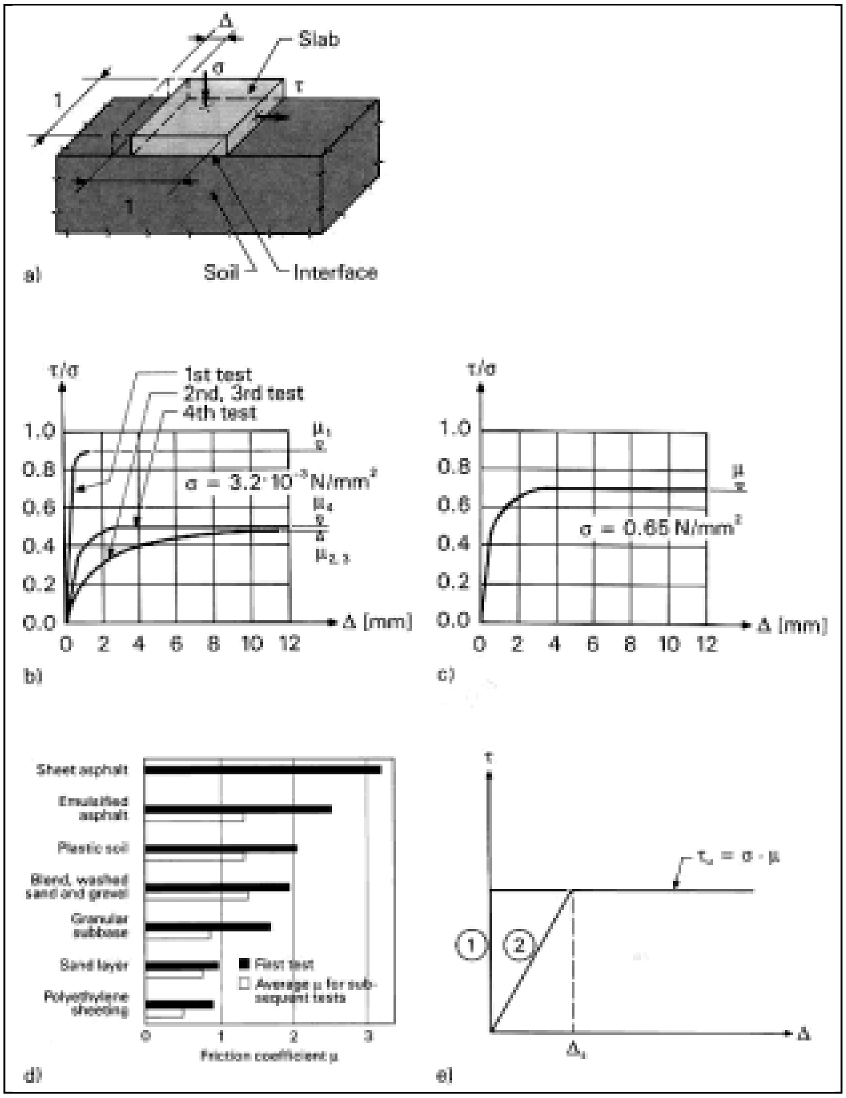


Figure 7 : Subgrade friction: (a) Problem and notation; (b) Test curves for a 125 mm thick concrete slab cast on polyethylene sheeting [2]; (c) Test curve from a direct shear test on a loose sand [9]; (d) Friction coefficients for 125 mm thick concrete slabs [2]; (e) Idealized shear-displacement relationships

interface, respectively. The displacement, Δ_s, at onset of pure sliding ranges from about 0.5 to 2 mm, depending upon the type of interface. For smooth interfaces, the value of Δ_s, will be lower than for rough interfaces. In Section 3.3. below the two models given in Fig. 7e are used for the discussion of the effects of subgrade friction on the effective prestress in post-tensioned slabs, see Fig. 17.

2.2. Why prestressing?

Prestressing offers the possibility of introducing a favourable system of active forces acting on the concrete, viz. anchorage forces, deviation forces and friction forces.

The in-plane components of the anchorage forces produce a certain prestress of the concrete, resulting in a higher cracking resistance, improved stiffness and watertightness, a reduced need for expansion joints and a better durability. Furthermore, an appropriate application of prestressing leads to a reduction in early hydration cracking, which is one of the major reasons for unsightly cracks.

The transverse components of the anchorage forces and the deviation forces allow a load balancing, resulting in an improved soil pressure distribution and an increased shear resistance. Fig. 8 illustrates the effect of prestressing on the soil pressure distribution. Consider an infinitely long continuous beam on an elastic foundation, see Fig. 8a. The beam is loaded by equally spaced columns, each applying a load F. A uniform modulus of subgrade reaction is assumed and a draped prestressing tendon with a profile made up of second order parabolas is used. The prestressing force, P, is chosen so as to balance either 50% or 100% of F. Figures 8b, 8c and 8d show the soil pressures due to F, P and the combined action of F and P respectively. As Fig. 8d demonstrates, a more uniform soil pressure distribution with a corresponding reduction of the maximum soil pressurecan be achieved by prestressing. In fact, the tendon profile can be selected so that for a particular loading case, e.g. for dead loads, an absolutely uniform soil pressure distribution

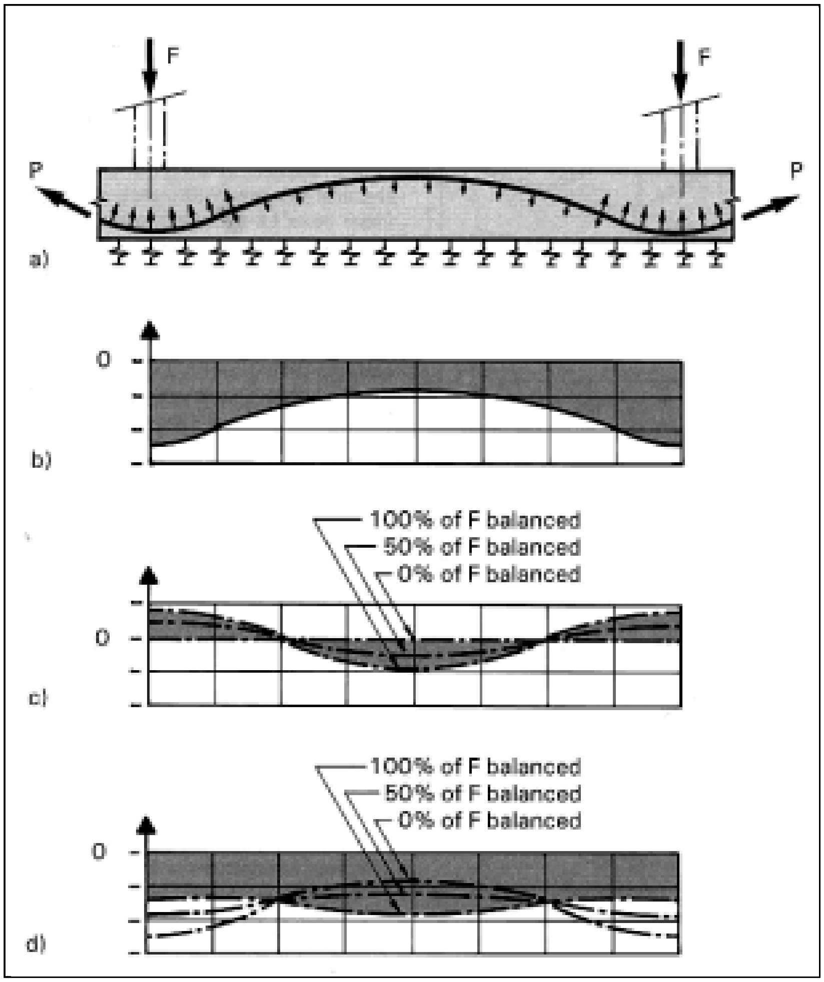


Figure 8: Effect of prestressing on soil pressure distribution: (a) Prestressed continuous beam on elastic foundation; (b) Soil pressure due to column loads; (c) Soil pressure due to prestressing; (d) Soil pressure due to combined action of column loads and prestressing

would result. Fig. 9 illustrates the effect of draped prestressing tendons on the punching resistance of a foundation slab. The punching resistance is increased by the sum of the components P·sin α which counteract the applied load F. It should be noted that, whereas the effective prestress in a foundation will be affected by subgrade friction to a certain extent, this will not be the case for the deviation forces.

Further advantages of prestressing include the reduced congestion of reinforcement and the concentrated application of anchorage forces, which creates favourable conditions for the transfer of large forces, such as at the head of piles in pile caps, see Fig. 10.

2.3. Type of prestressing

Basically, the designer has the choice between unbonded monostrand tendons and bonded multistrand tendons.

The unbonded monostrand system offers the following advantages:

- thin, light and flexible tendons and hence easy handling and maximum tendon eccentricity,
- small friction losses,
- corrosion protection of prestressing steelex works,
- no grouting necessary.

Typical applications of this system include pavements and slabs on grade for residential, commercial and industrial floors.

Whereas monostrands are primarily used for thin foundation slabs, multistrand tendons are suitable for larger foundations. Advantages of the bonded multistrand system include:

- full exploitation of the yield strength of the prestressing steel,
- ability to transfer large forces using large tendons,
- improved cracking behaviour by activation of bond forces.

Fig. 11 illustrates the combined action of nonprestressed and bonded prestressed reinforcements. After grouting of the tendons, the prestressing steel and non-prestressed reinforcing bars next to the tendons undergo the same strain increment. Hence with a suitable selection of the prestressing level the two reinforcements will reach their yield strengths more or less simultaneously. Fig. 11 reflects typical conditions using an effective prestress of 1,100 N/mm^2, non-prestressed reinforcing bars with yield and ultimate strengths of 420 and 500 N/mm^2 respectively, and strands with yield and ultimate strengths of 1,570 and 1,760 N/mm^2 respectively.

Whereas for bonded systems the behaviour at a section is more or less independent of adjacent regions of the system, this is generally not the case for unbonded systems. Because relative longitudinal displacements between concrete and steel are not prevented when there is no bond, the stress increase in unbonded tendons is related to the deformation of the entire structural system. Therefore, similarly to suspended slabs [10] and externally

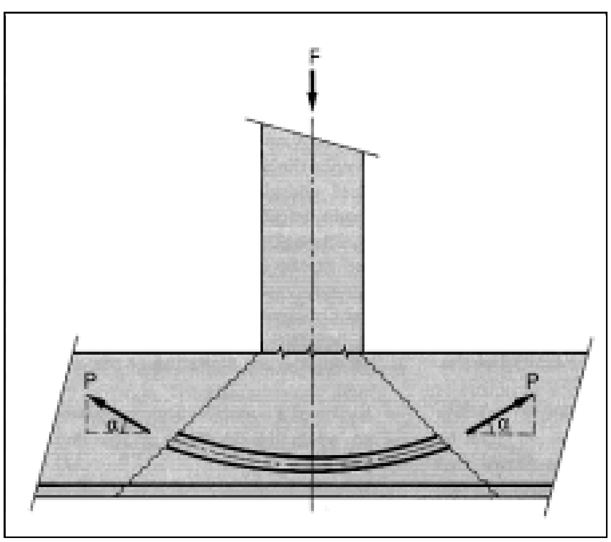


Figure 9: Effect of prestressing on punching resistance

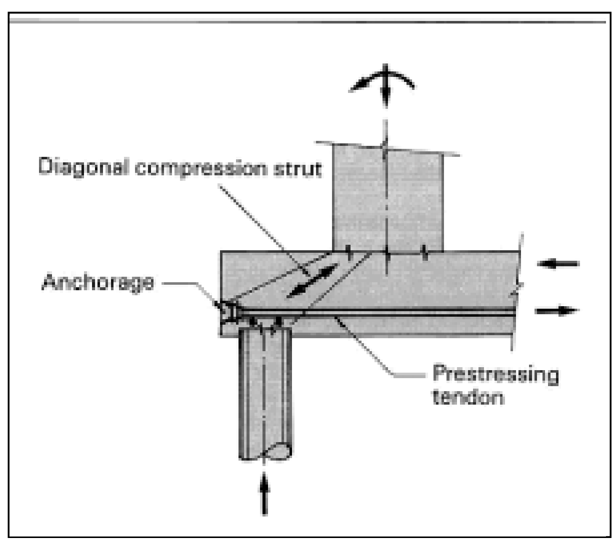


Figure 10: Strut and tie action in pile cap

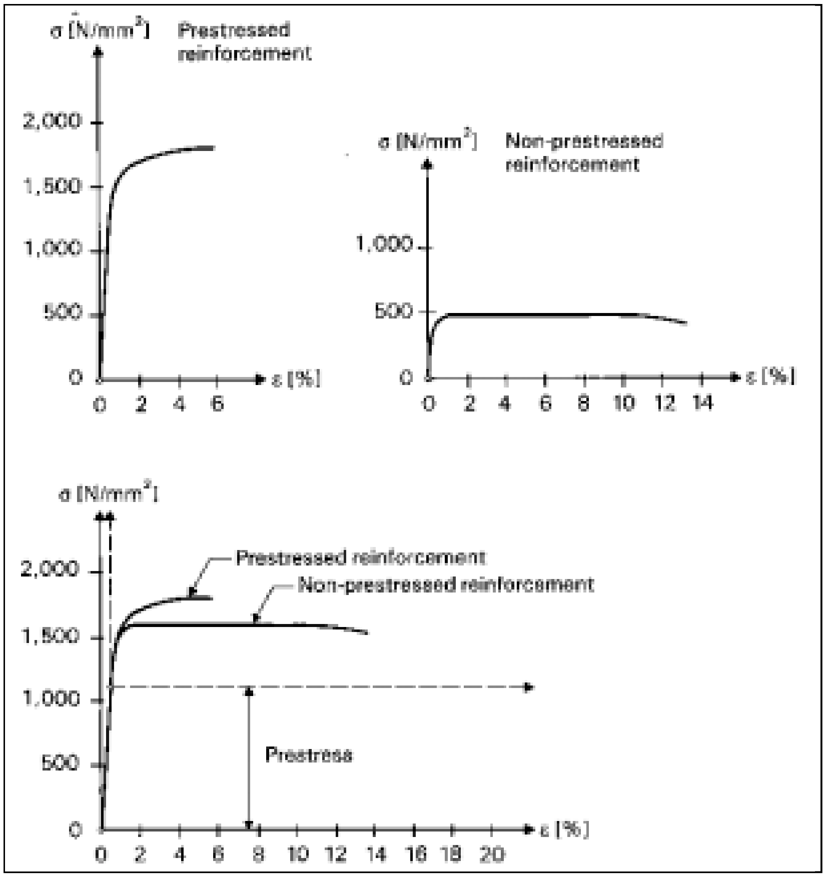


Figure 11 : Stress-strain diagrams showing combined action of prestressed and non-prestressed reinforcement

post-tensioned structures [11], the yield strength of unbonded tendons cannot be fully exploited in general. On the other hand, owing to the bond between concrete and reinforcement, bonded structures are capable of distributing imposed deformations among multiple sections, while for unbonded systems imposed deformations tend to be localized at only one or a few sections. Hence, bonded systems are inherently less sensitive to imposed deformations and secondary effects of any sort.

2.4. Amount of prestressing

In order to improve the cracking behaviour, a certain minimum amount of non-prestressed reinforcement is normally placed at the surfaces of prestressed foundations. Typical minimum reinforcement ratios are in the order of 0.1 %. Practical experience suggests both lower and upper limits for the effective prestress. The advantages of prestressing are most pronounced for effective prestress values in the range from 0.25 to 2.0 N/mm^2. For most practical applications, prestress values will range between the narrower limits of 0.75 and 1.5 N/mm^2.

Detailing requirements such as a minimum bar diameter to guarantee a sufficient stiffness of the reinforcing mesh and minimum and maximum bar spacings may impose further limitations on the choice of the nonprestressed reinforcement. The resistance provided by the combination of prestressed and non-prestressed reinforcements must at least equal the required resistance. Naturally, from the economic point of view, the aim is to minimize any excess resistance. Thus, the amount of prestressed reinforcement will be selected such that, together with the non-prestressed minimum reinforcement, it covers a substantial part of the required resistance. At locations where peak resistances are required, additional non-prestressed reinforcement will be provided.

The combination of prestressed and non-prestressed reinforcements addressed in the stressing. It is evident that the degree of partial prestressing, i.e. the percentage of the total resistance provided by the prestressed reinforcement, will vary from one point to another within a prestressed foundation structure. Therefore, to start a design with a preselected degree of partial prestressing would, in general, be ill-conceived.

3. Design Considerations

3.1. General

As for any concrete structure, the design of a prestressed foundation has to address the effects of both applied loads and imposed deformations.

With regard to applied loads there is, in general, little uncertainty about the overall resultants of the forces acting on a foundation. However, as discussed in Section 2.1., different approaches to the analysis of the soilstructure interaction result in different distributions of these forces. Thus, every approach results in a different set of forces and moments in the foundation that are in equilibrium with the overall resultant of the applied loads. The difference between any two such sets of internal forces and moments corresponds to a residual stress state. Similar residual stress states are produced by imposed deformations of any sort, e.g. due to shrinkage or temperature effects. Furthermore, residual stress states are influenced by the construction history and they are modified as soon as irreversible deformations due to cracking of the concrete, yielding of the reinforcement or slippage of the bond between concrete and reinforcement occur.

It is practically impossible to determine the actual stress state in a foundation structure. Nevertheless, reasonable designs can be achieved in a quite straightforward manner. The key lies in the fact that residual stresses are modified upon the occurrence of irreversible deformations. Thus, if the structure is provided with sufficient ductility in the first place, its dimensioning may in principle be based on any set of internal forces and moments that equilibrate the externally applied loads. With increasing loads, the residual stress state will simply be modified at the expense of deformations and hence thestructure will adapt itself to the applied loading.

In conclusion, therefore, the design will have two primary objectives. Firstly, it will attempt to provide the structure with sufficient ductility and redundancy to make it insensitive to imposed deformations. Secondly, the structural dimensions and details will be determined on the basis of a consistent equilibrium analysis to resist the applied loads. Important measures for reaching these objectives include:

- an appropriate limitation of the soil pressure,
- choice of a suitable structural system,
- an adequate detailing of the reinforcement,
- a suitable limitation of the reinforcement content,
- provision of an adequate amount of bonded reinforcement.

3.2. Structural systems

As discussed in Section 2.1., the linear soil pressure distribution method is a suitable simplification for relatively stiff foundations. On the other hand, the elastic foundation method is recommended for flexible foundations. In evaluating the relative stiffness of a foundation, a procedure proposed by Meyerhof may be used [12], [13].

Under a stiff foundation the relative settlements will remain within the order of 10% or less of the total settlement. This creates favourable conditions for the behaviour of the superstructure [9]. Furthermore, the simplifying assumptions underlying the engineering approaches discussed in Section 2.1. are well justified.

Fig. 12 shows a number of selected foundation systems. By joining slabs and walls in the form of a multi-cellular structure, such as the box foundation shown in Fig. 12a, a very stiff assemblage of individually flexible structural members can be achieved. Usual systems for the construction of mat foundations are indicated in Figures 12b to 12e. Other systems are illustrated in Figures 1 and 2.

The remainder of this section will concentrate on some comments regarding the dimensioning of slabs or beams that rest on the soil.

In the determination of the depth of such members, consideration has to be given to

- practical minimum thickness requirements,
- maximum allowable soil pressure,
- ductility requirements,
- shear resistance.

For stiff foundations the sectional forces and moments follow from the known reactions at the base of the superstructure and the linearly distributed soil pressure by applying simple statics. The dimensioning and detailing then proceed using conventional approaches.

For flexible foundations, the sectional forces and moments cannot be determined from simple statics. Instead, either the elastic foundation method or a more sophisticated approach have to be applied, see Section 2.1. A foundation can be stiff in an overall sense while locally it may be quite flexible. For example, such a case may occur for a box foundation with a slender bottom slab. For a preliminary dimensioning of such slabs it is sufficient to know the order of magnitude of the maximum sectional forces and moments. Several methods are available which may assist designers in this regard. They can be subdivided into:

- methods based on the theory of elasticity,
- semi-empirical methods,
- methods based on limit analysis.

Fig. 13 summarizes some results of an analysis of a beam on elastic foundation. Figures 13a and 13b show infinitely long beams subjected to single loads or couples applied at the centres or at the edges, respectively. The selected values given in Table I can be used to obtain an estimate of the maximum effects of applied loadings. The characteristic length

$$L = \sqrt[4]{\frac{4E_c \cdot I_c}{b \cdot k_s}}$$

where E_c = modulus of elasticity of beam
I_c = moment of inertia of beam
b = width of beam
k_s = modulus of subgrade reaction

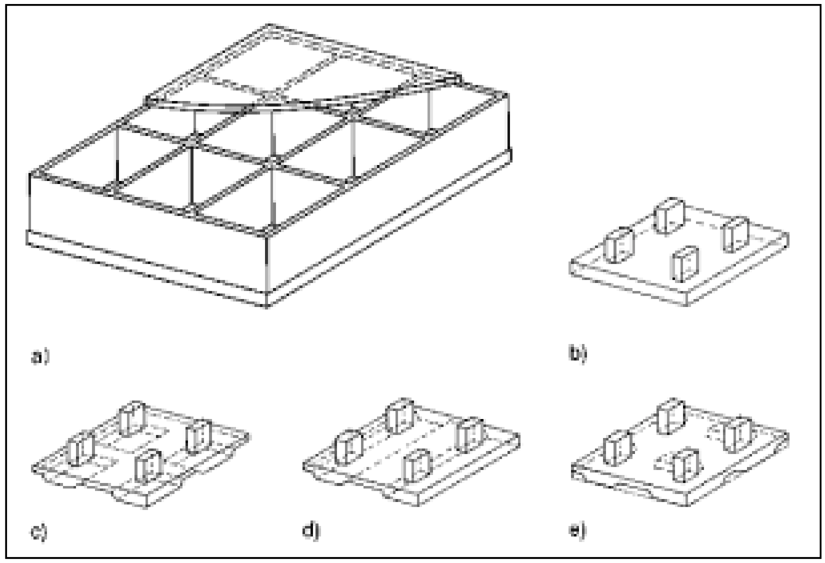


Figure 12: Selected structural systems: (a) Box foundation; (b) Flat plate; (c) Flat slab; (d) Oneway beam and slab; (e) Two-way beam and slab

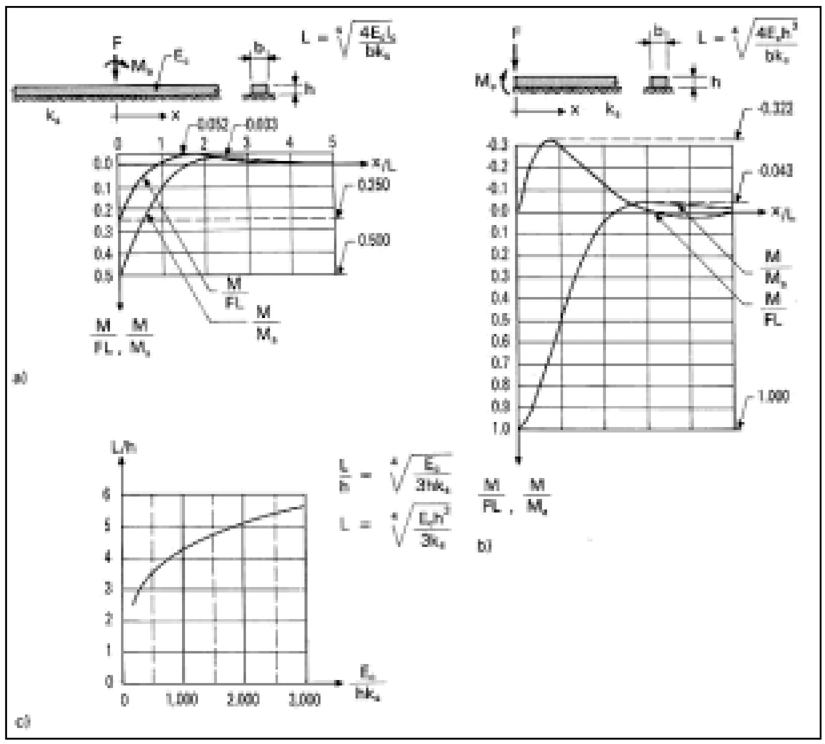


Figure 13: Beam on elastic foundation: (a) Centre loading; (b) Edge loading ; (c) Characteristic length

	Centre loading		Edge loading	
	Single load F	Couple M_o	Single load F	Couple M_o
Maximum moment	0.25 FL	0.50 M_o	0.32 FL	1.00 M_o
Maximum shear force	0.50 F	0.50 M_o/L	1.00 F	0.64 M_o/L
Maximum soil pressure	0.50 F/(bL)	0.32 $M_o/(bL^2)$	2.00 F/(bL)	0.42 $M_o/(bL^2)$

Table I : Selected results from beam on elastic foundation analysis

is a measure for the size of the influence region of an applied load. For a rectangular, solid cross-section the characteristic length L is of the order of two to five times the thickness h of the section, see Fig. 13c.

The relationships represented in Fig. 13 are directly applicable to strip foundations, grid foundations and most foundations with oneway or two-way beams where it can be assumed that the soil pressures are primarily

concentrated under the beams. Furthermore, these relationships can be used for slabs by applying the strip method [14].
Westergaard [15] considered interior, edge and corner loads applied to infinitely large plates resting on an elastic foundation, see Fig. 14. He evolved the formulae for the maximum moments and soil pressures given in Table II. The characteristic length for an elastic slab used in these formulae is equal to

$$L_s = \sqrt[4]{\frac{E_c \cdot h^3}{12(1-\nu_c^2) \cdot k_s}}$$

where E_c = modulus of elasticity of slab
h = thickness of slab
ν_c = Poisson's ratio of slab
k_s = modulus of subgrade reaction.

The formulae given in Table II for interior loading can be applied for loads at a distance of about one and a half to two times the characteristic length from the edge of the slab.
Losberg [16] considered failure mechanisms of infinitely large plates resting on an elastic foundation and subjected to single loads. One of his solutions is shown in Fig. 15. Fig. 15a shows the assumed fan-shaped failure mechanism under a single load F spread over a circular contact area of radius a. Losberg simplified his analysis by assuming a linearly variable soil pressure distribution. Fig. 15b gives the required flexural resistance as a function of the relative size of the contact area, a/L_s. The characteristic length, L_s, is of the order of two to four times the slab thickness, see Fig. 15c.

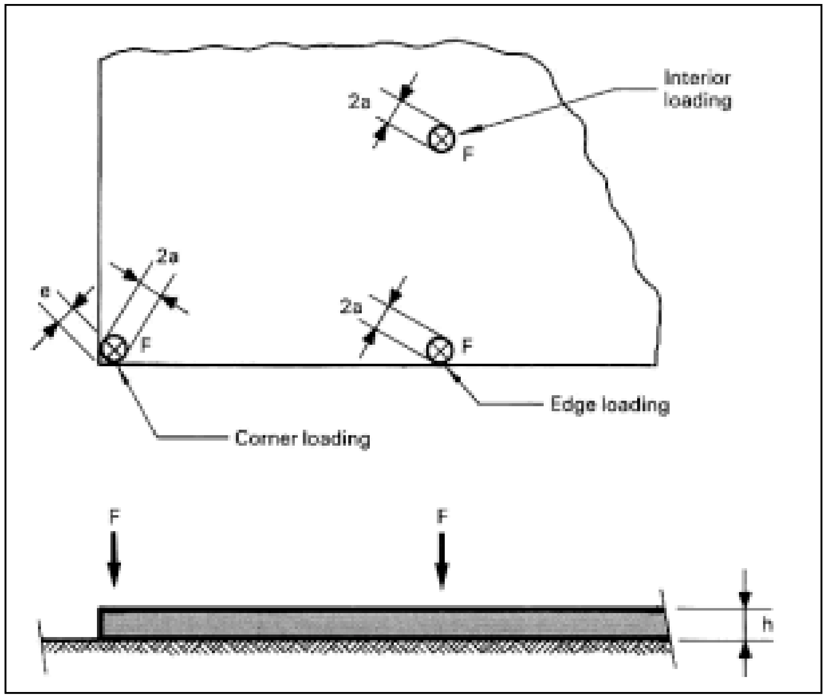


Figure 14: Slab on elastic foundation

	Interior loading F	Edge loading F	Corner loading F
Maximum moment	$\frac{F}{21.8}[\log_{10}(12 \cdot (\frac{L_s}{c})^4) - 0.44]$	$\frac{F}{11.3}[\log_{10}(12(\frac{L_s}{c})^4) + 0.62]$	$\frac{F}{2.0}[1-(\frac{c}{L_s}\sqrt{2})^{1.2}]$
Maximum soil pressure	$\frac{F}{8L_s^2}$	$3.3 \cdot \frac{F}{8L_s^2}$	$(8.8 - 7 \cdot \frac{e}{L_s}) \cdot \frac{F}{8L_s^2}$

$c = a \quad : \quad a \geq 1.7\,h$

$c = \sqrt{1.6 \cdot a^2 + h^2} - 0.675 \cdot h \quad : \quad a \leq 1.7\,h$

Table II: Westergaard's formulae

3.3. Effects of restraints and their mitigation

Volumetric changes due to shrinkage, temperature effects, prestressing and creep of the concrete are restrained both by subgrade friction and by restraining elements below and above the foundation, such as piles, elevator shafts, walls and so on. On the other hand, the foundation exerts restraining effects on the superstructure. Thus the designer generally faces a complex problem of mutual restraining actions.
The magnitude of the volumetric changes is in general not well known. Among other factors, it depends on the exposure conditions and the thicknesses of the different members. For example, pavements are typically subjected to severe temperature actions throughout their life span. On the other hand, a foundation of a building will experience only slight temperature variations after the construction period. Furthermore, soil moisture or ground water will reduce or even suppress shrinkage of the concrete.
The strain distribution through the thickness of a foundation slab is non-linear in general. General non-linear strain profiles can be split up into a constant, a linear and a non-linear component, i.e. axial deformations, curvatures and residual strains respectively.
Fig. 16 illustrates the behaviour of a long slab subjected to imposed, uniform curvatures. If the slab had no weight it would form a circular arch, in contact with the subgrade at the edges or at the centre for positive and negative curvature,

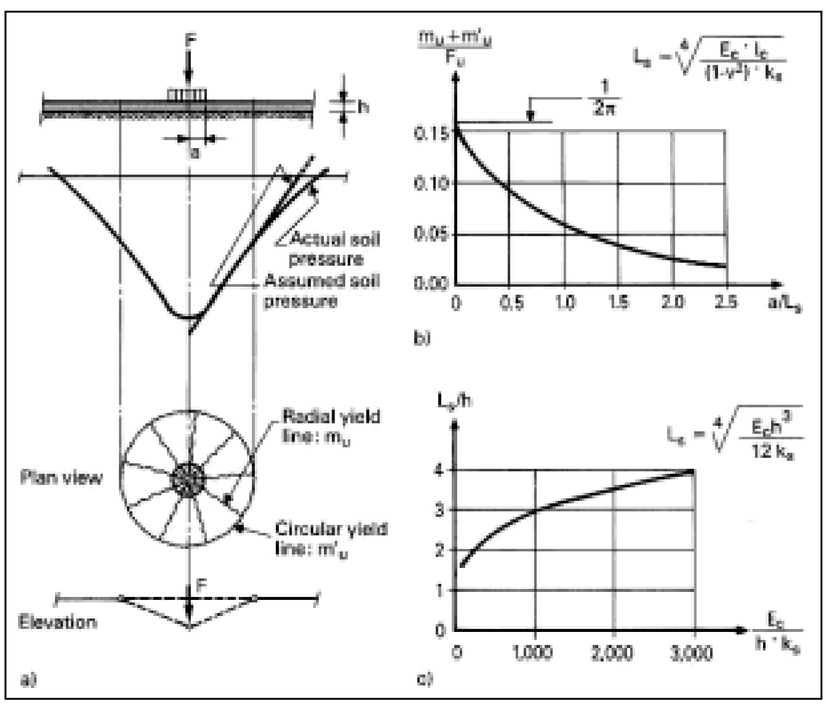


Figure 15: Limit analysis for slab on elastic foundation subjected to a single load: (a) Notation, soil pressure distribution and failure mechanism; (b) Required resistance; (c) Characteristic length

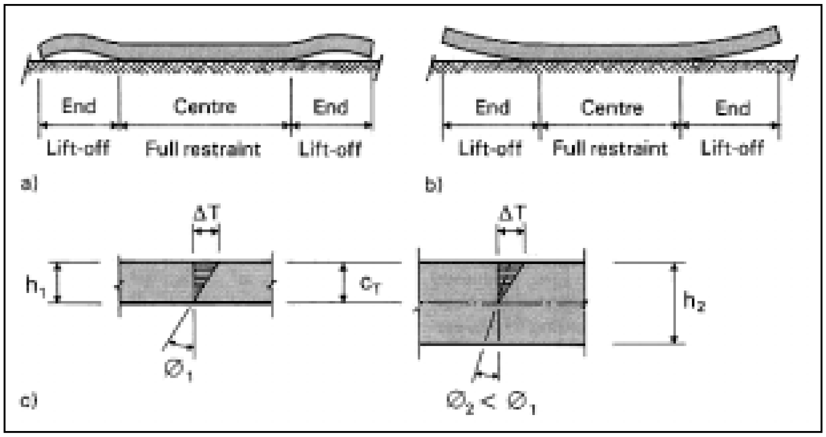


Figure 16: Imposed curvatures: (a) Positive curvature; (b) Negative curvature; (c) Thickness effect

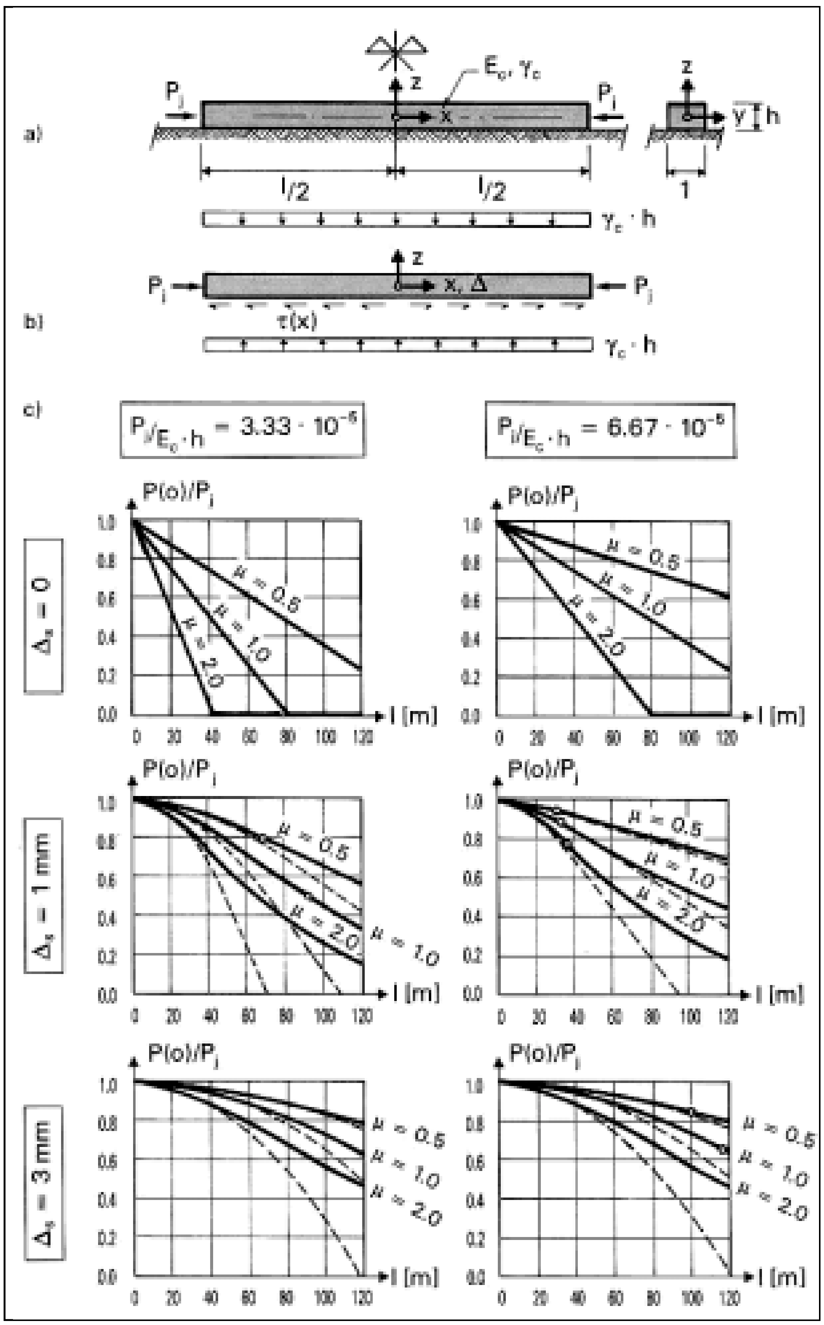


Figure 17: Loss of prestress due to subgrade friction: (a) System and notation; (b) Free-body diagram of slab; (c) Ratio of prestressing forces at centre and edge of slab

tive curvatures, respectively. The self-weight of the slab pushes down this arch, resulting in a straight, fully restrained region in the centre and two partially restrained, lift-off regions close to the edges.

The moments of the fully restrained central region are uniform and proportional to the slab stiffness. The moments in the partially restrained edge regions will be smaller. However, at the transition between the two regions there will be a peak moment about 20% greater than the uniform moment in the fully restrained region [17]. For shorter slabs, there may be no fully restrained region in the centre and hence such slabs will lift off and their moments will be reduced compared to long slabs.

Fig. 16c shows the effect of slab thickness on the magnitude of the imposed curvature due to a given temperature gradient. For a thin slab with a thickness h_1, equal to the influence depth of the temperature, c_T, a curvature Φ is obtained. For the same temperature gradient the curvature Φ_2 obtained in a slab of thickness $h_2 = 2h_1, = 2c_T$ will be equal to $\Phi_1/2$.

Under sufficiently large imposed curvatures, flexural cracks will occur. An unreinforced slab would tend to form a series of uncracked panels behaving as short slabs as discussed above. Concentrated rotations would occur at the transitions between panels. Correspondingly, crack width and crack spacings would both be large. For a reinforced slab, the reinforcement will prevent the development of concentrated rotations. As a consequence, crack spacings and crack widths will be limited.

Fig. 17 illustrates the influence of subgrade friction on the effective prestress in a foundation slab. Restraining actions due to other volumetric changes can be similarly treated. The slab strip shown in Fig. 17a is prestressed from both ends with the jacking force P_j. The free-body diagram of Fig. 17b shows the forces acting on the slab strip

- jacking force P_j,
- self-weight γ_c . h,
- subgrade friction τ.

By using the information given in Fig. 7, the diagrams shown in Fig. 17c can be established. These diagrams give the ratio of the effective prestressing force at the centre of the slab to the jacking force as a function of the total length of the slab, 1. Three values of friction coefficient, μ, are used, viz. μ = 0.5, 1.0, 2.0. Furthermore, two distinct levels of jacking forces are assumed, corresponding to prestress values of approximately 1.0 and 2.0 N/mm^2. Finally, three different values of Δ_s have been assumed. With $\Delta_s = 0$, Curve 1 of Fig. 7e, i.e. a rigid-plastic behaviour is considered. The four diagrams computed based on Δ_s = 1 mm and Δ_s = 3 mm pertain to an elastic-plastic behaviour, see Curve 2 of Fig. 7e. The six diagrams of Fig. 17c together cover a wide range of parameters encountered in practice. Note that the curves in Fig. 17c generally consist of two portions, corresponding to the elastic and plastic branches of Curve 2 of Fig. 7e.

Fig. 17c contains two sets of curves. While the solid lines accurately reflect the compatibility between the displacements of the slab and the subgrade the dashed lines are based on a simple approximation that ignores this compatibility. For the approximate solution the slab displacements due to the jacking force P_j. are considered to be unrestrained and the loss of prestress is computed as the integral of the associated subgrade friction forces. This sample approximation overestimates the loss of prestress but it is suitable for hand calculations and eliminates the need for evaluating differential equations resulting in hyperbolic functions. For relatively short slabs and/or low friction coefficients the approximations are quite reasonable.

Usual methods of analysis assume a rigidplastic shear-displacement relationship, i.e. $\Delta_S = 0$ [2], [3]. This results in the top two diagrams of Fig. 17c, which show a relatively pronounced influence of the slab length as well as the level of the jacking force on the effective prestress at the slab centre. For example, it can be seen that for a slab 40 m long and a friction coefficient of $\mu = 2.0$, there would be no effective prestress at the slab centre for the low jacking force level $P_j = E_c h/30{,}000$ while for the high jacking force level $P_j = E_c h/15{,}000$ the effective prestress at the centre would be 50% of that at the slab edges.

The usual method of analysis overestimates the influence of subgrade friction. In fact, Fig. 7 demonstrates that the actual shear-displacement relationships are better approximated by an elastic-plastic idealization. For the same 40 m long slab considered above, $\mu = 2.0$ and an assumed $\Delta_S = 1$ mm, the ratios of effective prestressing forces at the centre of the slab to the jacking forces amount to about 74% for both the low and the high level of the jacking force. For $\Delta_S = 3$ mm, corresponding ratios of 90% are obtained.

The usual method of analysis suggests that the length of a slab that can be effectively prestressed is proportional to the level of the jacking force, P_j, and inversely proportional to the friction coefficient, μ. For high levels of prestress and low friction coefficients, this assumption provides reasonable results. However, for relatively low levels of prestress and moderate to high friction coefficients, the loss of effective prestress along the slab is much less pronounced than is usually assumed.

The analysis underlying Fig. 17 assumes that the slab is prestressed while it is subjected to its self-weight only. In practice, foundation slabs often have to be prestressed in stages. Thus additional loads will be superimposed and subgrade friction will be increased. As a consequence, losses due to delayed jacking will generally be greater than reflected by Fig. 17c.

The jacking force level $P_j/E_c h$ used in Fig. 17c corresponds to a shortening of an unrestrained slab similar to an imposed axial deformation due to other volumetric changes. Hence the considerations underlying Fig. 17c can be generalized. In any case, there will be regions on either side of the centre of a slab within which the relative displacements between the slab and the subgrade will vary according to a hyperbolic function. For very long slabs, there will be an innermost, fully restrained region within which no relative displacements occur. This is similar to what was discussed in relation to Fig. 16. Furthermore, there may be regions next to the edges of the slab within which the relative displacements between slab and subgrade vary according to a second order parabolic function. Hyperbolic and parabolic variations of relative displacements correspond to the elastic and plastic branches of Curve 2 in Fig. 7e.

Fig. 18 provides a further illustration of these relationships. The slab strip shown in Fig. 18a is subjected to an imposed uniform shortening. The free-body diagram of Fig. 18b presents the restraining action of subgrade friction, resulting in the shear stress and tensile force diagrams given in Figures 18c and 18d, respectively. Fig. 18e differentiates between a fully restrained interior region and partially restrained exterior regions. As discussed above, the relative displacements between slab and subgrade vary according to a hyperbolic function close to the fully restrained region, whereas towards the slab edges a parabolic variation may occur. For short slabs there is no fully restrained interior region and the tensile force diagram will be as shown in Fig. 18f.

Apart from subgrade friction, restraining actions due to structural elements framing into a foundation must be considered. Several typical examples of such elements are illustrated in Fig. 19. Fig. 20 contains the results of an approximate investigation of the restraining effects of piles. The problem has been simplified by neglecting flexural deformations of the slab and by assuming a uniform modulus of horizontal subgrade reaction. With these assumptions, the potential losses of prestress are certainly overestimated. However, as Fig. 20b reveals, even with these pessimistic assumptions the losses are of the order of 10% or less of the jacking force for practical conditions.

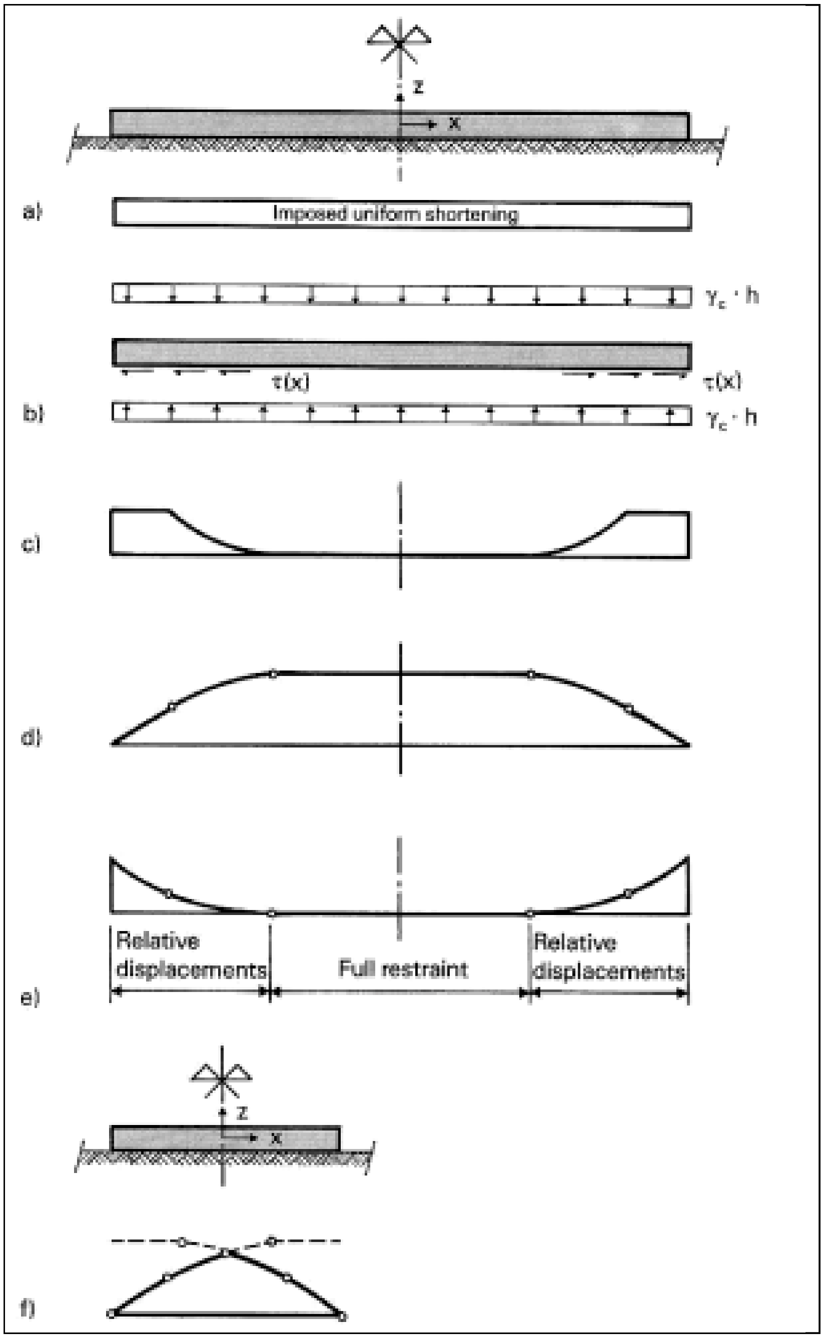


Figure 18: Imposed axial deformation: (a) Slab subjected to imposed uniform shortening; (b) Free-body diagram of slab; (c) Distribution of shear stresses; (d) Tension in slab; (e) Displacements of slab; (f) Tension in short slab

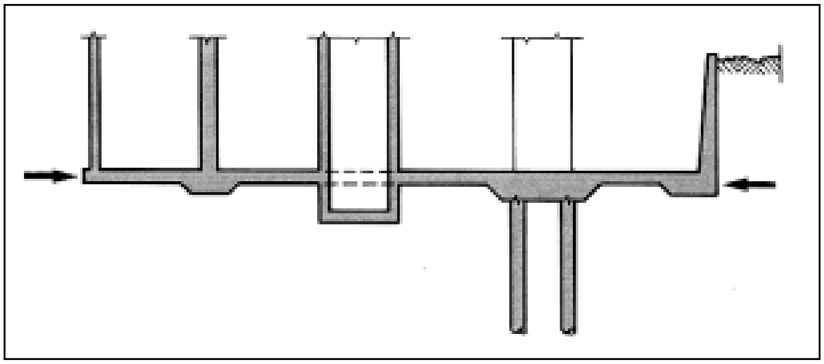


Figure 19: Restraining elements

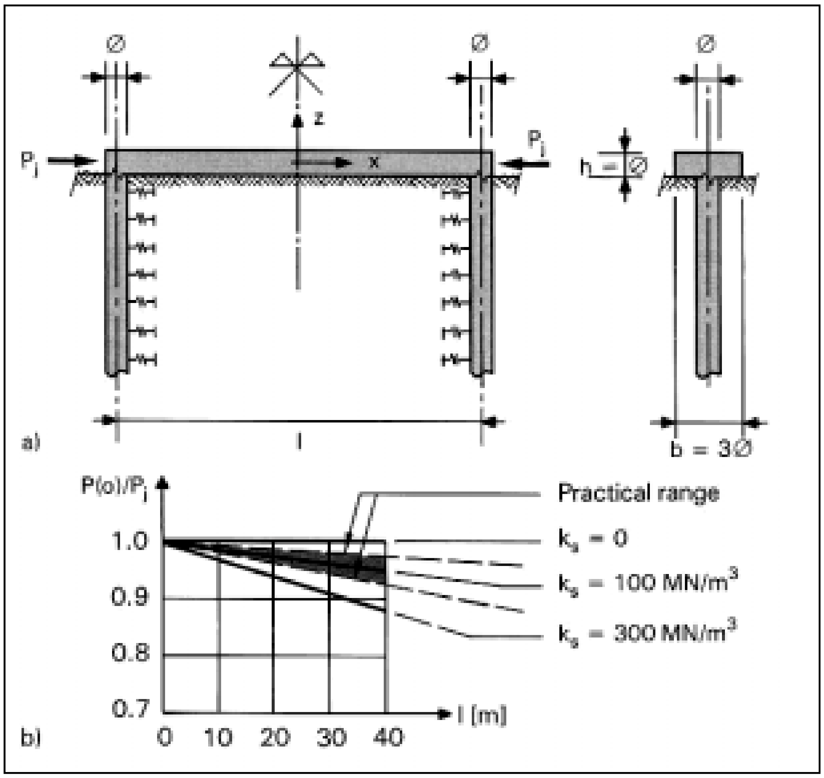


Figure 20: Restraint offered by piles: (a) System and notation; (b) Ratio of prestressing forces in the interior and at the edges of the slab

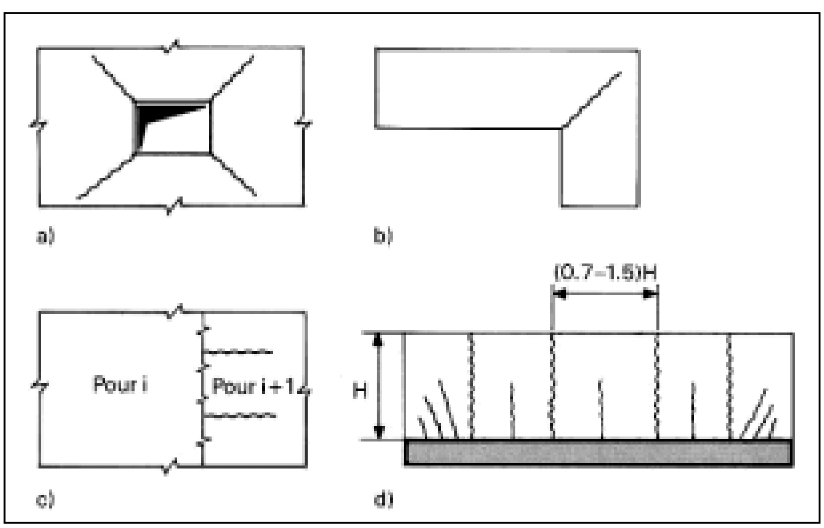


Figure 21 : Typical crack patterns: (a) Cracks at corners of slab opening; (b) Re-entrant corner cracking in L-shaped foundation slab; (c) Cracking at cold joint in foundation slab; (d) Cracks in wall on foundation

Under sufficiently large imposed axial deformations, cracks through the thickness of foundation slabs may develop. An unreinforced slab would tend to form a series of relatively large, uncracked panels separated by wide cracks at which the deformations would be concentrated. The purpose of the reinforcement is to enforce a narrow crack spacing and to limit crack widths.

As mentioned in Section 2.3., certain differences in the cracking behaviour of foundation structures prestressed with either bonded or unbonded tendons have to be taken into account. While crack initiation characteristics are essentially the same, the crack propagation behaviour is different. Slabs with unbonded tendons generally exhibit a poorer cracking performance because the lack of bonded reinforcement prevents them from mobilizing the concrete in the immediate vicinity of a crack. Consequently, a series of large slab segments separated by relatively wide cracks may be produced, while well distributed narrow cracks would occur in a slab with a sufficient amount of bonded reinforcement.

In evaluating the crack propagation characteristics of foundation slabs with unbonded tendons, it should be acknowledged that, whereas flexural cracks may be well distributed owing to the combined action of the effective prestress and restraints of in-plane deformations [10], [18], such a favourable behaviour may not occur in regard to cracks caused by imposed axial deformations.

Fig. 21 shows a number of typical crack patterns that have repeatedly been observed in practice. Re-entrant corners are particularly vulnerable to diagonal cracking, see Figures 21a and 21b. Another frequent cause of cracking lies in the different deformation characteristics of adjoining elements cast at different times, see Figures 21c and 21d.

Measures to mitigate restraint cracks include:

- adequate concrete mixing and curing techniques,
- appropriate casting and stressing sequence,
- addition or improved layout of non-prestressed and prestressed reinforcement,
- provision of structural separations,
- partial releases of sobgrade friction.

The importance of adequate concrete curing techniques is well-known and can hardly be overemphasized. In fact, post-tensioning has frequently been blamed for cracks which actually were due to improper curing.

Figures 22 to 24 summarize a number of casting and stressing sequences that have been applied with success. One of the distinct advantages offered by prestressing lies in the possibility of controlling hydration cracks by an early application of a certain prestress. Temporary joints are another measure that has frequently been used for reducing the effects of volumetric changes.

Figures 25 and 26 show favourable reinforcement layouts for mitigating restraint cracking in the vicinity of re-entrant corners or openings. By lapping tendons as indicated in these Figures, a known amount of prestress can be introduced into such regions and thus crack initiation may be delayed.

Significant irregularities both in the plan and in the elevation of a building may necessitate a complete structural separation of parts of a building, see Fig. 27. However, apart from considerable costs, structural separations involve other disadvantages such as a reduced redundancy of the overall structural system. As a further measure of crack mitigation, partial release of the subgrade friction at the base of a foundation should be mentioned. Fig. 28 summarizes different possibilities for the detailing of the soil-foundation interface.

Typically, the foundation concrete is placed on a thin layer of plain concrete, which is used for levelling the base, see Fig. 28a. As an alternative, a thin layer of sand topped by one layer of polyethylene sheeting can be used, as shown in Fig. 28b. Fig. 28c presents an improved layout, which utilizes a double layer rather than a single layer of polyethylene sheeting. Finally, Fig. 28d shows an alternative detail employing sheet asphalt or bitumen.

Whereas details aimed at reducing subgrade friction such as the ones shown in Fig. 28 have frequently been used for pavements and slabs on grade, their effectiveness in connection with building structures is reduced. Indeed, the addition of gravity loads during construction of the superstructure typically results in an increase of the subgrade friction which more than offsets any possible reduction of the friction coefficients.

Rather than investing in measures for reducing subgrade friction, it may be advisable to allocate certain funds for a one-time crack maintenance operation [19].

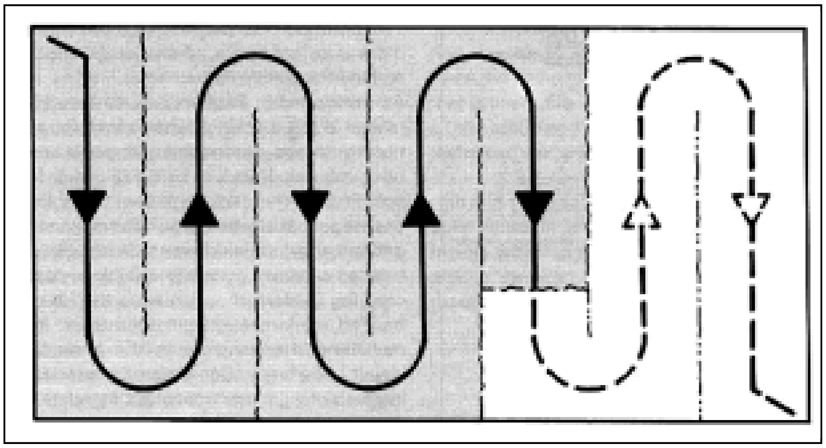


Figure 22: Suggested placing of concrete

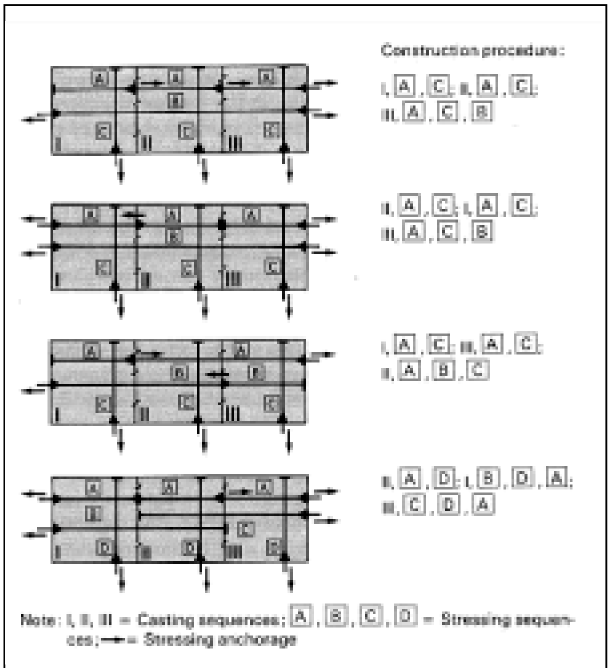
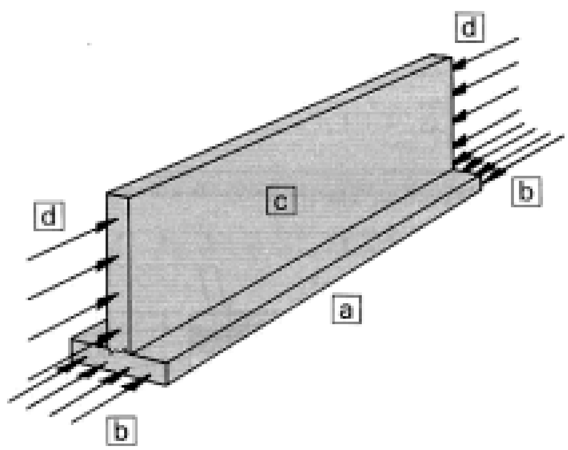


Figure 23: Possible casting and stressing sequences of foundation slab



Day	Activity
1	a Cast foundation
3	b Stress foundation tendons to 30%
7	c Cast wall
9	b Stress foundation tendons to 60% and
	d Stress wall tendons to 50%
18	b Stress foundation tendons to 100% and
	d Stress wall tendons to 100%

Figure 24: Possible stressing sequence of foundation slab and wall

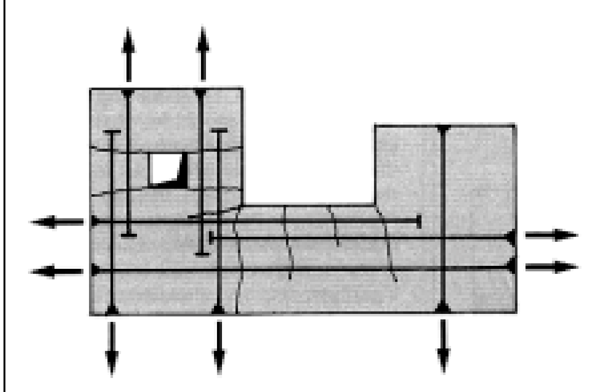


Figure 25: Favourable tendon layout in irregularly shaped foundation slab

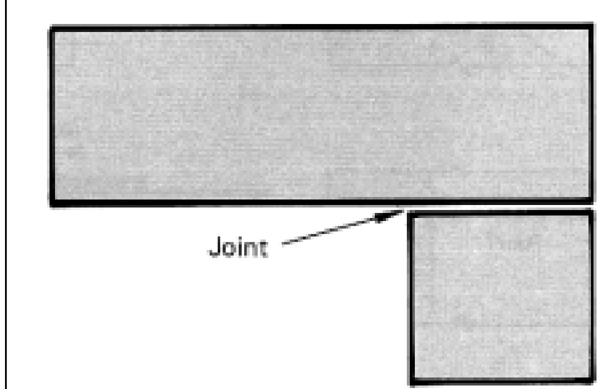


Figure 27: Structural separation

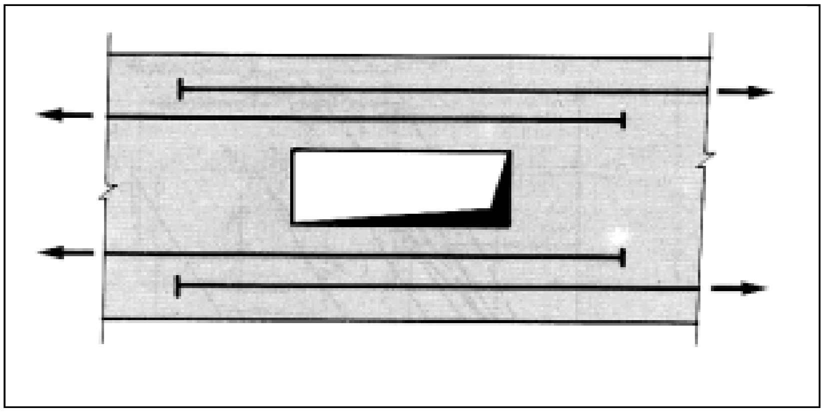


Figure 26: Overlapping of tendons at slab openings

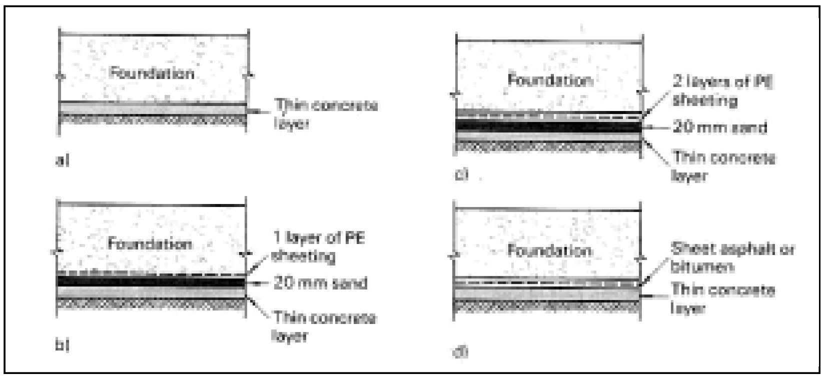


Figure 28: Possible detailing of interface at base of foundation

3.4. Checks at transfer

In order to control hydration cracking, it is of interest to stress tendons as early as possible. Furthermore, from the practical aspect it is desirable to stress all tendons simultaneously, rather than adopting a number of stressing stages. However, care must be exercised in order not to overstress the slab at transfer. This might occur because, on the one hand, the design strength of the concrete is not yet reached while, on the other hand, the gravity loads from the superstructure do not yet balance the deviation forces of the tendons.

Figures 29 and 30 present the effects of prestressing as well as a possible method of checking for sufficient strength at transfer. Consider the continuous foundation with the draped tendon profile shown in Fig. 29a. Points of inflection of the tendons are assumed to be located at the one-tenth points of the spans and the maximum tendon eccentricities are taken as equal to 40% of the slab thickness, h. Except for the self-weight and the prestressing of the foundation, no other applied loads are considered and the subgrade is modelled by assuming a uniform modulus of subgrade reaction, k_s. Figures 29c, 29d and 29e present primary, secondary and total moment diagrams, respectively. Provided that no lift-off of the foundation occurs, the secondary moments may vary between the negative of the primary moments and zero, depending upon the value of k_s, see Fig. 29d. The corresponding variation of the total moments is given in Fig. 29e. For practical values of ks, the extreme total moments always occur at the low points of the tendon profile. Note that, compared with the moments shown in Fig. 29e, the moments due to eccentricity of the subgrade friction forces can be neglected. This is demonstrated by Fig. 29f.

It should be noted that the assumption of no lift-off of the foundation is certainly justified for the final stage, where large column loads prevent any upward bending. At transfer, however, lift-off may occur. This effect is investigated in Fig. 30. Fig. 30a shows the same structural system as Fig. 29a and Fig. 30b illustrates the deflected shape of the foundation at transfer. The interaction curves given in Fig. 30d are based on the assumptions summarized in Fig. 30c. Fig. 30d comprises five solid lines, representing the strength of symmetrically reinforced sections according to Fig. 30c having mechanical reinforcement ratios $\omega_s = A_s f_y / hf_{ci}$ ranging from zero to 0.20, where A_s = crosssectional area of reinforcement, f_y = yield strength of reinforcement, h = thickness of section and f_{ci} = compressive strength of concrete at transfer. The two straight dashed lines in Fig. 30d correspond to the extreme moments indicated in Fig. 29e for k_s = 50 MN/m^3 and k_s = 200 MN/m^3, respectively. If lift-off of the foundation is taken into account, the influence of k_s on the extreme moments disappears for all practical intent and purposes. Furthermore, extreme moments increase more than proportionally to increasing values of the prestressing force. This is demonstrated by the thick solid curve in Fig. 30d, which indicates the increase of the extreme effects of prestressing. If this curve intersects the

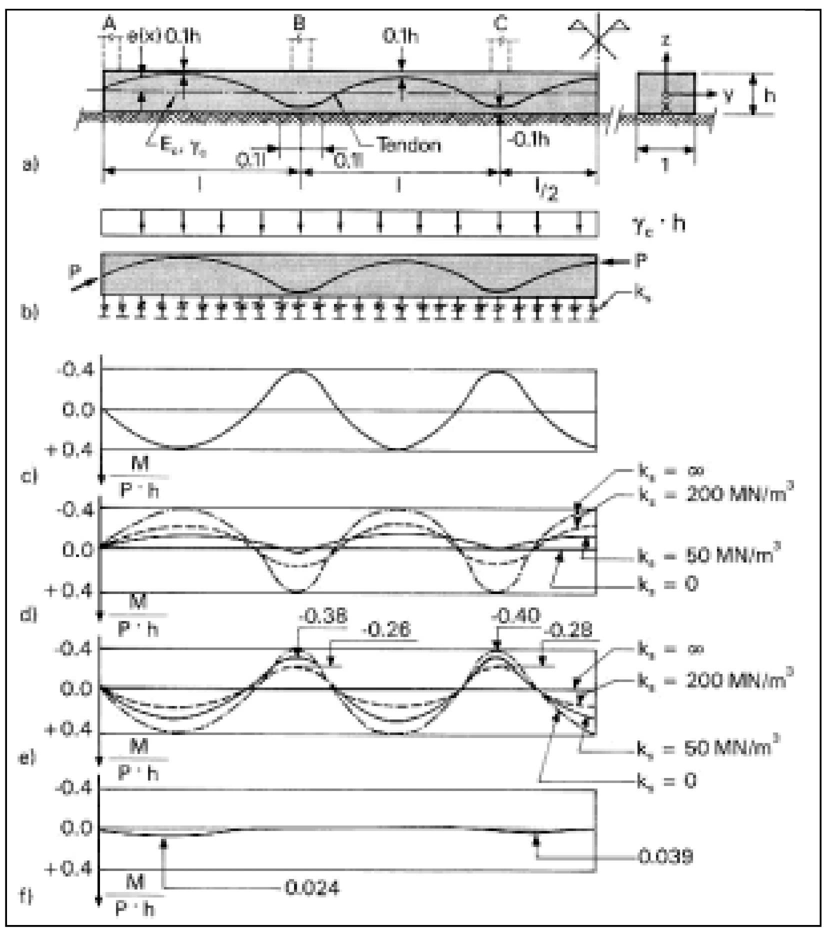


Figure 29: Flexural effects of prestressing in continuous foundation: (a) System and notation; (b) Static system; (c) Primary moments; (d) Secondary moments; (e) Total moments; (f) Moments due to eccentricity of subgrade friction

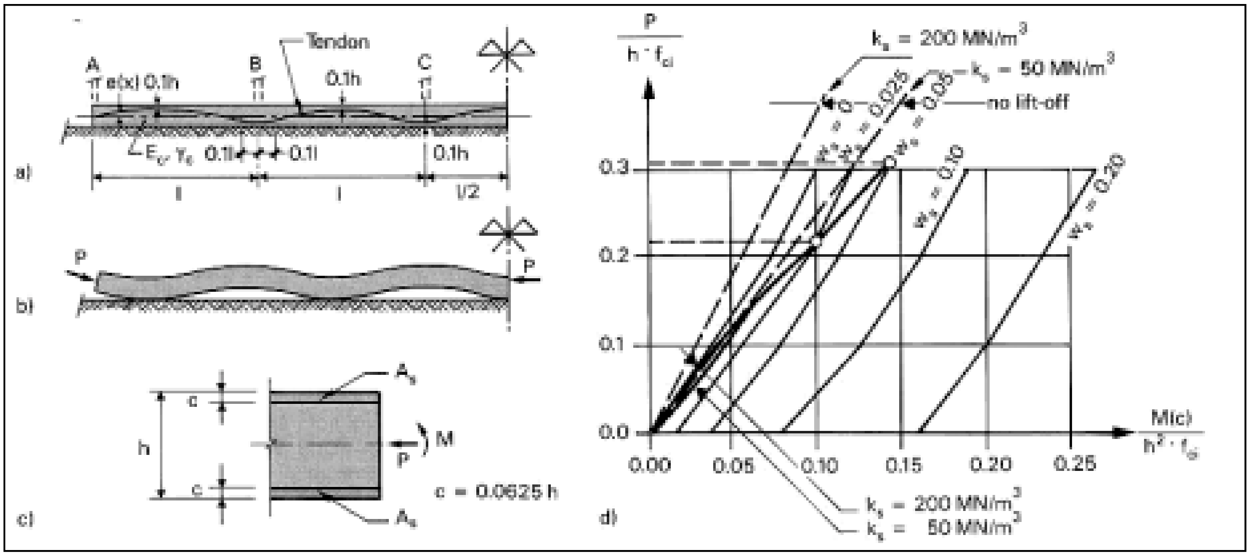


Figure 30: Check of flexural resistance of continuous foundation at transfer: (a) System and notation; (b) Deflected shape of foundation; (c) Assumptions for derivation of interaction diagrams; (d) Interaction of flexure and axial load

interaction curve for a given section, a failure condition will be reached. For example, for ω_s = 0.025 this condition is reached for P = 0.22 hf_{ci}. With f_y = 450 N/mm^2 and f_{ci} = 9 N/mm^2, this means that a reinforcement ratio A_s/h = 0.05% would theoretically suffice for a prestress P/h = 2.0 N/mm^2. Note that no safety factors have been included in this computation.

Whereas the results represented in Fig. 30d are directly applicable to foundation slabs with uniformly distributed tendons, more critical conditions may occur in slabs comprising banded tendons. This is due to the concentration of the effects of prestressing within a narrow width of the slab, which may result in premature local failures.

3.5. Checks under service conditions

Usual checks of serviceability limit states include comparisons of computed stresses in the concrete and in the reinforcement with allowable limits specified in codes. In general, effects due to loads and to imposed deformations are superimposed. Except for the uncertainty about initial residual stresses in the structure, such a procedure might still result in reasonable average stress values prior to cracking. However, it cannot cover the effects of cracking, since as soon as the structure cracks the influence of imposed deformations can no longer be treated via the consideration of stresses. Hence, at best, usual methods of stress analysis provide an indication of whether cracking will occur or not and they give an order of magnitude for the average stresses in the uncracked structure.

With regard to the behaviour of cracked structures, the usual methods of stress analysis are not capable of reflecting the actual stresses within the structure. Nevertheless, experienced designers have successfully applied such methods to obtain a basis for the dimensioning and detailing. However, as such an approach can theoretically only be justified on the basis of limit analysis of perfect plasticity, limit analysis procedures could just as well be applied in the first place.

Therefore, it is suggested to provide an adequate amount of bonded reinforcement and to perform ultimate limit state checks, rather than to focus on checks under service conditions, if a structure is expected to crack. Minimum bonded reinforcement is necessary to ensure a ductile and redundant behaviour of the structure and to eliminate dangerous strain localizations caused by imposed deformations.

3.6. Checks at ultimate

Compared with the ultimate limit state design of other structural members, the design of foundations does not involve any special difficulties. In particular, either the design may be based purely on the effects of loads or, alternatively, any suitable residual stress state may be superimposed onto these effects.

With regard to the dimensioning of transversely loaded foundation slabs, membrane forces are generally neglected since they are not necessary for equilibrium reasons. This corresponds to the usual practice of the dimensioning of transversely loaded slabs in buildings. Thus the stress resultants given in Fig. 31 are the only ones considered. The bending moments m_x and m_y and the twisting moment $m_{xy} = my_{yx}$ are accompanied by shear forces v_x and v_y.

The dimensioning for moments is usually based on the so-called normal moment yield criterion [20]. For an orthogonally reinforced concrete slab, one obtains

$$m_{xu} = m_x + k \left| m_{xy} \right|$$

$$m_{yu} = m_y + \frac{1}{k} \left| m_{xy} \right|$$

where m_{xu} and m_{yu} = flexural resistances due to prestressed and non-prestressed reinforcements in the x- and y-directions, respectively, and k = a positive factor which is frequently set equal to 1.

The remainder of this section concentrates on some remarks regarding the transfer of transverse shear forces [21].

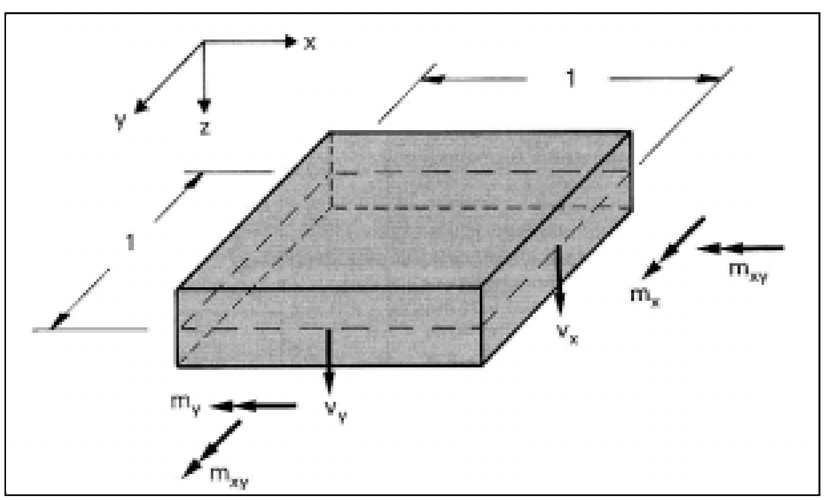


Figure 31 : Stress resultants on slab element

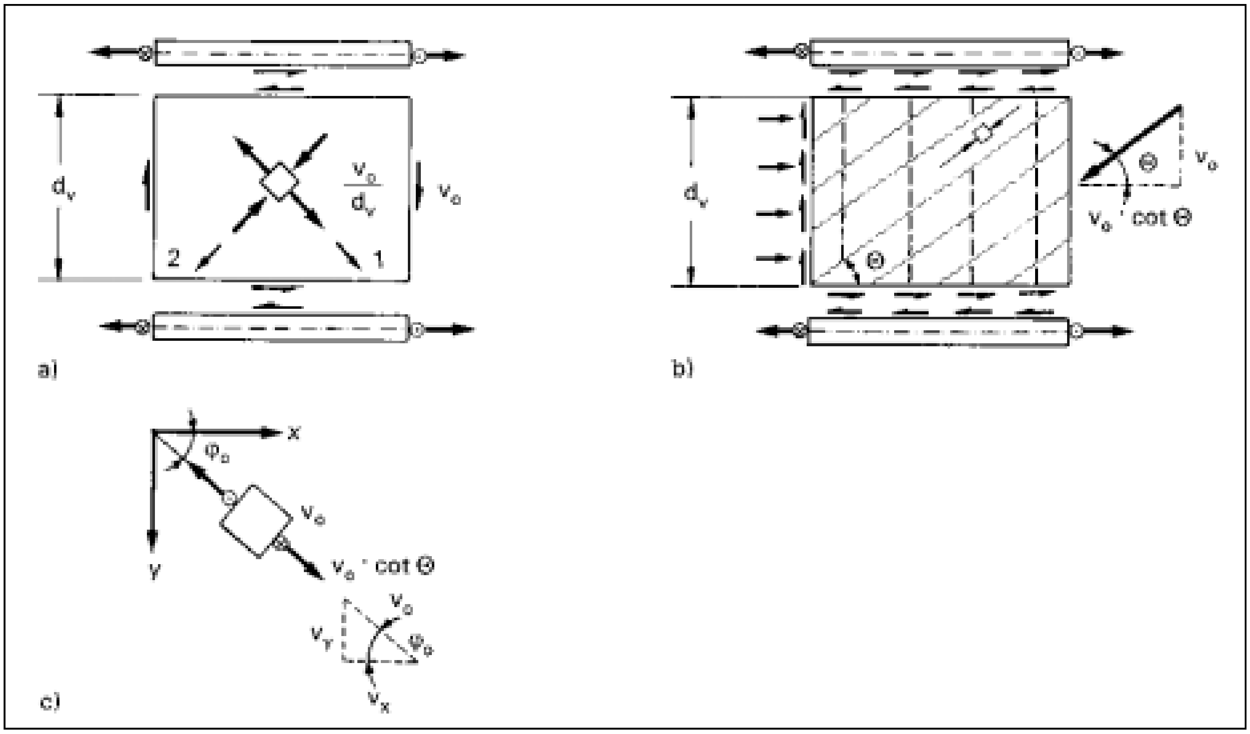


Figure 32: Shear transfer in slabs: (a) Untracked behaviour; (b) Cracked behaviour; (c) Principal shear

Provided the principal tensile stresses due to the shear forces do not exceed the tensile strength of the concrete, shear forces are carried by inclined compressive and tensile stresses in the interior of a slab, see Fig. 32a. If the tensile strength of the concrete is exceeded, diagonal compressive stress fields will develop in the concrete in the interior of the slab, see Fig. 32b. The diagonal compressive stresses correspond to tensile forces in the transverse and longitudinal reinforcement. This behaviour can easily be visualized by applying truss models [22].

In Figures 32a and 32b, the slab has been idealized as a sandwich consisting of two thin thin covers and a core of thickness d_v. Flexural and twisting moments are assumed to be carried by the covers, while shear forces are assigned to the core.

The shear force v_o indicated in Figures 32a and 32b is the so-called principal shear force

$$v_o = v_x^2 + v_y^2$$

which is being transferred in the direction

$$\varphi_o = \tan^{-1} \frac{v_y}{v_x}$$

as shown in Fig. 32c.

Resisting the principal shear force vo by a diagonal compressive stress field in the core concrete, as shown in Fig. 32b, necessitates a compression v_o .cot Θ which must be equilibrated by equivalent tensile forces in the covers of the sandwich model. Thus the longitudinal reinforcement required for bending and twisting moments must be strengthened to allow for shear transfer [21]. The geometric reinforcement ratio of the necessary transverse reinforcement amounts to v_o .tan Θ / ($d_v f_y$) where v_o = principal shear force, Θ = inclination of diagonal compressive stress field, d_v = effective depth in shear, and f_y = yield strength of transverse reinforcement.

Transverse components of the tensile forces in prestressing tendons should be taken into account in evaluating the shear resistance. For example, for the punching failure mechanism illustrated in Fig. 33, the beneficial action of the tendons crossing the punching cone equals the sum of all individual contributions P sin α in the x- and y-direction. Furthermore, Fig. 33 demonstrates that soil pressures q_u acting on the punching cone can also be deducted from the applied punching load V_u. This corresponds to the so-called staggering effect of shear design that has recently been discussed for shear in beams [23].

In order to apply conventional section-bysection design procedures, it is appropriate to introduce the resultant forces and moments due to prestressing at a section as externally applied loading acting on the section. With regard to stress increases beyond the initial prestress, a bonded, prestressed reinforcement can then be treated in a similar way to a non-prestressed reinforcement, while for the consideration of stress increments in unbonded prestressed reinforcements the deformations of the entire structural system have to be taken into account in general. As a simplification, potential stress increments in unbonded prestressing tendons are frequently neglected altogether [10], [18].

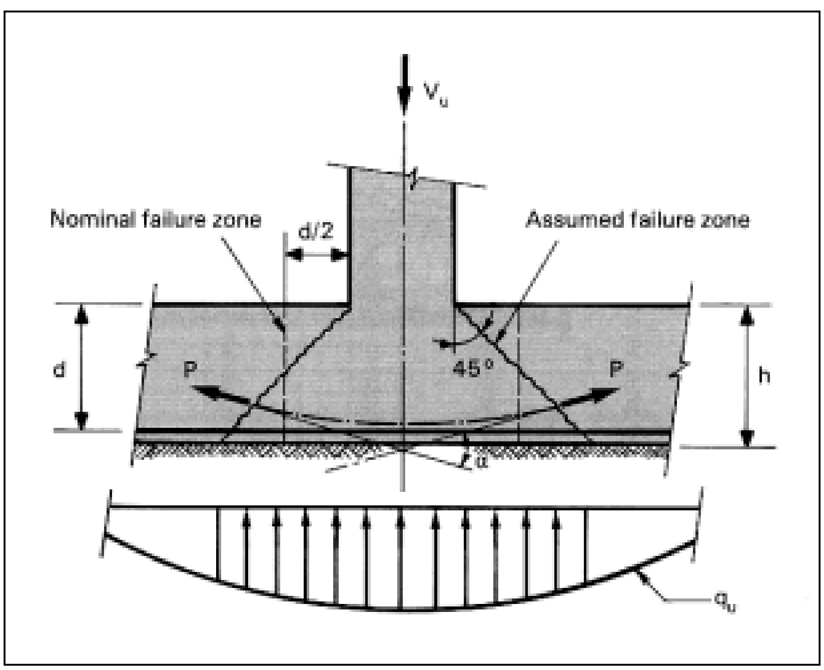


Figure 33: Punching shear

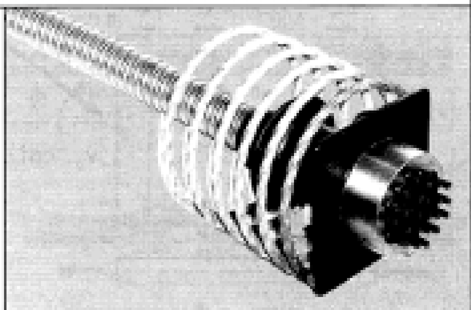
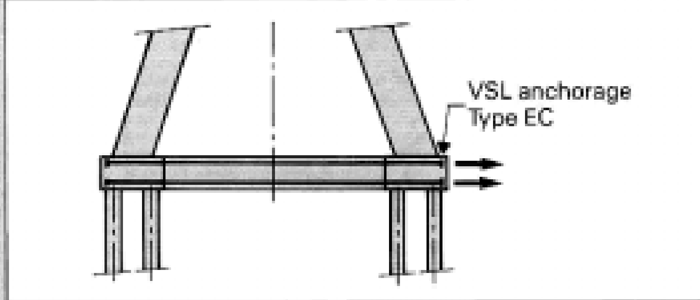
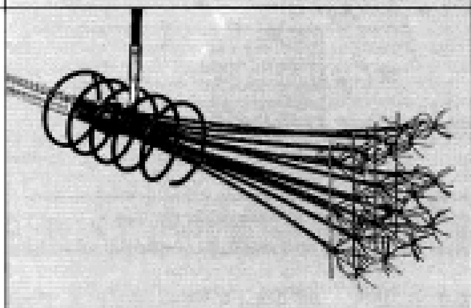
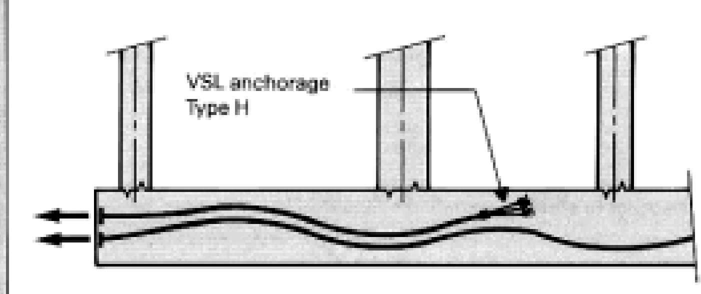
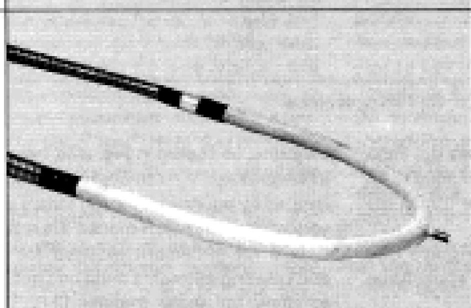
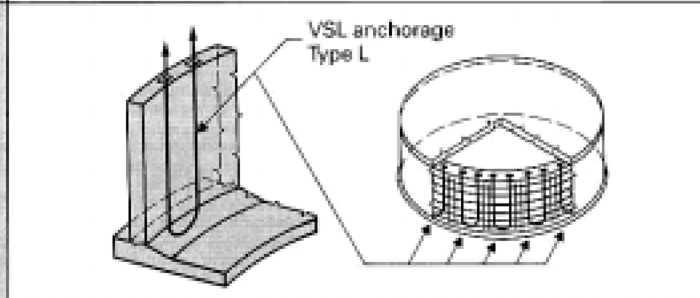
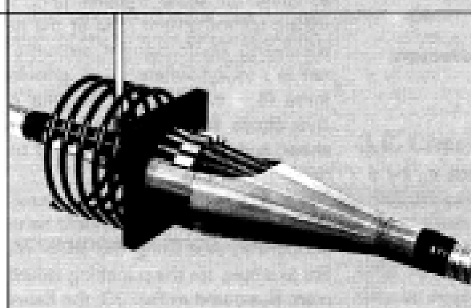
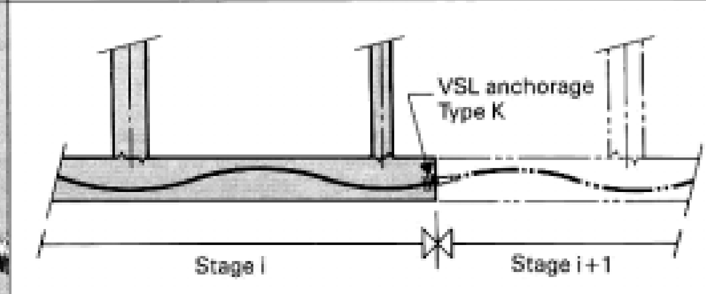
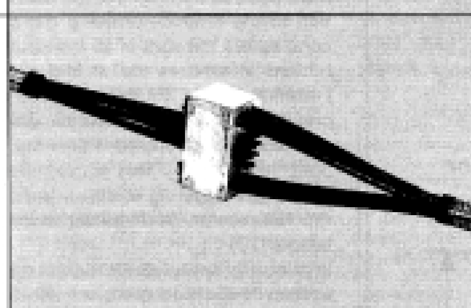
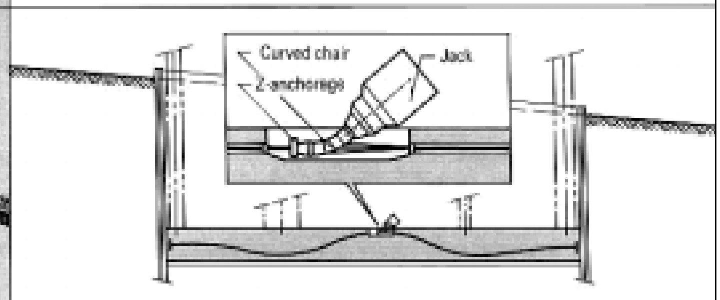
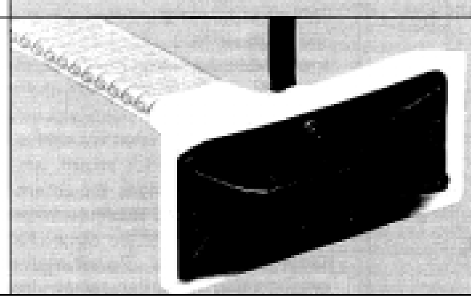
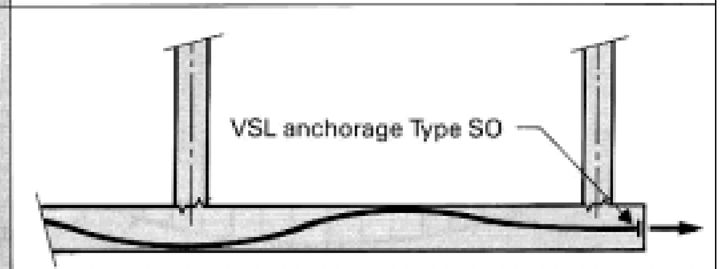
	Anchorage type	Typical application
EC		
H		
L		
K		
Z		
SO		

Figure 34: VSL Multistrand System - Anchorages and typical applications

4. Detailing

4.1. VSL Multistrand System

The VSL Multistrand Post-tensioning System offers a wide range of cable sizes and anchorage possibilities [24]. A number of anchorage types together with typical applications are illustrated in Fig. 34.
The stressing anchorage Type EC enables concentrated forces to be transferred. The strands can be installed after placing the ducts or after concreting.
The dead-end anchorage Type H is primarily suited to applications in the interior of a structure, and it permits a relatively smooth introduction of the prestressing force into the concrete. The strands must be installed before the concrete is cast.
The loop anchorage Type L is activated by stressing the tendon simultaneously at both ends. Strands can be installed after concreting.
The coupler Type K enables cables to be connected at construction joints, while the centre stressing anchorage Type Z enables tendons with inaccessible end anchorages to be stressed.
The VSL anchorage Type SO allows up to five strands placed in flat ducts to be stressed. This system is particularly suited to applications in relatively thin members.

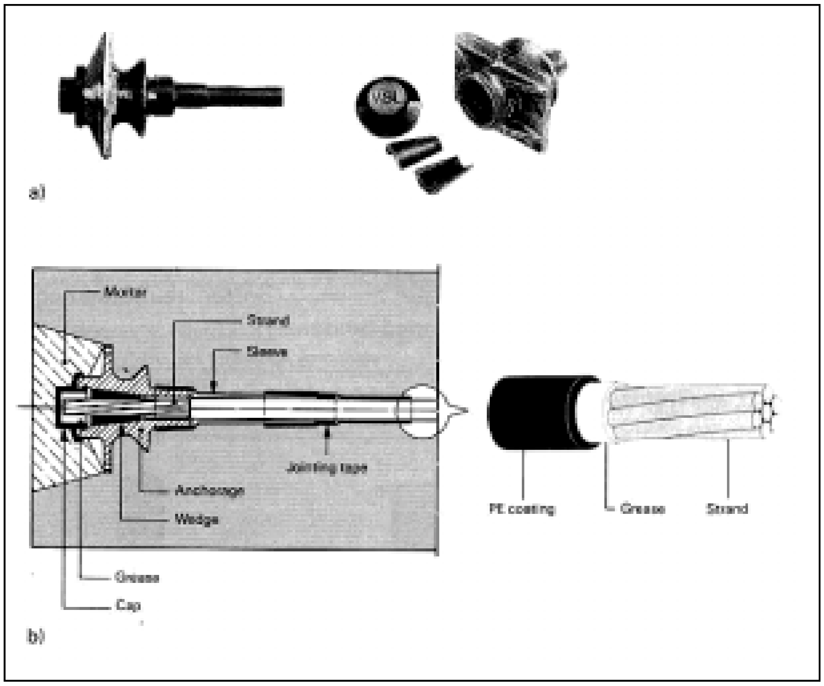


Figure 35: VSL Monostrand System: (a) Anchorage components; (b) Corrosion protection

4.2.VSL Monostrand System

Fig. 35a shows an end anchorage for monostrands. Stressing anchorages and dead-end anchorages are identical. Details of the special corrosion protection system are illustrated in Fig. 35b [25].
With regard to the application of monostrands, the reader is referred to Section 2.3. as well as to the literature [2], [3].

4.3.Tendon layout and profile

Fig. 36 illustrates various possibilities for the layout of prestressing tendons in mat foundation In any case several tendons are concentrated or banded along the column lines. Tendons placed in the areas between the banded tendons produce a more uniform soil pressure distribution. Furthermore, such distributed tendons produce a favourable prestressing effect near the edges or construction joints of mat foundations. If banded tendons only were used, the concrete near the edge between bands would receive only little if any prestress.
Placing of banded and distributed tendons in two directions requires thorough planning and detailing in order to avoid problems at tendon crossings. In particular, the placing sequence should be planned so that no threading of tendons is necessary. From this practical point of view, arrangements with banded tendons only or with banded and distributed tendons in only one direction may be preferable. However, from a structural point of view, a two-way layout of both banded and distributed tendons is generally superior.
Typical details of a one-way beam and slab foundation are shown in Fig. 37. This structural system employs banded tendons in the beams and distributed tendons in the slab.
The tendon profiles are composed of straight and parabolic or circular segments, see Fig. 38. In determining the tendon profile, attention should be paid to the allowable minimum radii of curvature for the particular tendons being used [24], [25]. Minimum lengths of straight tendon segments at anchorage locations must also be respected [24].
In order to ensure accurate placing of the tendons, appropriate cable supports must be used, see Fig. 39. The cable chairs should be sufficiently stable to prevent misplacements of the tendons during construction. Cable chairs must be sufficiently closely spaced to minimize wobble effects. Typically, a chair spacing of the order of 1 m is adopted.

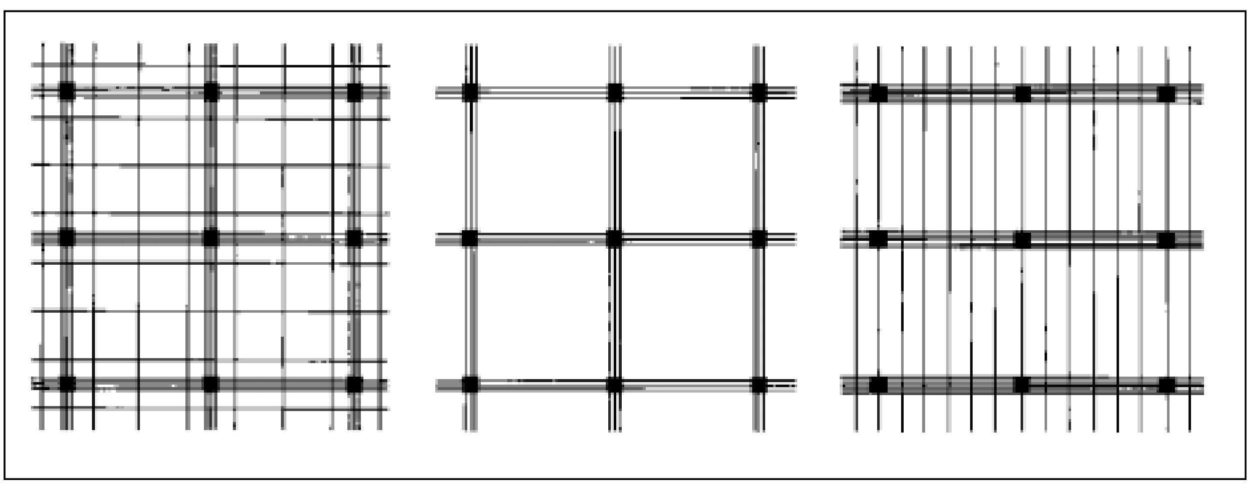


Figure 36: Typical tendon layouts

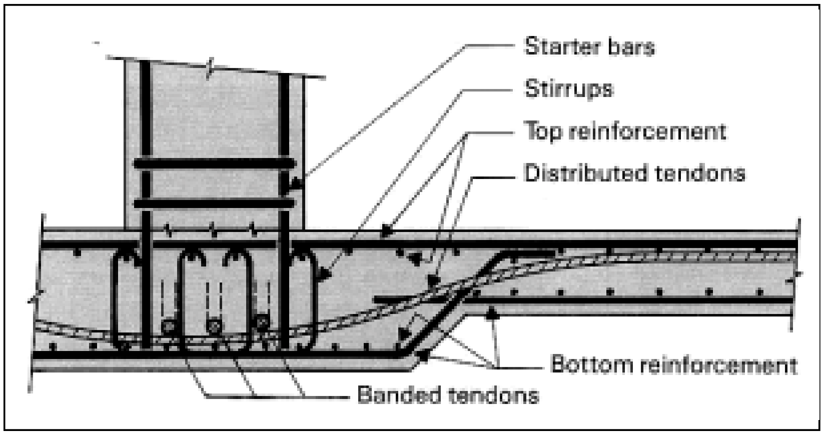


Figure 37: Reinforcing details for one-way beam and slab structure

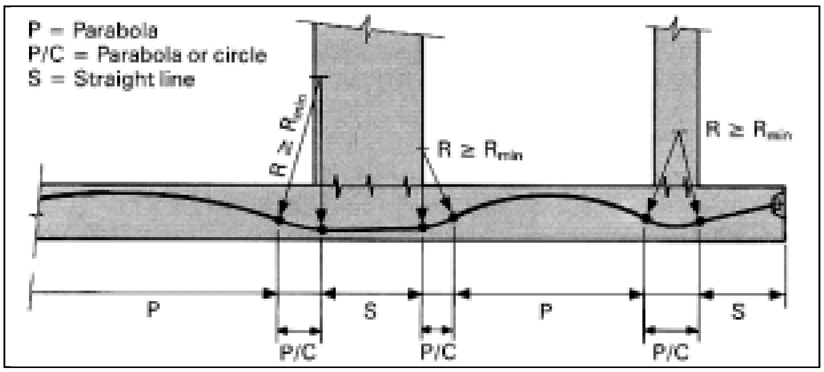


Figure 38: Typical tendon profile

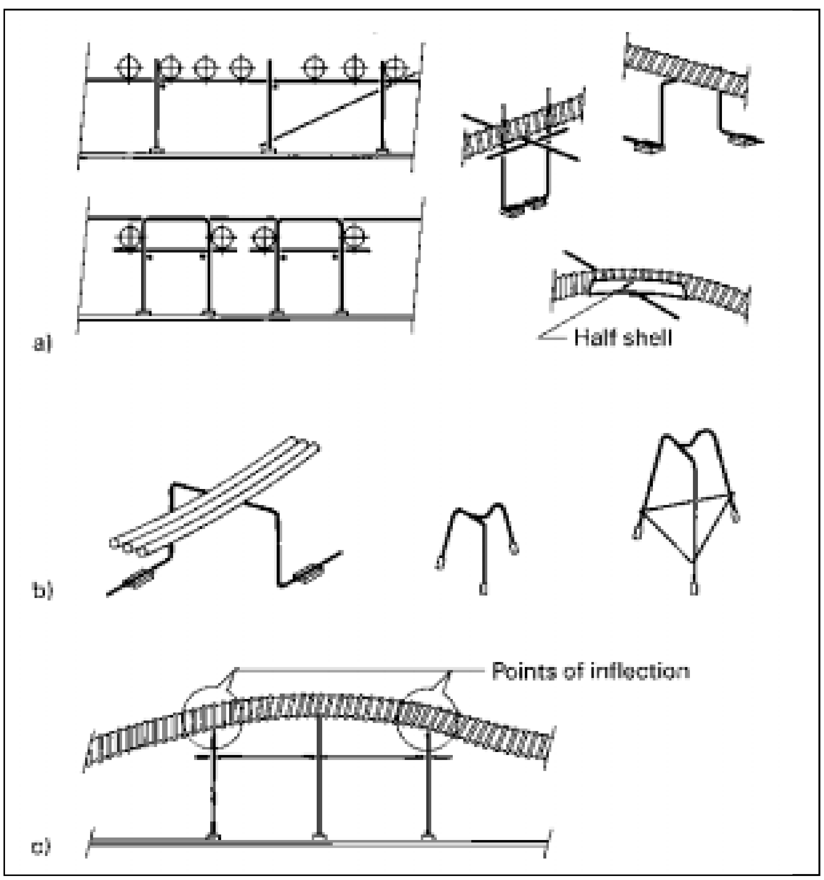


Figure 39: Cable supports: (a) Multistrand cable chairs; (b) Monostrand cable chairs; (c) Placing of chairs

vicinity of points of inflection of the tendon, closer spacings of the cable chairs may be required. In particular, this applies in regions near the high points of tendons, see Fig. 39c, and to any other situation where a particular slope of a tendon must be maintained.

Thorough planning and detailing of the tendon layout and profile must be complemented by adequate checking on site. In this regard, it should be recognized that, while the top surface of a foundation slab is well defined, its bottom surface cannot in general be determined with the same accuracy. Hence, checks of the cable profile should refer to the top surface. Unlike the situation with suspended slabs, it is not sufficient simply to measure the height of the cable chairs.

4.4. Additional considerations

As for any prestressed structure, a sufficient reinforcement must be provided in the cable anchorage zones of post-tensioned foundations to avoid bursting and spalling problems. It should, however, also be taken into account that proper functioning of the anchorages requires a sufficient concrete strength. The desire for an early full application of the prestressing forces is at variance with this requirement. One possible way of overcoming this difficulty is to use prefabricated anchorage blocks as shown in Fig. 40.

The use of waterstops at construction joints of foundation structures in ground water requires careful detailing of the non-prestressed and prestressed reinforcements in the joint region. However, the effective prestress at such a joint may eliminate the need for a waterstop, provided that appropriate care is taken in preparing the surface of the old concrete and in placing the new concrete [26].

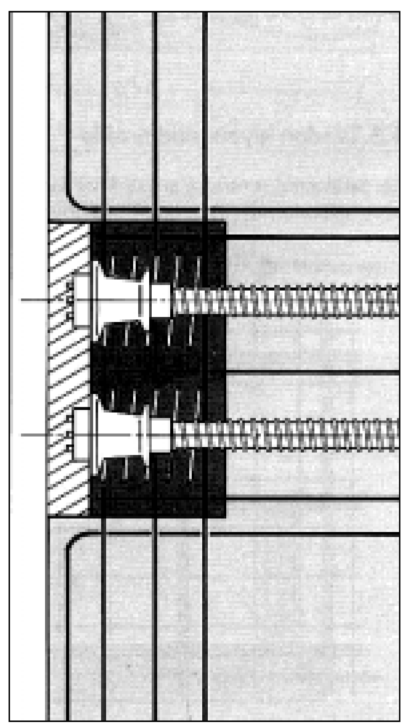


Figure 40: Prefabricated anchorage block

5. VSL Service Range

5.1. General

The VSL Organizations*) can offer a very comprehensive range of services in connection with post-tensioned foundations, namely:
- Consulting service to owners, engineers and contractors,
- The carrying out of preliminary designstudies,
- Assistance with the design of post-tensioned foundations,
- Supply and installation of tendons for post-tensioned foundations,
- Supply of materials and provision of equipement and supervisory personnel,
- The use of other VSL Systems, such as slab post-tensioning, slipforming (for building cores), soil and rock anchors, etc.

The VSL Organizations are in a position to provide these services on advantageous terms; for each case the possibilities and extent of the services will usually need to be clarified in discussions between the owner, the engineer, the contractor and the VSL Organization.

In many cases the application of several VSL Systems is possible on a single project. This enables the use of labour and material to be rationalized with consequent savings in costs.

*)The addresses of VSL Representatives will be found on the back cover of this Report.

At this point, reference may be made to other VSL publications which are of importance in the construction of post-tensioned foundations:
- Brochure «VSL Post-tensioning» [24]
- Brochure « VSL Slab Post-tensioning» [25]
- Technical report «Post-tensioned Slabs» [10]

In addition, the following VSL publications are available that may also be of interest in connection with post-tensioned foundations
- Brochure «VSL Slipforming»
- Brochure «VSL Soil and Rock Anchors»
- Various Job Reports
- VSL News Letters.

5.2. Preparation of a tender

The basic requirements for a tender for one or more of the above services, in so far as they concern the carrying out of detailed design, supply, installation and execution, are detailed drawings and specification documents. This applies both for structures which are about to be constructed and also for Client's proposals which require further technical development, or to which alternative proposals are to be prepared.

A VSL tender for post-tensioned foundations may consist of :
- Supply of material plus manufacture and complete installation of the cables, including provision of personnel and equipment, or
- Supply of material, provision of supervisory personnel and provision of equipment.

The first solution, in most cases, will prove to be the better one and therefore should be selected as a rule. The foremost reason is quality assurance. The durability over the lifetime of a structure indeed mainly depends on quality of materials and on quality of workmanship. The experience available with VSL, whose personnel are engaged exclusively on post-tensioning, is the most suitable for the effective manufacture and installation of tendons. Another reason is economy. A specialist worker can achieve a better rate of progress both by his experience and by the advanced type of equipment he has at his disposal. He will require less time to solve unforeseen problems on site.

In addition, considerable savings are possible if a Main Contractor investigates jointly with VSL how best to use the available material, plant, equipment and methods for a specific project. VSL has, for this purpose, built up its own design engineering staff. The combination of engineering skill with detailed knowledge of the possibilities and special features of the VSL Systems has proved to be an attractive service to Main Contractors for optimizing their construction work.

6. Examples of Application

6.1. Introduction

In this chapter we present to the reader some specific structures with post-tensioned foundations. These examples will illustrate some of the cases where post-tensioned foundations are applicable and at the same time they testify to VSL's worldwide experience in this field.

6.2.Executive Plaza Building, Fairfax, Va., USA

Owner Coakley and Williams, Inc., Greenbelt, Md.
Engineer Harvey Gordon and Associates, Alexandria, Va.
Contractor VECCO Concrete Construction, Springfield, Va.
Post-tensioning VSL Corporation, Springfield, Va.
Year of construction 1976

This office building consists of 11 supported floors and a roof (Fig. 41). The structure was originally designed with spread footings on soil with an allowable bearing capacity of 0.2 N/mm^2 (4,000 psf). A soft irregular substratum, however, was of concern to the engineer and consideration was being given to converting to deep foundations to preclude the possibility of differential settlements. Because of this problem, but primarily on account of the cost, VSL was asked to submit a post-tensioned mat redesign for consideration. This was selected for a number of reasons:

1. A mat foundation uses the total available area beneath a building for bearing. Foundation pressures could therefore be reduced, and the maximum safety against soil failure would be provided.
2. The amount of excavation was significantly reduced. This lowered costs and also provided a thicker layer of firm soil between the bottom of the foundation and the softer formation.
3. If both the mat and the upper floors were post-tensioned, frame moments and shears introduced by long-term creep and shrinkage of the slabs would be minimized.
4. The post-tensioned mat would provide deflection control and increase the probability of uniform bearing pressure beneath the mat. This in turn would reduce the amount of differential settlement.

Figure 41 : Executive Plaza Building

5. A post-tensioned mat would be water resistant and offer added protection for mild steel.
6. The total cost of the foundation, com pared with spread footings, was reduced by $ 39,800 (see table below).

The design of the post-tensioned mat was based on ACI 318-71 Building Code and BOCA Basic Building Code 1975. The thickness of the mat is 457 mm (18"); at each column it is provided with drop heads of additional 406 mm (16"). These serve many useful functions: they reduce the tendon material requirement, increase the mat's punching shear resistance and provide additional bond for the column dowels.
The mat measures 28.80 x47.40m (94'-6"x 155'-6"), giving a plan area of 1,365 m² (14,700 ft²). Column spacings vary between 6.10 and 7.39 m (20' and 24'-3"). All tendons are VSL Monostrands ∅ 13 mm (0.5") provided with stressing anchorages at both ends (Fig. 42). The tendons were distributed in a banded scheme (Fig. 43). Stressing was carried out in three stages at later dates, in order gradually to offset the increasing dead load as the structure went up. It took only two weeks to construct the mat [27].

Figure 42: Positioning of monostrand tendons

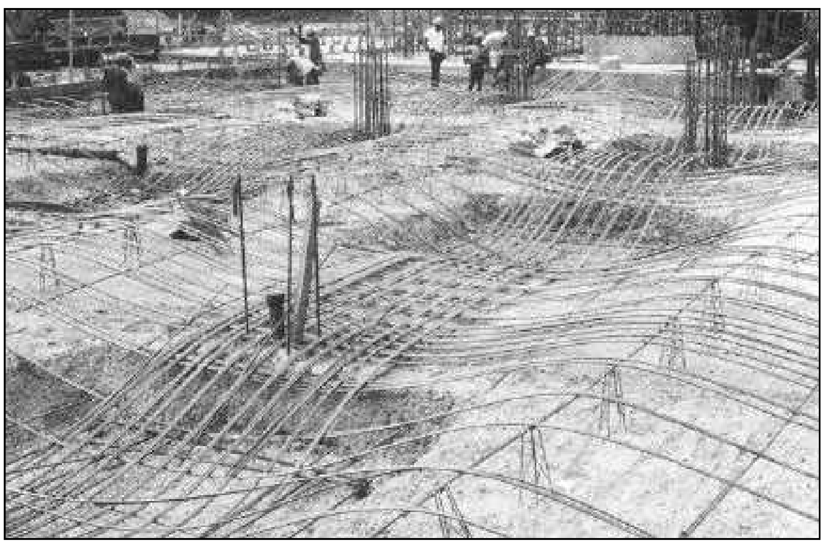


Figure 43: VSL Monostrands of the mat foundation

6.3. Government Office Building, Adelaide, Australia

Owner South Australia Government, Adelaide
Engineer Kinnaird Hill deRohan and Young Pty. Ltd., Adelaide
Contractor Fricker Bros. Pty. Ltd., Adelaide
Post-tensioning
VSL Prestressing (Aust.) Pty. Ltd., Thornleigh
Year of construction
1978

This structure consists of a three-level podium of 49x27 m supporting a seventeenlevel tower of 29.6 x 27 m. It is designed to withstand earthquakes. The building design brief did not include a basement. The use of individual pad foundations beneath each column was therefore discounted because of cost, although it was considered suitable for this building. The founding level required for pads would have meant very deep excavations.
Therefore a raft, be it post-tensioned or reinforced, was chosen to reduce bearing pressure and enable shallower founding levels to be used. The raft solution also enabled settlements to be limited to orders that would not affect the foundation materials of adjacent buildings. It was finally decided to use posttensioning to reduce construction time and costs. A comparison between a reinforced and a post-tensioned raft indicated that the latter would cost between 20 and 30% less in terms of labour and materials. An additional cost benefit became evident from a comparison of construction times; the reinforced concrete raft would probably have taken between 5 and 10 working days longer to construct.
The sub-base consisted of 450 mm of clayfree, graded quarry rubble. Its surface was trimmed and rolled. After final grading and rolling the surface was blinded-off with sand and rerolled before laying of the slip membrane on top. The latter consisted of two layers of Sizalkraft paper, overlayed with 0.2 mm polyethylene sheeting.
The design of the post-tensioned raft utilized the concept of (load balancing». The final design and choice of balanced loads and hence the level of prestress, however, was a complex matter. With an average column grid of 7.20 m and raft plan dimensions of 28 x 32.8 m, a raft thickness of 850 mm was adopted. The design raft bearing pressure was 0.22 N/mm², the sub-base / raft friction coefficient 0.30.
In total, 104 VSL tendons EE and EU 5-7 and

Item	Spread footings (base bid)				Post-tensioned mat			
General excavation	137 yd³	at $ 2 / yd³	=	275	1,005 yd³	at $ 2 / yd³	=	2,000
Footing excavation and backfill	3,112 yd³	at $ 6.50 / yd³	=	20,225				—
Concrete in place	1,917 yd³	at $ 40 / yd³	=	76,700	1,005 yd³	at $ 40 / yd³	=	40,200
Mild steel in place	113 Tons	at $ 400 / Ton	=	45,200	18 Tons	at $ 400 / Ton	=	7,200
Forming				—	825 ft²	at $ 1.40 / ft²	=	1,150
Post-tensioning material in place				—	40,000 lbs	at $ 1.15 / lb	=	46,000
Tendon support chairs				—				6,050
Cost to owner				$ 142,400				$ 102,600
post-tensioned mat savings = $ 39,800								

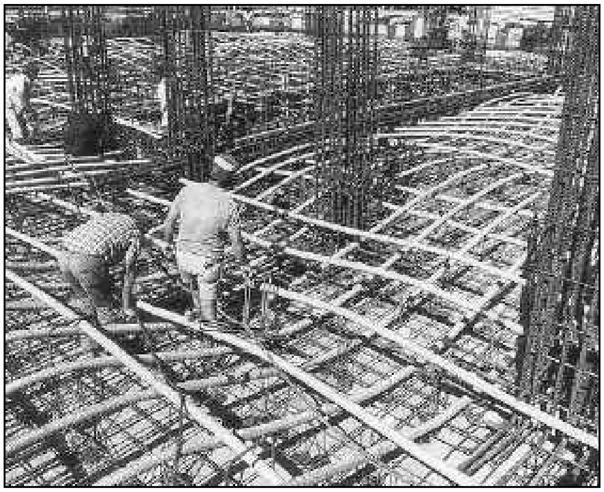


Figure 44: Foundation slab under construction

Figure 45: Concreting of foundation slab

5-12 at an average spacing of 700 mm were required (Fig. 44), of which 64 were stressed at both ends. The stressing of the raft tendons was completed prior to the commencement of any suspended floors. Stressing commenced at the centre of the raft and proceeded on a «Round-Robin» basis, stressing every fourth tendon until completion.

6.4. Raffles City, Singapore

Owner Raffles City (Private) Ltd., Singapore
Engineer Weiskopff & Pickworth, New York, USA
Contractor Ssangyong Construction Co. Ltd., Seoul, Korea
Post-tensioning VSL Systems Pte. Ltd., Singapore
Years of construction 1981-1982

Raffles City is the name of a building complex in the Beach Road area of Singapore. It comprises a 71-storey hotel, a 43-storey office tower block, a twin core 29-storey hotel and a podium building containing conference facilities and a shopping centre (Fig. 46).

To avoid the need for a foundation on piles more than 40 m long, which would have lengthened construction time by nine months, it was decided to construct a posttensioned slab foundation for the 71-storey hotel. Construction of the slab commenced in November 1981 and was completed in September 1982.

The foundation slab measures 35 x 40 m and is 5 m in total thickness. The upper 3.40 m were considered to be the compression element and therefore were only reinforced, whereas the bottom 1.60 m, being the tensile element, were orthogonally post-tensioned in three layers (Fig. 47). VSL tendons EP 5-31 (ultimate strength 5,700 kN), horizontally spaced at 440 or 450 mm, were used throughout. A total of 500 tonnes of posttensioning steel, 70 tonnes of supporting reinforcement, 60 tonnes of bursting reinforcement and 200 tonnes of cement for grouting were required.

VSL bid for the post-tensioning as a package which included

-design and supply of post-tensioning tendons,
-design of end block reinforcement,
-design and supply of tendon support chairs,
- installation of tendons,
- stressing and grouting of tendons.

Post-tensioning work started on June 28, 1982 and was completed in the record time of 85 days. This very tight target was achieved by having an average of 40 VSL employees on the site every day. The tendons were installed using the VSL Push Through Method (Fig. 48). After pushing, the compression fittings were mounted on the strands. Four automatic jacks of VSL type ZPE-460 were flown in specially for stressing (Fig. 49).

Figure 46: Raffles City buildings

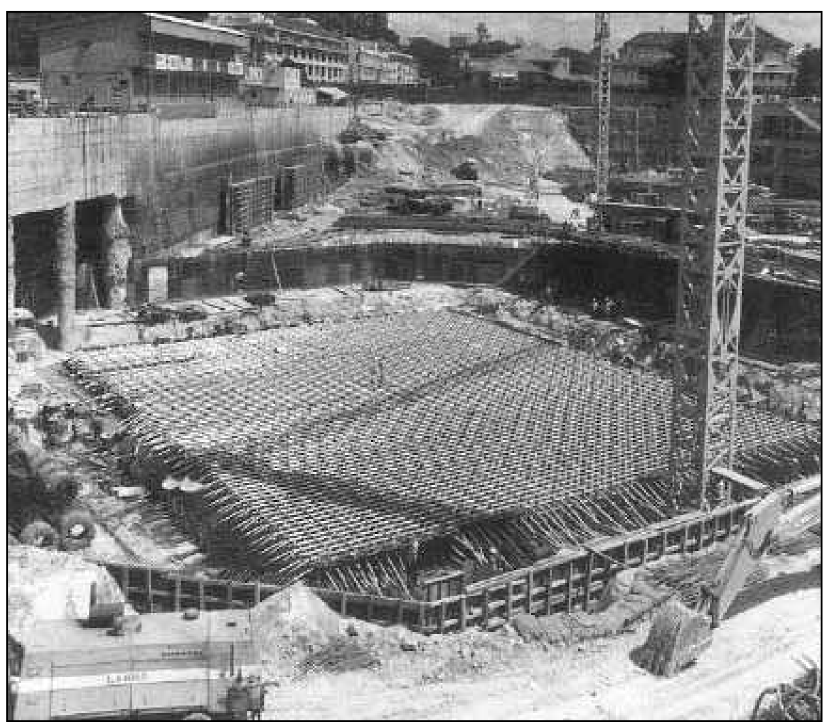


Figure 47: Foundation slab of 71-storey hotel during construction

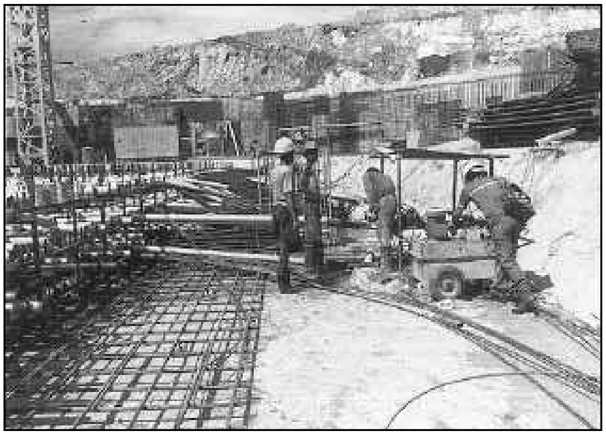


Figure 48: Pushing strand by strand into cable duct

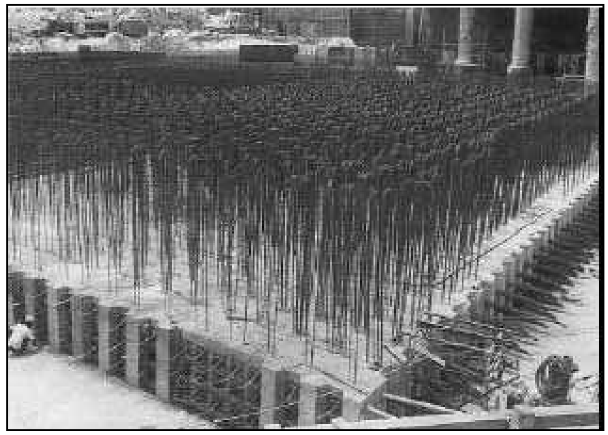


Figure 49: Lower part of slab concreted and partly stressed

6.5. Army Dispensary, Ittigen, Switzerland

Owner Federal Office for Public Works, Buildings Department, Berne
Engineer ITEC Ingenieurteam, Berne
Contractor Kastli Bau AG, Ostermundigen
Post-tensioning VSL INTERNATIONAL AG, Lyssach
Years of construction 1986-1987

This structure is an extension to the Army Dispensary at Ittigen, near Berne. The building was originally designed in reinforced concrete, but it was then decided to use posttensioned concrete in order to meet the exacting serviceability requirements. Post-tensioning enabled joints and sealing to be dispensed with and construction to be speeded up.

The building has a length of 80 m and a maxi mum width of 35 m. In the longitudinal direction, the columns are spaced at 7.50 m (except the end spans), while transversely the spacings include 5.00, 7.50 and 10.00 m. The foundation slab is up to 4 m below ground water level. Therefore exacting requirements regarding tightness and crack limitation were imposed.

The foundation slab has a thickness of 0.80 m. It was constructed in five stages (between November 1986 and April 1987); the last section (air-raid shelter) was not posttensioned (Fig. 50). In the other areas posttensioning was provided by VSL tendons 6-6 and 6-7 (breaking force 1,548 and 1,806 kN respectively). Anchorages of types EC, H, U, K and Z were used (Fig. 51). Where possible cables were installed after concreting by means of the VSL Push-Through Technique; the other tendons were prefabricated in the workshop and threaded into the previously placed ducts before concrete was poured (Fig. 52). The tendons run in both directions, approximately uniformly distributed, but with a certain concentration in the column strips. Minimum ordinary reinforcement of the order of 0.10 to 0.13 % was placed in each direction in lower and upper layers.

Half the tendons were stressed to full force as early as possible. The other half were stressed only after construction of three of the five upper slabs. At that time some of the end anchorages were no longer accessible and therefore Z anchorages were used as intermediate stressing anchorages for these tendons.

Figure 52: Installation of a tendon

6.6. Commercial Building, Hohlstrasse, Zurich, Switzerland

Owner Linco AG, Zurich
Engineer Jager AG, Zurich
Contractor Losag AG, General Contractors, Zurich
Post-tensioning VSL INTERNATIONALAG, Lyssach
Year of construction 1987

This building is situated in an industrial zone adjacent to freight railway lines. It consists of seven floors above ground and two floors below. In plan it measures 65.5x23 m. All floors and also the base slab are post-tensioned.

The base slab is located in the ground water. Therefore tightness of the substructure, i.e. base slab and surrounding walls, was required. For this reason both slab and walls were post-tensioned (Fig. 53). Longitudinally the

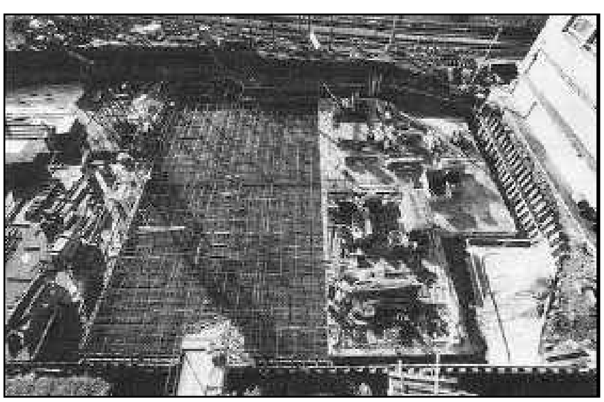


Figure 50: Construction (in stages) of foundation slab

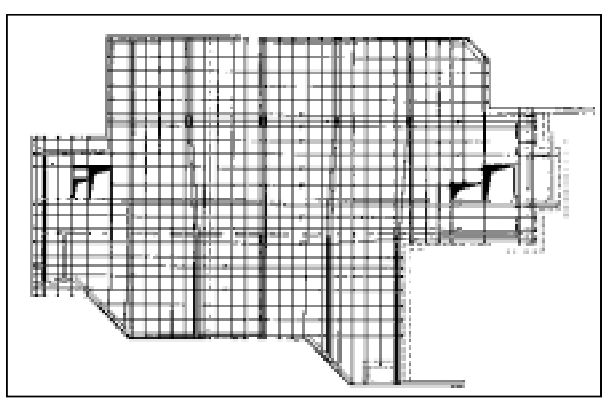


Figure 51 : Tendon scheme (construction stages 1-3)

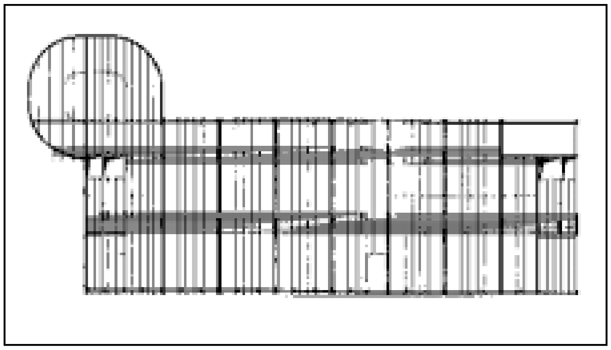


Figure 53 : Layout of base slab tendons

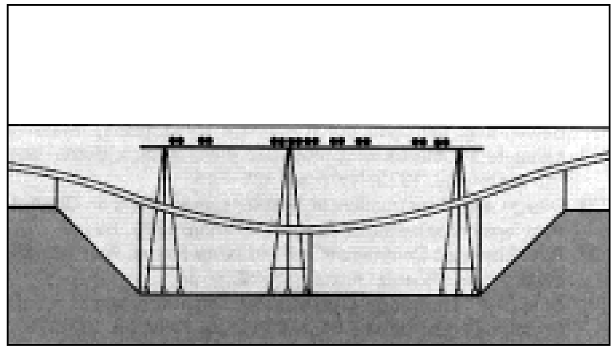


Figure 54 : Cross-section of smaller beam with tendons

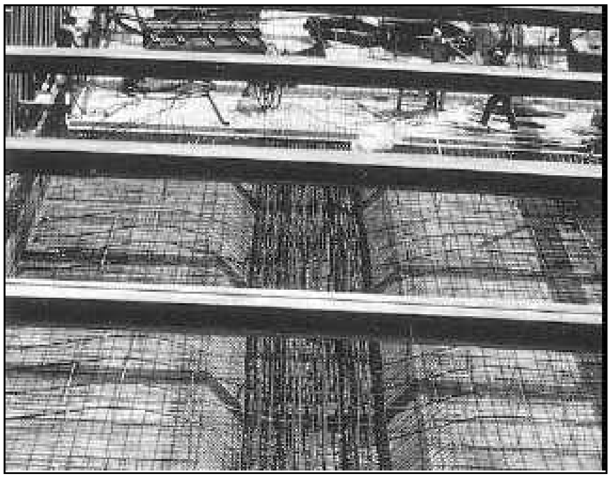


Figure 55: Base slab under construction

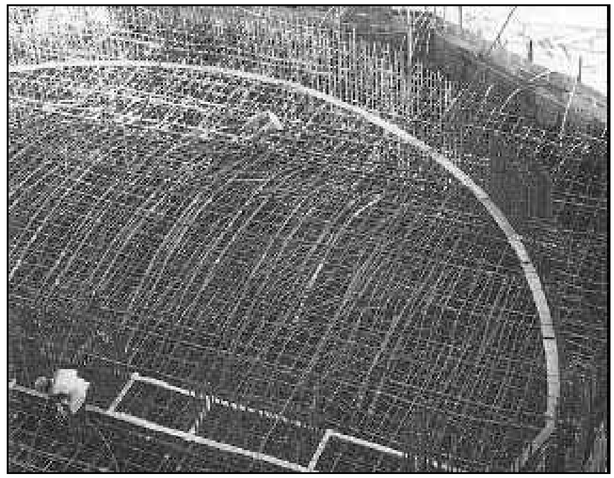


figure 56 : Partial view of base slab prior to concreting

base slab carries columns at 7.50 m spacings, the transverse spans being 9.00-8.754.75 m. The slab thickness generally is 0.45 m; in the longitudinal column axes it is thickened to 1.00 m so as to form beams 2.00 and 3.00 m wide respectively (Fig. 54). All tendons are VSL Monostrands ∅ 15 mm (0.6"). As the base slab was constructed in two stages, all the longitudinal tendons were provided with intermediate stressing anchorages (Fig. 55).

6.7. Clinker Silos, Pedro Leopoldo, Brazil

Owner CIMINAS (Cimento Nacional de Minas S.A.), Sao Paulo
Engineer H. Trachsel & H.J. Schibli AG, Olten, Switzerland
Contractor Joint Venture M. Roscoe / Moura, Schwark Ltda., Belo Horizonte
Post-tensioning Sistemas VSL Engenharia S.A., Rio de Janeiro
Years of construction 1973-1974

These two silos have an outside diameter of 26.64 m and a height of 42.00 m above the foundation slab. Each base slab is 1.70 m thick and rests on 232 piles. Each one is posttensioned with 144 VSL tendons EU 5-12, which are arranged in two layers. The first layer is located 400 mm above the lower face of the foundation slab, the second layer 400 mm below the upper face. In order to lead the tendon axes radially from the edge of the slab, the tendons are curved at their ends. In the central region of the slab, the tendons in each layer cross one another. Tendon lengths vary from 25.50 to 31.50 m (Figures 56 and 57).

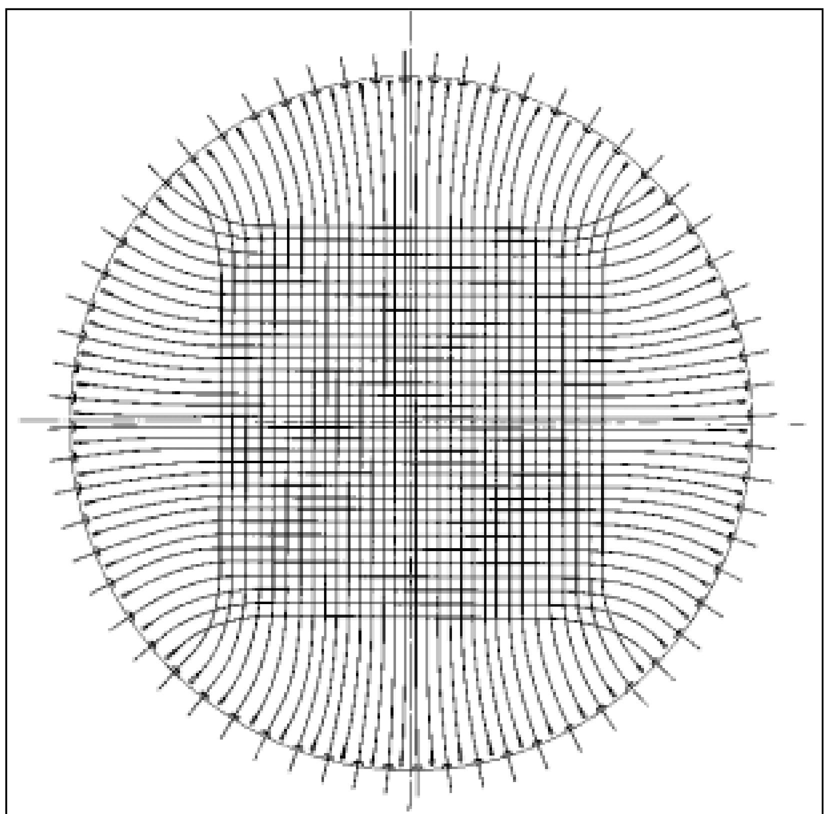


Figure 57: Tendon layout

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