POST TENSIONING IN BUILDING STRUCTURES

A PROJECT REPORT SUBMITTED TO JNTU, HYD

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BACHELOR OF TECHONOLOGY

IN

CIVIL ENGINEERING

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CERTIFICATE

This is to certify that the project report entitled "**POST TENSIONING IN BUILDING STRUCTURES**" being submitted by **B.DIXIT RAJ (08241A0114) , M.RUSHI VARMA (08241A0134) , K.VIJAYASENA REDDY (08241A0158) , G.VISHRUTH REDDY (08241A0159) ,** in partial fulfillment for the award of degree of **Bachelor of Technology** in **CIVIL ENGINEERING, GOKARAJU RANGARAJU INSTITUTE OF ENGINEERING AND TECHNOLOGY** (Affiliated to Jawaharlal Nehru Technology University, Hyderabad) is a bonafide report of the work carried out by them under my guidance and supervision from February 16th to April 10th 2012 and the project has been successfully completed. This project work has not been submitted to any other university or institute for any award or degree.

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Any errors and omissions are unintentional and are sincerely regretted.

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DECLARATION

We hereby declare that the project work "**POST TENSIONING IN BUILDING STRUCTURES**" submitted towards partial fulfillment of requirements for the Degree of Bachelor of Technology in Civil Engineering to Gokaraju Rangaraju Institute of Engineering and Technology, Hyderabad, is an authenticate work and had not been submitted to any other university or institute for any award of degree or diploma.

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ABSTRACT

The development of prestressing technology has one of the more important improvements in the fields of structural engineering and construction. Referring particularly to post tensioning applications, it is generally recognized how it opens the possibility to improve economy, structural behavior and aesthetic aspects in concrete solutions. As in modern days post tensioning has been most economical method when compared to the RCC works. The project discusses the activity of using post tensioning in slabs and beams in buildings. This method is quite used in the construction of shopping malls, theatres and multistory structures.

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

Post-tensioned slabs are a preferred method for industrial, commercial and residential floor slab construction**.** The increasingly extensive use of this method is due to its advantages and its nature of easy application to a wide variety of structure geometry and design solutions. The use of post-tensioned floor slabs and reinforced concrete core walls has become increasingly popular in high-rise construction.

Economics of the post-tensioning slab system are discussed including relative material contents, speed of construction, and factors affecting the cost of post-tensioning. Finally, a discussion on the flexibility of post-tensioned building structures in terms of future uses, new floor penetrations and demolition is presented.

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Introduction

When Eugene Freyssinet developed and patented the technique of prestressing concrete in 1928 he little realized the applications to which his invention would be put in future years. Spectacular growth in the use of prestressed concrete took place after the Second World War with the material used to repair and reconstruct bridges in Europe. It is now an accepted Civil Engineering construction material.

The A.C.I. Committee on Prestressed Concrete gives one of the most apt descriptions of post tensioned concrete.

`Prestressed Concrete is concrete in which there have been introduced internal forces of such magnitude and distribution that the forces resulting from given external loadings are counteracted to a desirable degree'.

In post-tensioning we obtain several distinct advantages: -

a) Designers have the opportunity to impart forces internally to the concrete structure to counteract and balance loads sustained by the structure thereby enabling design optimization.

b) Designers can utilize the advantage of the compressive strength of concrete while circumventing its inherent weakness in tension.

c) Post-tensioned concrete combines and optimizes today's very high strength concretes and steel to result in a practical and efficient structural system.

The first post-tensioned buildings were erected in the USA in the 1950's using unbonded post tensioning. Some post-tensioned structures were built in Europe quite early on but the real development took place in Australia and the USA. Joint efforts by prestressing companies, researchers and design engineers in these early stages resulted in standards and recommendations which assisted in promoting the widespread use of this form of construction in Australia, the USA and throughout the Asian region.

Extensive research in these countries, as well as in Europe more recently, has greatly expanded the knowledge available on such structures and now forms the basis for standards and codes of practice in these countries.

Since the introduction of post-tensioning to buildings, a great deal of experience has been gained as to which type of building has floors most suited to this method of construction. Many Engineers and Builders can identify at a glance whether the advantages of post-tensioning can be utilized in any particular situation.

Current architecture in India continues to place emphasis on the necessity of providing large uninterrupted floor space, flexibility of internal layout, versatility of use and freedom of movement. All of these are facilitated by the use of post-tensioning in the construction of concrete floor slabs, giving large clear spans, fewer columns and supports, and reduced floor thickness.

Post-tensioning in buildings can be loosely divided into two categories. The first application is for specialized structural elements such as raft foundations, transfer plates, transfer beams, tie beams and the like. For large multi-strand tendons used in these elements, 15.2 mm diameter seven wire strands are preferred.

1. ADVANTAGES OF POST-TENSIONED FLOORS

The primary advantages of post-tensioned floors over conventional reinforced concrete in-situ floors may be summarized as follows:

a. Longer Spans

Longer spans can be used reducing the number of columns. This results in larger, column free floor areas which greatly increase the flexibility of use for the structure and can result in higher rental returns.

b. Overall Structural Cost

The total cost of materials, labour and formwork required to construct a floor is reduced for spans greater than 7 meters, thereby providing superior economy.

c. Reduced Floor to Floor Height

For the same imposed load, thinner slabs can be used. The reduced section depths allow minimum building height with resultant savings in facade costs. Alternatively, for taller buildings it can allow more floors to be constructed within the original building envelope.

d. Deflection Free Slabs

Undesirable deflections under service loads can be virtually eliminated.

e. Waterproof Slabs

Post-tensioned slabs can be designed to be crack free and therefore waterproof slabs are possible. Achievement of this objective depends upon careful design, detailing and construction. The choice of concrete mix and curing methods along with quality workmanship also plays a key role.

f. Early Formwork Stripping

The earlier stripping of formwork and reduced back propping requirements enable faster construction cycles and quick re-use of formwork. This increase in speed of construction is explained further in the next section on economics.

g. Materials Handling

The reduced material quantities in concrete and reinforcement greatly benefit on-site carnage requirements. The strength of post-tensioning strand is approximately 4 times that of conventional reinforcement. Therefore the total weight of reinforcing material is greatly reduced.

h. Column and Footing Design

The reduced floor dead loads may be utilized in more economical design of the reinforced concrete columns and footings. In multi-storey buildings, reduced column sizes may increase the floor net let table area.

These advantages can result in significant savings in overall costs. There are also some situations where the height of the building is limited, in which the reduced storey height has allowed additional storey's to be constructed within the building envelope.

2. BONDED OR UNBONDED TENDON SYSTEMS

Post-tensioned floors can be constructed using either bonded or unbonded tendons. The relative merits of the two techniques are subject to debate. The following points may be made in favour of each.

a) Bonded system

For a bonded system the post-tensioned strands are installed in galvanized steel or plastic ducts that are cast into the concrete section at the required profile and form a voided path through which the strands can be installed.

The ducts can be either circular- or oval-shaped and can vary in size to accommodate a varying number of steel strands within each duct. At the ends a combined anchorage casting is provided which anchors all of the strands within the duct. The anchorage transfers the force from the stressing jack into the concrete. Once the strands have been stressed the void around the strands is filled with a cementitious grout, which fully bonds the strands to the concrete. The duct and the strands contained within are collectively called a tendon.

Figure1. Bonded systems.

The main features of a bonded system are summarized below.

1. There is less reliance on the anchorages once the duct has been grouted.
2. The full strength of the strand can be utilized at the ultimate limit state (due to strain compatibility with the concrete) and hence there is generally a lower requirement for the use of unstressed reinforcement.
3. The prestressing tendons can contribute to the concrete shear capacity.
4. Due to the concentrated arrangement of the strands within the ducts a high force can be applied to a small concrete section.
5. Accidental damage to a tendon results in a local loss of the prestress force only and does not affect the full length of the tendon.

b) Unbonded system

In an unbonded system the individual steel strands are encapsulated in a polyurethane sheath and the voids between the sheath and the strand are filled with rust-inhibiting grease.

The sheath and grease are applied under factory conditions and the completed tendon is electronically tested to ensure that the process has been carried out successfully. The individual tendons are anchored at each end with anchorage castings.

The tendons are cast into the concrete section and are jacked to apply the required prestress force once the concrete has achieved the required strength.

Figure 2. Unbonded Tendon systems.

The main features of an unbounded system are summarized below.

1. The tendon can be prefabricated off site.
2. The installation process on site can be quicker due to prefabrication and the reduced site operations.
3. The smaller tendon diameter and reduced cover requirements allow the eccentricity from the neutral axis to be increased thus resulting in a lower force requirement.
4. The tendons are flexible and can be curved easily in the horizontal direction to accommodate curved buildings or divert around openings in the slab.
5. The force loss due to friction is lower than for bonded tendons due to the action of the grease.
6. The force in an unbonded tendon does not increase significantly above that of the prestressing load.
7. The ultimate flexural capacity of sections with unbounded tendons is less than that with bonded tendons but much greater deflections will take place before yielding of the steel.
8. Tendons can be replaced (usually with a smaller diameter).
9. **A** broken tendon cause's prestress to be lost for the full length of that tendon.
10. Careful attention is required in design to ensure against progressive collapse.

Why a bonded system?

This is another question that arises. Why do we use bonded tendons? Well there are a number of advantages; higher flexural capacity, good flexural crack distribution, good corrosion protection and flexibility for later cutting of penetrations and easier demolition.

However there are some disadvantages such as an additional operation for grouting and a more labour intensive installation.

However, the main reason why bonded tendons are preferred relates to the overall cost of the structure and not just of the post-tensioning. With unbonded tendons it is usual to have a layer of conventional reinforcement for crack control.

Using bonded tendons there is no such requirement and therefore the overall price of bonded post-tensioning and associated reinforcement is less than for bonded tendons. For unbonded tendons the post-tensioning price may be less, but the overall cost of reinforcing materials is greater.

3. MATERIALS

Post-tensioned floors use all the materials required in a reinforced concrete floor- formwork, rod reinforcement and concrete- and, additionally, they use high tensile steel strand and the hardware specific to post-tensioning.

As a material, rod reinforcement in post-tensioned floors is exactly same as that in reinforced concrete in every respect. The normal high tensile steel, as used in rod reinforcement, has a yield stress of 460 N/mm^2 and a modulus of elasticity of 200 KN/mm^2. It has a poisons ratio of 0.3 and a coefficient of thermal expansion of 12.5×10^{-6} per degree centigrade. The strength of high tensioned steel is affected by rise in temperature, dropping from 100% at 300 degree centigrade to only 5% at 800 degree centigrade.

The technology for the production, compaction and curing of concrete is well understood and is not discussed here. Only the properties of concrete which are important for post-tensioning are considered.

Normal dense concrete, 2400 kg/m^3 density, is more common in post-tensioning. Light weight concrete, however, has certain advantages in the right circumstances. Both are dealt with in separate sections.

The properties of two concretes are quite different and it is not a good practice to use the two side by side; there may be problems from differential movement and the difference in their module of elasticity, shrinkage and creep.

a) FORMWORK

In a post-tensioned floor, the vertical edge boards of the form work need to have holes drilled through for the tendons to pass at live anchorages. During stressing, the concrete undergoes a slight reduction in length due to axial component of the prestress. Though trapping of formwork between any downstands is not a serious problem, the design of formwork should recognize the possibility. During stressing, the post-tensioned slab lifts off the formwork, so that there is a re-distribution of its weight. This may impose heavier loads on parts of the formwork than those due to the weight of wet concrete. In other respects, the formwork for a post-tensioned floor is similar to that for a reinforced concrete floor.

Each live anchorage is set in a recess, or anchorage pocket, in the slab edge. The pocket is formed using proprietary plastic formers supplied by the specialist prestressing hardware supplier. The formers are removed when the formwork is stripped. Expanded polystyrene blocks have sometimes used for this purpose but, in the authors experience they are often difficult to remove afterwards and pieces of polystyrene are left in the anchorage pocket. Attempts to remove these by burning causes heating of the anchorage and the strand at a critical point and use of chemicals may have a detrimental effect on concrete or steel.

Figure3. Formwork.

The pockets are normally tapering in shape to allow removal of the former, and are larger than the size of the anchorage in elevation to allow sufficient room for the jack to be coupled. The actual size depends on the anchorage dimensions and the clearance required for the jack; both of these vary between the various manufacturers. The pocket depth is sufficient to accommodate the anchorage, the projecting strand (about 30mm, 1.25inch) and the grease cap, and to provide adequate cover to the assembly.

Anchorage castings are temporarily attached to the vertical edge board. A single strand casting may be attached by long nails passing through holes at the corners of the anchorage casting, or by proprietary means. The nails get cast in the concrete and cannot be removed when the edge board is stripped. Anchorages for multistrand tendons are heavier and are supported by bolts which pass through sleeves of the same length as the pocket. The sleeve itself gets concreted and cannot be removed but the bolt is removed when the edge shutter is stripped.

A reinforced concrete floor does not undergo any significant longitudinal shortening during the first few days of casting; shrinkage does not occur while the concrete is wet during curing. In contrast, a post tensioned floor does shorten in length during stressing. The strain depends on the average prestress level.

For a slab with an average stress of 2 N/mm^2, the strain may be about 0.0001, i.e. a ten meter bay may shorten by 1 mm. A ten meter length of beam stressed to an average 6.0 N/mm^2 would shorten by 3 mm (1/8 inch). These strains are not large, but they may just trap the formwork between two vertical faces of a downstand. Removal of such a trapped soffit shutter may need some force, which can damage the arises of the downstands.

The difficultly can be avoided by a filler strip in the formwork, which can be removed before stressing, or by incorporating a strip of compressible material. In ribbed and waffle floors, removal of the forms is easier if the sides of the ribs are given a generous slope; say not less than 10 degrees.

A post-tensioned floor lifts of its formwork when prestress is applied-usually 3 or 4 days after casting. Soffit forms, therefore, become redundant at this stage and normally removed; some props are retained for construction loads. Early removal of formwork allows a faster turn-around, and it is normally possible to use fewer sets than would be required in a similar project in traditional reinforced concrete. Each formwork set would, therefore, be used many more times and it should be designed accordingly.

b) CEMENT

The cement used shall be any of the following, with the prior approval of the engineer-in-charge:

- Ordinary Portland cement conforming to IS : 269-1976*;
- Portland slag cement conforming to IS : 455-1976†, but with no more than 50 percent slag content;
- Rapid-hardening Portland cement conforming to IS : 8041-1978‡; and
- High strength ordinary Portland cement conforming to IS: 8112-1976§.

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*Specification for ordinary and low heat Portland cement (third revision).
†Specification for Portland slag cement (third revision).
‡Specification for rapid hardening Portland cement (*first revision*).
§Specification for high strength ordinary Portland cement.

c) AGGREGATES

All aggregates shall comply with the requirements of IS: 383-1970||.

The nominal maximum size of coarse aggregate shall be as large as possible subject to the following:

- In no case greater than one-fourth the minimum thickness of the member, provided that the concrete can be placed without difficulty so as to surround all prestressing tendons and reinforcements and fill the corners of the form.
- It shall be 5 mm less than the spacing between the cables, strands or sheathings where provided.
- Not more than 40 mm; aggregates having a maximum nominal size of 20 mm or smaller are generally considered satisfactory.

Coarse and fine aggregates shall be batched separately.
||Specification for coarse and fine aggregates from natural sources for concrete (*second revision*).

d) WATER

The requirements of water used for mixing and curing shall conform to the requirements given in IS: 456-1978*. However use of sea water is prohibited.

e) ADMIXTURES

Admixtures may be used with the approval of the engineer-in-charge. However use of any admixture containing chlorides in any form is prohibited.

The admixtures shall conform to IS: 9103-1979†.

f) PRESTRESSING STEEL

The prestressing steel shall be any one of the following:

- Plain hard-drawn steel wire conforming to IS: 1785 (Part I)-1966‡ and IS: 1785 (Part II)-1967§,
- Cold-drawn indented wire conforming to IS: 6003-1970||,
- High tensile steel bar conforming to IS: 2090-1962¶, and

- Uncoated stress relieved strand conforming to IS: 6006-1970**.

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*Code of practice for plain and reinforced concrete (*third revision*).
†Specification for admixtures for concrete.
‡Specification for plain hard-drawn steel wire for prestressed concrete: Part I Cold-drawn stress-relieved wire (*revised*).
§Specification for plain hard-drawn steel wire for prestressed concrete: Part II as-drawn wire.
||Specification for indented wire for prestressed concrete.
¶Specification for high tensile steel bars used in prestressed concrete.
**Specification for uncoated stress relieved strand for prestressed concrete.

All prestressing steel shall be free from splits, harmful scratches, surface flaws; rough, jagged and imperfect edges and other defects likely to impair its use in prestressed concrete. Slight rust maybe permitted provided there is no surface pitting visible to the naked eye.

Coupling units and other similar fixtures used in conjunction with the wires or bars shall have an ultimate tensile strength of not less than the individual strengths of the wires or bars being joined.

Modulus of Elasticity — the value of the modulus of elasticity of steel used for the design of prestressed concrete members shall preferably be determined by tests on samples of steel to be used for the construction. For the purposes of this clause, a value given by the manufacturer of the prestressing steel shall be considered as fulfilling the necessary requirements.

Where it is not possible to ascertain the modulus of elasticity by test or from the manufacturer of the steel, the following values may be adopted:

Type of steel	Modulus of elasticity, E, Kn/mm^2
Plain cold-drawn wires [conforming to IS: 1785 (Part I)-1966*, IS: 1785 (Part II)-1967† and IS: 6003-1970‡]	210
High tensile steel bars rolled or Heat-treated (conforming to IS: 2090-1962§)	200
Strands (conforming to IS: 6006-1970||)	195

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*Specification for plain hard-drawn steel wire for prestressed concrete: Part I Cold drawn stress-relieved wire (*revised*).
†Specification for plain hard drawn steel wire for prestressed concrete: Part II Asdrawn wire.

‡Specification for indented wire for prestressed concrete.
§Specification for high tensile steel bars used in prestressed concrete.
||Specification for uncoated stress relieved strand for prestressed concrete.

g) UNTENSIONED STEEL

Reinforcement used as untensioned steel shall be any of the following:

- Mild steel and medium tensile steel bars conforming to IS : 432 (Part I)-1966¶,
- Hot-rolled deformed bars conforming to IS : 1139-1966**,
- Cold-twisted bars conforming to IS : 1786-1979††, and
- Hard-drawn steel wire fabric conforming to IS: 1566-1967‡‡.

h) STORAGE OF MATERIALS

Storage of materials shall be as per IS: 4082-1978§§.

i) STRANDS

Strand, commonly in use in post-tensioned floors, is made from seven cold drawn high carbon steel wires. Six of the wires are spun together in a helical form around a slightly larger seventh straight centre wire. The strand is then given either a stress relieving treatment or it is run through a controlled tension and low temperature heat treatment process which gives it the low relaxation property.

For a more compact type of product, the seven wire strand is drawn through a die under controlled conditions of tension and temperature. In the process the individual wires are cold worked and compacted into a characteristic shape of the die-drawn strand. Compact strand, compared with normal strand, has a higher steel area for a given overall diameter and, therefore, it has a higher force to diameter ratio.

Drawing a strand through a die increases its strength but reduces its ductility the product is brittle and liable to failure without much warning. In order to avoid brittle and sudden failure the strand is annealed to increase its ductility.

Strand for post-tensioning is made of high tensile strength steel wire. A strand is comprised of 7 individual wires, with six wires helically wound to a long pitch around a center wire. All strand should be Grade 1860 MPa (270 ksi) low relaxation, seven-wire strand conforming to the requirements of ASTM A 416 "Standard Specification for Steel Strand, Uncoated Seven Wire Strand for Prestressed Concrete".

ASTM A 416 provides minimum requirements for mechanical properties (yield, breaking strength, elongation) and maximum allowable dimensional tolerances. Strand from different sources may meet ASTM A 416 but is not necessarily identical in all respects.

¶Specification for mild steel and medium tensile steel bars and hard drawn steel wire for concrete reinforcement: Part I Mild steel and medium tensile steel bars (*second revision*).
**Specification for hot rolled mild steel, medium tensile steel and high yield strength steel deformed bars for concrete reinforcement (*revised*).
††Specification for cold-worked steel high strength deformed bars for concrete reinforcement (*second revision*).
‡‡Specification for hard-drawn steel wire fabric for concrete reinforcement (*first revision*).
§§Recommendations on stacking and storage of construction materials at site.

Figure 4. Strand

Strand is mostly available in two nominal sizes, 12.7mm (0.5in) and 15.7mm (0.6in) diameter, with nominal cross sectional areas of 99mm ^{2} and 140mm ^{2} (0.153 and 0.217 square inches), respectively.

The majority of post-tensioning hardware and stressing equipment is based on these sizes. Strand size tolerances may result in strands being manufactured consistently smaller than or larger than nominal values. Recognizing this, industry ("Acceptance Standards for Post-Tensioning Systems", Post-Tensioning Institute, 1998 refers to the "Minimum Ultimate Tensile Strength" (MUTS) which is the minimum specified breaking force for a strand. Strand size tolerance may also affect strand-wedge action leading to possible wedge slip if the wedges and strands are at opposite ends of the size tolerance range.

Why use 12.7mm diameter strands?

A question that arises from time to time is why we use 12.7mm diameter strands for building works, when on face value 15.2mm diameter strands appears more cost effective. The first answer is that 12.7mm has a high strength per unit weight when compared to 15.2mm, which leads to a reduced cost. Secondly, and more importantly from an installation viewpoint, it allows greater flexibility in choosing the tendon we want to use. This is mainly due to the Recommended maximum tendon spacing being limited to 8 to 10 times the slab thickness. The Addition of a single 12.7mm strand in a tendon leads to a relatively small increase in overall tonnage and therefore cost, and allows for better customization of the design.

Of course there are times when 15.2mm strand should be used. This occurs when the tendon already contains the full 5 strands in a duct and the tendon spacing is not at the maximum allowed. In our experience this occurs in less than 10% of structures. If this is the case, we should substitute 15.2mm diameter strands and increase the tendon spacing. This leads to a reduction in the number of whole tendons and a subsequent reduction in anchorage costs and labour costs since less whole tendons have to be installed. As noted earlier, 15.2mm diameter should also be used for specialized structural elements and large civil engineering applications, where the aim is to use as few whole tendons as possible.

j) CONCRETE

The concrete shall be in grades designated as per Table 1.
The characteristic strength of concrete is defined as the strength of the concrete below which not more than 5 percent of the test results are expected to fall.

TABLE 1 GRADES OF CONCRETE

GRADE DESIGNATION	SPECIFIED CHARACTERISTIC COMPRESSIVE STRENGTH AT 28 DAYS N/mm2
M 30	30
M 35	35
M 40	40
M 45	45
M 50	50
M 55	55
M 60	60

NOTE 1 — In the designation of a concrete mix, letter M refers to the mix and the number to the specified characteristic compressive strength of 15-cm cube at 28 days, expressed in N/mm2.
NOTE 2 — For pre-tensioned prestressed concrete, the grade of concrete shall be not less than M 40.

PROPERTIES OF CONCRETE

Increase in Strength with Age — where it can be shown that a member will not receive its full design stress within a period of 28 days after the casting of the member (for example, in foundations and lower columns in multi-storey buildings); the characteristic compressive strength given in Table 1 may be increased by multiplying by the factors given below:

Minimum Age of Member When Full Design Stress is expected (Months)	Age factor
1	1.0
3	1.10
6	1.15
12	1.20

NOTE 1 — Where members are subjected to lower direct load during construction, they should be checked for stresses resulting from combination of direct load and bending during construction.

NOTE 2 — The design strength shall be based on the increased value of compressive strength.

Tensile Strength of Concrete — The flexural strength shall be obtained as per IS : 516-1959*. When the designer wishes to use an estimate of the flexural strength from the compressive strength, the following formula may be used:

$$f_{cr} = 0.7\sqrt{f_{ck}} \text{ N/mm}^2$$

where

f_{cr} = flexural strength in N/mm2, and

f_{ck} = characteristic compressive strength of concrete in N/mm2.

Elastic Deformation — The modulus of elasticity is primarily influenced by the elastic properties of the aggregate and to a lesser extent by the conditions of curing and age of the concrete, the mix proportions and the type of cement. The modulus of elasticity is normally related to the compressive strength of concrete.

In the absence of test data, the modulus of elasticity for structural concrete may be assumed as follows:

$$E_c = 5700\sqrt{f_{ck}}$$

where

E_c = short-term static modulus of elasticity in N/mm2, and

f_{ck} = characteristic compressive strength of concrete in N/mm2.

Shrinkage — The shrinkage of concrete depends upon the constituents of concrete, size of the member and environmental conditions. For a given environment, the shrinkage of concrete is most influenced by the total amount of water present in the concrete at the time of mixing and, to a lesser extent, by the cement content.

In the absence of test data, the approximate value of shrinkage strain for design shall be assumed as follows:

For pre-tensioning = 0.0003

For post-tensioning = $\frac{0.0002}{\log_{10}(t+2)}$

where
t = age of concrete at transfer in days.
NOTE — The value of shrinkage strain for design of post-tensioned concrete may be increased by 50 percent in dry atmospheric conditions, subject to a maximum value of 0.0003.

For the calculation of deformation of concrete at some stage before the maximum shrinkage is reached, it may be assumed that

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*Methods of test for strength of concrete.
half of the shrinkage takes place during the first month and that about three-quarters of the shrinkage takes place in first six months after commencement of drying.

Creep of Concrete — Creep of concrete depends, in addition to the factors listed above on the stress in the concrete, age at loading and the duration of loading. As long as the stress in concrete does not exceed one-third of characteristic compressive strength, creep may be assumed to be proportional to the stress.

In the absence of experimental data and detailed information on the effect of the variables, the ultimate creep strain may be estimated from the following values of creep coefficient (that is, ultimate creep strain/elastic strain at the age of loading):

Age at Loading	Creep Coefficient
7 days	2.2
28days	1.6
1 year	1.1

NOTE — the ultimate creep strain estimated as per above table does not include the elastic strain.
For the calculation of deformation at some stage before the total creep is reached, it may be assumed that about half the total creep takes place in the first month after loading and that about three-quarters of the total creep takes place in the first six months after loading.

Thermal Expansion — the coefficient of thermal expansion depends on nature of cement, the aggregate, the cement content, the relative humidity and the size of sections. For values of coefficient of thermal expansion for concrete with different aggregates, IS: 456-1978* may be referred to.

k) WORKABILITY OF CONCRETE

The concrete mix proportions chosen should be such that the concrete is of adequate workability for the placing conditions of the concrete and can properly be compacted with the means available. Suggested ranges of values of workability of concrete are given in IS: 456-1978*.

l) DURABILITY OF CONCRETE

The durability of concrete depends on its resistance to deterioration and the environment in which it is placed. The resistance of concrete to weathering, chemical attack, abrasion, frost and fire depends largely upon its quality and constituent materials. The strength alone is not a reliable guide to the quality and durability of concrete; it must also have adequate cement content and a low water-cement ratio.

*Code of practice for plain and reinforced concrete (*third revision*).

One of the main characteristics influencing the durability of concrete is its permeability. With strong, dense aggregates, a suitably low permeability is achieved by having a sufficiently low water-cement ratio, by ensuring as thorough compaction of the concrete as possible and by ensuring sufficient hydration of cement through proper curing methods. Therefore, for given aggregates, the cement content should be sufficient to provide adequate workability with a low water-cement ratio so that concrete can be thoroughly compacted with the means available.

REQUIREMENTS FOR DURABILITY

Minimum cement contents for different exposures and sulphate attack are given in Tables 2 and 3 for general guidance.

TABLE 2 MINIMUM CEMENT CONTENT REQUIRED IN CEMENT CONCRETE TO ENSURE DURABILITY UNDER SPECIFIED CONDITIONS OF EXPOSURE

EXPOSURE	PRESTRESSED CONCRETE	
	Minimum Cement Content kg/m3	Maximum Water-Cement Ratio
Mild — For example, completely protected against weather, or aggressive conditions, except for a brief period of exposure to normal weather conditions during construction	300	0.65
Moderate — For example, sheltered from heavy and wind driven rain and against freezing, whilst saturated with water, buried concrete in soil and concrete continuously under water	300	0.55
Severe — For example, exposed to sea water,	360	0.45

alternate wetting and drying and to freezing whilst wet subject to heavy condensation or corrosive fumes

NOTE — The minimum cement content is based on 20 mm nominal maximum size. For 40 mm aggregate, minimum cement content should be reduced by about 10 percent under severe exposure condition only; for 12.5 mm aggregate, the minimum cement content should be increased by about 10 percent under moderate and severe exposure conditions only.

To minimize the chances of deterioration of concrete from harmful chemical salts, the levels of such harmful salts in concrete coming from the concrete materials, that is, cement, aggregates, water and admixtures as well as by diffusion from the environments should be limited. Generally, the total amount of chlorides (as Cl–) and the total amount of soluble sulphates (as SO3 –) in the concrete at the time of placing should be limited to 0.06 percent by mass of cement and 4 percent by mass of cement respectively.

TABLE 3 REQUIREMENTS FOR CONCRETE EXPOSED TO SULPHATE ATTACK

CLASS	CONCENTRATION OF SULPHATES EXPRESSED AS SO_3			TYPE OF CEMENT	REQUIREMENTS FOR DENSE, FULLY COMPACTED CONCRETE MADE WITH AGGREGATES COMPLYING WITH IS : 383-1970*	
	In Soil		In Ground Water (Parts per 100 000)			
	Total SO_3 (Percent)	SO_3 in 2 : 1 Water Extract g/l			Minimum Cement Content	Maximum Free Water/ Cement Ratio
(1)	(2)	(3)	(4)	(5)	(6) kg/m^3	(7)
1.	Less than 0.2	—	Less than 30	Ordinary Portland cement or Portland slag cement	280	0.55
2.	0.2 to 0.5	—	30 to 120	Ordinary Portland cement (*see* Note 5) or Portland slag cement	330	0.50
3.	0.5 to 1.0	1.9 to 3.1	120 to 250	Ordinary Portland cement (*see* Note 5)	330	0.50

NOTE 1 — This table applies only to concrete made with 20 mm aggregates complying with the requirements of IS : 383-1970* placed in near-neutral groundwater of *p*H 6 to *p*H 9, containing naturally occurring sulphates but not contaminants such as ammonium salts. For 40 mm aggregate, the value may be reduced by about 15 percent and for 12.5 mm aggregate, the value may be increased by about 15 percent. Concrete prepared from ordinary Portland cement would not be recommended in acidic conditions (*p*H 6 or less).

NOTE 2 — The cement contents given in Class 2 are the minimum recommended. For SO_3 contents near the upper limit of Class 2, cement contents above these minimum are advised.
NOTE 3 — Where the total SO_3 in col 2 exceeds 0.5 percent, then a 2 : 1 water extract may result in a lower site classification if much of the sulphate is present as low solubility calcium sulphate.
NOTE 4 — For severe conditions such as thin sections under hydro-static pressure on one side only and sections partly immersed, considerations should be given to a further reduction of water cement ratio, and if necessary an increase in the cement content to ensure the degree of workability needed for full compaction and thus minimum permeability.
NOTE 5 — For class 3, ordinary Portland cement with C_3A content not more than 5 percent and $2C_3A + C_4AF$ (or its solid solution $4CaO, Al_2O_3, Fe_2O_3 + 2CaO, Fe_2O_3$) not more than 20 percent is recommended. If this cement is used for class 2, minimum cement content may be reduced to 310 kg/m^3.
*Specification for coarse and fine aggregates from natural sources for concrete (*second revision*).

m) CONCRETE MIX PROPORTIONS

Mix Proportion — The mix proportions shall be selected to ensure that the workability of the fresh concrete is suitable for the conditions of handling and placing, so that after compaction it surrounds all prestressing tendons and reinforcements if present and completely fills the formwork. When concrete is hardened, it shall have the required strength, durability and surface finish.

The determination of the proportions of cement, aggregates and water to attain the required strengths shall be made by designing the concrete mix. Such concrete shall be called 'Design mix concrete'.

For prestressed concrete construction, only 'Design mix concrete' shall be used. The cement content in the mix should preferably not exceed 530 kg/m^3.

Information Required — In specifying a particular grade of concrete, the information to be included shall be:
a) Grade designation,
b) Type of cement,
c) Maximum nominal size of aggregates,
d) Minimum cement content,
e) Maximum water-cement ratio, and
f) Workability.

In appropriate circumstances, the following additional information may be specified:
a) Type of aggregate,
b) Maximum cement content, and
c) Whether an admixture shall or shall not be used and the type of admixture and the conditions of use.

Design Mix Concrete

The mix shall be designed to produce the grade of concrete having the required workability and a characteristic strength not less than appropriate value given in Table 1. The procedure given in Indian Standard Recommended guidelines for concrete mix design (under *preparation)* may be followed.

n) PRODUCTION AND CONTROL OF CONCRETE

Quality of Materials — It is essential for designers and construction engineers to appreciate that the most effective use of prestressed concrete is obtained only when the concrete and the prestressing steel employed are of high quality and strength. The provisions of Production and control of concrete of IS: 456-1978* shall apply; except that no hand mixing shall be permitted in prestressed concrete work.

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*Code of practice for plain and reinforced concrete (*third revision*).

o) TENDONS

<u>Tendon protection</u>

Unbonded tendons

Unbonded tendons are protected by a layer of grease inside a plastic sheath. These materials should comply with the recommendations given in the draft **BS** EN 10138(16).Under normal conditions, the strand are supplied direct from the manufacturer already greased and sheathed. In no circumstances should PVC be used for the plastic sheath, as it is suspected that chloride ions can be released in certain conditions

Strand type	Steel number	Nominal tensile strength (MPa)	Nominal diameter (mm)	Cross-sectional area (mm^2)	Nominal mass (kg/m)	Characteristic value of maximum force (kN)	Maximum value of maximum force (kN)	Characteristic value of 0.1% proof force (kN)
12.9 'Super'	Y1860S7 1.1373	1860	12.9	100	0,781	186	213	160
15.7 'Super'	Y1770S7 1.1375	1770	15.7	150	1.17	265	302	228
15.7 'Euro'	Y1860S7 1.1373	1860	15.7	150	1.17	279	319	240
15.2 'Drawn'	Y1820S7G 1.1371	1820	15.2	165	1.290	300	342	258
Note: The table is based on information from BS EN 10138-3[1.6]								

Table 4. Strand type and steel number

Bonded tendons

Bonded tendons are placed in metal or plastic ducts, which can be either circular or oval in form. The oval duct is used in conjunction with an anchorage, which ensures that between four and six strands are retained in the same plane in order to achieve maximum eccentricity.

p) DUCTS

The duct is the sheath for the PT strand or bar. Because of tendon geometry and profile restrictions, the duct must be accurately fabricated, placed and secured. The duct is not only the conduit for the tendon during installation but also provides a protection layer (see figure) and is a grout channel for the tendon.

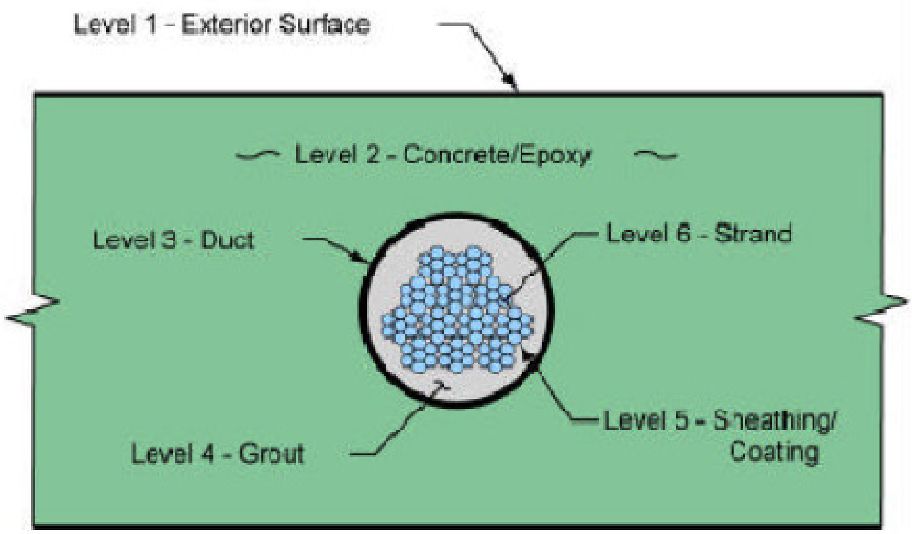


Figure 5. The levels of protection of PT steel.

Duct Materials

Duct materials are required to be high density polyethylene (HDPE), polypropylene or schedule 40 galvanized steel pipes.

Internal vs. External

Internal ducts are completely encased by the concrete and must be corrugated polypropylene. The corrugated duct provides better force transfer between the grout and surrounding concrete. The duct is placed in the concrete forms prior to concrete placement and must be securely fastened to maintain proper position, alignment and to prevent damage during concrete placement.

External ducts are only partially contained by the structural concrete. The anchorages and deviation blocks are encased within the structural concrete. In the deviation blocks and anchor areas, schedule 40 galvanized steel pipes are used for ducts because of their strength and because they can be accurately bent to the correct shape. HDPE pipe is used for external portions between the anchorages and any deviation blocks.

One difficulty with external tendons is keeping the tendon steel centered within the duct during grouting. The duct often sags and without careful effort will rest directly on the tendon, which prevents grout from covering the top of the tendon. This reduces the level of protection and if bleed-water is present, the probability of tendon corrosion and failure will increase.

q) ANCHORAGES

The design of anchorages is critical in external post-tensioning applications. The location of the anchorages must be carefully considered to not affect the structure adversely. The anchorage points apply additional horizontal force and vertical forces to the existing structure.

It is important that the structure be checked to ensure that it can support these additional forces. Anchorages are attached to a soffit (slab or beam); passed through a beam or column and anchored on the vertical surface; or attached to a bracket that envelops a column. Some factors that need to be considered in the design of anchorages are:

Attached to soffit:

When the forces are relatively small, the tendons can be attached to the slab soffit with drilled anchors. Below fig shows an example of a single strand anchorage at the soffit of a slab. The attachment devices must be able to handle the shear force that is the horizontal component of the tendon's force. They must also be able to clamp the anchorage to the soffit overcoming the vertical component of the tendon force so that the anchorages do not dislodge.

Through a beam or column:

External tendons can be easily attached to existing beams. The bearing plate of the anchorage must be designed to safely distribute the forces from the anchor to the concrete. The designer should also check the existing reinforcement in the beam to ensure that it can safely transfer the forces to the existing structure. Also, when passing though a beam, the existing beam must be able to handle the sideways bending induced by the tendon force. Figures below show the critical locations of induced forces when strengthening tendons pass through a beam.

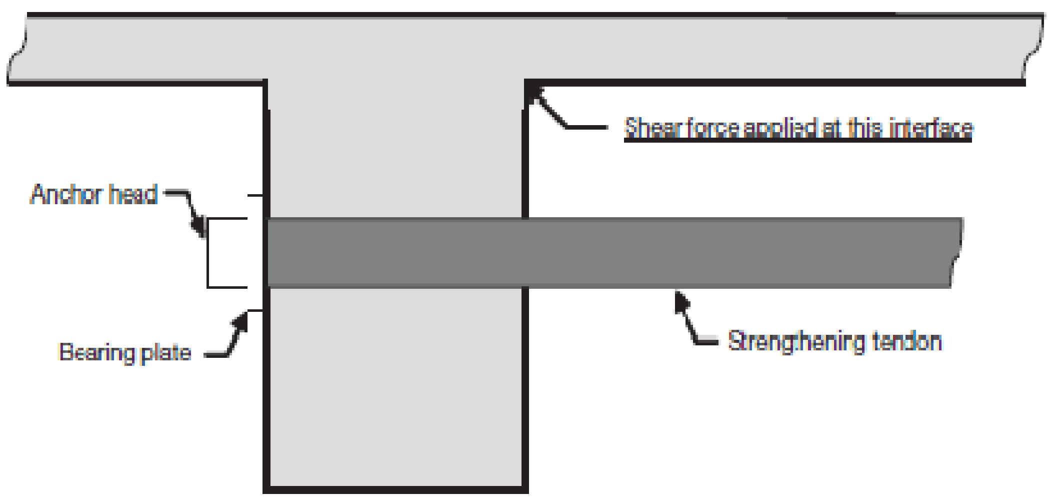


Figure 6. Cut through beam indicating critical location of shear force.

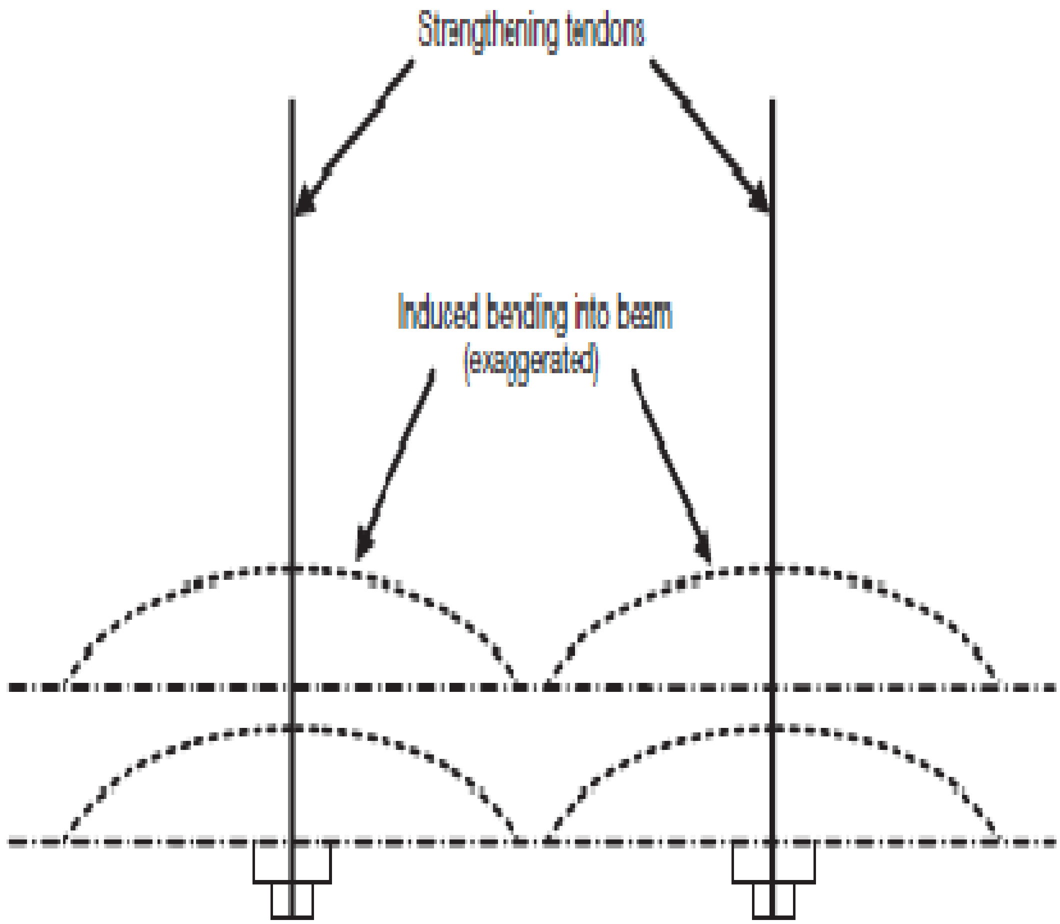


Figure 7. Plan view indicating induced bending in the beam.

Bracket at column:

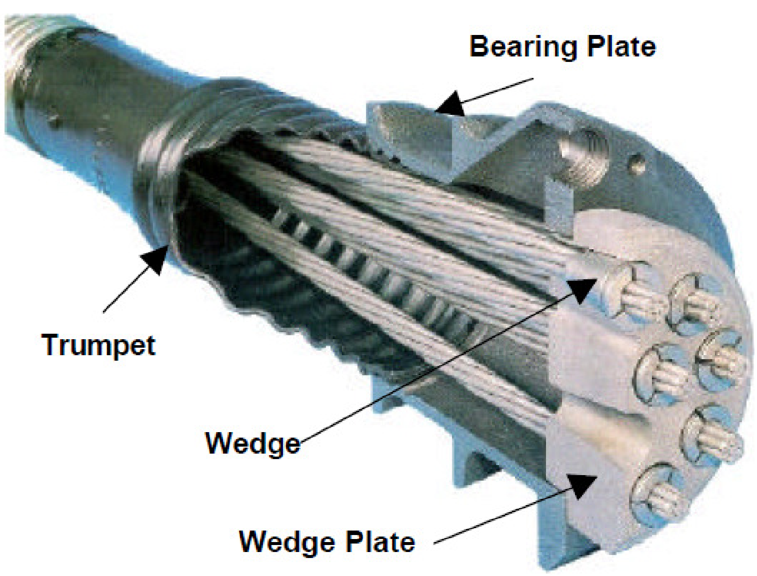
It may sometimes be possible to drill through the column or attach a bracket to support the anchorage at the columns. The bracket must be designed to have sufficient bearing to distribute the loads from the tendon to the concrete. The column must be checked for shear capacity at the anchorage location. Shear reinforcing may be used to reinforce the column. Fig. above shows an example of a column bracket. External strengthening applies axial forces on the structure, the Engineer should evaluate the axial capacity of the structure to confirm that axial forces can safely be transferred from the tendon to the structure

Figure 8. Multi-Strand Anchorage

Figure 9. Anchorage by looping the wires in a Slab.

Figure 10. Slab system anchorage components

r) WEDGES



The Post Tensioning Wedges are installed together around the cable and inside the cone openings at the edge of the tennis court. If you look closely at the wedges you can see that they have teeth that allow the steel part to grip the cable. The teeth are angled in a way that the cable can easily be pulled pass the wedges but when the cable is released the teeth cinch against the cable and the wedge shaped part is forced strongly against the embedded anchor. The excess cable now can be torched off and the post tensioned force is now fully engaged onto the slab.

Figure 11. Wedges

s) BARRELS

Wedges are locked into the barrel.In general barrels are cylindrical in shape.

Figure 12: Barrel

4) EQUIPMENT

The specialist equipment required for post-tensioning consists of the following items, not all of which would be required on site.

1. Stressing jack
2. Swaging jack
3. Strand threading machine
4. Strand cutters or shears
5. Grout mixer and pump

The strand threading machine and grouting outfit are required only for bonded tendons. Strand may be threaded into the sheathing either before placing the sheathing in position or after it has been placed but before concreting, or after concreting. The pushing machine would, of course, not be needed on site if the tendon lengths are short enough to be delivered to site ready threaded.

Jacks are designed to grip the strand(s) either at the front or at the rear. In the latter case the design of jack is mechanically simpler but it needs an extra length of strand (of the order of one meter but check for the particular jack to be used) which is cut off after stressing. The older jacks have the facility for pushing the wedge cone forward after stressing and this used to be a manual operation; now almost all jacks automatically move the wedges forward into the barrel to lock the strand as part of the automated stressing operation. Jacks are equipped with facility for measuring extension of strand and are calibrated with their hydraulic pumps so that a direct reading of jacking force is displayed.

Figure 13. Prestressing jack

Figure 14. Monojack

Figure 15.Grout mixer and pump

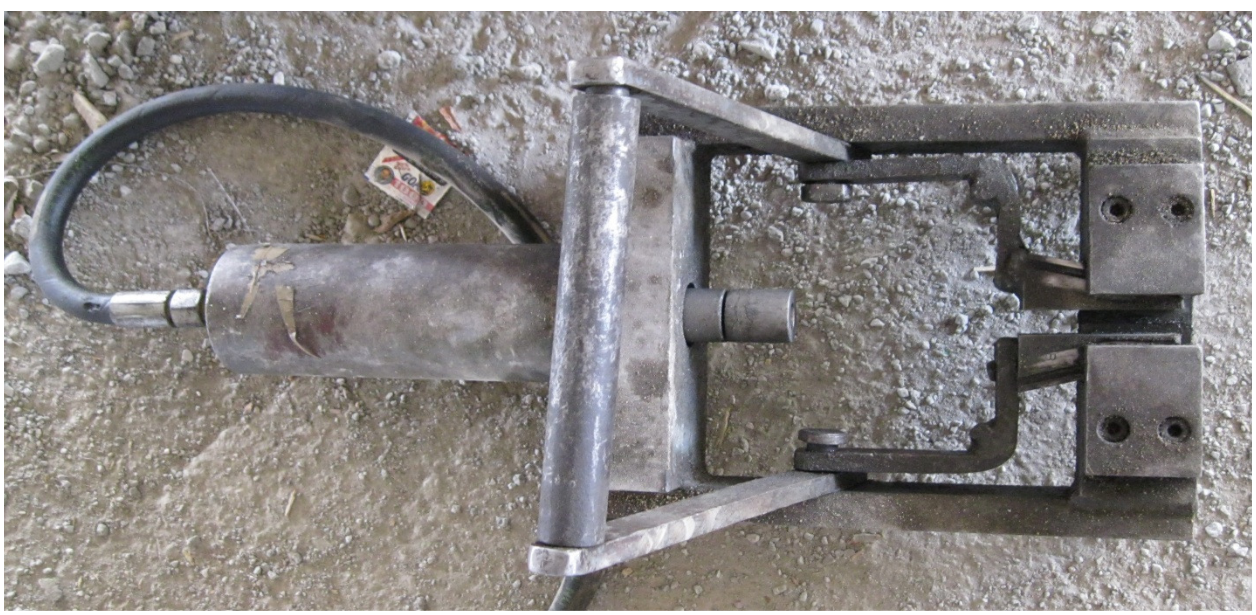


Figure 16. Strand threading machine

Apart from maneuverability with regard to the handling equipment and space available at a particular site, the important features of a jack of short stroke, may have to be stressed in stages. The process requires the anchoring cones to grip the strand and release it several times, which may weaken the serrations.

The sizes and weights of equipments vary in different systems but those for monostrand use are very similar in shape and size. These and their pumps are light to be handled manually. Flat tendons are usually stressed using the monostrand jack.

Multistrand jacks from different suppliers differ in shape, size and weight. Some are of a similar shape to the normal monostrand jack, but bigger, while others are much larger in diameter – up to 400 mm (16inch)—and shorter.

A multistrand jack is much heavier than a monostrand jack, it may weigh as much as 300kg. It therefore needs crane time during stressing. Because of the wide variation in equipment from different sources, no details are given. It is recommended that particulars are obtained from specialists in the area.

5. ASSEMBLY OF PRESTRESSING AND REINFORCING STEEL

PRESTRESSING STEEL

Straightening- The wire, as supplied, shall preferably be self-straightening when uncoiled. If it is not so, the wire may need to be mechanically straightened before use. In this event, care shall be taken to avoid alteration in the properties of the wire during the straightening process and preferably a test shall be made on a sample of the wire after straightening.

In the case of high tensile alloy steel bars, any straightening (or bending if the design provided for curved bars) shall be carried out by means of a bar-bending machine. Bars shall not be bent when their temperature is less than 10°C.

In no case heat shall be applied to facilitate straightening or bending of prestressing steel.

Arrangement of Wires and Positioning

All prestressing steel shall be carefully and accurately located in the exact positions shown in the design drawings. The permissible tolerance in the location of the prestressing tendon shall be ±5 mm. Curves or bends in prestressing tendon required by the designer shall be gradual and the prestressing tendon shall not be forced around sharp bends or be formed in any manner which is likely to set up undesirable secondary stresses.

The relative position of wires in a cable, whether curved orvstraight, shall be accurately maintained by suitable means such as sufficiently rigid and adequately distributed spacers.

In the case of post-tension work, the spacing of wires in a cable shall be adequate to ensure the free flow of grout.

The method of fixing and supporting the steel in the mould or the formwork shall be such that it is not displaced during the placing or compaction of the concrete or during tensioning of the steel.

The type of fixtures used for positioning the steel shall be such that it does not give rise to friction greater than that assumed in the design.

Jointing

High tensile wire other than hard-drawn wire may be joined together by suitable means provided the strength of such joints is not less than the individual strengths of the wires being joined.

Hard-drawn wire used in prestressed concrete work shall be continuous over the entire length of the tendon.

High tensile steel bars may be joined together by means of couplings, provided the strength of the coupling is such that in a test to destruction, the bar shall fail before the coupling.

Welding shall not be permitted in either wires or bars.

All cutting to length and trimming of the ends of wires shall be done by suitable mechanical or flame cutters. Where flame cutters are used, care shall be taken to ensure that the flame does not come into contact with other stressed wires or concrete.

Bars shall preferably be ordered to the exact length required. Any trimming required shall be done only after the bar has been tensioned and the grout has set.

Protection of Prestressing Steel and Anchorages — in all constructions of the post-tensioned type, where prestressing is initially carried out without bond, the prestressing tendon shall, at a subsequent date and generally not later than one week after prestressing, be given and adequate protection against corrosion.

Internal prestressing steel — Internal prestressing steel is best protected by a cement or cement-sand grout preferably in colloidal form. Care shall be taken to prevent segregation and, for that purpose, only fine sand shall be used.

The grout shall be placed under pressure, and it shall be ensured that the entire space between the duct and the prestressing tendon is properly filled with grout.

Where small ducts are encountered, it is advisable that water is flushed through prior to grouting, care being taken to see that all water is subsequently displaced by grout. In the case of butted assemblies, flushing with water shall be carried out only after the jointing material has properly hardened.

Injection shall proceed from one end or preferably in case of curved ducts from the lowest point of the curve, and shall be continued until the grout overflows from the other end.

External prestressing steel — the protection of external prestressing steel is usually best done by encasing the tensioned wires, cables or bars in a dense concrete secured to the main concrete, for example, by wires left projecting from the latter. If a cement-sand mix is used, the cover provided and its density should be adequate to prevent corrosion. Alternatively, the steel may be encased in bitumen or, where the steel is accessible for inspection and maintenance, paint protection may be provided.

The anchorage shall be adequately protected against damage or corrosion soon after the completion of the final stressing and grouting operations.

Cover

In pre-tensioned work, the cover of concrete measured from the outside of the prestressing tendon shall be at least 20 mm.

In post-tensioned work, where cables and large-sized bars are used, the minimum clear cover from sheathing/duct shall be at least 30 mm or the size of the cable or bar whichever is bigger.

Spacing

In the case of single wires used in pre-tension system, the minimum clear spacing shall not be less than greater of the following:
a) 3 times diameter of wire, and
b) 1 times the maximum size of aggregate.

In the case of cables or large bars, the minimum clear spacing (measured between sheathings/ducts, wherever used) shall not be less than greater of the following:
a) 40 mm,
b) Maximum size of cable or bar, and
c) 5 mm plus maximum size of aggregate.

Grouped Cables

Cables or ducts may be grouped together in groups of not more than four as shown in Fig. 16.

The minimum clear spacing between groups of cables or ducts of grouped cables shall be greater of the following:
a) 40 mm, and
b) 5 mm plus maximum size of aggregate.
The vertical distance between groups shall not be less than 50 mm
(See Fig. 17).

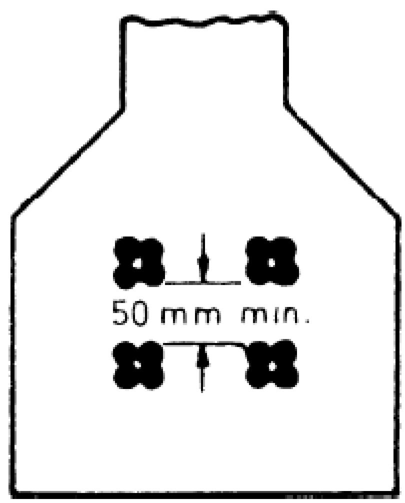


Figure 17. Spacing of cables

Sheaths and Extractable Cores

Sheaths shall be sufficiently water-tight to prevent concrete laitance penetrating in them in quantities likely to increase friction. Special care shall be taken to ensure water tightness at the joints.

They shall be preferably machine-manufactured and have bores sufficiently large to allow being easily threaded on to the cable or bar in long lengths.

The tubes or sheaths shall be of such strength as not to be dented or deformed during handling or concreting.

The alignment of all sheaths and extractable cores shall be correct to the requirements of the drawings and maintained securely to prevent displacement during placing and compaction of concrete. The permissible tolerance in the location of the sheaths and extractable cores shall be ± 5 mm. Any distortion of the sheath during concreting may lead to additional friction.

Reinforcing Steel

Provisions for assembly of reinforcement given in IS: 456-1978* shall apply.

The requirements of cover and spacing between bars shall conform to IS: 456-1978*.

6. DESINGN OF RCC BEAM

INPUT

Properties of material

density of steel	=	78.5	kn/m3
Grade of concrete (Fck)	=	40	n/mm2
Grade of steel (Fy)	=	500	n/mm2
width of beam(b)	=	1.5	m
depth of the beam(d)	=	1.5	m
density of concrete(γ)	=	25	kn/m3
thickness of slab(t)	=	0.15	m
bay width(w)	=	8.91	m
length of beam(l)	=	27.147	m

LOADINGS

live load(L.L)	=	5	kn/m2
dead load(D.L)	=	2	kn/m2

CALCULATIONS

self wt. of beam(γ*b*d)	=	56.25	kn/m
self wt. of slab(γ*t*w)	=	33.4125	kn/m
L.L+D.L in kn/m[(L.L+D.L)*w])	=	62.37	kn/m
Total wt. acting on beam(W)	=	152.0325	kn/m
Bending Moment(M=Wl^2/8)	=	14005.226	kn-m
factored B.M(M'=1.5*M)	=	21007.84	kn-m

CALCULATING DEPTH REQUIRED FOR A R.C.C BEAM

From IS:456 M'=0.36Fck*b*Xu(d'-0.42Xu)

Fck in kn/m2	=	40000	kn/m2
Fy in kn/m2	=	500000	kn/m2
d'2	=	2.6232203	
d'	=	1.6196359	m

CALCULATING Ast REQUIRED

	Ast=(Xu*0.36*Fck*b)/(0.87*Fy)		
Xu	=	0.7450325	m
Num	=	16092.702	
Denominator	=	435000	
Ast	=	0.0369947	m2
Youngs modulus	E=5000sqrtFck		
E	=	31622.777	
E in Kn/m2	=	31622777	kn/m2
moment of inertia (bd3/12)	=	0.5305	m4
deflection(5Wl4/384EI)	=	0.0640881	m
Actual deflection		64.09	mm
Longterm deflection	=	160.22	mm
(App. 2.5 times Short term deflection)			
Allowable deflection	=	0.0775629	m
As per Is 456			

Longterm deflection is more than allowable deflection, hence redesign
provide depth of 2.1m

d"	=	2.2	m
self weight of beam	=	82.5	kn/m
total weight acting on the beam	=	178.2825	kn/m
bending moment	=	16423.375	kn-m
moment of inertia(bd3/12)	=	1.331	m4
factored bending moment	=	24635.063	kn-m
Mu=0.87*Fy*Ast*d(1-(Ast*Fy/(b*d*Fck))			
Ast	=	0.02996	m2
Deflection	=	0.0299541	m
actual deflection	=	29.954137	mm
longterm deflection	=	74.885342	mm

longterm deflection is less than allowable deflection, hence it its safe

Check for shear

shear force (Vu=wl/2)	=	2419.9175	kn
Factored shear force(Vu'=1.5*Vu)	=	3629.8763	kn
Nominal shear stress(τv=Vu'/b*d")	=	1099.9625	kn/m2

As per Is456 clause 40.2.3, τv<4.0

Total depth(D)	=	2260	mm
100Ast/b*d"	=	0.9078788	

From table 19 of IS 456 for (100Ast/b*d)=0.9078 & M40, τc=0.631n/mm2

Tc	=	0.631	n/mm2

Providing 8 legged stirrups of 8mm dia

Dia	=	8	mm

total cross sectional area of stirrup legs

Asv	=	401.92	mm2
Vus=Vu'-τc.b*d"	=	1547576.3	n
spacing of stirrups	(0.87*Fy*Asv*D)/Vus		
Sv	=	255.32024	mm

Considering Torsion

Total depth(D)	=	2.16	M
breadth(b)	=	1.5	M
Tu	=	1500	kn-m
torsional moment(Mt)	=	2152.94	kn-m
Mt=Tu*((1+(D/b))1.7)			
Me=Mu+Mt	=	26269.8	kn-m
Ve=Vu+1.6(Tu/b)	=	5153.53	Kn
Mu=0.133*b*Fck*d*d			
D	=	1.80894	M
Ast=(Xu*0.36*Fck*b)/(0.87*Fy)	=	0.04934	m2
young's modulous(E)	=	31622.8	n/mm2
Moment of inertia(bd3/12)	=	1.25971	m4

actual deflection			
5Wl4/384EI	=	0.02699	M
longterm deflection (2.5 times of actual deflection)	=	67.4732	Mm
actual deflection (l/350)	=	77.5629	Mm

longterm deflection is less than actual deflection

CHECK FOR SHEAR

τv=Ve/bd	=	1590.59	n/m2
100Ast/bd	=	1.52276	
therefore τc=0.794 n/mm2			
Tc	=	0.794	n/mm2
providing 10 legged stirrups of 8mm dia			
Dia	=	10	Mm
total cross sectional area of stirruped legs			
Asv	=	628	mm2
Vus=Ve-τc.b.d	=	5150.95	Kn
spacing of stirrups	(0.87*Fy*Asv*D)/Vus		
Sv	=	114.555	

7. DESIGN OF P.T BEAM

length(l)	=	27.147	M
width(b)	=	1.5	M
depth(d)	=	1.5	M
force in each tendon	=	110	Kn
no of strands	=	96	
eccentricity(e)	=	650	mm
section modulus(Z)	=	0.5625	m3
bending moment(M)	=	11204.1812	kn-m
total wt of beam(W)	=	152.0325	kn/m
young's modulus(E)	=	31622776	kn/m2
moment of inertia(I)	=	0.421875	m4
Fy	=	500	n/mm2
P	=	10560	Kn
A	=	2.25	m2
P/A	=	4693.333333	kn/m2
Pe/Z	=	12202.66667	kn/m2
M/Z	=	19918.54436	kn/m2
Tensile stress=(P/A)+(Pe/Z)-(M/Z)	=	-3.02254436	n/mm2
compressive stress=(P/A)-(Pe/Z)+(M/Z)	=	12.40921102	n/mm2
We=(P.e.10)/l2	=	93.13943283	kn/m
deflection 1(W1)	=	0.080589564	M
deflection 2(W2)	=	0.049371459	
effective deflection(Wf)	=	31.21810542	mm
longterm deflection(2.5 times)	=	78.04526356	mm

slightly over than allowable deflection(77.563mm)

Xu=0.261d			
Me=Fpu.Ap(d-0.42Xu)	=	20747705990	n-mm

Mf=1.5M	=	16806.2718	kn-m
Me>Mf			
Ast=0.2%bd	=	4500	mm2
check for shear(Vu=Wl/2)	=	2063.613139	Kn
factored shear force(Vu')	=	3095.419708	Kn
nominal shear stress(τv=Vu'/b.d)	=	1375.742093	kn/m2
100Ast/b.d	=	0.2	
τc=0.34 n/mm2			
Vus=Vu'-τc.b.d	=	2330419.708	N
providing 10 legged stirrups of 8mm dia			
no of legs	=	10	
Dia	=	8	Mm
Asv	=	502.72	mm2
spacing of stirrups(Sv)	=	140.7578209	Mm

8. QUANTITY CALCULATIONS

R.C.C BEAM

CONCRETE:

Area of Concrete = $1.5*2.2-A_{st}-A_{sv}$
= $3.2696\ m^2$

Volume = $3.2606*27.147$
= $88.7608\ m^3$

STEEL:

Longitudinal reinforcement = A_{st}*Density of steel*Length of the beam
= 0.02996*7850*27.147
= 6384.594 Kgs

Shear Reinforcement= A_{sv}*density of steel*length of the beam
= 1049 Kgs

P.T BEAM

CONCRETE:

Area of Concrete = $A_g-A_{st}-A_{sv}$-Area of tendons
= $2.2354\ m^2$

Volume of concrete = Area of concrete * length
= $60.684\ m^3$

STEEL:

Longitudinal reinforcement = Bottom & top reinforcement
= $2*(4500*10^{-6}*7850)$
= 2*958.9678
= 1917.9356 kgs

Tendons Required = No. Of Tendons * Area of Tendons *Density of steel
= $96*100*7850*10^{-6}$
= 2045.798 Kgs

Shear Reinforcement = Asv*Density of steel* length of the beam
= $502.4*10^{-6}*7850*27.147$
= 107.0634 Kgs

S.No	PARAMETERS	R.C.C	POST TENSIONING	% OF SAVING
1	Width	1.5 m	1.5 m	0
2	Depth	2.2 m	1.5 m	31.82
3	Concrete	88.7608 m3	60.684m3	31.65
4	Steel			
5	Longitudinal	6384.594 kgs	1917.9356 kgs	70
6	Shear	1049 kgs	215.346 kgs	79.48
7	Tendon	________	2045.798 kgs	

Table 5. comparision of R.C.C & P.T beam

R.C.C	QTY	RATE/UNIT	TOTAL(RS)
Concrete	88.7608 m3	5000	4,43,804
Longitudinal	6384.594 kgs	56	3,57,537
Shear	1049 kgs	56	58,744

Table 6.Qquantity of RCC beam

POST TENSIONING	QTY	RATE/UNIT	TOTAL
Concrete	60.684m3	5000	3,03,420
Longitudinal	1917.9356kgs	56	1,07,404
Shear	215.346 kgs	56	12,059
Tendon/strand	2045.798	100	2,04,579

Table 7. Quantity of PT beam

Percentage savings on concrete = 31.63%
Percentage savings on steel = 22.45%
Total savings = 27.04%

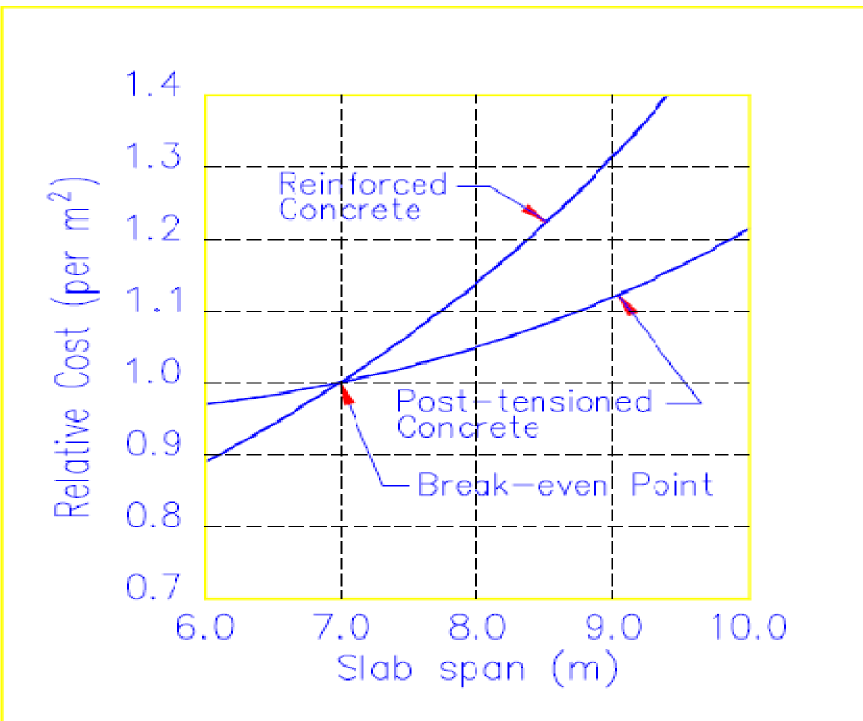
9. ECONOMICS

a) When is Post-tensioning Cost Effective?

The relative economics of post-tensioning versus other forms of construction vary according to the individual requirements of each case. In any basic comparison between post-tensioned and reinforced concrete one must consider the relative quantities of materials including formwork, concrete, reinforcement and post-tensioning. Other factors such as speed of construction, foundation costs, etc., must also be given consideration.

There is not always sufficient time or budget to carry out comparative feasibility studies for all structural solutions. There are however, some useful guidelines which can be employed when considering post-tensioned alternatives. As can be seen from graph 1 below, post-tensioned should be considered as a possible economic alternative for most structures when spans exceed 7.0 metres.

The graph illustrates two main points. Firstly, how with increasing span the difference in cost between reinforced and post-tensioned concrete flat slabs also increases. Secondly, using an index of one for a 7.0 m span how the cost will vary for other spans. For example, a post-tensioned 10.0 m span will cost approximately 20% more than a post-tensioned 7.0 m span.



Graph 1. Cost comparison - Reinforced vs. Post-tensioned flat slab.

b) Speed of Construction

Economics and construction speed are heavily linked in today's building construction environment. The speed of construction of a multi storey building is foremost in achieving economic building construction.

The key factor in the speed of construction of a post-tensioned framed building is expedient use and re-use of formwork. Post-tensioning allows for the early recovery of formwork by early stressing of tendons. Slab system tendons are usually stressed at the following minimum compressive strengths:-

a) Initial stress of slab tendons 24 hours after the pour of concrete for control of early shrinkage stresses at a minimum concrete strength of 7 MPa.

b) Final stressing of the slab system tendons may occur when the concrete has attained 22 MPa based on site cured cylinders in accordance with clause 19.6.2.8 of AS3600.

A typical floor cycle for a multi storey office development is shown below in figure below. This building has a floor area of approximately 1000 m2 and is divided into two pours per floor by a construction joint. It is normal to use two full sets of formwork in this type of construction.

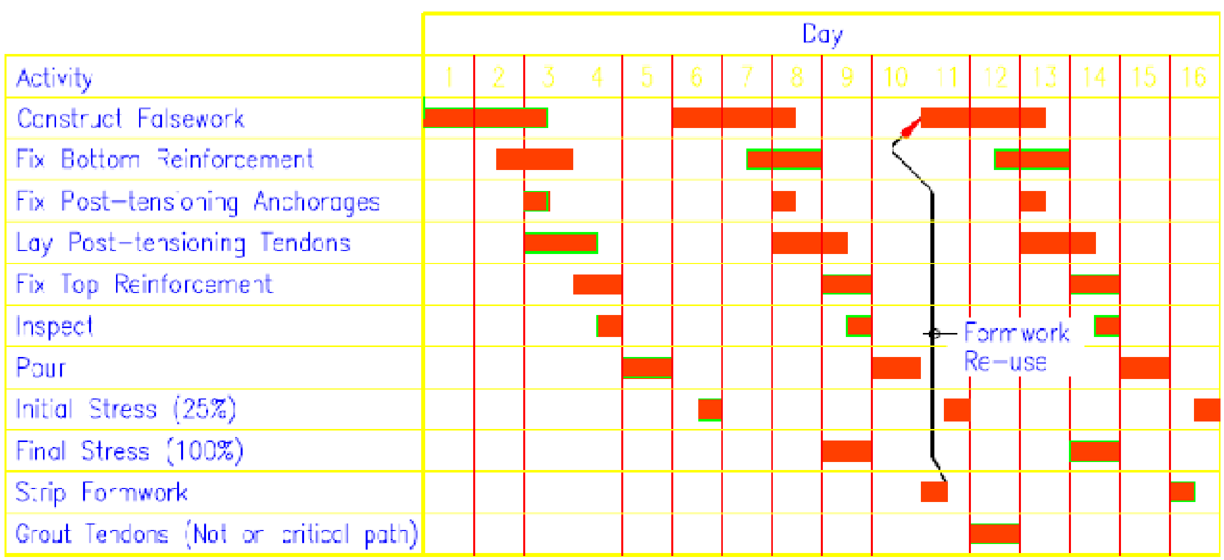


Figure 18. Typical 5 day construction cycle. Note that in tower construction it is usual to break the floor into a minimum of two pours. The above cycle is for a half floor with construction of the other half proceeding simultaneously.

c) Factors Affecting the Cost of Post-tensioning

Post-tensioning costs vary from project to project depending upon a number of factors. The cost of post-tensioning is most sensitive to the following influences.

1. Tendon Lengths

The major influence in the cost of post-tensioning depends primarily on the length of the tendon. Short tendons are relatively expensive in comparison with long tendons. The relatively high cost of short tendons results from the fixed cost such as establishment, anchorages and the stressing operation being pro-rated over a lesser tonnage. Experience has also shown the labour cost for larger tendons to be appreciably less than for short tendons. It becomes clear that designers should avoid, if possible, detailing with very short tendons.

2. Tendon Arrangement

The designer should always attempt to use as few tendons as possible. This leads to tendons being placed at the maximum spacing permissible in slabs (approximately 10 times the slab thickness for one way slabs, and approximately 8.5 times the slab thickness for two way slabs).

For example, if the design calls for a tendon containing 3 strands to be placed at 1125 mm centers; it would be more economical to use 4 strand tendons at 1500 mm centre's. If 5 tendons containing 4 strands each are required in a beam, it will be much more cost effective to use 4 tendons containing 5 strands each.

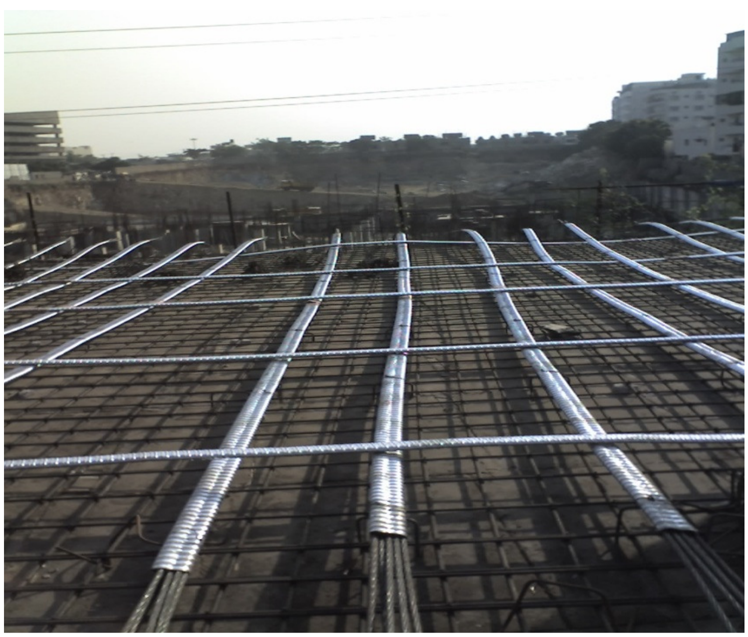


Figure 19. Tendon arrangement.

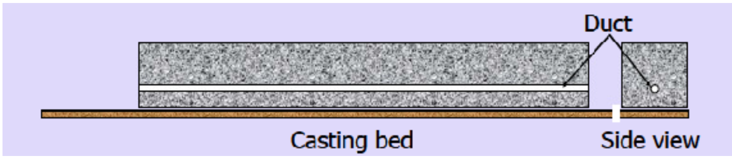
3. Stressing Access

Stressing of tendons from the perimeter of the building is preferable to top pocket stressing due to the following;

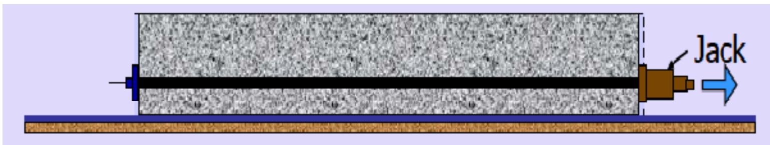
a) Clear access around perimeter permits a faster stressing cycle.

b) A curved nose needs to be used for stressing through a top side pan, which adds to the total loss of prestress through the jack and anchorage by 3%.

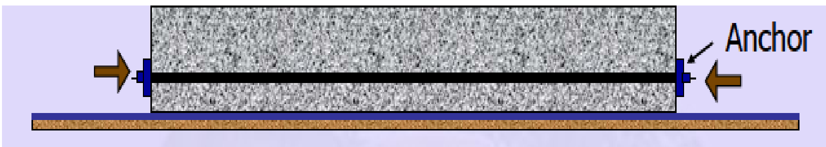
c) Access to the top side pans is restricted by the frames supporting the floor above and other construction debris, increasing the time required to stress the tendons.



a) Casting of concrete.



b) Tensioning of Tendons.



c) Anchoring the tendon at stressing end.

Figure 20. Stages of Post-Tensioning.

4. Structural System

The tendon installation and therefore fixing times for a banded slab is faster than for flat plate and flat slab structures.
In banded slabs the order of laying is:

a) Bands first
b) Slabs second
c) Slab distribution tendons last

With flat plates and flat slabs the tendons are interwoven which demands greater installation time than a comparable banded slab, and therefore increased labour costs.

5. Main Contractor

The ability of the main contractor to manage the sub contractors efficiently on site is paramount as this has a direct bearing on productivity and therefore labours costs. The main contractor also has a role to play ensuring all sub-contractors have continuity of work thereby minimizing down time. The prevailing industrial climate also plays a role.

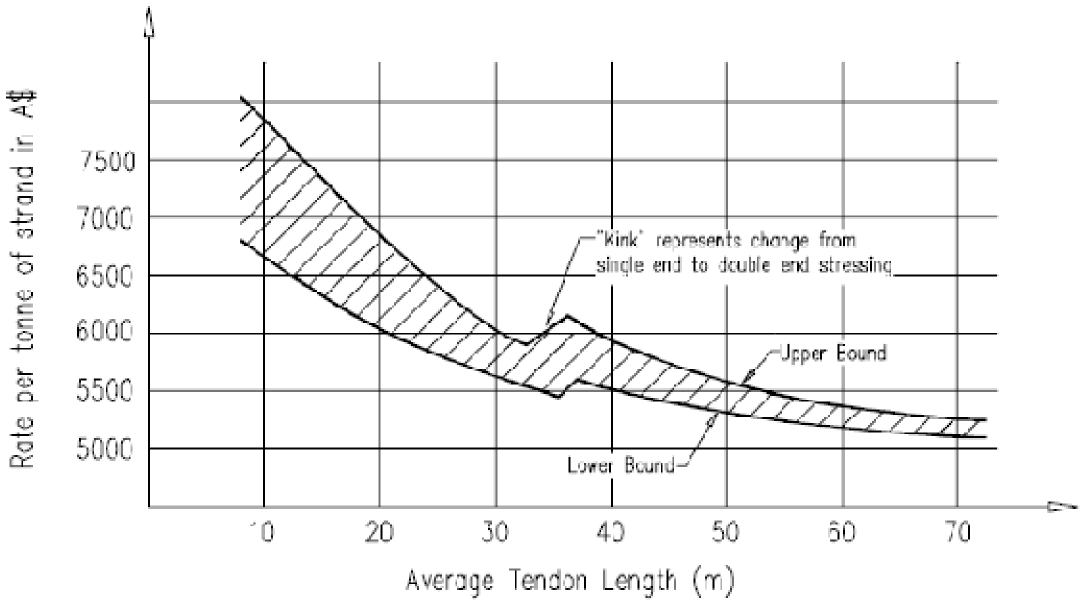
6. Site Access

Restricted site access to the construction site is likely to affect all aspects of the project as materials handling will be slower.

7. Treatment of Construction Joints

Wherever possible construction joints should be stitched using conventional reinforcement in lieu of post-tensioning couplers. The couplers are made up of expensive components and require significant labour and importantly, supervision, to install. Even if couplers are used, significant amounts of conventional reinforcement are required to keep the joint closed until the prestress is applied. Further guidance is offered in the next section.

Of the above, the average tendon length has the most significant impact on the cost of post tensioning and historically there is a reasonable correlation between the two, with the other influences creating a scatter of results which provide an upper and lower bound on the cost of post-tensioning. Figure 5 shows a general range of current costs in Australia based on using 4 strand tendons in a banded slab arrangement and stressing externally.



Graph 2. showing cost of post-tensioning per tonne of strand versus average tendon length. Note that the cost per tonne of strand is inclusive of labour to install, stressing, grouting, and post-tensioning materials such as anchorages, ducts, etc.

10. Economical Design

Of course, the economics of post-tensioned buildings is heavily dictated by the design of the structure. The designer has a role to play in the minimization of material quantities, the selection of the most economical structural system, and the simplification of the detailing allowing for ease and speed of installation. A few design considerations are briefly mentioned below.

a) Partial Prestressing

The advent of what is commonly termed partial prestressing has had a significant effect on the quantity of post-tensioning installed into building structures. Tensile cracking is allowed to occur, with crack control being provided by the bonded tendons and/or supplementary reinforcement. A cracked section analysis needs to be carried out to determine the cracked moment of inertia for use in deflection calculations as well as the steel stresses to confirm adequate crack control. The availability of computer software to carry out these calculations has meant that more often than not the amount of post tensioning is selected to satisfy deflection criteria.

b) Selection of Column Grid

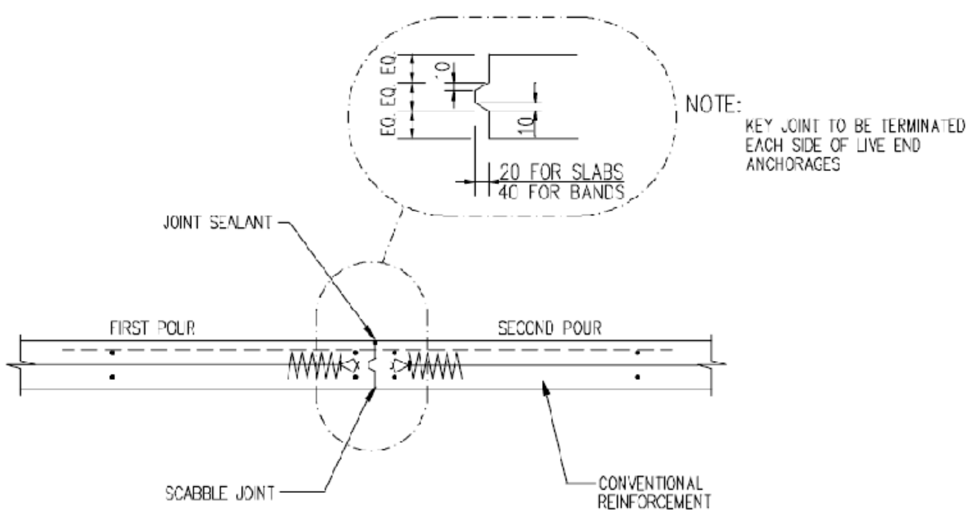
A column grid spacing of between 8 and 10 meters for car parks, shopping centers and offices usually results in the most economical structure while maintaining architectural requirements.

c) Formwork Layout

Formwork layout should be selected to enable quick fabrication with a minimum of form ply cutting. Widths of beams should be standardized in consultation with the main contractor and importantly, the width of the slab between bands should be selected as a multiple of 1200 mm to suit the standard formwork sheet widths.

d) Construction Joint Treatment

As mentioned previously the detail at the construction joint will play a significant role in the economics of the floor system. Post-tensioning couplers should be avoided due to their cost and slow installation. Construction joints should be stitched with conventional reinforcement as shown in figure below.



TYPICAL CONSTRUCTION JOINT DETAIL

Figure 21 – 'Stitched' construction joint detail.

Note that the amount of reinforcement required keeping a construction joint closed (say a crack width of 0.2 mm as for reinforced concrete) depends highly on the restraint of the overall frame. If the frame is very flexible, or alternatively if the construction joint is adjacent to very stiff elements such as core walls, then the amount of reinforcement required is quite low. On the other hand, if the frame is very stiff, large quantities of reinforcement will be required at which point an expansion joint should be positioned rather than a construction joint.

e) Simplicity In Detailing

As with all methods of construction the speed of installation is highly dependent upon the quality of the structural detailing. The designer needs to understand the installation process and be conscious of how their decisions on detailing affect all parties concerned on site. Detailing should be standardized and as simple as possible to understand. Congested areas should be carefully assessed and, as appropriate, large scale drawings and details produced.

f) Anchorage Reinforcement

Standardization of anchorage reinforcement is important. For the slab system, helical reinforcement is preferred by the main contractor due to the speed and ease of installation. It must be noted that by providing a single helix around an anchorage is not adequate since tensile forces are also generated between anchorages. It is usual to detail u-bars plus longitudinal reinforcement along the perimeter to control these forces and to reinforce the un-tensioned area between anchorages.

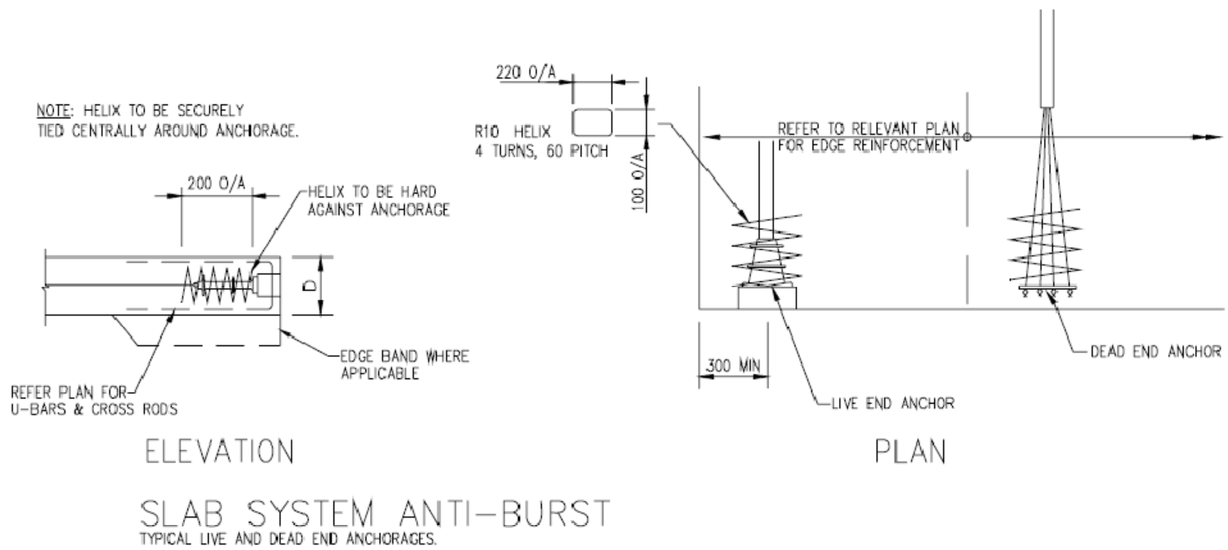


Figure 22 – Slab system anti-burst reinforcement.

g) L/D Ratios

Choosing the right L/D ratio for the structural system and applied loading is important. Choosing a high L/D ratio may minimize the amount of concrete, but will increase the amount of post-tensioning and/or reinforcement required, and perhaps cause increased vibration. Choosing a low L/D ratio in order to minimize post-tensioning may not secure the expected result due to minimum reinforcement rules and adequate residual compression levels to ensure shrinkage cracking is controlled.

h) Load Balancing

The selection of the load to be balanced by the post-tensioning tendons is an important factor in the economics of post-tensioned systems. One of the major advantages of post-tensioning is to reduce the long-term deflection of the structure; however selection of too high a load to balance may incur prestressing costs reducing the economy of the prestressed solution. A combination of a lower level of 'balanced load' and the addition of normal reinforcement at peak moment regions will prove to be a more economical solution in most applications. Table below is a guide to the amount of load to balance under a range of building uses.

Occupancy of building	Partitions and Other Superimposed Dead Load kPa	Live Load kPa	Load to Balance kPa
Car Parks	Nil	2.5	(0.7-0.85)SW
Shopping Centres	0.0 - 2.0	5.0	(0.85-1.0)SW
Residential (check transfer carefully)	2.0 - 4.0	1.5	SW + 30% of partition load
Office Buildings	0.5 - 1.0	3.0	(0.8-0.95)SW
Storage	Nil	2.4 kPa / m height	SW + 20% LL

Note: SW denotes self weight, LL denotes live load.

Table 8. General level of load to be balanced by post-tensioning tendons to give an economic structure.

i) Terminate Tendons Wherever Possible

Often the amount of post-tensioning required within a member varies across its length. 1For example, end bays usually require a greater level of prestress to control deflections than internal bays. Terminating the post-tensioning once it is not required can be achieved by either terminating whole tendons or terminating individual strands using a 'short dead end'.

j) The Use of Finite Element Analysis for Selected Projects

With the advent of sophisticated finite element analysis programs that are relatively easy to use, significant economy can be gained for selected projects. The types of structures benefiting from FEA methods are residential flat plate construction with an irregular supporting column grid and transfer structures such as transfer plates and raft foundations.

We find that the use of FEA methods for these types of structures allow for a better determination of structural load paths and enables the designer to detail and drape the post tensioning tendons to better reflect the slab bending moments. This is what leads to economy.

11.DETAILING

"Detailing for Post-tensioning" addresses the important, but often misunderstood details associated with post-tensioned structures. It has been written for engineers with a modern education who must interpret and use modern design codes. It is hoped that this report will be of interest to practicing engineers and aspiring students who want to "get it right the first time".

a) TENDON DISTRIBUTION

Various methods for distributing the tendons can be used. From a construction aspect it is normal to lay the tendons banded in one direction and evenly distributed in the other, which minimizes the amount of weaving and hence simplifies the laying procedure.

At the column floor intersection, where there is no shear reinforcement, it is recommended that a minimum of two tendons should pass within 0.5h of the column face for internal columns and a minimum of one tendon for external columns parallel to the slab edge. Only tendons within this distance are permitted to contribute towards the shear resistance
Where 1shear reinforcement is present the 0.5h limit may be increased to h.

For ribbed slabs or beams, the distributing of tendons is dictated by the spacing of members but generally one should Endeavour to have a tendon passing within 0.5h of the column face.

For situations where it is not practically possible to place the prestressing tendons within 0.5h from the column, reinforcement should be placed to bridge the vertical force from the adjacent tendon to the columns.

The reinforcement should:

- Be placed under the prestressing tendon.
- have sufficient area to transmit the vertical component of prestressing for that tendon to the column.
- extend a full anchorage length beyond the tendon
- lie within ***0.5h*** of the column and at least one bar should pass over the column.

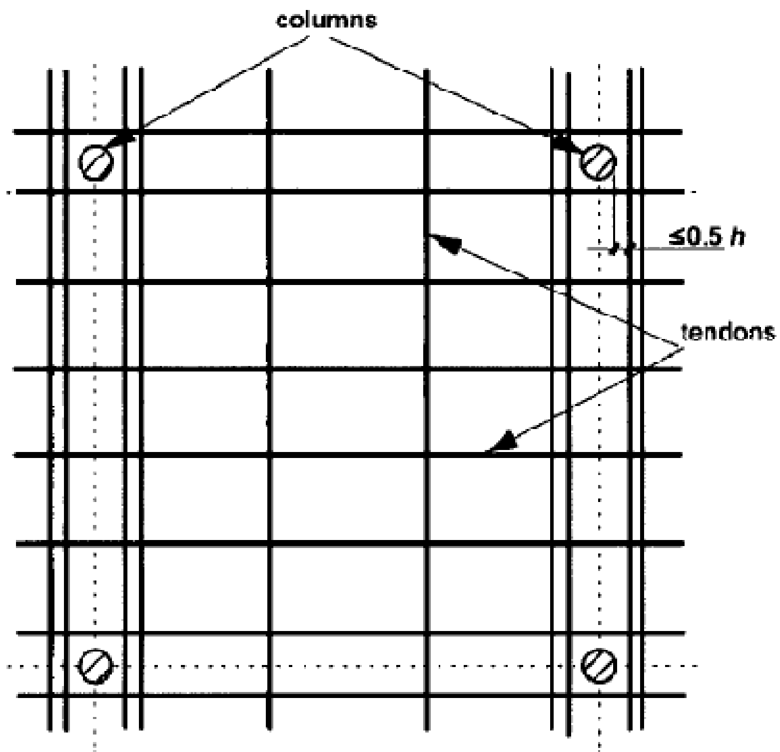


Figure 23.Position of tendons relative to columns.

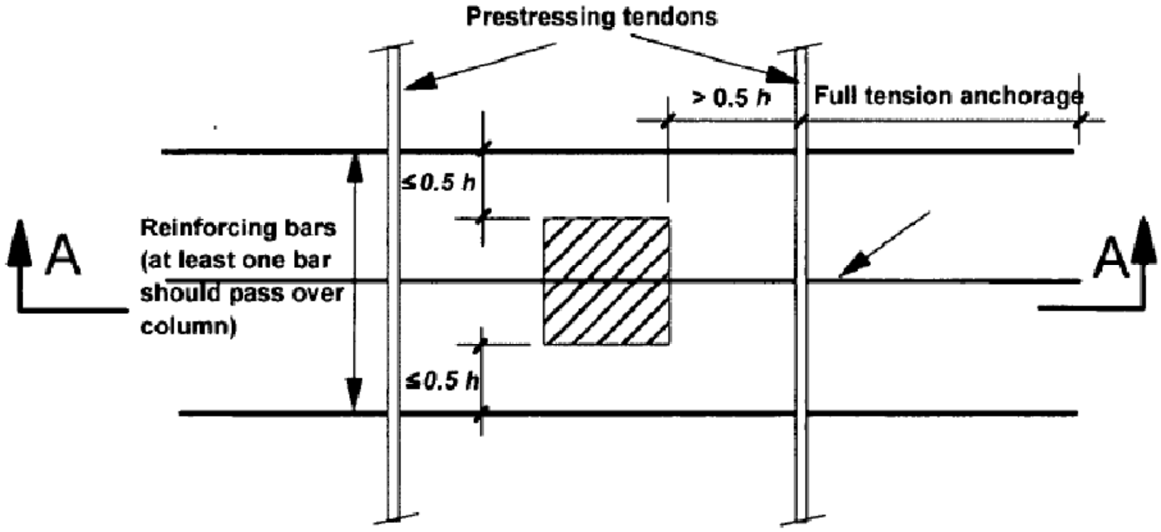
b) TENDON SPACING

For suspended post-tensioned slabs the maximum spacing of uniformly distributing tendons should not exceed **6** x the slab depth for unbonded tendons or **8** x the slab depth for bonded tendons. Where the banded option is used both ways then the maximum spacing between tendons should be restricted to 10 x the slab depth for bonded tendons. Greater spacing may be used where it can be shown that the slab is capable of spanning between the tendons and supporting the applied design loads.

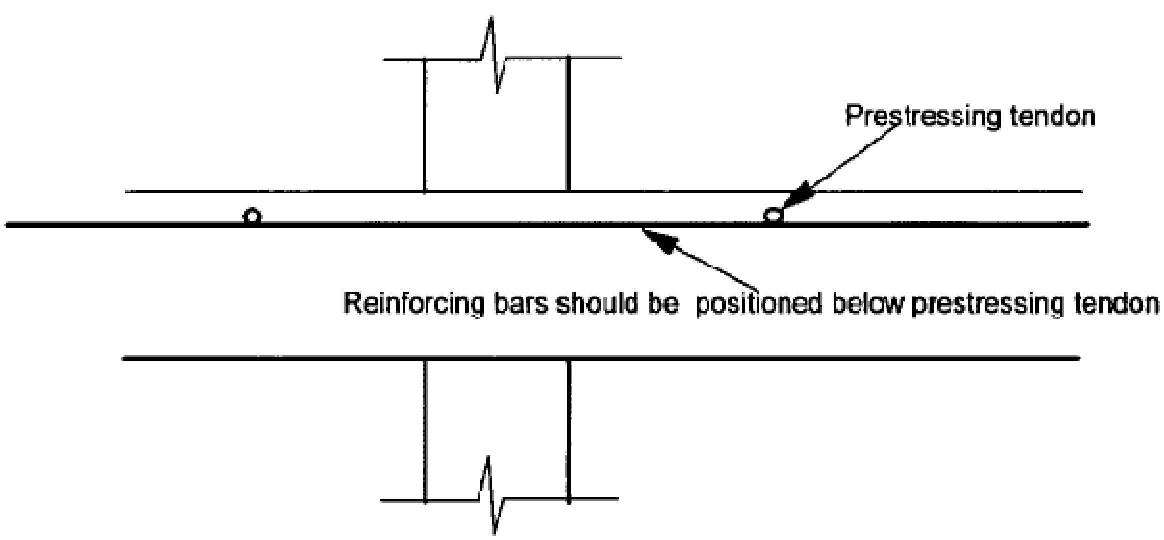
Unbonded tendons may be placed in groups if required. It is recommended that grouped tendons are laid side by side and do not exceed four tendons per group.

The minimum horizontal distance between ducts or groups of tendons should be the greater of 75mm or the group/duct width.

Should it be necessary to arrange the tendons in vertical layers in beams or ribs, and then it is recommended that the gap between the layers should be at least the vertical dimension of the tendon or duct. In the case of bonded tendons where oval metal ducts are used, it is recommended that their positions are staggered to ease the placing of concrete.



Plan view



Sectional view A-A

Additional reinforcement required where tendons are not within 0 .5h from the column.

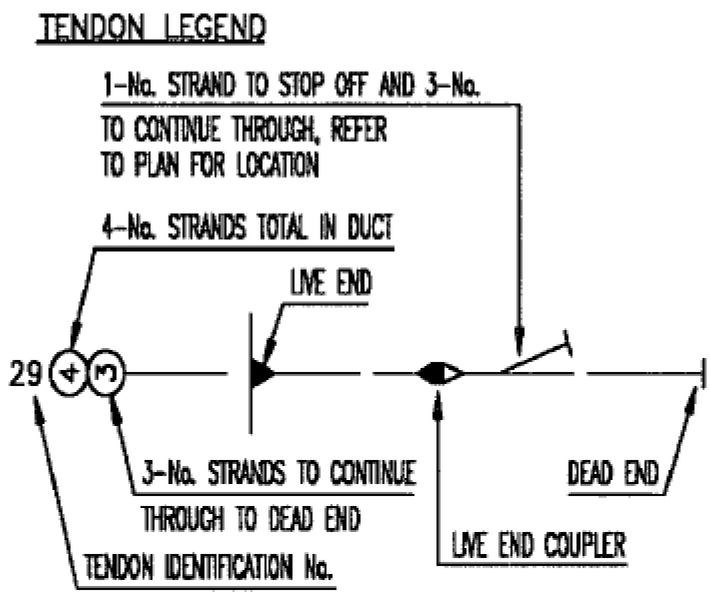
If tolerances on tendon positions are not stated, the values in the below table are taken.

Table 3: Tolerances on tendon positioning.

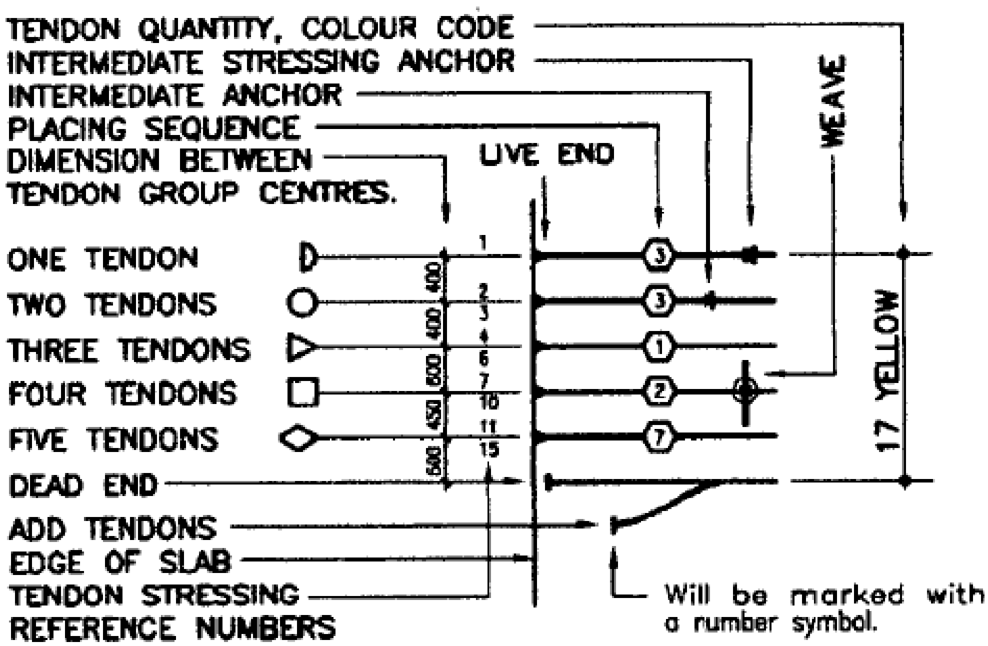
Slab thickness	Tolerances	
	Vertically	Horizontally
$h \leq 200$mm	$\pm h/40$	± 100mm
$h > 200$mm	± 5mm	± 100mm

c) TENDON NOTATION

Typical notation of tendons on drawings is shown in Figure. It is recommended that this legend figure is included on all tendon layout drawings



. Typical notation for use on tendon layout drawings.

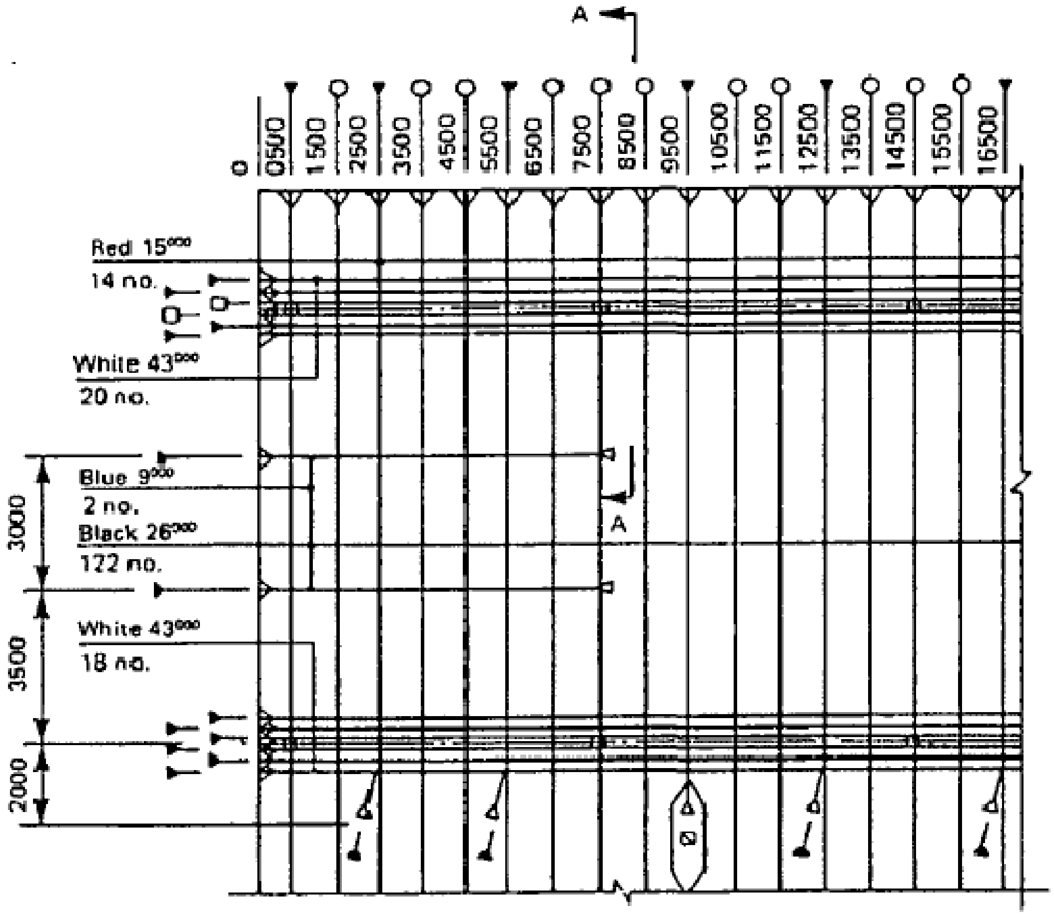


NOTE

When more than one symbol appears on a tendon group, the number of tendons will equal the sum of the symbols.

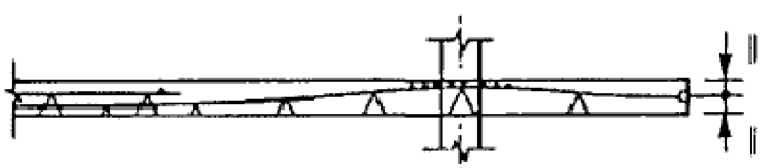
Typical method for unbounded tendons.

Figure 24. Typical notation for use on tendon layout drawings.



Tendon layout

Figure 25. Placing sequence.



Section A-A

Figure 26. Flat slab tendon layout.

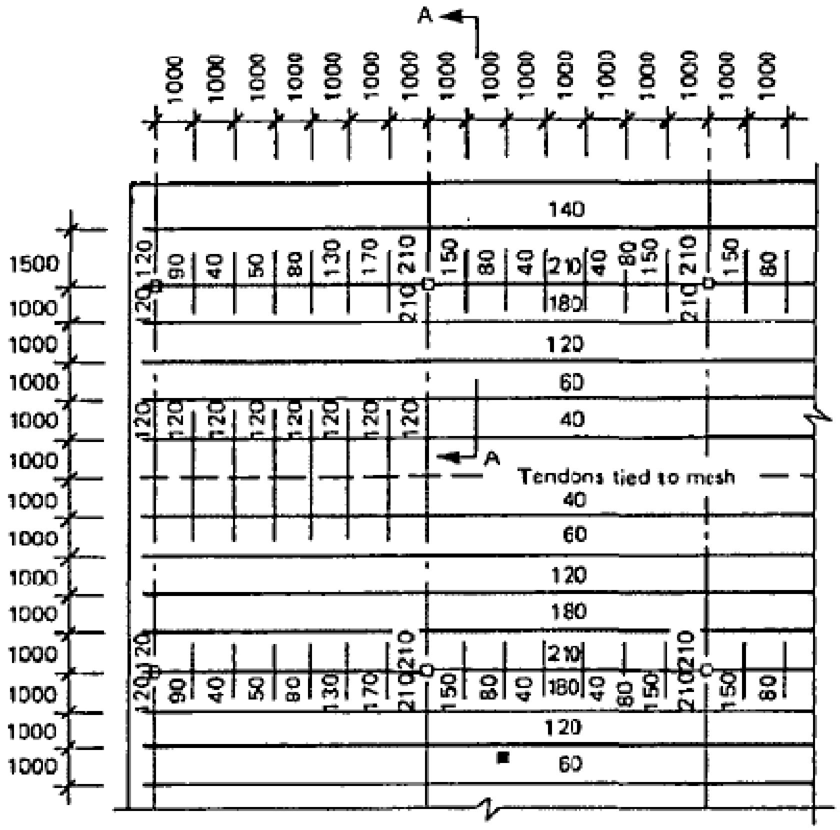


Figure 27. Typical tendon profile and support layout for slab.

Note

1. *Height given is from soffit of slab to underside of tendon.*
2. *Diameter of support bars is 10mm*

Flat slab tendon and support layout detailing.

Tendon profiles in the longitudinal and transverse directions are shown using an exaggerated scale for the vertical dimensions. These are usually given from the soffit of the slab to the centre line of the dud sheath and are plotted at intervals of lm. Closer centres may be necessary for sharp vertical curves. For ease of placement on site, shop drawings are detailed giving the vertical tendon position from soffit to underside of tendon.

d) TENDON SUPPORTS

The profile of the tendons is critical to the floor performance. It is therefore recommended that the support centres do not exceed 1 m. For ribbed slabs or beams, support bars can be adequately held by firm wire ties. Welding is not recommended and should be only used with extreme care. Figure shows a typical support bar layout.

The actual layout may be modified by the contractor depending on the support system adopted, so that the specified tendon profiles are attained and adequate support is provided.

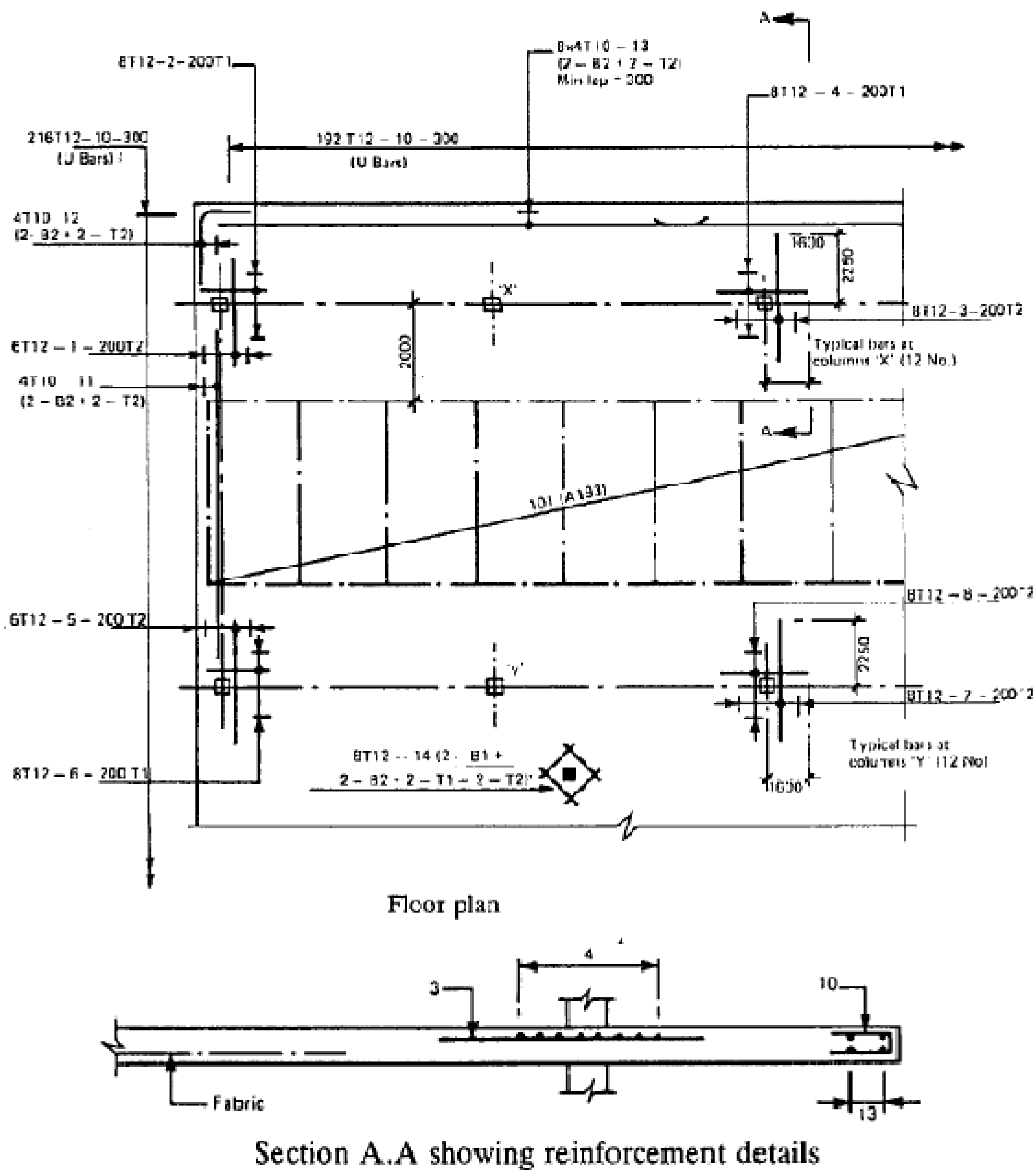


Figure 28.Flat slab reinforcement layout

12. STRUCTURAL SYSTEMS

The three most common floor systems used for building structures such as offices, shopping centres and car parks are the flat plate, flat slab and banded slab. For high rise construction a fourth system is widely used which consists of band beams at relatively close spacing spanning from the building perimeter to the service core.

Although economy of each of these depends primarily on the span and applied load, it is generally true to say that a band beam scheme is cheaper than a flat slab which in turn is cheaper than a flat plate.

To illustrate this analysis was carried out on a structural scheme for each of the three systems to show the percentage cost of each structural component. The schemes were based on a column grid of 8.5 m and imposed load of 5 kPa. A total relative cost figure, also shown, is obtained by multiplying each structural element by its cost rate. This rate varies from country to country, but the trend will remain unchanged. Refer to table 2 below.

FLOOR SYSTEM	Flat Plate	Flat Slab	Banded Slab
Concrete	25	24	24
Reinforcement	6	6	8
Post-tensioning	26	23	20
Formwork	43	47	48
TOTAL	100%	100%	100%
Relative total cost	1	0.97	0.96

Table 9. Percentage Floor cost

Post-tensioning is not limited to simple flat slabs and the range of structural types which can be economically stressed is almost limitless. Some of the most common floor systems are presented below along with recommended concrete sizes and span to depth ratios.

Note that the span to depth ratios given depend on the element being an internal bay, end bay or simple span. Figure explains the differences.

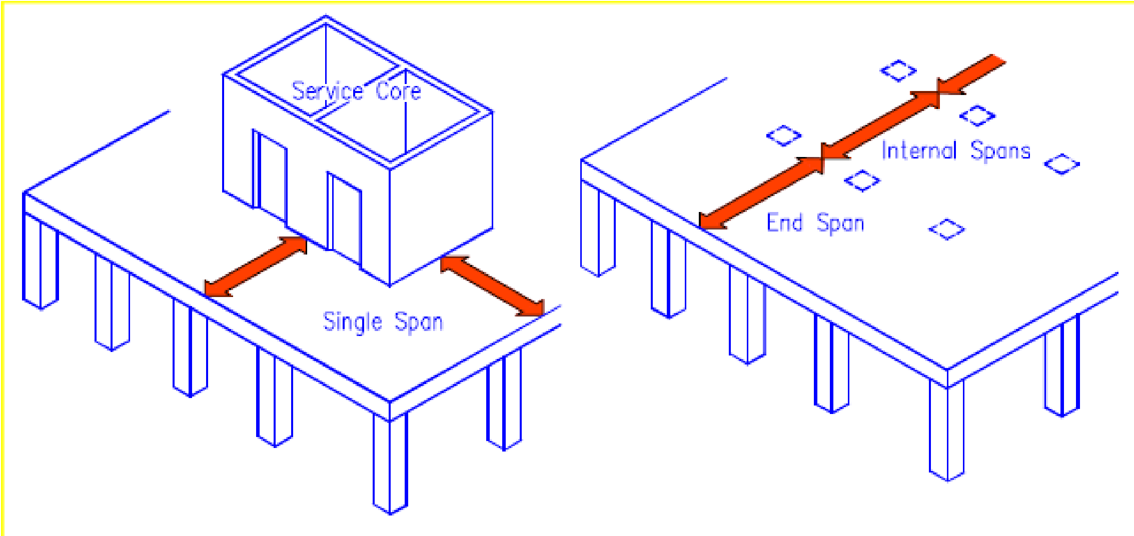


Figure 29.Examples of single spans, end spans and internal spans

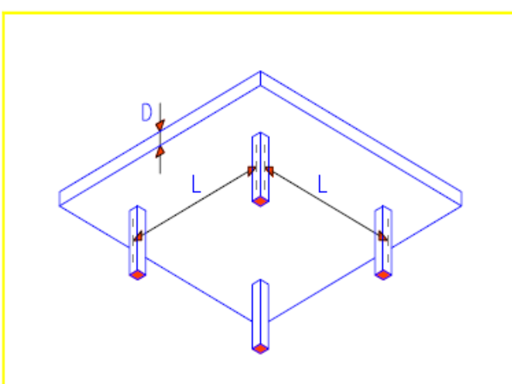
It should be noted that although often only of secondary consideration, the choice of the system can depend upon factors other than column spacing, imposed loads and economics. Headroom clearance for services or a flat soffit for aesthetics is two other possible governing criteria.

a. FLAT PLATE

This system is commonly used in Sydney for high rise residential construction where the span is usually 7 to 8 metres. The most attractive feature of this floor system is its flush soffit which requires simple formwork and greatly simplifies construction.

The depth of a flat plate is often dictated by shear requirements. Thinner slabs or longer spans can be constructed if column capitals or shear heads are employed.

Used	Where spans are similar both directions
Economic Span	Range 7.0 to 9.0 m
Imposed Loads	Up to 7.5 kPa



	Imposed Load (kPa)	Span/Depth Ratio
Single Span	3 5 10	33 31 28
End Span	3 5 10	39 36 32
Internal Span	3 5 10	45 42 38

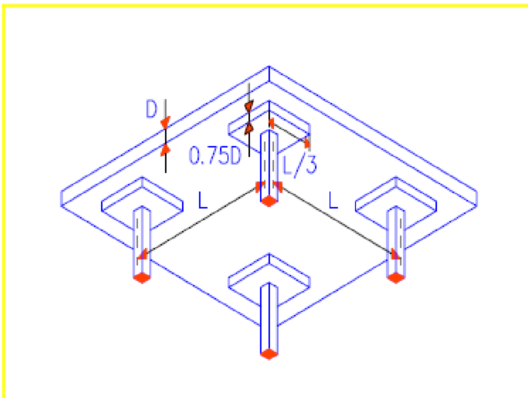
Figure 30. Flat Plate

b. Flat Slab

A widely used system today for many reasons - flat soffit, simple formwork and ease of construction, as well as flexibility for locating services. The economical span range over a flat plate is increase by the addition of drop panels.

The drop panels increase the flexural stiffness of the floor as well as improving its punching shear strength. This system provides the thinnest floors and can lead to height reductions and substantial savings in facade costs.

Used	Where spans are similar both directions
Economic Span	Range Up to 13.0 m
Imposed Loads	Up to 10.0 kPa



	Imposed Load (kPa)	Span/Depth Ratio
Single Span	3	38
	5	35
	10	32
End Span	3	46
	5	43
	10	40
Internal Span	3	52
	5	49
	10	45

Figure 31. Flat Slab

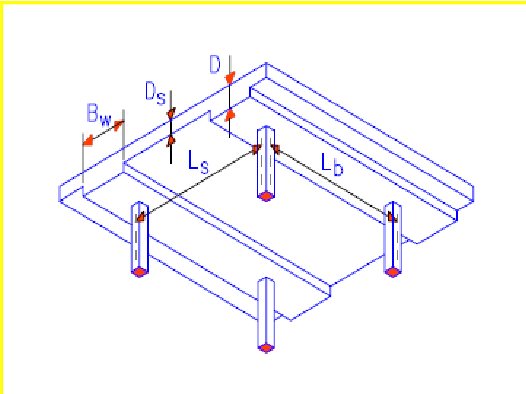
c. BANDED SLABS

This system is used for structures where spans in one direction are predominant. It is also a very common system due to minimum material costs as well as relatively simple formwork. In most circumstances the width of the band beam is chosen to suit the standard sizes of the formwork. The sides of the band can be either square, or tapered for a more attractive result.

The band beam has a relatively wide, shallow cross section which reduces the overall depth of the floor while permitting longer spans. This concrete section simplifies the formwork and permits services to easily pass under the beams. The post-tensioned tendons are not interwoven leading to fast installation and decreased cycle time.

The band beam system has another advantage which is not widely appreciated. In most circumstances depending on the actual geometry of the cross section the beam can be considered as a two way slab for fire rating and shear design. This enables considerable economies to be achieved in both post-tensioning and reinforcement quantities.

Used Span	predominant in one direction
Economic Span	Range Band Beam: 8.0 to 15.0 m
Slab: 6.0 to 10.0 m	Imposed Loads Up to 15.0 kPa



		Imposed Load (kPa)	Span/Depth Ratio
Slabs (Ls)	Single Span	3	38
		5	35
		10	32
	End Span	3	46
		5	43
		10	40
	Internal Span	3	52
		5	49
		10	45
Band Beams (Lb)	Single Span	3	20
		5	18
		10	16
	End Span	3	24
		5	22
		10	19
	Internal Span	3	27
		5	25
		10	22

Figure 32. Banded slabs

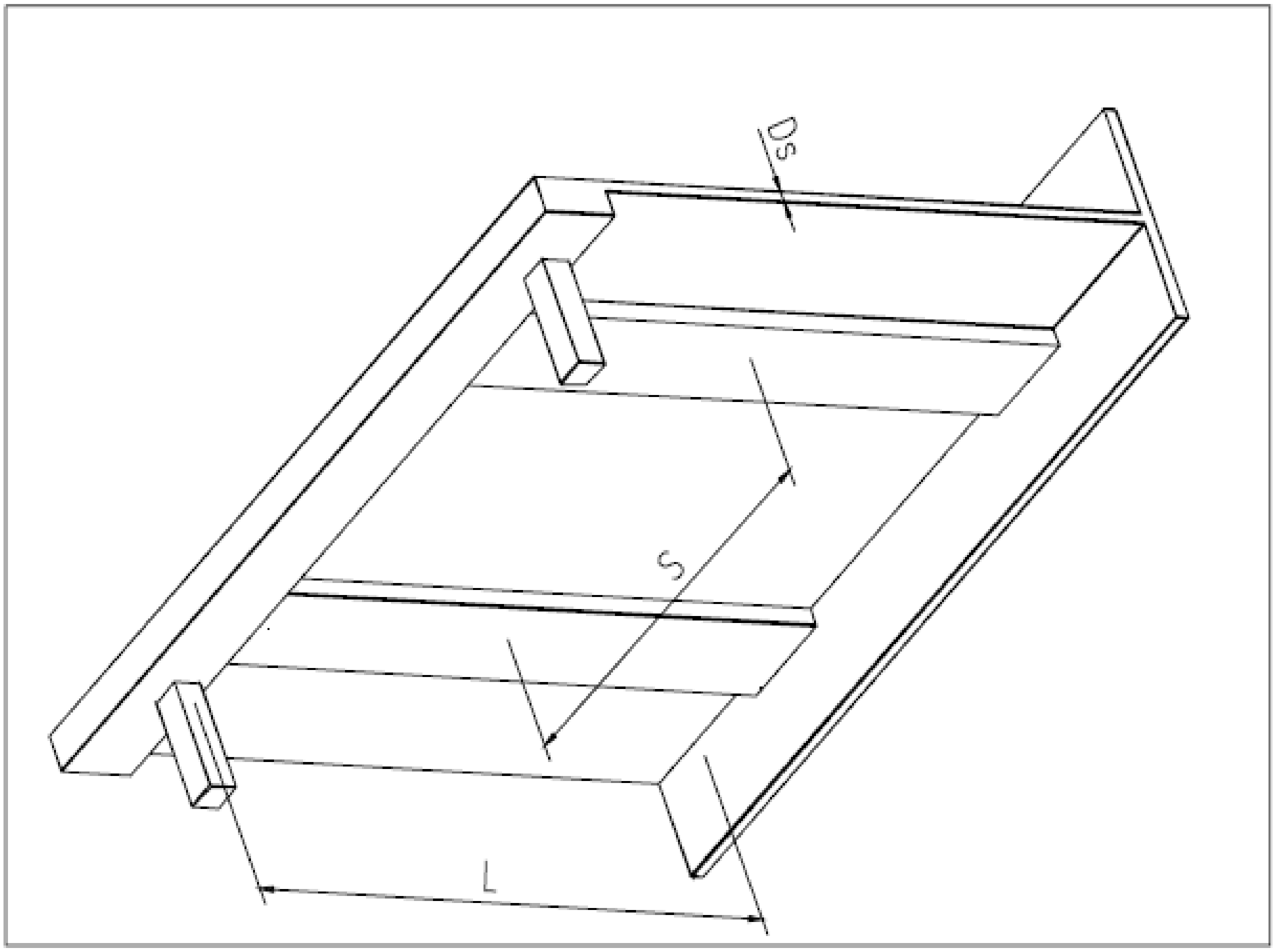
d. HIGH RISE BANDED SLABS

This system has gained favour over the past 10 to 15 years for high rise construction and consists of band beams at relatively close centres spanning between a perimeter beam and the service core. The system suits system formwork due to the amount of re-use in high rise construction.

Services may either pass under the shallow bands or, alternatively, pass under service 'notches' in the band soffit.

For clear slab spans in excess of 4.5m the use of post-tensioning is economical perpendicular to the bands and assist in reducing the weight of slab carried by the bands.

Used	Long span high rise construction
Economic Span Range	Band Beam: 9.0 to 15.0 m
Imposed Loads	Up to 7.5 kPa



	Slab Span	Band Width	Span / Depth Ratio
Band Beam (L)	S= 4800, Ds= 120 RC	bw= 1000	22
	S= 6000, Ds= 120 P-T, 150 RC	bw= 1500	20
	S= 8400, Ds= 160 P-T	bw= 1800	18

Figure 33. High Rise Banded Slabs

13. CONSTRUCTION DETAILS

a) EXTENT OF POURS

The size of pour is limited by:

- the amount of early thermal shrinkage that will take place. This is linked with the amount of un-tensioned reinforcement provided.
- the ability to complete the pouring and finishing of the surface of the concrete.
- Friction losses. With bonded tendons, these usually restrict the length of single-end stressed tendons to 35m, and double-end stressed to 65m for a slab and 25m and 50m, respectively, for a beam. The lower friction values for unbonded tendons extend these values to 45m and loom, respectively, for slabs. Longer lengths are achievable but the friction losses should be carefully considered.

Prestressing tendons may be continuous through construction joints allowing larger areas without any permanent joints. Allowances should be made in accordance with good practice to accommodate temperature variations by the provision of expansion joints on larger slabs.

b) PROTECTION OF ANCHORAGES

Tendons are normally anchored within the middle third of the slab to ensure adequate edge cover to the anchorage. Pocket formers at anchorages should be large enough to allow adequate trimming of the tendons after stressing, thus ensuring good end cover to the strand. Trimming should be carried out using a disc cutter or hydraulic shears. In no circumstances should the tendon be trimmed by flame cutting.

Pocket formers are normally proprietary plastic or polystyrene units that make up part of the anchorage fixings. It is recommended for unbounded tendons that, after trimming the strands, the wedges and the strand end are coated with grease of similar specification to that used in the tendon and that a watertight cap be applied over the coated area. The minimum end cover to this cap should be 25mm.

For bonded tendons after the strand has been trimmed then the anchor is treated with a resin sealer. The pockets for the anchorages are generally sealed with a sand cement dry pack. This has two functions. First it covers and protects the anchor and second it allows the air to be expelled during grouting. Where a pocket are exposed to the elements or for aesthetic purposes then the dry pack is left low and a suitable non-shrink mortar is post-applied after grouting operations are complete. In no circumstances should this mortar contain chlorides or other materials that could be harmful to the prestressing steel.

c) BACK-PROPPING

Back-propping may be required to ensure that the construction loads can be safely carried by the earlier construction stages, and this must be considered by the designer in a similar manner to normal reinforced concrete construction. Stressing sequencing should also be considered as this may induce greater loads. When using propping to resist the uplift forces from carrying cantilever decks check that decks are not overstressed due to load balancing action acting with the upward load from the props.

d) STRESSING PROCEDURE

The stressing forces and sequence of stressing should be specified on the drawings. This has to be planned in such a way that the prestress is applied as uniformly as possible, and that no overloading of the formwork occurs. For systems with banded and uniformly distributed tendons, the banded tendons should normally be stressed first to ensure this is the case. Wherever possible the use of different forces for tendons of the same size should be avoided.

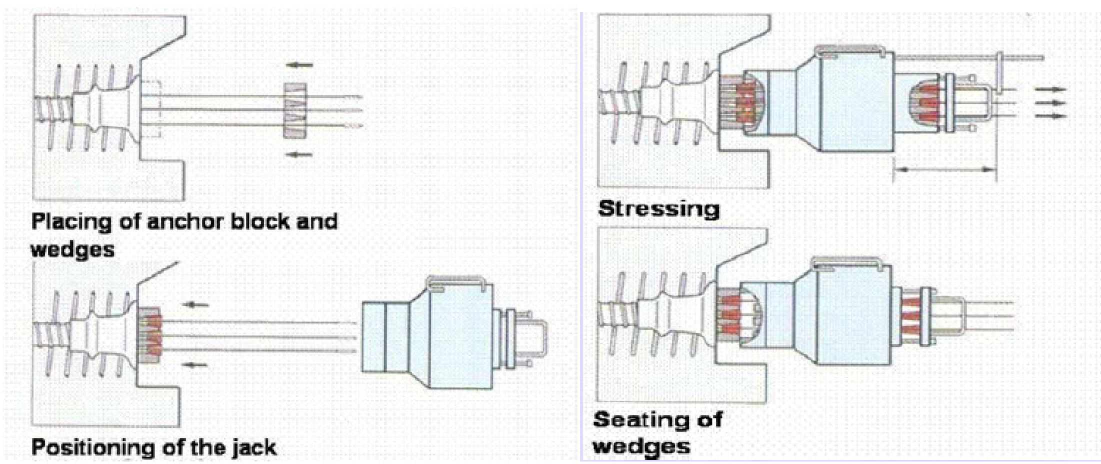


Figure 34. Sequence of anchoring.

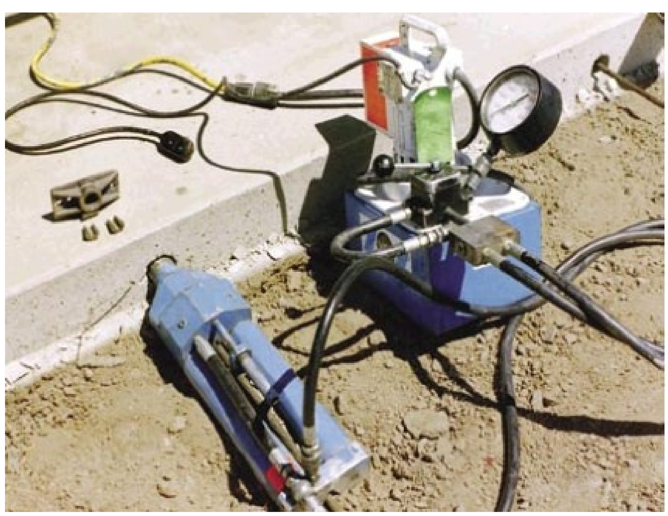


Figure 35: Stressing bonded tendons at slabs.

It should be noted that after stressing a bonded system and before grouting has taken place, it should be considered as an unbonded system. In members where early stressing is desired to reduce the risk of early shrinkage cracking, it is common to stress the tendons in two stages. The first stage is usually about 25% of the final prestress force, and is carried out as soon as the concrete has obtained adequate strength for the anchorage being used. This concrete strength is typically between 10 and 15MPa. It is important that sufficient site-cured cubes or cylinders are provided to determine the transfer strength (this is especially important in periods of cold weather). Where a slab or system of secondary beams is stressed across primary beams attention must be given to the sequence of stressing in order to avoid damage to the formwork of the primary beams.

e) GROUTING

Protection of the PT system is essential. The protection of the post-tensioning system is directly related to the overall durability and life span of the structure. The concrete that forms the structural member, the duct and the grout are all that separates the PT system from exposure to the air and weather. The high strength steel of the PT system is highly susceptible to corrosion and must be protected from exposure to moist and salt laden air. Additionally, the grout provides transfer of the prestressing force to the structural member in a bonded post-tensioning system.
The bonding of the grout to the post-tensioning steel accomplishes this in a manner that is similar to the bond developed by rebar in concrete.

Grout is the primary protection for the PT system; therefore, careful attention must be given to the grouting process. Cementitious grout provides an alkaline environment that passiveness the surface of the steel which inhibits the corrosion process. The grout must surround and be bonded with the steel to be effective. Specially blended grout materials, correctly mixed and effectively pumped into the tendon, are key to a successful grouting operation. The durability of the structure is directly affected by the quality of the grouting. It is essential that an adequate grout plan be developed and executed. The grout plan is an outline of the grouting operation from start to finish and will be discussed at length later.

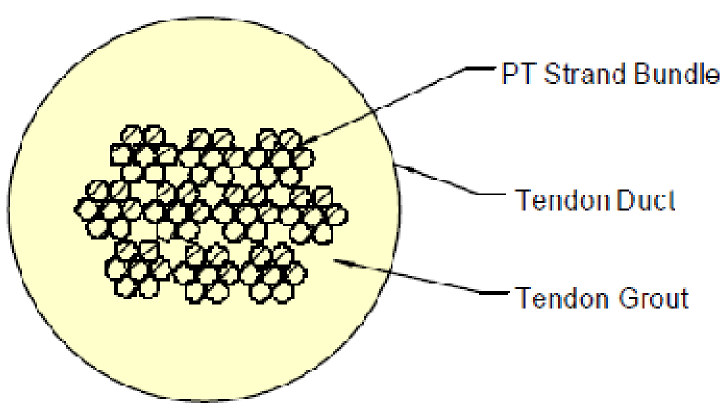


Figure 36. Duct, Grout and PT steel system.

Figure 37. Grouting Equipment.

f) **Grout Pipes**

The grout inlet pipes and outlet pipes are attached to the ducts. This is where the grout is pumped into the tendon and where air and excess water are expelled as the duct fills with grout. These pipes are attached to the ducts prior to placing concrete for internal tendons. They are attached to the external ducts during their assembly at the construction site.

Figure 38. Grouting Pipe.

g) SOFFIT MARKING

Tendon positions in flat slabs are not always apparent on completion of concreting. Recent practice has been to introduce soffit marking, where the cover to the tendon is less than the penetration of ceiling and service fittings. Unpainted zones indicate no tendons. Dark zones indicate tendons near the soffit and colored zones indicate tendons near the top of the slab.

Figure 39: Soffit marking used to indicate tendon position & arrangement..

14. FUTURE FLEXIBILITY AND PENETRATIONS

Post-tensioned floor slabs in Australia are now universally regarded as the most cost effective form of construction for shopping centres, office buildings, and carparks where spans exceed 7.5 metres. The preferred post-tensioning system used is the well proven `bonded' tendon utilising from 3 to 5 individual prestressing strands housed in oval ducting and anchored in flat fan shaped anchorage castings.

A question often asked of post-tensioned slab systems is what happens if we wish to make a penetration in the slab after construction.

From time to time it has been brought to our attention that certain members of the building profession see this question as a major obstacle and are reluctant to accept the use of prestressing in some types of buildings. This is often due to a perceived lack of flexibility in the structure when it comes to the formation of openings through the slabs some time after construction.

This section will outline the options available to enable the designer to produce a building which is both economic to construct and easy to modify in the future.

Planning's for openings

1. Possible Future Requirements

There is no doubt that during the lifetime of a structure the requirements of a tenant may alter with time or the tenant may change several times. Each new tenant will have his own requirements for mechanical, hydraulic and electrical services, as well as loading arrangements and general layout.

Therefore, for a building to remain readily gettable in the future it must have the flexibility to accommodate openings for stairs, services or lifts, and the possibility for changes in loading patterns.

2. Choice of Building System

No building system can be infinitely flexible in terms of future tenant requirements. Whether a slab is constructed from post-tensioned concrete, pre-cast concrete, structural steel or insitu reinforced concrete there will be certain areas such as main beam strips where holes cannot be accepted without significant difficulty.

3. Overall Structural adequacy

Whatever the building system and material, it is important to locate the new opening with due regard to the structural stability of the remaining portion of slab. The Structural Engineer must check the penetration location and advise any remedial work required, such as trimming beams, regardless of the structural system.

A post-tensioned slab will have similar strengths and weaknesses in terms of its flexibility as insitu reinforced concrete. For example, if a new stair well were to be located in the centre of a panel such that short cantilever slabs were retained on all sides to counterbalance the adjacent bays, it is quite possible that no additional supporting members would be required.

A precast system or steel framed structure may require the removal of the whole panel, construction of trimmer beams around the new void, and replacement of the slab to the surrounding edges.

Designing Post-Tensioned Slabs for Future Openings

1. Locating post-tensioning tendons

For post-tensioned slabs and beams where tendon positions may not be readily identifiable, soffit marking can be employed. Prior to casting the slab, stainless steel staples are use to secure the ducts to the formwork. When the formwork is struck, the position of the tendons is obvious, especially if the staple lines have been linked by painted lines. Alternatively, chalk lines can be marked on the slab top surface to aid in the locating of post tensioning tendons. This procedure will assist in locating openings away from tendons.

2. Structural systems in post-tensioned concrete

a) Band Beam and Slab

For rectangular grids the band beam and slab solution may be appropriate. This is the system typically used for shopping centres and car parks due to the economic benefit and relative insensitivity to floor height restrictions.

Normally band beams span in the long direction and impose the same constraints on whole placement as would a steel or reinforced concrete beam. However, small hydraulic type penetrations (approximately 150 mm diameter) can usually be accommodated without the need for remedial action.

The slabs however, are usually quite lightly prestressed with tendons in one direction only at approximately 1500 mm centres. Reasonable size openings or large slots are therefore easy to accommodate without the need to cut post-tensioning tendons.

b) Flat Slabs and Flat Plates

For structures requiring minimum floor to floor height and regular grids the two-way post tensioned flat slab is usually the most cost effective solution.
The normal installation procedure would concentrate the tendons into 'column strips' along the column grids at approximately 600 mm centres with tendons away from the column strip at approximately 1400 mm centres.

Consequently small holes for services could be located without the need to cut tendons. Using this structural system it is possible to leave the central panel as traditionally reinforced and designed as a `soft zone' to easily accommodate large openings. The cost penalty for the extra reinforcement required would need to be offset against the perceived benefits.

c) Ribbed and Waffle Slab

Larger grids or heavier loads may dictate the introduction of ribs spanning either one-way or two-way (waffle) depending upon the aspect ratio of the grid.
Rib spacing for post-tensioned slabs are generally larger than for reinforced concrete, being typically 1.2 to 1.5 metres. Consequently small to moderate holes can easily be cut through the topping slab without disturbing the ribs. Indeed with tendons confined to the ribs their location is readily identifiable, assisting in the sitting of the openings.

Cutting of Tendons

1. Bonded Tendons

Bonded tendons are located within oval shaped galvanised ducts which are injected with cement grout following the post-tensioning procedure. Consequently when such a tendon is severed, the free end will become de-tensioned but after a short transmission length the full tendon force will be effective. This distance is in the order of 800 to 1000 mm.

Present quality assurance methods and supervision ensure that the tendons have been adequately grouted after the application of prestress.

If a penetration is required that will need the termination of a bonded tendon, then the procedure follows that for a fully reinforced structure.

Cutting a bonded post-tensioned tendon is, structurally, the same as cutting through conventional reinforcement. The tendon, however, needs to be `terminated' in order to give full corrosion protection (as does conventional reinforcement).

Tendons are easy to cut using a disc cutter. In fact, cutting tendons requires less effort than for a fully reinforced slab due to the relative amount of reinforcing material to be cut.

2. Unbonded Tendons

These tendons come individually greased and plastic coated and are therefore permanently de-bonded from the slab.

When unbonded tendons are severed, the prestressing force will be lost for the full length of the tendon.

When contemplating the cutting of an unbonded tendon it is therefore necessary to consider the aspects as noted below.

a) Cutting the tendons.

The strand is packed with grease which prevents an explosive release of energy when the tendon is severed. Even so a gradual release of force is recommended. This can be achieved by using two open throat jacks back to back. After cutting the strand the force can be gently released by closing the jacks.

b) Propping the slab.

Adjacent spans may require temporary propping depending upon the number of tendon severed at one time. It is rare for a slab to carry its full design load. A design check based on actual loading at the time of the modification may show props to be unnecessary.

c) Forming the hole.

When the edge of the slab is re-concreted new anchors are cast in to enable the remaining lengths of tendon to be re-stressed, thus restoring full structural integrity. The above operations are not difficult but will require the expertise of a post-tensioning sub-contractor.

15. DEMOLITION OF POST TENSIONED STRUCTURES

In the case of post-tensioned structures using bonded tendons, demolition can be carried out using techniques similar to those used to demolish reinforced concrete structures. Due to its induced compression the concrete is significantly harder and whilst tendons are made from high tensile strand there is considerably less steel to cut and generally concrete sections will be thinner than comparable reinforced concrete structures.

Only in the case of transfer slabs or beams, which have been progressively stressed, must extra precautions be taken to avoid upward bursting of concrete as the self weight of the structure above is progressively removed.

The cutting of unbonded tendons may result in dramatic collapse of a structure, but properly considered, can be used to advantage, enabling rapid demolition of large areas as the force in the supporting tendons is released.

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16. SUMMARY

It is not uncommon for post-tensioning to be rejected in certain types of building project due to a perceived lack of flexibility. This, in the majority of cases, is based more on a fear of the unknown than on sound technical knowledge.

With a little forethought it can be seen that post-tensioning need not mean a dense mat of tendons in all directions. Tendons are usually spaced sufficiently far apart to allow penetrations of reasonable size to be made later, without cutting through the tendons.

Where there is a reasonable possibility that a penetration may be required in the future, slabs can be built with `soft zones' to allow later perforation by voids without cutting tendons.

Should it be necessary to cut tendons this can easily be achieved using well established methods and in short, whilst the modification of a post-tensioned slab may require more planning than other forms of construction, its use will present the client with a building which is both economical to construct and flexible for its life.

17. CONCLUSION

In conclusion it is worthy to reinforce a few key points.

There is a definite trend towards large spans in buildings due to the fact that there is now more emphasis on providing large uninterrupted floor space which can result in higher rental returns. Post-tensioning is an economical way of achieving these larger spans. For spans 7.5 meters and over, post-tensioning will certainly be economic and, as the spans increase, so do the savings.

The most significant factor affecting the cost of slab system post-tensioning is the tendon length. Other factors create a scatter of results leading to an upper and lower bound. Not with standing this, it is always advisable to obtain budget prices from a post-tensioning supplier.

The main structural schemes available are the flat plate, flat slab and banded slab, with the latter generally leading to the most cost-efficient structure. However, other factors such as floor to floor heights, services, etc., must be taken into account in the selection of the floor structure. For high rise construction and highly repetitive floor plates, the use of more specialized structural schemes is appropriate with emphasis on systems formwork.

It is not uncommon for post-tensioning to be rejected in certain types of building project due to a perceived lack of flexibility. However, tendons are usually spaced sufficiently far apart to allow penetrations of reasonable size to be made later, without cutting through the tendons. Should it be necessary to cut tendons this can easily be achieved using well established methods.

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