POST-TENSIONING MANUAL

SIXTH EDITION

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Efforts to develop the 6th Edition began in 1997. The manual has come to fruition thanks to the efforts of hundreds of people. This eight-year effort involved contributions from all segments of the post-tensioning industry. Key contributors are identified below; however, there are many others—too numerous to list—who assisted in this project.

The Editors

Because of the extended development time for this manual, editorial responsibilities were shared by several individuals. Dr. Pawan R. Gupta, PTI Technical Director, was the chief editor for the final published version. He was preceded by Dr. Fadi Alkhairi, former PTI Technical Director, Larry Krauser, DYWIDAG-Systems International USA, Inc. and Juergen Plaehn (retired).

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The Reviewers

An extensive review process was used to ensure the technical accuracy and quality of the manual. For each chapter, the author prepared a draft that was submitted for several rounds of review. The first level of review was provided by staff and selected individuals with expertise relative to the particular subject matter. The edited draft was then reviewed by the Technical Advisory Board (TAB). The TAB-approved draft was then reviewed and edited by a professional writer, Ms. Coleen Bondy. The finalized chapter drafts were then reviewed by staff and an editorial task group to ready them for printing.

PTI is particularly indebted and would like to express its appreciation to the members of the Technical Advisory Board who put in countless hours in the review of this manual. Many corrections and technical enhancements resulted from these reviews. Without the hard work and dedication of the TAB members listed below, this manual would not have been possible.

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TABLE OF CONTENTS

INTRODUCTION

1.1 HISTORY

The first patent for prestressed concrete was issued to P.H. Jackson of San Francisco in 1886. However, modern development of prestressed concrete is usually attributed to Eugene Freyssinet of France. In 1928, Freyssinet began to use high-strength steel wire for prestressing concrete.

Earlier attempts at producing prestressed concrete using normal-strength reinforcement had been unsuccessful. After being precompressed, concrete continues to shorten with time in a process called creep. Shrinkage of the concrete due to loss of moisture over time also causes shortening. Creep and shrinkage together may cause the concrete to shorten nearly 0.1%.[1.1] With lower-strength reinforcement, it is not possible to elongate the steel in the prestressing operation by more than about 0.15%. Consequently, when lower-strength reinforcement is used to prestress concrete about two-thirds of the prestressing in the reinforcement is lost due to shortening. High-strength steel wire, on the other hand, can be elongated about 0.7% in the prestressing operation.[1.1] Even with the losses due to creep and shrinkage, over 80% of the prestressing remains. To further reduce losses due to creep and shrinkage and to make possible much higher levels of precompression, Freyssinet also recommended the use of higher-strength concrete.

By 1939, Freyssinet had designed conical wedges for anchoring the wires at the ends of prestressed members as well as special jacks for use in stressing and anchoring the wires. In 1940, Professor Gustave Magnel of Belgium developed a system of curved, multi-wire tendons in flexible rectangular ducts. Further development of post-tensioning was interrupted by World War II. However, the shortage of steel in the post-war years gave impetus to the use of prestressed concrete in replacing the war damaged bridges throughout much of Europe. Although France and Belgium led the development of prestressed concrete, England, Germany, Switzerland, Holland, Russia, and Italy quickly followed.[1.2]

The first use of post-tensioning in the U.S. was on the Walnut Lane Bridge in Philadelphia in 1949. This landmark bridge had precast girders post-tensioned with the Magnel system. The first post-tensioning in U.S. building construction was in the mid to late 1950s in buildings using the lift-slab construction method. The early development of the prestressed concrete industry in North America was predominantly oriented toward factory production of precast-prestressed elements for highway bridges. There were, however, many notable exceptions where post-tensioned prestressed concrete construction was utilized. In the 1960s, post-tensioned box girder bridges were widely used in California and other Western states. During the same period, the use of unbonded tendons for building floor systems became more widespread. The use of post-tensioned nuclear containment also began in the 1960s. The 1970s saw the emergence of new applications, including the use of post-tensioned foundations for single and multi-family residences on expansive and compressible soils, and the use of prestressed rock and soil anchors for a variety of tie-back and tie-down structural applications.

1.2 USAGE

As a result of these new markets, and a growing awareness of the advantages of post-tensioning among engineers, architects, contractors and owners, the use of post-tensioning increased more than five-fold in the period 1965-1985. Since 1985, post-tensioning usage has continued to grow at a rapid pace, averaging 8.5% annual growth as shown in Fig. 1.1.

Fig. 1.1 Post-Tensioning Shipments in North America: 1986 to 2004[1.3]

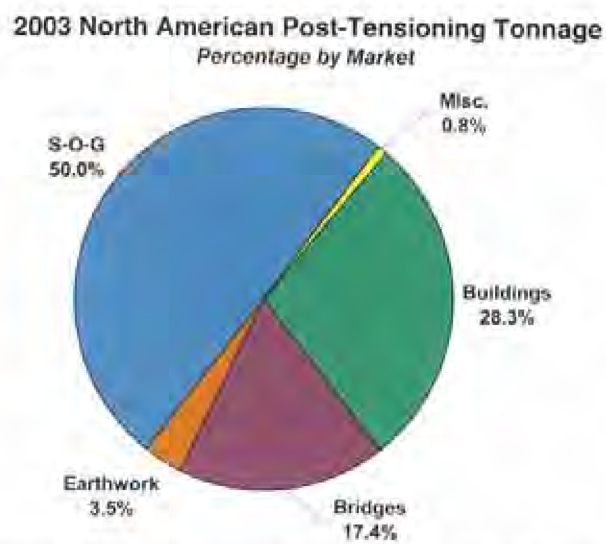

Fig. 1.2 2003 Post-Tensioning Usage in North America Broken Down by Application[1.3]

In the last 50 years, prestressed concrete has grown to be a multibillion-dollar industry in North America and is used in many different construction applications. Fig. 1.2 shows the relative usage of post-tensioning by market segment.

1.3 STATE-OF-THE-ART DEVELOPMENTS

Over the years, there have been a number of significant technological developments that have helped advance the state-of-the-art of post-tensioning and have contributed to its continued growth.[1.4,1.5] These developments include:

- Introduction of strand systems
- Development of ductile iron castings for single-strand tendons
- Introduction of the "load-balancing" design method
- Introduction of "banded" tendon layout for 2-way slab systems
- Segmental bridge construction
- Use of computers for analysis and design
- Formation of the Post-Tensioning Institute
- Improvements in corrosion resistance

In the United States, early unbonded post-tensioning systems used high strength stress relieved steel wires, bars, or strand. The button-headed tendon system used ¼ inch wires bundled together, greased, and wrapped with Sisal Kraft paper as sheathing. The wires ran through a stressing head and were "button headed" to anchor them. Machinery was used to cold-upset the ends of the wires to create the button-head anchors. The post-tensioning elongation was held with shims or a threaded nut; Fig. 1.3.

The button-headed tendon system had two major problems. The first problem was that the tendons had to be an exact length. Any deviation between the tendon length and the length between forms required either a new tendon or moving the edge forms before pouring the concrete. Second, because the shims and the stressing washer ended up on the outside edges of the constructed slab, they had to be covered with a second concrete pour.[1.6]

Another early unbonded tendon system utilized solid bar with diameters of ⅞ inches to 1⅜ inches, greased and wrapped with a Sisal Kraft paper.

The first strand tendon system—developed by Edward K. Rice and others at Atlas Prestressing Corp.—used ½ in. seven-wire prestressing strand and an anchorage assembly manufactured of coiled wire and a plate and anchored with two half wedge chucks; Fig. 1.4. The strand was also greased and Kraft paper wrapped.

The strand system was much more economical than the button-headed tendon system and eliminated all of its major construction drawbacks. By 1970, virtually all post-tensioned tendons supplied for building construction were strand tendons.

Following the Anchorage Alaska earthquake in 1964 in which there were a number of post-tensioning system failures, a ductile iron casting was introduced as a substitute for the coiled wire anchorage. By the mid 1970s, single strand tendons with ductile iron castings and wedge chuck anchorages were used for most unbonded post-tensioning applications.

Corrosion has been the biggest problem faced by the industry. When some of the early post-tensioning systems began to exhibit corrosion problems, it was realized that some tendon sheathings and coatings could not adequately resist corrosion in the most aggressive environments.

Early sheathing of the strand was crude at best. The tendons were greased by hand, and then wrapped by hand in a spiral fashion with Sisal Kraft paper so the tendon would not bond to the concrete. This was a labor intensive

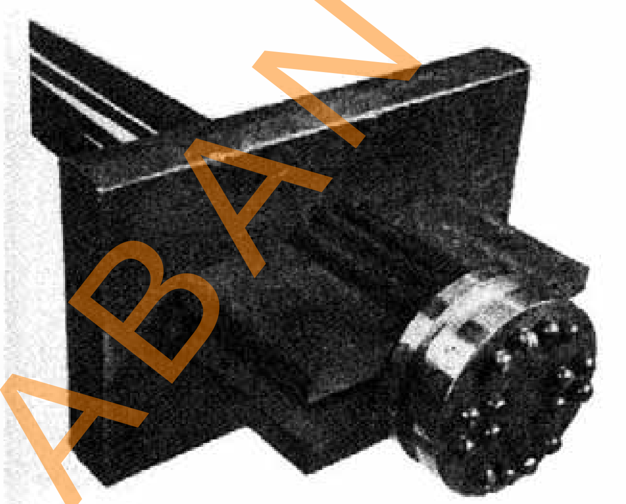

Fig. 1.3 Button Head Anchorage

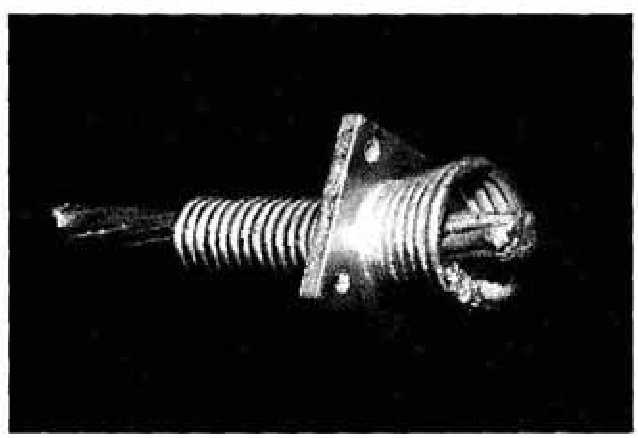

Fig. 1.4 First Strand Tendon Anchorage Developed by Edward K. Rice

process, and the paper wrap was not waterproof and did not provide long term protection for the steel.

Another sheathing system consisted of a semi-rigid plastic tubing. The tubing was laid in a trough and strand from a reel was run through a greasing device and then pushed through the tubing. This "push-through" system proved to be very susceptible to corrosion and failure. Often, the plastic tube sheathing on the tendon would get damaged and water would enter the space between the tube wall and the strand. Although the grease was intended to fill the annular space between the strand and the wall of the tube, with the push-through process it never completely filled the voids. As a result, it was virtually impossible to keep water from entering the tendon at some point in the construction process.

Later a "cigarette wrapped" plastic sheathing was developed. In this process, a machine took strand from a reel, ran it through a greasing apparatus, and rolled a 40 mil thick ribbon of plastic sheet around the strand. The edges of the rolled plastic ribbon were then heat sealed with a flame to form the completed sheathing. This greatly reduced the labor cost of producing single-strand tendons; however, this equipment was very operator sensitive. The heat sealed lap joint of the sheathing often failed; particularly when the tendons were installed in cold weather.

In the 1980s, the current day system of extruding plastic sheathing around the greased strand was developed. This eliminated the lapped joint problem in the sheathing and resulted in a watertight covering that tightly encased the grease with no voids.

In early applications, automotive-type greases were mainly used and were later found to be susceptible to oxidation and emulsification. Many of these early greases would lose their corrosion resistive qualities over time.

During the last two decades, significant improvements in the durability aspects of unbonded and bonded post-tensioning systems have been implemented. Improved tendon material specifications and the advent of the encapsulated unbonded post-tensioning systems have largely solved the corrosion problems with unbonded systems. Likewise, detailed guidance and improved specifications for grouting and ducts have resulted in enhanced performance and durability of bonded post-tensioning systems.

1.4 POST-TENSIONING INSTITUTE

The Post-Tensioning Institute (PTI) was formed in 1976 with 16 member companies. Most of these companies had previously been part of the post-tensioning division of the Prestressed Concrete Institute, and the separate organization was established to permit them to cooperate in the area of post-tensioning with clearer identity. Today, members of the Institute include major post-tensioning materials fabricators in the U.S., Canada and Mexico, and manufacturers of prestressing materials in the U.S., Canada, Mexico, Japan and Europe, and companies supplying miscellaneous materials, services and equipment used in post-tensioned construction. In addition, the Institute has more than 700 professional engineers, architects and contractors as individual professional members.

PTI provides research, technical development, marketing and promotional activities for companies engaged in post-tensioned prestressed construction. Its publications are a major communication system for disseminating information on post-tensioning design and construction technology. PTI concentrates on development of specifications and design recommendations, publication of technical literature on applications of post-tensioning, structural research and an annual program of technical seminars to disseminate information on post-tensioned design and construction technology.

The Post-Tensioning Institute published the first Post-Tensioning Manual in 1972. The Manual, which was initiated by the PCI Post-Tensioning Division prior to organization of the PTI, provided the basics of design and construction and provided an overview of post-tensioning technology. Subsequent editions were expanded to reflect the growing uses of post-tensioning.

This book is intended for students, educators, contractors, inspectors, building officials, as well as practicing engineers and architects. The text is intended to provide basic guidance and the essential principles for various uses of post-tensioning applications. In many instances, more detailed technical guides exist as standalone PTI publications; where this is the case, these more comprehensive references have been identified and referenced throughout the Manual.

1.5 CHANGES FROM EARLIER EDITIONS

The 6th Edition has been significantly reworked from the 5th Edition published in 1990. Some of the changes and enhancements that have been incorporated include:

- Chapter 2, *Applications* has been updated to include more recent examples of post-tensioning applications.
- Chapter 3 *Post-Tensioning Systems* has been rewritten and contains general information on state-of-the-art post-tensioning systems. Chapter 2 of the 5th Edition *Post-Tensioning Systems* has been eliminated from the Manual. The information in this chapter provided a comprehensive overview of then current material suppliers and their proprietary systems. In an effort to keep this information up to date, it will be available as a frequently updated standalone publication from the Post-Tensioning Institute.[1.7]
- Chapter 4 *Specifying Post-Tensioning* has been expanded to provide more comprehensive guidance in specifying post-tensioning for various applications.

- Chapter 5 *Analysis and Design Fundamentals* has been updated to reflect current code provision.
- Chapter 6 *Detailing and Construction Procedures for Buildings* has been completely rewritten to reflect current construction and detailing practice.
- Chapter 7 *Design Examples* includes new examples of building design.
- Chapter 15 *Prestressed Rock and Soil Anchors* has been updated to reflect the latest in design, installation and testing.
- Chapter 18 *Fire Resistance* has been updated to reflect current code and design practice.

In addition, new chapters have been added, including:

- Chapter 8 *Seismic Design of Post-Tensioned Concrete Structures*
- Chapter 9 *Post-Tensioned Concrete Floors*
- Chapter 10 *Post-Tensioned Parking Structures*
- Chapter 11 *Post-Tensioned Slabs-on-Ground*
- Chapter 12 *Bridges*
- Chapter 13 *Stay Cables*
- Chapter 14 *Storage Structures*
- Chapter 16 *Design of Prestressed Barrier Cable Systems*
- Chapter 17 *Prestressed Concrete Under Dynamic Loads and Fatigue*
- Chapter 19 *Durability*
- Chapter 20 *Inspection*
- Chapter 21 *Post-Tensioning Institute Certification Programs*

The post-tensioned specifications, design, detailing and construction chapters contained in the previous edition have all been updated or rewritten to reflect current practices. While much of the text is independent of specific code provisions, the requirements of the American Concrete Institute's Building Code Requirements for Structural Concrete (ACI 318) and the American Association of State Highway and Transportation Officials' (AASHTO) Bridge Design Specification have been incorporated where appropriate. ACI Code references in all chapters of this manual refer to the 2002 Edition (ACI 318-02) due to the fact that most chapters were reviewed and approved by the Technical Advisory Board (TAB) when ACI 318-02 was current.

1.6 SUMMARY

It is hoped that publication of the extensively rewritten and expanded 6th Edition of the Manual will not only provide application awareness but contribute to increased understanding and effective use of the powerful structural, economic and esthetic advantages afforded by post-tensioning.

Considerable effort has been made to ensure that information in this Manual is accurate. However, as the Post-Tensioning Institute does not prepare engineering plans, it cannot accept responsibility for any errors or oversights in the use of Manual material or in the preparation of plans and specifications. This publication is intended for the use of professionals competent to evaluate and implement the significance and limitations of its contents and who will accept responsibility for the application of the contents.

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APPLICATIONS

2.1 INTRODUCTION

Post-tensioning is a highly efficient structural system that offers many benefits in a wide range of construction, repair, and rehabilitation applications. Post-tensioning has been successfully used for small as well as large projects over the last 30 years. The efficiency stems from being able to use high strength materials, to structurally utilize the entire cross section, to vary the force and location of the reinforcing to best resist applied loads, and to control the timing of when the prestressing force is applied to the structure.

Post-tensioning offers a perfect balance of two materials which complement each other. Concrete is strong in compression and relatively weak in tension. The tensile strength of concrete is about 10% of its compressive strength. Prestressing steel, on the other hand, has a very high tensile strength (270,000 psi for strand) which is about four times that of common reinforcing bars. By combining the two, a structural member can resist both compressive and tensile forces caused by various loads. This results in greater efficiency in resisting tensile as well as compressive stresses resulting from the applied loads.

Post-tensioning can be used in all facets of construction from buildings and bridges to highway pavements, slabs-on-ground and ground anchors. It has also been used for rehabilitation and retrofit applications. In this chapter, specific project examples are presented to illustrate the wide range of applications and versatility that can be achieved by the use of post-tensioning. The project examples highlight the benefits that were achieved by post-tensioning.

2.2 BUILDINGS

Post-tensioned concrete is used in commercial buildings, residential apartments, high-rise condominiums, office buildings, parking structures, and mixed-use facilities such as hotels and casinos. Benefits of post-tensioning include:

- A significant reduction in the amount of concrete and reinforcing steel required.
- Thinner structural members as compared to non-prestressed concrete, resulting in lower overall building heights and reduced foundation loads.
- Aesthetically pleasing structures that harness the benefits of cast-in-place structures with curved geometries, and longer, slender members with large spaces between supports.
- Superior structural integrity as compared to precast concrete construction because of continuous framing and tendon continuity.
- Monolithic connections between slabs, beams, and columns that can eliminate troublesome joints between elements.
- Profiled tendons that result in balanced gravity loads (typically a portion of dead load only), significantly reducing total deflection.
- Better crack control, which results from permanent compressive forces applied to the structure during prestressing.
- Post-tensioning reduces overall building mass, which is important in zones of high seismicity.

Post-tensioning also offers the following construction advantages as compared to steel, non-prestressed concrete and precast construction:

- Faster floor construction cycle
- Lower floor weight
- Lower floor-to-floor height
- Larger spans between columns
- Reduced foundations

High early-strength concrete allows for faster floor construction cycles. The use of standard design details of the post-tensioned elements, minimum congestion of prestressed and non-prestressed reinforcement, and earlier stripping of formwork after tendon stressing can also significantly reduce the floor construction cycle. Greater span-to-depth ratios are allowed for post-tensioned members as compared to non-prestressed members. This results in a lighter structure and a reduction in floor-to-floor height while maintaining the required headroom.

The following sections give examples of building projects and highlight the use of post-tensioning to achieve some important architectural/structural requirements.

2.2.1 Office Buildings

2.2.1.1 New York State Comptrollers Building, Albany, NY

Approximately 2000 employees from the office of the New York State Comptroller and the Department of Taxation and Finance moved into the 15-story, 450,000 sq ft office building in Albany New York. Fig. 2.1 shows the completed structure.

Typical plan dimensions of a floor in the building are 104 ft by 304 ft. Typical spans range from 30 to 50 ft; a 9 in. thick post-tensioned flat slab with 9 ft by 9 ft drop panels was used for bays up to 34 ft- 6 in. in span. The drop panels were made continuous in the 50 ft direction to form a wide shallow beam between the supports. 18 in. deep beams were used at the perimeter to support the exterior cladding elements and provide lateral resistance. The structure is designed for 80 mph wind loads. The lateral load resistance is provided by moment frames that include the flat slab, edge beams, and columns. To achieve higher

Fig. 2.1 Office of the New York Comptroller, Albany, NY
Courtesy of Weidlinger Associates Inc. and Portland Cement Association

early strength, 6000 psi concrete was used in the floor slabs. This allowed a two to three day construction cycle for typical floors, even during the winter months. The overall cost of the project drove the decision for use of post-tensioning on the project. The project was finished on time and under budget. The project highlights the time and money savings that can be achieved through the use of post-tensioning.

2.2.1.2 One Overton Park Office Building, Atlanta, GA

Overton Park is zoned to include 1.45 million sq ft of office space, 400 units of high-rise luxury residential space, a 200-room hotel, four restaurants and retail space. The first phase of the project, which included a 14-story office building, was completed in 2001. It incorporates the use of long-span post-tensioned beams and semi-lightweight concrete to reduce the overall weight, thus reducing column sizes and saving considerable foundation cost. The total floor thickness was reduced to 20 in., which allowed a lower floor-to-floor height, further reducing cladding costs.

The 35,000 sq ft floor plate was built with a 30 in. by 20 in. deep typical girder spanning 42 ft. Post-tensioning stresses in the girders were around 375 psi. Post-tensioning increased the stiffness of the girders, increasing the effectiveness of the beam-column frames. This allowed the 200 ft tall building to be built without shear walls, resulting in time and cost savings to the owner. It also allowed increased flexibility for interior space planning. Fig 2.2 shows an overall view of One Overton Park from the street.

Fig. 2.2 One Overton Park Office Building, Atlanta, GA
Courtesy of Walter P. Moore and Associates, Inc.

Fig. 2.3 Penterra Plaza, Denver, CO
Courtesy of S. A. Miro, Inc.

2.2.2 Condominiums/Residential Buildings

2.2.2.1 Penterra Plaza, Denver, CO

Penterra Plaza is the first landmark residential high-rise in the Denver Technological Center. The L-shaped tower consists of 23 stories of condominiums over a 3-story below-grade precast parking garage. The structural system of the tower is a post-tensioned flat slab with cast-in-place columns and shear walls. The condominium consists of 560,000 sq ft of post-tensioned slab with 266 luxury units ranging in size from 600 to 3600 sq ft. Story-to-story heights vary from 9 ft-4 in. to 12 ft-0 in.

Because the tower is located over the parking garage, an irregular column layout was required to accommodate drive aisles and ramps. This resulted in an uneven arrangement of bay sizes ranging from 30 ft-0 in. by 30 ft-0 in. to 12 ft-0 in. by 20 ft-0 in. Traditional non-prestressed cast-in-place construction or steel framing would have required a complicated layout of beams and girders in order to achieve the required column arrangement. The use of an 8 in. thick post-tensioned slab resulted in much lower structure depths at each floor, and significantly simplified formwork. In addition, shear rail reinforcement eliminated the need for drop caps at columns. The flat plate allowed the use of "flying" forms and a much shorter floor construction cycle. The reduced floor-to-floor heights also reduced the overall height of the building, resulting in significant cost savings for the building skin. The lighter floor system also resulted in reduced foundation costs.

The flexibility of the post-tensioned slab system also allowed the two wings of the building to be connected by two round sections of flat slab. Rather than requiring a complicated arrangement of cantilevered beams, the support of these round sections was greatly simplified by placing distributed post-tensioning tendons with a horizontal curvature around the radii. The design of the slab utilized

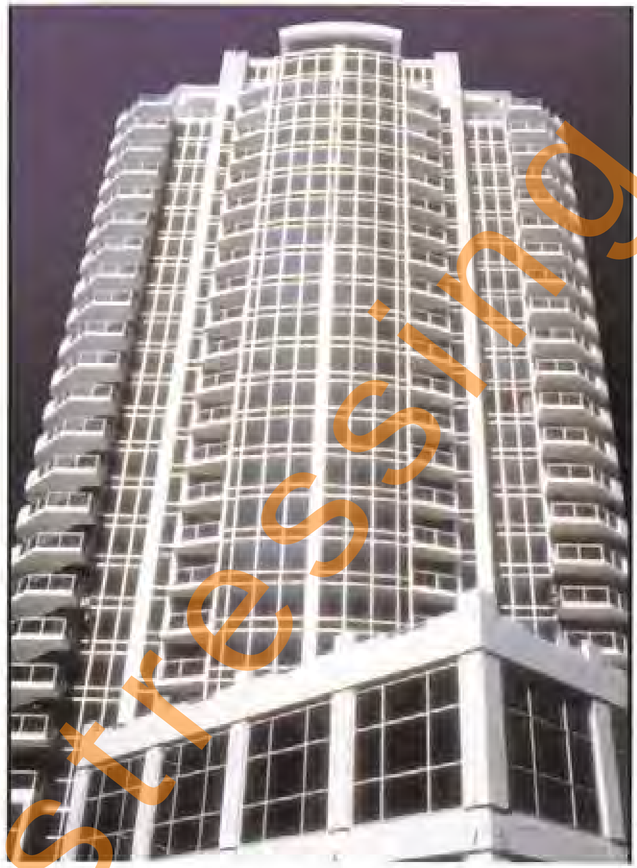


Fig. 2.4 One Pacific Tower, Seattle, WA
Courtesy of Cary Kopczynski & Company, Inc.

only a minimum of non-prestressed reinforcement, with long-term deflections being dramatically reduced because of the load-balancing effect of the post-tensioning. Fig. 2.3 shows the second floor concrete placement.

Two-story open spaces and internal stairs were easily accommodated by adjusting the post-tensioning reinforcement. Similarly, the varying unit layouts between floors also resulted in a large number of scattered plumbing and mechanical penetrations through each floor. Coordination of these penetrations within the structure was simplified because there were no beams or other members to avoid.

2.2.2.2 One Pacific Tower, Seattle, WA

This 27-story tower is one of the premier condominiums in downtown Seattle. The building's free-form perimeter has full-height glass unobstructed by beams. Large cantilevers create dramatic open spaces for living rooms and balconies. The tower is rotated 45 degrees to the parking pedestal to maximize views of the natural surroundings; see Fig. 2.4.

All seismic frames were located at the building interior, allowing floor-to-ceiling glass at the perimeter. Unbonded post-tensioning accommodated the irregular pattern of columns required by the architectural layout. The flat plate post-tensioned floors also reduced the floor-to-floor height and allowed several extra levels within the building's 240 ft height limit.

The 45 degree rotation of the tower on the parking pedestal created framing challenges at the transition. A combination of bonded and unbonded post-tensioned transfer beams were designed to carry the tower columns and redirect loads to the pedestal framing. Post-tensioned flat plate construction minimized floor-to-floor height in the subterranean parking levels, which resulted in reduced excavation. This also created a cost reduction for the shoring system.

Through careful planning and by designing for constructability, the concrete frame was completed on schedule and budget despite the complex architectural layout. Concrete mixes, reinforcing details, and construction sequencing were planned to maximize the economies realized through standardization. Once the typical tower floor was reached, the contractor poured one level every week.

The challenging floor plan, open perimeter, and internal seismic system would have been difficult to frame with any material other than cast-in-place post-tensioned concrete.

2.2.2.3 Turnberry Place Luxury Estate Residences, Las Vegas, NV

Turnberry Place is located on Paradise Road and Riviera Boulevard, across from the Las Vegas Hilton and one block from the Las Vegas Strip.

The floor system of the towers consists of 8 in. thick post-tensioned concrete flat plates that span 25 ft to 30 ft in each direction. Using both round and rectangular columns, the sizes range from a diameter of 36 in. in the parking garage to 18 in. by 32 in. in the tower. Located strategically around the elevator and stair cores are 24 in. thick shear walls that help resist wind and seismic forces. A 9.5 ft thick mat foundation supports the columns and walls.

High-strength concrete ranging from 5000 psi to 8000 psi allowed consistent cycling of forms and contributed to quick access of floors to other trades on the project. The rapid pace of concrete construction allowed completion of the first tower in 18 months. The project manager predicted that the post-tensioned concrete system saved about 6 months in the construction schedule as compared to a steel frame structure.

Turnberry's quick turnaround of the first two towers has led to strong sales for the third tower, which recently broke ground and is 60% pre-sold. It is estimated that the project is about 18 months ahead of schedule. Fig. 2.5 shows an overall view of the complex.

2.2.2.4 Acqua Vista, San Diego, CA

Acqua Vista is an 18-story, post-tensioned residential building with 382 units located in the Little Italy district of San Diego. Twin towers extend from floors 8 through 18 in this cast-in-place concrete structure, with an open courtyard and pool situated between the towers at the eighth floor level. There are two levels of underground parking, with 417 parking spaces.

The project is located directly over an ancient fault line, and is within two kilometers of the Rose Canyon Fault Zone (part of the San Andreas Fault system). Earthquakes

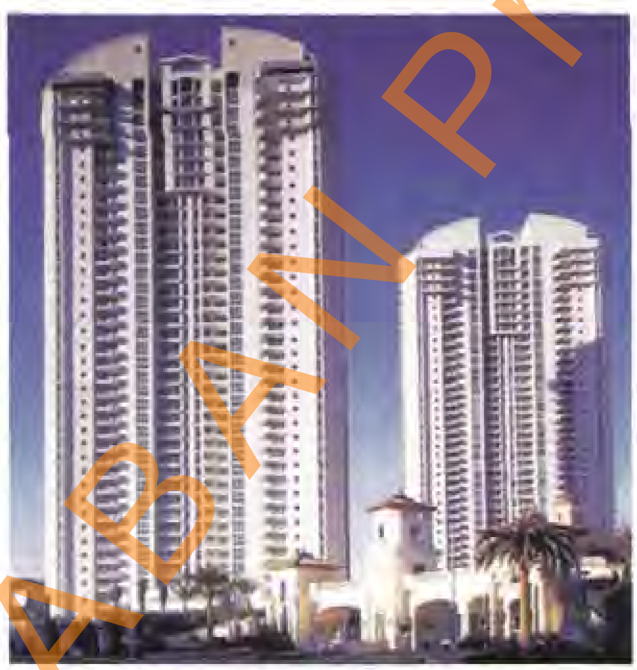


Fig. 2.5 Turnberry Place Luxury Estate Residences, Las Vegas, NV
Courtesy of Martin & Peltyn, Inc. and Portland Cement Association

Fig. 2.6 Acqua Vista, San Diego, CA
Courtesy of Poggemeyer Design Group, Inc.

at the site tend to be shallow, strong, short, and frequent aftershocks are common. Concrete shear walls founded on a mat foundation provided a lateral resisting system. Shear walls were typically 18 to 24 in. thick, with heavy boundary reinforcing.

The floor and roof systems consisted of cast-in-place post-tensioned flat plates supported by concrete columns. Typical bay spacing ranged from 26 to 34 ft. The typical floor thickness was 8½ inches, to support standard residential floor loads. The Level 8 podium slab and roof slabs were 10 in. thick, to support the pool and deck area and the large mechanical units, respectively. A 3-D, finite-element post-tensioning software was used to model the floor geometry, including openings and changes in slab thicknesses. Slab closure pours used at large slab areas minimized the effects of concrete shrinkage.

Post-tensioning allowed for thinner sections, less slab deflection, more favorable stresses at service loads, and better crack control compared to non-prestressed construction. Post-tensioning also provided superior performance of diaphragm action at building irregularities, resisting tensile forces resulting from separation of "wings" at re-entrant building corners. Thinner slabs reduced building height and therefore saved material cost and labor. Post-tensioned slabs span farther than non-prestressed slabs, allowing for wider column spacing and fewer columns. Fig. 2.6 shows a view of Acqua Vista during construction.

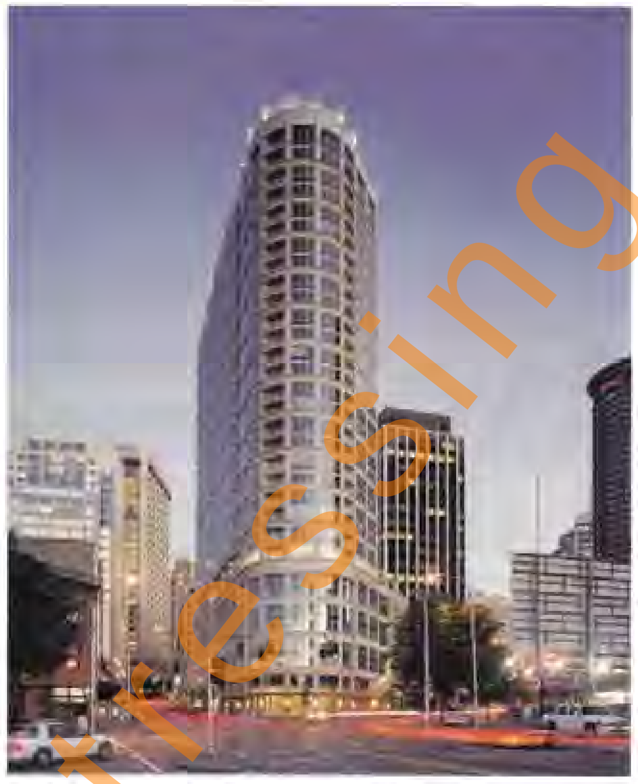


Fig. 2.7 Metropolitan Tower, Seattle, WA
Courtesy of Cary Kopczynski & Company, Inc.

2.2.2.5 Metropolitan Tower, Seattle, WA

Metropolitan Tower is an elegant addition to the Seattle skyline. Constructed in 2001, the 320 ft high building includes 24 stories of luxury condominiums over a 7-story parking podium. Noteworthy features include the unique shape of the building and the long, cantilevered balconies around the perimeter that provide unobstructed views; see Fig. 2.7.

The floor system for both the parking structure and residential tower consists of 7.5 in. thick post-tensioned flat plates with spans from 27 ft to 30 ft. Typical non-seismic columns are 24 in. square for the full building height. The slab layout was developed to maximize forming efficiency and streamline construction.

Lateral resistance is provided by shearwalls and ductile moment-resisting frames, with approximately 70 percent of the seismic forces carried by the shearwalls. This system is both well balanced and highly redundant. The shearwalls, which are 24 in. thick full height, are located around elevator and stair cores. At the top of the parking structure, reduced lateral forces allowed for a reduction in wall lengths. Maintaining constant wall thickness and column sizes for the full building height enhanced constructability and reduced overall construction time. Concrete strengths and reinforcing quantities are reduced at higher levels of the building to account for the smaller loads.

The building is supported on a floating foundation mat covering the full site. Tying the mat to the perimeter walls minimized settlements and lowered construction costs. Post-tensioning the tower floors reduced slab thickness, which lowered the building weight and helped make this foundation solution feasible.

Normal weight concrete with a specified compressive strength of 7000 psi at 56 days was utilized for the post-tensioned slabs. This strength achieved compatibility with the high-strength column concrete. Eliminating the need to "puddle" high-strength concrete at the columns significantly decreased both construction time and labor costs. For early stressing purposes, the slab concrete was specified with a compressive strength of 3000 psi at three days.

Cast-in-place post-tensioned concrete greatly reduced the floor-to-floor height when compared to a structural steel option, which also resulted in significant savings in the façade, HVAC, electrical, plumbing, and vertical transportation systems. Utilizing the slab soffit as the ceiling eliminated the need for a suspended ceiling, and also resulted in substantial cost savings. Furthermore, the inherent fire resistance of concrete provided the prescribed fire resistance without additional fireproofing. Residents will enjoy safe and quiet living for many years to come.

Fig. 2.8 Hyatt Grand Resort, Huntington Beach, CA
Courtesy of Ficcadenti & Waggoner Consulting Structural Engineers, Inc.

2.2.3 Hotels

2.2.3.1 Hyatt Grand Resort, Huntington Beach, CA

Huntington Beach has been working for a number of years to upgrade its beachfront to attract more visitors to the laid-back atmosphere in Surf City. The Hyatt Regency Grand Coast Resort construction strategically contributes to this transformation by adding a major resort and conference center. Opened in 2003, the Mediterranean-style hotel provides luxury accommodations while preserving the relaxed beach community atmosphere; see Fig. 2.8.

The resort's four-story, 500,000 sq ft super-structure offers 517 guestrooms and 57 suites. It is configured in four wings extending out toward the ocean and offering spectacular views of the Pacific while surrounding the hotel's scenic courtyards. The indoor and outdoor function space of both the hotel and the 110,000 sq foot conference center are supported on a two-level, 400,000 sq ft underground parking garage providing 1000 parking stalls. The resort boasts retail shops, restaurants, and its very own pedestrian bridge across the Pacific Coast Highway, allowing convenient access to the beach and the famed Huntington Beach Pier.

To minimize floor-to-floor heights, upper hotel levels are 8 in. thick post-tensioned concrete floor plates where approximately 500,000 sq ft of slab was cast in a three-month period. Spanning 29 ft in typical bays, the floor system allowed flexibility in locating the 14 by 28 in. columns. To accommodate open areas at the lower floors, about 100 post-tensioned concrete girders transferred column loads with minimal deflection. With dimensions up to 4 ft in width by 4.5 ft in depth, the transfer girders utilized up to 44 harped tendons 0.6 in. in diameter. The selection of a post-tensioned system for the hotel enabled flexibility in varying the layout at different levels, provided the required fire separation without the additional fireproofing measures, and eliminated lost ceiling space. Concrete also provided favorable sound attenuation and vibration control, essential for a luxury hotel.

Combining hotel guestrooms at the upper levels, common gathering and convention areas at the plaza level, and parking below grade created challenging project architecture. Concrete provided the flexibility to accommodate the need for varying column layouts with minimum floor depth and the speed to achieve the desired schedule.

2.2.3.2 Grove Hotel, Boise, ID

The Grove Hotel and Bank of America Centre has become the focal point of social and sporting events in Idaho. Prominently located in the heart of downtown Boise, the

Fig. 2.9 Grove Hotel, Boise, ID
Courtesy of Cary Kopczynski & Company, Inc.

project provides a vibrant mix of services for the entire community. It includes a luxury hotel, 5000-seat arena that hosts major concerts and conventions, retail space, fine dining, and a sports bar. The arena also serves as home to a hockey team, the Boise Steelheads; Fig. 2.9.

Schedule was a key factor in the decision to use cast-in-place post-tensioned concrete. The design was fast-tracked, with shoring approved by the building department prior to the final building design being submitted. The speed and simplicity of the unbonded post-tensioning system complemented the fast pace of the project. The ability to stress tendons and strip forms three days after concrete placement was crucial to the schedule. With only 24 months to complete the entire project and open it to the public, every day counted. When it came time to "drop the puck" for the opening hockey game, the arena was ready.

The benefits of using post-tensioning in hotel and residential applications are well known, and were fully exploited in this project. By taking advantage of flat plate post-tensioned slabs, floor-to-floor height was minimized by using slab soffits as finished ceilings for the units. Post-tensioning allowed the slab thickness to be minimized while maintaining the required design strength and serviceability, along with reduced deflection and greater crack control.

Reducing floor-to-floor heights benefited the project both above and below grade. In the tower, the building height was lowered, reducing cladding costs and vertical utility runs. In the subterranean parking, raising the slab-on-grade elevation achieved shoring and excavation savings. This was significant because the water table was immediately below the grade slab. If a deeper structural system had been used, the lowest parking level would have required full waterproofing. Post-tensioned flat plates solved the problem.

The grand ballroom provided an unusual post-tensioning challenge. With one end of the ballroom located under the residential tower, three exterior tower columns supporting fourteen floors required transferring to create a 100 ft clear span. The solution involved a two-story truss with post-tensioned beams as top and bottom chords at the fourth and sixth floors. The beam at the fourth floor was designed to support several floors above with only one central shoring point, allowing the remainder of the formwork to be removed. With the fourth and sixth floors supported by the post-tensioned edge beams, tower construction proceeded unimpeded. Structural steel diagonals were then welded to inserts cast into the columns. Once the diagonals were in place, the center shore was removed. Vertical deflection of the 100-ft-long truss was only ½ in. The bottom chord was designed for zero net tension under dead load, allowing the full section properties of the composite truss to be achieved.

The many challenges posed by the Grove Hotel and Bank of America Centre allowed post-tensioning to demonstrate its speed, versatility, and economy. Post-tensioning is a major contributor to the project's success.

Fig. 2.10 Moody Gardens, Galveston, TX
Courtesy of Walter P. Moore and Associates, Inc.

2.2.3.3 Moody Gardens, Galveston, TX

Moody Gardens is one of the top-tier entertainment complexes in the southern United States, featuring three signature pyramids (with aquarium, museum, and tropical rain-forest themes), an IMAX theater and an artificial white-sand beach which are surrounded by a nine-story hotel with an adjoining convention center; Fig. 2.10. Moody Gardens ranks among the Houston area's favorite tourist attractions, drawing more than 2 million visitors annually to Galveston Island.

The Phase V addition to the hotel completed in 2003 provided a 125-room expansion to the existing 300-room facility. Considering the complex column layout and the small floor-to-floor heights required to match the existing structure (typically 9 ft-8 in.), a post-tensioned slab was the obvious choice. Post-tensioning allowed the floor thickness to be reduced by 20%.

The sawtooth profile of the building, designed to provide scenic picture windows and balconies overlooking the West Bay just off the Gulf of Mexico, drove the irregular column layout.

The coastal location exposes the exterior slabs to the harsh saltwater environment. Thus, the project used an encapsulated system of post-tensioning to ensure the structure had added durability in a corrosive environment.

The typical floor plate of the hotel consisted of 8 in. two-way post-tensioned flat plate with 14 in. by 24 in. concrete columns and 12 in. shear walls at the stair and elevator cores. Stud shear rails were used for punching shear reinforcement.

2.2.4 Mixed Use

2.2.4.1 Four Seasons Hotel and Tower, Miami, FL

The Four Seasons Hotel and Tower offers sweeping views of downtown Miami, Biscayne Bay and the Atlantic Ocean. Rising 70 stories into the sky in the heart of the Brickell district, the Four Seasons Hotel and Tower Miami is the newest multi-use building to dominate South Florida's cityscape. Opened in October 2003, the 789-foot-tall structure has earned the distinction of being the tallest residential building south of Manhattan. It is considered to be among the 50 tallest buildings in the world; see Fig. 2.11.

Comprising 222 hotel rooms, 84 condominium hotel residences and 186 condominiums, the Four Seasons also features 200,000 sq ft of office space and a 45,000 sq ft spa and sports club.

Twelve-inch-thick post-tensioned structural slabs and transfer beams eliminated the need for interior columns and provided column-free space that allowed flexibility in the layout of the condominiums. The typical spans of the slab ranged between 40 and 50 ft between supports. Eliminating the interior columns kept the load transfer at the perimeter, where it was efficiently handled by the perimeter structural frame that formed a part of the lateral load-resisting system.

The contractor successfully achieved a four-day cycle for both the post-tensioned floor slabs and the shear walls, the latter of which was three floors ahead of the slab crews from the beginning of structural work.

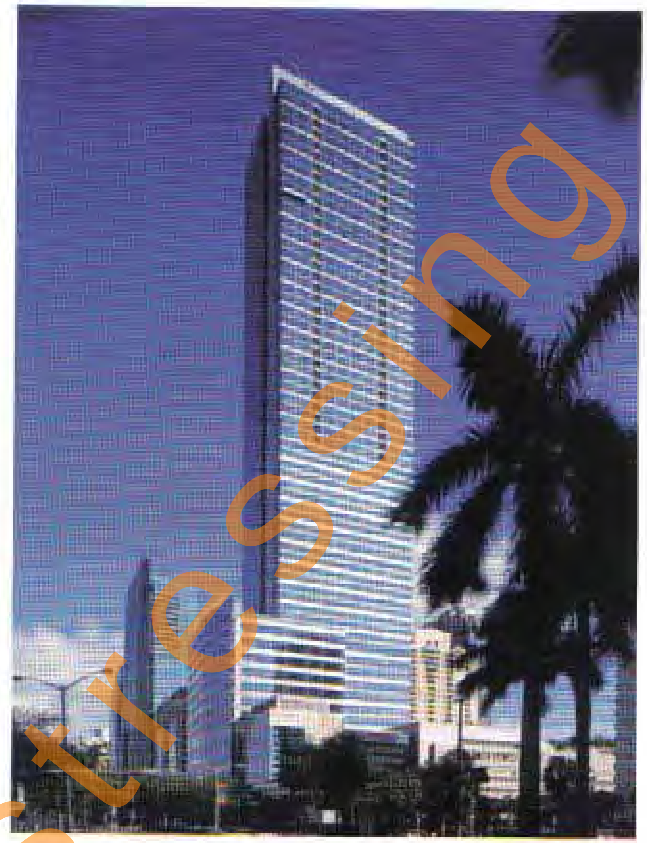


Fig. 2.11 Four Seasons Hotel and Tower, Miami, FL
Courtesy of DeSimone Consulting Engineers and Suncoast Post-Tension L.P.

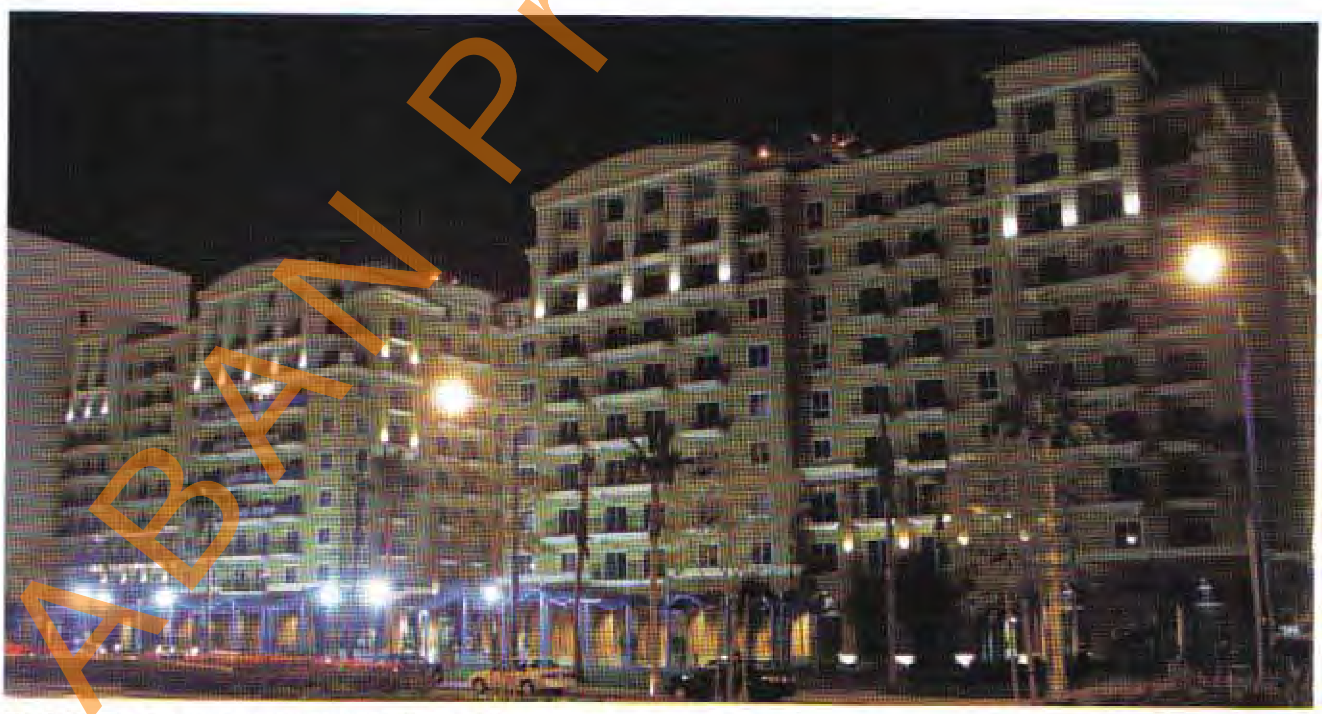


Fig. 2.12 The Park at Harbor View, Long Beach, CA
Courtesy of BFL Owen & Associates

2.2.4.2 The Park at Harbor View, Long Beach, CA

The Park at Harbor View is a mixed-use facility in Long Beach, Calif. that overlooks historical buildings and landmarks. The development is designed to provide a transition from a high-rise office building zone on the north end to a residential low-rise building with ocean views to the south. The project includes 246 condominium units, 538 apartment units, 25,000 sq ft of retail/commercial space, and a 500-room hotel; Fig. 2.12. This project provides much-needed housing in the urban setting of Long Beach, nestled between Los Angeles and Orange Counties.

Concrete structures with post-tensioned decks provide a superior structural solution for mid-rise buildings in Southern California. Desirable fire rating is achieved in concrete structures with virtually no additional cost. They also provide superior vibration and noise control, which offers occupants a higher level of comfort. Use of post-tensioned flooring reduces the slab thickness, thus reducing the overall weight of a structure up to 30% when compared with a reinforced concrete structure. This also improves the seismic design and performance of the structure.

The typical floor system in the Long Beach towers and the parking garage consisted of a 7-inch-thick post-tensioned concrete slab. Twelve-inch post-tensioned concrete slab was used at the podium level to span over typical spans of about 27 to 30 ft. The floor system incorporated column capitals and shallow post-tensioned beams at selected locations where unusual loading conditions or long spans occurred.

A reinforced concrete shear wall system provided lateral force resistance for the garage and towers. Both the garages and the buildings above are divided into three independent structures separated by seismic expansion joints 3 to 4 inches wide. Due to plan and height irregularities, the shear walls were distributed based on 3-D analysis for each structure. The thickness of the walls varies between 14 inches at the towers to 18 inches at the garage levels.

2.2.4.3 The Espirito Santo Plaza, Miami, FL

Espirito Santo Plaza is a sparkling 37-story glass tower with simple, elegant lines and a sculptured exterior. Located in Miami, the building welcomes visitors with a dramatic concave figural arch, beautifully symbolizing the gateway to Latin America; see Fig. 2.13.

The 483 ft high tower is a 750,000 square foot mixed-use building divided, from bottom to top, into office, hotel and residential floors. Between the hotel and residential floors, there are two mechanical levels, and a sky-lobby level. The sky lobby has an expansive 10-story high atrium which is enclosed with a glass wall and a skylight. The residential floors wrap around this atrium.

The framing of the office floors was organized to span between the exterior columns and the central building core, eliminating the need for interior columns. The framing consists of a 7.5 in. post-tensioned one-way slab supported by 28 in. deep post-tensioned beams that span up to 50 ft.

Fig. 2.13 The Espirito Santo Plaza, Miami, FL
Courtesy of Leslie E. Robertson Associates

The framing of the hotel floors consists of a 6.5 in. post-tensioned flat plate supported by a grid of columns. At the 16th floor, a system of post-tensioned girders transfers the hotel column loads to the core walls and exterior columns of the office floors below. The transfer girders were typically post-tensioned in two stages.

The residential floors consist of a 7 in. post-tensioned flat plate. Since the residential columns do not line up with the hotel columns, they are also transferred at the 25th floor to the core walls and exterior columns of the hotel floors below.

2.2.4.4 Sam Nunn Federal Center, Atlanta, GA

The Sam Nunn Federal Center in downtown Atlanta combines many different federal agencies across the city into one central location; Fig. 2.14. While post-tensioning is used throughout this multi-use, multi-building complex, the most benefit is gained in the 24-story office tower. The tower, including four levels of basement parking, gains from the use of semi-lightweight concrete in conjunction with 20 in. deep total structural depth to allow for lower floor-to-floor heights and lighter overall loads. This signif-

Fig. 2.14 Sam Nunn Federal Center, Atlanta, GA

icantly reduces the size of the columns and thickness of the mat foundation.

The typical exterior span of this 41,000 gross sq ft floor plate is 47 ft-6 in. Post-tensioning and haunches at the interior support allow for this long, shallow span to support a live load of 80 psf required by the owner, plus additional superimposed loads. Post-tensioning stresses for the typically 36 in. wide by 20 in. deep girders are around 350 psi.

The lateral system for this 400 ft tall structure is a dual system of shear walls and concrete beam-column frames, resisting both wind and seismic forces. Post-tensioning increased the stiffness of the frame members, allowing the post-tensioned girders to be more effective despite their shallow depth. This also resulted in a savings in the shear wall construction.

2.2.5 Casinos

2.2.5.1 Silver Legacy Hotel and Casino, Reno, NV

Silver Legacy is one of Reno's newest casino hotels. The 38-story Silver Legacy Resort Casino has 1720 rooms and is styled on a Victorian theme. It is the tallest hotel in northern Nevada. The property encompasses 85,000 sq ft of exciting gaming space and 90,000 sq ft of convention space; Fig. 2.15.

The floors for the hotel rooms were typically 7 in. thick, incorporating a one-way flat slab design. Uniformly dis-

Fig. 2.15 Silver Legacy Hotel and Casino, Reno, NV
Courtesy of DYWIDAG-Systems International, USA

tributed unbonded tendons were placed only longitudinally in each tower spanning the typical 25 ft-8 in. bays. The attached parking levels consist of a beam-and-slab design. Both the beams and slabs were tensioned with unbonded monostrand tendons. The typical beam dimensions are 16 in. by 37.5 in. Excavation began on the structure in early January 1994. The doors officially opened almost a year and a half later, in 1995. The complete structure includes approx. 7000 tons of steel reinforcement, 334 tons of tendons and more than 96,000 cubic yards (73,400 m^3) of concrete.

2.2.5.2 The Borgata Hotel Casino, Atlantic City, NJ

The Borgata Hotel Casino in Atlantic City, N.J. is a 1.53 million sq ft, 42-story, 2000-room hotel structure; Fig. 2.16. The hotel tower is a cast-in-place concrete structure with post-tensioned flat plate floor construction.

The 510-foot-long floors are framed with 5000 psi, 7½ in. thick post-tensioned flat plate. The benefits of post-tensioned construction were most apparent with these spans, where the span/depth ratios are 49. None of the other structural systems investigated during the schematic design phase of the project came close to matching this ratio.

A cylindrical zone (see photo) occurs at each end of the tower, giving the structure a unique and distinctive architectural appearance. The circular floor areas in these cylindrical zones were economically designed and constructed with cast-in-place post-tensioned concrete despite irregular column locations and numerous openings in the floors. Perimeter columns in the circular areas were spaced at 35ft-0in. on center. The larger column spacing in the circular end zones necessitated the use of an 8½ in. thick slab within these areas. The curved slab edges were designed as

8½ in. deep "in-slab" beams. Their design and construction was simply a matter of installing a curved "band" of tendons along the slab edges to support the façade and tributary floor slab area.

Perimeter slab edges support a glass curtain wall façade. Attachment of this façade to the floor slabs required careful attention to slab edge deflections and the means by which these deflections could be controlled. Additional tendons were added at perimeter slab edges to provide the required stiffness. The additional tendons eliminated virtually all dead load and long-term creep deflections that would otherwise have occurred due to the self-weight of the structure.

The use of cast-in-place post-tensioned flat plate construction provided the Borgata project team a structural system that had flexibility to easily accommodate irregular column locations and slab openings, curved slab edges, and low floor-to-floor heights—all while doing so efficiently and economically.

2.2.5.3 The Golden Moon Hotel & Casino, Philadelphia, MS

The Golden Moon Hotel & Casino doubled the size of the existing Pearl River Resort in Philadelphia, Miss., 98 miles from Jackson, providing 571 new rooms in a 30-story, 843,000 sq ft luxury hotel. The casino offers 90,000 sq ft

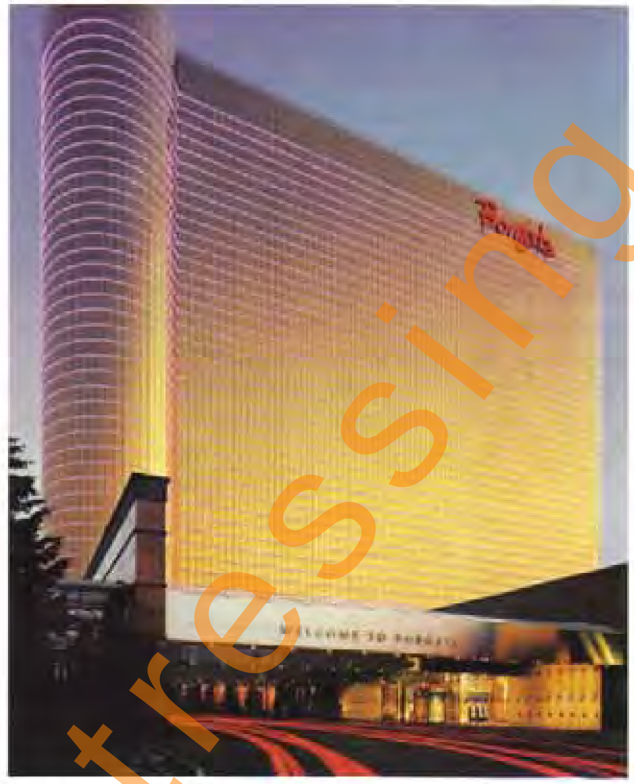


Fig. 2.16 The Borgata Hotel Casino, Atlantic City, NJ
Courtesy of Cagley Harman & Associates

Fig. 2.17 The Golden Moon Hotel & Casino, Philadelphia, MS
Courtesy of Walter P. Moore and Associates, Inc.

of gaming space, 44,000 sq ft of retail, a new cultural center, 1200-seat theater and a 1520-car parking deck; Fig. 2.17. The $177 million hotel-casino is the first phase of a multiphase, master-plan expansion of the casino complex developed as an economic engine by the Mississippi Band of Choctaw Indians.

The structural engineers selected a post-tensioned flat plate concrete structural frame for the curved-face, sloping-roof 30-story hotel tower. There were economic advantages to using the post-tensioned flat plate floor framing system. It allowed the architects to minimize the floor-to-floor heights while maximizing column-to-column spacing. This maximized the amount of usable square footage for the owner. It also provided economic benefits to the owner by reducing the cost associated with formwork and the duration of construction. The contractor used flying forms to maintain the high rate of construction to meet the owner's accelerated construction schedule and reduce the overall cost.

The project met or exceeded all of the owner's goals. It opened on time, beginning to generate a revenue stream just 22 short months after groundbreaking. Occupancy was almost 100% through its first few months of operations, exceeding projections. Its on-budget opening was well received by the community. More than 2000 new jobs were created, raising the overall Choctaw workforce by 20%.

2.2.6 Research/Academic Institutions

2.2.6.1 Knowlton Hall, Ohio State University, OH

Knowlton Hall is a four-story building for the school of architecture at Ohio State University. A majority of the 168,000 sq ft structure is exposed, including the columns, walls and floor slabs. The building features unique floor plates at every level, with large floor openings and offsets in elevation that create sight lines from end-to-end of the 480-foot long building. In plan, the footprint of the building follows the curvature of the streets that border the site, creating a gracefully undulating façade clad in white marble shingles. The floor slabs at the third and roof levels support the façade, creating alternately sweeping and tightly curved slab edges; Fig. 2.18. The façade progressively steps out at each level, requiring cantilevered slabs as long as 10 ft at the third level and 13 ft at the roof. The columns in the structure are laid out so that in plan it appears that they are randomly placed. Contrary to typical document preparation practices, this project did not have any column grids. Both the slab edges and the column locations were defined using coordinates obtained from the architect's electronic file.

Most of the 12-inch post-tensioned concrete floor slabs are exposed, and the many openings in the floor plates permit views of the relatively thin floor structure. The edges of the floor slabs are cantilevered from the columns and left

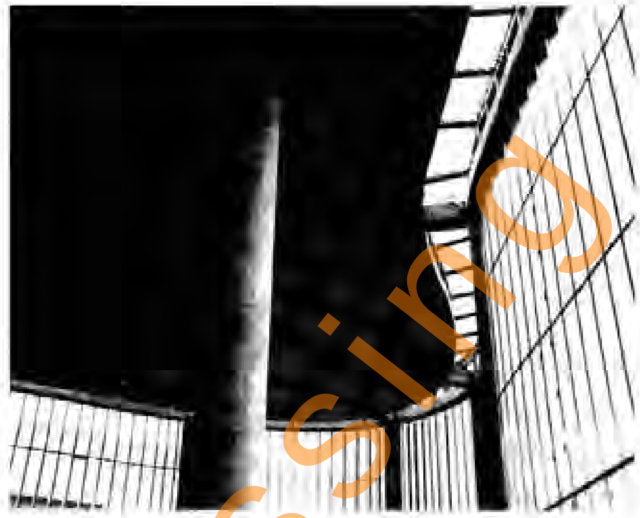


Fig. 2.18 Knowlton Hall, Ohio State University, OH
Courtesy of Shelley Motz Baumann Hawk, Inc.; formerly Lantz, Jones & Nebraska, Inc.

exposed, creating an impression that the slabs are floating in space. The floating effect is most pronounced at the many stair locations at the edges of the third level, where the stairs seemingly defy gravity as they span between levels while surrounded by space. The most dramatic of the exposed slabs is the 32 in. thick slab supporting the roof garden, an 80-foot by 60-foot area soaring almost 50 ft above the main entrance plaza below. The slab spans 50 ft and cantilevers 25 ft, creating a spectacular effect on everyone who enters the building.

The project contains a number of unique features, all of which dictated the use of post-tensioned concrete as the primary structural component. The requirements of a thin structure and the non-linear column layout made concrete a favorable choice. Additionally, concrete naturally possesses the required 2-hour fire rating while also providing an aesthetically appealing exposed structure. The large column spacing, dramatic cantilevers, and high live load requirements drove the need for post-tensioned concrete. Without post-tensioning, the column spacing would have been limited significantly. Likewise, the long cantilevers would not work without the strength and deflection performance that post-tensioning provides. Post-tensioning also eliminated the large quantities and the congestion of steel reinforcing bars that are typical of reinforced concrete slab construction. An additional benefit to the owner is that cracking in the exposed floor slabs is virtually eliminated, creating a much more pleasing appearance.

Knowlton Hall provided a big challenge and a big reward for everyone involved in the project, from the architects to the contractors. Without post-tensioning, the dramatic spaces and soaring cantilevers would not have been possible, and the building would not have the effect that it does on all who experience it.

Fig. 2.19 Levine Hall, University of Pennsylvania, PA
Courtesy of CVM Structural Engineers

2.2.6.2 Levine Hall, University of Pennsylvania, PA

Levine Hall is a six-story concrete framed building with post-tensioned floors housing offices, labs, meeting spaces and a 150-seat auditorium for the Computer and Information Science Department of the University of Pennsylvania; Fig. 2.19.

Levine Hall is built in between and connects two existing University of Pennsylvania facilities, the Graduate Research Wing of Moore Building and Towne Building. The exterior walls of the building are fully glazed. The architectural goal was to have the floor construction concealed as one looks at the building through the glass walls. The 14 ft floor-to-floor height and extensive use of exposed concrete provides a "loft-like" feel and maximizes the usable interior space.

There are three major column lines which run north and south approximately 22 ft apart. The two outmost column lines are set approximately 12 ft away from the exterior walls on the east- and west-facing elevations of the building. The main post-tensioned beams span north and south along the column lines. The concrete post-tensioned floor spans east to west between the beams and cantilevers out 12 ft from the outermost column line on the east and west faces of the building. The concrete floor is 7 in. thick between the column lines and 10 in. thick in the cantilever section. This narrow floor profile accomplishes the design goal of concealing the floor construction as one looks at the building through the glass walls as well as enhancing the "loft-like" feel of the interior. The lateral resisting system is composed of the post-tensioned floor slab diaphragms tied to cast-in place concrete shear walls around the elevator shaft and two stair wells in the building.

2.2.6.3 Biosciences Research Building, Vanderbilt University, TN

The Biosciences Medical Research Building is a 384,737 sq ft research and educational facility located on the campus of Vanderbilt University in Nashville, Tenn. It is known informally on campus as "MRB III"; Fig. 2.20. The facility is shared between Vanderbilt University and Vanderbilt University Medical Center. The total project cost was $79 million and the construction duration was 27 months.

The building is a 10-story concrete frame, which utilizes a fully bonded post-tensioned reinforced concrete structure. There is a two-story steel frame mechanical penthouse. The main loading dock for the hospital was located on the site chosen for the Medical Research Building. The University decided that the building would have to be designed to leave the existing loading dock in place. This necessitated a 64 ft by 120 ft clear area without columns. The area above the loading dock was supported using a floor depth two-way post tensioned truss system that supported the ten floors above the loading dock area. These elements were stage tensioned, which allowed the addition of tendon force as floors were constructed.

An existing lab building built in 1927 also had to be connected to the new facility. This required that the floor elevations in the new facility matched the floors of the existing building. The floor-to-floor heights in the existing lab were below the minimum requirement of the new laboratories. This made the floor thickness in the new facility critical. With post-tensioning it was possible to provide a thinner slab with comparable strength and deflection performance. This allowed the new facility to be built to the current standards for headroom while matching the floor elevations of the existing building.

The post-tensioned system was the best choice to meet the owner's needs on this project. The system was economical, durable and flexible.

Fig. 2.20 Biosciences Research Building, Vanderbilt University, TN
Courtesy of PTSI

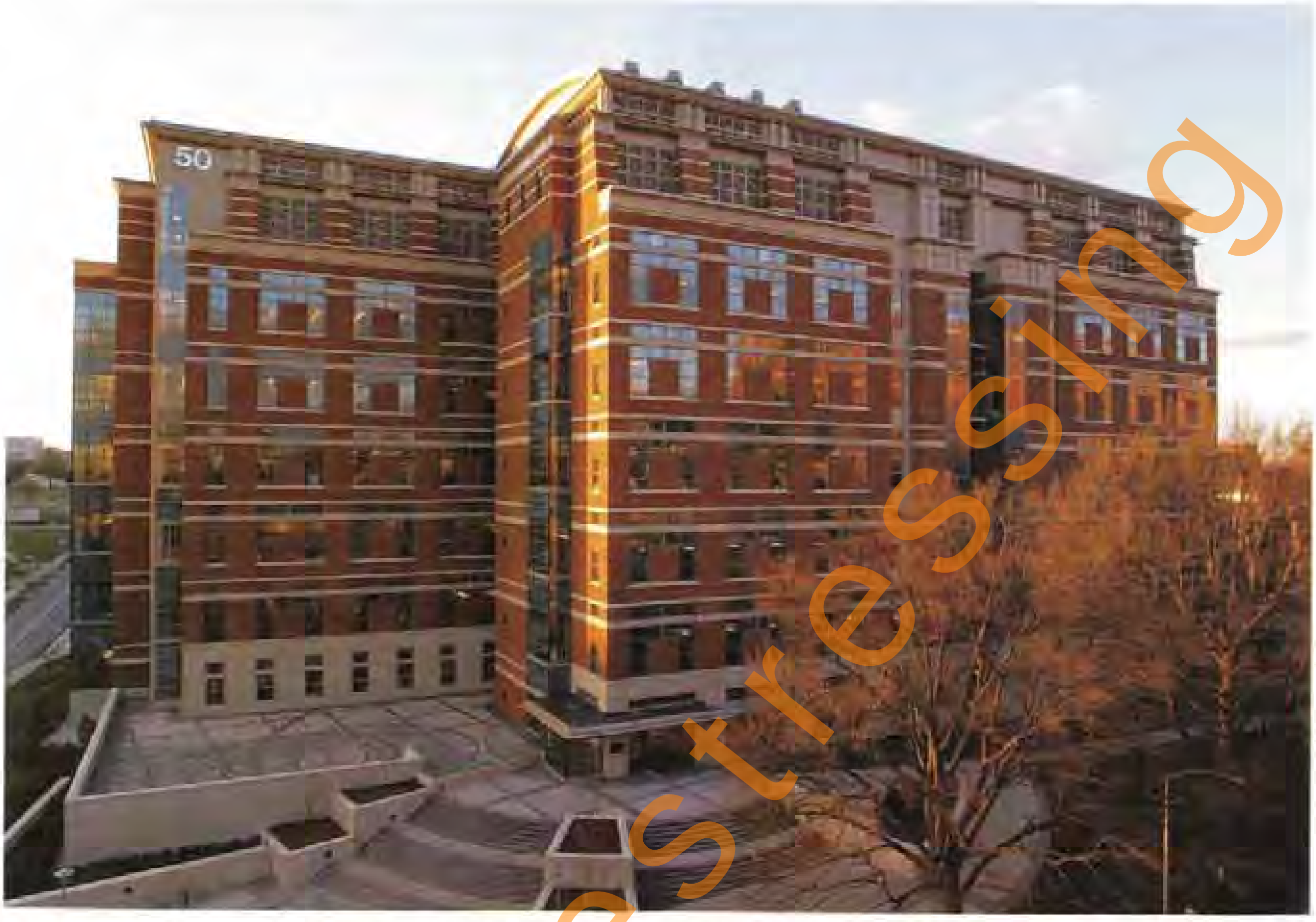


Fig. 2.21 National Institutes of Health - Louis Stokes Labs, Bldg 50, Bethesda, MD
Courtesy of Heery International Inc.; formerly HLM Design

2.2.6.4 National Institutes of Health - Louis Stokes Labs, Bldg 50, Bethesda, MD

The Louis B. Stokes Consolidated Laboratory is the first major design commission at the National Institutes of Health (NIH) Bethesda campus to be designed under the new guidelines and the recently completed Master Plan. The primary project goal for this state-of-the-art 294,000 sq ft biomedical research laboratory is to maximize the usable interior laboratory space to support the structural and cell biology research work of several institutes, including the National Center for Human Genome Research; Fig. 2.21.

At the beginning of the project a schematic structural matrix evaluation of five feasible structural systems was made. The five different structural systems evaluated were: a cast in place "skip joist" concrete floor system; waffle slab concrete floor system; unbonded post-tensioned concrete flat slab; and two separate composite steel systems (one with lightweight concrete, and one with normal weight). The evaluation factors, in order of importance used in the matrix, were: cost, vibration, flexibility, floor-to-floor height, and constructability. The use of a 9½ in. unbonded post-tensioned concrete flat slab with 9½ in. drops at columns proved to be the most favorable system, and it was proposed for the facility. After a great deal of discussion with NIH, a thorough peer review by an independent structural engineering consulting firm, and several vibration studies, it was accepted by the NIH and constructed.

Post-tensioning of the concrete allowed for a thinner concrete floor system, with less vibration, and less long-term deflection. The thinner slab allowed a lower floor-to-floor height, reducing the cost of the exterior skin of the building. This resulted in a floor system that cost less than a reinforced concrete floor system with the same 33-foot span distances. The post-tensioned floor system required 2.25 psf of non-prestressed steel, 1.02 psf of post-tensioning strand, and 0.66 cu ft/sq ft of concrete. A comparable reinforced concrete system would have required 4.3 psf of reinforcing steel and 0.94 cu ft/sq ft of concrete. The estimated savings with the post-tensioned system is in excess of $2 per sq ft.

2.3 PARKING STRUCTURES

Millions of sq ft of cast-in-place post-tensioned parking structures are built in North America every year. An independent survey of parking garage construction in the United States for the year 2000 showed that cast-in-place post-tensioned systems were the structural alternative most often selected.[2-1] The survey showed that post-tensioned structures had a significant lead in market share in terms of both number of structures built and volume (number of spaces).

Numerous post-tensioned parking structures built today are constructed as free-standing structures. Others are constructed as part of hotels, condominiums, apartment and office buildings, and other facilities. An excellent source of information on post-tensioned parking structures can be found in the PTI publication *Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures*.[2-2] The publication provides a comprehensive reference for the design, construction, and maintenance practices that will ensure long-term durability and minimize lifecycle costs of cast-in-place post-tensioned concrete parking structures.

The use of post-tensioning in parking structures offers several advantages, including initial and lifecycle cost savings, low maintenance costs, crack control and watertight structures, smooth riding surfaces, lighting and security, fire resistance, aesthetics, reduced structural depth, longer spans, deflection and vibration control, seismic resistance, durability, and structural integrity. The above advantages are discussed in more detail in Chapter 10.

The project examples listed below illustrate some of these advantages.

2.3.1 Commercial

2.3.1.1 McKinney Place, Houston, TX

McKinney Place is a 13½-story, 1214-space parking structure that has provided much-needed parking capacity in the heart of downtown Houston, Texas; Fig. 2.22. Upon completion, in a closeout meeting, the owner described the project as "an absolute homerun."

The floor structural system consists of one-way post-tensioned slabs and beams. The typical bay is 19 ft-10 in. by 58 ft-7½ in. The typical slab thickness is 5 in. and the typical beam size is 18 in. by 34 in. This results in a beam depth of 29 in. below the slab soffit. This same 29 in. soffit dimension was maintained throughout the building, even at the levels where the structure varied somewhat.

The use of post-tensioning allowed for the long spans required with this parking structure design. It facilitated the use of the typical garage beam forming system that is currently widely used. The project site covered half a city block, but was constrained by an existing building and three adjacent streets. Additional columns and a different structural system would not have been desirable.

Fig. 2.22 McKinney Place, Houston, TX
Courtesy of Walter P. Moore and Associates, Inc.

Initially, there were discussions about possibly designing McKinney Place as a precast parking structure. Then the well-known durability advantages provided by a cast-in-place post-tensioned parking structure were communicated to the owner. The owner agreed that cast-in-place post-tensioned design was the best way to proceed. The post-tensioning was fully encapsulated from end to end. This provided the owner with a durable structure that was flexible enough to accommodate future tenant needs.

The lateral load-resisting system utilizes frames in one direction and the ramps in the other, and the frames and ramps were designed accordingly.

2.3.1.2 Main Street Parking Garage, San Mateo, CA

The San Mateo Main Street Parking Structure is a 159,000 sq ft garage located in the city's downtown next to a commuter railroad right-of-way. The project included construc-

Fig. 2.23 Main Street Parking Garage, San Mateo, CA
Courtesy of International Parking Design, Inc.

tion of a 380-stall, five-level (one below grade) parking structure and improvements to a pedestrian area between the garage and the Century Theatres Complex; Fig. 2.23. The project also included the demolition of a 75,000 sq ft garage, construction of 3500 sq ft of ground-level retail space, installation of fire protection and security systems, and the removal and relocation of storm-drain sewer lines and other utilities located under Main Street between First and Third avenues. The final construction cost of the garage was $12,217,000, which included adjacent street improvements and landscaping as well as the Main Street pedestrian area. One of a kind, the project posed several key challenges.

The project site is located in the heart of the city's historic downtown and the parking structure was conceived to accommodate the growing demand for the revitalized downtown business sector and new multiplex theater. Extensive mitigation was necessary including sensitivity to construction noise, dust, and vibration. Demolition of an existing parking structure had to be done with great care in order not to damage any of the nearby buildings. The exterior design elements of the parking structure were designed to complement the theater and other surrounding structures. This was achieved by using painted stucco, precast concrete, and ceramic tile with metal decorative trim and light fixtures specially designed for the project. Other design features were utilized in conjunction with city lighting, signage, and site hardscape standards.

2.3.2 Airports

2.3.2.1 Sky Harbor Consolidated Rental Parking Structure, Phoenix, AZ

The city of Phoenix consolidated rental car center is a 2.5 million sq ft space with 40,000 parking spaces. When completed, it will be Arizona's largest building. The project is just west of Interstate 10 on Lower Buckeye Road.

Located under the flight paths of one of the nation's busiest airports, the project couldn't go too high. At the same time, designers needed to preserve room for 40,000 covered parking spaces and nine free-standing rental car service centers. The design was based on getting the smallest footprint on the site and limited by FAA height restrictions. From the air, the consolidated rental car facility resembles a person with outstretched arms holding a large ball; see Fig. 2.24. The "arms" are a half-circle elevated roadway that allows the ferry buses to pull up, unload the customers and continue down the ramp back to the airport.

Designers created massive 61 by 61 ft bays supported by columns and beams. A 5 in. post-tensioned slab is used between the beams. The design of the project was very complicated and large transfer beams were necessary to distribute the load. One large, 85 ft long beam, dubbed the "Barney Beam" for its purple color-coding in the plans, supports a large area with heavy bus loading. The beam is 7 ft-0 in. wide and 6 ft-0 in. deep heavily reinforced with

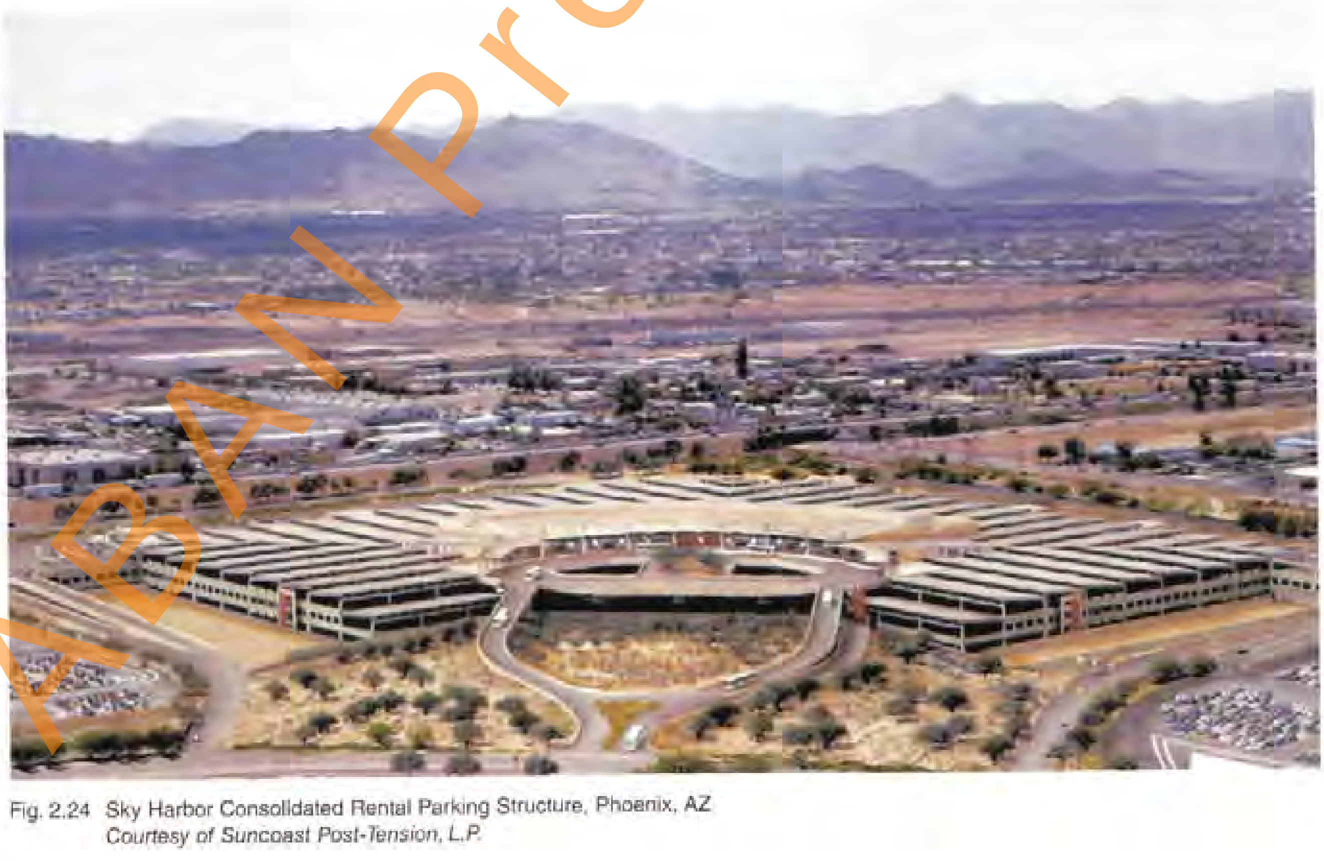


Fig. 2.24 Sky Harbor Consolidated Rental Parking Structure, Phoenix, AZ
Courtesy of Suncoast Post-Tension, L.P.

115 tendons. Other critical design factors included concerns about vibration from the large buses and the expansion and contraction of such a large concrete structure.

The project used 130,000 cubic yards of concrete, 10,000 tons of rebar, and 760 miles of post-tensioned tendon. The total project cost was $256 million.

2.3.2.2 BWI - Consolidated Rental Parking, Baltimore, MD

Located in the center of the Washington/Baltimore corridor, Baltimore Washington International Airport (BWI) is one of the fastest-growing airports in North America and serves as a powerful economic engine for the region. By the late 1990s, this rapid increase in service led to growing concerns about parking availability at the airport. Subsequently, a five-year improvement plan to upgrade the airport's functionality was announced that included provisions for a new state-of-the-art parking garage which would consolidate the airport's eight rental car companies into one central location; Fig. 2.25.

The new parking facility includes approximately 3.5 million sq ft of parking space. More than 1 million sq ft of the structure consists of elevated, post-tensioned, cast-in-place concrete. The construction team for the new garage presented the owner, the Maryland Aviation Administration, with the most advanced techniques and materials wherever possible. This included a cast-in-place concrete system, which featured bonded post-tensioning. This system was chosen based on the owner's need for maximized durability and minimized maintenance over the structure's projected life cycle.

The post-tensioning system worked particularly well with garage functionality requirements. While standard commercial parking garages typically have 20 ft to 30 ft wide by 54 ft long bays, the column grid spacing for this project was increased to 60 ft by 60 ft to provide each car rental group with as much clear space at the bottom level as possible for staging and multi-directional traffic. Additionally, the floor-to-floor height was increased to 19 ft in order to provide customers with the feel of an open structure.

Another interesting feature of the project was concrete placement sequencing. The original design layout called for 93 concrete placements at approximately 10,000 sq ft each. Based on past experience and careful pre-planning, the team opted to increase the placement size to 33,000 sq ft. Each placement required approximately 1100 cubic yards of concrete. This change provided multiple benefits that improved the schedule. The number of concrete placements was reduced from 93 to 36. Associated bulkheads and joints were eliminated, and the tendon stressing and grouting operations were more streamlined.

The facility, which offers customers one-stop rental car shopping, holds 8300 spaces and frees up more than 1000 prime parking spaces in BWI's terminal parking garage. More than 73,000 cubic yards of cast-in-place concrete

Fig. 2.25 BWI – Consolidated Rental Parking, Baltimore, MD
Courtesy of VSL

were used to construct the foundations and superstructure during the two-year, fast-track construction cycle. Careful pre-planning, pre-construction mockups and collaborative input by all project participants were the keys to success on this post-tensioning project.

2.3.2.3 Humphrey Terminal, Minneapolis, MN

With double-digit air travel demand increases, the Metropolitan Airports Commission (MAC) embarked on the construction of a new terminal at the Minneapolis-St. Paul International Airport as one of the cornerstones of a multi-billion dollar expansion program to serve the Twin Cities into the new millennium. Air travel and parking demand studies revealed a major parking capacity shortfall due to the "buildout" of parking capacity at the Lindbergh Terminal. Results of these studies indicated the need to provide a parking and multi-modal transportation facility consisting of:

- 400 stalls of valet parking
- 700 stalls of short-term parking
- 9200 general or long-term parking stalls
- 1900 stalls for employee parking
- Transit center-serving shuttles, taxis and buses
- Light-rail transit station providing intermodal passenger and employee transfers

The structure decision matrix evaluated a number of project elements, including maintenance costs and structure flexibility. This process resulted in the selection of a cast-in-place post-tensioned concrete parking structure. Concrete utilizing 5 percent silica fume was chosen for incorporation into the concrete mix design to provide reduced maintenance costs and a long life.

The eight-story structure consists of 7 in. thick slab supported on 33 in. deep beams spaced 27 ft on center; Fig. 2.26. Plans also call for a new light-rail transit station to connect the parking structure and the terminal.

The consultant led a large, diverse team charged with developing innovative solutions for a complex project within a constrained timeframe. The result is the new high-tech Humphrey terminal parking and transit center. The facility incorporates state-of-the-art construction features for a user-friendly facility that will weather Minnesota winters and provide low service maintenance costs as travel demands continue to increase.

2.3.3 Underground Parking Structures

2.3.3.1 Underground-Entry Pavilion, CA

In this unique project, a multi-level, cast-in-place post-tensioned concrete parking structure was placed completely underground, and will be concealed by the re-installation of the original landscaping. When completed, the parking structure is virtually invisible. Innovative post-tensioning

Fig. 2.26 Humphrey Terminal, Minneapolis, MN
Courtesy of SRF Consulting Group, Inc.

Fig. 2.27 Underground-Entry Pavilion, CA
Courtesy of Watry Design, Inc.

design solutions were required to maintain efficient use of the underground space and economy in the design.

The building geometry is 202 ft by 275 ft in plan, with three below-grade parking levels and a plaza at grade, for a total of 220,800 sq ft. The typical parking decks have 6 in. one-way concrete slabs that span 20 ft-0 in. between 14 in. by 36 in. long-span beams. The plaza framing follows this bay spacing, but with an 8 in. deck on 24 in. by 48 in. beams. Perimeter concrete walls provide soil and lateral support. Fig. 2.27 shows the pavilion during construction.

Post-tensioning provides many advantages for parking structures, including relatively shallow floor-to-floor heights and long column-free spans. These benefits were critical on the parking levels of this project, as well, making post-tensioning a logical choice. Reduced floor-to-floor heights in an underground building provide direct savings in excavation, temporary shoring, and retaining wall costs.

In order to economically install this post-tensioning system underground, an internal stressing scheme was developed. The beams were stressed along a closure formed through the central ramp bay. This unique approach created beams that are essentially cantilevered in the center of the building; the closure strips, which were poured later, do not contain tendons.

A second challenge in the underground assembly is the presence of stiff perimeter walls that resist diaphragm shrinkage. To minimize cracking under these conditions, the connections from the slabs and beams to the walls were delayed. Slab closures provided temporary separations from the bearing walls. Columns that support the beams were poured in normal sequence, but the in-fill wall panels were installed a minimum of 45 days later. Close coordination with the contractors ensured minimal cost impact for these unusual designs and sequencing.

Post-tensioned concrete construction is well suited to variability in form. This project used custom beam sizes to maximize the load-carrying capacity of the members within the restricted height. The parking decks were warped for adequate drainage. The plaza level reinforcement was customized to support the precise load patterns of fire truck accessible roads, several 45,000 lb magnolia trees, and extensive planting.

On this project, post-tensioning helped achieve architectural effect as well. The entry is marked with a slender, post-tensioned cantilevered frame. A post-tensioned walkway canopy will be added to the above-ground landscape. The thin canopy profile is achieved with a double-cantilevered, post-tensioned form designed specifically for this site. The elegance of this canopy feature reflects the creative potential of post-tensioned design.

2.3.4 Mixed Use

2.3.4.1 The Pike at Rainbow Harbor Parking Structure, Long Beach, CA

The Pike Parking Structure fulfilled a long-sought goal of the Queensway Bay Plan, envisioned by the City of Long Beach over a decade ago: to link the vibrant nightlife of Pine Avenue and the Convention Center activity with the serene beauty of the Aquarium of the Pacific and the colorful boats coming and going from Rainbow Harbor. The parking structure assists in tying together the elements to form a unique entertainment district, serving both visitors and residents. The entertainment center includes a 14-screen multiplex theater, GameWorks, The Laugh Factory, a nightclub, and a wide variety of restaurants.

The seven-level parking structure provides 2211 spaces for shops and visitors. To ease parking and keep traffic flowing efficiently, circulation is one-way and spaces are angled at 70 degrees. The 10 entry/exit lanes flow by means of the circular ramp at the west end of the structure, allowing the structure to be filled or emptied quickly. This 475-foot-long cast-in-place post-tensioned concrete structure is designed in a "streamlined moderne" style with metal panel accents similar to those used on the Aquarium of the Pacific Parking Structure located just across the street. At the east end, the six geared elevators form a 140-foot-tall tower, foot-wide starburst, echoing the shape of the starburst light; see Fig. 2.28. At the Pike's west end, the spiral exit ramp is accented with blue lights and a circular triple-tiered tower that caps the spiral ramp reaching 115 ft.

Fig. 2.28 The Pike at Rainbow Harbor Parking Structure, Long Beach, CA
Courtesy of International Parking Design, Inc.

2.4 BRIDGES

From the inception of prestressing in Europe, a primary application has been the construction of prestressed concrete bridges. Today, the use of post-tensioning is considered to be a major factor in the design and construction of concrete bridges in North America. Control of cracking, reduced structural depth, ease of accommodating curved roadway alignment, durability, low maintenance costs, and potential for economical construction of long spans are some of the more important reasons for the rise in the use of post-tensioning in bridge construction.

2.4.1 Cast-in-Place Box Girders

Post-tensioned box-girder designs have become the dominant form of bridge construction in California. The longer spans (commonly up to 300 ft or more), shallower girder depths, and continuously curved superstructures made available by post-tensioning provide superior aesthetics to precast designs. Post-tensioning also provides more flexibility in the layout of span lengths, bent configurations and roadway geometrics than precast I-girders or reinforced concrete box girders. Post-tensioning combines longer spans with the ductility and seismic performance of cast-in-place construction.

In addition to the economic and technological advantages related to this type of construction, the aesthetic potential of cast-in-place prestressed concrete bridges is unsurpassed. One of the most important aspects of good total design is the appearance of the structures within a project. It is generally accepted that concrete is superior in appearance, and blends more readily than do other types of construction. Cast-in-place concrete, particularly box-girder construction, allows the use of architectural treatments, such as curved surfaces and finishes that enhance the appearance of the structure. The internal voids in box girders provide an excellent place to conceal unsightly utilities that detract from the appearance.

Following are several project examples for some of the major cast-in-place bridge work carried out in North America.

2.4.1.1 SR-125/SR-94 Interchange, San Diego County, CA

The project is a four-level freeway interchange, connecting the new six-lane State Route 125 and existing State Route 94 in San Diego County, Calif., with a footprint that includes right-of-way in three separate communities: La Mesa, Lemon Grove and Spring Valley. The interchange contains seven individual structures with a total length of 9900 ft, more than a half million sq ft of deck, and columns exceeding 109 ft in height. There are a total of 47 individual spans, ranging from 114 to 291 ft in length; Fig. 2.29.

The entire project is constructed of cast-in-place concrete. The superstructures are post-tensioned box girders with integral bent caps and hinge diaphragms. The deck, soffit slab and most bent caps are reinforced concrete; however, the design includes large (up to 72 ft long), post-tensioned, highly-skewed outrigger bents.

Built in an existing urban setting, the interchange sits above a commercial district, and is surrounded by subdivisions with residential properties both below and overlooking the site. Residents were particularly concerned about visual impacts. The long, graceful spans and clean lines available with post-tensioned box girders contributed to the development of attractive layouts, themes, and architectural details for the bridge structures. With the use of post-tensioning, the engineers were able to develop structural designs with superior seismic performance, roadway geometrics, and aesthetic appearance. The result is a facility which meets the needs of the traveling public as well those of the surrounding community.

2.4.1.2 Camino Ruiz, San Diego County, CA

Winding north through Black Mountain Ranch from Carmel Valley Road, the Camino Ruiz Bridge will serve as the major thoroughfare for the planned development and open space of over 5000 acres. Gracefully reaching more than 400 ft across a wildlife corridor, with a height over 50 ft, the Camino Ruiz Bridge serves as a critical infrastructure component for this development.

Fig. 2.29 SR-125/SR-94 Interchange, San Diego County, CA
Courtesy of Dokken Engineering

Fig. 2.30 Camino Ruiz, San Diego County, CA
Courtesy of T.Y. Lin International

The bridge's unique structural design provides the community with an architectural landmark; Fig. 2.30. The three-span, post-tensioned concrete bridge utilizes a parabolically haunched superstructure, tapering to a dramatically thin 4 ft-6 in. while providing a main span of 160 ft. The structure uses extremely long 11 ft overhangs on each side of the box girder. The long overhangs make up nearly half of the overall bridge width, thereby keeping the box girder width narrow and sleek.

The bridge column configuration is also unique to this bridge. In the transverse direction the columns flare to meet the descending haunch of the superstructure and sloping exterior girders of the box. Not only does this gradual flare provide aesthetic continuity of flow between the superstructure and columns, it allows the column to taper to a relatively narrow base. In the longitudinal direction the columns are a thin, prismatic 4 ft, keeping the architecture simple and clean.

2.4.2 Segmental

Cast-in-place (CIP) segmental construction was first developed in Germany by Dyckerhoff & Widmann in the 1950s immediately after World War II. The new, innovative construction method led to construction of concrete bridge spans in excess of 700 ft (210 m). The technology for cast-in-place construction was adapted and extended for use with precast segments in the Choisy-le-Roi Bridge over the Seine River south of Paris in 1962.

CIP segmental cantilever is most advantageous when the use of falsework is impractical and the bridge is built over a waterway channel. In this scenario, the cantilever method with cast-in-place segments provides an economical solution. CIP construction also reduces the high degree of dimensional control needed during the manufacturing and construction process for precast segmental bridges.

Cast-in-place segments are usually 10 to 13 ft long, and are tied to adjoining segments via post-tensioning tendons. Pier segments are 20 to 30 ft long and are cast (or placed) first, typically in smaller segments due to the large weight of a pier segment. Balanced cantilevers using traveling forms are typically used in CIP segmental bridge construction. Traveling forms at the tips of the cantilevers move in opposite directions at the same rate to cast new segments, thereby maintaining balance and stability. The result is a cantilever that grows in length from the pier as it approaches midspan. The maximum length of any given cantilever will be approximately half the span length between any two successive piers. During construction, tendons are placed near the top fiber to support the cantilevered self-weight of the segments plus construction dead load. Fig. 2.31 illustrates the Balanced Cantilever method.

The use of precast segmental box girders erected in cantilever (without falsework) combines many of the advantages of CIP post-tensioned bridges with the potential of remarkably speedy construction. A major innovation for construction of precast segmental bridges was the introduction of the launching gantry in the mid 1960s. The gantry makes it possible to move segments over the completed part of the structure and place them in cantilever over successive piers. In this case, an overhead truss or "launching girder," riding above the superstructure pier, places the precast segments, alternating from one cantilever to the other in order to balance loads. Erection in many cases can also be accomplished by the use of cranes.

Fig. 2.31 Balanced Cantilever Method

The advantages of precast segmental bridge construction include:

- The economy of precast prestressed concrete construction is extended to a span range of 100 to 400 ft (30 to 120 m). Longer spans may be economical where use of heavy erection equipment is feasible.
- Precast segments may be fabricated while the substructure is being built, and rapid erection of the superstructure can be achieved.
- Higher quality control because of the repetitive industrialized manufacturing techniques, with the inherent potential of achieving high-performance concrete.
- The need for falsework is eliminated and all erection may be accomplished from the top of the completed portions of the bridge. This may be of particular importance for high-level crossings or in cases where it is necessary to minimize interference with the bridge environment.
- The effects of creep and shrinkage are substantially minimized through the use of precast segments that have matured to full strength.

The contractor and/or manufacturer must exercise a high degree of dimensional control during the manufacturing and erection of the precast segments. The following project examples illustrate the application of CIP and precast segmental cantilever construction in various parts of the country.

2.4.2.1 Sagadahoc Bridge, Bath/Woolrich, ME

Sagadahoc Bridge provides an attractive means to link the City of Bath and the Town of Woolwich, Maine; Fig. 2.32. The former lift-span bridge was outdated and created extensive traffic delays during the summer tourist season, and daily during shift changes at Bath Iron Works, Maine's largest employer. With the opening of the new bridge, the time required to disperse traffic at shift changes has shrunk from one hour to just seven minutes and the city no longer needs to supply police for traffic control. The new high-level structure also means the end of delays for ship traffic.

The main span of Sagadahoc Bridge is 420 ft, a U.S. record for a precast concrete segmental span built in balanced cantilever. The long span was desirable in order to minimize the number of piers in the river, as well as the location of the navigational channel. A two-cell precast segmental box girder was used to carry the bridge deck over the river. Segments were up to 69 ft wide and weighed 100 tons. However, this eliminated half of the precasting and erection operations and expense. A casting yard was established upriver from the bridge site in an enclosed building, allowing superstructure production to continue through the harsh Maine winter. To avoid the expense of a temporary bridge a large portion of the first span was completed in a second phase, after the traffic moved to the new bridge and the upper deck of the old bridge was demolished. The bridge is post-tensioned both longitudinally and transversely to zero-tension criteria, preventing cracking. Both longitudinal and transverse post-tensioning tendons were overlapped in anchor ribs at the construction joints to allow for phasing.

Fig. 2.32 Sagadahoc Bridge, Bath/Woolrich, ME
Courtesy of Figg Engineering Group

Through engineering excellence, Sagadahoc Bridge provided a bridge that is also aesthetically pleasing. The completed cost of the project was $3 million less than the Maine DOT budget. In addition to setting a record for span length for this structure type, this combination of engineering, aesthetics and financial responsibility has set a high benchmark in civil engineering.

Fig. 2.33 Broadway Bridge, Daytona Beach, FL
Courtesy of Figg Engineering Group

2.4.2.2 Broadway Bridge, Daytona Beach, FL

Broadway Bridge carries U.S. No. 92 (International Speedway Boulevard) over the Intracoastal Waterway, linking the famed speedway with the beaches. The Orlando Sentinel has referred to the new Broadway Bridge as "Daytona Beach's newest permanent art exhibit." The 3008 ft precast concrete segmental bridge was dedicated on July 20, 2001. The bridge is comprised of twin parallel structures that elegantly cross the Halifax River, which serves as the Intracoastal Waterway at this point; Fig. 2.33. Broadway Bridge replaces a 50-year-old, functionally obsolete bascule bridge that restricted vehicular and boat traffic and had become too expensive to operate and maintain. The new bridge features 262 ft spans and a 65 ft vertical clearance, allowing vehicular and boat traffic to move freely.

The Broadway Bridge has total deck square footage of 260,152 sq ft. Segments were cast in Flagler Beach, Fla., and barged 20 miles down the Intracoastal Waterway to the site. The 352 segments that comprise the bridge are 48 ft wide and vary in depth from 13 ft to 7 ft-9 in. The elliptically shaped piers are cast-in-place. Due to the existing street elevation at the western landing of the bridge, a 34,500 sq ft cast-in-place flat slab with bi-directional post-tensioning was constructed. The bi-directional post-tensioning limits cracking, reducing intrusion in the harsh saltwater environment. The flat slab transitions to concrete segments once adequate clearance is achieved above the mean water levels. The Broadway Bridge was built as a balanced cantilever.

As a precast concrete segmental bridge, the Broadway Bridge will be durable and easily maintained for many years despite the corrosive salt water environment of the coastal waterway. High performance concrete was enhanced with fly ash and calcium nitrate for increased durability. Segmental precast concrete construction was selected for the Broadway Bridge based on its durable nature, along with the economy and speed of construction.

Broadway Bridge has been warmly embraced by the community of Daytona Beach. The growing importance and emphasis on bridge aesthetics is well demonstrated in the design of Broadway Bridge. All elements of the bridge—the shapes, color, lighting, gateway entrance, pedestrian walkways and more—were designed to be in harmony with the waterfront and to project the community's image of itself.

2.4.2.3 JFK Air Train, New York, NY

The JFK Airport Light Rail System includes the construction of six new airline terminal stations, connected by 11 miles of rail, nine miles of which are elevated on a precast concrete segmental structure; Fig. 2.34. The elevated structure is comprised of 461 spans, using 5408 precast concrete segments—the most of any bridge in America. This light rail system allows passengers to travel between airport terminals (Central Terminal Area), long term and employee parking (Howard Beach) and rental car facilities (Federal Circle). The elevated structure also crosses over and runs within the 7 ft-8 in. median of the Van Wyck

Fig. 2.34 JFK Air Train, New York, NY
Courtesy of Figg Engineering Group

Expressway, a six-lane congested urban highway. The Van Wyck portion is 2.3 miles in length and connects the system to Jamaica Station (hub for the Long Island Railroad, New York Subway System and 40 bus lines). It has been estimated that as many as 34 million passengers will use the rail system each year via 32 driverless, environmentally friendly trains. Construction of the superstructure was completed in August, 2001, three months ahead of schedule.

The JFK Airport Light Rail Transit system is unique in many ways. The successful progress of this complex project is a direct reflection of the teamwork and unity developed and enjoyed by the designers, contractors and owner.

2.4.2.4 Lower Screwtail Bridge, Beeline Highway, AZ

Lower Screwtail Bridge, with a total length of 1080 ft, was the first cantilevered cast-in-place concrete segmental bridge built in Arizona; Fig. 2.35. It is part of an approximately $30 million ADOT contract, budgeted for the construction of Mesa-Payson Highway (SR-87), 60 miles northeast of Phoenix. The four-span bridge was built above a rugged, 125-foot-deep canyon in a very environmentally sensitive area, home of endangered wildlife and at the intersection of two historic roads.

Fig. 2.35 Lower Screwtail Bridge, Beeline Highway, AZ
Courtesy of DYWIDAG-Systems International, USA

The bridge was designed as a cantilever structure during construction, where the two piers supported the weight of concrete segments, two form travelers and live construction loads. Cantilevers on each side of the pier were created with 11 bridge segments, each segment being 16 ft long, 45 ft wide and 10 to 19 ft deep. One unique feature of the project was the 4.5% superelevation and a 1200 ft horizontal radius

Typical segmental construction started after the pier was poured and ready to support load. A single-box (one-cell) form traveler was placed on each side of the pier table. Self-weight of the structure was supported by installing rebar and short post-tensioning tendons, transversally (top slab) and longitudinally (top and bottom slab) of the bridge. Once the concrete was poured and had reached required strength, the post-tensioning tendons were stressed over the pier and the form travelers moved forward to the next segment. Symmetrical placement of form travelers to each side of the pier table was essential for maintaining construction balance. The construction cycle for each segment was typically one week. Longitudinal post-tensioning tendons were also installed inside the structure two-webs, for tensioning at the abutment locations after all four spans were completed.

2.4.2.5 Acosta Bridge, Jacksonville, FL

The 1645-foot long Acosta Bridge over the St. Johns River in downtown Jacksonville, Fla. was designed as a cast-in-place concrete segmental continuous structure built in cantilever with a variable depth and arched soffit, the longest of its kind in the United States; Fig. 2.36. Construction was by balanced cantilever, casting segments in two directions from each pier table, alternating from one side to the other in small increments to maintain balance. The design intent was to place the river pier foundations in shallow 35-foot depth water and avoid 90-foot depths in the center of the river, yet provide a 75-foot vertical clearance for watercraft in high-tide water at the navigation channel, which is not in the usual center of the riverbed but hugging the north shore. The south channel pier haunch is 38 ft deep and the north (navigation) channel pier haunch is 28 ft deep.

Designing a replacement bridge constructible in multiple stages within established right-of-way while accommodating various temporary detour patterns, all within a congested downtown setting, was a challenging feat. Construction of the new bridge and demolition of the old bridge took place immediately adjacent to a major railroad trestle, adding to the challenge. Vehicular traffic crossing the river at this location could not be interrupted; therefore, the first half of the new span was constructed. Traffic transferred to this new span, the original bridge was demolished, and the new bridge completed. This required an innovative plan for maintaining traffic flow on adjacent downtown streets and associated ramps, and following a detailed procedure for the demolition of the old bridge so

Fig. 2.36 Acosta Bridge, Jacksonville, FL
Courtesy of T. Y. Lin International and Fred Wilson & Associates, Inc.

as not to adversely impact the first half of the new structure, nor the adjacent railroad bridge.

The connection of the new bridge to the downtown at grade street network required a four-level interchange ramp system at the north end of the project. These ramps were designed and constructed to provide proper clearances and grades for the subsequent construction of the Automated Skyway Express downtown people mover system, which was constructed within the restricted median area of the new bridge.

2.4.2.6 Tsable River Bridge, Vancouver Island, BC, Canada

The bridge is located about 22 kilometers south of Courtenay on the Inland Island Highway in British Columbia. Tsable is the largest bridge on the island, as well as the most expensive. Given the enormous challenges of the $15 million project, it is even more notable that it came in on time and under budget.

The different concrete structural schemes investigated include arch; cable-stayed; a three-span, balanced cantilever, cast-in-place concrete box girder; a four-span, balanced cantilever, both precast and cast-in-place concrete box girder; and a six-span incrementally launched concrete box girder bridge. The four-span, cast-in-place concrete box girder, built using the balanced cantilever method, emerged as the most cost-effective concrete solution for this crossing.

Building the structure involved building a 400-meter deck using the cast-in-place balanced cantilever method. The bridge was essentially built outward from two piers and the end abutments until the four spans joined. Concrete was alternately poured into a cell box segment that spanned out from either side of the pier tops supported by a "traveling form." To pour and cast a subsequent segment, the traveling form was anchored to the previously cast segment. During construction, once a segment had cured as a hollow concrete cell, attached post-tensioning strands were used to join the segments together. The cross-section of the Tsable Bridge has the largest single-cell trapezoid box in North America.

Several environmental factors played into the type of bridge selected for design and construction. One important factor was minimizing the number of trees that would be removed for construction; Fig. 2.37. In addition, the river itself is a vital salmon spawning watercourse. To protect the fish habitat, work affecting the foundations at Pier 2 and Pier 3 could only be performed from June through September. The pier foundations and temporary work areas

Fig. 2.37 Tsable River Bridge, Vancouver Island, BC, Canada
Courtesy of T. Y. Lin International

had to be set back from the main and side channel banks by about 10 meters. These reasons, and others, predicated the design of a cantilever single-cell box girder bridge.

2.4.2.7 Smart Road Bridge, Blacksburg, VA

Located outside of Blacksburg, Va., the 1985 ft Smart Road Bridge is a cast-in-place concrete segmental box-girder bridge with 472 ft spans that was built in balanced cantilever with form travelers. With the bridge deck 175 ft above the valley floor, the Smart Road Bridge is the tallest bridge in the Commonwealth of Virginia: Fig. 2.38. The bridge connects to the previously completed 1.7-mile section of roadway that was the first phase of the project. These two phases of the Smart Road project serve as a state-of-the-art test bed for transportation research such as advanced communications systems, variable message signs, experimental pavements, and intelligent transportation systems. The facility has the capability to simulate variable weather conditions, using the 75 snow-making towers to create fog, rain or snow. Combined with the ability for variable lighting conditions, vehicle and driver reactions to a variety of conditions can be monitored with the extensive monitoring equipment located along the roadways.

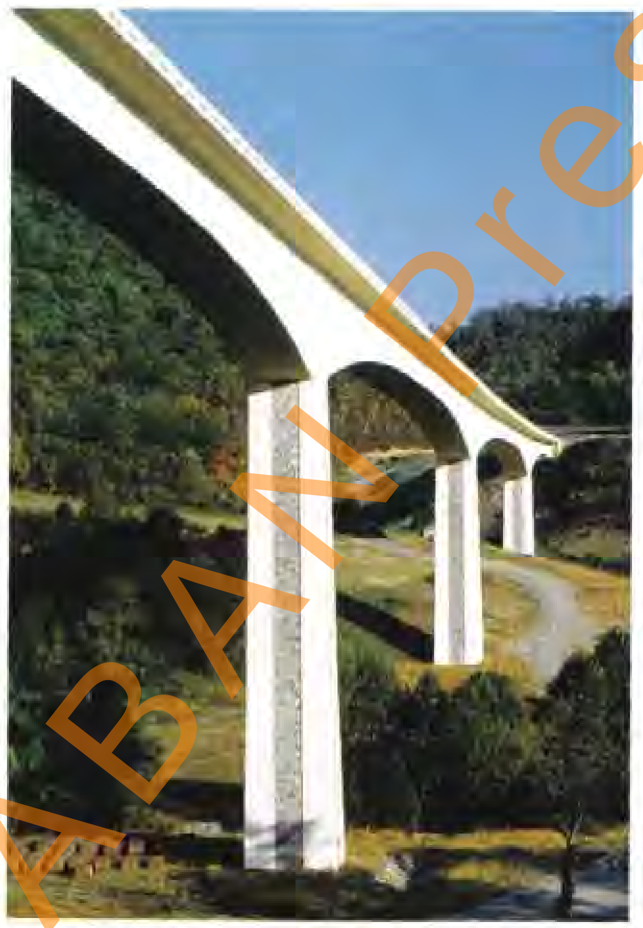


Fig. 2.38 Smart Road Bridge, Blacksburg, VA
Courtesy of Figg Engineering Group

Once the third and final phase of the project is completed, the 5.7-mile Smart Road will provide a public and vital link to Interstate 81 from the town of Blacksburg and Virginia Tech. The Virginia Department of Transportation (VDOT) determined that a concrete segmental bridge was the preferred construction technology from aesthetic, economic and maintenance points of view. An added benefit of the open box girder is that it could be used to accommodate the various testing equipment associated with the mission of the Smart Road.

The bridge consists of a single continuous unit, with expansion joints located only at each of the ends. In addition to the reduced number of bearings, this further reduced the amount of future maintenance. Steel finger joints were used to accommodate large movements at these locations. This minimal number of expansion joints also allows traffic to pass over the bridge at low noise levels, which is desirable in this rural valley. Both the substructure and superstructure of the bridge consist of high-performance concrete with low permeability and a compressive strength of 8000 psi. For the superstructure, longitudinal post-tensioning tendons with 19 ft-0.6 in. diameter strands were used for both the top slab cantilever tendons and the bottom slab continuity tendons. There are 18 cantilever tendons per web in each cantilever. The interior spans have 11 continuity tendons per web anchored in blisters on the bottom slab. The exterior spans have four bottom slab tendons per web. All the superstructure reinforcement is epoxy coated. Reinforcement in the piers and footings is black, non-coated steel.

The Smart Road Bridge was designed and constructed as a post-tensioned segmental bridge to satisfy the functional, aesthetic and economic requirements of the project. The combination of functional goals was exceeded by this new concrete bridge, which also met the demanding construction challenges of the rural site without adversely impacting the setting. The ability to house monitoring and testing equipment in the bridge was beneficial, along with the ability to support research on the bridge in the future. In addition to the functional requirements, it was important that the bridge not intrude upon the beauty of the Ellett Valley, but instead complement the surrounding area.

2.4.3 Cable-Stayed Bridges

Cable-stayed bridges and suspension bridges may seem similar as both have superstructures that hang from cables, and both have towers. But the two bridges support the superstructure in very different ways, and depend upon how the cables are connected to the towers. In suspension bridges, the cables ride freely between the towers, transmitting the load to the end anchorages. In cable-stayed bridges, the load in the cables is directly transferred to the towers.

Cables in cable-stayed bridges can be attached to the bridge superstructure in a variety of ways, including radial and par-

Fig. 2.39 Grijalva River Bridge, Villahermosa, Tabasco, Mexico
Courtesy of Mexpresa

allel patterns. In a radial pattern, cables extend from several points on the bridge superstructure to a single point at the top of the tower. In a parallel pattern, cables are attached at different points along the tower and run parallel to one another.

The use of cable-stayed superstructures has made possible dramatic increases in the economical span range of concrete bridges. They are cost effective because of the inherent material efficiency of the high-strength tension elements. Cable-supported bridge decks can bridge long spans without the need for intermediate piers. Main spans of up to 1200 ft (365 m) are not uncommon, and record construction times of less than a year have been accomplished.

Cables used in such bridges have evolved from the technological advances in post-tensioning. High-strength prestressing steel wires, strands, and bars are the prevalent materials for modern cable stays. Their anchors and stressing techniques have been adapted from existing techniques used for post-tensioning purposes.

Most modern cables are proprietary developments of the major post-tensioning system suppliers. Their basic components (i.e., strands, bars, wires, and anchors) must satisfy special stay cable performance requirements against fatigue and environmental protection.

2.4.3.1 Grijalva River Bridge, Villahermosa, Tabasco, Mexico

The 2001 finished cable-stayed bridge over the Grijalva River is part of the extension of an existing major boulevard in the city of Villahermosa; Fig. 2.39. Architectural requirements restricted the height of the new bridge to that of an existing three-span bridge built in the 1950s. However, the State of Tabasco wanted the new bridge to cross the river in a single span without supports in the river. The cable-stayed approach was selected for the project. This resulted in a streamlined slender structure that was both functional and architecturally appealing. The two inclined pylons in a V shape make the structure slender and provide a unique architectural character to the bridge.

The post-tensioned pylons support a 0.8 m deep, 240 m long deck through cable stays. The deck, consisting of a slab over longitudinal post-tensioned edge beams and transverse post-tensioned ribs, is longitudinally fixed to one pylon and free to move over the rest of supporting points. This was important in order to reduce the seismic forces. The stay-supported section is divided into a 116 m long main span plus two lateral sections, each divided into 37 plus 25 m long spans by torsion-control bents.

The cable stays consist of 23 to 28 parallel 0.6 in. strands each individually protected against corrosion with two nested barriers: epoxy coating-fill plus HDPE sheathing. Every strand bundle is covered by a thermo-elastic band, helically wrapped, which protects it against UV rays, temperature changes, fire, dust and rain, while allowing air circulation. The helical shape of the band is also a path for raindrops to gradually move down. Saddles were used to cross the slender pylon columns. They included individual tubes and a system to attach the stays to the pylons to transfer the unbalanced loads.

The design and construction of the bridge, under a fast-track scheme, took only 15 months. All concrete was cast in place with traveler forms and temporary stays for the main section. This proved to be a cost-effective solution, contributing to the total cost of only US $135 per sq ft for the entire structure.

2.4.3.2 Puente de la Unidad (Bridge of Unity), Monterrey, Mexico

As part of a project to provide a new transportation corridor in Monterrey Mexico, the State of Nuevo Leon decided to build a cable-stayed bridge across the Santa Catarina River as the main component of the elevated highway. The river is normally calm, but it is prone to flooding in large storms. The desire was to cross the channel with a single span, but also take the opportunity to build a landmark structure. The result was the Puente de la Unidad, which is Spanish for "Bridge of Unity"; Fig. 2.40. The bridge aes-

Fig. 2.40 Puente de la Unidad (Bridge of Unity), Monterrey, Mexico
Courtesy of International Bridge Technologies, Inc. and VSL Mexico

thetics were a major concern for the owner. In particular, a concrete mix with white cement and aggregates was used for all visible parts of the bridge. This structure completes the ring road around Monterrey and will certainly become a new landmark for this modern industrial city.

The total length of the cable-stayed bridge is 997 ft. This bridge features a unique asymmetrical structure with a 186 m main span; a 60-degree-inclined central pylon with two planes of stay cables; deck width varying from 33 m to 24 m; erection in one-directional cantilever using a moveable scaffold; complex pylon straddling four lanes of traffic; a pedestrian promenade along the centerline of the bridge with access through the pylon; and on- and off-ramps merging the main roadway on both sides of the pylon. Nearly all of the main structural components were made of high-performance 7250 psi concrete. Thirteen pairs of harped stay cables support the main span and are anchored on the outside of the roadway. One pair of vertical stays is anchored into the back of the pylon. The main span carries two lanes of traffic in each direction with a pedestrian walkway provided in the middle.

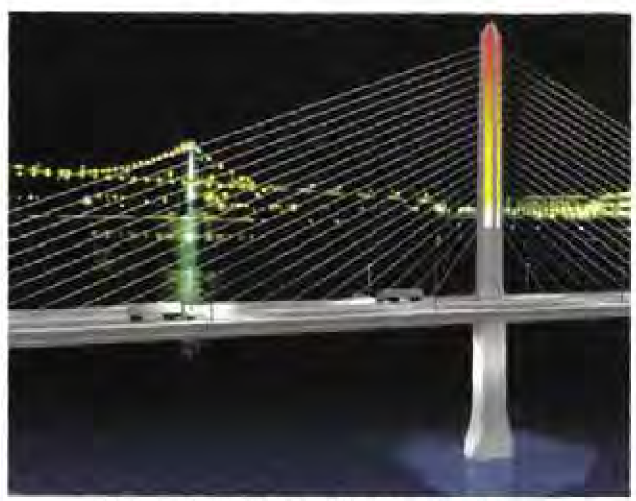
This complex structure was designed and built on an aggressive two-year schedule. To achieve this goal, the Secretary of Public Works adopted a "fast-track" approach: the first five months were used for preliminary design, development of bid documents, and contractor selection. The detailed design was completed with input from the selected contractor. In particular, the construction methods for the superstructure and pylon could be modified to optimize the schedule.

2.4.3.3 Maumee Stay Cable Bridge, Toledo, OH

The new six-lane high-level bridge over the Maumee River is located downstream from the existing Craig Memorial Bridge near Toledo, Ohio. The region's new signature landmark bridge has a vertical clearance of 36.5 m and a 122 m wide shipping channel; its overall length is 2682 m.

The stay cables, which carry the bridge deck in a single plane of stays and from one pylon only, are designed in accordance with the revolutionary "cradle system" developed by Figg Bridge Engineers, Inc., of Florida. In the new system the stays are not individually anchored into the pylon, but a continuous cable stay runs from the bridge deck, through the cradle at the top of the pylon, and back down to the bridge deck in the subsequent span. In addition to significant reductions in material costs and construction period, the "cradle system" allows the pylon to be slender and aesthetically pleasing because the need for pylon cable anchors is eliminated. Utilization of traditional anchor systems would have required the pylon to be at least an additional 3 m in width; Fig. 2.41.

The stay cables for the Maumee Bridge consist of 82 ft to 156 ft-0.6 in. epoxy-coated strands encased in stainless



Computer simulation of the Maumee Bridge

Fig. 2.41 Maumee Stay Cable Bridge, Toledo, OH
Courtesy of Figg Engineering Group and DYWIDAG-Systems International, USA

steel sheathing. One hundred and fifty-six strands per stay cable were used, which was a world record for cable-stayed bridge structures. The high-strength steel strands are encased in a stainless steel sheathing and each strand will pass through its own stainless steel tube in the cradle assembly, eliminating strand-to-strand interaction in the curved portion of the cable.

The project provides for 40 "reference" strands, which may be individually removed at desired time intervals for inspection (after 15 years, 50 years, etc.), in order to verify the condition of the stays during the life of the bridge. The chosen stay cable design allows the owner savings of over US $3 million, with a total construction volume of about US $240 million.

2.4.3.4 Leonard P. Zakim Bunker Hill Bridge, Boston, MA

The Leonard P. Zakim Bunker Hill Bridge is the crown jewel of Boston's multi-billion-dollar Central Artery/Tunnel Project. This newest "Gateway to Boston" is evidence that many communities no longer view their bridges solely as a means to get from point A to point B; Fig. 2.42. The bridge epitomizes the philosophy of form following function; a signature structural form was borne out of a multitude of functional requirements and stringent site constraints. With its slender towers and light superstructure, the bridge is an extremely efficient structure with few ornamental aspects. It also is a case study of how geometric refinements, refined analysis, application of innovative and efficient structural systems and details, and selection of optimal materials can combine to provide efficient solutions to a diverse array of technical challenges on a highly complex project. The Zakim Bridge should lay to rest the belief that structural efficiency can only be achieved by sacrificing aesthetic impact and visual appeal. The bridge's engineers were as focused on the visual effects of their design solutions as their technical merits. The bridge's 10 lanes will ease gridlock that has plagued Boston's downtown elevated highway system for decades. Its unique two-lane cantilevered roadway, carrying northbound traffic from the Sumner Tunnel and North End, will help to eliminate bottlenecks created by traffic at Logan International Airport.

Fig. 2.42 Leonard P. Zakim Bunker Hill Bridge, Boston, MA
Courtesy of HNTB Corporation

Constructed on a highly congested urban site, with existing facilities both above and below ground, the Zakim Bridge incorporates numerous design features that avoid conflict and interference with existing facilities and minimize adverse impacts on its surroundings. Many bridge elements were limited in physical size due to these geometric constraints and the client's desire for a slender, elegant structure. Prudent use of post-tensioning allowed the design team to strike a balance between such constraints and the need to provide high-quality structural members that met serviceability and strength demands. Provision of active reinforcement in reinforced concrete bridge elements (post-tensioning) was both necessary and desirable:

- To enhance the serviceability and durability of the concrete bridge by eliminating or limiting the level of tensile stresses in concrete elements subjected to deicing salts and other environmental factors.
- To eliminate or control bursting and splitting stresses due to anchorage of stay cables and at locations of application of other large concentrated loads.
- To enhance shear and torsional resistance of concrete elements by providing pre-compression.
- To provide adequate strength in relatively small elements carrying substantial loads.

The main span places the two tower foundations on land, providing a channel free of piers in the water immediately upstream of the Charles River locks and dam. Constrained by the Orange Line subway tunnel and an active ventilation building on one side and the existing bridge on the other, the tower width at the deck level accommodates only eight of the bridge's 10 lanes. The two remaining lanes are cantilevered to the outside of the eastern cable plane (within the main span). The CA/T Project involves depressing the Interstate 93 arterial roadway below ground as it cuts through downtown Boston. The need to tie in to the I-93 tunnel as it emerges required a very slender bridge profile at the south end. The overlap of the existing bridge and new bridge at the end of the south back span made anchorage of cables along the median of the roadway the only viable solution for the back spans. The main span is supported with two cable planes along the longitudinal edge girders. This unique cable geometry necessitates the inverted Y towers. The towers are widest at the roadway level and are bent back below the deck, forming a diamond shape.

The Zakim Bridge lays claim to numerous engineering firsts. It is the first major cable-stayed bridge asymmetri-

Fig. 2.43 South Sixth Street Viaduct, Milwaukee, WI
Courtesy of HNTB Corporation

cal in cross section, the first major hybrid cable-stayed bridge in the United States and the world's widest cable-stayed bridge to date. A two-lane ramp cantilevered to the outside of its east cable plane makes the bridge asymmetrical in cross-section, while unequal back span lengths and the height of its two towers make the structure asymmetrical in all three dimensions.

2.4.3.5 South Sixth Street Viaduct, Milwaukee, WI

Wisconsin's first design-build transportation project is the replacement of the South Sixth Street Viaduct in downtown Milwaukee. The Sixth Street Viaduct serves as a southern gateway connecting the City of Milwaukee to the historic restaurant district through the Menomonee Valley; Fig. 2.43. To complete this key southern connection, the city looked to a design-build team to bring a 30% RFP concept to reality within an aggressive 24-month schedule.

The cable-stayed bridges and approaches consist of bi-directionally post-tensioned flat slab construction using 8000 psi, high-performance concrete (HPC). An optimization of the cross-section resulted in the use of a variable-depth concrete slab with depth changes mimicking the depth of a segmental box girder top slab. In the longitudinal direction, 19 ft-0.6 in. strand tendons installed in 4 in. nominal diameter high-density polyethylene ducts were used. Transverse post-tensioning consists of 4 ft-0.6 in. strand tendons installed in flat polyethylene ducts. Both longitudinal and transverse post-tensioning are banded to enhance both the efficiency and ultimate strength of this superstructure system. Longitudinal tendons exceeded 600 ft in length in some cases and were double-end stressed to maximize efficiency.

Ungrouted, greased, and sheathed stay cables were selected for this project as the optimal choice considering durability and economy. In order to monitor the condition of the stay cables throughout the life of the bridge, an additional strand was added to each stay cable as a "reference" strand. At five to 10 year intervals, a strand from selected stay cables is removed and inspected for signs of corrosion.

The project team successfully designed and constructed the 3700-foot project, including dual flat-slab cable-stayed bridges and twin movable bridges, within an aggressive 24-month schedule from award to bridge opening. In contrast, a typical cable-stayed or movable bridge has a final design schedule of 18 to 24 months, and a construction schedule between 24 and 36 months. The design-build method of project delivery clearly saved at least 18 months and as much as two years in calendar time to bridge opening.

2.4.4 P/T Bridge Decks

Post-tensioning is an ideal choice for bridge decks. Post-tensioned bridge decks can span longer distances requiring fewer cross members to be fabricated and installed. This makes the bridges economical and speeds up the construction.

Post-tensioned decks have a built-in pre-compression which significantly reduces cracking. The encapsulated tendons are isolated from the surrounding environment and are not affected by the deicing salts. With very little amount of reinforcing steel, post-tensioned decks are more durable as compared to reinforced concrete decks. Post-tensioning of bridge decks is a popular choice for segmental and cast-in-place bridges.

2.4.4.1 The Reconstruction of Historic Wacker Drive, Chicago, IL

Wacker Drive is a major two-level viaduct bordering the north and west sides of Chicago's downtown "Loop." The two-mile-long, two-tiered elevated roadway was built between 1924 and 1926 under the direction of Charles H. Wacker. Owing to severe corrosion of the reinforcing steel and spalling of the concrete cover, the Chicago Department of Transportation decided to replace the existing 75-year-old viaduct with a bi-axially post-tensioned, high-performance concrete slab structure; Fig. 2.44.

Through a multi-faceted public information campaign, the city and its design and construction management teams were able to maintain access to the 57 high-rise buildings on Wacker Drive while minimizing disruptions to pedestrians and Loop traffic. On an average weekday, more than 60,000 pedestrians and 60,000 vehicles use the upper and lower levels with another 125,000 vehicles crossing over the roadway. The project was completed on time and within budget.

2.4.5 Precast Spliced Girders

Precast pre-tensioned I-beams are not very efficient when used as simply supported beams, especially for longer bridge spans. They are better utilized when they are made continuous for live load and superimposed dead load using on site post-tensioning of the simply supported beam segments.

There are several advantages for using post-tensioning as opposed to non-prestressed reinforcement to make precast pre-tensioned I-beams continuous:

- Short segments that could not otherwise be manufactured in one piece can be spliced into longer spans, thereby creating a more efficient continuous bridge.
- The introduction of precompression in the negative moment zones, which translates into a crack-free deck surface above the piers.
- Post-tensioning of the spliced girders ensures that deck slabs are stressed below the cracking limit (i.e., improved durability).
- Spliced girders can easily adapt to horizontally curved alignments, as they can be laid out in a piece-wise arrangement, then post-tensioned for continuity.

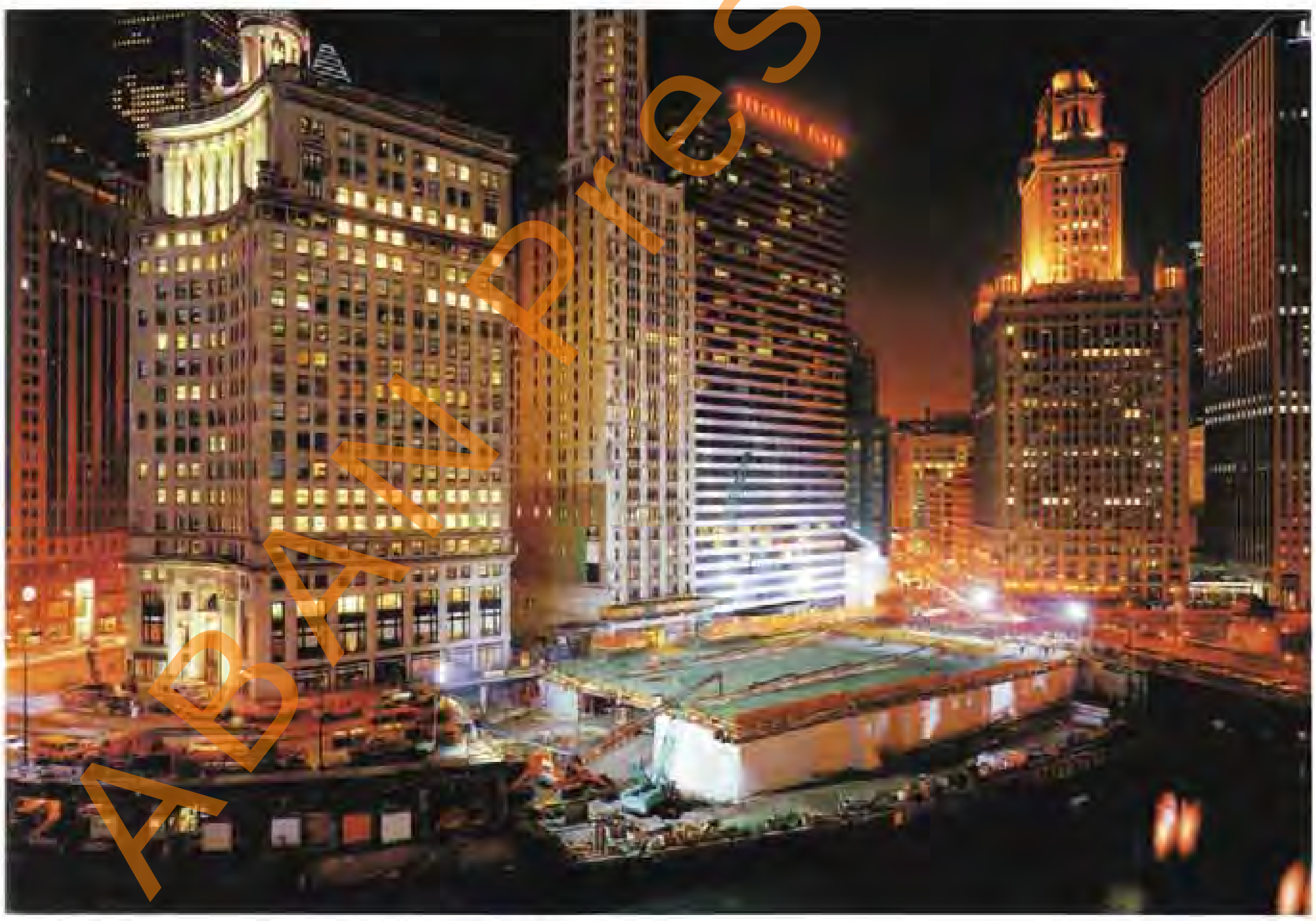


Fig. 2.44 The Reconstruction of Historic Wacker Drive, Chicago, IL
Courtesy of Alfred Benesch & Company

Fig. 2.45 Twisp River Bridge, North Cascade Mountains, Twisp, WA
Courtesy of Washington State Department of Transportation

2.4.5.1 Twisp River Bridge, Twisp, WA

An example of a post-tensioned spliced girder bridge is the Twisp River Bridge shown in Fig. 2.45. The structure replaces a four-span cast-in-place concrete T-beam bridge built in 1935 that had become functionally obsolete. The bridge crosses the Twisp River, which is home to several endangered fish species. Under normal circumstances, WSDOT would have designed a two-span prestressed concrete girder bridge with an intermediate pier in the river. Environmental restrictions, however, allowed work below the ordinary high water line only during the months of July and August, which made the installation of an intermediate pier difficult. This obstacle meant that WSDOT would have to come up with a new innovative solution.

In this case, the solution was a 197 ft (60.0 m) long single-span bridge using the new deep WSDOT girder sections, which would eliminate construction in the river during the September to June fish closure. New 7.87 ft (2.4 m) deep WSDOT W95PTMG girder sections were used in the construction. The girders were delivered to the site in three precast, pre-tensioned segments, erected on falsework bents, and post-tensioned together after the roadway deck was placed. High-performance concrete (HPC) with a 28-day strength of 8000 psi (55 MPa) was used for the girders. The girder closure pours required a strength of 5000 psi (35 MPa), while all other cast-in-place concrete had a specified strength of 4000 psi (28 MPa).

2.5 STORAGE STRUCTURES

The versatility, economy, watertightness and long-term durability of post-tensioned concrete make it an ideal material for environmental and other storage structures. Post-tensioned containment structures have proven records for superior performance, durability, and economy. They are well suited to store a wide range of liquid and solid materials, including potable water and wastewater, low temperature liquefied gases, chemicals, oil, grain, cement and clinker.

Fig. 2.46 White River Water Treatment Facility, Indianapolis, IN
Courtesy of Jorgensen & Close Associates, Inc.

Fig. 2.47 Foothills Water Treatment Plant Reservoir No. 3, Denver, CO
Courtesy of Bates Engineering, Inc.

2.5.1 Water Storage Tanks

2.5.1.1 White River Water Treatment Facility, Indianapolis, IN

This 7-million-gallon facility is an example of a rectangular P/T concrete tank with rounded corners. This design-build tank has horizontally and vertically post-tensioned walls. It also has a monolithically placed, post-tensioned two-way flat plate mat foundation designed for four feet of water uplift. Monolithically placing and post-tensioning the two-way flat plate roof helped to prevent cracking and possible water contamination. The tank is fully buried, with two feet of backfill over the roof. The corners are rounded to a 40-foot inside radius; see Fig. 2.46.

2.5.1.2 Foothills Water Treatment Plant Reservoir No. 3, Denver, CO

Denver Water has built seven buried circular cast-in-place post-tensioned concrete water storage reservoirs since 1982 and has found them to be economical. Denver's post-tensioned tanks have had excellent serviceability (durability) and low maintenance compared to other types of reservoir construction.

The Foothills Reservoir is believed to be the largest circular post-tensioned water storage reservoir in the United States. The reservoir statistics reflect the huge size of the circular post-tensioned concrete tank:

- Over 1 million feet (189 miles) of prestressing strand including 42 miles of unbonded tendons in the floor slab, 91 miles in the bonded tendons in the wall and 56 miles of unbonded tendons in the roof slab (Fig. 2.47)
- 10,500 cubic yards of concrete
- 3000 tons of cement
- 305 tons of reinforcing steel
- 29,841,000 gallons of storage capacity

One of the notable events during construction was the single-day placement of the floor slab concrete. To eliminate construction joints and risk of water leakage at the joints, the floor slab of the reservoir was monolithically placed. More than 400 people completed the 122,000 sq ft, 3250-cubic yard placement in just 14 hours.

Post-tensioning made this construction possible and economical. It has been estimated that post-tensioned concrete construction saved up to $4 million compared to a reinforced concrete reservoir.

2.5.1.3 Waste Water Treatment Plant, Lehighton, PA

This project required an oxidation ditch, with three tanks of about 2 million gallons each; an aerobic digester, with three 112,700-gallon digester tanks; and two 154,400-gallon sludge holding tanks. Post-tensioning provided an innovative approach that would maximize durability while minimizing maintenance.

Fig. 2.48 Waste Water Treatment Plant, Lehighton, PA
Courtesy of VSL

The tanks featured internal vertical and horizontal bonded post-tensioning in plastic duct; Fig. 2.48. Post-tensioning was also used for the slab-on-grade floor, horizontal and vertical walls, and the elevated slab. The concrete floors of the ditches and the digester were designed and constructed to prevent uplift from groundwater pressure. The floors and footings were placed on a 12 in. thick layer of compacted crushed stone. A minimum compressive stress of 250 psi was obtained.

2.5.1.4 Egg-Shaped Sludge Digester Tanks, Bottrop, Germany

Although they have not been used extensively in North America, egg-shaped sludge digester tanks are gaining increasing usage and acceptance throughout the world, due to the shape's inherent advantages. The need for high durability, combined with the desirability of lightweight, thin-walled shells, have made post-tensioned concrete the most practical and effective method of construction for this type of structure. Post-tensioning greatly reduces the amount of reinforcing steel that would otherwise be required, thus reducing steel congestion and facilitating construction.

Fig. 2.49 Egg-Shaped Sludge Digester Tanks, Bottrop, Germany
Courtesy of VSL

Fig. 2.50 Seward Ash Silo, Seward, PA
Courtesy of VSL

The four prestressed concrete digesters at Bottrop, Germany, each with a capacity of 19,600 cubic yards (15,000 m^3), are the largest tanks of their kind ever constructed; Fig. 2.49. Each tank is 155 ft (47.47 m) tall, with a diameter of 89 ft (27.16 m) at the midpoint.

2.5.2 Silos

2.5.2.1 Seward Ash Silo, Seward, PA

The Seward Silo project involved the post-tensioning of three interconnected ash silos. The overall design-build project involved the construction of a new, state-of-the-art 208-megawatt power plant designed to burn low-grade coal.

The silos are believed to be the first interconnected silos built in the world using post-tensioning as the primary circumferential reinforcement; Fig. 2.50. The bonded post-tensioning included 366 horizontal tendons in the walls, using galvanized duct with 4 to 12 - 0.6 in. strands per tendon. Strand installation, stressing, and grouting operations were completed at the four intersection wall locations and the eight external pilasters.

Post-tensioning combined with slip-form casting permitted rapid construction. The slip was completed in nine days. Once the slip was completed, access to work was achieved using swing-stages supported with steel beams placed at the roofs of the silos.

2.5.2.2 Saraburi, Thailand

The 80-foot tall, 215-foot diameter clinker storage silos in Saraburi, Thailand are the largest post-tensioned silos in the world. The use of post-tensioning in the silos resulted in both economy and increased strength.

The circumferential post-tensioning in the 20 in. thick walls provided effective counterbalance to the huge static and dynamic loads on the walls. This resulted in thinner walls, reduced rebar congestion, and better cracking behavior. Overall, the quantity of concrete and the construction cost were reduced.

2.5.3 Nuclear Containment

The safety of a nuclear power station is the first design consideration in every reactor structure. It follows that the modes of failure of a pressure vessel must be predictable and should not be approached by any credible fault conditions. It is highly desirable that warning should be given by slowly progressive failure modes.[2.3] These requirements can be attained with post-tensioned prestressed concrete.

Two types of post-tensioned prestressed concrete reactor structures are used. In the first, the complete pressure circuit embracing reactor and heat exchangers are placed within the one concrete vessel (a prestressed concrete reactor vessel). The Fort St. Vrain prestressed concrete reactor vessel near Denver, Colo., was the first of its type in the United States. Fig. 2.51 shows details of the Fort St. Vrain reactor vessel. The vessel is an approximate hexagonal prism, 106 ft high and 61 ft across the flats. The internal cavity is 75 ft in height and 31 ft in diameter.

The second type of post-tensioned concrete reactor structure is called a containment vessel. In this type of design, the reactor is contained in a steel pressure vessel connected by external ducts to heat exchangers. The complete system is then surrounded by a larger more voluminous containment structure. Fig. 2.52 shows construction of the post-tensioned containment structure of the Palisades Nuclear Plant near South Haven, Mich.

2.6 GRANDSTANDS AND STADIUMS

Stadiums are landmark structures that define cities and even countries. They are some of the largest structures built by man. They are designed and constructed for use during short periods of time with life safety as a primary function of design. While functionality is important, typically the primary goal of a designer is to build a structure that people will remember and relate to a particular locale. To that end, post-tensioning offers the designer many possibilities that would not be available with reinforced concrete, precast concrete, or steel structures. The use of post-tensioning in the structure allows exceptional design flexibility, aesthetically pleasing architecture, and long design life with low maintenance.

Post-tensioning is used in a variety of ways when constructing stadiums and grandstands. Post-tensioning is used in conjunction with cast-in-place, precast, or steel construction. Some examples of applications follow:

- **Raker Beams** – Cantilevered raker beams improve the sight lines by eliminating support columns. Post-tensioning allows reduced sizes of the individual beams and longer cantilevers.
- **Ring Beams** – Ring beams take thrust from domed structures and minimize movement at the base of domes. Post-tensioning allows reduced sizes of the individual beams and provides an easily understood

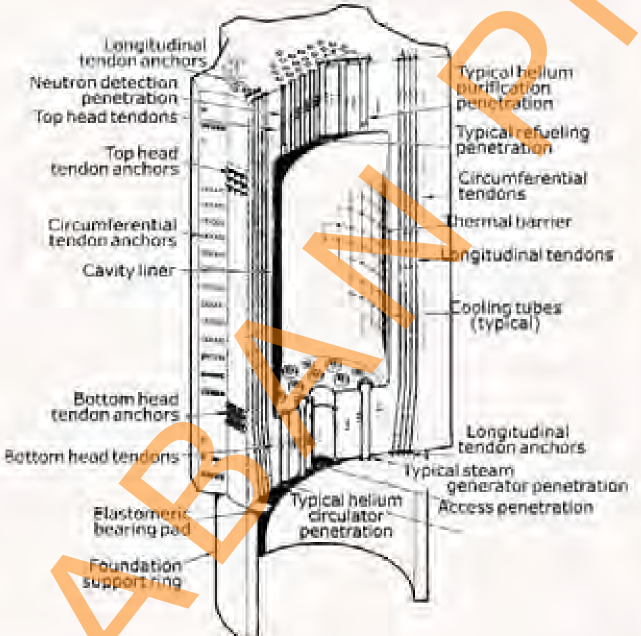


Fig. 2.51 Details of St Vrain Reactor Vessel

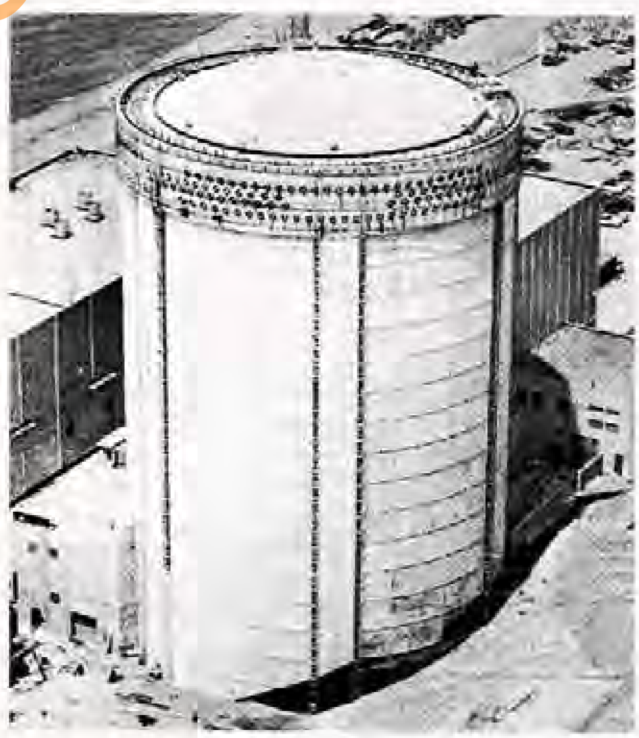


Fig. 2.52 Construction View of Post-Tensioned Containment Structure of Palisades Nuclear Plant near South Haven, MI

method of transferring the forces into the structure and foundation.

- **Support Stays** – Stays are used to support roof structures, improving sight lines by eliminating support columns or allowing longer-span roof systems. Post-tensioned stays transfer the load back to a column or support pier.
- **Tie-Downs** – Tie-downs help transfer tension loads to the foundations. Post-tensioned tie-downs are used to hold the backs of cantilevers and keep them from overturning. The tendons can be embedded in columns/piers or be exposed tendons similar to stays.
- **Floor Plates** – Floor slabs around the structure acting as walkways and supporting auxiliary functions (food courts, locker rooms, media rooms, etc.) are many times economically constructed with post-tensioned concrete floor systems.
- **Foundations** – Including post-tensioning in foundation design can enhance the performance of concrete mats and foundation beams.

All types of post-tensioning systems are used in stadiums and grandstands. Unbonded, bonded, single-strand, multiple-strand, bar, and stay cable systems have been used. The application will normally dictate the type of system used.

2.6.1 Invesco Field at Mile High, Denver, CO

Post-tensioning was used extensively at the new 76,152-seat home to the Denver Broncos (NFL) and the Colorado Rapids (MLS); Fig 2.53. The structural framing for the concourses of the stadium consists of 8-inch-wide, 29-inch-deep post-tensioned circumferential pan joists framing to post-tensioned 33-inch-wide by 29-inch-deep transverse girders. This design was developed through post-bid, value engineering exercises with the concrete sub-contractor. The two-way post-tensioned system was considered due to the long joist spans (40 ft to 50 ft) and high-superimposed loads (3 in. topping slabs, 100 psf live loads and numerous masonry walls).

Early studies had concluded that a system of cast-in-place girders and columns with precast concrete double tee infill, which has been used for several other NFL stadiums, was the most economical. Concerns with the original precast infill system were as follows:

- Curved geometry
- Flying formwork
- Structural depth
- Coordination between precast and cast-in-place
- Excessive topping slab

Fig. 2.53 Invesco Field at Mile High, Denver, CO
Courtesy of Walter P. Moore and Associates, Inc.

Fig. 2.54 Precast, Post-Tensioned Arch Roof Construction of The Calgary Olympic Oval

Fig. 2.55 Interior View, Calgary Olympic Oval

Schematic pricing of the two-way post-tensioned alternative indicated that approximately $2 million could be saved over the original scheme. Creative long-span forming and shoring systems, and a structural design that eliminated the need for reshoring, resolved initial concerns about schedule and quality of finish. The post-tensioned system also allowed for a shallower structural depth, providing additional mechanical plenum space and allowing for the floor-to-floor height at one of the suite levels to be reduced for additional cost savings. Advantages to the use of this two-way, post-tensioned system were:

- **Cost** – Approximately $2 million savings were achieved over the original structural system.
- **Geometry** – The cast-in-place concrete system was better able to handle the varying geometry and loading.
- **Structural depth** – Post-tensioning typically allowed for the structural depth to be reduced by 5 in.
- **Durability** – As an open-air stadium, exposure to the elements was a major concern. Post-tensioning allowed the tensile stresses to be limited, therefore reducing the number and size of flexural cracks.
- **Reduced congestion** – The original girder system typically required multiple layers of reinforcement. Post-tensioning significantly reduced the amount of reinforcement required and eliminated the need for multiple layers of reinforcement in most members.
- **Deflection** – With significant extents of masonry, most of the concrete framing was under strict deflection limits. In most cases, post-tensioning allowed for these deflection limits to be met without increasing member sizes.

2.6.2 Olympic Oval, Calgary, AB, Canada

The 1988 Winter Olympics speed skating events were staged in the first 400-meter covered speed skating oval in the world. The roof structure for this huge facility consisted of a series of post-tensioned intersecting concrete arches (Fig. 2.54) which allowed the use of precast concrete components as the primary load-carrying system for the very large, long-span roof. The system as used on this project consists of a number of precast arch segments, each identical in cross-section and length, joined at their intersection points by cast-in-place concrete infill nodes. Splicing of reinforcing at the nodes and continuous concentric post-tensioning throughout the full length of each arch provided continuity.

The grid of arches intersected the length of the building at an angle of 67 degrees. This increased the overall length of the arches by only 9% while providing a system that was internally stable and that effectively distributed thermal stresses and concentrated loads. The intersecting arch system was conceived following extensive research and cost comparison on alternative roof structures, including timber domes, steel arches and space frames, and fabrics. The completed structure (Fig. 2.55) provides an attractive environment for both participants and spectators.

2.7 STAGED CONSTRUCTION/TRANSFER GIRDERS

2.7.1 O'Hare Airport Transit Station, Chicago, IL

When the City of Chicago extended its rapid transit line to O'Hare International Airport, a new station was constructed beneath the existing six-story O'Hare parking garage. Because the column spacing in the existing garage was not compatible with the operation of, or movement within, the station, the caissons supporting two rows of columns had to be cut off. A system of seven transfer girders was needed to support the garage over the top of the station. Each girder, with dimensions of 10 ft by 10 ft by 75 ft, supports five stories of the existing garage plus the ground-level parking.

Fig. 2.56 Transfer Girder Construction, O'Hare International Airport, Chicago, IL

The advantage in using post-tensioned prestressed concrete was the ability to prestress the girders in two entirely opposite ways. The temporary post-tensioning was in the top of the girder, to resist loads on the cantilevered ends while the girder was supported on the existing garage caissons. The final post-tensioning was in the bottom of the girder to resist the column loads from the garage above, while the girder is supported at the ends. Prior to the final post-tensioning, the temporary post-tensioning was released. The use of post-tensioning permitted all the necessary load transfers to be achieved with virtually no movement of the existing structure.

Special features of the design and construction of the transfer girders:

- The girders were cast on the ground (Figs. 2.56 and 2.57), before excavation of the subway station, eliminating the need for shoring and scaffolding under the formwork.
- The girders were supported on the existing caissons during excavation of the station (Fig. 2.58), eliminating the need for temporary supports. Special temporary post-tensioning was required for this stage because the support of the girder by the garage columns and caissons represented the opposite of the final structural action of the girders.

Fig. 2.57 Completed Transfer Girder, O'Hare International Airport, Chicago, IL

Fig. 2.58 Excavations Under Transfer Girders Supported on Original Caissons, O'Hare International Airport, Chicago, IL

Fig. 2.59 Cutting Off Original Cassions, O'Hare International Airport, Chicago, IL

Fig. 2.60 Final Station Construction Phase, O'Hare International Airport, Chicago, IL

After construction of the permanent supports for the girders, before cutting off the existing garage caissons, precisely computed final post-tensioning and de-tensioning was applied to transfer the existing load from the caissons to the new supports. Very careful and sophisticated analysis was required to ensure that the caissons were essentially unstressed (neither compression nor tension) when they were cut off. Fig. 2.59 illustrates the process of cutting off the original caissons. The second and higher floors of the garage were in full and normal use during construction, even while the caissons were being cut off. In Fig. 2.60, the completed transfer girders are in place during the final station construction phase.

2.7.2 Georgia Tech Student Athletic Complex II, Atlanta, GA

The renovation and expansion of the student athletic complex involves the adaptive reuse of the existing Olympic Aquatic Center. The Aquatic Center was constructed in 1995 to serve as the swimming and diving venue for the 1996 Summer Olympic Games. The expansion of the facility involved the construction of a long-span concrete frame structure that spans over the existing 50-meter competition and diving pools and spectator seating area; Fig. 2.61.

The structural system supporting the multi-purpose gymnasium floor has a clear span over the existing aquatic cen-

Fig. 2.61 Georgia Tech Student Athletic Complex II, Atlanta, GA
Courtesy of Continental Concrete Structures

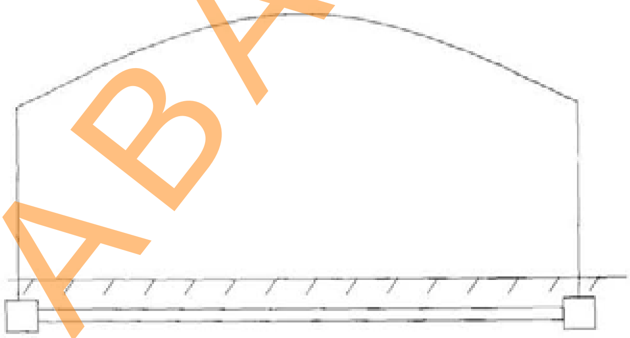
ter and is approximately 60 ft above the pool decks. The long span concrete structure consists of 10 parallel structural concrete frames. The frame girders are post-tensioned and span 170 ft with a 40-ft cantilever. The girders are 13 ft deep and support an 8 in. thick slab that spans 26 ft. The slab is post-tensioned.

The girders were constructed in a staged sequence with incremental post-tensioning at each stage. Staged construction was used to reduce the construction loads imposed on the pool floors. The construction sequence involved casting the first stage, which was slightly more than half the girder depth, and applying the first increment of tensioning. After post-tensioning, this stage was self-supporting and had sufficient strength to support the second stage of concrete. Forms were released after post-tensioning the first stage, and therefore any further loads to the pool floors were eliminated. After post-tensioning the second stage, the third stage was cast. The design of the first and second stages acting together was sufficient to carry the final third stage of concrete. The post-tensioning of the third stage completed the construction of the girder.

Concrete was selected for the structural system principally due to its inherent mass and stiffness and its natural resistance to vibration anticipated from aerobic dancing and the running track. Vertical accelerations due to these activities were specified to be less than 5 percent of gravitational accelerations. Concrete was also chosen because of its corrosion resistance and durability. Corrosion was a major concern on this project due to the high level of airborne chlorine and moisture from the pools below.

The tight construction schedule required completion of the first stage post-tensioning within 48 hours following the concrete pour. The design of the girders required an initial strength of 5000 psi at the time of post-tensioning, which was achieved within 24 hours.

a) Tension Tie in an Inverted Suspension Bridge



b) Underground Tension Tie

Fig 2.62 Post-Tensioned Tension Members

2.8 TENSION MEMBERS

Post-tensioning is an excellent medium for tension members and ties. Post-tensioned members use much higher-strength steel than structural steel; therefore, the steel area is less for the same required force, allowing a sleeker, more slender member.

Post-tensioning is well suited for arch tension members. Steel arched bridges easily accommodate post-tensioned tension ties. The tension ties in these cases will most likely consist of single or multiple-strand tendons encased in steel or plastic pipe and grouted for corrosion protection. Fig. 2.62(a) shows an example of post-tensioning in an inverted suspension bridge.

Another application is an underground tension tie that contains the column thrust force from long-span overhead steel structures. Large single-span metal buildings exert an enormous outward force at the base of the columns. The column foundations can be tied together with post-tensioning to overcome the column thrust. The tendons are buried under the earth or floor slab. Multiple unbonded single-strand tendons or single bonded multiple-strand tendons are used; Fig 2.62(b). The tendons are placed in a trough that can be concreted, grouted, or earthen-filled. Corrosion protection of the tendons must be considered in the design. Care must be exercised not to damage the tendons if excavating after they are placed.

2.8.1 Post-Tensioned Tension Rings

2.8.1.1 University of West Virginia Field House, Morgantown, WV

Post-tensioned tension rings may be used to take the thrust from domed structures and to minimize movements at the base of the dome and resulting stress development throughout the dome surface. Fig. 2.63 shows the construction of the 360-foot diameter domed field house at the University of West Virginia, in Morgantown. The horizontal thrust from the dome is carried by the 4 ft-6 in. by 5 ft-0 in. ring located on top of the 34 ft tall columns. The ring is post-tensioned by 14-40 wire tendons. Tendons in 282 ft lengths were used in overlapping arcs to post-tension the 1130-foot circumference. Eight notches formed on the outside of the ring provided tensioning locations. At each tensioning location, seven tendons pass continuously through the ring, and seven tendons are terminated for tensioning.

2.8.2 Post-Tensioned Tie Beams

2.8.2.1 U.S. Air Force Museum, Eugene Kettering Building, Dayton, OH

The U.S. Air Force Museum in the Eugene Kettering Building in Dayton, Ohio is designed in the shape of an arc with a 283 ft (86 m) radius. The arched structure was designed to handle dead load, snow load, the weight of the

Fig. 2.63 Construction of Domed Field House, University of West Virginia Field House, Morgantown, WV

trusses and that of the aircraft and other exhibits that will hang from them. Each truss delivers significant forces to the foundations. Thus the structure needed to be stable enough to resist sliding and overturning forces. To provide this stability, the roof trusses were connected by pins to reinforced concrete buttresses at each end; Fig. 2.64. To resist lateral thrust, the design includes tie beams of post-tensioned concrete beneath the foundation slab.

Each tie beam is 20 in. square and contains a single bonded post-tensioning tendon consisting of up to 11 half-inch (13 mm) diameter strands. The strands for each tendon are housed in a corrugated galvanized metal duct.

The post-tensioned tie beams were used to accommodate the increase in gravity loads on the structure at each stage of construction. The tension ties were stressed in two stages. First, the tendons were partially tensioned before the roof was built. Once the arch trusses, joists, and standing-seam roof deck were in place, the tendons were stressed again, this time to 100 percent of the design jacking force.

Fig. 2.64 U.S. Air Force Museum, Eugene Kettering Building, Dayton, OH
Courtesy of VSL

2.9 ROCK AND SOIL ANCHORS

Anchors were introduced in the 1930s and first used to anchor structures in rock. By the 1960s, anchors were commonly used for all types of major structures in Europe. Since these early applications, anchor technology and acceptance has continued to grow. Today, engineers specify permanent tiebacks for a variety of applications. Some common applications are described below.

2.9.1 Retaining Walls

Ground anchors are commonly used in the construction of retaining walls. A permanently anchored wall can substantially reduce the cost and construction time.

2.9.1.1 Crockett Interchange Project, Carquinez Bridge, Crockett, CA

Wall R1 was the biggest wall on the site and one of the biggest of its kind in the world. Standing nearly 67 ft (20.5 m) at its tallest point and ranging 550 ft (168 m) in length, the wall cut a massive face into the existing hillside; Fig. 2.65. Because of the unstable soils, variable water table, and extreme seismic loads, the anchors used in this wall had to be designed to reach far into the hillside and retain sizable loads. Of the 342 anchors, 200 were over 98 ft (30 m) in length with the longest at 134 ft (40.8 m). Test loads of these 6- to 8-strand anchors ranged from 225 to 337 kips (1000 to 1500 kN).

2.9.2 Dam Tiedowns

Permanent ground anchors are used to anchor a dam and increase its resistance to sliding and overturning.

2.9.2.1 Rio Dam, Orange and Rockland Utilities, Inc., New York, NY

The Rio Dam was constructed in the 1920s to provide power to southeastern New York. In a recent rehabilitation project, 22 tiedowns were installed for increased resistance

Fig. 2.65 Crockett Interchange Project, Carquinez Bridge, Crockett, CA
Courtesy of DYWIDAG-Systems International, USA

to sliding and to stabilize the slightly arched spillway to meet current Federal Energy Regulatory Commission (FERC) standards; Fig. 2.66. High capacity, 1936-kip anchors were required for the rehabilitation along with an innovative corrosion protection system. The 55-strand tendons ranged from 134 to 161 ft in length. The system of corrosion protection covers the entire length of each tendon with a large diameter, heavy wall corrugated sheath.

2.9.3 Earth Stabilization

Permanent anchors are used to stabilize or repair existing walls that may be subject to overturning or founded on soil or rock that is part of a slide. Normally, an existing wall can be stabilized with permanent anchors at a fraction of the cost of constructing a new wall—without disturbing the rock or soil behind the wall.

2.9.3.1 Laguna Niguel Landslide, Laguna Niguel, CA

In the early morning hours of March 19, 1998, a dramatic landslide occurred in the city of Laguna Niguel. As the displaced material moved downslope, several houses and condominiums were severely damaged or destroyed.

Fig. 2.66 Rio Dam, Orange and Rockland Utilities, Inc., New York, NY
Courtesy of Paul C. Rizzo Associates, Inc. and DYWIDAG-Systems International, USA

Fig. 2.67 Laguna Niguel Landslide, Laguna Niguel, CA
Courtesy of Schnabel Foundation Company

Fig. 2.68 Ground Anchors to Stabilize the Basement for WTC Cleanup
Courtesy of DYWIDAG-Systems International, USA

Post-tensioned ground anchors were used to stabilize the area and protect the remaining structures in the neighborhood; Fig. 2.67. The stabilization scheme included a permanent caisson and tieback wall at the head of the slide, permanent tiebacks within the slide cut, and replacement of the slide material. Each anchor had a tendon with nine 0.6-inch strands and double corrosion protection. Anchors were up to 210 ft long and tested to one and a half times the design load.

2.9.4 Excavation Stabilization

Excavations for office buildings, hospitals, hotels, colleges, retail centers, condominiums, and parking structures frequently require vertical cuts to be made on or near property lines. Excavation walls can be economically stabilized using temporary anchors. Safety is often improved by eliminating cramped excavations and reducing the time and area required for standard construction methods. Fig. 2.68 shows post-tensioned ground anchors that were used to stabilize the excavation during the massive clean-up of the World Trade Center in the aftermath of the Sept. 11, 2001 attack.

2.9.5 Resist Uplift

Ground anchors are also used to resist hydrostatic uplift forces. Concrete structures located below the water table will "float" out of the ground unless they are sufficiently heavy or tied down. Ground anchors were used to resist uplift of more than 50 ft of water pressure in the construction of the Opera House in Copenhagen, Denmark; Fig. 2.69.

2.9.6 Tower Tiedowns

2.9.6.1 Wind Turbine, Toronto, ON, Canada

The first wind turbine built to produce power for Toronto's electrical grid went into service at the end of January 2003; Fig. 2.70. The 94-meter-high, 750-kilowatt turbine is the foundation for a future that includes green electricity generated without producing emissions. The tower consists of 115 cubic yards (150 m^3) of concrete with a 13,200 lb (6000 kg) steel tube anchored to the ground by 59 ft (18 m) long rock anchors.

Fig. 2.69 Opera House in Copenhagen, Denmark
Courtesy of DYWIDAG-Systems International, USA

Fig. 2.70 Wind Turbine, Toronto, Ontario, Canada
Courtesy of DYWIDAG-Systems International, USA

2.10 POST-TENSIONED SLABS-ON-GROUND

Post-tensioning has been used successfully in elevated structures for many years. The inherent benefits of post-tensioning are also being realized in slab-on-ground applications. Residential foundations, light industrial foundations, heavy industrial foundations, sport courts, institutional foundations and pavements all benefit from the enhanced performance and cost savings post-tensioning provides.

2.10.1 Residential Slab-on-Ground Foundations

The slab-on-ground market is the fastest-growing segment in the post-tensioning industry. Slab-on-ground material shipments have increased an average of 13% per year over the last decade, with approximately 362,000 homes constructed in 2004 utilizing post-tensioning slab-on-ground foundations.

2.10.1.1 Residential Ribbed Foundations on Expansive Soils

Post-tensioning has proven to be an efficient solution for problems associated with ground-supported residential foundations on expansive soils.[2.4,2.5] The compressive stresses resist the anticipated tension stresses induced by the soil movements, enhancing the performance over a typical non-prestressed foundation. Cost benefits are achieved by reductions in quantities of concrete, steel and excavation which in turn reduce labor costs.

A typical post-tensioned foundation on expansive soil consists of a monolithic "ribbed" foundation with a 4 - 5 in. thick slab, a perimeter beam and interior beams spaced in both directions at 10 - 15 ft. Post-tensioning is accomplished using 0.5-in. single-strand tendons distributed in both directions, initially stressed to 33,000 lb, to provide a residual compressive stress ranging from 50 to 100 psi. Fig. 2.71 shows a typical single family residential ribbed foundation. The post-tensioned reinforcing is optimized to reduce the size and/or increase the spacing of the stiffening ribs while constructing a foundation that is sufficiently stiff to accommodate the shrinking and swelling of the supporting soil. This type of construction is widely used by production and custom builders to construct both single-family and multi-family homes in areas throughout the United States that contain soils classified as moderate to highly expansive.

2.10.1.2 Residential Uniform Thickness Foundations

In less expansive soils a uniform thickness foundation is utilized. First used in California and referred to as a "California slab." Fig. 2.72 shows a typical residential uniform thickness foundation. Typical thickness ranges from 7.5 in. to 12 in. The slight increase in the material costs of this system are compensated by the reduction in labor and equipment costs. Because of the elimination of the stiffening ribs, these slabs can be constructed very rapidly, eliminating the need for labor and equipment to dig the ribs and

Fig. 2.71 Residential Ribbed Foundation
Courtesy of Jack W. Graves Jr., GSI Post-Tension

dispose of the resulting soils. This is a substantial benefit in sandy clay soils where the trenches require shoring to keep the sides vertical. It can also benefit the construction of foundations supporting two- and three-story multi-family structures that can have a complex system of interior bearing walls. The thicker slab is usually sufficient to distribute the loads from the internal bearing walls without additional footings. Post-tensioned foundations of uniform thickness are commonly used throughout the southwest.

2.10.1.3 Residential Foundations on Stable Soils

Post-tensioned foundations are also used in areas with stable soils to reduce cracking, reduce or eliminate control joints, increase flexural capacity and improve constructability. Reducing the control joints also improves the serviceability and eliminates durability problems. Typically constructed as a relatively thin (4 in. to 5 in.) slab with monolithic footings to transmit the loads imposed by the structure into the supporting soils, these slabs contain about 50 to 70 psi of compressive stress after overcoming the losses resulting from subgrade frictional resistance, elastic shortening, concrete shrinkage and thermal strains. Post-tensioned foundations supporting single-family and multi-family homes are used by custom and production builders throughout the United States who want the benefits provided by reducing cracking and reducing or eliminating joints.

2.10.2 Light Industrial Foundations

Post-tensioning offers an economical solution for light industrial facilities. Post-tensioning can be utilized to eliminate or minimize open joints and cracks to maximize productivity. The increased stiffness of the slab is also beneficial for applications where high floor flatness is required.

2.10.2.1 Granbury Municipal Airport, Granbury, TX

The City of Granbury was looking for an economical solution for the design and construction of three 50 ft – 4 in. by 294 ft – 7 in. hangars, totaling 44,482 sq ft. The original non-prestressed design specified a 6 in. thick floor reinforced with #4 rebar @ 16 in. o.c. each way with 4 #5 rebar continuous in the perimeter beams. The design required numerous expansion joints and saw cut joints.

The post-tensioned redesign resulted in a 4 in. thick floor reinforced with 0.5 in. single-strand tendons in both directions; Fig. 2.73. A final compressive force of approximately 200 psi was achieved, utilizing a two-stage stressing operation that reduced shrinkage cracks and eliminated approximately 3775 ft of saw cut joints.

In addition to increased strength, cost benefits were achieved by reductions in quantities of concrete (275 cubic yards less or 33%), reinforcing (12.8 tons less or 52%) and the additional long-term savings realized by eliminating all saw cut maintenance costs.

Fig. 2.72 Residential Uniform Thickness Slab-on-Ground

Fig. 2.73 Granbury Municipal Airport, Granbury, TX
Courtesy of Jack W. Graves Jr., GSI Post-Tension

2.10.3 Heavy Industrial Foundations

Post-tensioning is increasingly being used for large area industrial and commercial floors. Owners of industrial facilities have come to realize that floors are integral parts of their industrial processes and can significantly affect efficiency and production. As a result, they have become more and more concerned about optimizing the quality of their floors. Post-tensioning can be utilized to eliminate or minimize open joints and cracks to maximize productivity for the facility owners.

The main purpose of industrial slab-on-ground foundations is to safely transfer applied loads from the superstructure to the soil. Prestressing these foundations provides a combination of active forces acting on the concrete, resulting in a higher crack resistance, improved stiffness, watertightness, increased durability, reduced thickness and fewer joints. Bonded and unbonded post-tensioning are both options for industrial slab-on-ground foundations.

Fig. 2.74 Universal Leaf Tobacco Plant, Tarboro, NC
Courtesy of Blair Industries Inc.

Fig. 2.75 Pharmaceuticals Plant in Lafayette, IN
Courtesy of VSL

2.10.3.1 Universal Leaf Tobacco Plant, Tarboro, NC

The new T-shaped building is a 1.3-million sq ft facility divided into a central processing area, a shipping area, and three large rooms for storing green tobacco; Fig. 2.74. The 700,000 total sq ft of foundation for the shipping and green storage areas were designed as 5-in. thick, post-tensioned slabs on grade.

The primary design objective for the post-tensioned foundations in the shipping and green storage areas was to achieve crack-free floors with no control joints. Tendons ran the full length of each of the rooms and provided a compressive stress of about 250 psi. In addition to providing the owner with a floor that was virtually free of cracks, joints and curling, the post-tensioning allowed thinner slabs, reduced quantities of reinforcement and eliminated the cost of armored joints.

2.10.3.2 Pharmaceuticals Plant in Lafayette, IN

This project is a 23,000 sq ft slab-on-ground foundation for a major pharmaceutical plant; Fig. 2.75. The foundation was designed for live, dead, forklift, heavy storage, and impact loads. Dead load included internal and external concrete masonry walls, and steel columns and roof trusses. A two-strand bonded tendon system was used for the 6-in. slab, and a multistrand bonded system with plastic duct was used in the 36-in. deep edge beams. The tendon layout provided a compressive stress of approximately 330 psi in the slab and approximately 300 psi in the edge beams.

The use of post-tensioning made it possible to build a highly durable, crack-free slab with no construction joints.

2.10.4 Mat Foundation

Post-tensioned mat foundations are occasionally used as an alternative to other types of foundations, particularly where unfavorable soil conditions at a building site make

Fig. 2.76 Large Mat Foundation

pile foundations more expensive. The function of post-tensioning in the mat is essentially to "pick up" the column loads and distribute them more or less uniformly over the entire plan area of the mat. As illustrated in Fig. 2.76, the tendon layout required for this purpose can be visualized as an upside down flat plate.

2.10.5 Sport Courts

Post-tensioning is a preferred method of constructing concrete sport courts. The advantages include improved serviceability, durability, drainage characteristics and controllable ball skid length. It also provides the flexibility to receive many types of topping systems. Post-tensioned concrete courts provide the additional advantages of reducing or eliminating control joints and resisting and controlling cracking due to the pre-compression of concrete. A post-tensioned court also has a higher flexural capacity as compared to a slab with non-prestressed reinforcing to control temperature and shrinkage cracking. Post-tensioned sport courts are typically constructed as thin slabs (4 in. to 5 in. thick) with single-strand unbonded tendons and little or no non-prestressed reinforcing.

Fig. 2.77 The Lakes Tennis Academy, Frisco, TX
Courtesy of Jack W. Graves Jr., GSI Post-Tension

Fig. 2.78 Bristol International Raceway, Bristol, TN
Courtesy of Jack W. Graves Jr., and VSL

2.10.5.1 The Lakes Tennis Academy, Frisco, TX

The Lakes Tennis Academy features 12 new courts, constructed in two monolithic foundations, measuring 240 ft-0 in. by 250 ft-0 in. and 125 ft-0 in. by 240 ft-0 in., totaling 90,000 sq ft; Fig. 2.77. Foundations are 4 in. thick with 12 in. by 12 in. perimeter beams, placed on two layers of 6 mil. polyethylene sheathing over a properly prepared subgrade. The foundations are reinforced with 0.5 in. single- strand tendons in both directions and stressed in a two-stage operation providing a residual compressive force of approx. 150 psi.

2.10.5.2 Tempe Union High School Tennis Court Reconstruction, Tempe, AZ

Beginning in 1997, the Tempe Union High School District began a program of removing their deteriorated asphalt tennis courts and replacing them with concrete courts. Initially designed as non-prestressed 6 in. thick slabs with #4 rebar located in the middle of the slab and spaced at 12 in. on center in both directions, the courts were redesigned as post-tensioned slabs with equivalent flexural capacity. The resulting post-tensioned design called for a 5 in. thick slab with ½ in. diameter strands provided at 4 ft on center in both directions to produce a compressive stress in the foundation of approximately 100 psi. Since the quantity of post-tensioned reinforcing was significantly reduced, the reinforcing costs were significantly lower. In addition the school district saved the cost of 1 inch of concrete spread over the 90,000+ sq ft of courts that were constructed.

2.10.6 Pavements

Joints are the "weak link" in reinforced concrete pavements. Most concrete pavement distress is associated with joints and/or cracks. Prestressed pavements have been used to reduce the number of joints and to improve the life of concrete pavements.

2.10.6.1 Bristol International Raceway, Bristol, TN

The Bristol International Raceway was originally constructed in 1961. It is one of 119 raceways currently sanctioned by the National Association for Stock Car Auto Racing (NASCAR) and is considered to be the "world's fastest half-mile speedway," with average speeds exceeding 120 mph. The track is 0.533 miles long, varies between 40 to 70 ft wide and incorporates 36 degree banked turns; Fig. 2.78. The high bank turns are the steepest in motorsports and provide some of the most exciting competition in all of autoracing. The track configuration results in one of the most demanding facilities for both driver and machine.

The original track was constructed of asphalt. This material performed adequately in the early years, when average speeds were under 100 mph. As speeds increased over the years, the asphalt became a major problem in the turns. The asphalt would break up, resulting in unsafe conditions and loss of traction. After several unsuccessful attempts to resurface the track with various asphaltic designs, the owner decided to investigate a concrete alternative.

A design concept was proposed that utilized the single-strand system along with welded wire fabric. An ambitious construction schedule of 42 days was allowed between the issuance of the contract and the completion of the concrete overlay for required tire testing.

The track was designed to have a minimum residual compression due to post-tensioning of 150 psi after all losses. An optimum pour sequence was established that would eliminate as many construction joints as possible and allow efficient use of construction crews to meet the demanding time schedule and allow proper stressing. The final design utilized 16 pours ranging in length from 292 ft in the straight-aways to 140 ft in the high-bank turns. This limited construction joints to 12 with only 4 closure pours being required. Construction of the track proceeded on schedule

Fig. 2.79 Greater Rockford Airport Runway, Rockford, IL
Courtesy of CTS Cement

and is performing beyond expectations. Many in the auto racing community believe post-tensioned concrete will be the preferred choice on many future overlays and new facilities.

2.10.6.1 Greater Rockford Airport Runway, Rockford, IL

In 1993, a post-tensioned pavement was constructed at the Greater Rockford Airport in Rockford, Ill. The project featured a 1200-ft long, 75-ft wide taxiway; Fig. 2.79. Normally, a reinforced concrete pavement of this size would require over 80 transverse joints and two longitudinal joints. By using post-tensioning combined with fibrous concrete and shrinkage-compensating cement, no transverse joints or longitudinal joints were required on the prestressed concrete portion of the project. This innovative combination also allowed the pavement thickness to be reduced to 7 in. compared to the 15 in. of the reinforced concrete pavement alternative. The first cost of the joint-free prestressed concrete pavement was 8% less than the 15 in. reinforced concrete pavement.

After 10 years of service, it has been reported that the prestressed pavement is performing excellently, with a Pavement Condition Index (PCI) estimated to be over 98.[2.6]

Fig. 2.80 I-35 Project, Waco, TX
Courtesy of The Transtec Group, Inc.

2.10.6.3 Highways

Prestressed pavements have not been used extensively on highways in the U.S.; however, their potential for reducing maintenance and improving long-term performance is gaining renewed attention from pavement designers worldwide.

2.10.6.3.1 I-35 Project, Waco, TX

Located on Interstate 35 in McLennan County, Texas, the 200,000 sq ft prestressed concrete pavement overlay was 38 ft wide. It consisted of two adjacent strips 17 and 21 ft wide, placed on visqueen bond breaker and 2 in. thick asphalt leveling course over the existing concrete pavement. There are nine 240 ft long and seven 440 ft long sections in the one mile of the pavement.

The new slabs were anchored to the pavement below by a continuous 3 ft wide by 2 in. deep transverse recess located at the middle of each post-tensioned section of the pavement to allow elastic shortening and thermal movement at both ends. The transverse end joints were protected by a steel armored expansion joint anchored with Nelson deformed bars to each side of the joint. Stainless steel dowel bars were used to provide load transfer across the expansion joints, which were designed to allow up to 3 in. of thermal movement.

The anchorages for the longitudinal single-strand (unbonded) post-tensioning system were bolted to the armored joints to provide compression to the ends of the slab. Tendons were stressed from the staggered stressing block-outs at the middle of each slab; Fig. 2.80. To elimi-

Fig. 2.81 I-35 Frontage Road Precast Pavement, Georgetown, TX
Courtesy of DYWIDAG-Systems International, USA

nate shrinkage cracking, first stage stressing was conducted not more than eight hours after the concrete was placed when the concrete had reached a compressive strength between 1400 psi and 2300 psi. Final stressing was performed 48 hours after the concrete was placed. The final longitudinal average compression was 200 psi.

2.10.6.4 Prestressed, Precast Pavements

Transportation agencies are under growing pressure to come up with new and innovative ways to build durable, longer lasting pavements that can be constructed quickly to minimize inconvenience to the traveling public. The use of prestressed precast concrete pavement is one such innovative technique for constructing pavements that is receiving a great deal of attention from pavement designers.

Using precast panels joined with post-tensioning, pavements can be constructed during the night or over a weekend when traffic volumes are significantly lower than peak travel times. This can significantly reduce or even eliminate traffic delays normally associated with standard concrete pavement construction, thereby significantly reducing user delay costs. These costs can be further minimized over the life of the pavement because of improved durability. Prestressing not only benefits the durability of the pavement by greatly reducing or even preventing cracking, but also significantly reduces the required pavement thickness.

The Georgetown precast pavement pilot project was initiated by Texas Department of Transportation as a means for evaluating and refining the precast pavement concept (Fig. 2.81). The project incorporated prestressing in both the transverse and longitudinal directions. Similar projects have been initiated in other parts of the U.S.

Fig. 2.82 The Holy Cross Church in Santa Cruz, CA
Courtesy of VSL

Fig. 2.83 The National Concrete Masonry Association's Model Concrete Home
Courtesy of VSL

2.11 MASONRY STRUCTURES

Masonry, like concrete, is strong in compression and weak in tension. As a construction material, masonry has been used primarily for vertical members subject to gravity loads.[2.8] However, other loads (such as in-plane shear, out-of-plane lateral, and deformations caused by floor slabs) may be applied to masonry walls. If these lateral loads and deflections are large enough, the weight of the masonry will not be sufficient to resist the resulting bending moments, and the walls must be reinforced. Post-tensioning is an effective way of reinforcing a masonry wall to enhance its strength, durability, and performance. Post-tensioning can be applied both vertically and horizontally. In the vertical direction, post-tensioning increases the vertical axial load initially applied by the weight of the wall. In the horizontal direction, post-tensioning is typically applied for support such as a beam.

Prestressing bars or strands may be used to post-tension masonry walls. Bars are typically threaded at both ends. The bar may be full height or in multiple sections depending upon the height of the wall. The bars are connected to special anchor bolts that are cast into the foundation. Tendons can be tensioned intermediately at various elevations or at the top of the wall only. The stressing anchorages usually consist of bearing plates and nuts. The force in the tendon can be measured with a calibrated ram or by using direct tension-indicating washers.

With strand post-tensioning, special anchorages are usually cast into masonry units for both fixed- and stressing-end anchors. Duct or pipes are attached to the anchorage units in short lengths, which permits the masonry units to be easily placed over the ducts as the wall progresses. Additional lengths are added as the wall goes up until the masonry cap unit with the stressing-end anchor is installed. Unbonded sheathed strand is installed into the duct or pipe prior to grouting the masonry wall. After the grout has achieved the specified strength, the tendon is stressed.

The Holy Cross Church in Santa Cruz, Calif. was strengthened to resist seismic loads with strand tendons placed in core-drilled holes. Fig 2.82 shows the structure after completion of the strengthening.

Fig. 2.84 Barrier Cables in Nike Parking Garage
Courtesy of Cary Kopczynski & Company, Inc.

Fig. 2.85 Preheat Tower, CO
Courtesy of VSL

The National Concrete Masonry Association's model concrete home for the future, Lifestyle 2000, used post-tensioning to strengthen the foundation walls. Single-strand unbonded 0.5 in. tendons were used in the walls and spaced 32 in. apart. The tendons were eccentrically located within the block cavities to maximize their effectiveness in resisting lateral earth pressure; Fig. 2.83.

2.12 BARRIER CABLES

The selection and design of a vehicle barrier system is an important element in the structural design of many structures, particularly parking garages. On open-sided structures, some type of barrier system must be erected at the perimeter and at the open edges of the ramps to prevent automobiles and pedestrians from falling from the open sides. The use of prestressed 7-wire steel strand provides an attractive and cost-effective solution to these situations. Fig. 2.84 illustrates the use of barrier cables for the Nike Parking Garage in Beaverton, Ore.

2.13 REPAIR AND REHABILITATION

Post-tensioning has been effectively used to rehabilitate/strengthen the flexural and shear capacity of both reinforced and prestressed concrete members. An active external force is applied to the structural member

Fig. 2.86 Strengthening of Retail Store Parking Garage, Springfield, VA

using post-tensioned (stressed) tendons to resist new loads. In most cases post-tensioning adds a minimal amount of weight to the existing structure. The repairs can be performed quickly with very little disruption to the occupants. The following sections give some specific examples where post-tensioning has been effectively used to rehabilitate existing structures.

2.13.1 Preheater Tower, CO

Within two weeks of initial start-up, cracks were discovered in the concrete columns at three different levels in the 328 ft tall preheater tower at a Portland Precalciner Cement Plant in Colorado. These cracks coincided with the terminations of flexural reinforcement in the concrete beams. Upon discovery of the cracks, production was stopped in order to assess the condition of the tower and begin investigation of the cause of the cracks. Engineering analysis concluded that the anchorage of the beam flexural reinforcement into the columns for both negative and positive moment (positive under lateral loads) would require strengthening and that all beam/column joints required retrofitting.

A scheme of core drilling the beams along their length and installing grouted post-tensioning was selected for strengthening the beams and beam column joints. The core drilling of up to 87 ft long holes axially in the beams required considerable amount of skill and precision. Ground-penetrating radar was used to monitor the progress of the holes as they were drilled and corrections were made as necessary. The post-tensioning was installed in the holes, stressed and grouted to complete the rehabilitation of the towers.

This method afforded the repair team the ability to execute a fast-track schedule, despite challenging circumstances, which included working high on the exposed structure through a cold winter with severe wind conditions. The unique retrofit also resulted in a structure that is stronger, more serviceable, and more durable than the original tower; Fig. 2.85.

2.13.2 Strengthening of Retail Store Parking Garage, Springfield, VA

A Springfield, Va. home improvement store utilizes a multi-level parking structure to accommodate patrons. The main entrances to the store are located on the first elevated deck while vehicular entrances are located on the ground level as well as first elevated deck. The garage is a precast prestressed concrete structure that consisted of 60 ft double tees supported on 30 ft inverted T-beams; Fig. 2.86.

Due to a limitation of the original design, a 9500 lb gross vehicular weight (GVW) restriction was imposed, severely affecting the ability of the store to serve the contractor-type business. To resolve this challenge, the owner aimed at increasing the capacity to approximately 20,000 lb GVW. Evaluation of the garage by the Engineer-of-Record revealed that achieving this new capacity required flexural upgrade of the structural components. Several strengthening options were evaluated by the contractor and led to a unique and cost-effective solution that was least disruptive to store operations and pedestrian and vehicular traffic.

The strengthening solution consisted of a bonded reinforced concrete overlay, external post-tensioning systems, and externally bonded FRP, to increase the load carrying capacities of the slab, double tees, and main girders of the garage, respectively. External post-tensioning, an active strengthening technique, was selected for upgrading the double tees that provided active force to offset effects of the new bonded overlay and produced a strengthening system compatible with the existing construction. This unique system consisted of two loops of 0.6-inch strands, one on each side of a stem, that were anchored using heavy-wall steel pipes installed at the ends of the double tee stems. The system is stressed at the center of each stem using a special steel anchoring block and stressing rams. The post-tensioned system was specially detailed and manufactured to be completely watertight. The project was completed on time, within budget, and with minimal disruptions to store clients.

2.13.3 University Hall Tension Ring Repair, Charlottesville, VA

University Hall, located in Charlottesville, Va., is the home of the University of Virginia men and women's basketball teams. With a seating capacity of more than 9000, it serves as a key sports and entertainment facility for the region. The building, similar but smaller in structure to the Houston Astrodome, was originally completed in 1965. It is covered by an 87-meter diameter concrete dome, which consists of 32 pre-cast segments with a cast-in-place thrust ring that was originally prestressed by being wrapped with smooth wire encased in concrete. During a structural inspection in the fall of 1998, it was discovered that the prestressing steel strands in the ring beam had experienced pitting corrosion and that some wire and strand breakage had occurred.

The repair scheme adopted consisted of strengthening the ring beam with external tendons. Deviator blocks were installed in the tension ring at the columns where the pre-cast shells were supported. Post-tensioning tendons were then installed and stressed. In order to ensure that a uniform load was applied to the structure, the complete loop was stressed simultaneously in four sections. After stressing was completed, the strands were grouted with a high-quality, protective cement grout.

The strengthening of the University Hall ring beam utilized a unique application of post-tensioning to provide a

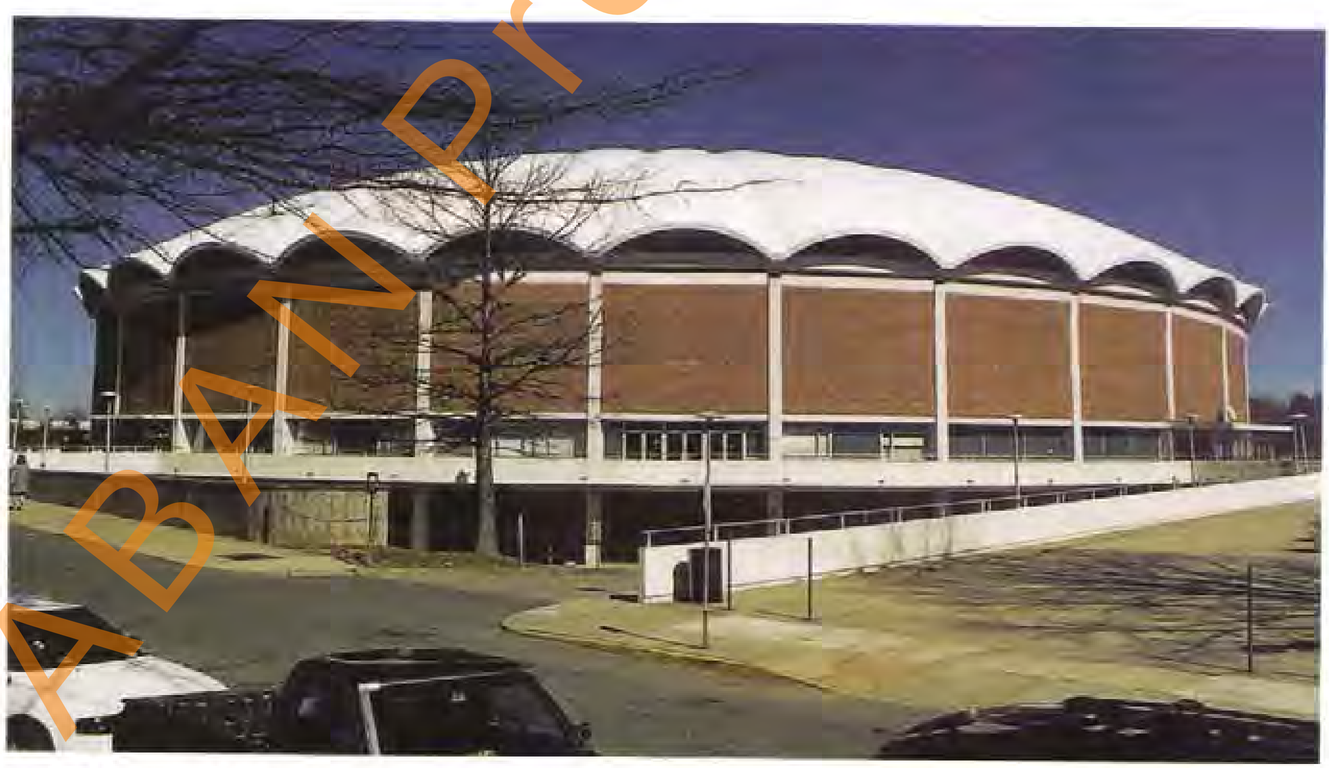


Fig. 2.87 University Hall Tension Ring Repair, Charlottesville, VA
Courtesy of VSL

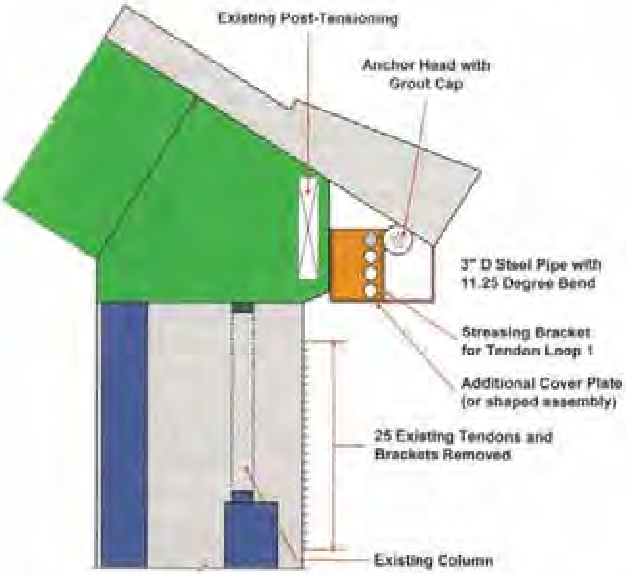


Fig. 2.88 Repair Detail for University Hall Tension Ring, Charlottesville, VA
Courtesy of VSL

Fig. 2.89 Fallingwater Rehabilitation
Courtesy of Mario Suarez and VSL

structurally elegant and aesthetically pleasing result; Fig. 2.87. The team devised an efficient and unobtrusive strengthening solution that could be accomplished during an extremely short time frame while allowing complete access to the facility during construction operations, Fig. 2.88. The project was completed on time and under budget.

2.13.4 Fallingwater, PA

Painted into the picturesque southwestern Pennsylvania landscape is a national treasure that is also one of Frank Lloyd Wright's most famous architectural home designs; Fig. 2.89. Unfortunately, the famous structure, Fallingwater, was at risk of failure. The challenges date back to the original construction in 1937. The concrete beams that support the hung-in-space living room, its two adjoining terraces, and the master bedroom terrace above were under-designed. The beams sagged during construction and continued to creep every year. By 1994 they were an alarming 4 to 7 inches out of level. Given the great amount of concern to public safety, the owner chose to install temporary shoring beneath the main level terrace until a permanent strengthening scheme could be designed.

The primary requirements for the permanent repair system were strength and aesthetics. The system would need to be strong enough to halt vertical creep while being invisible to visitors. After careful review, the design team selected external post-tensioning to retrofit the structure. Since post-tensioning was an active system, in addition to the increasing the capacity it offered a means of reducing the current deflections. Post-tensioning was also attractive from an aesthetic viewpoint because it could be hidden in the floor cavity between the girders and be virtually invisible to the public.

Thirteen-strand tendons were placed on each side of two of the girders. One 10-strand tendon was placed on the western side of the third girder. Eight 0.6-inch single-strand tendons were slated for the east-west direction. Reinforced concrete blocks doweled into the sides of the existing girders were used to anchor and profile the post-tensioning tendons. Small openings were cut into the existing south parapet wall to gain access for multi-strand tendons for stressing. Dead-end anchors were placed at the north end of the girders.

Stressing operations were carefully staged and sequenced. The four single-strand tendons were stressed in the east-west direction and then the four multi-strand tendons were stressed in the north-south direction. Stage stressing was beneficial because it allowed engineers to visually inspect the structure and monitor strains and deflections periodically. The single-strand tendons were tensioned to jacking forces of approximately 43 kips each. The 10-strand and 13-strand tendons were post-tensioned to jacking forces of 300 kips and 390 kips respectively. The multi-strand tendons were grouted with a high quality, low-bleed cementitious grout mixture.

All of the renovation work on the project was completed in a 10-week period during the winter months when the building normally closes to the public. No time was lost for visitor services. The client was thrilled with the entire process. The final result exceeded everyone's expectations and is a testament to its success. The renovation process did not have any architectural impact on the project. The team proudly claims that the house looks exactly the same after the renovation as it did before the work started.

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POST-TENSIONING SYSTEMS

3.1 GENERAL

Prestressing is a method of reinforcing concrete. Externally applied loads induce internal stresses (forces) in concrete during the construction and service phases of a member. The concrete is prestressed to counteract these anticipated stresses during the service life of the member.

There are two commonly used methods of prestressing concrete.[3.1] One is called pre-tensioning. The prefix "pre" means that the prestressing steel is stressed before the concrete is cast. This method consists of first stressing high-strength steel strands or wires between buttresses, and then casting the concrete around the steel. Once the concrete has reached a certain specified strength, the steel is cut between the ends of the members and the buttresses to transfer the prestressing forces to the concrete. This process typically takes place at a precast plant and requires the completed pretensioned concrete member to be trucked out to the job site and then assembled.

The other method of prestressing concrete is called post-tensioning. The prefix "post" means that the prestressing steel is stressed after the concrete is cast. Instead of stressing the high-strength steel between buttresses at a precast plant, the steel is simply installed on the job site after the contractor forms up the member. The high-strength steel is housed in a sheathing or duct that prevents it from bonding to the concrete. The steel is attached to the concrete at the ends of the member by specially designed anchorage devices. Once the concrete has cured (hardened), the steel is stressed to induce forces in the concrete. Post-tensioning has all of the advantages of prestressed concrete while allowing the builder the freedom to construct the member in any location, including its final position in the structure (cast-in-place).

3.2 TYPES OF POST-TENSIONING SYSTEMS

In most post-tensioned construction, the prestressing tendons are embedded in the concrete before the concrete is cast. These internally post-tensioned systems can be either bonded or unbonded. In some bridge and retrofit applications, the post-tensioning tendons are mounted outside the structural member. These are referred to as external post-tensioned systems.

In unbonded systems, the strand is kept unbonded to the surrounding concrete throughout its service life. In bonded systems grout is injected in the ducts to bond the prestressing strand to the surrounding concrete after it has been stressed. Once the grout has cured (hardened), the system behaves as an integral system without any relative movement between the steel and concrete. Most of the internally grouted post-tensioned systems are considered to be bonded. Unbonded systems allow relative movement between the strand and surrounding concrete throughout its service life. Most single-strand systems and all external post-tensioning systems fall under this category.

3.2.1 Unbonded Post-Tensioning Systems

The tendons in an unbonded system typically consist of single-strands that are coated with a corrosion-inhibiting coating and protected by extruded plastic sheathing. This allows the strand to move inside the plastic sheathing and prevents ingress of water. The strands are anchored to the concrete using ductile iron anchors and hardened steel wedges. The tendon is supported by chairs and bolsters along its length to maintain the desired profile. Fig. 3.1 shows the typical components and construction sequence for an unbonded system.

Depending on the exposure of the single-strand unbonded system it can be classified as a standard or an encapsulated system. Encapsulated systems are required for aggressive environments where there is a possibility of tendon exposure to chlorides or other deleterious substances. Encapsulated tendons are designed to prevent any ingress of water during and after construction. Fig. 3.2 shows an example of a standard and encapsulated tendon.

3.2.2 Bonded Post-Tensioning Systems

Bonded post-tensioning systems consist of tendons with multiple strands or bars. The strands or bars are placed in corrugated galvanized steel, high density polyethylene (HDPE) or polypropylene (PP) ducts. Depending on the site conditions and system used, the strands may be installed before the concrete is placed or the ducts may be installed without the strands. The strands are then pulled or pushed through the ducts. Once the concrete has hardened the tendons are stressed and the ducts filled with grout. Inlets and outlets are provided at high/low points to ensure that the grout fills the ducts completely. Fig. 3.3 shows the components of a typical multistrand grouted system. The grout provides an alkaline environment and protects the prestressing strands from corrosion. It also bonds the strands to the surrounding concrete.

3.2.3 External Post-Tensioning Systems

Tendons in an external post-tensioned system are installed outside of the structural concrete member except at anchorages and deviation points. External tendons can be either straight between anchorages or can run through deviator blocks to create a harped profile. External tendons are used in bridges, retrofit and repair applications. Fig. 3.4 shows the application of external tendons in a retrofit application.

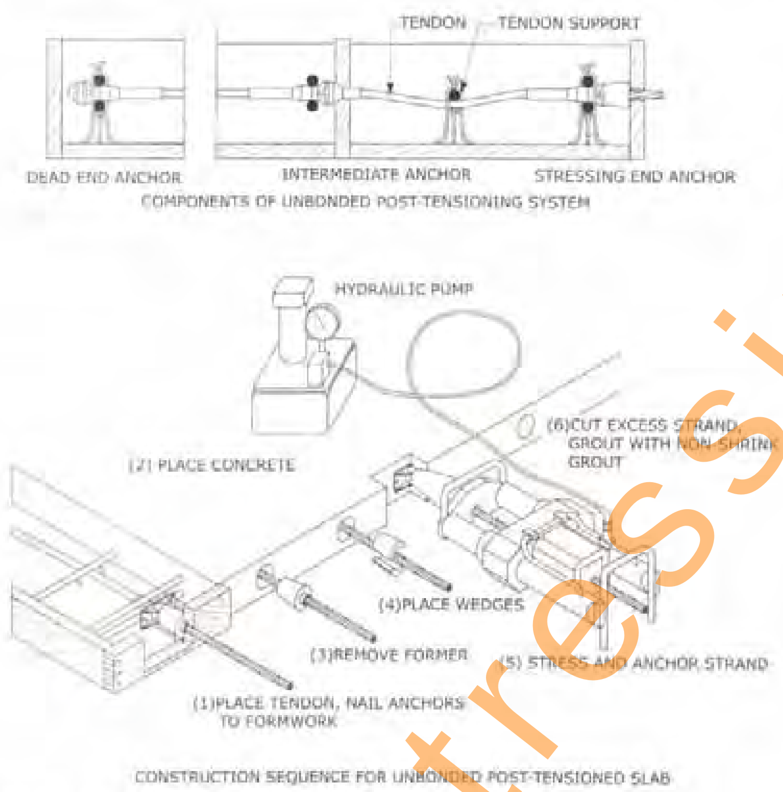

Fig. 3.1 Components and Construction Sequence for Unbonded Post-Tensioning System[3.2]

Fig. 3.2 Standard and Encapsulated Tendon Assembly

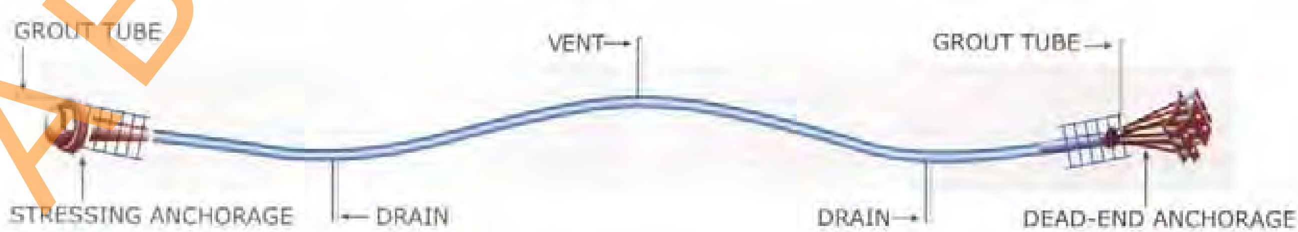

Fig. 3.3 Components of a Multistrand System[3.3]
Courtesy of VSL

Fig. 3.4 External Tendons in a Retrofit Application
Courtesy of Seneca Structural Engineering, Inc.

Prestressing steel used in external post-tensioned systems is either greased and plastic sheathed (as in a typical unbonded single-strand tendon), or enclosed in a duct which is subsequently filled with grout. Greased and plastic-sheathed tendons and associated hardware are covered with a coating such as metal lath and plaster, when required to provide fire protection. Regardless of tendon coating, externally post-tensioned systems allow relative movement between the tendon and the member to which it is attached and are therefore considered to be unbonded in practice.

3.3 COMPONENTS OF A TENDON

Post-tensioning systems can be broadly divided into three categories—single-strand, multistrand, or bar, depending on the type of prestressing steel used. This section discusses these categories and is intended to provide a general overview of the various available systems. Information on design, detailing and material specifications of most of the commercially available systems is presented in a separate publication, *PTI Directory of Post-Tensioning Systems, Products, and Services*.[3.4]

The entire assembly of high-strength prestressing steel, end anchorage, and the duct is referred to as a "tendon." In a single-strand post-tensioning system, the tendon consists of a single 7-wire steel strand, P/T coating, fixed end anchorage, stressing end anchorage, and intermediate anchorage for long tendons. A specially shaped piece of plastic called a pocket former is used at the stressing end to create a pocket in the concrete for fitting the stressing jack and embedding the anchors. After the tendon is stressed, the pocket is grouted with a high-strength non-shrink grout to prevent ingress of moisture. Fig. 3.1 shows the components and construction sequence of an unbonded system.

In a multistrand system, multiple strands are typically installed in a single duct. As discussed in Section 3.3.4, ducts create voids in the concrete and may be made of galvanized steel, HDPE or PP. Multistrand anchors are specially designed devices that are supplied by the post-tensioning supplier. These devices are intended to anchor multiple strands to concrete and are specially designed to accommodate the concentrated forces produced in the anchorage zone. Strands in a bonded system are typically installed inside the duct without any P/T coating. Once the strands have been stressed, the ducts are grouted to bond the strands to the surrounding concrete and protect the strands from corrosion. In very aggressive environments epoxy coated strands may be used. Galvanized strands are generally used only for barrier cable applications. Fig. 3.3 shows the components of a multistrand system.

Bar tendons have either single or multiple bars. The tendons consist of high-strength steel rods, anchorages and ducts. Bar couplers are used for long tendons. Fig. 3.5 shows an assembly of a single-bar tendon.

The following sections describe each of the components of an unbonded and multistrand tendon. Bar tendons can be used in bonded as well as unbonded applications.

3.3.1 Prestressing Steel

The prestressing steel used in a post-tensioning system can be either high-strength 7-wire strand or bar. For most prestressing applications, 7-wire strands are made by helically wrapping six wires around a central straight wire. The diameter of strands supplied in North America typically ranges from 0.375 in. to 0.6 in. Strands are typically low

Fig. 3.5 Bar Tendon Assembly
Courtesy of DYWIDAG-Systems International, USA

relaxation steel with an ultimate tensile strength of 270 ksi. The low relaxation properties are achieved by a process called *stabilizing*. In this thermo mechanical process, the high-strength steel strand is stretched to a pre-determined tension and heated. This results in a substantial increase in its resistance to relaxation. Almost all of the steel strand produced in North America is low relaxation. Fig. 3.6 shows the assembly of a bonded and unbonded system with 7-wire strands.

Prestressing bars typically have an ultimate strength of 150 ksi, and diameters ranging from 0.625 in. to 2.5 in. Couplers are used to connect bars and lengthen the bar tendons. The types and configurations of bars vary by suppliers. Bar tendons are typically used when short, straight tendons are required.

3.3.2 Anchorages

Anchorages are mechanical devices that transmit the tendon force to the concrete. For single-strand tendons this includes wedges that grip the strands and a bearing plate that transfers the tendon force to the concrete. Fig. 3.7 shows a typical anchorage assembly for a single-strand tendon and Fig. 3.8 shows a few types of anchorages used for multistrand systems. Bearing plates with nuts are used in bar systems, as shown in Fig. 3.5.

3.3.2.1 Stressing End Anchorages

Stressing end anchors are used to stress the strand on site. A pocket former is typically used during forming and casting operations to embed the stressing anchors in the concrete. After stressing, the tendon tails are cut, and the pocket is grouted with non-shrink grout to prevent the ingress of water. Figs. 3.7, 3.8(a) and 3.5 show the stressing anchorage arrangements for unbonded, multistrand, and bar systems respectively.

3.3.2.2 Fixed End Anchorages

For unbonded systems, the fixed-end anchorages are typically installed at the fabrication facility before the tendons are shipped to the project site. This involves stressing the tendon to a specified load to seat the wedges securely in the anchor. This ensures that no slippage occurs at the fixed end during the stressing operation. Fixed-end anchorages are used when the tendon is stressed from one end only.

Proprietary anchorage systems are commonly used in multistrand systems. For multistrand tendons, the anchorage at the fixed end can be achieved by splaying the strands or bonding the strand to the concrete for a sufficient length beyond the end of the member [see Fig. 3.8(b)]. However, this method requires the tendons to be placed before the concrete is placed. Some fabricators prefer to use the

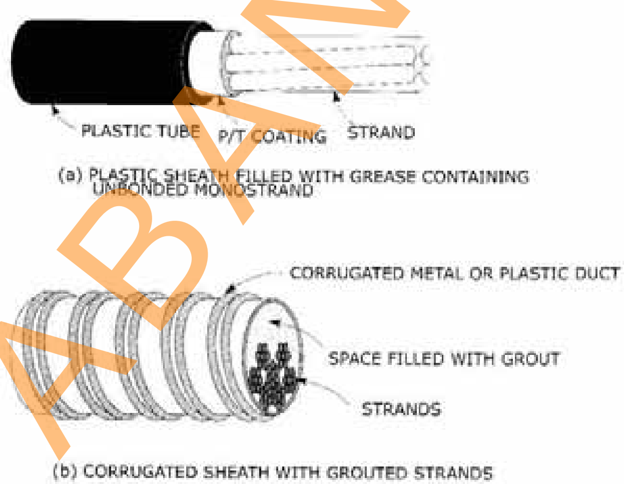

Fig. 3.6 Unbonded and Bonded Systems[3.2]

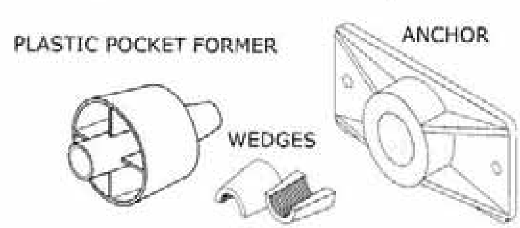

Fig. 3.7 Stressing End Anchorage Assembly[3.2]

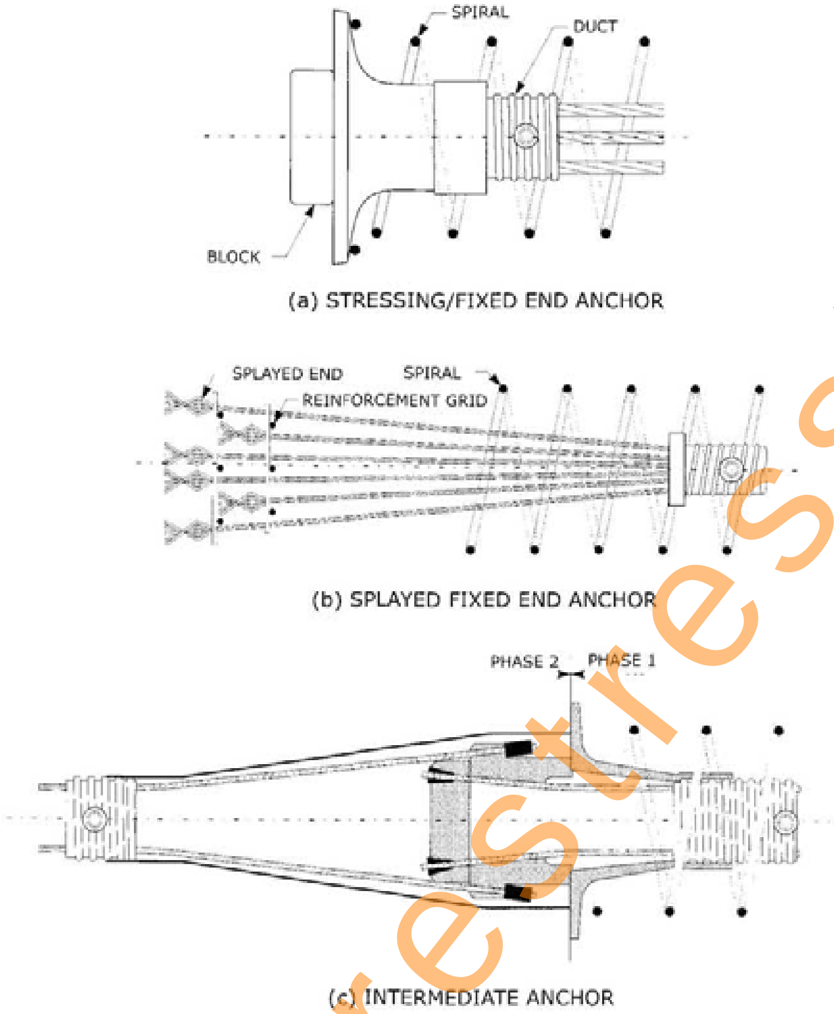

Fig. 3.8 Anchorage Systems for Bonded Construction

stressing end anchor as shown in Fig. 3.8(a) at the fixed as well as at the stressing end. This allows the contractor flexibility in placement of strands after the concrete has been placed.

3.3.2.3 Intermediate Anchorages

When the tendon is very long, or for staged construction, it may be necessary to provide a construction joint along the length of the tendon. An intermediate anchorage is required to stress the strand at a construction joint. Figs. 3.1 and 3.8(c) show typical intermediate stressing anchorages for unbonded and multistrand systems respectively.

3.3.3 P/T Coating

Strands in unbonded construction are coated with a corrosion-inhibiting material that typically consists of special grease. The coating is usually applied to the strand as a part of the extrusion process. It acts as a barrier for ingress of water, inhibits corrosion of the steel and lubricates the strand so that it can move independently of the surrounding concrete.

3.3.4 Duct/Sheathing

Ducts are used in bonded and, in some cases, in external post-tensioning to provide a void that permits the installation and stressing of strands after the concrete has been placed and hardened. The ducts also provide protection to the post-tensioning strands after construction. Ducts for post-tensioning systems can be either rigid or semi-rigid and made from ferrous metal, HDPE or PP. Ducts may be round, oval or flat. For bonded post-tensioning, the ducts are corrugated to facilitate the transfer of force between the tendon and the concrete. In contrast, the ducts for external post-tensioning usually have smooth walls.

Use of HDPE or PP ducts is recommended for corrosive environments. Plastic ducts provide a non-corrosive impermeable barrier between the concrete and the grout. Metallic ducts are usually galvanized to provide a degree of corrosion protection both before and after construction. Galvanized ferrous ducts also provide a barrier to water ingress but are not impermeable and may corrode over time in aggressive environments. This may lead to an increase in the penetration of moisture and chlorides or other deleterious substances, potentially reducing the long-term durability of the structure.

Ducts are normally joined with fittings and sleeves that minimize grout leakage and water ingress. Recently, a number of specially designed fittings have been developed for plastic duct systems that are essentially watertight. These fittings, if properly installed, can significantly decrease the amount of water that gets into the tendons and can greatly enhance the structure's long-term durability.

Plastic sheathing is used for unbonded post-tensioning. Polyethylene is directly extruded onto individual strands that are coated with P/T coating [Fig. 3.6(a)]. The plastic sheathing provides a barrier that is in direct contact with the concrete and permits the lubricated strand to slide independently during stressing and service loading. The plastic must be impermeable to water and other corrosion-causing contaminants such as chloride or other deleterious substances, and serves as a barrier to corrosion. The sheathing must also be sufficiently durable to permit handling in the field and stressing without causing breaks and tears that would expose the underlying steel strand.

3.3.5 Grout

In bonded construction the ducts containing the strands are filled with cement grout as soon as possible after stressing of the tendons. The grout serves several important functions. First the grout bonds the strand to the duct and hence to the surrounding concrete, facilitating the transfer of force between the tendon and the concrete. Second, the grout provides a cementitious cover that slows the ingress of water and corrosion-causing contaminants. Third, the alkalinity of the grout creates a passive environment for steel, further inhibiting corrosion.

To be effective, the grout must essentially fill the voids in the tendon. To do so, it must be fluid enough to be easily pumped over long distances in confined spaces without excessively high pumping pressure that could burst the duct or damage the structure, and it must maintain its fluidity during the grouting operations. As discussed in greater detail in Section 4.4.6, non-bleed grout should be used to provide continuous encapsulation of the tendons. Detailed information about specifying grout and grouting procedures can be found in Section 4.4.6.

3.4 CHOICE OF POST-TENSIONING SYSTEM

The choice of post-tensioning system usually involves trade-offs between structural and construction considerations. Usually more than one type of system may be feasible for a particular application. The decision is typically governed by economic considerations. Unbonded tendons are common in building construction. The bonded system is typical of bridge construction and is usually used in larger members such as beams and girders. External post-tensioning is used extensively for repair and rehabilitation applications, and in some bridge and building structures. Bar systems (which can be bonded or unbonded) are typically feasible in relatively short members where harped or straight tendons are required, and are common in vertical applications such as walls, piers and reinforced masonry.

The choice of a bonded or unbonded post-tensioning system involves the technical characteristics and differences inherent in each type of tendon and the economics related to those differences. The important technical considerations are strength, corrosion protection, and redundancy. Each is discussed in the following sections.

3.4.1 Strength of Bonded and Unbonded Systems

A properly grouted bonded tendon results in bond between the tendon and the surrounding concrete. When such bond exists, a change in strain in the concrete adjacent to the tendon results in same change of strain in the prestressing steel. This condition is known as "strain compatibility." This compatibility of strain between the concrete and the prestressing steel means that a bonded tendon will develop more force at design (factored) loads than will an unbonded tendon with the same cross-sectional area [see ACI 318[3.5] equations for stress at nominal strength in bonded and unbonded tendons, Eqs. (18-3) and (18-4/5) respectively].

For equivalent flexural strength between the two systems, additional bonded non-prestressed reinforcement is normally added in the unbonded system, supplementing the lower force in the unbonded tendon. The additional bonded non-prestressed reinforcement in the unbonded system serves the same purpose as the grout in the bonded system. A minimum amount of non-prestressed reinforcement is also required by ACI 318-02 when unbonded tendons are used to provide flexural performance and crack distribution equivalent to members with bonded tendons (see ACI 318-02 Sections 18.9 and R18.9).

3.4.2 Corrosion Protection of Bonded and Unbonded Tendons

Bonded tendons are protected from corrosion by the surrounding grout. If the cementitious grout comes in contact with all surfaces of the prestressing steel, the resulting alkaline environment is extremely effective in resisting corrosion.

Corrosion protection in unbonded tendons is provided by a factory-applied corrosion-inhibiting coating material and an extruded plastic sheath. In highly aggressive corrosive environments the tendon can be completely encapsulated, providing additional protection against corrosion. Unbonded tendons fabricated in accordance with current tendon material specifications[3.6] have an excellent performance record in resisting corrosion and are believed to be equivalent in corrosion resistance to a properly grouted bonded tendon.

3.4.3 Redundancy and Safety of Bonded and Unbonded Tendons

In an unbonded tendon all prestressing force is transferred to the concrete by the anchorages alone. A failure in the unbonded tendon at any point will cause a loss of prestress force throughout the entire length of the tendon between anchorages. In a bonded tendon, the prestressing force is transferred to the concrete through a combination of bearing at the anchorages and bond with the concrete along the full length of the tendon. A failure in a bonded tendon will, of course, reduce or eliminate the prestressing force at the failed section, but the full force should be unaffected at and beyond a full development length on either side of the failed section.

This condition has long been recognized by building codes and experienced design professionals. It is mitigated by the addition of bonded reinforcing steel in one-way unbonded beams and slabs, which provides a secondary load path, independent of the tendons, in the event of a large loss of prestress force. The minimum amount of reinforcing required in ACI 318-02 Section 18.9 is effective in providing this secondary load path. ACI/ASCE Committee 423 recommends consideration of providing a fully developed system of non-prestressed bonded reinforcing in all one-way unbonded beams and slabs with the capacity, independent of the prestressing tendons, to resist an applied load equal to the entire unfactored dead load and 25% of the unfactored live load. Thus, in the unlikely event that *all* prestress force is lost in the member, it will still have the capacity to resist, without collapse, the full member dead load plus a realistic percentage of the design live load (see ACI 423.3R-05 Section 2.2.1).[3.7]

Because of the inherent redundancy of two-way systems, the loss of even a large amount of prestress force in unbonded two-way systems is not considered to present the same vulnerability as in one-way systems. In tests of a nine-panel two-way unbonded slab performed at the University of Texas in 1975 all of the tendons in the center panel in both directions were intentionally detensioned, simulating the loss of the entire center panel, and the loss of half of the prestress force in the four edge panels. Under those extreme conditions the slab was still capable of supporting its full service dead and live load.[3.8]

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3.6 *Specification for Unbonded Single Strand Tendons*, 2nd Edition, Post-Tensioning Institute, Phoenix, AZ, 2000.

3.7 *Recommendations for Concrete Members Prestressed with Unbonded Tendons*, ACI 423.3-05, American Concrete Institute, Farmington Hills, MI, 2005.

3.8 *Design of Post-Tensioned Slabs Using Unbonded Tendons*, 3rd Edition, Post-Tensioning Institute, Phoenix, AZ, 2004.

DEFINITIONS OF COMMONLY USED TERMS

Definitions of terms as used in this manual are as follows:

Added Tendons: Tendons, usually short in length, placed in specific locations such as end bays to increase the structural capacity at the location without having to use full-length tendons.

Admixture: Material other than water, aggregate, or hydraulic cement, used as an ingredient of concrete or grout and added to concrete or grout before or during its mixing to modify its properties.

Aggressive Environment: An environment in which structures are exposed to direct or indirect applications of deicing chemicals, seawater, brackish water, or spray from these water sources; and salt-laden air as occurs in the vicinity of seacoasts. Aggressive environments also include structures where stressing pockets are wetted or are directly in contact with soils which contain chloride levels considered by the geotechnical engineer to be harmful to metals.

Anchor Cavity: The opening in the anchor or anchor block designed to accommodate the strand passing through and the proper seating of the wedges.

Anchor Nut: Threaded device that screws onto a threaded bar and transfers the force from the bar to the bearing plate.

Anchor: See Anchorage.

Anchorage Zone: The portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section. Its extent is equal to the largest dimension of the cross section. For anchorage devices located away from the end of the member, the anchorage zone includes the disturbed regions ahead of and behind the anchorage.

Anchorage: A mechanical device comprising all components required to anchor the prestressing steel and permanently transfer the post-tensioning force from the prestressing steel to the concrete.

Anticipated Set: The expected movement of the wedges into the anchorage during the transfer of the prestressing force to the anchorage device.

Back Stressing: A stressing procedure that ensures that the wedges are properly seated into the anchor at a given location on the tendon.

Back-Up Bars: Reinforcing bars placed in concrete in the anchorage zone to position the anchor and help in distributing the loads.

Banded Tendons: Group(s) of closely spaced tendons in slabs placed together in a narrow strip, usually along the column line.

Bar: Bars used in post-tensioning tendons conform to ASTM A722, *Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete*. Bars have a minimum ultimate tensile strength of 150,000 psi (1035 MPa). Type 1 Bar has a plain surface and Type 2 Bar has surface deformations.

Barrel Anchor: A cylindrical metal device housing the wedges and normally used with a bearing plate to transfer the prestressing force to the concrete.

Barrier Cable: High-strength steel strands erected around the perimeter of a structure and at open edges of ramps to prevent automobiles and pedestrians from falling over the open sides.

Bearing Plate: A plate which bears directly against the concrete and is part of an overall anchorage system.

Blowout: A localized concrete failure which occurs during or after stressing.

Bonded Tendon: Tendon in which prestressing steel is bonded to concrete either directly or through grouting.

Bulkhead: See Edge Form.

Bursting Steel: Reinforcing steel used to control the tensile bursting forces developed at the bearing side of the anchor as the concentrated anchor force from the stressed tendon spreads out in all directions.

Cable: A term used by some to denote a prestressing strand or a single-strand tendon.

Camber: An upward deflection that is caused by the application of prestressing force. Camber is intentionally built in a structural element or form to improve appearance or to nullify the deflection of the element under the effects of loads, shrinkage, and creep.

Cantilever: Any horizontal structural member projecting beyond its vertical support.

Casting: See Anchorage.

Chair: Hardware used to support or hold post-tensioning tendons or reinforcing bars in their proper position to prevent displacement before and during concrete placement.

Chuck: See Barrel Anchor.

Coating: Material used to protect the prestressing steel from corrosion and reduce the friction. (See P/T Coating.)

Concrete Slurry: Cementitious paste mixed with aggregate fines. (From ready mix concrete.)

Constructor: The person, firm, or organization who had entered into a contractual agreement with the Owner to construct the project and who has the prime responsibility for the overall construction of the project in accordance with contract documents.

Corrosion Inhibiting Coating: See P/T Coating.

Coupler: A device designed to connect ends of two strands together, thereby transferring the prestressing force from end to end of the tendon.

Creep: The time-dependent deformation (shortening) of concrete under sustained stress (load).

Curvature Friction: Friction resulting from bends or curves in the specified prestressing tendon profile.

Dead-End Anchorage: See Fixed-End Anchorage.

Design Professional: The person, firm, or organization responsible for preparing the contract documents of the project. (In most cases the person or firm is licensed in the jurisdiction of the project).

De-tensioning: Releasing the prestressing force from the tendon.

Distributed Tendons: Single or group of tendons in a slab that are uniformly distributed, usually perpendicular to the banded tendons and spaced at a maximum of eight times the slab thickness or 5 ft (1.5m).

Donut: See Barrel Anchor.

Drape: See Profile.

Duct:	A conduit (plain or corrugated) to accommodate prestressing steel for post-tensioning installation.
Eccentricity:	Distance between the center of gravity of the concrete cross-section and center of gravity of the prestressing steel at any point along the length of a member
Edge Form:	Formwork used to limit the horizontal spread of fresh concrete on flat surfaces such as floors.
Effective Force:	See Effective Prestress.
Effective Prestress:	Stress remaining in prestressing steel after all losses have occurred.
Elastic Shortening:	The shortening of a member that occurs immediately after the application of the prestressing force.
Elongation:	Increase in the length of the prestressing steel under the applied prestressing force.
Encapsulated System:	A post-tensioning system that prevents the ingress of water into the tendon during all stages of construction, and isolates the strand and anchorage from contact with the concrete.
Engineer:	See Design Professional.
Fixed-End Anchorage:	An anchorage at the end of a tendon where stressing jack is not attached during stressing operations. Fixed-end anchorages are typically attached to the strand at the fabrication plant.
Force:	When used in post-tensioning applications force is the load applied to the structure by the tendon.
Friction Loss:	The loss of force in a prestressing tendon resulting from friction created between the strand and sheathing due to curvature and wobble during stressing.
Gauge:	A device used to measure the hydraulic pressure delivered by the hydraulic pump.
Grease:	See P/T Coating.
Grout:	A mixture of cementitious materials and water, with or without mineral additives, admixtures or fine aggregate, proportioned to produce a pumpable consistency without segregation.
Hand Seating Tool:	A small handheld device used to properly align (seat) the wedges in the anchor prior to attaching the jack to the strand for stressing.
HDPE:	Acronym for High Density Polyethylene plastic. HDPE has a minimum density of 0.941 gm/cm^3 in post-tensioning applications.
Honeycombing:	Voids in the concrete caused by inadequate consolidation.
Initial Concrete Strength:	The strength of the concrete necessary for the post-tensioning operation to begin. Typically specified by the design engineer or post-tensioning material supplier.
Initial Force:	See Initial Prestress.
Initial Prestress:	The force in the tendon immediately after transferring the prestressing force to the concrete. This occurs after the wedges have been seated.
Inlet:	Opening used to inject grout into the duct.
Installation Drawings:	Drawings furnished by the post-tensioning supplier showing information such as the number, size, length, marking, location, elongation, and profile of each tendon to be placed.

Intermediate Anchorage: An anchorage located at any point along the tendon length, which can be used to stress a given length of tendon without the need to cut the tendon. Normally used at concrete pour breaks.

Jack Calibration: A chart showing related gauge pressure to actual force applied to a tendon.

Jack Gripper Plates: Steel plates designed to hold the jack grippers in place in the jack.

Jack Grippers: Wedges used in the jack to hold the strand during the stressing operation.

Jack: A mechanical device (normally hydraulic) used to apply force to a prestressing tendon.

Jacking Force: The maximum temporary force exerted by the jack on the tendon.

Lift Off: The field procedure used to verify the force in a tendon.

Live-End: See Stressing-End.

Material Certifications: Documentation from manufacturer that confirms that the quality of material supplied meets all project requirements.

Modulus of Elasticity: Ratio of stress to corresponding strain for tensile or compressive stresses below proportional limit of material.

Monostrand: One single-strand.

Multistrand: More than one single-strand in a tendon.

Multi-Use Splice Chuck: A coupler made for repeated use.

MUTS: Minimum Ultimate Tensile Strength

Non-Aggressive Environment: All environments not specifically defined as aggressive, including enclosed buildings.

Nose-piece: The front part of the jacking device that fits into the stressing pocket, to align the jack with the anchor.

Outlet: Opening to allow the escape of air, water, grout and bleed water from the duct during grouting operation.

Owner: The person, firm, or organization that initiated the design and construction of the project, provides or arranges for funding, is responsible for partial and final payments and who will take possession and ownership of the project upon completion.

P/T Coating: Material used to protect against corrosion and reduce friction between prestressing steel and sheathing. For unbonded applications P/T coating should meet or exceed the performance criteria outlined in the PTI *Specifications for Unbonded Single Strand Tendons*.

Pocket Former: A temporary device used at the stressing end to provide a cavity that can be grouted after the prestressing operation is complete.

Post-Tensioning Installer: Contracting entity responsible for unloading the post-tensioning materials, storing and protecting them on the job site at all stages of handling, storage, placement, tendon installation, stressing, and tendon finishing in accordance with the contract documents and this specification.

Post-Tensioning Supplier: Contracting entity responsible for providing all components of the post-tensioning system including the tendons, anchorages, couplers, field placement drawings, and stressing equipment, and delivering them to the job site.

Post-Tensioning: Method of prestressing in which prestressing steel is tensioned after concrete has hardened.

Potable Water: Water as defined by EPA (Environmental Protection Agency) to meet drinking water standards.

Prestressed Concrete: Structural concrete in which internal stresses are introduced to reduce potential tensile stresses in concrete resulting from applied loads.

Prestressing Steel: High-strength steel, most commonly a 7-wire strand, used to impart prestress forces to concrete.

Pretensioning: A method of prestressing in which the tendons are tensioned before the concrete has been placed.

Profile: The path of a tendon in concrete from end to end.

Pump: A hydraulic pump used to provide hydraulic pressure to the stressing jack.

Quality Assurance: Actions taken by an Owner or his representative to provide assurance to the owner that the work meets the project requirements and all applicable standards of good practice.

Quality Control: Actions taken by the Contractor to ensure that the work meets the project requirements and all applicable standards of good practice.

Reference Point: The painted mark placed on a tendon tail used to measure the elongation of a tendon after stressing.

Seating-Loss: The relative movement of the wedges into the anchor cavity during the transfer of the prestressing force to the anchorage resulting in some loss of prestressing force.

Sheathing: A material encasing prestressing steel to prevent bonding of the prestressing steel with the surrounding concrete, provide corrosion protection, and contain post-tensioning coating.

Shipping List: A detailed list of specific materials included in a particular shipment of material.

Shop Drawings: See Installation Drawings.

Slab Bolster: Continuous hardware used to support or hold post-tensioning tendons in place prior to and during concrete placement. See Chair.

Slurry: See Concrete Slurry.

Special Inspector: Individual certified by the International Code Council (ICC) to conduct special inspections of prestressed concrete.

Splice Chuck: See Coupler.

Split Donut: See Troubleshooting Anchor.

Split Pocket Former: A temporary two-piece device used at the intermediate end during casting of the concrete to provide an opening in the concrete, allowing the stressing equipment access to the anchor cavity.

Stage Stressing: Sequential stressing of tendons in separate steps or stages in lieu of stressing all the tendons during the same stressing operation.

Strand Slippage: Slippage or relative movement of strand with respect to wedges during force transfer. See seating loss.

Strand: High-strength steel wires helically placed around a center wire. For unbonded tendons typically a 7-wire strand.

Stressing End Anchorage: The anchorage at the end of a tendon where the stressing jack is attached to the tendon during stressing operations.

Stressing Equipment: Consists normally of a jack, pump, hoses, and a pressure gauge.

Stressing Force: See Jacking Force.

Stressing Pocket: The void created by the pocket former between the stressing anchor and the edge of the concrete to allow access for the stressing equipment. After stressing this void is filled in with an approved grout to provide protection for the tendon end.

Stressing Record: A permanent record of the actual tendon elongations after stressing produced by the inspector.

Stressing-End Anchorage: The anchorage at the stressing-end of a tendon which is used to stress the prestressing steel (strand).

Stressing-End: The end of the tendon at which the prestressing force is applied.

Subcontractor: A person, firm, or organization engaged by the Contractor to provide selected construction activities, materials or other specialized construction or engineering services.

Temperature Tendons: Tendons used to resist shrinkage and temperature stresses.

Tendon Bundle: A bundle of individual coiled tendons banded together.

Tendon Coil: An individual monostrand tendon coiled and wired together.

Tendon Group: More than one strand of Prestressed steel tied together to form a tendon.

Tendon Profile: See Profile.

Tendon Support System: The required support bars, chairs, bolsters and other accessories required to maintain the tendon profile.

Tendon Tail: The excess strand protruding beyond the stressing-end anchor.

Tendon: In post-tensioned applications, the tendon is a complete assembly consisting of anchorages, prestressing steel, and sheathing with post-tensioning coating for unbonded applications or ducts with grout for bonded applications.

Thixotropic: The property of a material that enables it to stiffen in a short time while at rest, but to acquire a lower viscosity when mechanically agitated. The process is reversible.

Threshold Inspector: This is a term employed by certain states to define a qualified professional engineer who inspects structures of certain defined parameters, and who also inspects the post-tensioning tendons.

Troubleshooting Anchor: A special anchor used for structural modification or repair of existing tendons. The anchor consists of a removable segment which allows it to slide onto an existing strand. The segment is then returned and tightened by screw or bolt.

Ultimate Strength: The tension force or stress that is required to fail a steel element in tension.

Unbonded Tendon: Tendon in which prestressing steel is prevented from bonding to concrete and is free to move relative to concrete. The prestressing force is permanently transferred to concrete at the tendon ends by the anchorages only.

Uniform Tendon: See Distributed Tendons.

Water-Reducing Admixture: An admixture that either increases the slump of freshly mixed grout without increasing the water content or maintains the slump with reduced amount of water due to factors other than air entrainment.

Wedge Plate: The hardware which holds the wedges of a multi-strand tendon and transfers the tendon force to the bearing plate.

Wedges: Pieces of tapered metal with serrations, which bite into the prestressing steel (strand) during transfer of the prestressing force.

Wedge-Set: See Seating-Loss.

Wobble Friction: The friction caused by the unintended deviation of the tendon.

Yield Strength: The stress at which a material exhibits a specific limiting deviation from the proportionality of stress to strain.

SPECIFYING POST-TENSIONING

4.1 REFERENCE STANDARDS

Specifications are an integral part of any design project because they ensure that material, construction methods, equipment, and design follow a set of standards, codes of practice, and industry guidelines. These, in turn, ensure acceptable performance of post-tensioned structures.

Post-tensioning project specifications should detail and set forth all of the relevant requirements for materials and system components, for fabrication and handling of tendons, and for installation in the structure. Standard specifications are available to address most, if not all, of these requirements. In some instances, the Design Professional may want to supplement these standard provisions with additional requirements to reflect project-specific needs. (For example, in a highly corrosive environment, the designer may specify additional corrosion protection to improve durability.)

The Design Professional should be aware of the scope of the appropriate standards and should carefully evaluate their adequacy for the intended application.

Fig. 4.1 identifies many of the standard specifications that are available and when they should be used for both bonded and unbonded post-tensioning. Some of the common standards and their respective sources are discussed in the following sections.

4.1.1 Post-Tensioning Institute

Acceptance Standards for Post-Tensioning Systems[4.1]– Provides requirements for approval and acceptance of post-tensioning systems. Includes requirements and qualification tests for prestressing materials, bearing and wedge plates, wedges, connections, and sheathing. Not intended for unbonded single-strand tendon systems.

Specification for Unbonded Single Strand Tendons[4.2]– Provides performance criteria for materials, and requirements for fabrication and installation of unbonded single-strand tendons. Main specification is intended for building construction and general post-tensioning applications. Supplement includes requirements for light commercial and residential construction.

Specification for Grouting of Post-Tensioned Structures[4.3]– Provides requirements for selection, design and installation of cementitious grouts and ducts.

Specification for Seven-Wire Steel Strand Barrier Cable Application[4.4]– Provides requirements for installation and tensioning of post-tensioned barrier cable systems. Applicable only for 7-wire prestressing strand.

Recommendations for Stay-Cable Design, Testing and Installation[4.5]– Not a specification, but includes recommendations for fatigue testing, corrosion protection, saddle testing and in-service monitoring and inspection.

Recommendations for Prestressed Rock and Soil Anchors[4.6]– Not a specification, but includes recommendations for anchor grouting, tendon bond length encapsulation, heat shrinkable sleeves, corrosion protection, anchor design, and construction.

4.1.2 American Society for Testing and Materials (ASTM)

A416/A416M Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete – This specification covers two types (low-relaxation and stress-relieved) and two grades (Grade 250 and 270) of 7-wire, uncoated steel strand for use in pretensioned and post-tensioned prestressed concrete construction.

A421/A421M Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete – This specification covers two types of uncoated stress-relieved round high-carbon steel wire commonly used in prestressed linear concrete construction: 1) Type BA wire, which is typically used for applications in which cold-end deformation is used for anchoring purposes (button anchorage); and 2) Type WA wire, which is used for applications in which the ends are anchored by wedges, and no cold-end deformation of the wire is involved (wedge anchorage). Supplement I describes low-relaxation wire and relaxation testing requirements.

A722/A722M Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete – This specification covers uncoated high-strength steel bars intended for use in post-tensioned prestressed concrete construction or in prestressed ground anchors. Bars must have a minimum ultimate tensile strength of 150 ksi (1035 MPa). Two types are covered: Type I bar has a smooth round profile and Type II bar has surface deformations (similar to common reinforcing steel bars).

A882/A882M Standard Specification for Filled Epoxy-Coated Seven-Wire Prestressing Steel Strand – This specification covers the material and application requirements for fusion-bonded epoxy coating applied to 7-wire prestressing steel strand. It is intended for use with low-relaxation strand that conforms to ASTM A416 (Grade 250 or Grade 270).

A475-98 Standard Specification for Zinc-Coated Steel Wire Strand This specification covers five grades of zinc-coated, steel-wire strand, composed of a number of round, steel wires, with four weights of zinc coatings, suitable for use as guys, messengers, span wires, and for similar purposes.

4.1.3 American Concrete Institute (ACI)

ACI 117-90 Standard Specifications for Tolerances for Concrete Construction and Materials[4.7]– This specification provides standard tolerances for concrete construction. This document is intended to be used as the reference document for establishing tolerances for concrete construction by specification writers and ACI committees writing standards.

ACI 301 Specifications for Structural Concrete for Buildings[4.8]– This reference specification covers materials and proportioning of concrete; reinforcing and prestressing steels; production, placing, finishing, and curing of concrete; and formwork design and construction. Section 9 of ACI 301 sets forth requirements for site-cast, post-tensioned structural members and includes provisions for submittals, quality assurance, materials and products and execution.

ACI 423.6-01 Specification for Unbonded Single-Strand Tendons[4.9]– Provides performance criteria for materials and requirements for fabrication and installation of unbonded single-strand tendons.

4.1.4 American Association of State Highway and Transportation Officials (AASHTO)

AASHTO LRFD Bridge Construction Specifications[4.10]– Chapter 10 of this reference specification covers furnishing, placing, and tensioning of prestressing steel for cast-in-place and precast concrete. The specification provides performance criteria for end anchorages and couplers used for bonded and unbonded systems. Also included are performance tests for members subjected to cyclic, sustained, and monotonic loadings. The specification includes various requirements for the placement of ducts, post-tensioning steel, and anchorage hardware. It covers requirements for protecting the prestressing steel from the time of manufacture to grouting. Performance criteria for ducts include those for metal ducts, polyethylene ducts, duct area, and duct fittings. General tensioning requirements of bonded multi-strand post-tensioning systems is provided.

The remainder of this chapter presents an overview of the various considerations that are essential to properly specifying post-tensioning. Note: the discussion that follows is not intended to be a substitute for any of the referenced standards. The Design Professional should carefully evaluate the adequacy and appropriateness of this information for use on any specific project. In addition, the Design Professional should always consult local codes for additional specifications that might be applicable to a specific project.

4.2 POST-TENSIONING MATERIALS

4.2.1 Prestressing Steel

There are generally four classifications for prestressing material:

1. **Strands** – The most commonly used prestressing material in North America is 7-wire carbon steel strand. Seven-wire strand has a center wire enclosed tightly by six helically wound outer wires. Strand conforming to ASTM A416 Grade 270 has a minimum ultimate strength of 270 ksi (1860 MPa). Grade 250 is also available (with an ultimate strength of 250 ksi) for use for barrier cable applications. ASTM A416 also sets forth other requirements for strands, such as strand size, tolerances, workability, bending, fatigue, stress corrosion and hydrogen embrittlement, and bond.

   For each grade, there are two types of steel: low relaxation and stress relieved (normal relaxation). Almost all of the prestressing strand supplied today is low-relaxation steel. Up until the 1970s, stress-relieved strands were common; however, they are rarely used today. Relaxation is defined as the reduction in force over time in a highly stressed tendon at a given elongation. Low-relaxation strand must conform to the Supplement I requirements of ASTM A416, which limit relaxation loss after 1000 hours of testing (see 4.2.2.2) to 2.5% at 70% of minimum ultimate tensile strength (MUTS) or 3.5% at 80% of MUTS.

   Stress relieved strand, in contrast, is not subject to any relaxation loss limit under ASTM A416. Relaxation losses for such tendons typically run at 4.5%, 8%, and 12% of the initial stress in the free tendon (i.e., strand is not associated with a concrete element) for an initial stress equal to 0.6, 0.7, and 0.8 of MUTS, respectively. Minimizing relaxation loss reduces overall prestress losses, and as a result enables the designer to take advantage of a higher final prestressing force after all other losses have occurred. Chapter 5 discusses prestress losses in more detail.

   If a structure is exposed to an aggressive environment, the designer may elect to specify a corrosion protective coating for the strand such as epoxy coating or hot dip galvanizing. As noted previously, epoxy-coated strands shall conform to ASTM A882 *Standard Specification for Epoxy-Coated Seven Wire Strand*. Galvanized strand shall conform to ASTM A475-98 *Standard Specification for Zinc-Coated Steel Wire Strand*. Neither epoxy-coated or galvanized strand are widely used in general post-tensioning applications in North America; the Design Engineer should evaluate the local availability of these materials before specifying.

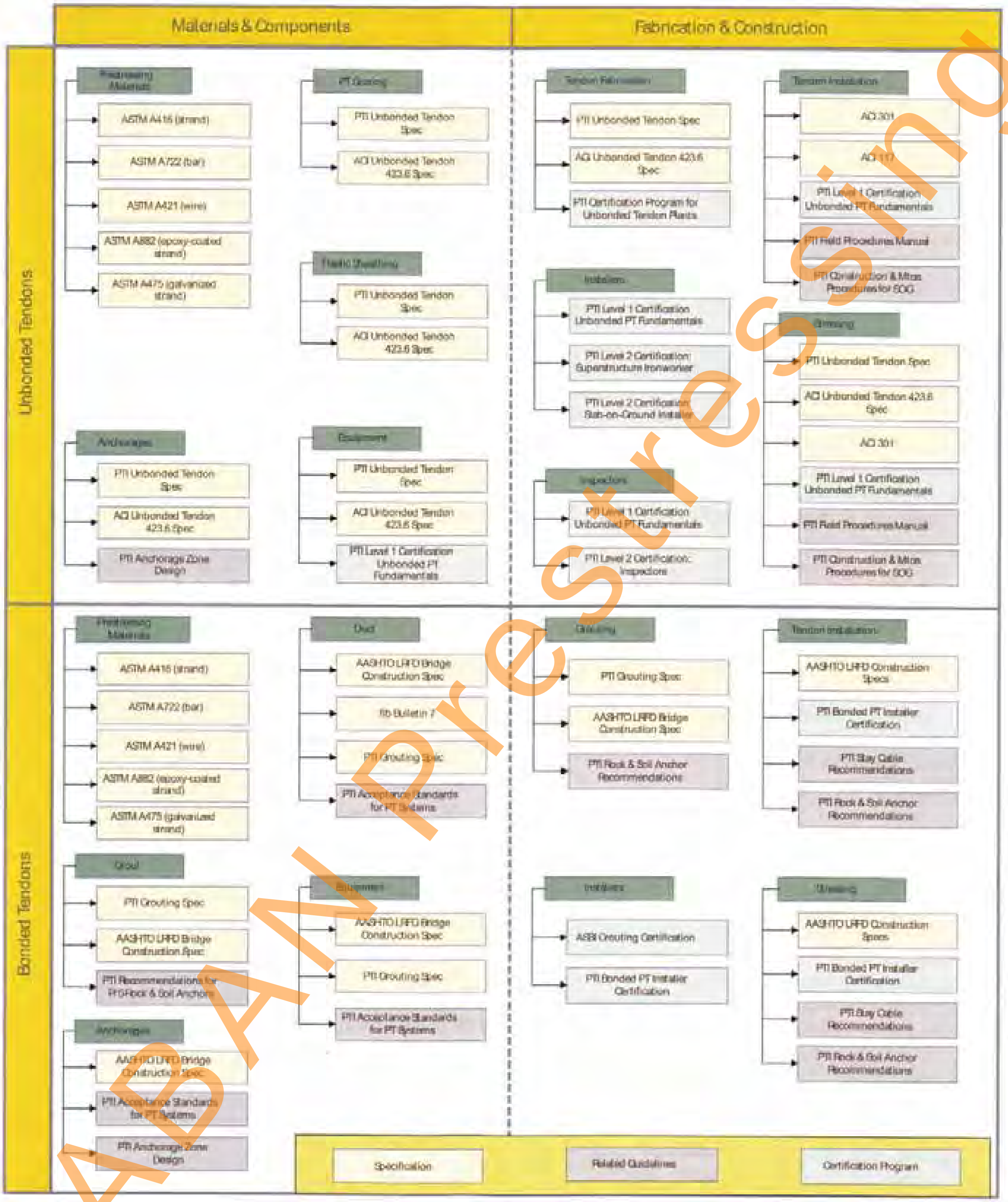

Fig. 4.1 Flowchart Illustrating Commonly-Used Standard Specifications and Recommendations for Post-Tensioning

2. **Bars** – Prestressing bars are high-strength steel bars that are cold-stressed to not more than 80% MUTS and then stress relieved to produce the desired mechanical properties. They have a minimum ultimate tensile strength of 150 ksi (1035 MPa). Prestressing bars are rolled from properly identified heat ingot- or strand-cast steel. They can be manufactured as a smooth round (Type I) or with deformations similar to a common reinforcing bar (Type II). Deformed prestressing bars have deformations that are arranged in a thread pattern permitting the use of screw-on couplers and nuts. Plain round bars must be threaded before they can be used with nut/bearing plate anchoring systems. Bars used in post-tensioned structures must meet the requirements of ASTM A722 *Specifications for Unbonded High-Strength Bar for Prestressed Concrete*, including Supplementary Requirements S1 and S2. These requirements include chemical composition, dimensions, and tensile properties.

3. **Wires** – Wires used for prestressing generally conform to ASTM A421 *Uncoated Stress-Relieved Wire for Prestressed Concrete*. Rods are used to manufacture wires by the open hearth or electric furnace process. Heat treatment is then used to stress-relieve the wires so that the desired mechanical properties are achieved. Wires are manufactured with various cross-sectional shapes and surface conditions: round vs. oval, smooth vs. indented, ribbed or crimped. ASTM A421 also has a supplement for low-relaxation wires. Wires are rarely used for post-tensioning applications in the United States; however, they are still used to a greater degree in other parts of the world.

4. **Special Prestressing Material** – Several other materials, including composites and stainless steels, have been proposed for use as a prestressing material. Many of these hold promise of greater durability, higher strength and/or lighter weight. Unfortunately, experience with many of these new materials is limited and standard specifications are not presently available.

   If a special prestressing material is to be used, the Design Engineer must carefully evaluate the material to ensure that it has been properly tested to establish that its properties, including ductility, bending, fatigue, relaxation, bond, susceptibility to mechanical damage, ability to withstand hot and cold temperatures, and resistance to chemical attack, are acceptable and suitable for the intended post-tensioning system. Special prestressing materials should be evaluated in accordance with PTI's *Acceptance Standards for Post-Tensioning Systems*.[4.1]

4.2.1.1 Properties of Prestressing Steel

4.2.1.1.1 Mechanical Properties

Certain mechanical properties of prestressing steel must be known to properly design a post-tensioned structure. ASTM specifications identify requirements for: MUTS f_{pu}; yield limit f_{py}; modulus of elasticity E_p; and the total elongation under load. In most cases, the design strength of unbonded tendons f_{ps} will be substantially less than the yield limit, f_{py}. For bonded construction, the design strength will be greater than or equal to f_{py}. Typical mechanical properties for low-relaxation strands, wires, and bars are shown below in Table 4.1.

Table 4.1 - Typical Mechanical Properties of Standard Prestressing Steels

Prestressing Steel	f_{pu} ksi (MPa)	f_{py} ksi (MPa)	E_p ksi (MPa)	% Elongation [Gauge Length]	Relaxation
Low-Relaxation 7-Wire Strand Grade 270 per ASTM A416/416M	270 (1860)	0.90 f_{pu}	28,500 (196,500)	3.5 [24 in. (610mm)]	[2.5% @ 70% MUTS] or [3.5% @ 80% MUTS]
Stress-Relieved Wire per ASTM A421/421M	235-250 (1620–1725)	0.85 f_{pu}	29,000 (200,000)	4 [10 in. (250mm)]	————
Low-Relaxation Wire per ASTM A421/421M	235-250 (1620–1725)	0.90f_{pu}	29,000 (200,000)	4 [10 in. (250mm)]	[2.5% @ 70% MUTS] or [3.5% @ 80% MUTS]
Prestressed Bars Grade 150 per ASTM A722	150 (1035)	Type I: 0.85 f_{pu} Type II: 0.8 f_{pu}	29,700 (205,000)	4 [20 bar dia.] 7 [10 bar dia.]	————

These typical values are often used for design purposes; however, the actual material properties for the prestressing steel supplied to the project may vary and may exceed specification minimums. Knowing the actual properties can be important during inspection and future rehabilitation. For example, when evaluating out-of-tolerance elongations during stressing, the Design Engineer should compare the actual values (e.g. the modulus of elasticity) as given on the supplier-provided mill certificates and the value assumed in design. In many instances, the difference may explain the observed elongation.

4.2.1.1.2 Ductility

Ductility is an essential property of a prestressing material. Standard specifications prescribe ductility requirements, which are usually expressed as a minimum percent elongation in the gauge length under total load. For ASTM A416 prestressing strand, the minimum elongation is specified as 3.5% using a gauge length of not less than 24 in. (610 mm). For ASTM A722 prestressing bars, the minimum percent elongation after rupture is 4% and 7% for Type I and Type II prestressing bars, respectively.

4.2.1.1.3 Static and Fatigue Testing

Tendons in prestressed concrete structures and ground anchors normally do not experience stress cycling significant enough to cause fatigue problems. For those applications where fatigue is a concern, such as post-tensioned bridges and cable-stayed bridges,[4.5] fatigue resistance can be increased by proper material selection and anchorage design. Tendon fatigue will depend on the type of structure and whether the tendon is bonded or unbonded. The strand-wedge connection is the most sensitive part of a tendon in regards to fatigue resistance.

Where fatigue is a possible concern, the Design Engineer should confirm that the intended post-tensioning system has been dynamically tested and qualified in accordance with the PTI *Acceptance Standards for Post-Tensioning Systems*[4.1] for bonded tendons and in accordance with Ref. 4.1 for unbonded tendons. For unbonded systems on bridges, AASHTO[4.10] requires that a representative anchorage and coupler specimen, as well as tendon, be dynamically tested without failure, 500,000 cycles from 60 to 66 percent of MUTS, and 50 cycles from 40 to 80 percent MUTS.

4.2.1.2 Packaging of Strand

ASTM A416/M416 requires that strands be well protected during shipment against mechanical injury, which includes damage from corrosion, stress corrosion, or hydrogen embitterment through contact with deleterious chemicals. The ASTM Specification leaves the details of the protection method to the Specifier.

Some strand manufacturers, in cooperation with the California Department of Transportation, have developed effective corrosion protective packing known as CAL-WRAP, which meets the corrosion protection requirements of Section 50 of California Department of Transportation's *Standard Specifications.*[4.11] For building projects, standard packaging provided by the strand manufacturer is normally adequate.[4.1]

Tendons must be packaged in a manner that prevents physical damage to the strand during transportation and protects the material from deleterious corrosion during transit and storage.

4.2.2 Anchorages and Bearing Plates

4.2.2.1 Anchorages

As discussed in Chapter 3, anchorage devices transfer the tendon force to the surrounding concrete. In many instances, particularly on multi-strand anchors, these transfer forces can be very high. The highest force in a tendon usually occurs at the time of stressing when the tendon is typically stressed to 80% MUTS. Thereafter, the force in the tendon decreases as a result of seating losses, creep, shrinkage, and relaxation.

Anchorages are expected to develop at least 95% of the actual ultimate strength of the prestressing steel. This requirement provides a substantial safety margin between the ultimate tendon capacity and the tendon design strength.

For unbonded tendons, the post-tensioning supplier typically attaches fixed end anchorages at the plant. The supplier must also provide sizes and quantities of stressing end anchorages on the *Loose Hardware Sheet* (see Section 4.3.1.2.2).

Anchorages for bonded tendons must be installed by the Contractor in accordance with the design documents and the requirements stipulated by the anchorage device supplier. Stressing-end tendon anchorages must be placed perpendicular to the face of the form used.

Due to the dynamic interrelationship of the component parts during the transferring of force to wedge-type anchorages, the casting and the wedge should always be considered as one design unit. Wedges are typically designed to preclude premature failure of prestressing steel due to notch or pinching effects under static and fatigue test loading. Component parts from different manufacturers must not be used without substantiating test data.

4.2.2.2 Bearing Plates

The transfer of the tendon force from the anchorage device to the concrete is usually accomplished by the use of bearing plates. There are two classes of bearing plates: basic and special.

Basic bearing plates are commonly used on unbonded, single tendons. Basic bearing plates are designed to meet specific design criteria based on the distribution area (see Section 3.1 of Ref. 4.1) and do not require testing.

Special bearing plates are typically used on bonded, multistrand applications. They are often proprietary, and tend to be more massive, complex and costly than basic bearing plates. Most suppliers have developed special bearing plates. They have special shapes, frequently have multiple bearing surfaces, and often are ductile iron castings. Such bearing plates nominally produce very high local bearing stresses on the concrete and, therefore, require spirals or equivalent confinement reinforcement in the local zone.

Design of local zone (confinement) reinforcing is the responsibility of the post-tensioning supplier and will not typically be shown on the contract drawings. It is not uncommon during stressing of a large multi-strand tendon for bearing stresses to exceed 10,000 psi (65 MPa). Because the behavior of such plates cannot be readily established using analytical procedures, their adequacy must be established by tests. Refs. 4.1 and 4.10 set forth basic bearing plate design criteria and special testing requirements. For large multi-strand tendons where special bearing plates are required, most specifiers require that the bearing plates conform to one of these standards.

The PTI publication *Anchorage Zone Design*[4.12] provides detailed guidance for the design of bearing plates and anchorages. Additional information and requirements can be found in the *AASHTO LRFD Bridge Design Specifications.*[4.13]

4.3 SPECIFYING UNBONDED SINGLE-STRAND TENDONS

4.3.1 General

PTI's *Specification for Unbonded Single Strand Tendons*[4.2] and ACI's 423.6/423.6R-01 *Specification for Unbonded Single-Strand Tendons and Commentary*[4.9] set forth general materials, system components, fabrication and installation requirements for most unbonded single-strand tendon applications. These standards cover both aggressive and non-aggressive environments. The PTI specification is applicable to all unbonded tendon applications, while the ACI specification is not intended for residential or light commercial applications. It is usually sufficient to reference one of these standards in the contract specifications. However, the Design Professional should be fully aware of the scope and requirements of any referenced standard and must carefully evaluate its adequacy for a given project. Key elements of these specifications are discussed below.

4.3.2 Aggressive Environment

An aggressive environment is defined as one in which structures are subjected to direct or indirect applications of deicing chemicals, seawater, brackish water, or spray from these sources; structures in the immediate vicinity of seacoasts exposed to salt-laden air; and structures where anchorage areas are in direct contact with soil. Stressing pockets that are not maintained in a normally dry condition after construction should also be considered to be exposed to an aggressive environment. With the exception of those located near the coast, nearly all enclosed buildings (office buildings, apartment buildings, warehouses, and manufacturing facilities) are considered to be non-aggressive environments. The Design Engineer should determine if the structure or any part of it is to be exposed to an aggressive environment. Consideration should be given to such areas as location of stressing-end and intermediate anchors, construction joints, planters, balconies and swimming pools. The determination of whether the structure is in an aggressive environment will primarily impact tendon finishing and the need for encapsulated tendons.

In aggressive environments, anchorages must be protected against corrosion either by plastic encapsulation or epoxy coating. Encapsulation systems provide a watertight connection of the sheathing to the anchorage and a watertight enclosure of the wedge cavity, and prestressing steel is required at all anchorages including at the fixed-end, intermediate anchorage, and stressing-end to ensure proper corrosion protection of the anchor, wedge, and prestressing steel. Anchorages must be designed to attain watertight encapsulation of prestressing steel; on stressing-end anchorages the tendon tail and gripping part of the anchorage must be capped at the wedge cavity to completely seal the area against moisture. Standard specifications[4.2,4.9] require that encapsulated anchorages be hydrostatically tested to confirm that all connections remain watertight.

The use of epoxy coatings is also acceptable, however special inspection is required to identify damage that can occur to the epoxy system during transportation, handling, and installation. Damaging the epoxy coating would breach the encapsulation and make the system unacceptable. The use of bare metallic anchorages produced from a material that is subject to corrosion is unacceptable.

4.3.3 Fabrication

Fabrication of unbonded tendons is the process of applying a protective coating (grease or wax) and extruding a plastic sheathing on the strand, cutting a tendon to a specified length, marking it for a specific position in the structure, applying the fixed-end anchor, positioning the intermediate anchors (if required) and coiling and securing the tendons into bundles for shipment to the jobsite. Quality of fabrication is essential to ensure performance and to minimize problems during installation.

Project specifications should, whether by reference to the aforementioned standards or by special provision, spell out appropriate fabrication requirements such as required certifications, material identification, handling and storage, and testing. It is highly recommended (and required by the

PTI and ACI specifications) that unbonded single-strand tendons be fabricated in a plant that has been certified to meet the requirements of PTI's Program for Certification of Plants Producing Unbonded Single-Strand Tendons, and its accompanying Manual,[4.14] or equivalent. The PTI program involves detailed plant inspections and review of the supplier's records, test data, fabrication procedures, materials, equipment and quality control program. It sets forth stringent quality control procedures for fabrication and provides an independent certification that the plant and its personnel are capable of producing unbonded single-strand tendons in conformance with the PTI *Specification for Unbonded Single Strand Tendons*.[4.2]

4.3.3.1 Handling and Storage

The post-tensioning supplier is responsible for the fabrication and packaging of unbonded tendons. Individual tendons must be secured in bundles using a tying product that does not damage the sheath. Damage is defined as a rupture or breach in the sheathing, which would allow the possible intrusion of moisture into the tendon. The tendon sheath must be protected from damage by banding materials. Padding material shall be used between any metal banding and the tendon.

4.3.3.2 Identification

For unbonded tendons, *Tendon Fabrication Sheets* are produced by the post-tensioning supplier prior to shipping and are derived from the post-tensioning installation drawings. These sheets show relevant project information such as the intended installation location for the material being supplied (e.g., floor or pour number). The remainder of the *Tendon Fabrication Sheet* lists tendon quantities, tendon lengths, anchorage configurations, identification markings (e.g., color codes), and loose hardware quantities. Anchorage configuration of each tendon can be determined from the symbols on the *Tendon Fabrication Sheet*. The symbols are as follows:

———> a tendon with a fixed-end anchorage at one end and a stressing anchorage at the other end.

<———> a tendon with a stressing anchorage at each end.

——>——> a tendon with an intermediate and end stressing anchorages.

In addition to *Tendon Fabrication Sheet(s)* that are provided for unbonded tendons, there may be other shipping documents such as a *Loose Hardware Sheet* that shows quantities and tractability information for loose hardware shipped (e.g., stressing end anchorages and wedges, pocket formers, etc.). These are important documents that are to be maintained as part of the permanent project record.

Test data for anchor/wedge systems are typically submitted in a material data submittal package prior to shipping any materials to a project. *Material Certification Sheets* list the actual physical properties of the prestressing steel, such as nominal diameter, cross-sectional area, grade, type of prestressing steel (i.e., low vs. normal relaxation), and modulus of elasticity. All PTI Certified Plants must keep *Material Certification Sheets* on file and they must be sent upon request if the need should arise.

4.3.3.3 P/T Coating

An effective P/T coating must be a compound with appropriate moisture-displacing and corrosion-inhibiting properties. The PTI and ACI unbonded tendon specifications set forth performance criteria for P/T coating that include key properties such as: corrosion resistance, water content, compatibility with sheathing, separation and flow characteristics (Table 1, Ref. 4.2).

These criteria are considered to be baseline tests to ensure that minimum corrosion protection properties are provided. New developments of coating materials may not meet some of these test requirements, and in such cases, other and more comprehensive tests may be necessary to ascertain their adequacy.

4.3.4 Construction

4.3.4.1 Tendon Installation

Tendons must be placed as shown on the construction drawings. It should be realized that the high and low points along the tendon profile are the most critical locations. A smooth curve must be maintained between these points. The contract documents must also specify the maximum distance between tendon support per the following section.

Table 4.2 - Typical Profile Tolerances[4.2,4.15]

Member Depth	Deviation from Tendon Design Profile
Elevated concrete with depths less than or equal to 8 in. (200 mm)	¼ in. (6 mm)
Elevated concrete with depths greater than 8 in. (200 mm) and less than 24 in. (610 mm)	⅜ in. (9.5 mm)
Elevated concrete with depths greater than 24 in. (610 mm)	½ in. (13 mm)
Slabs-on-ground with ribbed slabs with depths greater than 4.5 in.(114mm)	middle third of slab thickness
Slabs-on-ground with ribbed slabs with depths less than 4.5 in.(114mm)	middle half of slab thickness
Slabs-on-ground with uniform thickness	⅓ of slab thickness not exceeding 1 in. (25 mm)

4.3.4.2 Support System

Prestressing tendons must be supported in a manner that does not cause damage to the sheathing. Supports should be spaced at intervals not exceeding 4 ft. (1220 mm).

4.3.4.3 Tolerances

Placing tolerances are specified in Refs. 4.2 and 4.15, and ACI 117.[4.7] It is recommended that the most restrictive of these be specified. If warranted by the design or project conditions, special tolerances should be specified in the project specifications. Typical vertical tolerances of tendon placement are shown in Table 4.2.

These vertical tolerances are primarily for beams and slabs; tolerances for other types of members should be specified in the contract documents. Placing tolerances should be considered when establishing tendon cover requirements. This is particularly important in applications exposed to deicing chemicals or saltwater environments where use of additional cover is recommended.

The horizontal locations of tendons in slabs are not generally critical; however, excessive wobble (i.e., unintended curvature) must be avoided. Horizontal deviations of up to 12 in. (305 mm) are allowed to avoid openings, ducts, chases and inserts. Such deviations should have a minimum radius of curvature of 480 strand diameter [20 ft (6.5 m) for ½ in. diameter strands]. This minimum should be adjusted proportionally for other strand diameters.

Care should be taken in placing stressing end anchorages as close as possible to the location shown on the design drawings. Some installers will apply a small amount of P/T coating to the tip of the pocket former that fits into the anchorage cavity. This helps to form a seal that keeps cement paste from working its way into the anchorage cavity and is an acceptable practice. The specifications should clearly indicate that any part of the pocket former that comes into contact with concrete be free of P/T coating.

4.3.4.4 Sheathing Inspection, Damage, and Repair Procedure

After the tendons have been installed in the forms and prior to concrete placement, the sheathing must be inspected for possible damage. Rips and tears up to 3 in. (75 mm) long are allowed as long as they do not cover an excessive amount of the tendon length. If the damage covers an excessive amount of tendon length, the P/T coating must be restored and the sheathing repaired. Project specifications should include a repair guide and must require that sheathing repairs be watertight and without air voids.

Sheathing damage may be repaired with an approved self adhesive, moisture resistant tape. For more information about evaluation of commonly used tape repair materials, see PTI's *Field Procedures Manual for Unbonded Single Strand Tendons*.[4.16]

4.3.4.5 Tendon Finishing

Tendon tails can be cut in a number of ways, including oxyacetylene cutting, abrasive wheel, hydraulic shears, or gas plasma cutting. Contract drawings can specify any additional special requirements pertaining to the method of cutting the strand tails. If oxyacetylene cutting is used, flames must be directed away from the wedges. When using hydraulic shears or gas plasma cutting, the specifications should make reference to the operational instructions of the equipment supplier. Tendon tails must not be cut until the Design Engineer has approved the measured elongations. Once approved, cutting should be carried out as soon as possible. It is important that the strand length protruding beyond the wedges after cutting of the tendon tail be between 0.5 in. (15 mm) and 0.75 in. (20 mm). If too much strand is left protruding, proper concrete coverage will not be maintained when the recess is grouted.

For slabs-on-ground, it is typical to see a contractor use an abrasive wheel as opposed to a cutting torch. This is because most slabs-on-ground are constructed on residential projects that typically contain large amounts of combustible materials. This makes the use of abrasive wheels safer. If abrasive wheels are used, the specifications must include a provision requiring the Contractor to install a 1 in. (25.4 mm) plastic cap over the strand tail prior to grouting. If an encapsulated system is used, the length of the strand that can be left protruding beyond the wedges must be specified. Too much strand protruding beyond the wedges may prevent the watertight cap from being properly installed. A watertight cap filled with P/T coating should be installed in accordance with the supplier's instructions.

4.3.4.6 Concrete Placement

During concrete placement, the contractor must ensure that the position of post-tensioning tendons and non-prestressed reinforcement remain unchanged. All pump lines, chutes, and other concrete placing equipment must be supported above the tendons. If tendons are moved out of their designated positions during concreting, they must be adjusted back to their correct position.

4.4 SPECIFYING BONDED TENDONS

4.4.1 General

As described in Chapter 2, bonded post-tensioning is characterized by the use of multiple strands that are typically placed (either by pushing or pulling) inside a plastic or metallic duct embedded inside the concrete member. The specification of bonded post-tensioning differs from unbonded single-strand post-tensioning in several ways, including: the use of multi-strand anchors, ducts, grouting, stressing and inspection.

4.4.2 Multi-Strand Anchorages

Most bonded post-tensioning uses multi-strand anchorages where multiple strands have one common anchor. These anchors are generally larger and more complex than single-strand anchors and require specific testing and qualification.

Because anchorage zones are very critical regions in post-tensioned structures, the Contractor must place all anchorage devices exactly as shown on the contract plans. The Design Engineer must approve any change in anchorage zone details, including hardware, reinforcement, and concrete block-out and consolidation.

Anchor and wedge cavities should be clean and free of any cement paste. Anchor heads should be protected from dirt and debris during handling and installation.

4.4.3 Storage and Handling

The Design Engineer must reject strands that show excessive rust. The use of corrosion-inhibiting compounds should be considered a secondary means of corrosion protection, except in areas with high humidity or marine environments. In such areas, the Project Engineer will typically require that the prestressing steel be packaged at the source using corrosion inhibitor, or by wrapping the strands with paper impregnated with a corrosion inhibitor. Bonded strands cannot be coated with any material that will affect the bonding of the steel to the grout material.

Bars are not easily damaged by corrosion because of their relatively low strength, and relatively small ratio of surface area to cross-sectional area. Therefore, they do not require any special corrosion protection during shipping and handling.

Care must be taken in selecting the proper platform for storage. Prestressing steel must be stored on some sort of dunnage (i.e., timber or pallets) that keeps it from being contaminated by dirt and protects it from water and ground.

4.4.4 Identification

Bonded prestressing steel, and associated anchor assemblies and wedges, must all have coding information that traces the material to a specific lot. It is the responsibility of the post-tensioning supplier to record this information for materials shipped to a project. It is the responsibility of the Contractor to maintain traceability of the materials delivered by installing them in their proper locations on site. Tags on the prestressing steel must not be removed until the dispenser pack is ready to be installed.

Test data for anchor/wedge systems are typically submitted in a material data submittal package prior to shipping any materials to a project. *Material Certification Sheets* list the actual physical properties of the prestressing steel, such as nominal diameter, cross-sectional area, grade, type of prestressing steel (i.e., low vs. normal relaxation), and modulus of elasticity. All PTI Certified Plants must keep *Material Certification Sheets* on file and they must be sent upon request should the need arise.

4.4.5 Ducts

Ducts and duct couplers must be tight enough to prevent the entrance of cement paste, and strong enough to retain their shape and resist damage during handling and concrete placement. Ducts can be made of galvanized strip steel, steel pipe, high-density polyethylene (HDPE), or polypropylene (PP). The duct material should not adversely react with the concrete, prestressing element or grout.

Plastic ducts may add to the system's durability by providing a non-corrosive impermeable layer between the concrete and the grout. Most flat duct systems are of corrugated plastic. When using corrugated plastic ducts, the guidelines provided by the post-tensioning supplier must be closely followed for assembly of the duct system, especially when air-tightness is required.

Galvanized steel ducts are not impermeable and may corrode in aggressive environments. With any type of duct, care should be taken that the end anchorage does not become a route for ingress of aggressive agents such as moisture, chlorides and sulfates. For corrugated metal ducts, care must be taken during installation to ensure that the duct is not bent too sharply, causing the folded seams to open. Smooth steel ducts are used where ducts are subjected to very high loadings during concrete placement (e.g., concrete dropped from a high elevation, the use of large vibrators, etc.).

Ducts, whether made of plastic or steel, can be either smooth or corrugated. Corrugated ducts are embedded inside a concrete member and must provide sufficient bond transfer to the surrounding concrete. Smooth duct is typically used for external tendons.

4.4.5.1 Duct Wall Thickness

The required wall thickness of a duct varies with diameter, depth and spacing of corrugations and hardness. The thickness must be adequate to resist grout pressure, denting during handling and installation, and damage from concentrated forces at support points. Very large ducts may require increased wall thickness.

4.4.5.2 Diameter of the Duct

The internal diameter of the duct depends on the nominal cross-sectional area of the prestressing steel. Table 4.3 gives the minimum nominal internal duct area for various types of tendons. If for any reason the duct/steel area ratio falls outside the given limits, it shall be proven by tests that proper grouting, corrosion protection and bond transfer are possible. Table 4.4 shows the typical duct diameter for different numbers of strand per duct.

Table 4.3 - Minimum Nominal Internal Area of the Duct

Tendon Type	Nominal Internal Area of the Duct
Strand – push-through placing method	2.25 times prestressing steel area
Strand – pull-through placing method	2.5 times prestressing steel area
Single prestressing bar	0.25 in. (6mm) larger than the outside diameter of bar
Short tendons [less than or equal to 100 ft. (30 m)]	2.0 times prestressing steel area

Note: In the case of space limitations, the minimum duct area may be reduced to 2.0 times prestressing steel area.

Table 4.4 - Typical Tendon Strand Duct Sizes[4.1]

Number of Strands in Each Duct		Duct Internal Diameter
0.5 in. (12.7 mm) Strand	**0.6 in. (15.24 mm) Strand**	
5	4	2 in. (50 mm)
12	9	2 ½ to 3 in. (65 to 75 mm)
19	12	3 to 3 ½ in. (75 to 90 mm)
27	19	3 ½ to 4 in. (90 to 100 mm)
31	22	4 to 4 ½ in. (100 to 115 mm)
34 – 38	24 – 27	4 ½ to 5 in. (115 to 130 mm)

4.4.5.3 Protection of the Ducts

Ducts shall be protected against crushing, excessive bending, dirt contamination and corrosive elements during transport, storage and handling. In normal circumstances, it is not necessary to protect ducts against corrosion after they are placed within the concrete structure. In special cases, however, such as when empty ducts are placed a considerable time before the insertion of the prestressing elements, protection of the ducts is required to avoid corrosion of the duct's interior surfaces, which may increase friction during stressing operations.

4.4.5.4 Repair of Ducts

In case of damage, ducts shall be sealed with tape (or other suitable material), or by splicing a duct coupler over the damaged section to form a seal that prevents cement paste from entering the ductwork during concrete placement and to prevent leakage during grouting operations.

Table 4.5 - Recommended Maximum Spacing of Duct Supports

Type of Duct	Maximum Support Spacing
Galvanized metal round duct	4 ft (1.22 m)
HDPE round duct (no strands installed in duct prior to placing concrete)	2 ft (0.61 m)
HDPE flat duct (strand installed in duct)	2 ft (0.61 m)
HDPE flat duct (no strands installed)	1 ft (0.305 m)

If damage is localized (e.g., dents at ends), the damaged portions can be cut off. Ducts that are damaged along their entire length, or those that have holes due to pronounced rusting, must be discarded.

4.4.5.5 Duct Support Spacing

Duct supports must be constructed per the approved contract documents and should be installed at the locations and heights shown on the post-tensioning installation drawings. Unless otherwise specified, the duct support spacing shall not exceed the values in Table 4.5.

Care should be taken to avoid a potential problem with flat ducts collapsing if there is no strand or other device installed inside the duct prior to concreting.

For curved structures, large radial tendon forces tend to deform or cut through the duct wall, and hence, must be limited. Bearing forces of 250 lb/in. (44 N/mm) on HDPE ducts are considered acceptable.[4.1] Testing might be the best way to accurately predict the performance of a particular plastic duct. If specified, such tests should also simulate the abrasion caused by the prestressing steel during stressing.

Strand installation inside plastic ducts must be carefully planned to avoid duct damage. For instance, pulling large and long cables into the duct is likely to cause abrasion damage. Equivalently, pushing individual strands at a high speed into the duct requires special bullets for capping the strand ends to prevent rupture.

4.4.5.6 Ducts for External Tendons

Ducts for external tendons, including their splices, must be vapor tight, seamless or welded, and capable of resisting at least 150 psi (1 MPa) grout pressure. The duct material must be resistant to the particular project exposure and environment.

Table 4.6 - Recommended Tolerances for Ducts

Position	Location	Tolerance
Vertical	Slabs; longitudinal draped superstructure tendons over supports; superstructure tendons in top or bottom of member	¼ in. (6 mm)
	Draped superstructure ducts in middle half of web depth; vertical ducts in webs	1 in. (25 mm)
	Horizontal ducts in foundations; vertical tendons in pier shafts	½ in. (13 mm)
Lateral	Ducts in slabs or foundations	½ in. (13 mm)
	All other conditions	¼ in. (6 mm)

It should be noted that spiral wound ducts do not satisfy these requirements, and should be avoided for use in external tendons.

4.4.5.7 Duct Couplers and Mandrels

Couplers must be staggered in adjacent ducts and must not be located in curved zones. Joints at coupler locations must be sealed with duct tape or other approved material.

Mandrels are inflatable tubes that are placed inside the duct assemblies to prevent collapse during concrete placement. This may be required in very deep concrete members where the Design Engineer or post-tensioning supplier has determined that the ducts may not withstand the weight of the concrete being dropped.

4.4.5.8 Duct Tolerances

Duct tolerances should be per Table 4.6.

4.4.6 Grouting

Proper grouting is essential to ensure that the tendons, the duct and the surrounding concrete are integrally bonded and perform as a unit structurally. Grouting also provides corrosion protection to the tendons and is a critical part of ensuring the durability of a post-tensioned structure. Experience has shown that proper specification of materials and of workmanship are both critical in order to achieve good grouting.

Ref. 4.3, PTI's *Specifications for Grouting of Post-Tensioned Structures* provides guidelines for the grouting of post-tensioned elements commonly used in all types of structures. This guide specification provides minimum requirements for the selection, design and installation of cementitious grouts and ducts for post-tensioned systems used in concrete construction.

For specialized grouting applications, such as main cables in cable-stayed bridges or prestressed ground anchors, Ref. 4.3 may be used for the general properties and quality assurance aspects of the grout materials. In addition to these general design and construction considerations, there are specific aspects and specialized procedures that must be considered in these types of applications. For further information on ground anchor grouting, see Ref. 4.6.

Key specification factors for successful grouting include: materials, design, equipment, personnel, construction, and quality assurance/quality control.

4.4.6.1 Grouting Materials

Over the last 40 years, most grouts used in post-tensioned construction have been a simple mixture of portland cement and water (sometimes referred to as "neat cement" grout). Water/cement ratios have typically ranged from 0.47 to 0.53. In some instances, an expansive and/or non-bleeding admixture has also been specified.

For the most part, post-tensioning grouts have performed satisfactorily. However, on some projects, especially in aggressive environments, such as marine and northern climates where chlorides or sulphates were encountered, these grouts did not perform as expected. In general, the observed problems have been attributed to a combination of poor materials (e.g. high-bleed, high-permeability grout, cracked ducts, etc.) and improper workmanship.

Recent advances in material technology have made high performance grouts a reality. With prudent use of additives/admixtures and proper grout mix design, it is feasible to produce post-tensioning grouts that are extremely workable, corrosion resistant and that have essentially no bleed. However, to achieve these high performance grouts in practice requires a comprehensive grouting specification and careful selection of materials. Detailed specifications regarding grout design can be found in Ref. 4.3.

Key factors to consider in selecting grout materials include the following:

- Cement hydration rate, as it affects working time and set time
- Grout fluidity (both initial fluidity and changes in fluidity) as a function of time and temperature
- Volume control
- Permeability
- Strength
- Bleeding characteristics
- Level of corrosion protection required for prestressing steel

The ingredients of post-tensioning grouts can include: cement, mineral additives, admixtures, aggregate and water.

4.4.6.1.1 Grout Mix Design

In the PTI Guide Specification,[4.3] grouts are divided into four classes: Class A, B, C, and D. The class of grout used for a specific application depends on the severity of exposure. Class A grout is used in non-aggressive exposure applications. Class B grout is for aggressive exposures. Class C grout is prepackaged grout material suitable for aggressive exposures but may also be used in non-aggressive environments. Class D grouts are specialized grouts for critical applications where the properties and performance characteristics of the grout material must be carefully controlled.

Class A, B, or C grouts typically consist of Type I or II portland cement, potable water, mineral additives and other specified admixtures. Deliberate addition of chlorides to the grout should not be permitted. Because of their specialized nature, Class D grouts may include these and other ingredients.

When proportioning materials for grout, the following parameters should be considered: setting time, grout strength, pumpability, fluidity and bleed.[4.3]

4.4.6.1.2 Mineral Additives

Mineral additives can be used to modify the properties and behavior of grouts. These include fly ash, ground granulated blast furnace slag, and silica fume, all of which are widely used in portland cement concretes. The potential benefits of using these mineral additives include:

- Lower permeability
- Control of bleeding
- Higher long-term strength
- Lower grout temperature during curing

These commercially available materials consist of fine powders and may contain hazardous ingredients, necessitating careful handling at the job site. In addition, the effect that these additives have on other grout properties must be carefully evaluated. For example, with certain cements, fly ash additions will extend the setting time of the grout, and thus extend the time during which settlement shrinkage will occur. Certain detrimental fly ash-cement reactions can also occur.[4.17]

Other detailed information regarding mineral additives can be found in Ref. 4.3.

4.4.6.1.3 Admixtures

Admixtures can also be used with grout and provide several advantages, namely: set control, water reduction, air-entrainment, bleed control, volume control, corrosion inhibition, and pumpability. Admixtures are typically liquids (less frequently, dry powders) that are added to concretes and mortars in small amounts to influence fresh and hardened properties and characteristics of the material.

4.4.6.1.3.1 Water-Reducing Admixtures

Water-reducing admixtures can serve two functions when used with post-tensioning grouts. They can increase the fluidity of the material in the fresh state while maintaining a given water content. Alternatively, they can be used to maintain a desired level of fluidity at significantly lower water contents.

4.4.6.1.3.2 Air-Entraining Admixtures

Air-entraining admixtures are used in those instances where freeze/thaw damage is a possibility. Historically, air-entraining admixtures have not been specified or used in grouts for bonded, post-tensioned construction. There is little evidence that freeze/thaw damage has been a problem for post-tensioning grouts even though many structures are located in freeze/thaw environments. Air-entraining agents can increase cohesiveness and lower bleeding rates of grout. However, when used, fine aggregates must also typically be specified in order to develop a stable air void system.

4.4.6.1.3.2 Anti-Bleeding Admixtures

Anti-bleeding admixtures are an important consideration for grouts for bonded, post-tensioned construction, particularly in those instances where strands are used in high vertical lifts. Bleed is defined as the autogenous flow of mixing water within, or its emergence from, newly placed grout, and is caused by the settlement of the solid materials within the mass and filtering action of strands, wires and bars. Bleeding of grout in bonded ducts is largely influenced by the interstices formed in the strand by the space between the king wire and the perimeter wires which act as a capillary tube. Gravity causes a pressure head to form due to the difference in elevation between the crown and the trough of the duct profile or the height difference in the lift of vertical ducts. Temperature, rheology and fluidity all are important factors in bleeding of the combined system. The following discussion from Ref. 4.18 about bleed in grouts explains the significance of bleed in more detail:

> "Bleed is the process of sedimentation of a solution – in this case water and solid particles (cement, admixtures, and fillers if used). Bleed water tends to migrate to higher points or in tall tendons, may congregate at intermediate lenses. Separated water can become trapped in the tendon. For strand type tendons there is also an additional mechanism of water transport where the water is filtered through the interstices between the core wire and the outer six wires causing a water separation mechanism. The denser grout drives the water up the interstices causing exaggerated bleed.
>
> Bleed can be a long-term concern in post-tensioned grout applications. Bleed water trapped in the duct cannot be readily evaporated. It is usually reabsorbed by the hardened grout and results in a void. Whether there

is a void or entrapped bleed water, potential corrosion sites exist particularly if corrosion-causing contaminants are available. Bleed pockets, also known as bleed lenses, typically are formed at high points in the duct due to the bleed water traveling upwards. This high point may be at the top of a vertical duct, at an anchorage region, or at the top of an intermediate draped high point. In very tall applications such as cable-stays or piers, the bleed water may not reach the very highest point, but may instead congregate at intermediate points forming additional bleed lenses. Higher vertical rises result in higher pressure heads and will require stronger anti-bleed properties. The amount of bleed may be considerable in a grout that does not contain bleed-reducing agents."

To assure minimal bleed, in most cases a thixotropic grout is needed. A thixotropic grout is simply one that exhibits gel-like properties at rest, but that becomes fluid when agitated (by mixing and pumping). Webster's defines thixotropy as "the property of various gels of becoming fluid when disturbed."[4.19]

A thixotropic anti-bleed grout retains water rather than allowing segregation. A gelling agent (also known as thixotropic agent or stabilizer) is added as an admixture to the grout and imparts a thixotropic nature to the grout. Anti-bleed admixtures tend to increase set time, but when used at the proper dosage, set times remain under 12 hours.

4.4.6.1.3.4 Expansion-Causing Admixtures

Expansion-causing admixtures are sometimes used in grouts because it is thought that the resulting grout expansion will help fill any void that is present. However, research has shown that certain gas-forming admixtures will provide an interconnected air void system leading to increased chloride permeability of the grout, whose major function is to prevent the prestressing steel from chloride and carbonation induced corrosion. In addition, there is little evidence that the use of gas-forming expansion agents will improve volume stability. In fact, one study[4.20] suggests that there is no material benefit. Until such time that these types of admixtures have been shown to be beneficial, their use in post-tensioning grouts is not recommended.

4.4.6.1.3.5 Corrosion-Inhibiting Admixtures

Some admixtures, such as calcium nitrite, have been shown to be beneficial in mitigating steel corrosion in concrete structures,[4.21] but limited research is available about their benefit in grouts for bonded post-tensioning.[4.20] The use of this type of admixture in grouts for bonded post-tensioned construction may be considered in aggressive environments subject to deicing chemicals, sea-spray exposure and seacoasts.

4.4.6.1.4 Aggregates

Historically, aggregates have not been widely used in post-tensioning grouts. Aggregate, if used, must have a very fine gradation (Ref. 4.3 specifies a maximum size of 1 mm) to facilitate movement of the grout through the duct and to provide total encapsulation of the prestressing elements.

Aggregates are used in grout material to reduce permeability, improve volume stability and reduce drying shrinkage. Although aggregates can make a positive contribution to the overall performance of these grouts, their use should be considered very carefully.

4.4.6.2 Grouting Design Considerations

The Design Engineer should be aware that certain design details might impact subsequent grouting operations and the future durability of the structure. Duct inlets and outlets should be carefully located to ensure proper grouting and to minimize possible water and chloride ingress into the duct.

Project specifications should determine the class of grout to be used on a given project. Grout mix designs are to be submitted, tested, and approved for use prior to commencing the grouting operations on a project. For details on the different classes of grouts, see Ref. 4.3.

4.4.6.2.1 Duct Inlets and Outlets

Inlets are used for injecting the grout into the duct, while outlets allow the escape of air, water, grout, and bleed water. Inlets are typically provided near low points of the tendon, while outlets are near high points. Outlets at low points provide for drainage and an additional inlet when grouting from the original inlet becomes difficult due to blockages or high pumping pressures. The use of transparent plastic tubing from outlets will allow better visual examination for bleed water. The location of inlets and outlets must be detailed on the post-tensioning field placement drawings.

Inlets and outlets along the length of the duct must be airtight to prevent cement paste from infiltrating the duct during concrete placement, and to prevent grout from escaping during the grouting operations.

The position of inlets and outlets is related to the direction in which the grout flows, the inclination of the ducts, anchorages, and couplers, and the allowable grout pressure. In some situations, they may be interchanged to make grouting and re-grouting possible. Appendix E in Ref. 4.3 contains illustrations of typical inlet and outlet locations. Inlets and outlets are typically located at:

- The anchorage area of the tendon (inlet or outlet)
- The high points of the duct, when the vertical distance between the highest and lowest point is more than 20 in. (500 mm) (outlet)

- At or near the lowest point of a tendon (inlet)
- At all low points; should be free draining (outlets)
- Major changes in the cross-section of the duct, such as couplers and anchorages (inlets or outlets)
- Other locations recommended by the Design Engineer or the post-tensioning supplier

It is recommended that an additional grout outlet be provided within a short distance downstream of a high point outlet.

The locations of inlets and outlets must be clearly shown on the working (shop) drawings. Outlets and vents near high points are fixed vertically to ensure they are higher than the top of the trumpet or the crown of the duct. In this case, ends of the outlets must remain above these high points. Vents at the bottom of the low points may be used when provisions are made to drain the ducts of moisture prior to grouting the ducts.

When detailing grout inlets and outlets, particular attention should be paid to any location where a significant directional change in the tendon profile or a change in cross-section occurs.

4.4.6.3 Grouting Equipment

Grouting equipment consists of measuring devices for water, cement and admixtures, a mixer, a storage hopper (holding reservoir) and a pump with all the necessary connecting hoses, valves, pressure gauge, and testing equipment. Accessory equipment shall provide for accurate solid and liquid measures of all materials to be batched. The capacity of the equipment must be sufficiently large to ensure that the post-tensioning duct or ducts to be grouted can be filled and vented without interruption at the required rate of injection. Under normal conditions, the grout equipment shall be capable of continuously grouting the longest tendon on the project in not more than 30 minutes once mixing has commenced.

4.4.6.3.1 Grout Mixing

The equipment used for grout mixing must be capable of continuous mechanical mixing and agitation that will produce uniform distribution of materials, passing through screens, and pumped in a manner that will completely fill the ducts.

4.4.6.3.2 Grout Injection Equipment

Injection equipment must be capable of continuous operation with little variation of pressure and must include a system for recirculating the grout while actual grouting is not in progress. The equipment must be capable of maintaining pressure on completely grouted ducts and be fitted with a valve that can be locked off without loss of pressure in the duct. Using compressed air to aid in the pumping of grout should not be allowed.

4.4.6.3.3 Air Compression and Flushing Equipment

The specifications should require that the Contractor have oil- and water-free compressed air available on-site for the purposes of: 1) checking free passage of the ducts; 2) blowing out any excess water; and 3) checking the ducts for leaks. In addition, the Contractor should have adequate flushing equipment and a potable water supply available to facilitate complete removal of the grout from the duct if necessary.

4.4.6.3.4 Grouting Attachments

Accessories, such as grouting and venting attachments—including seals between grout caps and anchors—are necessary to accomplish the successful grouting operation of bonded tendons. Grouting and venting attachments must be capable of holding positive pressure (usually at least 150 psi [1 MPa]). Proper venting must allow for the filling of all voids in anchors and ducts with grout.

4.4.6.4 Personnel Qualifications

Good workmanship is also very important for ensuring proper grouting. Specifications should require that grouting operations be carried out by workers who are trained for and experienced in the tasks required. In addition, grouting should only be performed under the immediate control of a person skilled in various aspects of grouting. This person should be trained and certified in grouting procedures. One program that provides the requisite training and an independent assessment of grouting skills and knowledge is the American Segmental Bridge Institute's (ASBI) Grouting Certification Training program. For all post-tensioning projects involving a significant degree of grouting it is recommended that the project contract specify that at least one person who will be performing the grouting be ASBI certified.

Table 4.7 - Permissible Intervals Between Tendon Installation and Grouting

Exposure	Permissible Intervals Between Tendon Installation and Grouting Without use of a Corrosion Protection
Very Damp Atmosphere or over salt water (Humidity > 70%)	7 days
Moderate Atmosphere (Humidity < 70% > 40%)	20 days
Very Dry Atmosphere (Humidity < 40%)	40 days

4.4.6.5 Grouting Construction Procedures

Strands placed inside the ducts should be protected against corrosion, unless they are grouted as soon as possible after stressing. The time from tendon installation in ducts to grouting should be within the limits of Table 4.7. Temporary protection might include water-soluble oils, sealing of ducts to prevent moisture from getting through to the tendons, and continuous pumping of super dry air.

Grouting should proceed as soon as possible after stressing of the prestressing steel in the ducts. To minimize possible adverse effects, it is recommended that the time from installing the tendons in the ducts in an unstressed condition to grouting after stressing not exceed the periods in Table 4.7 unless approved by the post-tensioning supplier. Precautions should be taken at all times to minimize exposure of the tendons to weather and on-site conditions that may promote corrosion.

Additional corrosion protection measures may include the use of water-soluble oils applied by the prestressing steel manufacturer or at the job site. However, care must be exercised because water-soluble oils may have an adverse effect on bond.[4.22]

4.4.6.5.1 Preparation for Grouting Operations

Before proceeding with grouting operations, the following should be checked with respect to both materials and equipment:

- Temperature and possible caking of dry packaged materials, such as cement or prepackaged grout
- Shelf life of all materials including prepackage grouts
- Weight of the bags for mix design proportioning
- Weight of containers of the liquid or mineral admixtures
- Performance of the equipment to be used, including stand-by equipment
- Leak-free connections to inlets and outlets and proper operation of valves
- Ducts, anchorage block-outs, openings, inlets and outlets must be kept clean and free of debris, fuel, oils, other contaminants and site trash at all times prior to and after installing the tendons

Prior to grouting, oil-free compressed air should be blown into the ducts to remove water and debris blockages that may interfere with the injection. It is recommended that ducts then be air pressure-tested to locate any identifiable potential grout leaks.

4.4.6.5.2 Storage of Grouting Materials

Cement and other prepackaged materials can be delivered in bags and stored in a building, bin, or other location that is both weatherproof and conveniently located near the work to be performed. Open-air storage should be permitted if approved by the post-tensioning supplier and if stored on a raised platform and adequate waterproof covering is provided. If temperatures fall below 32°F (0°C), care must be taken in storing the grout material. Storage in a warm environment can help reduce the effects of delayed setting and excessive bleeding associated with cold storage of materials.

For practical considerations, prepackaged grouts should not be stored for more than one month before they are used. Bags of cement containing "clumps" should be rejected as they may not mix properly and may clog the pumping equipment or block the free flow of the grout into the ducts.

4.4.6.5.3 Grout Injection

The method of injecting grout shall ensure complete filling of the ducts and complete surrounding of the strand or bar with grout. Grouting must continue without interruption so that grout flows continuously in the same direction from the inlet to the outlet. A continuous, one-way flow of grout shall be maintained within a grouting stage. Grouting of a tendon (or designated group of tendons) shall be performed in one operation. The grouting rate shall be slow enough to avoid air entrapment and segregation of the grout and ensure complete filling of the duct. The optimum speed or rate of grouting will depend on the type of grout, size of duct, amount of steel inside the duct, duct surface profile (i.e. smooth vs. corrugated) and equipment size.

Grout should be injected from near the lowest end of tendons in an uphill direction. In most instances, grout must be used within 30 minutes of the first addition of water to ensure the flowability of the grout. Grouting from the end anchorage is acceptable if it can be shown to completely fill the duct. It is recommended that caution be exercised with "oval" or "flat" ducts (e.g., rectangular sections with rounded ends) when used in thin sections. This is especially important when the flat thin side becomes parallel to and within approximately two duct thicknesses of the face of the section. Such caution is necessary in order to avoid exerting excessive lateral force from the grouting pressure and possibly splitting or cracking the section. Similar precautions should be considered for closely spaced and parallel banks of ducts.

In very high vertical ducts, when the grouting pressure becomes excessive, the grout should be placed in lifts. The lifts should be limited in height so as to prevent excessive pressure and bleeding. Additional inlets should be used as needed to facilitate grouting each lift.

4.4.6.5.4 Temperature Considerations

Grouting at 40°F (4°C) or lower temperatures should be avoided because it significantly impedes the strength development of grouts, quantity, and rate of bleed. When temperatures are below 32°F (0°C), ducts must be kept free of water to avoid damage during freezing. If cold temperatures are expected, use of a freeze-resistant grout should be considered.

In high ambient temperatures (e.g., above 100°F [38°C]), the temperature of the grout shall not exceed 32°C (90°F). Since the length of time over which the grout is workable reduces with high temperature, the contractor should be required to demonstrate that the actual grouting could be accomplished within the expected temperature range; the use of pumping equipment with extra output capacity may be necessary.

In general, it is not practical to cool down the structure or the post-tensioning ducts before grouting (e.g., by flushing with ice water). The cooling measures implemented should therefore aim at keeping the grout temperature low before grout enters the ducts to delay as much as possible the setting of the grout. Several techniques can be employed to reduce the temperature of the components quite effectively at the job site. Careful shading of the dry materials out of the hot sun has been found to be effective. Ice added to the mixing water can also reduce the temperature of the freshly mixed grout to a level low enough to avoid flash set.

4.4.6.6 Grout Quality Assurance/Quality Control

The project specifications should detail the inspection and testing that will be required to ensure quality grouting. It is recommended that this include the considerations described below.

4.4.6.6.1 Material Certifications

The Contractor should be required to provide written certification that all ingredients used in the grout meet the applicable ASTM Standards or any other specification requirements. This usually includes test reports for: cement, mineral additives, chemical admixtures and for any other grout ingredients. For prepackaged grouts, the manufacturer shall supply the current certified mill test reports for the product.

4.4.6.6.2 Laboratory Testing

It is advisable, particularly on any project involving a significant amount of grouting, that trial batches of the proposed grout mix be prepared and tested by an independent laboratory using the same materials and equipment that are used on the job site, prior to the scheduled start of production grouting. Tests of trial grout should include:

- Setting time
- Grout strength
- Permeability
- Volume change
- Pumpability and fluidity
- Bleed
- Corrosion
- Wet density

Detailed information regarding the methods and acceptance criteria for the above tests can be found in Ref. 4.3.

The Design Engineer can waive the laboratory trial test requirements for a prepackaged grout that has previously met the necessary qualifying performance tests; or if the results of earlier tests on grouts with the same design, same source of materials, procedures and equipment are satisfactory and within the acceptance requirements.

4.4.6.6.3 Grouting Plan

It is also beneficial to require the contractor to submit a written plan for grouting operations prior to the start of construction. This plan should address the materials and equipment to be used; the procedures for mixing, pumping, duct cleaning, handling blockages and possible regrouting; and the direction/sequencing of grouting. In addition, the plan should detail the inspection procedures that will be used.

The preparation of a grouting plan, though not necessary on every project, will help the construction team anticipate and respond to unforeseen construction difficulties (e.g. equipment breakdown, weather delays, etc.) and will help ensure that grouting is done in accordance with project requirements.

4.4.6.6.4 Grout Field Testing

Project specifications should also detail the quality control that will be required during grouting construction. Field testing may include: field trial tests, field mock-up tests, and production tests.

1. **Field Trials**: Field trial tests of grout are designed to demonstrate that the grouting equipment, methods, and procedures are appropriate. Field trial tests conducted prior to initiation of production grouting will provide an early indication of potential problems. If specified, field trial batching and testing should be performed with the same materials, personnel, and equipment used in production grouting and in accordance with appropriate test procedures.
   The Design Engineer may choose to waive field trial testing if results of earlier tests on grouts with the same mix design, exact materials, equipment and procedures were satisfactory and within the acceptance requirements, or if the scope of the project does not warrant such testing.

2. **Field Mock-Up Tests**: When faced with complicated tendon geometry, a new type of duct, or other features that might raise concerns about successful grouting, the Design Engineer may wish to specify that field mock-up tests be performed. Field mock-up tests of grouts are designed to verify and demonstrate that the materials, outlets, inlets, mixer, grouting equipment, methods, and procedures are appropriate and will result in complete filling of the duct.
3. **Production Tests**: Production tests should be specified to ensure that grouting execution is of high quality and in conformance with specification requirements. At a minimum, it is recommended that the following tests be performed during production grouting:
   - One pressure bleeding test per day
   - Two wet density (mud balance) tests per day
   - One strength test per day
   - Two fluidity (flow cone) tests every 2 hours during the grouting operation
   - One volume test per day (if an expansive admixture is used)

   The number and frequency of these tests should be detailed in the project specifications.

4.4.7 Tensioning Requirements

4.4.7.1 General

Project specifications should detail and set forth all of the relevant requirements for equipment, stressing requirements and limitations, elongation tolerances, acceptance criteria for wire failures, and directives for calculating post-tensioning loss due to friction. Standard specifications are available which address most, if not all, of these requirements. In some instances, the Design Professional may want to supplement these standard provisions with additional requirements to reflect project-specific needs.

4.4.7.2 Equipment

4.4.7.2.1 Stressing Equipment

Project specifications should limit specialty equipment used on the job site to that furnished by the supplier of the post-tensioning system. The installer need not supply typical hand tools familiar to most ironworkers.

Each stressing ram must be equipped with a pressure gauge for determining the jacking pressure; each ram and gauge must be calibrated as a unit. The post-tensioning supplier or an independent laboratory shall perform initial calibration of rams and gauge(s). Load cells used to calibrate the stressing equipment must be calibrated within the past 6 months. For each ram and gauge unit used on the job site, the post-tensioning supplier must furnish certified calibration charts to the Design Engineer prior to stressing. The post-tensioning supplier must further verify that ram and gauge serial numbers match those listed on the calibration sheets.

Calibrations subsequent to the initial calibration with a load cell may be accomplished using a master gauge. Permanent gauge readings can be easily verified by installing a master gauge to a quick-attach hydraulic manifold that allows quick and easy installation of the master gauge.

Any jack repair, such as replacing seals or changing the length of the hydraulic lines, is cause for recalibration using a load cell. No extra compensation will be allowed for the initial or subsequent calibrations or for the use and required calibrations of the master gauge.

Following are the requirements and limits for proper jacking (for detailed commentary on each of these requirements, refer to Section 7.1 of Ref. 4.1):

- Stressing equipment must produce jacking forces at least 80% of tendon MUTS.
- Jacking force test capacity must be at least 95% of tendon MUTS.
- Jack cylinder area is to be permanently identified on the jack.
- Wedge seating methods must ensure uniform seating of wedge segments and uniform wedge seating losses on all tendon strands.
- Proper devices must prevent accumulation of deferential seating losses during cycling.
- The use of wedge power seating capability for jacks used for stressing tendons less than 20 ft (6 m) in length.
- In order to hold the pressure, hydraulic power units must be equipped with pressure relief and check valves.
- Hydraulic flow controls must have features for fine adjustments.
- Static bursting pressure must be at least four times the maximum operating pressure for hydraulic hoses and fittings.
- Gauges must be calibrated (independent of the jacks) to read actual pressures at the expected maximum pressure range.
- Gauge diameter must be at least 6 in. (150 mm) and display increments of 100 psi (0.50 MPa).

All strands in each tendon must be stressed simultaneously using a multi-strand jack. Mono-strand type jacks may be used for stressing flat-duct multi-strand tendons. This equipment may also be used for stressing straight multi-strand tendons in situations where access prevents the use of larger rams.

4.4.7.2.2 Strand Installation Equipment

When laying out equipment and during concrete placement, the Contractor must ensure that the post-tensioning steel pre-assembled in ducts be accurately held in place.

When the prestressing steel is installed after the concrete has been placed, the Design Engineer must verify that: 1) all tendons are free and unbonded to the surrounding duct; and 2) ducts are free of water and debris. Some projects might specify checking the ducts to ensure they are free of obstructions and have not been damaged.

The Contractor must make sure that all anchorage devices or block-outs for anchorages are set and held such that their axis coincides with the axis of the tendon, and that all tendons are normal to anchor plates in all directions. Furthermore, the post-tensioning tendons must be placed in each web such that the force is equally distributed in all webs or as required by the contract document. For box girders with more than two girder stems, at the Contractor's option, the prestressing force may vary up to 5% from the theoretical required force per girder stem, provided the required total force in the superstructure is distributed symmetrically about the centerline of the typical section.[4.10]

Except for flat duct systems, it is usually preferable to install the prestressing steel after concrete placement. Strands installed after the concrete is poured are either pulled or pushed. Strands can be installed individually or as an entire bundle. Pulling and pushing equipment is not system related, where any equipment that does not cause strand damage can be used provided that:[4.1]

- Pulling lines shall have a capacity at least 2.5 times the dead weight of the tendons when used for essentially horizontal tendon installation. For vertical tendons, the safety factor on the win line shall follow OSHA safety requirements.
- Pushing wheels made from metal shall not be used.
- Bullets for checking duct clearance prior to concreting shall be rigid and be ⅛ in. (3 mm) smaller than the inside duct diameter. Bullets for checking duct clearance after concreting shall not be less than ¼ in. (6 mm) smaller than the inside duct diameter.

Pull through method on an entire bundle of strands should be carried out after concrete placement as the forces exerted on the duct at high and low points may be high enough to damage the duct.

Post-tensioned bars are typically pushed through the duct. When using couplers, the duct diameter must be large enough to accommodate the diameter of the coupler throughout the length of the duct, or the diameter of the duct must transition to a larger size at the appropriate locations. More on the push through and pull through methods can be found in Ref. 4.23.

4.4.7.3 Stressing

Stressing equipment for bonded, multi-strand post-tensioning is system related. A jack designed for one particular system is unlikely to fit another without major modifications. The design of the tendon anchorage assembly has to be coordinated with the design of the jack chair, wedge seating devices, tube bundles, and automatic or manual jack stressing heads.

All post-tensioning steel must be tensioned with hydraulic jacks such that the post-tensioning force is not less than that required by the plans or approved shop drawings, or as otherwise approved by the Design Engineer. Tensioning must be carried out with allowances for all losses, including frictional and anchorage seating losses. Stressing should not commence before the concrete reaches the specified initial compressive strength shown on the contract drawings.

Strand tails must have the minimum required length for the equipment to be used. Length varies and depends upon the stressing equipment.

With the exception of bar tendons, at least one strand must be stressed to 20% of the jacking force, to be used for measurement reference.

4.4.7.3.1 Jacking Stress

For low-relaxation strands, the maximum temporary stress (jacking stress) prior to seating in the post-tensioning steel must not exceed 80% of MUTS. Tendons must never be overstressed to achieve the expected elongation.

4.4.7.3.2 Initial and Permanent Stresses

The post-tensioning steel must be anchored at initial stresses that will result in the long-term retention of permanent stresses or forces of no less than those shown on the plans or the approved shop drawings.

Permanent stress and permanent force are the stress and force remaining in the post-tensioning steel after all losses, including long-term creep and shrinkage of concrete, elastic shortening of concrete, relaxation of steel, losses in the post-tensioning steel from the sequence of stressing, friction and unintentional wobble of the ducts, anchor set, friction in the anchorages and all other losses peculiar to the post-tensioning system. Permanent stress in low-relaxation post-tensioning tendons must not exceed 74% of MUTS.

4.4.7.3.3 Stressing Sequence

Except as noted on the plans or the approved shop drawings, permanent post-tensioning tendons in continuous members are usually stressed from both ends.

Single end stressing is permitted when the following conditions are satisfied:

- Space limitations prohibit double end stressing.
- The calculated elongation of the post-tensioning steel at the second end is ½ in. (13 mm) or less and wedges are power seated.
- Single end stressing applied at alternate ends of paired adjacent post-tensioning tendons is required to produce a symmetrical force distribution in agreement with the plan design.
- Staged stressing requiring the stressing of some tendons before others must be carried out in accordance with the approved shop drawings, or as otherwise approved by the Design Engineer.

When the sequence of stressing individual tendons is not specified in the contract documents or on the approved drawings, the stressing of post-tensioning tendons shall be performed in a sequence to minimize eccentric force on the member.

For flat tendon systems, strands are usually stressed individually, alternating from each side of the anchorage plate to evenly distribute the force at the anchor and decrease the possibility of concrete failure due to eccentric loading.

4.4.7.3.4 Record Keeping

The following record of the post-tensioning materials, operations, stressing, and grouting must be kept for each tendon stressed:

- Project name, financial project ID
- Contractor and/or subcontractor
- Tendon location, size and type
- Date tendon was first installed in ducts
- Reel number for strands and heat number for bars
- Tendon cross-sectional area
- Modulus of elasticity
- Date stressed
- Jack and gauge numbers per end of tendon
- Required jacking force
- Gauge pressures
- Elongations (theoretical and actual)
- Anchor set
- Stressing sequence
- Stressing mode (one end/two ends)
- Witnesses to stressing operation (Contractor and inspector)
- Date grouted
- Any other relevant information
- Complete copy of all stressing and grouting operations to be provided to the Design Engineer

4.4.7.4 Elongations and Agreement With Forces

It is the responsibility of the Contractor to provide a record of elongation for each tendon to the Design Engineer. Furthermore, the Contractor must ensure that the forces being applied to the tendon and the elongation of the post-tensioning tendon can be measured at all times. Stressing tails must remain uncut until the Design Engineer approves the stressing records.

All tendons must be tensioned to a preliminary force necessary to eliminate any take-up in the tensioning system before elongation readings are started. This force will range between 5 and 25% of the final jacking force. A dynomometer (or any other approved method) is used for the purposes of measuring the initial force. Each strand must be marked prior to final stressing to permit measurement of elongation and to ensure that all anchor wedges are properly set.

According to Section 10.10.1.4 of AASHTO LRFD Construction Specifications,[4.10] when a discrepancy between gauge pressure and elongation of more than 5% in tendons over 50 ft (15,000 mm) long, or 7% for tendons less than 50 ft (15,000 mm), is identified, the entire operation must be carefully checked for the source of error. ACI 318-02, section 18.20.1 uses 7% regardless of length. The error must be identified and corrected before proceeding. For provisional ducts over 50 ft (15,000 mm) in length, the discrepancy between gauge pressure readings and elongations may be increased to 7% before investigation into the source of the error. In any case, if there is a discrepancy found between gauge pressure and elongation, the load used as indicated by the gauge pressure shall produce a slight over stress rather than under stress.

In the event that agreement between the observed and calculated elongations at the required force falls outside the acceptable tolerances, the entire operation must be checked and the source of error determined and remedied to the satisfaction of the Design Engineer before proceeding further. In this case, and at the discretion of the Design Engineer, additional tests might be requested without additional compensation to the Contractor. The tendons must never be overstressed to achieve the calculated elongation.

4.4.7.5 Friction

The Design Engineer must calculate frictional and anchor set prestress losses using assumed values of friction and wobble coefficients. These values must be shown on the Contract Plans.

The Contractor must submit an independent set of calculations of prestress losses due to friction and anchor set based upon the expected actual coefficients and values for the post-tensioning system to be used. Shop drawings must show a typical tendon force diagram, after friction, wobble and anchor set losses, as well as the actual coefficients used in the calculations.

If, in the opinion of the Design Engineer, the actual friction significantly varies from the expected friction, post-tensioning operations must be revised so that the final tendon force is in agreement with the shop drawings.

When friction must be reduced, graphite may be used as a lubricant, subject to the approval of the Design Engineer. Lubricants used to decrease friction must be flushed from the duct as soon as possible after stressing is completed by use of lime-treated potable water. This should be immediately followed by dry blowing the duct with oil-free air.

4.4.7.6 Wire Failures in Post-Tensioning Tendons

Multi-strand post-tensioning tendons having wires that fail by breaking or slippage during stressing may be accepted provided that not more than one wire in any strand is broken and the area of broken wires does not exceed 2% of the total area of the prestressing steel in the member. The width of the slab member to be used for this calculation shall be designated by the Design Engineer.

4.4.7.7 Finishing of Tendons

Upon completion of the stressing operations, and once the Design Engineer approves measured elongations, tendon tails must be cut off using an abrasive saw, and either capped or encased in concrete to seal the anchorages against leakage. An oxy-acetylene torch may be used provided that project specifications do not prohibit this method. Cutting must be carried out within ¾ in. (20 mm) away from the wedge. Electric arc welders shall not be used.

Bar tendons should be cut approximately 1 in. (25 mm) behind the anchorage nut.

4.4.8 Identification and Testing

4.4.8.1 Identification

Post-tensioning material shipped to the job site must be assigned a lot number and tagged for identification purposes. This applies to wires, strands, bars, and anchorage assemblies.

A Manufacturer's Certificate of Compliance, a mill certificate, and a test report must accompany each lot of wire or bar, or each reel of strand. The mill certificate must include the following information:

- Elongation at a specific load
- Modulus of elasticity
- Area of steel
- Breaking load
- Load at 1% extension
- Ultimate elongation
- Stress-strain curve of the actual prestressing steel intended for use

4.4.8.2 Testing of Multi-Strand Tendons

Project specifications should identify the frequency of sampling for each component to be tested, including strand, bar, duct, couplers, anchorages, and grout material. It's important to test multi-strand tendons so that length and force differences after stressing and seating are minimized. Frequently, length variations between strands in a multi-strand test tendon are the most critical test variables. However, they have little practical significance for establishing the strand-wedge connection performance.

Test tendon installation inaccuracies and non-uniform seating losses in individual strand-wedge connections, including those in the stressing equipment, result in length and force differences of individual strands, which can be large enough to distort test results. The strand flares behind the anchorages further aggravate the non-uniform tendon force distribution to the individual strands. Under such conditions, it is difficult to determine the actual force in the particular strand that breaks first, which establishes the breaking force of the tendon.

Strand-wedge connections used in bonded applications do not require dynamic testing.

Bond transfer length between anchorages and the zone where full prestressing force is required under service and ultimate loads must be sufficient to develop the minimum specified ultimate strength of the prestressing steel. When anchorages or couplers are located at critical sections under ultimate load, the ultimate strength required of the bonded tendons must remain less than the ultimate capacity of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.[4.10]

Depending on the specific project requirements, the Contractor may be required to perform material testing in the field prior to commencing work on the project. Tests may be required to include the following factors.[4.23]

4.4.8.2.1 Tendon Modulus of Elasticity Test

Used to accurately determine tendon elongations during stressing. Project specifications must specifically require this test to be performed by the post-tensioning installer. The test is used to settle disputes when the recorded elongations fall out the specified tolerances. If the modulus of elasticity from field testing varies from the theoretical modulus used in the post-tensioning field drawings by 1% or more, the theoretical elongations on these drawings need to be adjusted using the modulus obtained through field tests.

4.4.8.2.2 In-Place Friction Test

Used to demonstrate that the friction characteristics, losses, and resulting tendon forces are in agreement with the design assumptions. This test is done in place (on a tendon in the completed structure) on each size and type of tendon. Size is defined as the number of strands and/or bars per tendon. Type is defined as the type of tendon family (i.e., cantilever unit, externally draped tendon, continuous beam, etc.). If the field-measured elongations vary relative to the theoretical elongations by ±7% (unless noted otherwise), the post-tensioning supplier will be required to review the conditions and correct the variance (e.g., by calculating the elongations based on the revised friction values).

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ANALYSIS AND DESIGN FUNDAMENTALS

5.1 BASICS OF POST-TENSIONED CONCRETE

5.1.1 Introduction

Concrete is strong in compression and relatively weak in tension. The tensile strength of concrete is about 10% of its compressive strength. In the flexural design of non-prestressed concrete members, the tensile strength of concrete is neglected; it is assumed that the concrete is cracked at all load levels and all tensile stresses are resisted by reinforcing steel. Less than half of the concrete cross-section of a typical non-prestressed concrete member is actually used to resist flexural compressive stresses. Resisting loads in bending with non-prestressed concrete is inherently inefficient-most of the concrete serves only to separate the reinforcing steel from the compression zone.

In a post-tensioned concrete member, some or all of the reinforcing steel is put into tension shortly after the concrete is placed and hardened, by elongating it with hydraulic jacks and anchoring it against the concrete with mechanical anchorage devices. This induces compressive forces and bending moments into the concrete, precompressing areas of the cross-section that will be subsequently subjected to tensile stresses from applied loads. This greatly increases the applied load producing first flexural cracking, and results in greater efficiency in resisting stresses resulting from applied loads. The increased efficiency in the use of the concrete material in post-tensioned members leads to a number of advantages over non-prestressed concrete, as described below.

1. **Reduction in dead load**:

- Post-tensioned concrete members generally contain about 30% less concrete than non-pre-stressed members designed for equivalent loads and performance.
- This reduction in dead load results in savings not only in the member itself, but in all of the other structural members whose required strength is a function of dead load, including columns, bearing walls, shear walls, and foundations.
- Post-tensioned floor systems are particularly cost-effective in zones of high seismicity where lateral earthquake forces are directly related to dead load.

2. **Reduced structural depth**:

- Because of the more efficient use of material, post-tensioned concrete members can provide equivalent or superior performance to non-prestressed members with significantly less structural depth.
- This reduces building height, and the cost of all related building components, such as plumbing and electrical systems and curtain walls.
- In multistory buildings the reduced structural depth possible with post-tensioned concrete often permits the adding of one or more floors with no increase in total building height.

3. **Reduced deflections**: In post-tensioned members, most of the dead load produces no deflection. This greatly reduces both instantaneous deflections and long-term deflections caused by flexural creep.
4. **Improved durability and fatigue resistance**

Post-tensioned concrete members are investigated at three distinct loading stages:

1. **At transfer of prestress force**:

- Behavior is elastic; plane sections remain plane; stresses are proportional to strains.
- Prestress force is maximum before any short or long term losses.
- Applied loading is minimum (no live or super imposed dead load).
- Flexural stresses are limited to permissible values specified in governing codes.

2. **Under service loading:**

- Behavior is elastic; plane sections remain plane; stresses are proportional to strains.
- Applied loading is full unfactored dead and live loads. Live loads depend on occupancy and are specified in the governing building code.
- Prestress forces are at effective levels, after all short and long-term losses.
- Prestress force is selected such that flexural stresses, deflections, and cracking are each limited to permissible values specified in governing building codes, and any additional criteria imposed by the Licensed Design Professional.

3. **At nominal strength**:

- Behavior is inelastic; plane sections are assumed to remain plane but stresses are not proportional to strains.
- Applied loading is full factored dead and live loads.
- Prestress forces are at nominal strength levels, specified in governing codes.
- Moments are redistributed inelastically in accordance with specified limits in governing building codes to produce the most cost-effective patterns.
- Usable flexural and shear capacity at every section must exceed moments and shears produced by applied factored loads and unfactored secondary moments without exceeding permissible concrete

and steel stresses and/or strains specified in the governing code.

- Non-prestressed reinforcement is added if required to satisfy strength requirements and any minimum reinforcement requirements specified in the governing codes.

Most contemporary design and analysis of post-tensioned concrete members is accomplished with the use of commercially marketed computer programs. These programs have been used since the late 1970s for the design of thousands of existing post-tensioned concrete structures, have been "debugged" for many years, and are highly reliable. Nonetheless it is important that the Licensed Design Professional understand the underlying concepts used in programs, which will be addressed in the remainder of this chapter.

5.1.2 Difference Between Analysis and Design

The structural *analysis* of a post-tensioned concrete member involves the systematic engineering examination of an existing or proposed member where all geometry, loading, reinforcing, and other properties are known. The structural *design* of the member involves starting with architectural criteria for geometry (spans and permissible depths) and occupancy (loading), and determining, by iteration, the final dimensions, material properties, and reinforcing in order to satisfy a set of known criteria, the building code as a minimum, and any additional criteria imposed by the Licensed Design Professional.

Although this is not immediately obvious, all structural design involves an iterative process of simply guessing an initial design, a starting point, then analyzing the design, modifying it based on the results of the analysis, and performing the analysis again on the modified design. This cycle continues until the designer is satisfied with the final design. The number of iterations involved in the design process is a function of how closely the initial "guess" resembles the final design. In post-tensioned concrete design, design aids are available to ensure that the "first guess" at a design will be reasonably similar to the final design, and will require few or no iterations. Long-established guidelines for span-to-depth ratios can be of great help in establishing initial values for structural depth; and normal ranges for average prestress compression stresses (F/A) and balanced loads are helpful in determining an initial value for prestress force.[5.1]

5.2 FLEXURAL ANALYSIS

5.2.1 Freebody Diagrams

Two unique freebody diagrams (*FBD*) are used in the flexural analysis of post-tensioned concrete members. The first, called the "classic" or "combined" *FBD*, consists of both the concrete member and the tendon within the concrete. In the second *FBD*, called the "equivalent load" *FBD*, the tendon is mentally removed from the member and replaced by the loads it exerts on the concrete. The equivalent load *FBD* actually consists of two freebodies, one for the concrete alone and the other for the tendon alone. Fig. 5.1 shows the combined *FBD* with the tendon and concrete combined; Fig. 5.2 shows the equivalent load *FBD* with the tendon and concrete separated.

In these two figures, the *FBD*s shown represent the simplest indeterminate models available for demonstrating all important fundamentals of post-tensioned concrete analysis. Complications such as column stiffness, additional spans, pattern live loading, and more complicated tendon profiles such as compound curves and harps, albeit realistic, add only mathematical complexity to the model, without demonstrating anything fundamental or unique about the behavior of post-tensioned concrete, and are not addressed herein. It is assumed that the Licensed Design Professional can incorporate them as required.

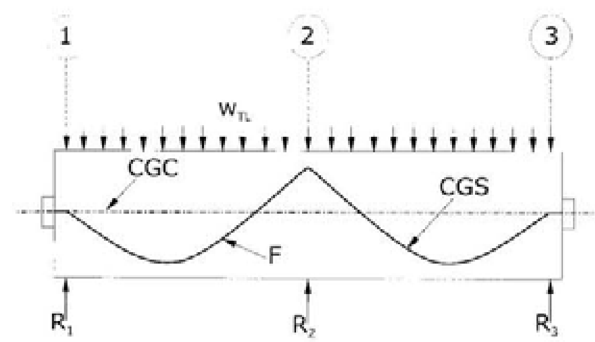


Fig. 5.1 Classic or "Combined" Freebody Diagram

In Fig. 5.1 the beam has a constant cross-section, applied external unfactored dead and live loads w_{TL} (factored total load w_u), a mathematically definable tendon profile (assumed to be parabolic in the examples herein), prestress force F, and knife-edge supports with no column stiffness. At the two ends of the beam the centroid of the tendon force F (the CGS) is assumed to be applied at the centroid of the beam cross-section (the CGC). If it is not applied at the CGC it can be transformed to that position with the addition of an appropriate moment acting at the ends of the beam.

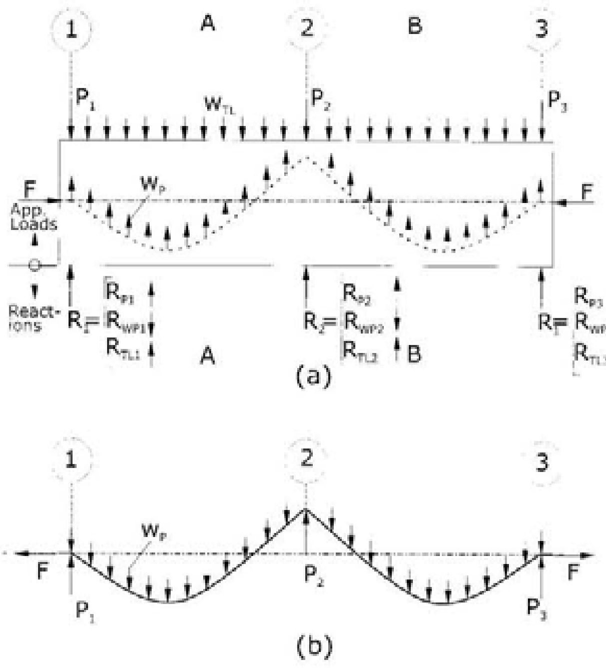


Fig. 5.2 Equivalent Load Freebody Diagrams

In Fig. 5.2(a) the tendon has been mentally removed from the beam and replaced with the set of loads it exerts on the beam concrete, F, P_1 through P_3, and w_p. These loads are called the "equivalent loads," and they are in equilibrium with themselves. The tendon itself is also shown in Fig. 5.2(b), equilibrated in its precise profile with the same set of loads, except equal and opposite in sense to those acting on the concrete. It is emphasized that the equivalent loads are completely general in nature. For example, the loads P_1 through P_3 are not necessarily applied directly into the supports and w_p is not necessarily the same in each span; nor is it necessarily a uniform load as shown. The only requirement for the equivalent loads is that they be in equilibrium with themselves, which is obvious from Fig. 5.2(b) where they equilibrate the tendon. The loads are shown the way they are in Fig. 5.2 for convenience (they are a simple and recognizable set of equivalent loads) but they can be any general set of equivalent loads in any shape or direction, which will equilibrate the tendon in a definable profile within the concrete.

Equivalent tendon loads are a function of the tendon force and the tendon profile. They are produced when one or more of the following conditions exist:

- A change in the slope of the tendon *between two points along* the tendon profile
- A discontinuity in the location of the CGC or CGS *at a point*
- A discontinuity in the prestress force *at a point*, such as at interior or exterior anchorages
- A change in the prestress force *between two points* along the tendon profile, such as that produced by tendon friction with the adjacent concrete

Some of the commonly used equivalent tendon profiles and the equivalent loads they produce with a constant prestress force are shown in Table 5.1.

5.2.2 The Equivalent Load Freebody Diagram

The equivalent load *FBD* is commonly used to calculate elastic flexural concrete stresses at transfer and service loading, and for the calculation of secondary reactions and moments. It has the following advantages:

- It is extremely physical and easy to visualize. Once the equivalent loads have been determined, the member can be analyzed like any non-prestressed concrete member acted upon by a set of loads.
- Any convenient structural analysis method can be used, including moment distribution and matrix methods.
- It is mathematically simple. Secondary moments and the actual vertical location of the tendon profile are not required to be known.

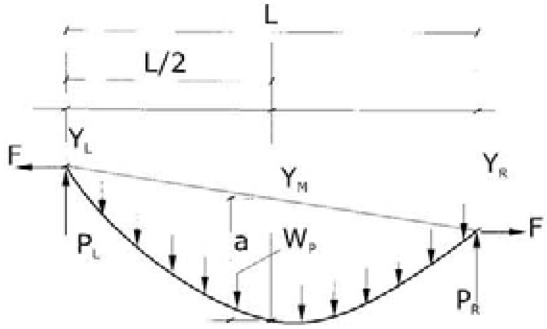
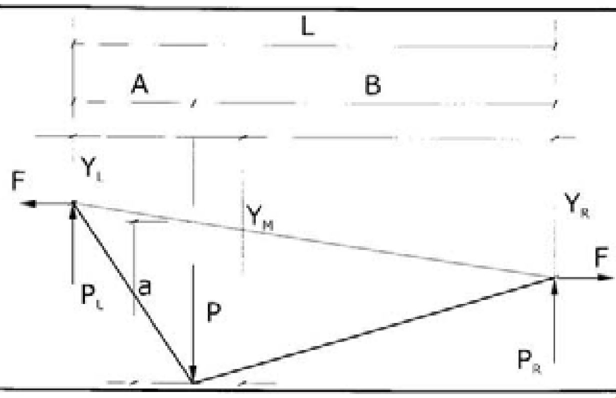
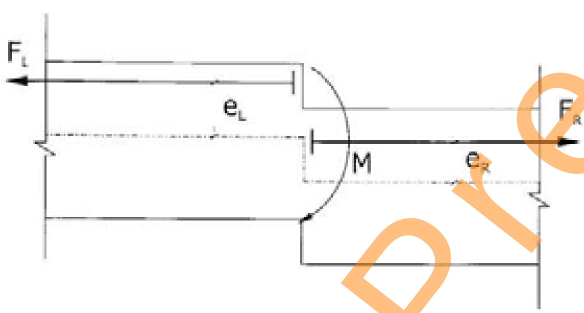
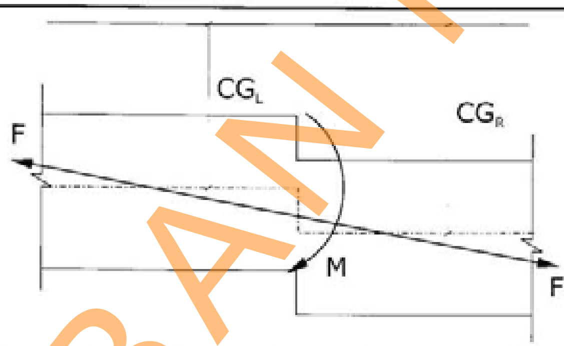
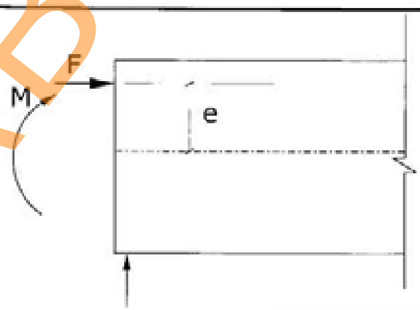
The equivalent load *FBD* cannot be used for nominal strength calculations, because nominal strength in concrete flexural members is based upon an internal tension-compression couple between the resultant compressive force in the concrete and the tensile force in the prestressing steel (and non-prestressed reinforcement), and in the equivalent load *FBD* the tendon and concrete are separated.

5.2.2.1 Concrete Flexural Stresses Using the Equivalent Load *FBD*

The equivalent loads produce a set of reactions acting on the concrete-only *FBD* shown in Fig. 5.2(a). It is important to distinguish between the reactions to the equivalent loads and the equivalent loads themselves, as shown in Fig. 5.2(a). Each reaction has three components. The first is the part of the total reaction that equilibrates the downward applied equivalent loads P_1 through P_3. These reactions are designated as R_{P1} through R_{P3} respectively at supports 1 through 3. The second component equilibrates the upward equivalent load w_p and is designated R_{WP1} through R_{WP3} at supports 1 through 3. The third component equilibrates the applied load w_{TL} and is designated R_{TL1} through R_{TL3} at the three supports. To calculate concrete flexural stresses at any cross-section A-A, a cut in the beam is made at that point, and the portion of the beam to the left of section A-A is isolated, as shown in Fig. 5.3.

The forces acting on cross-section A-A include a horizontal force F applied at the CGC, and a moment M_{net} which equilibrates the applied total load w_{TL} and its reaction R_{TL1}, and the "balanced load" moment M_{bal} which equili-

Table 5.1 - Equivalent Loads With Constant Prestress Force

TENDON PROFILE	EQUIVALENT LOADS
	**Parabola** $a = Y_M - \left(\frac{Y_L + Y_R}{2}\right)$ $w_P = \frac{8Fa}{L^2}$ $P_L = \frac{w_pL}{2} + \frac{(Y_R - Y_L)F}{L}$ $P_L = \frac{w_pL}{2} - \frac{(Y_R - Y_L)F}{L}$
	**Single-Point Harp** $a = Y_M - \left(Y_L + \frac{A}{L}(Y_R - Y_L)\right)$ $P = \frac{FaL}{AB}$ $P_L = \frac{PB}{L} + \frac{(Y_R - Y_L)F}{L}$ $P_L = \frac{PA}{L} - \frac{(Y_R - Y_L)F}{L}$
	**Section Discontinuity** $M = F_Re_R - F_Le_L$
	**Section and Force Discontinuity** $M = F(CG_R - CG_L)$
	**At Tendon Anchorage** $M = Fe$

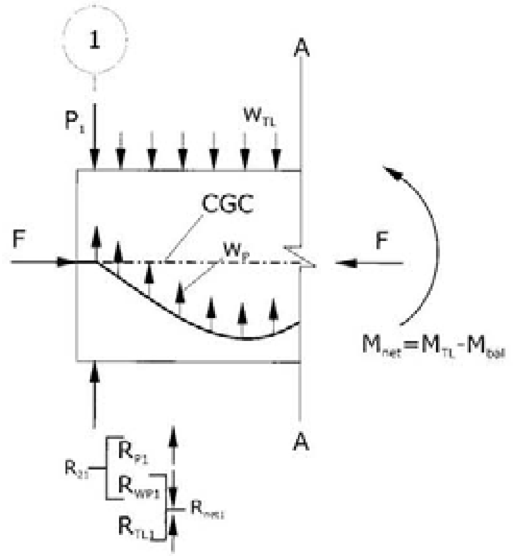


Fig. 5.3 Free Body Diagram for Calculating Concrete Flexural Stresses

brates the equivalent loads w_{TL} and P_1 and their reactions R_{P1} and R_{WP1}. It is noted that the algebraic sum of the reactions to the equivalent loads R_{P1} and R_{WP1} is called the "secondary" reaction R_{21} at support 1 (see Section 5.2.2.2), and since P_1 and R_{P1} are equal and opposite, they do not contribute to M_{net}, thus the reaction equilibrating M_{net}, w_{TL}, and w_p is simply the algebraic sum of R_{WP1} and R_{TL1}, which is called R_{net1} in Fig. 5.3. The vertical shear force acting on section A-A, which equilibrates all of the vertical loads and reactions acting on the freebody, is not shown.

Using the equivalent load *FBD*, the extreme fiber flexural stresses acting on cross-section A-A can be calculated with the simple expression:

$$f = -\frac{F}{A} \mp \frac{M_{net}}{S} \tag{5.1}$$

where A is the area of the cross-section and S is the appropriate top or bottom section modulus, and compression stresses are negative and tension stresses are positive. Note that the eccentricity e and the secondary moment M_2 do not appear in this equation, and are not required when calculating flexural concrete stresses with the equivalent load *FBD*.

5.2.2.2 Secondary Reactions and Moments Using the Equivalent Load Freebody Diagram

If the applied total load is removed from the concrete-only *FBD* shown in Fig. 5.2(a) the result is shown in Fig. 5.4. This *FBD* contains the equivalent loads acting on the concrete beam, and the reactions to those equivalent loads.

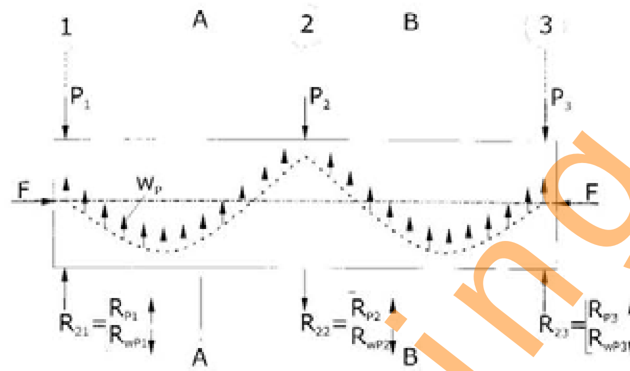


Fig. 5.4 Equivalent Loads and Reactions

Each reaction contains two components, one equilibrating the downward applied equivalent loads P_1 through P_3 which act upwards on the bottom of the beam and are designated R_{P1} through R_{P3} at the three supports. These reactions are statically determinate and can be calculated from the *FBD* of the tendon shown in Fig. 5.2(b). The other component is the reaction equilibrating the upward equivalent load w_p. These reaction components act downward on the bottom of the beam (they hold the beam down), and are designated R_{WP1} through R_{WP3} at the three supports. These reactions are statically indeterminate and must be calculated using an indeterminate analysis of the concrete beam itself. For statically indeterminate structures, the reactions R_{pi} are not equal to the reactions R_{WPi}, thus the weightless beam actually has external reactions produced to satisfy support compatibility requirements caused by the internal tendon loads. These reactions are called the "secondary" reactions and are designated as R_{21} through R_{23} in Fig. 5.4. If the tendon shown in Fig. 5.2(b) is now returned to its original position in the concrete-only *FBD* shown in Fig. 5.4, the set of equivalent loads acting on the tendon will cancel the set of equal and opposite equivalent loads acting on the beam. The *FBD* of the beam will then appear as in Fig. 5.5.

Note that the equivalent loads have "disappeared," yet the reactions to the equivalent loads (the secondary reactions R_{21}, R_{22} and R_{23}) remain acting on the freebody. The secondary reactions must be in equilibrium with themselves, because there is no other load acting on the beam. The secondary reactions produce moments in the beam as shown in Fig. 5.5. Secondary moments, because they are produced by reactions induced into beam supports, and do not involve loads applied between supports, are always straight lines between supports.

Secondary moments may also be calculated in a less physical, but simpler manner by considering Fig. 5.6, which shows an equivalent load concrete-only *FBD* isolated from the portion of the beam to the left of Section B-B, accompanied by the *FBD* of the removed tendon. The total moment M_{bal} acting on Section B-B can be expressed as

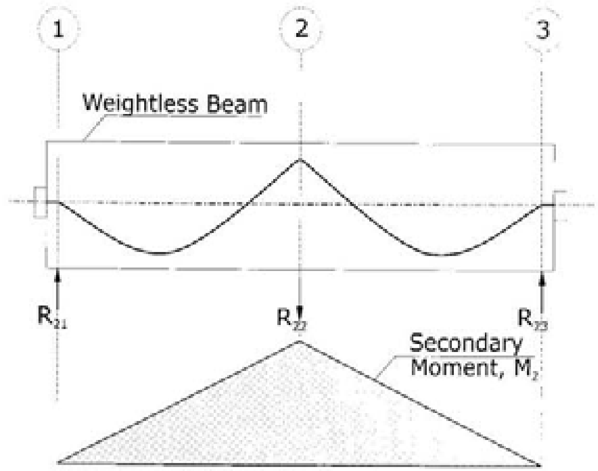


Fig. 5.5 Weightless Beam with Secondary Reactions and Moments

the sum of the moments M_{equiv} caused by the equivalent tendon loads alone (w_p, P_1 and P_2), and the secondary moment M_2 produced by the reactions R_{21} and R_{22} alone. Therefore:

$$M_{bal} = M_{equiv} + M_2 \tag{5.2}$$

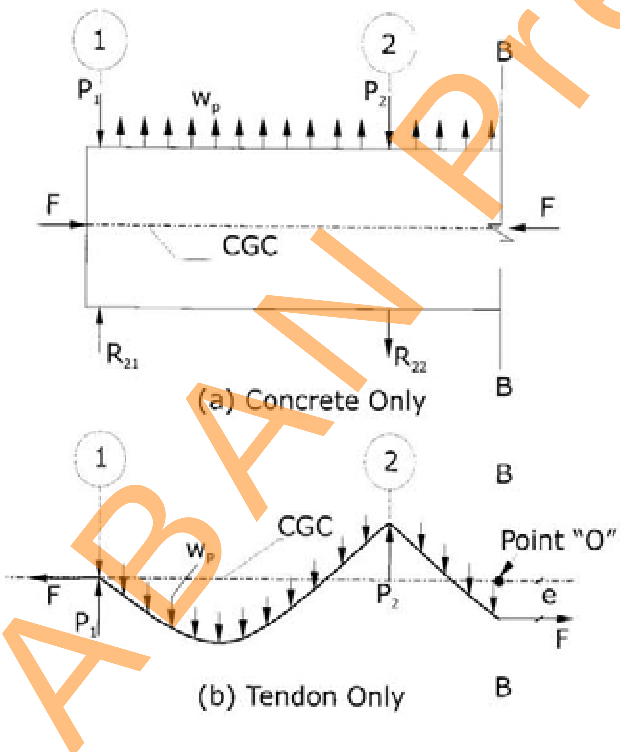


Fig. 5.6 Concrete-Only and Tendon-Only *FBDs*

The tendon in Fig. 5.6(b) has negligible flexural stiffness, so the moment acting on it at Section B-B (and every other section) is zero. Taking moments about point O, and recognizing that the loads w_p, P_1 and P_2 are the same set of equivalent loads applied in Fig. 5.6(a) but in opposite sense, where e is the distance between the CGC and CGS at Section B-B.

$$Fe = M_{equiv} \tag{5.3}$$

Combining Eqs. 5.2 and 5.3, and eliminating the term M_{equiv}, yields the following simple expression for M_2:

$$M_2 = M_{bal} - Fe \tag{5.4}$$

The term Fe is often called the "primary moment."

5.2.3 The Combined Freebody Diagram

The combined *FBD*, consisting of the tendon and the concrete member, can be used to calculate flexural concrete stresses. However, it is much more cumbersome for this purpose than is the equivalent load *FBD*, because both tendon eccentricity and secondary moments must be known at all points. The primary usage of the combined *FBD* is in calculations for nominal strength, where it *must* be used.

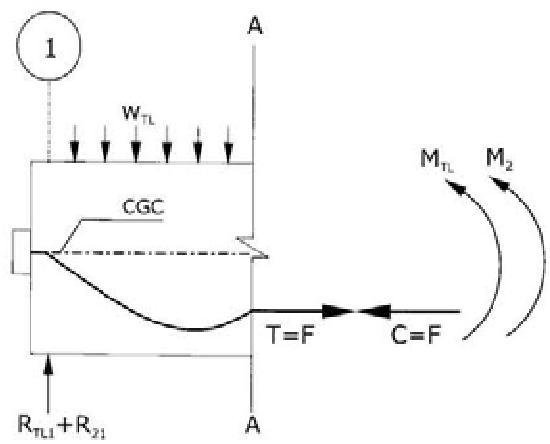


Fig. 5.7 Section A-A in Combined *FBD*; C and T are at the Same Level

5.2.3.1 Concrete Flexural Stresses Using the Combined Freebody Diagram

The combined or classic *FBD* is shown in Fig. 5.1. If a section were cut at A-A, between gridlines 1 and 2, the portion of the beam to the left of Section A-A can be isolated as shown in Fig. 5.7 (the shear force is not shown). It is convenient mathematically to move the resultant compression force in the concrete upwards until it coincides with the CGC. This induces a moment on the cross-section equal to Fe which is shown in Fig. 5.8.

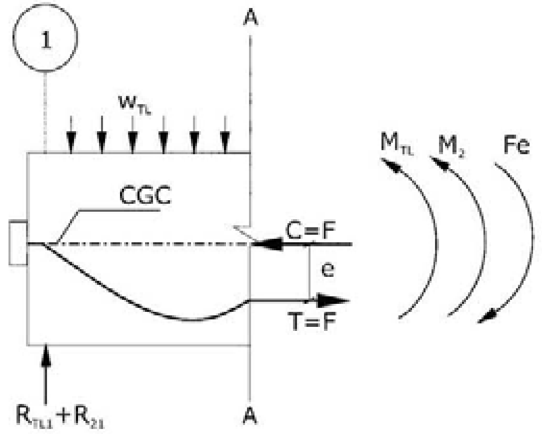


Fig. 5.8 Section A-A in Combined *FBD*; C is Moved to CGC of Section

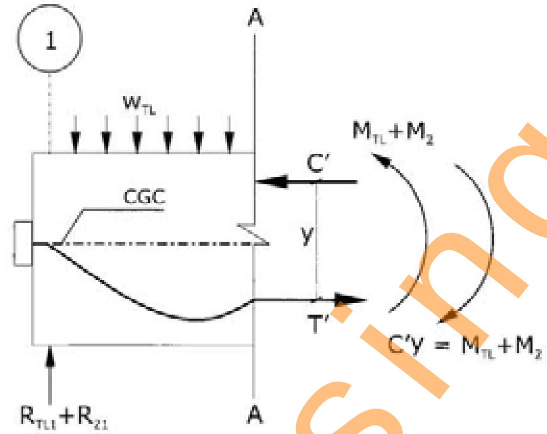


Fig. 5.9 Separate T' and C'

The flexural concrete stresses acting at the extreme fibers of the cross section can be calculated using the following equation:

$$f = -\frac{F}{A} + \frac{\mp M_{TL} \mp M_2 \pm Fe}{S} \tag{5.5}$$

Note that the use of the combined *FBD* requires that the tendon eccentricity e and the secondary moment M_2 be known. This involves considerably more mathematical effort than using the equivalent load *FBD* requires.

5.2.3.2 Nominal Strength Using the Combined Freebody Diagram

Fig. 5.7 shows the combined *FBD* of a portion of a post-tensioned beam to the left of Section A-A under service loads. The applied moment acting on Section A-A is the sum of M_{TL}, the moment equilibrating the total load w_{TL} and its reaction R_{TL}, and M_2, the moment equilibrating the secondary reaction R_2. The tensile force in the steel (under service loads) is T' (= $A_{ps}f_{se}$) and is equal to the compressive force in the concrete C'. The shear force acting on Section A-A is not shown.

If C' is separated upward from T' by a distance

$$y = \frac{M_{TL} + M_2}{C'}$$

the freebody will appear as in Fig. 5.9.

The two applied moments cancel each other and can be omitted from the diagram for convenience. The diagram can therefore be shown as in Fig. 5.10, where the stresses and strains acting on cross-Section A-A are shown.

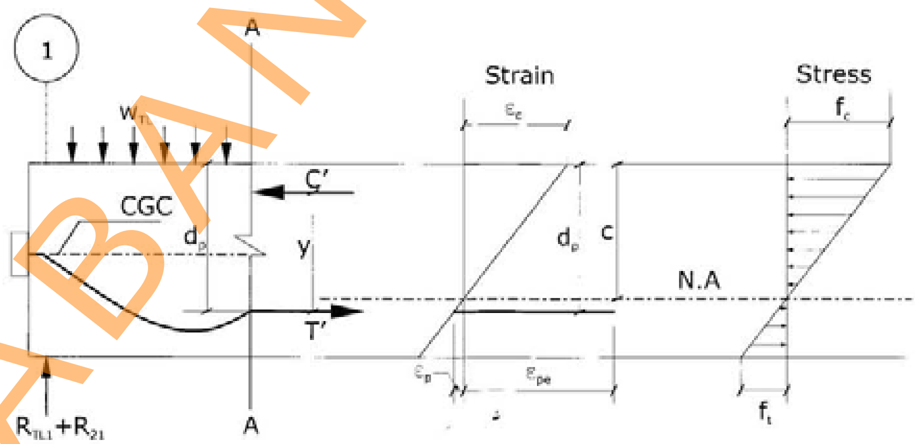


Fig. 5.10 *FBD* Showing Service Load Stresses and Strains

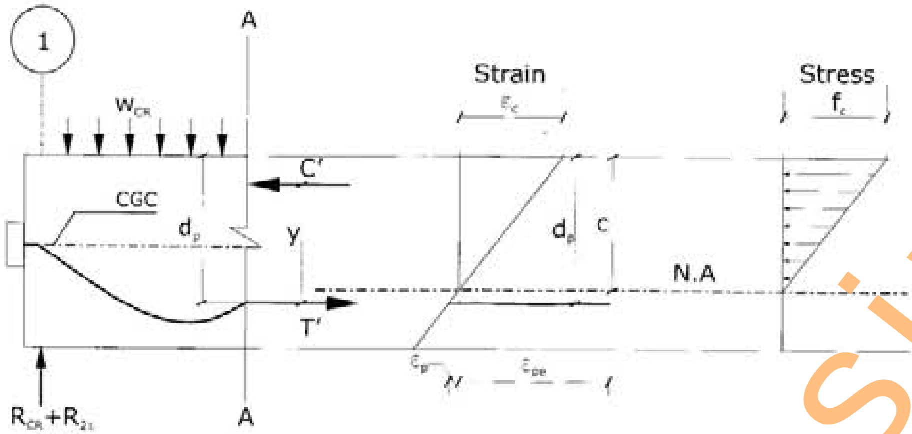


Fig. 5.11 *FBD* at First Cracking

In Fig. 5.10, the net tensile strain ε_p in the post-tensioned reinforcement is very small because the neutral axis N.A is very close to the CGS. Also, the concrete tensile stress f_t is below the concrete modulus of rupture and has not produced flexural cracking.

As the applied load increases above the service level, first cracking will occur at the tensile face. The *FBD* at first cracking is shown in Fig. 5.11.

At first cracking, the net tensile strain at the level of the prestressed reinforcement is still small and the strain in the steel is very close to the effective strain.

As the load increases beyond the cracking load, the neutral axis rises and the strain in the prestressed reinforcement increases significantly. At a certain load the relationship between concrete stress and strain becomes non-linear, and, finally, the concrete reaches its crushing or ultimate strain, commonly assumed to be 0.003. At this point the stress in the prestressed reinforcement is f_{ps}, the tensile force in the prestressed reinforcement is $T_p = A_{ps}f_{ps}$, and the nominal strength of the beam M_n is $T_p y$ where:

$$y = \frac{M_u + M_2}{T_p}$$

For bonded prestressing steel f_{ps} can be determined with the actual stress-strain diagram (with a strain of $\varepsilon_{pe} + \varepsilon_p$). For unbonded prestressing steel, f_{ps} is determined using empirical equations specified in building codes. The *FBD* of the beam at nominal strength is shown in Fig. 5.12.

The magnitude and location of the resultant compressive force C in the concrete can be accurately modeled with a rectangular compressive zone (or "stress block") with a uniform stress of $0.85f'_c$ and a depth of $\beta_1 c$ where β_1 is 0.85 for f'_c up to and including 4000 psi, reducing at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi, down to a minimum of 0.65. The *FBD* at nominal strength with this rectangular stress block is shown in Fig. 5.13. The nominal strength of the beam in Fig. 5.13 is $M_n = T_p(d_p - a_c/2)$.

Fig. 5.14 shows the combined *FBD* with non-prestressed tension and compression reinforcement. Note that the addition of compressive reinforcement A'_s reduces the total compressive force acting on the concrete, reduces the depth of the compression block a, raises the neutral axis (decreases c), and increases the net tensile strain ε_t in the extreme tension reinforcement. Taking moments about C, the nominal strength of the beam shown in Fig. 5.15 is:

$$M_n = A'_s f'_s\left(\frac{a_c}{2} - d'_s\right) + A_{ps}f_{ps}\left(d_p - \frac{a_c}{2}\right) + A_s f_y\left(d_t - \frac{a_c}{2}\right) \tag{5.6}$$

Building codes limit the maximum amount of prestressed and non-prestressed reinforcement in order to prevent a premature brittle failure. Codes also specify minimum amounts of bonded non-prestressed reinforcement for post-tensioned concrete members with unbonded tendons. These limitations are discussed in Section 5.3.3.

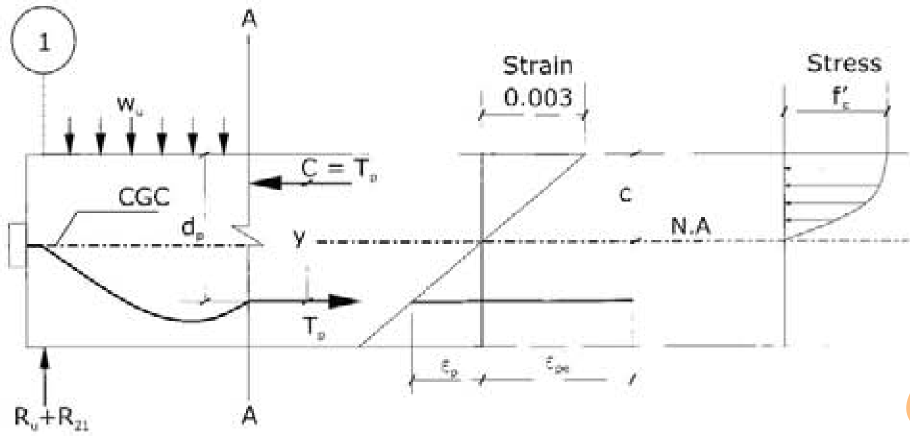


Fig. 5.12 *FBD* at Nominal Strength

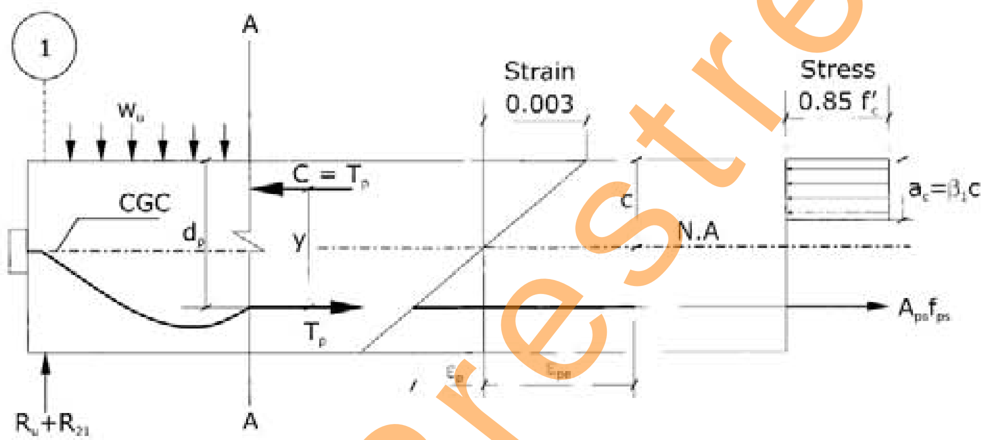


Fig. 5.13 *FBD* at Nominal Strength with Rectangular Stress Block

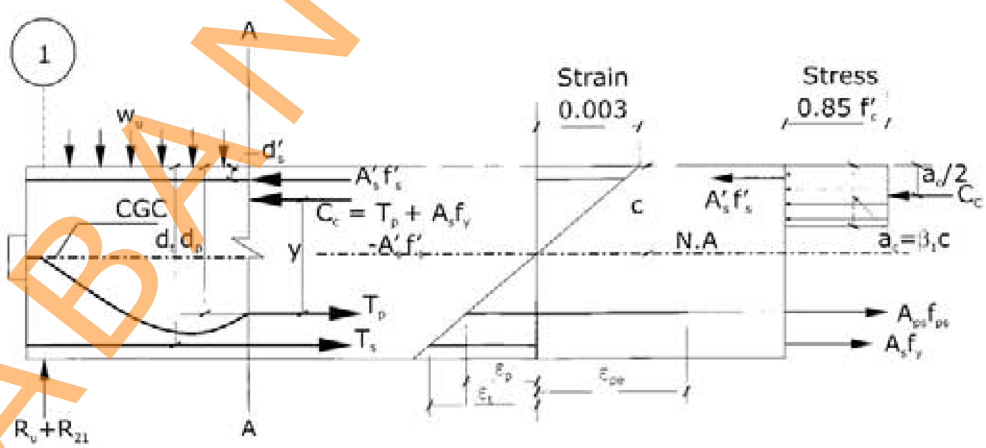


Fig. 5.14 Nominal Strength with Non-Prestressed Reinforcement

5.3 FLEXURAL DESIGN

It is standard practice in the design of post-tensioned concrete members to select the final prestressing force (area of prestressing steel A_{ps} or number of tendons of specified size) based upon service load criteria. Typically this is done by selecting an effective prestressing force (after all losses) that limits concrete flexural stresses under service loads to some predetermined values, and one or both of the following arbitrary criteria:

- Minimum average compressive stress F/A
- Equivalent loads, "balancing" some minimum percentage of service dead loads

Once selected, based upon the service loading stage, the amount of prestressed reinforcement remains constant through the other design phases. The selected amount of prestressed reinforcement is used to check concrete stresses at the transfer condition, and is also used at the nominal strength stage, where supplemental non-prestressed reinforcement may be required to either satisfy strength or minimum reinforcement criteria. The three design load stages are discussed in the following sections.

5.3.1 Building Codes

It is beyond the scope of this document to address all of the requirements for post-tensioned concrete in codes other than the ACI Building Code. It is very possible that more post-tensioned concrete structures are designed in accordance with the ACI Building Code (i.e. ACI 318-02) than any other code. For that reason, citations to and discussions of code requirements in this document (including references to section numbers and terminology) will refer to ACI 318-02. It is recognized that other codes and standard practices may vary somewhat from the requirements of the ACI Code. For design requirements unique to bridges, see Chapter 12.

5.3.2 Service Loading

Service loading consists of the actual unfactored dead loads (concrete weight and superimposed dead load) and code-specified live loads. ACI 318-02 classifies prestressed concrete flexural members as Class U (Uncracked), T (Transition), or C (Cracked), based upon the computed extreme fiber tensile stress f_t present in the concrete under service loading (18.3.3).

- Class U: $f_t \leq 7.5\sqrt{f'_c}$
- Class T: $7.5\sqrt{f'_c} < f_t \leq 12\sqrt{f'_c}$
- Class C: $f_t > 12\sqrt{f'_c}$

Serviceability requirements for deflection and crack control vary depending on the Class of member designed (see ACI 318-02 Table R18.3.3). Two-way post-tensioned slab systems are required to be designed as Class U members (18.3.3). Most post-tensioned one-way slabs and beams are designed as Class U or T members. Class C is useful for the design of one-way slabs and beams with heavy transient live loads.

Service load design consists of selecting a prestress force and profile which satisfy the following criteria:

- Extreme fiber concrete flexural stresses are less than or equal to those permitted for the appropriate Class (U or T).
- Serviceability criteria (deflection and crack control) are in conformance with the requirements of the governing building code.
- Average prestress compression stress (F/A) is within acceptable range.
- Equivalent "balanced loads" are within acceptable range.

5.3.3 Nominal Strength

Once the prestressing force has been determined by considering service load criteria, the ultimate strength of the member is designed. Strength design consists of providing a flexural capacity at every section which is greater than or equal to the demand strength. The demand strength is the bending moment acting at the cross section produced by factored applied dead and live loads, and unfactored secondary moments. Demand moments calculated by elastic theory may be inelastically adjusted using moment redistribution (see Section 5.3.3.5 herein and ACI 318-02 Section 18.10.4).

5.3.3.1 Prestressing Steel Stress at Nominal Strength

The stress at nominal strength in unbonded prestressing steel is determined by Eqs. 18-4 and 18-5 of ACI 318-02. These equations are repeated below.

For members with span-to-depth ratios (L/h) less than or equal to 35 (most beams):

$$f_{ps} = f_{se} + 10000 + \frac{f'_c}{100\rho_p} \qquad \text{ACI 318-02 Eq. (18-4)}$$

For members with span-to-depth ratios (L/h) greater than 35 (most slabs):

$$f_{ps} = f_{se} + 10000 + \frac{f'_c}{300\rho_p} \qquad \text{ACI 318-02 Eq. (18-5)}$$

For bonded tendons, stress at nominal strength can be determined from the actual stress-strain diagram for the prestressing steel used, or it can be determined with Eq. (18-3) in ACI 318-02:

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left\{\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}\left(\omega - \omega'\right)\right\}\right]$$

ACI 318-02 Eq. (18-3)

This equation can be simplified without a significant sacrifice in accuracy to:

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left\{\frac{A_{ps} f_{pu}}{b d_p f'_c}\right\}\right] \tag{5.7}$$

5.3.3.2 Upper Limit on Reinforcing

For many years the ACI Code placed an upper limit on the total amount of prestressed and non-prestressed reinforcement with the use of the following equation, which was applicable only to prestressed concrete members:

$$\omega = \frac{A_{ps} f_{ps} + A_s f_{ps} - A'_s f'_s}{b d_p f'_c} \le 0.36\beta_1 \tag{5.8}$$

This requirement ensures ductility at nominal strength, and guarantees that the stress values assumed for prestressed and non-prestressed reinforcement will be achieved prior to crushing the concrete. In ACI 318-02 the upper limit on reinforcing is established, for all reinforced concrete members, prestressed or non-prestressed, with limitations on ε_t, the net tensile strain in the extreme tensile reinforcement (the reinforcement farthest from the compression face). For tension-controlled flexural members, ε_t must be equal to or greater than 0.005. This is equivalent to a reinforcing index $\omega = 0.32\beta_1$, about a 12% reduction in the maximum amount of permissible reinforcing. The minimum strain value of 0.005 is equivalent to $c/d_t = 0.375$ maximum.

5.3.3.3 Minimum Reinforcing Requirements

The ACI Code (Section 18.9) requires a minimum amount of bonded reinforcement in all flexural members with unbonded post-tensioning tendons. These requirements ensure ductility and crack distribution equivalent to members with bonded reinforcement, and prevent concentrated cracking and "tied-arch" behavior. The minimum required amount of reinforcing varies depending on whether the member spans in one or two directions.

For one-way beams and slabs, the requirement is:

$$A_s = 0.004\,A \qquad \text{ACI 318-02 Eq. (18-6)}$$

where A is the area of the cross-section between the CGC and the extreme tensile face. The reinforcement required by ACI 318-02 Eq. (18-6) is uniformly distributed over the precompressed tensile zone as close as practicable to the extreme tension fiber. For solid thickness one-way slabs the requirement is equivalent to a minimum steel area of 0.2% of the cross-sectional area in zones of flexural tension, at the top of the slab over supports and at the bottom of the slab at midspans.

For two-way slabs, the Code requires a minimum area of bonded reinforcement in each direction in negative moment regions at the top of the slab over columns equal to:

$$A_s = 0.00075\,A_{cf} \qquad \text{ACI 318-02 Eq. (18-8)}$$

where A_{cf} is the larger cross-sectional area of the two equivalent frame slab-beams intersecting at the column. Reinforcement required by ACI 318-02 Eq. (18-8) is required to be placed between lines that are 1.5h outside opposite faces of the column. In positive moment areas of two-way slabs no minimum amount of bonded reinforcement is required unless the computed flexural tensile stress at service load exceeds $2\sqrt{f'_c}$. This is the only condition in the ACI Code that does not require minimum bonded reinforcement in a member with unbonded tendons. When the flexural tensile stress exceeds this value, the minimum amount of bonded reinforcement is:

$$A_s = \frac{N_c}{0.5\,f_y} \qquad \text{ACI 318-02 Eq. (18-7)}$$

where N_c is the tensile force acting on the concrete cross-section (the resultant force of the triangular tensile stress block) at service load.

The minimum lengths of all of the bars required for these minimum steel requirements are one-third the clearspan centered in positive moment areas, and one-sixth the clear span on each side of the column in negative moment areas. When this minimum bonded reinforcement is required for moment strength in either positive or negative moment regions, or to satisfy ACI 318-02 Eq. (18-7) in positive moment regions, the bars must be developed in accordance with Chapter 12 of ACI 318-02. This means that they must extend a full development length beyond the point at which they are no longer required.

Minimum required amounts of reinforcing in post-tensioned concrete members with bonded tendons are specified in ACI 318-02 Section 18.8.2 and discussed in the next section.

5.3.3.4 Cracking Moment

ACI 318-02 Section 18.8.2 requires a factored "load" at least 1.2 times the "cracking load," using a modulus of rupture $f_r = 7.5\sqrt{f'_c}$ for all prestressed concrete members except two-way post-tensioned slabs with unbonded tendons and flexural members with shear and flexural strength at least twice that applied by factored design loads. The requirement is intended to prevent "abrupt flexural failure developing immediately after cracking" (Commentary R18.8.2). The abrupt flexural failure is a sudden tensile failure of reinforcement as the tensile force is transferred from concrete to steel at first cracking. This behavior has never been observed in unbonded tendons since there is no strain compatibility between the steel and the concrete and no sudden transfer of stress from concrete to prestressing steel. ACI Committee 423[5.2] has recommended for many years waiving the $1.2M_{cr}$ requirement for *all* post-tensioned members with unbonded tendons.

This requirement is commonly satisfied, when required, at each cross-section as follows:

$$\phi M_n \geq 1.2\left(7.5\sqrt{f'_c}\,S + \frac{SF}{A} + M_{bal}\right) \tag{5.9}$$

where S is the appropriate top or bottom section modulus.

5.3.3.5 Moment Redistribution

Moment redistribution is a term that describes the behavior of a continuous indeterminate concrete member (two or more spans) after first yielding occurs at some cross-section of the member.[5.3] As applied load is increased on an indeterminate member, the response is initially elastic (deflections, moments, and shears are linearly proportional to applied load and can be calculated by elastic indeterminate theory) up to the load where yielding first occurs in any cross-section. The applied load producing first yielding at any cross-section is called w_1. Incremental applied load w_2 greater than w_1 is assumed to produce inelastic rotation at the yielded section, without change in applied moment. Since w_2 produces no incremental moment at the yielded cross-section, incremental moments resisting w_2 are developed at sections other than the initially yielded section. Thus after first yielding, moments are redistributed to other cross-sections of the member which are still elastic. As w_2 increases, eventually other sections will yield and develop hinges. When enough hinges have developed in any span of the member to make it unstable (that is, a mechanism rather than a flexural failure), the member is considered to have failed. The load at which a mechanism forms in any span is called the "limit" load in that span.

Moment redistribution is used in the design of continuous concrete members by providing a flexural capacity ϕM_n at the negative or positive moment regions of the member (or both) that is less than the moment at the same point calculated by elastic theory. The reduced moment capacities must be statically consistent with moments at other sections of the member under the same loading condition.

Moment redistribution provides the designer of continuous post-tensioned concrete beams and slabs a valuable tool for cost-efficient design. Understanding and taking advantage of the effects of inelastic behavior in indeterminate members generally permits the Licensed Design Professional to reduce both the maximum elastic positive and negative moments when live load is "skipped" (arranged in patterns that produce maximum possible positive and negative moments at all sections), thus narrowing the envelope of demand moments across the spans, and reducing the amount of reinforcing required for any given factor of safety. Moment redistribution also often permits the "shifting" of moments from cross-sections that are less efficient in resisting moment to those that are more efficient, resulting in further savings in reinforcing. Significant changes have been made in the 2002 ACI Building Code that simplify and unify moment redistribution in both prestressed and non-prestressed continuous beams and slabs. A detailed description of moment redistribution is found in Ref. 5.3.

5.3.4 Transfer of Prestress Force

Most post-tensioned concrete members are cast-in-place. The transfer loading stage rarely controls the design of cast-in-place post-tensioned concrete members, where most of the dead load is present at the time the tendons are stressed. An exception would be in slabs or beams supporting significant loading from floors above, where the entire post-tensioning force may have to be applied without the full superimposed dead loads acting on the member. Cast-in-place post-tensioned concrete members controlled by the transfer loading stage are likely to be poorly proportioned, highly stressed, uneconomical, subject to subsequent restraint-to-shortening problems, and should be avoided. Despite the fact that the transfer loading stage rarely controls, building codes require that it be investigated. The ACI Building Code (ACI 318-02) does this by limiting concrete flexural stresses.

At transfer, the prestress force is based upon a stress in the prestressing steel of $0.7f_{pu}$ [18.5.1(c)]. All equivalent loads acting at transfer must be based upon this steel stress. It is commonly assumed that the prestress force and all equivalent loads acting at transfer are 7/6 times the effective values (after all losses). Loading at the transfer stage should be based upon the concrete dead load only, unless the designer knows with certainty that additional superimposed loads will be present. The specified concrete compressive strength at transfer is f'_{ci}. Under those conditions, flexural stress in the concrete is limited to $0.60f'_{ci}$ in compression and $3\sqrt{f'_c}$ in tension.

5.4 SHEAR

While often considered to be less "interesting" than flexural design, proper design for shear is nonetheless of extreme importance. Contrary to flexural failures, which provide ample warning of impending failure in the form of deflection and cracking and rarely result in complete collapse, shear failures are brittle, can occur suddenly with no warning, and have resulted in catastrophic collapse.

A fundamental principle in the design of concrete members is to ensure flexural failure before shear failure, guaranteeing adequate warning of overload prior to a sudden failure. The ACI Code accomplishes this by requiring smaller strength reduction factors (ϕ) for shear than for flexure, and a general conservatism in the equations predicting concrete shear capacities (V_c). Designers are cautioned against providing flexural capacities significantly larger than required by Code without a similar increase in shear capacity. Concrete members with flexural capacities substantially larger than shear capacities, even if both exceed Code requirements, are susceptible to progressive shear failures in the event of a catastrophic loading event (such as an explosion or similar impact). Members with shear capacities in excess of their flexural capacities tend to isolate local shear failures by surrounding them with flexural failures (hinges), limiting or preventing the spread of shear failure to adjacent supports.

ACI 318-02 requires that the factored shear force acting on any concrete cross-section must be less than or equal to the usable shear capacity of the section:

$$V_u \leq \phi V_n \qquad (5.10)$$

where the nominal shear capacity consists of two components:

$$V_n = V_c + V_s \qquad (5.11)$$

V_c is the nominal shear strength of the concrete cross-section alone and V_s is the nominal shear strength of the shear reinforcement present at the cross-section. It should be noted that the strength reduction factor ϕ for shear is 0.75 (ACI 318-02 Section 9.3.2.3) and for tension-controlled flexural members it is 0.9 (Section 9.3.2.1). Shear design procedures differ for one-way slabs, two-way slabs, and beams. Each is discussed next.

5.4.1 Shear in One-Way Slabs

Shear stresses are generally very low in properly proportioned, uniformly loaded one-way slabs, and rarely if ever control their design. Attention must be paid to concentrated or line loading. Shear design for concentrated loads is similar to punching shear design in two-way slabs, discussed in Section 5.4.3. Shear reinforcement is not normally used in one-way slabs, where design for shear simply involves the selection of a slab thickness that provides adequate shear capacity. Shear capacity for one-way slabs with uniform or line loading can be conservatively based upon ACI 318-02 requirements for non-prestressed members:

$$V_c = 2\sqrt{f'_c}\, b\, d \qquad \text{ACI 318-02 Eq. (11-3)}$$

5.4.2 Shear in Beams

Shear strength V_c in prestressed concrete beams is determined by considering the following three equations:

$$V_{cn} = \left(0.6\sqrt{f'_c} + 700\,\frac{V_u d}{M_u}\right) b_w d \qquad \text{ACI 318-02 Eq. (11-9)}$$

$$V_{ci} = 0.6\sqrt{f'_c}\, b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \geq 1.7\sqrt{f'_c}\, b_w d$$

ACI 318-02 Eq. (11-10)

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3 f_{pc}\right) b_w d + V_p$$

ACI 318-02 Eq. (11-12)

Where:

$$M_{cr} = \left(\frac{I}{y_t}\right)\left(6\sqrt{f'_c} + f_{pe} - f_d\right) \qquad \text{ACI 318-02 Eq. (11-11)}$$

Note that M_{cr} in ACI 318-02 Eq. (11-11) is defined differently from the cracking moment included in Eq. (5.9) in Section 5.3.3.4.

The smaller of V_{ci} and V_{cw} is compared to V_{cn}, and the larger selected for the controlling shear strength V_c. V_{cn} is a simplified, conservative prediction for V_c that can be used in all cases. V_{ci} and V_{cw} are more accurate, more complex mathematically and generally less conservative predictions for V_c considering two known types of shear behavior in prestressed beams. ACI 318-02 Eq. (11-12) for V_{cw} predicts a type of shear failure known as "web shear cracking," caused by diagonal tension in the beam web at zones of high shear and small moments (an exterior support with minimal column stiffness). In the web cracking failure mode the shear crack initiates near the vertical center of the beam and progresses diagonally upward and downward. ACI 318-02 Eq. (11-10) for V_{ci} predicts a type of shear failure called "inclined shear cracking" that occurs in areas of high shear and high moment (interior supports of continuous beams). In the inclined cracking failure mode the crack initiates in flexure at the extreme tension fiber (normally the top), progresses downwards until it initiates a diagonal tension failure at which point it forks and becomes diagonal.

In the shear equations, the Code permits a minimum value of 0.8h for d, except for the d in the term $(V_u d)/M_u$ in ACI 318-02 Eq. (11-9), where d has no minimum.

The nominal strength of vertical shear reinforcement (stirrups) in prestressed concrete beams is:

$$V_s = \frac{A_v f_y d}{s} \leq 8\sqrt{f'_c}\, b_w d \qquad \text{ACI 318-02 Eq. (11-15)}$$

Combining Eqs. (5.10), (5.11), and ACI 318-02 Eq (11-15) results in the following equation for the required area of shear reinforcement per unit horizontal length of beam:

$$\frac{A_v}{s} = \frac{V_u - \phi V_c}{\phi f_y d} \qquad (5.12)$$

ACI 318-02 Section 11.5.5 requires a minimum amount of shear reinforcing (stirrups) in beams where:

$$V_u > \frac{1}{2}\phi V_n \qquad (5.13)$$

Except for beams where the total depth h is smaller than the largest of the following three quantities:

- 10 inches
- 2.5 times the flange thickness
- One half the web thickness

When shear reinforcement is required, the minimum amount of shear reinforcement per unit of length of beam is the smaller of the following two quantities:

$$\frac{A_v}{s} = \frac{50\, b_w}{f_y} \qquad (5.14)$$

$$\frac{A_v}{s} = \frac{A_{ps}\, f_{pu}}{80 f_y\, d}\sqrt{\frac{d}{b_w}} \qquad (5.15)$$

5.4.3 Shear in Two-Way Slabs

Two-way post-tensioned slabs are typically supported on isolated square, rectangular, or round columns. The slab area around these columns is subject to large shears and moments, and is susceptible to a unique type of shear behavior called "punching shear," in which the column literally "punches" through the slab, often with catastrophic consequences.

To prevent punching shear failure, the ACI Code requires, at each column, that the total factored shear, and a portion of the unbalanced factored slab moment, be transferred from the slab to the column through a section of slab concrete surrounding the column known as the "critical section." A critical section exists just outside the column, and just outside each change in slab thickness in the vicinity of the column, that is, at each drop cap. The critical section follows the column or drop cap plan shape (it is geometrically similar to the shape of the column or the cap), and it is located at a horizontal distance of d/2 from the edge of the column or drop cap. The shears and moments acting on the critical section produce stresses that are limited by Code to certain permissible values.

The unbalanced moment at each column (the moment transferred from slab to column) is transferred partly by direct flexure (in the contact area between slab and column), and partly by direct and torsional shear stresses on the critical section, thus:

$$M_u = M_f + M_v \qquad (5.16)$$

where M_u is the total factored moment transferred from the slab to the column, M_f is the portion transferred in direct flexure, and M_v is the portion transferred by shear stresses on the critical section. The fraction of the total unbalanced moment, which must be transferred by shear stresses on the critical section, is:

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{c_1 + d}{c_2 + d}}} \quad (5.17)$$

And the moment transferred by shear stresses on the critical section is:

$$M_v = \gamma_v M_u \quad (5.18)$$

Punching shear stress calculations assume the shear is applied at the centroid of the critical shear section, rather than at the centroid of the lower column. Fig. 5.15 shows a *FBD* of an exterior joint including the columns (represented by a single lower column) and the portion of slab inside the critical section. Equilibrium of this freebody requires a counterclockwise moment $V_u e_x$ which reduces the applied moment on the critical section.

The applied moment acting on the critical section is thus:

$$M_v = \gamma_v M_u - V_u e_x \quad (5.19)$$

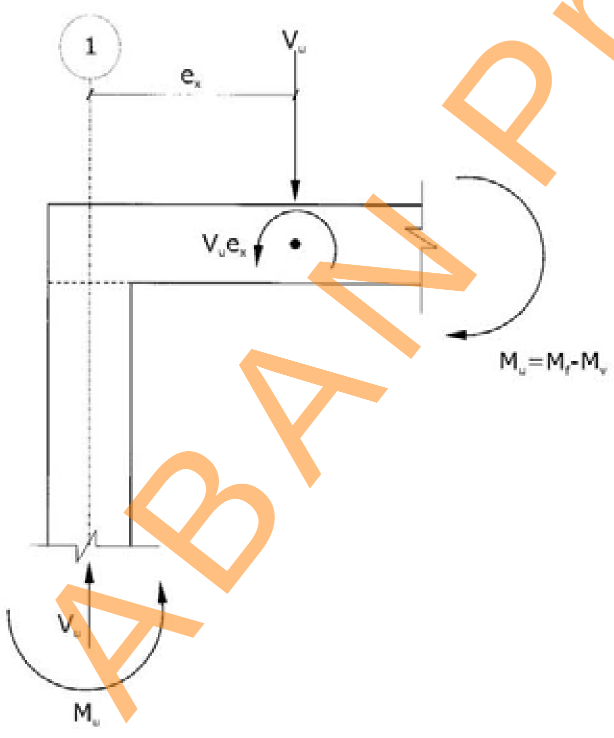


Fig. 5.15 Equilibrium at Edge or Corner Columns

Applied stresses acting on the critical section are calculated using the following equation:

$$f_v = \frac{V_u}{A_c} \pm \frac{M_v x_{L,R}}{J_c} \quad (5.20)$$

The allowable stress acting on the critical section for prestressed slabs is:

$$v_c = \left(\beta_p \sqrt{f'_c} + 0.3 f_{pc}\right) b_0 d + V_p$$

ACI 318-02 Eq. (11-36)

Where:

$$\beta_p = \frac{\alpha_s d}{b_0} + 1.5 \le 3.5$$

α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns; however, limitations on the proximity of free edges restricts the use of ACI 318-02 Eq. (11-36) to interior columns. For edge and corner columns, v_c is the smallest of the following three stresses:

$$v_c = \phi\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c} \quad \text{ACI 318-02 Eq. (11-33)}$$

$$v_c = \phi\left(\frac{\alpha_s d}{b_0} + 2\right)\sqrt{f'_c} \quad \text{ACI 318-02 Eq. (11-34)}$$

$$v_c = 4\phi\sqrt{f'_c} \quad \text{ACI 318-02 Eq. (11-35)}$$

5.5 VARIABLE PRESTRESS FORCE

Design methods for unbonded post-tensioned concrete flexural members in buildings have generally assumed that the effective prestress force is constant for the full length of the tendon. It was felt that the variable tendon force that exists due to friction at the time of stressing distributes with time until the force becomes equal at all points along the tendon. Consistent with this assumption, effective tendon forces have been normally based upon the average initial force in the tendon minus the long-term losses. On the other hand, the design of post-tensioned bridge members often considers the tendon force to be variable.

A study published in 1991[5.4] concludes that the assumed redistribution of tendon force does not occur and that the design of unbonded post-tensioned members should be based upon a variable effective prestress force calculated by subtracting long-term losses from initial tendon forces acting at each point along the tendon profile. Bondy[5.5] studied the ramifications of variable prestress force in typical post-tensioned concrete beams used in buildings and concluded that it has negligible impact when the tendon stressing length is less than 100 ft (stressed from one end only) and 200 ft (stressed from both ends).

5.6 PRESTRESS LOSSES

Loss in stress in post-tensioned reinforcement can be divided into two categories, short-term and long-term. Short-term losses occur between the time the stressing jack first starts to elongate the prestressing steel and the time the initial prestress force is transferred to the concrete by the anchorages. Long-term losses are all losses that occur after that point

Short-term losses include:

1. **Friction loss** (for information on how to calculate frictional losses, see Appendix A) –
   - Caused by friction between the prestressing steel and the tendon sheathing
   - Addressed in ACI 318-02 Section 18.6.2
2. **Seating of wedges at transfer of prestressing force** (for information on how to calculate losses due to seating of wedges, see Appendix A) – The wedges travel approximately 0.25 in. after the jack releases the tendon reducing the stress in a certain length of prestressing steel extending from the anchorage (friction resists movement of the prestressing steel caused by wedge travel).
3. **Elastic shortening of concrete** – The initial prestressing force shortens the concrete elastically, and the portion of the elastic shortening that occurs after an individual tendon has been anchored reduces the stress in the prestressing steel.

Long-term losses include:

1. **Concrete volume change** (shortening) caused by shrinkage and creep
2. **Relaxation of prestressing steel**

A definitive study of long-term loss calculations is presented by Zia,[5.6] et al. Precise determination of losses in post-tensioned concrete members is not critical to their behavior. Any reasonable estimate of losses is acceptable. A 100% variance in the estimate for total long-term losses generally results in less than a 10% difference in the stress in the prestressing steel at nominal strength.

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NOTATION

A	Cross-sectional concrete area
A_c	Cross-sectional concrete area of the critical punching shear section
A'_s	Cross-sectional area of unstressed longitudinal compression steel
A_p	Cross-sectional area of prestressed steel
A_s	Cross-sectional area of unstressed longitudinal tension steel
A_v	Cross-sectional area of shear reinforcement (stirrups)
a	Tendon sag (the maximum offset from the chord, the line connecting the two highpoints in each span)
a_c	Depth of rectangular compression stress block at nominal strength
b, b_w	Minimum web width of a T-beam
b'	Width of rectangular concrete compression stress block at nominal strength
b_o	Perimeter of the critical punching shear section
C	Total compression force acting on freebody cross-section at nominal strength ($= C_c + C_s = T_p + T_s = T$)
C_c	Compression force acting on freebody cross-section *resisted by concrete* at nominal strength
C_s	Compression force acting on freebody cross section *resisted by the compressive steel* at nominal strength
C'	Total compression force acting on the freebody cross-section under service load ($= T'$)
CGC	Centroid of concrete cross-section
CGS	Center of gravity of prestressing steel
c	Distance from extreme compression fiber to neutral axis
c_1	Column dimension parallel to beam span
c_2	Column dimension perpendicular to beam span
d	Distance from extreme compression fiber to the centroid of the resultant total tension force ($T_p + T_s$). In shear calculations *only* d need not be less than $0.8h$
d'_s	Distance from extreme compression fiber to centroid of unstressed compression steel A'_s
d_p	Distance from extreme compression fiber to centroid of prestressing steel A_{ps}
d_t	Distance from extreme compression fiber to centroid of extreme tension steel (steel farthest from compression fiber)
e	Eccentricity, distance between the CGS and the CGC
e_x	Horizontal distance from column centerline to centroid of the full critical punching shear section
F	Effective prestress force
f_c	Flexural concrete compressive stress
f'_c	Concrete compression strength at 28 days
f'_{ci}	Concrete compression strength at time of stressing
f_d	Extreme fiber flexural tensile stress caused by unfactored dead load
f_{pc}	Average concrete compression F/A
f_{pe}	Extreme fiber flexural compressive stress caused by equivalent tendon loads at the fiber *where tension is caused by applied gravity loads*
f_{ps}	Stress in prestressing steel at nominal member strength (ultimate stress)
f_{pu}	Specified maximum tensile stress in prestressing steel
f_r	Modulus of rupture in concrete, the flexural tensile strength or the stress assumed to produce first cracking (normally $7.5\sqrt{f'_c}$)
f_{se}	Effective stress in prestressing steel after all losses
f'_s	Stress in compressive reinforcement at nominal strength
f_t	Flexural concrete tensile stress
f_v	Combined shear stress acting on the punching shear critical section due to direct shear and a portion of the unbalanced moment
f_y	Yield stress of unstressed steel
h	Total member depth
I	Moment of inertia
J_c	"Polar" moment of inertia of the critical punching shear section about a horizontal centroidal axis perpendicular to the plane of the equivalent frame
L	Beam span between support centerlines
M_2	Secondary moment
M_{bal}	Balanced or equivalent load moment
M_{cr}	Moment in excess of the unfactored dead load moment which produces an extreme fiber tensile stress of $6\sqrt{f'_c}$ (used in beam shear calculations for V_{ci})
M_{design}	$M_u + M_2$ (the demand moment)
M_{DL}	Unfactored dead load moment
M_{equiv}	Moment which equilibrates the tendon balanced, or equivalent, loads only (not including the reactions to those loads, which are called the secondary reactions)

M_f	Portion of the total unbalanced moment M_u at a joint which is transferred by direct flexure between slab and column
M_{LL}	Unfactored live load moment
M_{max}	$M_u - M_{DL}$
M_n	Nominal moment capacity (without ϕ factor)
ϕM_n	Usable moment capacity
M_{net}	$M_{TL} + M_{bal}$
M_{TL}	$M_{DL} + M_{LL}$
M_u	Applied moment caused by factored dead and live loads
M_v	Portion of the total unbalanced moment M_u at a joint which must be transferred by eccentric shear stresses on the critical punching shear section
NA	Neutral axis
R_2	Secondary reaction
S	Section modulus
S_b	Section modulus at the bottom beam fiber
S_t	Section modulus at the top beam fiber
s	Stirrup spacing measured along length of beam
T	Total tension force acting on the freebody cross-section at nominal strength (= $T_p + T_s = C$)
T_p	$A_{ps}\,f_{ps}$, tensile force in prestressing steel at nominal member strength (the ultimate prestress force)
T'	Total tension force acting on the freebody cross-section under service load (= $A_{ps}\,f_{se} = C'$)
T_s	$A_s\,f_y$, tensile force in unstressed tension steel (normally rebar) at nominal member strength (the ultimate rebar tensile force)
t	Slab thickness
V_c	Controlling nominal concrete shear strength (determined from V_{cn}, V_{ci}, V_{cw})
V_{ci}	Nominal shear strength for "inclined cracking" type of shear failure
V_{cn}	Nominal concrete shear strength. Can be used for V_c in lieu of V_{ci} or V_{cw}
V_{cw}	Nominal shear strength for "web cracking" type of shear failure
V_d	Unfactored dead load shear
V_i	$V_u - V_{dl}$
V_l	Unfactored live load shear
V_n	Nominal shear capacity $V_c + V_s$ (without ϕ factor)
ϕV_n	Usable shear capacity
V_p	Vertical component of prestress force (the shear "carried" by the tendons)
V_s	Nominal shear strength of shear reinforcement (stirrups)
V_{TL}	$V_{DL} + V_{LL}$
V_u	Applied factored total load shear (the demand shear)
v_c	Allowable combined shear stress acting on the critical punching shear section
w_{bal}, w_p	Tendon balanced, or equivalent, load
w_{DL}	Unfactored dead load
w_{LL}	Unfactored live load
w_{net}	$w_{TL} + w_{bal}$
w_{TL}	$w_{DL} + w_{LL}$
w_u	Factored dead plus live load
$X_{L,R}$	Distance from the *centroid of the critical punching shear section* to its left and right faces
Y_L	Distance from datum line to CGS at left end beam highpoint
Y_M	Distance from datum line to CGS at a low point
Y_R	Distance from datum line to CGS at right end beam highpoint
y	Distance between resultant tension and compression forces acting on cross-section
y_t	Distance from concrete centroid to the extreme fiber *where tension is caused by applied gravity loads*
α_s	A term used in determining v_c, α_s = 40 for interior columns, 30 for edge or edge parallel columns, and 20 for corner columns
β_1	Factor that varies with concrete strength f'_c, b_1 is 0.85 for strengths up to and including 4000 psi, then reduces continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi down to a minimum of 0.65
β_c	Ratio of long side to short side of a rectangular column (β_c = 1 for round columns)
ε_c	Extreme fiber concrete compressive strain
ε_{pe}	Effective strain in prestressed reinforcement after all losses
ε_t	Net tensile strain in extreme tensile reinforcement
γ_p	A factor used in the calculation of f_{ps} for bonded tendons, 0.40 for stress-relieved steel, 0.28 for low-relaxation steel
γ_v	The decimal fraction of the total unbalanced moment at any joint of a two-way system which must be transferred from slab to column by eccentric shear stresses on the critical punching shear section
ϕ	Capacity reduction factor (0.9 for flexure, 0.75 for shear)
ρ_p	$A_{ps}/b'd_p$
ω	Reinforcing index $(T_p + T_s - C_s)/(b'\,d_p\,f'_c)$

DETAILING AND CONSTRUCTION PROCEDURES FOR BUILDINGS

6.1 GENERAL

The primary emphasis of this chapter is on detailing and construction procedures for buildings with unbonded tendons. Some of the material presented is also applicable to buildings with bonded tendons and other applications. These applications may require specialized considerations beyond the scope of this chapter. Additional information on construction of post-tensioned buildings is available in the PTI *Field Procedures Manual for Unbonded Single Strand Tendons.*[6.1]

6.2 DESIGN ISSUES

6.2.1 Information on Structural Drawings

The project construction documents typically include design drawings and specifications from all of the disciplines involved. The design drawings indicate the scope of the project and give sufficient detail for the contractor to estimate the work and produce detailed installation/shop drawings.

In general, the structural drawings show the geometry of all elements of the structure and the required non-prestressed as well as post-tensioned reinforcing. For post-tensioned slabs, the designer typically specifies the effective prestress force per linear foot in the slab and the tendon profile for each design strip. Total effective force and tendon profile is typically specified for post-tensioned beams. Fig. 6.1 shows a plan for a post-tensioned slab. In addition to the floor plan, the licensed design professional is also expected to provide sufficient detail on the drawings to illustrate the intent of the design. Section 6.4 shows some of the commonly used details and standard notes for post-tensioned construction. Post-tensioning fabricators frequently have experienced structural designers on staff who review the structural drawings and produce shop drawings that address construction and stressing sequence.

6.2.2 Floor Shortening and Restraint Cracking

If it is not adequately addressed in design and construction, floor shortening can be a source of distress in both the structural and non-structural elements. Restraint can occur due to the stiffness of the columns or other stiff lateral load resisting elements such as shear walls, foundation walls, and non-structural elements that are not temporarily or permanently isolated from the structure.

Restraint to floor shortening is a major source of cracking and distress in post-tensioned structures. Non-structural elements should be isolated from the structure by means of joints and physical separations. Joint and separation details should be clearly shown on the structural drawings and carefully inspected during all stages of construction. Building owners should be made aware that some movement is expected at joints over the life of the structure.

Structural elements that cannot be isolated from floor movements must be designed and detailed to absorb the movements. There are four factors that contribute to the shortening of cast-in-place post-tensioned floors:

- Elastic shortening due to pre-compression
- Creep shortening due to pre-compression
- Shrinkage of concrete
- Temperature variation

It is important for the designer to understand the effect of floor shortening on the various components of the structure and account for it in the design.

6.2.3 Measures to Mitigate Restraint Cracking

Floor shortening can cause cracking of both the floor slabs and the vertical supporting elements. Although it may be impossible to completely eliminate cracking, it can be significantly reduced by taking appropriate steps during design. Some techniques that can be used to mitigate restraint cracking are discussed in this section.

6.2.3.1 Planning Layout of Restraining Members

The most effective method of preventing restraint cracks is to ensure that columns and walls are correctly located. Stiff elements such as shear walls should be located at, or near, points of zero expected movement. Fig. 6.2 shows examples of both favorable and unfavorable wall arrangements. Cracks sometimes develop in the slab in the vicinity of shear walls, even if they are favorably located. Aalami et. al.[6.2,6.3] provide recommendations regarding additional non-prestressed reinforced that should be provided in order to control cracking. Fig. 6.16 shows a possible detail of this reinforcement.

6.2.3.2 Expansion Joints

Slabs of irregular plan geometry are particularly susceptible to cracking. Fig. 6.3 shows a small slab area appended to a larger rectangular region. An expansion joint can be added between the two slab sections to create a structural separation. Restraint forces are minimized because each slab is allowed to move independently towards its respective location of zero expected movement. If an expansion joint cannot be provided, the effect of potential shortening and movements between stiff elements must be investigated and additional reinforcing provided as necessary to minimize apparent cracking.

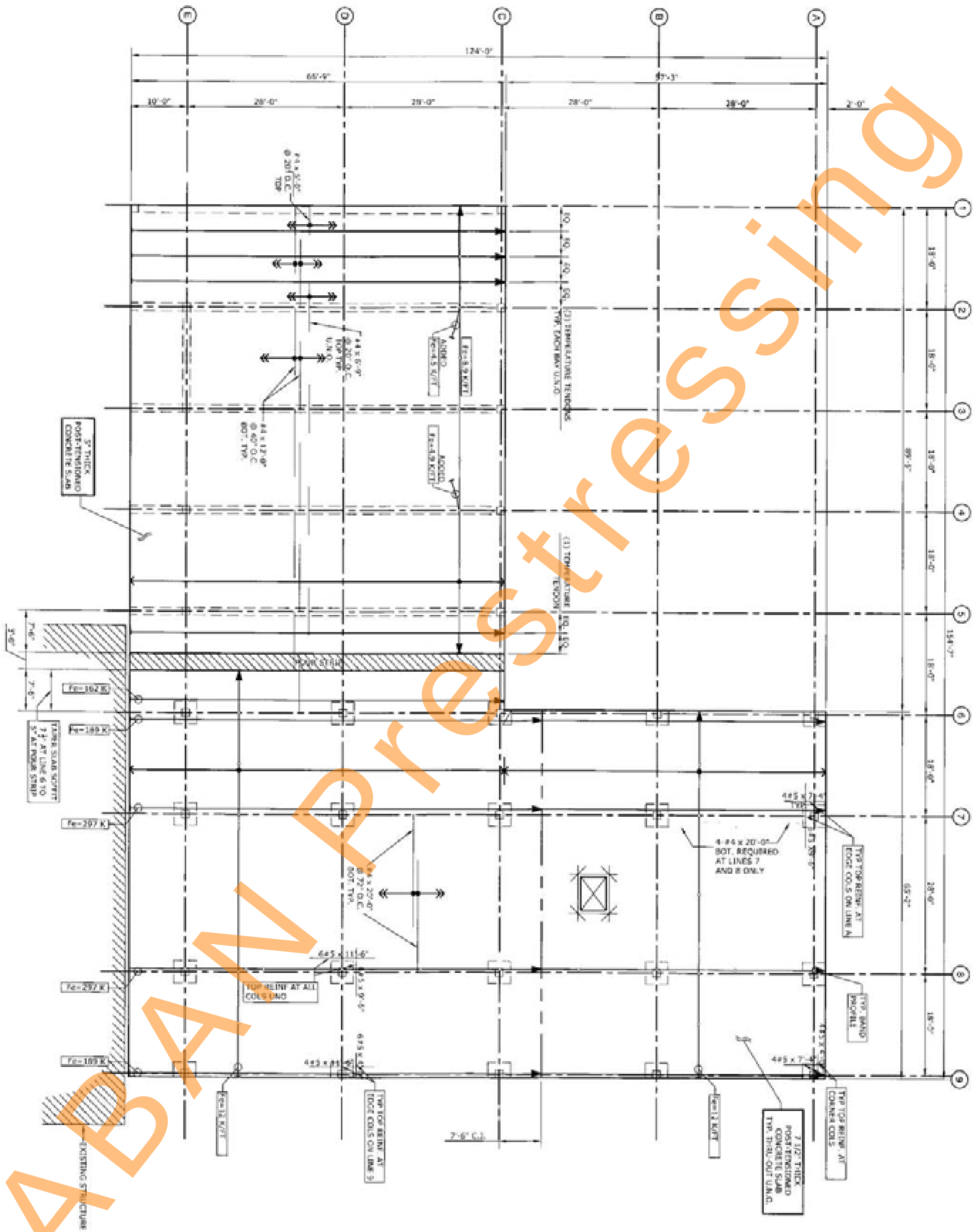


Fig. 6.1 Sample Framing Plan for One- and Two-Way Slab, Refer to Figs. 6.22 and 6.23 for Shop Drawings

6.2.3.3 Closure Strips

Closure strips are temporary slab separations that allow different sections of a slab to move independently until the closure strip concrete is placed and the adjacent slab sections are connected. In order to be effective in mitigating restraint cracks, closure strips must remain open long enough for a sufficient amount of the slab shortening to have occurred. Additional information on shortening calculations and detailing requirements are available in Section 2.8 in the reference *Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures.*[6.4] If the construction schedule or other considerations do not allow a closure strip to remain open for the required time, an expansion joint should be considered. Fig. 6.15 shows typical reinforcing for a closure strip.

6.2.3.4 Expansion Joint and Closure Strip Spacing Guidelines

The following general limitations on the lengths between closure strips and expansion joints are recommended, unless other details or methods are specified to mitigate cracking caused by restraint to shortening:

- If the slab length is less than 250 feet, no closure strip or expansion joint is necessary.
- For slab lengths between 250-325 feet, provide one centrally located closure strip.
- If the slab length is between 325 and 400 feet, consider using two closure strips open for at least 60 days.
- For slabs greater than 400 feet an expansion joint is recommended.

These guidelines may need to be modified for locations with significant temperature changes. In addition, it is assumed that the slab is regular in shape and stiff elements such as shear walls are favorably located near points of zero expected movement. As already discussed, plan location of stiff elements and geometry of the slab play an important role in the determination of expansion joint and closure strip location.

6.2.3.5 Special Movement Details

In some cases, details can be developed to allow some movement of the floor slab relative to its supporting elements. This helps to relieve the restraint forces and avoid cracking. These connection details need to be adequately designed and detailed on the structural drawings. Special care is required during construction to ensure that movement joints perform as designed. Fig. 6.4 shows some examples of movement details that have been used successfully.

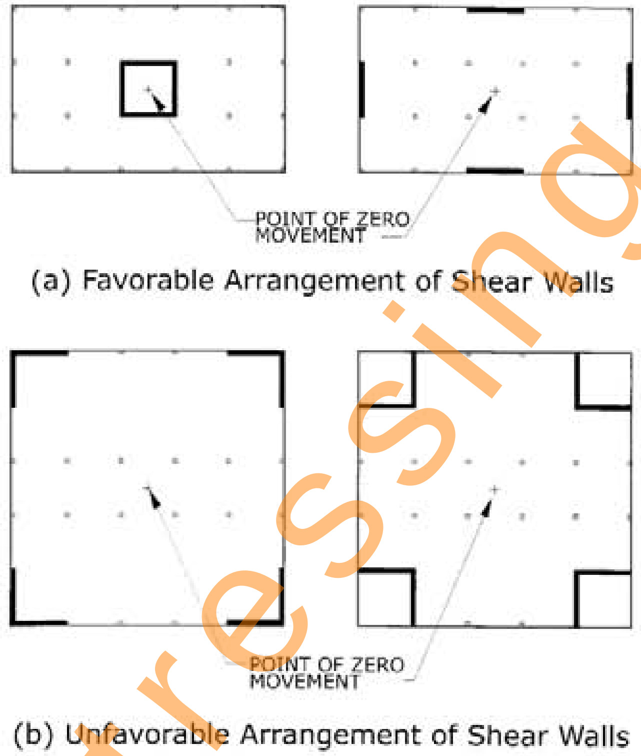


Fig. 6.2 Arrangement of Restraining Elements

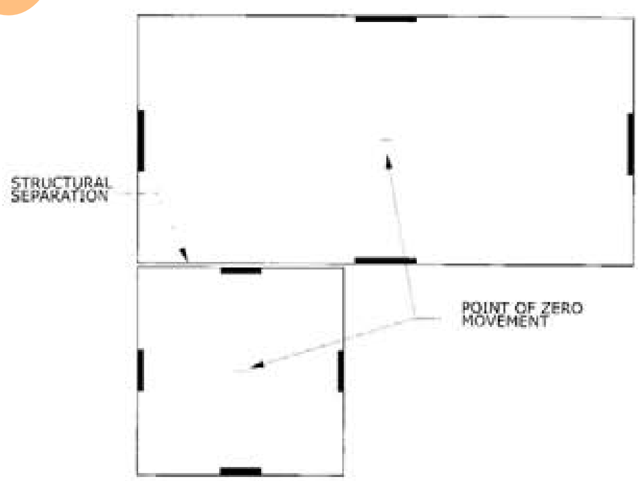
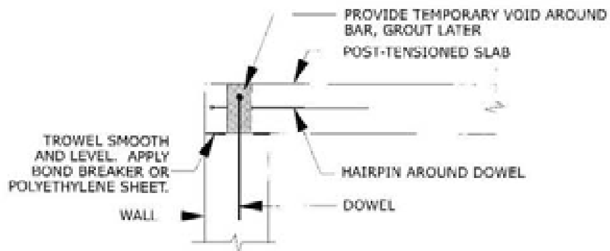
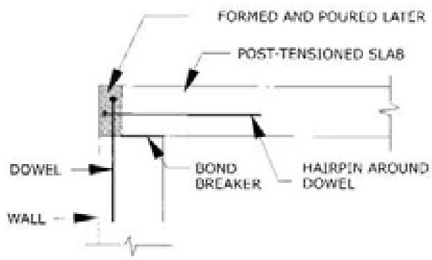


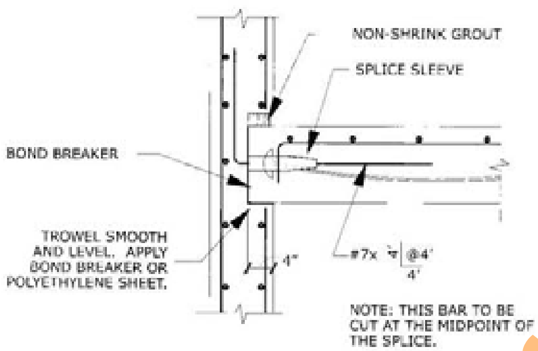
Fig. 6.3 Separation Between Areas Forming an Irregular Shape



(a) Wall-Slab Release Connection Detail (Option #1)



(b) Wall-Slab Release Connection Detail (Option #2)



(c) Connection of Post-Tensioned Slab To Wall With Splice Sleeves

Fig. 6.4 Temporary Release Details

6.3 CONSTRUCTION ISSUES

6.3.1 Tendon Layout

6.3.1.1 Overall Tendon Layout for Two-Way Slabs

The most prevalent tendon layout for two-way post-tensioned slabs consists of tendons located in a narrow band over columns in one direction and uniformly distributed tendons in the orthogonal direction. This layout offers ease of placement and faster overall construction as compared to the other layouts. Refer to Chapter 9 for further details. The sequence of construction involves placing at least two distributed tendons over the support, followed by banded tendons in the perpendicular direction. The remaining distributed tendons are then placed over the bands. This layout allows the placement of tendons without interweaving. Banded and distributed tendons generally do not cross at their high or low points, with the exception of distributed tendons over the supports. This layout also allows most of the strands to be placed with the maximum allowable drape and increases the efficiency of the framing system.

6.3.1.2 Tendon Profiles

Tendon profiles are generally parabolic for slabs and beams. In some cases where a beam transfers significant concentrated load, designers may choose harped tendons to allow for a more efficient load transfer to the supports.

6.3.1.3 Detailing of Tendon Layout

6.3.1.3.1 Minimum Tendons Over Supports

ACI 318-02 requires that a minimum of two tendons in each direction pass directly through the critical shear section over the columns. However, it is preferable to have the tendons pass through the column core if possible; this provides better integrity to the system and prevents possibility of progressive collapse.

6.3.1.3.2 Distributed Tendons

The minimum spacing of the tendons is typically not an issue in unbonded post-tensioned construction. If the tendons are too close, it is common practice to bundle the tendons together. This allows easy placement of tendons.

The maximum spacing between any two tendons or tendon bundles is limited to the lesser of 5 ft or eight times the thickness of the slab.

6.3.1.3.3 Bundling of Tendons

For ease of construction, it is common practice to bundle several unbonded tendons together. Five tendons per flat bundle is the maximum recommended for floor slab construction.

For beams, the strands are placed in a round or rectangular bundle. There is no code limitation on the number of unbonded strands that can be bundled in a beam. However, consideration should be given to the detailing of the anchorage zone and satisfactory consolidation of concrete below the bundle. Generally in beams the width of a tendon bundle should be limited to six tendons.

6.3.1.3.4 Curvature in Tendons

Tendons that are curved horizontally exert lateral forces that can produce concrete failure, particularly at slab openings. This risk can be minimized if tendon curvature is less than that shown in Fig. 6.14a. For greater curvatures (tighter radius of curvature), properly placed hairpins must be used as shown in Fig. 6.14b. (Note: the details shown in the figure are for common slab construction with ½ in. nominal diameter unbonded tendons.)

6.3.1.3.5 Tendon Supports

Tendon supports are used to position and secure tendons in their designated profile. The supports are generally #4 reinforcing bars. For tendon heights (center of gravity) greater than 1.25 in. above the soffit, the support bars are secured on chairs typically spaced at 4 ft on center. For tendon heights of 1.25 in. or less, slab bolsters are used. The spacing of support bars depends upon the designated profile and the type of tendon, but usually does not exceed 4 ft. In bonded post-tensioning with larger stiffer ducts it may be possible to increase the spacing of the tendon supports. The recommendations of the post-tensioning supplier should be followed in this case. Using a continuous #4 bar at each support point for uniformly spaced tendons in a flat slab and providing a minimum lap of 24 in. at splice points in the support will tend to limit shrinkage cracks that may form between the time the slab concrete is placed and the tendons are stressed.

6.3.1.3.6 Tolerance in Tendon Profile

Tolerances for deviations from the tendon profiles and layouts shown on the placing diagrams are typically specified as follows:

- In the vertical (normal to the plane of the slab) direction, ¼ in. from the specified vertical profile for members up to 8 in. thick; ⅜ in. for members between 8 in. and 24 in.; and ½ in. for members over 24 in. thick. It is noted that these post-tensioning tolerances are generally more restrictive than rebar placement tolerances required by ACI 318-02.[6.5] It should be noted that even these tolerances at their maximum limit could result in a decrease of up to 20% in the design balanced load produced by the post-tensioning tendons.
- In the horizontal direction (plane of the slab) tolerances should be as indicated above in the section "Curvature in Tendons."

6.3.1.4 Detailing Around Openings

Tendons should be properly detailed around small or medium sized openings. A minimum clearance of 6 in. should be maintained around all blockouts; sharp bends and transitions should be avoided. For larger openings, it is desirable to reinforce the top and bottom of the slab at the opening with diagonal bars to control cracking initiated at the corners of the openings. In some cases, additional structural reinforcing may be required around the slab perimeter to distribute the forces around the opening. Additional hairpin reinforcement should be considered where there is a significant horizontal change in profile of the tendon. Fig. 6.14 shows typical tendon layout details around openings.

6.3.2 Detailing of Anchorage Zones

Large concentrated forces are introduced at tendon anchorages. Additional reinforcement may be required to resist the bursting forces in these areas. ACI 318-02 gives detailed guidelines for determining the allowable forces in these regions and general procedures for the design of anchorage zones using the strut-and-tie approach. Detailed procedures for the design of post-tensioning anchorage zones are also given in the PTI Design Guide *Anchorage Zone Design*.[6.6] Research by Sanders et. al.[6.7] has shown that anchorage zones in typical slabs which have groups of four or more ½ in. diameter single strand unbonded tendons with horizontal anchors spacing of 12 in. or less can be reinforced according to Fig. 6.7 or with a similar detail using closed stirrups or headed studs. Similar reinforcement should be provided for anchorages located within 12 in. of slab corners.

6.3.2.1 Anchorage for Added Tendons

Additional tendons are frequently used to increase the post-tensioning force in the exterior bays, long interior spans or non-uniform loading patterns. Bonded reinforcement capable of transferring a portion of the factored jacking force into the concrete behind the anchors is required at these locations. This reinforcement is intended to control cracking which might develop due to localized tensile stresses generated as a result of compatibility requirements for deformation ahead of, and behind the anchors. Some design professionals specify that intermediate anchors be staggered at least 12 in. in the direction of the tendon to further reduce the potential of cracking.

6.3.3 Joints

The maximum slab length between construction joints is generally limited to 100-150 ft to minimize the effects of slab shortening, and to avoid excessive loss of prestress due to friction. When special consideration is given to reducing the effects of axial shortening on both the slab system and the substructure elements, longer distances between construction joints may be used. In such cases, the structure may be segmented with closure strips or temporary stressing joints to minimize the movement and restraint during post-tensioning due to early volume changes. Section 6.2.2 gives a detailed description of some of the techniques that can be used to reduce restraint cracking. Tendons should typically be stressed from both ends when their length exceeds 100 ft.

6.4 GENERAL NOTES/STANDARD DETAILS

6.4.1 General Notes

This section is intended as an example of the General Notes that might be found on the structural drawings for an unbonded post-tensioned concrete parking or building structure. The notes are limited to the post-tensioning activity and are neither complete nor applicable for every project. In some instances, they may conflict with accepted practice in certain areas of the country and may be inappropriate for certain types of construction. Some of this information may also be contained in the specifications.

The notes are grouped into seven sections: General, Material, Installation, Concrete Pour, Stressing, Inspection and Allowances.

GENERAL

1. **Material, Installation, Stressing and Finishing Specifications**: The tendon shall meet the requirements set forth in the *Specifications for Unbonded Single Strand Tendons* (Latest Edition) published by the Post-Tensioning Institute (PTI).
2. **Marking of Tendon Location**: If desired by the owner, the horizontal position of slab tendons shall be marked on the soffit of the slab. This may be accomplished by attaching markers to the slab formwork or by spray-painting the formwork along the tendon path just prior to placement of concrete. The paint marks will transfer to the concrete soffit to permanently mark the tendon locations and can be hidden by suspended ceiling systems.
3. **Embeds**: Where permanent fixtures such as curtain wall systems, handrails, fire protection equipment, lights, and security devices must be connected to post-tensioned slabs or beams, they shall be attached using embeds. Drilled anchors shall not be allowed on post-tensioned slabs unless there is written authorization from the Engineer of record. No dissimilar metals shall be in contact.
4. **Fasteners and Inserts**: Fasteners or inserts shall not be shot or drilled into post-tensioned slabs after the concrete is placed unless there is written authorization from the Engineer of record. Drilled or power-driven fasteners will only be permitted if they are located to avoid the tendons and anchorages and it can be shown that they will not spall the concrete. Special attention shall be given to braces for column and wall forms to ensure that they are located between tendon groups.
5. **Openings**: Opening, penetration and insert locations shall be determined to the fullest extent possible prior to tendon layout. No changes shall be made in the field without prior written approval of the Engineer of record. Core drilling is not allowed without prior written approval of the Engineer of record.
6. **Formwork**: For multi-level structures, the formwork shall extend beyond the slab edge or scaffolding shall be provided to allow adequate room for the stressing operation.
7. **Installation Drawings**: The Contractor shall submit installation drawings to the Engineer of record a minimum of four weeks prior to scheduled installation of the tendons. Installation drawings shall show the tendon layout, all anchorage locations, and tendon supports with all details necessary to ensure proper installation. The review of installation drawings by the Engineer of record is only for general compliance with the intent of the structural drawings and specifications.
8. **Field Placement Review**: The tendon, mild steel and slab embed installation shall be reviewed by the Engineer of record or the Engineer of record's designated representative prior to the concrete pour. The Engineer of record shall be notified at least five days in advance of the date all placements of tendons, reinforcing steel and slab embeds will be completed.
9. **Field Foreman**: The Field Foreman responsible for the placement, stressing and finishing of all post-tensioning material (including all mild reinforcement necessary for proper installation of the post-tensioning tendons) shall be PTI Certified with a minimum of five years experience in this capacity for this type of construction.

MATERIALS

1. **Strand Quality**: Post-tensioning tendons shall use low-relaxation strand conforming to the following:

Seven-wire strand ASTM designation	A 416
Minimum Ultimate stress	270 ksi
½ in. diameter strand area	0.153 in^2

   One sample of each reel or heat shall be tested by an approved laboratory. Mill certificates may be submitted in lieu of independent testing if approved by the Engineer of record. Test results or mill certificates shall be submitted to the Engineer of record before any tendons are fabricated or installed.
2. **P/T Hardware Quality**: All anchorages, couplers, and miscellaneous hardware shall be approved by governing agencies and the Engineer of record.
3. **Sheathing**: Unbonded tendons shall be encased in a slippage sheathing which shall be manufactured by a process that provides watertight encasement of the corrosion inhibiting coating material (P/T coating) so as to prevent the internal migration of any water. Sheathing shall be of sufficient strength and durability to resist damage during normal fabrication, trans-

portation, installation, and concrete placement operations. Minimum sheathing thickness shall be 0.050 in. Tears in the sheathing shall be repaired by replacing the P/T coating and restoring the watertightness. Tendons shall be protected during shipping and handling to avoid damage to the tendon sheathing during transportation and offloading at the jobsite, and avoid exposure to deicing salts or any other form of corrosive element.

4. **Concrete**: Strength and material data for all concrete mixes to be used on the project shall be provided to the Engineer of record as early in the project as possible with a minimum of one week prior to their use.

INSTALLATION

1. **Installation of Unbonded Tendons**: If the post-tensioning supplier does not install the post-tensioning material, detailed instructions for the installation and stressing of the tendons shall be furnished to the installer. The contractor responsible for hiring the independent post-tensioning installer shall ensure that the installation crew meets the standards set forth by PTI Certification Programs. The post-tensioning material supplier shall provide initial jobsite technical assistance to instruct the installer on any special requirements of their system to ensure proper installation, stressing and finishing of all post-tensioning material.
2. **Tendons**: Tendons shall be shop-fabricated with preassembled fixed-end anchorages. Plastic pocket formers shall be used at all stressing-ends to recess the anchor castings so that the required cover is achieved.
3. **Tendon Placement**: Care shall be taken that tendons are located and held in their designated positions as shown on the approved installation drawings. Except as noted in the construction documents or approved by the Engineer of record, tolerances for the vertical location of the prestressing steel shall not be more than ±¼ in. for slab thickness less than 8 in., ⅜ in. for concrete with dimensions more than 8 in. but not more than 2 ft, and ½ in. for concrete with dimensions more than 2 ft. Access shall be provided to stressing-ends.
4. **Tendon Groups**: Tendons in beams shall be grouped to provide adequate clearance to mild reinforcing and facilitate concrete placement. A maximum of six tendons is allowed per group unless noted otherwise on the approved installation drawings.
5. **Tendons Over Columns**: In two-way slab construction, a minimum of two tendons shall be placed directly over the supporting column (within the column cage), in each orthogonal direction. (Note: ACI 318-02 allows the tendons to pass outside the columns as long as they are within the critical shear section, however, it is preferable to have the tendons pass through the column core if possible, as this provides integrity to the system and prevents possibility of progressive collapse.)
6. **Tendon Adjustments**: Small deviations in the horizontal spacing of the slab tendons will be permitted when required to avoid openings, inserts, and dowels with specific location requirements. Where tendon locations interfere with each other, one tendon may be moved horizontally in order to avoid the interference.
7. **Twisting**: Twisting or entwining of individual tendons within a group shall not be permitted. Entwining of groups within a beam shall not be permitted.
8. **Vertical Profiles**: Profiles shall conform to control points shown on the drawings and shall have an approximate parabolic drape between supports unless noted otherwise. Low points shall be at midspan unless noted otherwise. Harped tendons shall be straight between high and low points.
9. **Horizontal Profiles**: If tendons must be curved horizontally to avoid openings or other obstructions, tendon groups shall be flared such that a minimum of two inches of separation is maintained between each individual tendon. Tendons shall be flared a maximum of 1:6. If tendons are flared at more than 1:12, #3 hairpins at 12 in. on center shall be used to transfer the horizontal radial force to the concrete, unless noted otherwise on the approved installation drawings.
10. **Tendon Cover**: All dimensions showing the location of prestressing tendons are to the center of gravity of the tendon (cgs) unless noted otherwise. A minimum of ____in. concrete cover at the top of the slab and____ in. concrete cover at the bottom of the slab is required. (*Cover to be specified by the licensed design professional; will vary from project to project.*)
11. **Minimum Chairing for Slab Tendons**: Tendons shall be supported on reinforcing bars spaced at a maximum of 4 ft on center and secured to the support bar at each tendon/support bar intersection to ensure that the correct vertical and horizontal location is maintained during the placing of the concrete. Supports shall be installed to prevent excessive movement during placement of concrete.
12. **Support Bars**: Support bars shall be minimum #4. Support bars spanning across the capital or drop panel shall be #6 or greater and generally placed parallel to the banded tendons and below the lower layer of top reinforcement (parallel to uniformly spaced tendons).
13. **Anchorages**: Anchorage devices shall be recessed a minimum of 2 inches. Two continuous #4 backup bars shall be placed behind all anchorages unless otherwise noted.

14. **Blockouts/Pockets**: All blockouts or pockets required for access to anchorages in beams or slabs shall be adequately reinforced so as not to decrease the strength of the structure. All blockouts and pockets should be sealed in such a manner as to eliminate water leakage through or into the blockout or pocket. Location of all blockouts and pockets shall be approved by the Engineer of record.

15. **Pipes**: Plastic or metal conduits may be embedded in the slab providing the following criteria are met:

    a. The outside diameter of the conduit does not exceed ¼ of the slab thickness or 2 in., whichever is less.

    b. Conduits with outside diameter greater than or equal to 1 in. are located within the middle third of the slab.

    c. Conduits with outside diameter smaller than 1 in. may be located anywhere within the slab as long as the minimum cover requirements are observed.

    d. Center-to-center spacing of the conduits is not less than three times the diameter of the largest conduit or 6 in., whichever is greater, with no more than three conduits per six-foot width of slab.

    e. Conduits do not contact, interrupt or displace the post-tensioned tendons or the mild reinforcing.

    f. No conduit may be placed within a distance equal to the largest column dimension from the face of the column, unless approved by the Engineer of record.

    g. No conduits may be placed with a distance equal to the depth of slab from the edge of the drop, unless approved by the Engineer of record.

    It is undesirable to have a large number of conduits entering the slab at one location. If this occurs, the conduits must be fanned out immediately. Additional mild reinforcement shall be added top and bottom until Item d above is met.

16. **Penetrations**: Penetrations shall not be permitted in beams or drop caps unless shown on post-tensioning drawings or typical details and approved by the Engineer of record.

CONCRETE PLACEMENT

1. **Concrete Placement**: Prior to placement of the concrete, forms shall be cleaned of all debris and dirt. When concrete is placed in post-tensioned slabs, special care shall be taken at column drop caps and drop panels. The pump hose shall be inserted into the column drop panel below the reinforcement and filled until the concrete reaches the top reinforcing layer. Concrete elevation shall be monitored to avoid floatation of the top reinforcing. After the drop panel is full of concrete, concrete shall be placed over the top reinforcing layer to specified slab thickness. Concrete shall be adequately vibrated in and around column drop panels.
2. **Pumped Concrete**: If concrete is placed with pumps, hoses shall not be allowed to rest on the reinforcing (tendons or rebar).
3. **Concrete Consolidation**: The Contractor shall take precautions to ensure complete consolidation and densification of concrete behind all post-tensioning anchorages. Care shall be taken not to allow the vibrator to contact the reinforcing or the post-tensioning tendons.
4. **Chlorides:** Maximum water soluble chloride ion in concrete shall not exceed 0.06 percent by weight of cement.

STRESSING OF TENDONS

1. **Tendon stresses shall conform to the following:**

    a. Maximum tendon jacking stress — 0.94 f_{py} < 0.80 f_{pu}

    b. Maximum tendon stress immediately after prestress transfer — 0.82 f_{py} < 0.74 f_{pu}

    c. Maximum tendon stress at anchors and couplers after anchorage-set — 0.7 f_{pu}

2. **Effective Force**: Forces shown on structural drawings are effective forces after all short and long term losses. The post-tensioning supplier shall provide friction and long-term loss calculations for the Engineer of record's review.
3. **Concrete Strength at Stressing**: Concrete shall reach a minimum compressive strength of f'_c = 3000 psi prior to stressing. Minimum concrete strength shall be established by breaking concrete test cylinders cured at the job site under conditions similar to the curing of the post-tensioned elements. Stressing shall not commence until the concrete reaches the specified strength. Tendons should be stressed within 72 hours after the concrete is placed to minimize early age concrete shrinkage cracking. This may not apply to elements that are stage stressed where only a portion of the total post-tensioning forces is applied within the 72 hours.
4. **Calibration**: Each stressing jack and gauge combination shall be individually identified and calibrated as a unit to known standards at intervals not exceeding 6 months. With a written approval of Engineer of record, it may be permissible to calibrate the gauges to a master gauge of known accuracy, provided the jacks are calibrated to the same master gauge. Copies of the calibration certificates for each jack and gauge combination being used shall be submitted to the Engineer of record for review and reviewed copies kept at the job site and shall be available upon request.
5. **Tendon Stressing**: The stressing operation shall be under the immediate control of a person who is a PTI Certified Installer experienced in this type of work. Continuous inspection and recording of elongations and stressing equipment gauge pressures by an inde-

pendent inspector hired by the owner is required during all stressing operations. The independent inspector shall be certified under the PTI Certified Installer program.

6. **Stressing Sequence**: To prevent overloading of the forming system during the stressing operation, the type of formwork system being used shall be considered when determining the appropriate stressing sequence. In general, uniformly distributed tendons shall be stressed before banded tendons in two-way slab construction. Slab tendons shall be stressed before beam tendons in one-way slab construction. Additional stressing sequence requirements shall be as specified below. Special consideration shall be given to the stressing sequence of transfer girders.

   (Insert one of the following as appropriate)

   Two-Way Slab Sequence

   1. Stress continuous distributed tendons
   2. Stress continuous banded tendon
   3. Stress added distributed tendons
   4. Stress added band tendons

   One-Way Slab and Beam Sequence

   1. Stress temperature tendons
   2. Stress continuous uniform slab tendons
   3. Stress beam tendons

7. **Elongations**: Field readings of elongations and/or stressing forces shall not vary by more than ±7% from calculated required values shown on the installation drawings. If the measured elongations vary from calculated values by more than ±7%, stressing operation shall be suspended until the cause of the variation from specified elongation is determined and corrected to the satisfaction of the Engineer of record. The elongation reports should be submitted the same day the stressing operation is completed and the elongation report should be approved or rejected within 96 hours after stressing.

8. **Member Forces**: The post-tensioning force provided in the field for each structural member shall not be less than the requirements shown on the structural drawings. In this context, structural members are beams or slabs, each serving their respective tributary area.

9. **Tendon Ends**: Tendon ends within a section of the project shall not be cut until all post-tensioning tendons in that section have been satisfactorily stressed and the Engineer of record's approval is obtained. If an encapsulated system is specified, the tendon ends shall be protected with a grease-filled cap within one day of cutting off the tendon ends. Connection of the cap to the anchorage shall be watertight.

10. **Grouting of Anchorage Pockets**: To minimize moisture access to the tendons, anchorage pockets shall be filled with non-shrink grout as soon as practical after stressing. Grout containing chlorides shall not be used.

11. **Shoring**: Unless full shoring is required to carry the floors above, the shoring supporting slabs and beams may be stripped when all tendons have been satisfactorily stressed and the Engineer of record's approval is obtained. Re-shore in accordance with the approved shoring plan. Shoring requirements for stage-stressed transfer girders shall be as noted on the drawings.

    In areas supporting a partial span such as near a pour strip or construction joint, the shoring in the partial span shall stay in place until the remaining section of span has been poured and stressed. In some cases, the immediate back span may also need to remain shored until the adjacent span is completed. If this is required, it shall be specified on the construction documents and the post-tensioning installation drawings.

INSPECTION FOR PRESTRESSING STEEL

Continuous special inspection shall be provided during the placing of reinforcing steel and post-tensioning tendons for all structural concrete. Tendon placement and integrity shall be inspected prior to placement of concrete. During all stressing of post-tensioned concrete, the special inspection shall include recording the field-measured elongation and jacking force for each tendon. For good quality control, independent verification to tendon tails, end cap installation for encapsulated systems and grouting of anchorage pockets is recommended.

1. **Admixtures**: No admixtures shall be added to the concrete mix without the approval of the Engineer of record. Admixtures or concrete containing chlorides shall not be used in post-tensioned slabs.

MATERIAL ALLOWANCES

1. **Reinforcing Steel Allowance**: The Contractor shall provide ___ pounds of reinforcing steel for the Engineer to use at the Engineer's discretion during construction.
2. **Post-Tensioning Allowance**: The Contractor shall provide ___ pounds (ASTM weight) of post-tensioning steel for the Engineer to use at the Engineer's discretion during construction.
3. **Crack Repair Allowance**: The Contractor shall provide for ___ ft of crack repair during and after construction.

Allowance to be specified by the licensed design professional; will vary from project to project.

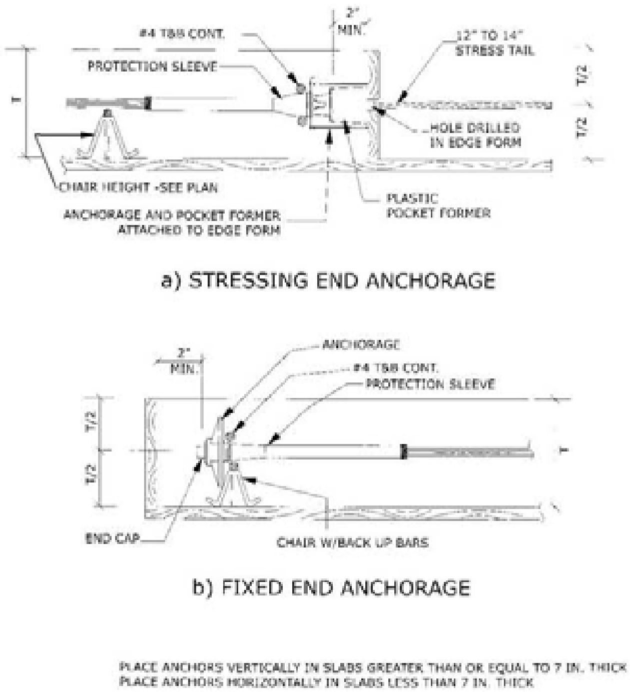


Fig. 6.5 Encapsulated System of Anchorages

6.4.2 Standard Details

Many specialized details have been developed for post-tensioned concrete structures. The recommended procedure is for the licensed design professional to show all of the details related to the post-tensioning layout and supports, the anchorage zone reinforcing, and the supplemental reinforcing on the structural drawings.

The following selected details illustrate typical design and detailing practices for unbonded post-tensioning in aggressive environments. The licensed design professional should carefully review the project requirements to determine the appropriateness of a particular detail.

6.4.2.1 Tendon Anchorage Zone

The anchorage zone, one of the most critical concrete regions, resists the bearing and tension forces of unbonded tendons during the service life of the post-tensioned structure.

Fig. 6.5 shows typical details of stressing and fixed-ends for an encapsulated system. These details apply to uniformly spaced tendons and temperature tendons in one-way slab applications.

Fig. 6.6 shows typical details for the stressing and fixed-ends of banded tendons. Unless otherwise called out on the structural contract drawings or the post-tensioning installation drawings, anchorage zones for groups of six or more, ½ in. diameter single strand tendons with an anchor spac-

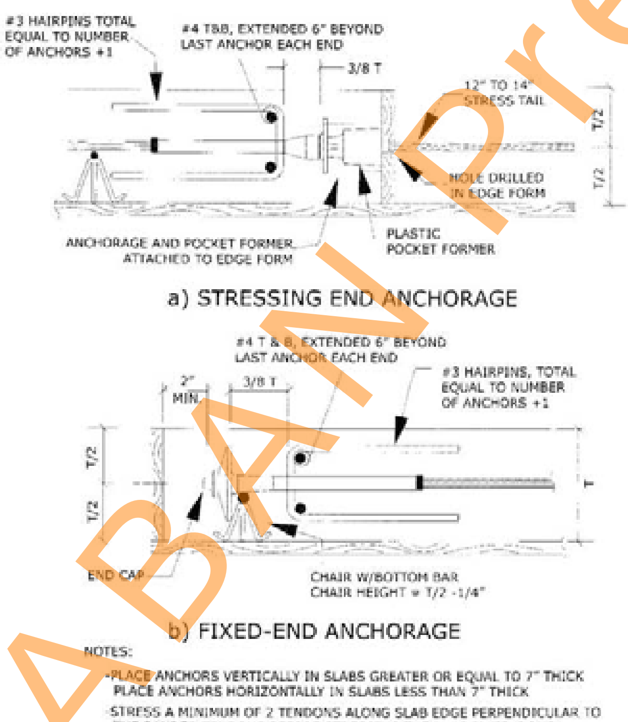


Fig. 6.6 Banded Tendon Anchorage

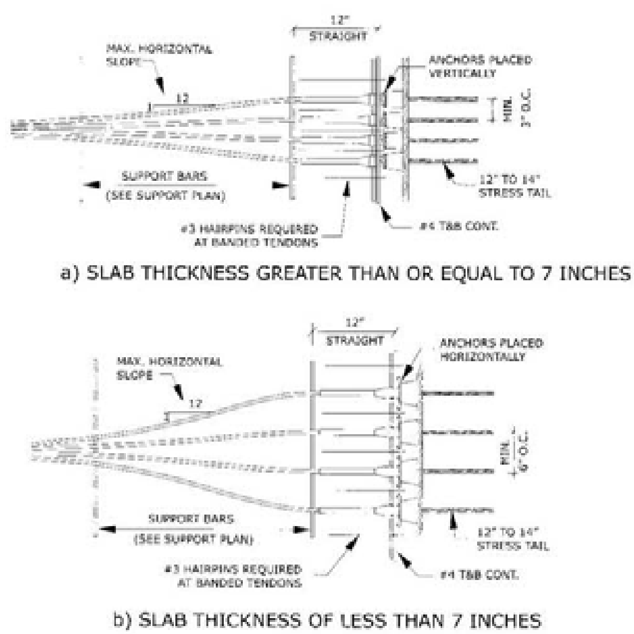


Fig. 6.7 Horizontal Flare of Tendons at the Anchorage Zone

ing of 12 in. or less should be reinforced with hairpins as shown on Fig. 6.6. The hairpins should be a minimum of 9 in. long; many designers specify longer hairpins so that all the hairpins used on the project are the same length. Additional #4 bars are typically used to secure the hairpins in place; the number of bars required will depend on the length of the hairpins.

Fig. 6.7 shows the horizontal flaring of tendons at the anchorage zone.

Fig. 6.8 shows reinforcement for anchorage zone penetrations.

6.4.2.2 Tendons Over Column Supports for Two-Way Slabs

The mild-steel reinforcing arrangement and the layout of the banded and uniform tendons over column supports must be detailed well enough for the field personnel to understand the tendon layout, the layers of top reinforcement and the support system. Figs. 6.9 and 6.10 show typical two-way slab tendon layouts at interior and exterior column supports. At interior columns, ACI 318-02 requires that a minimum of two tendons in each direction pass directly through the critical shear section over the columns. However, it is preferable to have the tendons pass through the column core; this provides better integrity to the system and prevents the possibility of progressive collapse. Tendons in both directions have essentially the same eccentricity at the support.

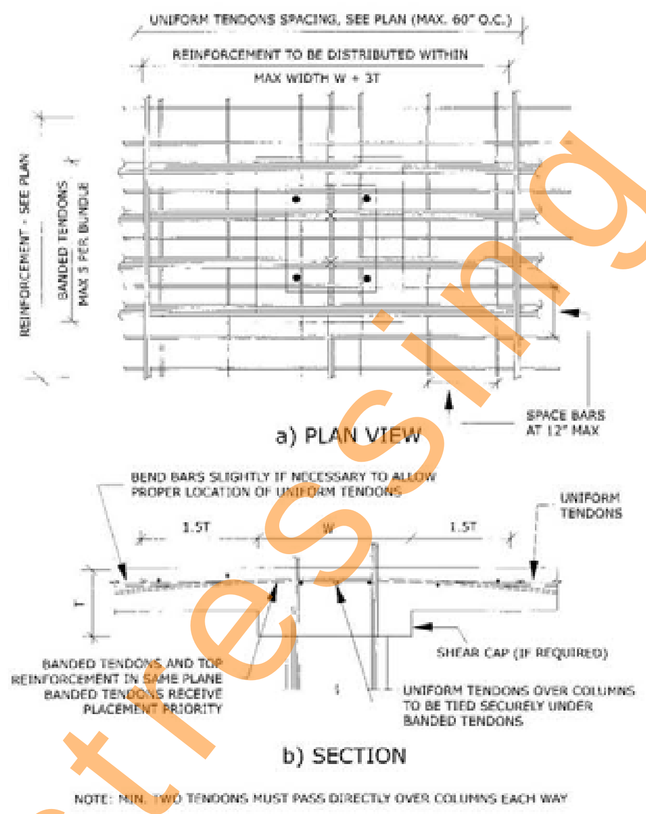


Fig. 6.9 Two-Way Slab System, Interior Columns

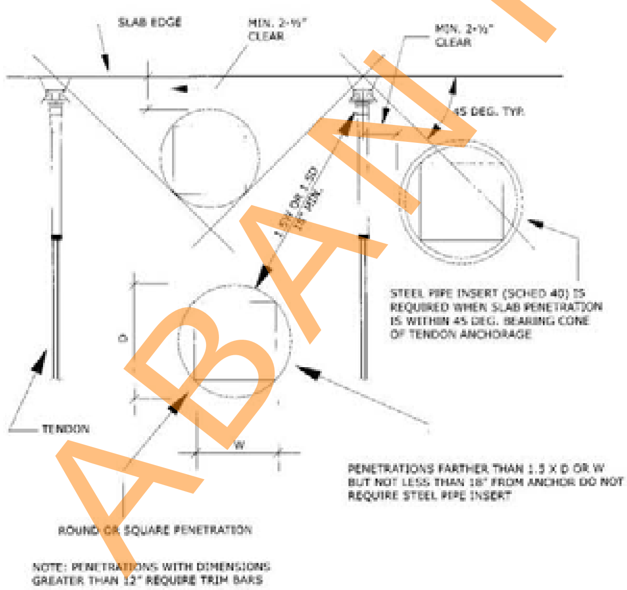


Fig. 6.8 Reinforcement at Anchorage Zone Penetrations

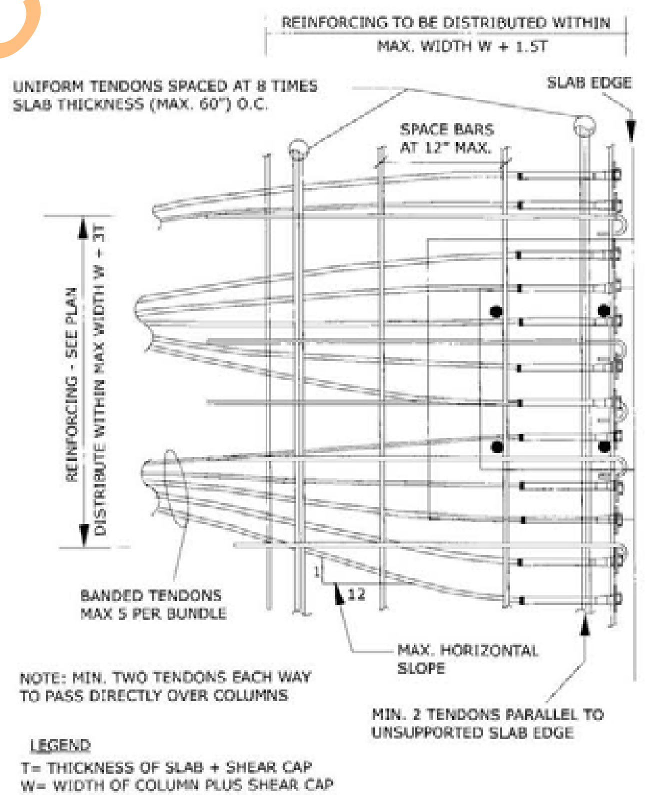


Fig. 6.10 Two-Way Slab System, Exterior Columns

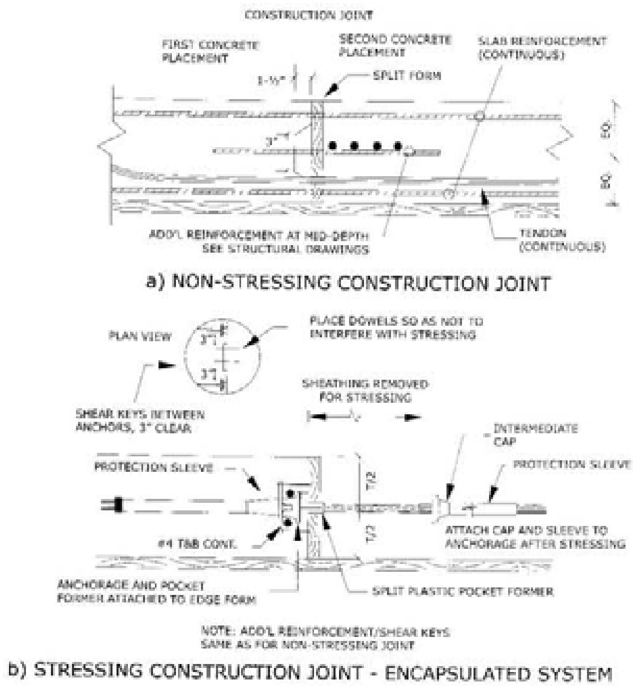


Fig. 6.11 Construction Joints

6.4.2.3 Joints, Stressing Blockouts, Added Tendons, Openings, and Closure Strips

Construction joints are used both to limit the size of the concrete placement and allow for intermediate stressing of long tendons. All construction joint locations must be approved by the Engineer of record. Fig. 6.11 shows typical details for both non-stressing and stressing construction joints.

Blockouts are also sometimes used to provide access for stressing. All blockout locations must be approved by the Engineer of record. Fig. 6.12 shows typical reinforcing for a stressing blockout. The overlapping bars extending in from either side of the blockout would be bent up out of the way of the equipment during stressing. Bar lengths should be such that there is a 2 ft minimum lap length. Note that for slabs greater than ten inches thick, the bars shown can be continuous through the blockout since they would not interfere with the stressing jack.

Partial length (added) tendons that terminate at the interior of the structure must be profiled so that both the fixed and stressing ends are at mid-depth of the slab. When more than two tendons terminate at a given location, the fixed-ends should be staggered to minimize potential for cracking the slab. Fig. 6.13 shows details for the fixed-end of added tendons.

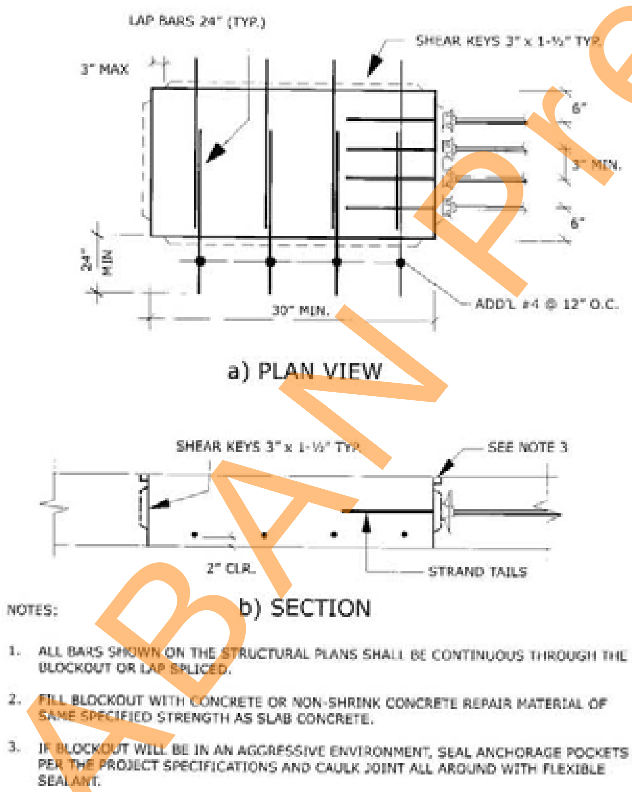


Fig. 6.12 Stressing Blockout

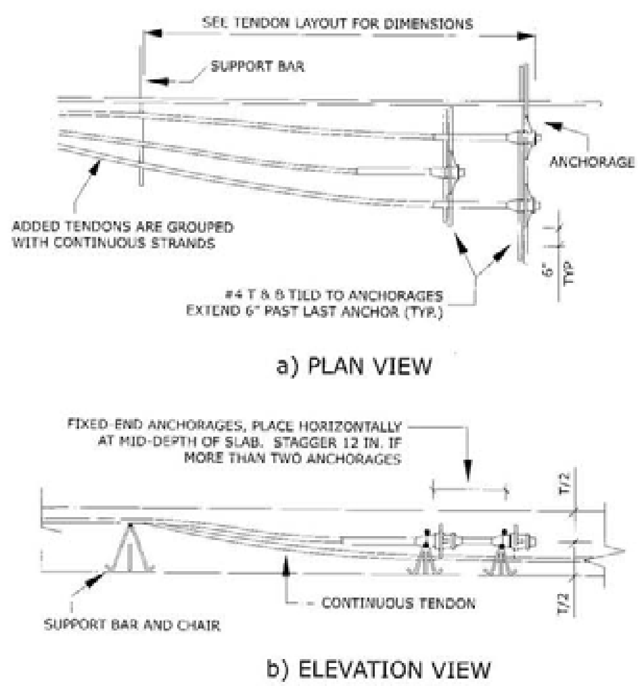


Fig. 6.13 Added Tendons

Fig. 6.14(a) shows the placement of tendons around openings. Fig. 6.14(b) shows typical hairpin reinforcement when the horizontal slope of the tendons is greater than 1:12. If there are several tendon bands, the hairpins should be staggered so that each band is enclosed within the turn of the hairpin.

Closure strips may be used to reduce the restraint effects on post-tensioned members. They are also used to improve constructability and provide access for stressing. The closure strip location and the length of time that the closure strip must remain open should be determined by the Engineer of record, based on an evaluation of the structure's expected shortening. Closure strips used to provide access for stressing can usually be filled as soon as the tendons have been cut off and the anchorage pockets have been grouted. Fig. 6.15 is an example of the reinforcing layout for a typical closure strip.

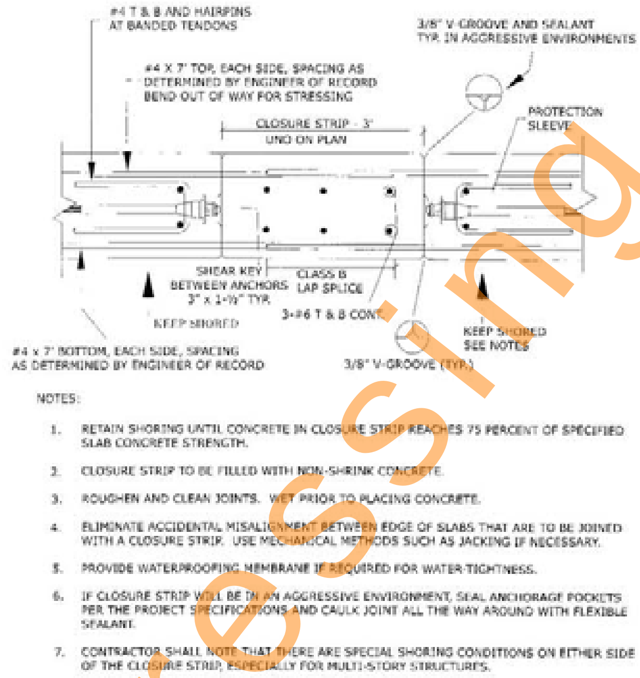


Fig. 6.15 Typical Closure Strip Detail

6.4.2.4 Added Reinforcing Steel at Walls

If not properly detailed, restraint from stiff elements such as shear walls may cause slab cracking. Fig. 6.16 shows recommendations for added reinforcing steel in the slab at interior and exterior walls.

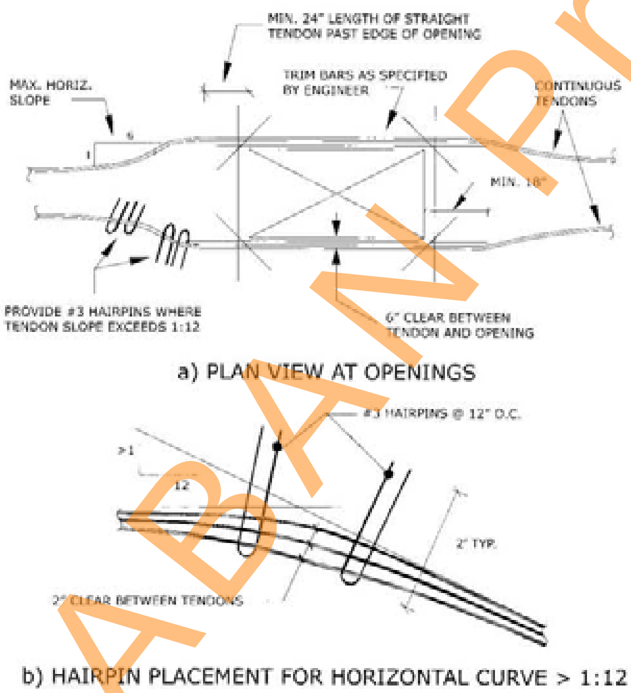


Fig. 6.14 Tendon Layout at Openings

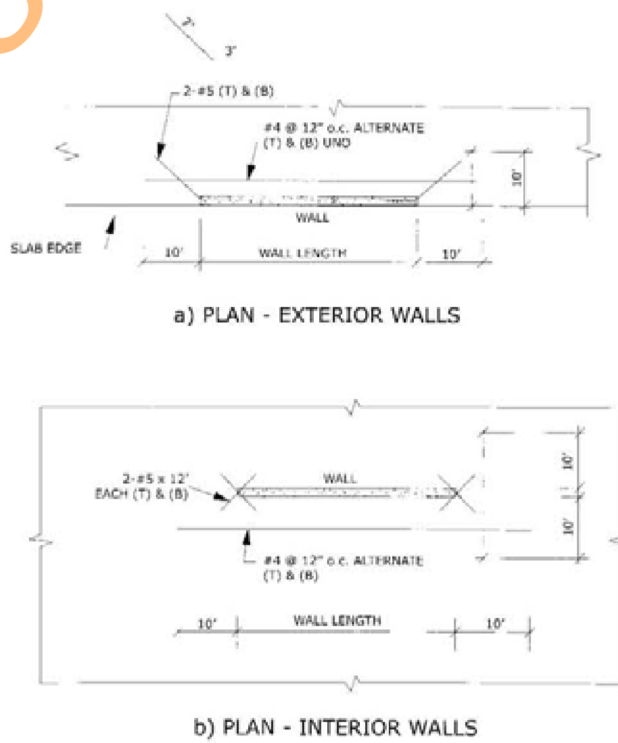


Fig. 6.16 Added Reinforcement at Bearing and Shear Walls

6.4.2.5 Typical One-Way Slab Details

Fig. 6.17 shows typical details for one-way slabs including temperature tendons and the required minimum steel according to ACI 318-02. The top beam reinforcing should never be used as the high point support for the uniform tendons. A separate support system should be provided. Support bars are typically placed 4 to 6 inches from the edge of the beam.

As shown on Fig. 6.17(b), temperature tendons should be laid flat (without profile) at approximately the mid-depth of the slab. The number of tendons required will depend on the slab depth and distance between beams. The tendons are typically spaced evenly between the effective beam flange widths. ACI 318-02 specifies a minimum of 100 psi; some Designers specify 150 psi to 200 psi for parking garages.

The uniform tendons may be used as part of the support system for the temperature tendons provided the CGS of the temperature tendons remains within the kern (middle third) of the slab. Similarly, temperature tendons may be used as part of the support system for the uniform tendons as long as the correct profile is achieved.

6.4.2.6 Tendon Profiles

Fig. 6.18 shows typical tendon profiles. The most commonly used profile for both slabs and beams is a reversed parabola with inflection points at span length/10 or span length/12.

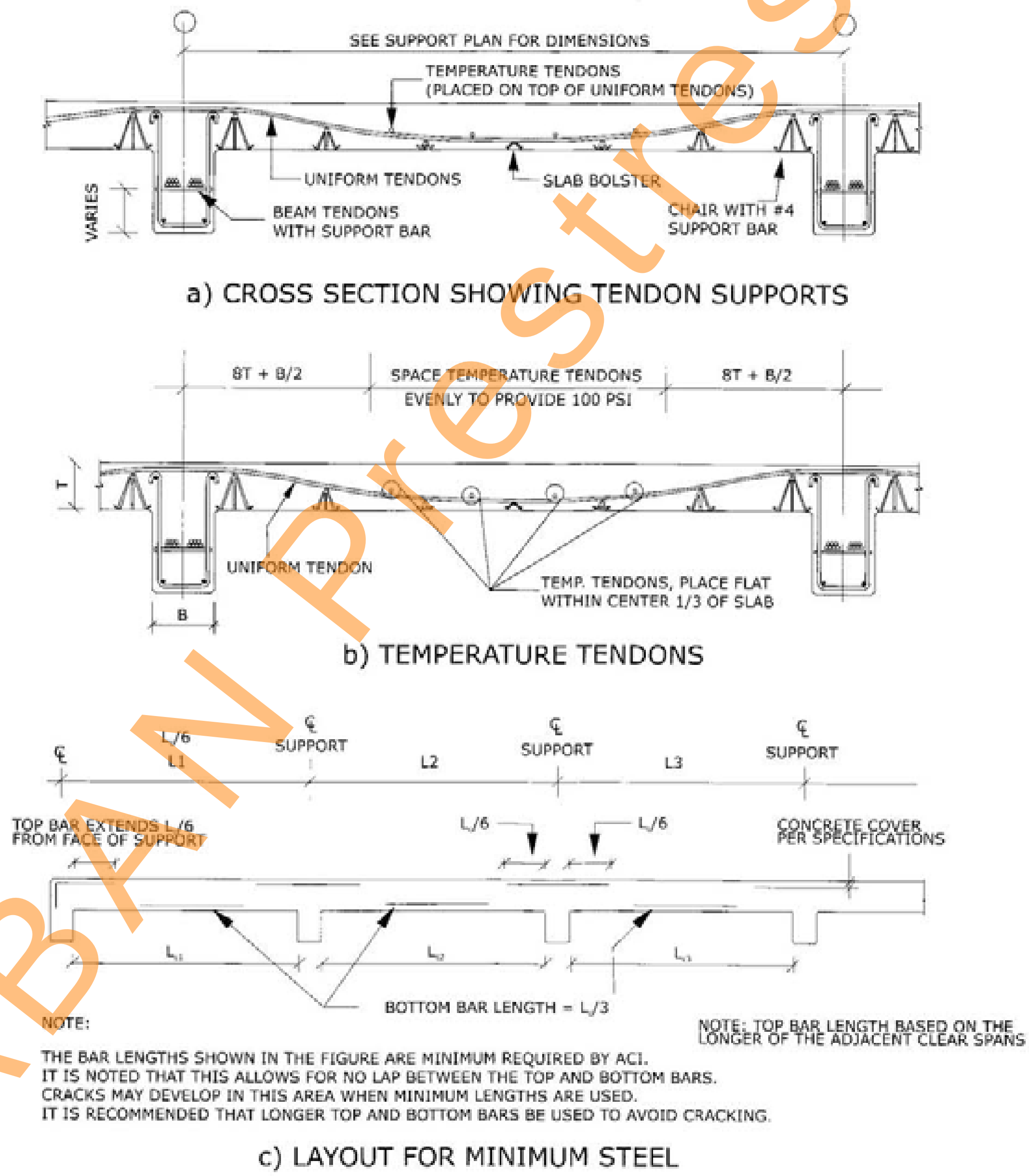


Fig. 6.17 Typical One-Way Slab Details

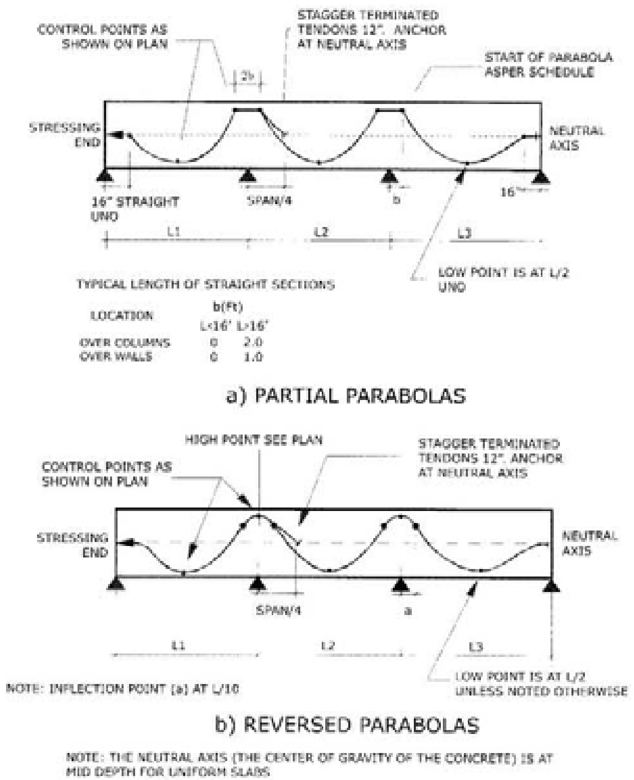


Fig. 6.18 Tendon Profiles

6.4.2.7 Beams

Fig. 6.19 shows tendon profile and reinforcing diagrams for single and multi-span post-tensioned beams. These diagrams would typically be accompanied by a beam schedule indicating the tendon profile in terms of high point at left end of span, low point, and high point at right end of span. The schedule would also indicate the size and number of the longitudinal reinforcing bars and the size and spacing of the stirrups. Note that these diagrams show the most general case. In a typical beam, partial length additional reinforcing would only be needed at the bottom of the beam at midspan and at the top of the beam at the supports.

To ensure that the tendon profile is maintained during concrete placement, the tendon support bars must be stable and properly secured to the beam stirrups. The tendon support system is critical due to the large prestressing forces typically required for beams. If tendon bundles are not properly secured to the support system, the tendons may move horizontally and cause improper concrete consolidation.

If the support system itself is not secured properly, the tendons can be displaced vertically, causing a reverse curvature. Tensile forces resulting from reverse curvatures may be high enough to cause the concrete to split. The spacing of beam tendon supports should not exceed 48 inches and

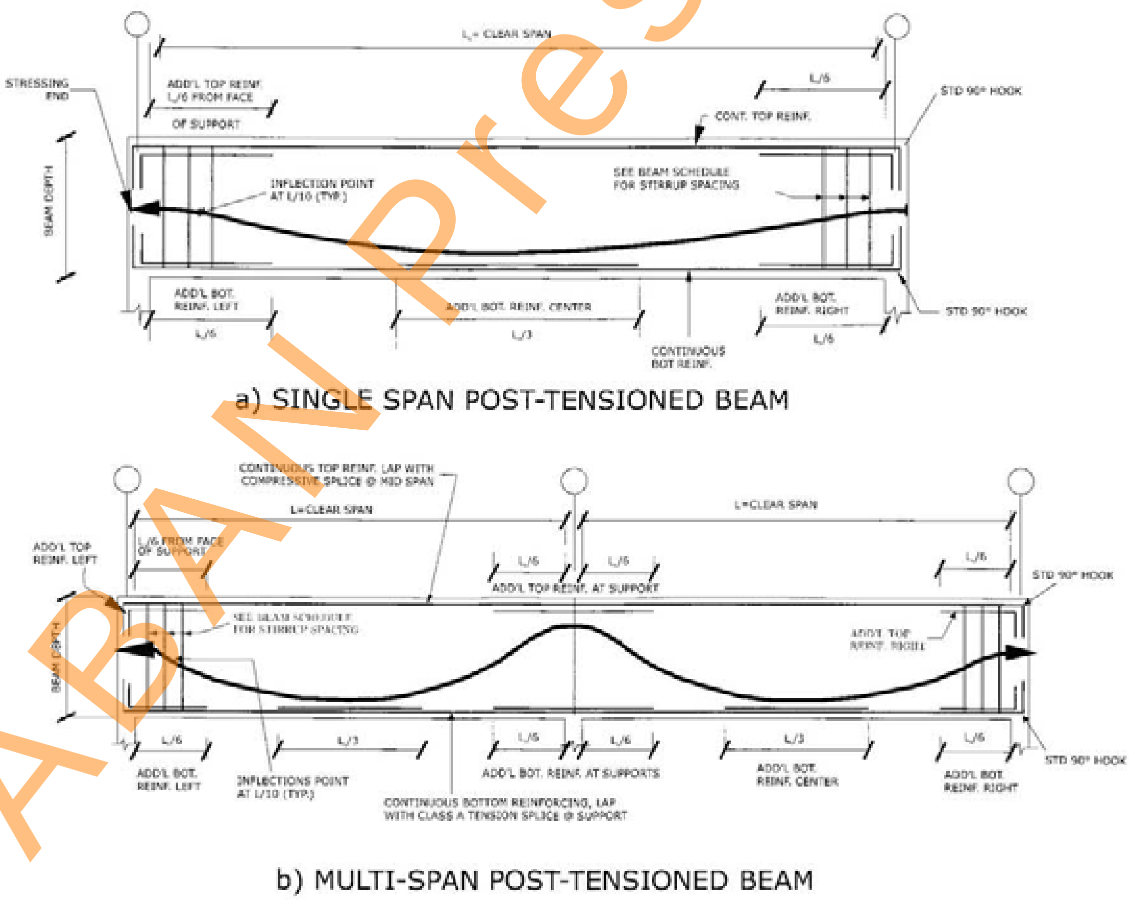


Fig. 6.19 Typical Beam Details

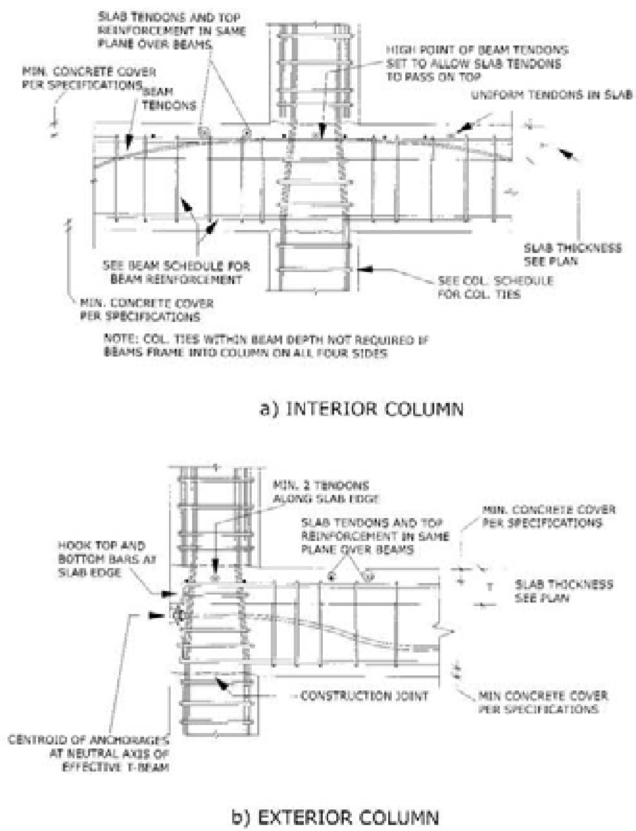


Fig. 6.20 Beam Column Connections

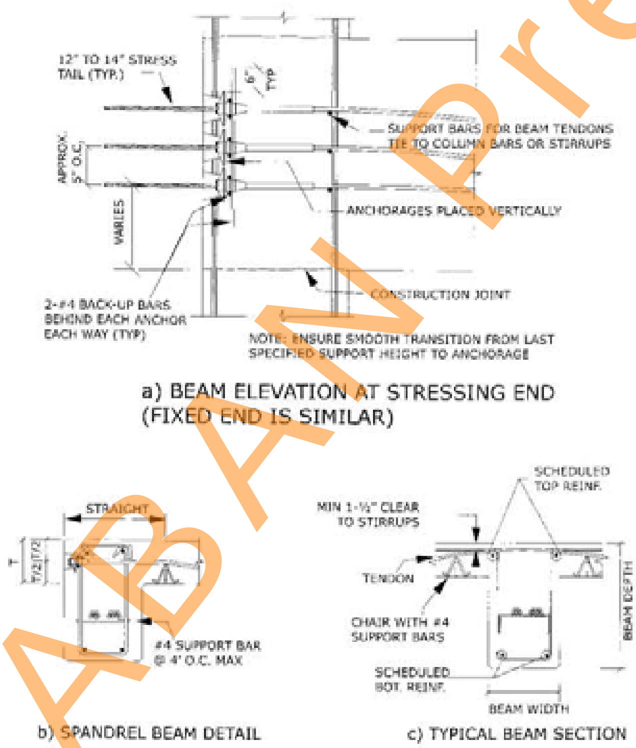


Fig. 6.21 Beam Details

should correspond to the typical stirrup spacing required by the Engineer of record, e.g., if the required stirrup spacing is 20 inches, the post-tensioning support system should be 40 inches on center. This will ensure that the tendon supports will coincide with stirrup locations so that they can be secured to the stirrup legs at the proper height.

Typical details for beam-column joints at interior and exterior columns are shown in Fig. 6.20. Typical details showing tendon and anchor placement in beams are shown in Fig. 6.21.

6.5 CONSTRUCTION PROCEDURES

6.5.1 Document Flow

In accordance with standard industry practice for construction, general details for unbonded post-tensioned members are typically provided by the Engineer of record. The details on the structural drawings show the member geometry and reinforcing layout. Using this information, the post-tensioning material supplier prepares more specific drawings, called installation/shop drawings that show locations of individual tendons or tendon groups, anchorages, and supports for maintaining the tendon profile. The installation drawings should be submitted to the Engineer of record for review before fabrication begins.

It is essential that details for the tendons, mild steel reinforcement, conduit, ductwork, and other embedment items be reviewed and coordinated by the Architect, Engineer and General Contractor during the preparation of installation drawings. The installation drawings prepared by the different material suppliers may show incompatible or conflicting layouts. In most cases, details can be adjusted at the installation drawing stage to accommodate all embedded items. When conflicts arise either during the development of installation drawings or during construction, the tendon layout should govern over other element or embedment locations unless otherwise indicated by the Engineer of record. In situations where the locations of other elements or embedments are critical, the Engineer of record and the post-tensioning material supplier should be notified and the appropriate adjustments made to the positioning of the tendons and/or anchorages. These adjustments will be shown on the approved post-tensioning installation drawings.

6.5.1.1 Installation/Shop Drawings

Installation/Shop drawings of post-tensioned slab should only be prepared using the most current issue of the Engineer of record's structural drawings marked "ISSUED FOR CONSTRUCTION." These installation/shop drawings should show the number, size, length, marking, elongation, and location in plan of all tendons, as well as the method of tendon support. Should structural drawings be changed after approval, the approved installation/shop drawings should be replaced with the revised, re-approved drawings. Care should be taken to update all sets. Superseded drawings should be preserved. The field installation should be undertaken from the most recently approved installation/shop drawings. Fig. 6.22 and 6.23 show examples of typical tendon layout and tendon support layout for a post-tensioned slab.

6.5.1.2 Shipping Lists

Each shipment of post-tensioning materials delivered to the job site should be accompanied by a list detailing specifically those materials included in the shipment. The quantity of material delivered should be checked against the shipping list at the time the materials are unloaded. Discrepancies should be reported to the post-tensioning material supplier immediately upon discovery. Failure to provide timely notification may result in project delays while replacement materials are fabricated and shipped. To ensure quality post-tensioning materials, it is required that the materials be ordered from a PTI Certified Plant. *Specifications for Unbonded Single Strand Tendons*[6.8] gives the minimum requirements and acceptance criteria for unbonded tendons.

6.5.1.3 Material Certifications

The physical properties of post-tensioning materials are described by material certificates provided by the post-tensioning material supplier when required by the contract documents or requested by the builder or Engineer of record. Such certificates may accompany the shipment to the job site or arrive by mail.

6.5.1.4 Jack Calibrations

An individual stressing unit is comprised of a hydraulic pump, jack and pressure gauge. Each jack and pressure gauge are calibrated as a "set" and should never be separated unless re-calibrated prior to use. All equipment "sets" used by a post-tensioning subcontractor should be accompanied by a current certified calibration chart, relating gauge pressure to force applied to a tendon. The calibration chart should be issued by an independent testing laboratory or by the post-tensioning company supplying the equipment. The calibration should be checked and re-certified every six(6) months. As a visual safety check, gauge faces should be marked to show maximum pressure reading for stressing.

6.5.1.5 Stressing Records

Stressing record forms should be available for use by stressing crews and project inspectors whenever stressing is undertaken. The completed stressing records should be submitted to the Engineer of record for review and approval prior to cutting of the tendon tails.

6.5.2 Formwork

The forming system is a major factor in determining the economy and construction speed of cast-in-place post-tensioned construction. A three-day per floor cycle can be achieved for completely cast-in-place post-tensioned building projects when careful consideration is given to the design, and detailing, of the forming. Fig. 6.24 shows the flying forms used for the one-way post-tensioned slab. Flying forms have been developed to accommodate complicated structural geometrics including up-turned and/or down-turned spandrel beams. The use of flying forms or large panel prefabricated forms is recommended whenever practicable. In all cases, selection of the basic structural system for a project should be made after careful consideration of the available forming methods. For conventional shored plywood and dimensioned lumber forming, economy can be achieved through repetition, simplicity of details, use of reasonable shapes, and provisions for easy installation and removal of bracing.

Forms should be drilled to receive tendon stressing hardware and bearing plates in accordance with the shop drawings. Fabrication and placement details provided by the post-tensioning materials fabricator will show end anchorage details, bolt hole dimensions, tendon identification, spacing, profile, stressing data, clearance requirements for the stressing equipment, and anchorage blockout dimensions.

Forms or scaffolding are often extended beyond the tendon terminal to provide space for the stressing and finishing operations.

6.5.3 Tendon Placing

Tendons are usually shipped to the job site in coils about 5 feet in diameter. The coil is secured by ties at intervals to prevent premature uncoiling. Each tendon is individually marked and clearly identified for its location in the job. Tendons may be handled mechanically or manually. Care should be exercised in unloading and handling the tendons to prevent damage to the sheathing. Belt or webbing slings are recommended when tendons are handled mechanically. Some damage to tendon sheathing may occur in handling. Usually this can be repaired in the field with approved repair tape.

The placing sequence number for tendons is indicated on the placing drawings. Coiled tendons should be transported to the deck according to placing sequence number. Each coil should be positioned near the slab edge where the stressing is to be done (shown in details). With the coil in a vertical plane, ties at the stressing end should be cut first and the tendon unrolled along the path that it will take in its final position. Remove the other ties only as the tendon is uncoiled to help prevent premature and sudden straightening of the tendon. After all tendons marked for the initial placing sequence have been uncoiled in their approximate position, tendons with the second placing

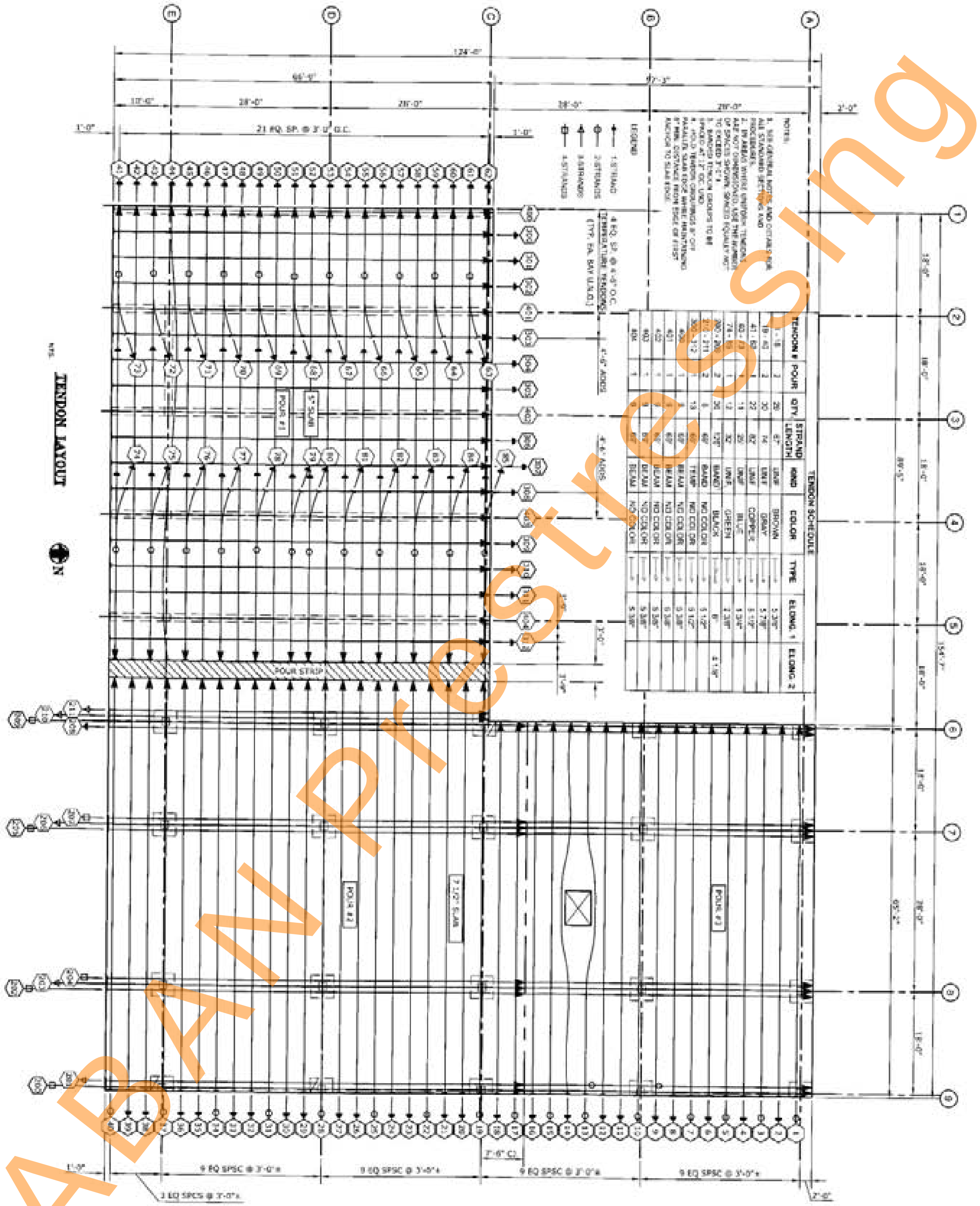


Fig. 6.22 Tendon Layout for One- and Two-Way Post-Tensioned Slabs, Refer to Fig. 6.1 for Structural Drawing
Courtesy of VSL

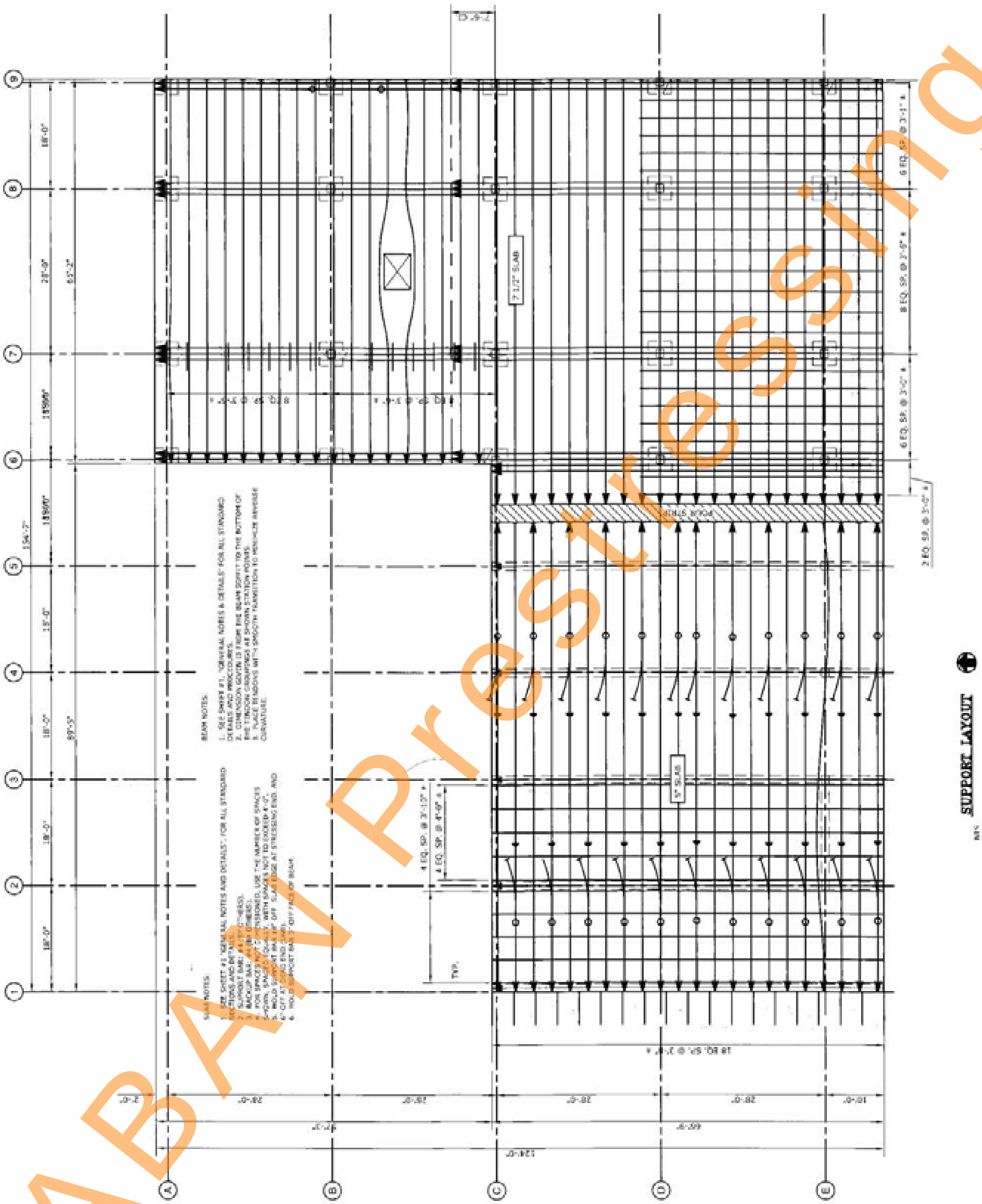


Fig. 6.23 Tendon Support Layout for One- and Two-Way Post-Tensioned Slabs, Refer to Fig. 6.1 for Structural Drawing
Courtesy of VSL

Fig. 6.24 Flying Forms

sequence number may be placed as described above. Remaining tendons should be placed in numerical sequence.

If the tendons are not installed by the post-tensioning materials fabricator, the fabricator usually provides the technical assistance necessary to instruct job personnel in proper placing, stressing, and grouting procedures. Workmen who place reinforcing steel are generally employed to place post-tensioning tendons.

Placement of tendons should normally precede placing reinforcing steel. However, in joists, beams, or girders a reinforcing cage may be used with open top stirrups to allow the reinforcing to be fabricated and placed prior to the tendon placement. Tendons may then be placed down through the open stirrups and secured in final position. Each tendon is designed for a specific location in the structure. Should some discrepancy be noted the post-tensioning materials fabricator should be consulted immediately.

When welding or burning near tendons, care must be exercised to avoid overheating the tendon, and to keep molten slag from coming in contact with the tendon. Grounding of welding equipment to the tendon should not be allowed.

6.5.4 Concrete Placement

The American Concrete Institute's *Guide for Measuring, Mixing, Transporting, and Placing Concrete*[6.9] presents detailed procedures for these activities which are generally applicable to cast-in-place post-tensioned construction. The American Concrete Institute has also published Recommended practices for hot and cold weather concreting which present special considerations relevant to placement of concrete when the weather conditions are unfavorable. Some cast-in-place post-tensioned construction is accomplished in unusually hot or cold weather. Admixtures known to have no injurious effects on steel or concrete may be used. Additives and admixtures that contain chloride (e.g. calcium chloride) should not be used in concrete for post-tensioned construction because of their potential to cause corrosion.

Prior to placing concrete, tendon profiles should be checked at high and low points. If the tendon sheathing or duct has been damaged, repairs should be made to prevent concrete from bonding to the tendon or from entering the duct.

Horizontal alignment should be checked to ensure minimum horizontal deviations and proper concrete cover at openings. Care should be exercised to prevent concrete from entering pockets, sheathing, or anchorage hardware. A workman should be assigned to maintain proper tendon alignment slightly ahead of concrete placement.

Concrete should be placed in such a manner that tendon alignment and reinforcing steel positions remain unchanged. Special attention must be given to vibration of concrete at tendon anchorages to ensure uniform compaction at these points. Voids behind the bearing plate, or insufficient concrete strength will cause concrete failure. Careful vibration and proper curing will eliminate most such difficulties. Voids behind the bearing plate should be repaired prior to the stressing operation.

Concrete should be cured in accordance with ACI recommendations to ensure proper concrete strength.

6.5.5 Stressing Operations

When the stressing is not performed by the post-tensioning materials fabricator, all necessary stressing equipment is usually furnished by the post-tensioning fabricator and delivered to the job site according to a prearranged schedule.

When tests of field-cured cylinders indicate that the concrete has reached the proper strength (usually 3000 psi) the stressing operation may begin. It is essential that the shoring be left in place at least until the stressing is complete and the stressing records have been reviewed and accepted by the licensed design professional.

Tendons should be stressed only when proper data and experienced personnel are present. Refer to the approved calibration data for proper ram area, forces and gauge readings for each tendon. PTI's *Field Procedures Manual for Unbonded Single Strand Tendons*[6.1] gives detailed guidance on the procedures to be followed during stressing operations.

Stressing is monitored in two ways. First, the gauge reading on the pump may be translated into force in the tendon at the anchorage. This information is generally provided in a tendon stressing data table by the post-tensioning fabricator. Second, the measured elongation is compared against the theoretical elongation of the tendon as shown on the approved installation/shop drawings.

It is generally required that the tendon force measured by gauge pressure agree within 7 percent with the tendon force determined by elongation measurements as shown on the approved installation/shop drawings. The modulus of elasticity of 7-wire strand can vary somewhat from the 28,500,000 psi average value normally used to determine the elongations shown on the approved installation/shop drawings. Because a variation of 1,000,000 psi in the modulus of elasticity represents a difference of about 4% in elongation, it is always preferable to use the actual modulus of elasticity of the strand used on the project (supplied by the post-tensioning materials fabricator) when comparing tendon elongation and gauge pressure in the field. The tendon elongation is affected by the variation in force due to friction losses throughout the tendon length. For this reason, friction losses should be considered in translating tendon elongation measurements into tendon forces. The elongation measurement provides a measure of the average force throughout the length of the tendon, whereas the gauge pressure gives the force in the tendon at the anchorage.

Stressing equipment supplied by post-tensioning materials fabricators has been carefully designed and incorporates reasonable factors of safety. Occasionally, flaws in material are undetected or the equipment may have been misused. For this reason, extreme caution should be exercised at all times as stressing is carried out at extremely high pressures. THE PRIMARY SAFETY RULE IS TO KEEP PERSONNEL FROM THE AREA DIRECTLY IN BACK OF STRESSING EQUIPMENT, OR BETWEEN THE EDGE OF THE BUILDING AND THE EQUIPMENT. Failure during the stressing operation may cause serious injury to any personnel in the immediate vicinity of the stressing equipment. All of the usual concrete construction safety regulations apply to post-tensioned construction.

Should stressing reveal that voids or other concrete deficiencies exist, (i.e. the bearing plate begins to recede into concrete), release all pressure on the equipment at once, remove the faulty concrete and, with the approval of the Engineer of record, patch the void with suitable material that will attain the required strength before attempting to re-stress the tendon. Calcium chloride or admixtures containing calcium chloride should not be used in any patching operation.

Exercise care in operating the pump to retract the ram. Do not allow pressure build-up on the return side, as this may damage the ram seal.

6.5.6 Form Removal and Re-Shoring

Shoring must be left in place until the stressing operation is completed and elongations/tendon forces reviewed by the Engineer of record. Edge or pocket forms, and bulkheads should be removed well ahead of the stressing operation. Beam or side forms may be removed prior to stressing with permission from the Engineer of record.

Removal of shoring and forms may follow immediately after the stressing operation and review of the tendon forces and elongations by the Engineer of record. After stressing, re-shoring may be required to prevent overloading during subsequent construction.

6.5.7 Protection of End Anchorages

Stressing pockets shall be filled with non-metallic, non-shrink, chloride-free grout as soon as possible after stressing but not later than 10 days after stressing. If the tails cannot be cut within 10 days then temporary protection shall be provided. See *Specification for Unbonded Single Strand Tendons*[6.8] for additional protection requirements.

6.6 SPECIAL ISSUES

6.6.1 Lift-Off Procedures

Should a lift-off procedure be contemplated, the Engineer of record must be consulted prior to proceeding with the tendon lift-offs. The purpose of a lift-off is to verify the force in a tendon at the stressing end after it has been stressed. A lift-off may be appropriate when the recorded elongation is out of the specified tolerance. Project specifications may call for selected force verification using the lift-off method. A "lift-off test" may be conducted by use of the standard hydraulic stressing jack on previously stressed and anchored unbonded post-tensioning tendons to determine the residual effective force in the tendon at the anchorage. The lift-off test is preferable and most easily done before the stressing tails of the tendons have been cut off. While it may be possible to conduct a lift-off test after the stressing tails have been cut off, this possibility is determined by the length of tendon protruding beyond the wedges in the stressing pocket as well as the possibility of connecting the hydraulic jack to this length of tendon, (this may be dangerous).

When the tendon is initially stressed and anchored, the wedge seating that occurs develops a mechanical-friction force between the strand, wedges and anchorage casting. During the lift-off test, it is necessary to stress the tendon in excess of the residual effective tendon force at the anchorage by an amount equivalent to this mechanical-friction force in order to break the wedges loose and then determine the force remaining in the tendon. This process will be reflected during the lift-off test by stressing to a level (reflected on the gauge attached to the ram) sufficient to break the wedges loose, and a subsequent reduction in the gauge pressure to reflect the residual force in the tendon. It should be understood that the lift-off test determines the residual force in the tendon at the anchorage. Determination of the force level in the tendon at other locations requires detailed consideration of friction and wedge seating effects. IN GENERAL, LIFT-OFFS ARE TO BE AVOIDED AS A ROUTINE REQUIREMENT.

6.6.2 De-Tensioning Tendons

It may sometimes be required to de-tension tendons after they have been stressed. The de-tensioning operation should be carried out under the immediate supervision of an experienced stressing operator. It is essential that during de-tensioning the operator follow safety precautions similar to stressing. Several techniques are available for de-tensioning, depending on the experience of the contractor and project conditions. A licensed design professional should review the de-tensioning procedure and recommend a de-tensioning sequence. The most common procedures used to de-tension tendons are: Heating the wedges until the tendon slips; it may also be done by exposing the strand at an interior location and grinding the strand with a hand grinder or burning it with a welding torch. Because there is a heavy concentration of stresses behind the anchor, it is not advisable to chip directly behind the anchors. As demonstrated in section 6.6.4 special jacks are also used in sensitive applications to safely de-tension post-tensioning strands.

6.6.3 Splicing Tendons

Tendons are sometimes too short to reach an edge form because of misplacement or incorrect cutting length. If the tendon is in one pour only and not continuous, every effort should be made to replace the short tendon with a tendon of proper length instead of using couplers. If tendons are continuous from another pour, thus making tendon couplers necessary, the Engineer of record and the post-tensioning material supplier should be notified. The coupler location should be determined by the post-tensioning material supplier such that the coupler is centered in the member and not at a point of tendon curvature. Fig. 6.25 shows the details of coupler installation. Couplers should not be located side by side. If more than one tendon requires splicing, couplers should be staggered at half bay increments per tendon group.

A PVC pipe of sufficient inside diameter to hold the coupler and of sufficient length to allow for subsequent elongation movement shall be used. An additional piece of sheathed strand of sufficient length to reach the edge form is required along with two pocket formers. P/T coating shall be used to fill the void in the PVC pipe. The tapered tip of the pocket former that normally fits inside the anchor cavity can be cut off when being used for splicing thereby reducing the length of the PVC pipe needed. The original strand is first cut with a saw or abrasive plate at the coupler location and one pocket former is placed on the strand. Mark the strand before coupling to make certain that the proper length of strand has been fully inserted into the coupler. The coupler is then connected to the original strand. The PVC pipe is placed over the coupler. The second pocket former is placed over the new strand (the strand is marked) and inserted into the coupler. A pocket former is taped to one end of the PVC pipe which is then packed tightly with P/T coating, allowing no air voids. The second pocket former is affixed to the PVC pipe completing a tightly sealed coupler.

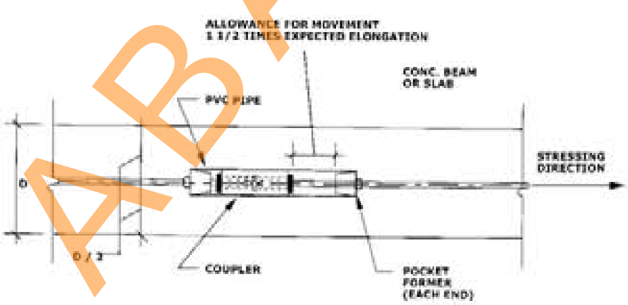


Fig. 6.25 Coupler Installation[6.1]

Fig. 6.26 Tendon Locations Marked at Bottom of Slab by Use of Spray Paint on the Form Prior to Concrete Placement
Courtesy of Seneca Structural Engineering, Inc.

The tendon coupler's location within the PVC pipe must permit the coupler to move the required elongation amount in the direction of stressing. Allowance for movement in both directions must be provided when the tendon is to be stressed from both ends. Conservatively, a minimum of 1.5 times the total expected elongation at the splice location shall be allowed for. A dark crayon or paint mark on the deck will facilitate locating the coupler after the pour, should that become necessary if the above procedure was not properly followed.

6.6.4 Future Slab Penetrations and Openings

Slab penetrations in post-tensioned buildings floors should be preplanned. Tendon layouts for most floor systems provide reasonably large areas of concrete without tendons which permits flexibility in location of openings and penetrations. For penetration of slabs following construction it is necessary to locate the tendons and to locate the openings to avoid tendon damage. If future penetrations are anticipated it may be desirable to mark the tendon location on the bottom of the concrete slab as illustrated in Fig. 6.26. In this case the concrete was marked by spraying the forms with paint along the tendon lines just prior to placement of concrete. Sufficient amount of paint was transferred to the concrete to permanently locate the tendons. Forms can also be marked with construction crayons or by other physical means to provide a mark locating each tendon in the bottom of the slab. Other non-destructive testing methods may also be used to locate the tendons prior to cutting into the slab.

In some cases, it may be necessary to make large openings in post-tensioned floors after construction. In these cases care should be taken to avoid major structural members for the openings. Depending on the size of the opening it may be required to de-tension some tendons. Fig. 6.27 shows an example of where a stair opening was required to interconnect floors at 10 separate floor levels in a structure after construction. Stair openings of 14 ft by 14 ft were cut into the post-tensioned slabs without the use of extensive shoring, and in most cases, without requiring additional support beams. The process of constructing the opening is illustrated in Figs. 6.27, 6.28, and 6.29. After removing the concrete in the opening location, the tendons were cut with a torch while restrained at the edges of the opening by heavy metal clamps as shown in Fig. 6.27. The metal clamps were used to provide a slow release of the tendon force after the tendons were cut. This process permitted de-tensioning of all tendons through the stairwell openings with no damage to the glazing around the perimeter of the buildings. New tendon anchorages were cast at the edge of the opening as shown in Fig. 6.28, and after concrete reached the necessary strength, the tendons were re-stressed. Finally, forms were installed as shown in Fig. 6.29, and concrete was placed around the forms to finish the openings.

Fig. 6.27 Procedure for De-Tensioning Tendons at Stairwell Opening

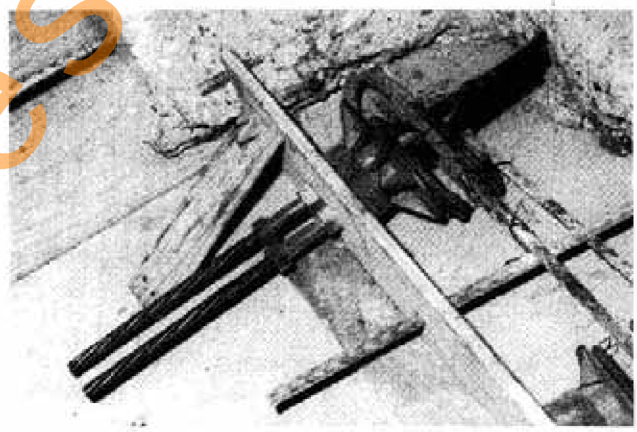


Fig. 6.28 Anchorage Reset at Perimeter of Opening Ready for Concrete Placement

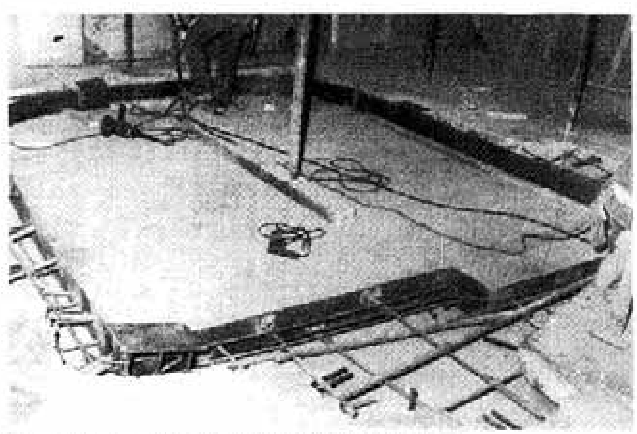


Fig. 6.29 Forming of Stairwell Opening in Place

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6.2 Aalami, B. O., "One-Way and Two-Way Post-Tensioned Floor Systems," *PTI Technical Notes*, Issue 3, October 1993, pp. 1-10.

6.3 *Design Fundamentals of Post-Tensioned Concrete Floors*, Post-Tensioning Institute, Phoenix, AZ, 1999.

6.4 *Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures*, Post-Tensioning Institute, Phoenix, AZ, 2001.

6.5 *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-02, American Concrete Institute, Farmington Hills, MI, 2002.

6.6 *Anchorage Zone Design*, Post-Tensioning Institute, Phoenix, AZ, 2000.

6.7 Sanders, D. H., Breen, J. E., and Duncan III, R. R. , *Strength and Behavior of Closely Spaced Post-Tensioned Monostrand Anchorages*, Post-Tensioning Institute, Phoenix, AZ, 1987.

6.8 *Specification for Unbonded Single Strand Tendons*, 2nd Edition, Post-Tensioning Institute, Phoenix, AZ, 2000.

6.9 *Guide for Measuring, Mixing, Transporting, and Placing Concrete*, ACI 304R-89, American Concrete Institute, Farmington Hills, MI, 1989.

DESIGN EXAMPLES

7.1 INTRODUCTION

In this chapter, five numerical examples are presented to illustrate various fundamental concepts of post-tensioned concrete analysis and design. The examples were selected with simplified geometries to illustrate basic concepts through easy-to-follow step-by-step hand computations. In all five examples, related references to the American Concrete Institute's *Building Code Requirements for Structural Concrete* (ACI 318-02)[7.1] *and Commentary* (ACI 318R-02) are given.

The first two examples in Sections 7.2 and 7.3 illustrate a complete design carried out for one- and two-way slab systems, respectively. Because the majority of post-tensioned slabs in the United States are constructed with unbonded tendons, bonded tendons design examples are not covered. More detailed information about the design of one-way and two-way slabs with unbonded tendons can be found in Ref. 7.2.

Section 7.4 discusses the design of a simply supported T-beam post-tensioned with bonded tendons. Bonded tendons are commonly used in transfer beams and bridge superstructures.

In Section 7.5, a numerical example is presented to illustrate the various concepts discussed in Chapter 5. These include load balancing, equivalent loading, secondary moment computations, and stress computations at the extreme fiber. Secondary moments are calculated using two approaches: a free-body diagram approach, and the $M_{bal} - Fe$ approach.

Section 7.6 presents an example illustrating the application of the strut-and-tie model in the anchorage zone design of a wide shallow beam. Further examples involving end anchorage design of various types of structures, such as box girder bridges and precast beams, can be found in Ref. 7.3.

7.2 DESIGN OF A ONE-WAY SLAB IN A PARKING STRUCTURE

7.2.1 Given Information

Assume f'_c = 4000 psi

Design the one-way parking slab shown in Fig. 7.1 for 50 psf live loads.

7.2.2 Determine Slab Thickness

From Table 9.3 (See chapter 9) for one-way slab

Span / depth = $L/48$

$L/48 = 18 \times 12/48 = 4\text{-}½$ in.

For durability use, 1 in. additional top cover = 5-½ in.

Note: 5 in. thick slabs are commonly used with 18 ft spans in non-aggressive environments.

7.2.3 Determine Loads

Dead	150 × 5.5 / 12 = 69 × 1.2	= 82.5 psf use 83 psf
Live	= 50 × 1.6	= 80 psf
Total	= 119 psf, unfactored	
	= 163 psf, factored	

7.2.4 Estimate Balanced Load

Assume that 65% of the dead loads are balanced by the post-tensioning.

$w_{bal} = 0.65 \times 69 = 45$ psf

7.2.5 Determine Frame Properties

Since the slab is of uniform thickness EI is constant. The slab is supported on beams hence there is no column stiffness.

Thus:

Say slab stiffness: $k_{interior} = 1$

$k_{endspan} = ¾$

7.2.6 Calculate Fixed-End Moments (FEM)

7.2.6.1 Dead Loads

Interior Span = 0.069 (18²)/12 = –1.86 ft-k

Exterior Span = 0.069 (18²)/8 = –2.79 ft-k

Conduct a Moment Distribution for Dead Load Moments at supports

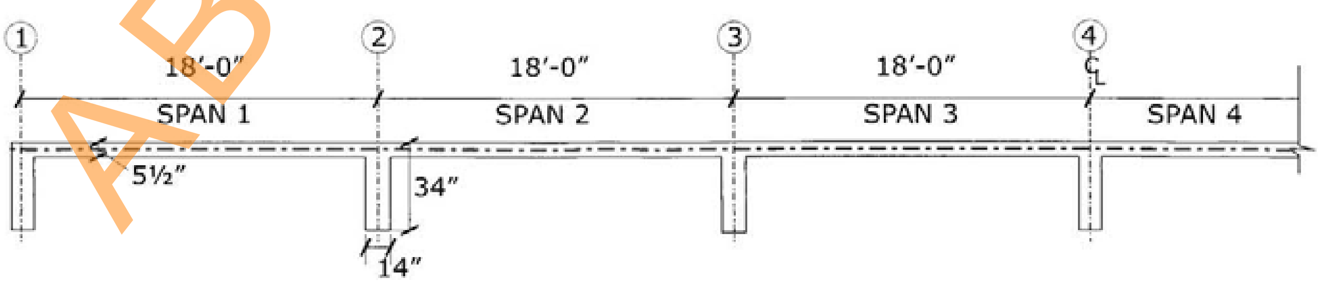

Fig. 7.1 Slab Cross-Section

Table 7.1 - Moment Distribution for Dead Load

	2		3		4	
DF	0.43	0.57	0.5	0.5	0.5	0.5
FEM	-2.79	-1.86	-1.86	-1.86	-1.86	-1.86
Dist.	+0.40	-0.53				
Carry-over			+0.27			
Dist.			-0.13	+0.14		
Sum	-2.39	-2.39	-1.72	-1.72	-1.86	-1.86

Moments have units of ft-k

7.2.6.2 Balanced Loads

Assume 45 psf balanced load at all spans. Then, by ratio of 45/69 times the above moments, the moments at supports due to balanced loads only are (Note sign change):

Table 7.2 - Balanced Load Moments

2	3	4
+1.56 ft-k	+1.12 ft-k	+1.21 ft-k

7.2.6.3 Live Loads (Patterns)

Case 1 – Live loads on Span 1 and 2 (Maximum –ve moment at grid 2):

Interior Span $= 0.05\ (18^2)/12 = 1.35$ ft-k

Exterior Span $= 0.05\ (18^2)/8 = 2.02$ ft-k

Table 7.3 - Moment Distribution for Live Loads; Case 1

2		3		4	
0.43	0.57	0.5	0.5	0.5	0.5
-2.02	-1.35	-1.35	0		
+0.29	-0.38	+0.67	-0.68		
	-0.33	+0.19			
-0.14	+0.19	-0.10	+0.09		
-1.87	-1.87	-0.59	-0.59		

Case 2 – Live loads on Span 1 and 3 (Maximum +ve moment in Span 1):

Table 7.4 - Moment Distribution for Live Loads; Case 2

2		3		4	
0.43	0.57	0.5	0.5	0.5	0.5
-2.02			-1.35	-1.35	
+0.87	-1.15	-0.68	+0.67	+0.67	-0.68
	+0.34	+0.57	-0.33	-0.33	
+0.15	-0.19	-0.45	+0.45	+0.17	-0.16
	+0.22	+0.10	-0.08	-0.22	
+0.09	-0.13	-0.09	+0.09	+0.11	-0.11
-0.91	-0.91	-0.55	-0.55	-0.95	-0.95

Positive moment at middle of first span:

$wl^2/8 - 0.91/2 =$

$$\frac{0.050 \times 18^2}{8} - 0.45 = 1.57 \text{ ft-k}$$

Case 3 – Live loads on Span 2 and 3 (Maximum –ve moment at grid 3):

Table 7.5 - Moment Distribution for Live Loads; Case 3

2		3		4	
0.43	0.57	0.5	0.5	0.5	0.5
	-1.35	-1.35	-1.35	-1.35	
-0.58	+0.77	0	0	+0.67	-0.68
	0	-0.39	-0.33	0	
	0	+0.03	-0.03	0	
		-1.71	-1.71		

Note: Maximum -ve moment at grid 4 will be similar to that at grid 3

Case 4 – Live loads on Span 2 and 4 (Maximum +ve moment in Span 2):

Table 7.6 - Moment Distribution for Live Loads; Case 4

2		3		4		5	
0.43	0.57	0.5	0.5	0.5	0.5	0.5	0.5
	-1.35	-1.35	0	0	-1.35	-1.35	0
-0.58	+0.77	+0.67	-0.68	-0.67	+0.68	+0.67	-0.68
0	-0.33	-0.39	+0.34	+0.34	-0.33	-0.33	0
-0.14	+0.19	+0.36	-0.37	-0.33	+0.34	+0.17	-0.16
	-0.18	-0.10	+0.17	+0.19	-0.09		
-0.08	+0.10	+0.14	-0.13				
-0.80	-0.80	-0.67	-0.67				

Maximum positive moment in Span 2:

$wl^2/8$ – Average FEM $= 0.05\ (18^2)/8 - (0.8 + 0.67)/2$

$= 1.29$ ft-k

Note: A simplified two cycle moment distribution procedure for calculating moments due to skipped live loads is presented in the PCA publication *Continuity in Concrete Building Frames.*[7.4]

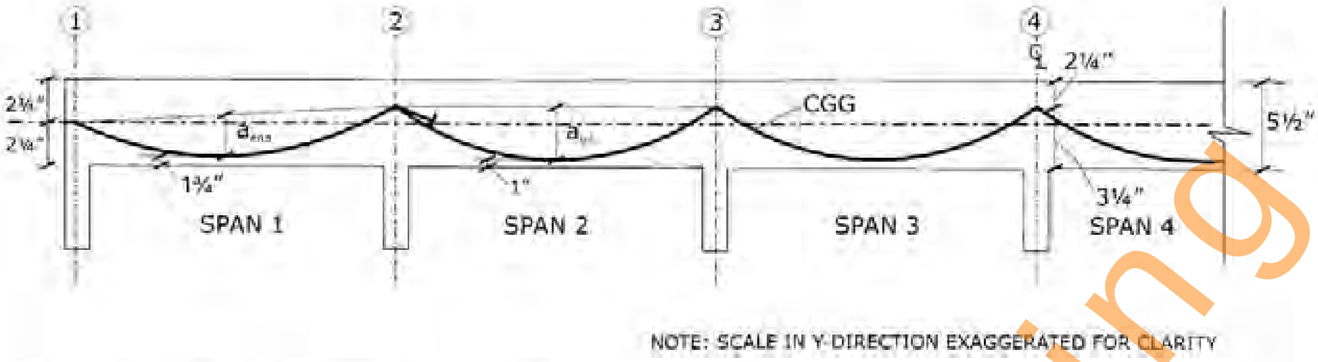

Fig. 7.2 Initial Tendon Profile Layout (Measured to CGS)

7.2.7 Determine Post-Tensioning Force and Tendon Profile

7.2.7.1 Determine Profile

Assume cover for 2 hour fire rating; i.e. ¾ in. typical bottom cover (see Table 18.2, Chapter 18), 1½ in. @ end spans (considered unrestrained).

Use 2 in. typical top cover in consideration of traffic and exposure to weather and deicing chemicals.

Tendon Sag at endspan:

$$a_{end} = \frac{2\frac{3}{4} + 3\frac{1}{4}}{2} - 1\frac{3}{4} = 1.25 \text{ in.}$$

Tendon Sag at typical span:

$$a_{int} = 5\tfrac{1}{2} - 2\tfrac{1}{4} - 1 = 2.25 \text{ in.}$$

$$F = \frac{w_{bal}L^2}{8a}$$

7.2.7.2 Determine Tendon Force

End span:

$$F_{end} = 0.045\,(18^2)/(8 \times 1.25/12)$$
$$= 17.5 \text{ k/ft}$$
$$F/A = 17.5/(5.5 \times 12) = 265 \text{ psi}$$

Interior span:

$$F_{int} = 0.045\,(18^2)/(8 \times 2.25/12) = 9.72 \text{ k/ft}$$
$$F/A = 9.72/(5.5 \times 12) = 147 \text{ psi}$$

7.2.8 Combine Moments and Adjust to Critical Section at Face of Support.

The PCA publication *Continuity in Concrete Building Frames*[4] Suggests modification of design moments to the face of columns or supports in accordance with the following diagram:

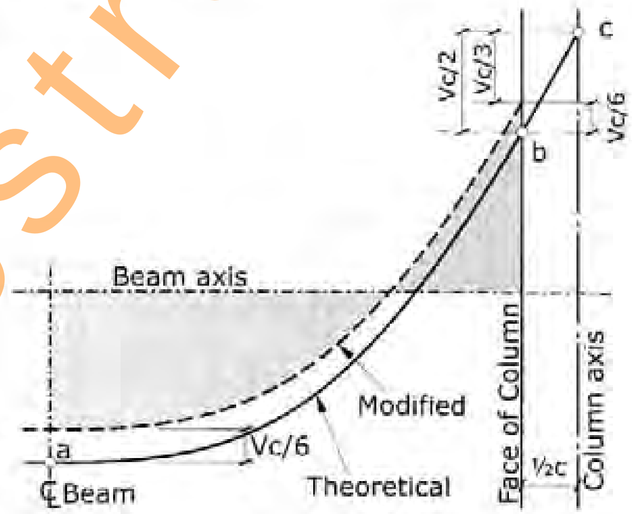

Fig. 7.3 Correction to Design Moment at Face of Column

The Vc/6 correction at the column face and midspan from the theoretical curve represents the effects of an assumption of an infinite moment of inertia over the width of the column or support. The reduction in negative moment is commonly taken as Vc/3 at the face of the supports and Vc/6 at midspan.

$$\text{Min } V_{net} \text{ @ support} = 0.074 \times 18/2 = 0.666 \text{ k}$$

$$\text{Section Modulus } S = \frac{bh^2}{6} = \frac{12t^2}{6} = 60.5 \text{ in}^3$$
$$= 5.04 \text{ in}^3/\text{ft}$$

Table 7.7 - Service Load Moments and Tensile Stresses at Face of Supports

Loading	Magnitude psf	Moments (ft-k at support) 2	3	4
Dead Load	0.069	-2.39	-1.72	-1.86
Live Load	0.050	-1.87	-1.71	-1.71
Balanced Load	0.045	+1.56	+1.12	+1.21
Net Load	0.074	-2.70	-2.31	-2.36
Min V_{net} at Support (k)		0.67		0.67
$Vc/3 = 0.67 \times 14/12 \times 1/3$		+0.26		+0.26
Face Moment		-2.44		-2.10
M/S		+0.484		+0.417
P/A		-0.265		-0.147
Stress at top of slab (ksi)		+0.219		+0.270

Maximum net tensile stress = 0.270 ksi = $4.27\sqrt{f'_c}$. Design could be recycled with slightly lower balanced load. However, this will be considered satisfactory for illustrative purposes. Use post-tensioning force of 17.5 k/ft in the end spans and 9.72 k/ft in the typical interior spans.

7.2.9 Calculate Design Moments

7.2.9.1 Calculate Secondary Moments

The balanced load moment includes both the "primary" effect of post-tensioning, M_1, (due to force times eccentricity) and the secondary effects, M_2, (due to moment restraint of supports). This can be expressed as:

$$M_{bal} = M_1 + M_2 \quad \text{or} \quad M_{bal} = Fe + M_2$$

Where:

M_1 = Fe = Primary Moment

M_2 = Secondary Moment

Since we have previously found the moments due to balanced load, M_{bal}, and have defined force and profile, secondary moments are easily found for design moment calculations by:

$$M_2 = M_{bal} - Fe$$

Balanced load moment correction to face of support:

$$\frac{Vc}{3} \text{ at support } 2 = 0.045 \times 9 \times \frac{14}{12} \times \frac{1}{3} = 0.16 \text{ ft-k}$$

Table 7.8 - Calculation of Secondary Moments

Supports	2	3	4
Balanced Moment at Supports	+1.56	+1.12	+1.21
-Vc/3	-0.16	-0.16	-0.16
Balanced Moment a at Face	+1.40	+0.96	+1.05
Fe = 17.5 k × 0.5/12 =	-0.73		
Fe = 9.72 k × 0.5/12 =		-0.41	-0.41
Secondary Moments M_2 =	+0.67	+0.55	+0.64

Moments shown have units in ft-k

7.2.9.2 Calculate Design Moment

Combine 1.2 dead + 1.6 live + 1.0 M_2

Table 7.9 - Calculation of Design Moments at Face of Supports

	Span 1 Midspan	2	Span 2 Midspan	3	Span 3 Midspan	4
1.2 D	+1.93	-2.89	+0.89	-2.06	+1.21	-2.23
1.6 L	+2.51	-2.99	+2.05	-2.74	+2.05	-2.74
1.0 Sec	+0.34	+0.67	+0.61	+0.55	+0.60	+0.64
M	+4.78	-5.19	+3.56	-4.25	3.86	-4.33
V	1.47	1.47	1.47	1.47	+1.47	1.47
Vc/3		+0.57		+0.57		+0.57
Vc/6	-0.29		-0.29		-0.29	
M_{face}	+4.49	-4.62	+3.27	-3.68	+3.57	+3.76

7.2.10 Check Flexural Capacity

Calculate capacity utilizing minimum bonded steel area = 0.002bh (equivalent to 0.004A) in accordance with ACI 318-02 equation (18-6).

7.2.10.1 Determine Ultimate Tendon Force

From equation (18-5) of ACI 318-02:

$$f_{ps} = f_{se} + 10,000 + \frac{f'_c}{300\rho_p}$$

Stressing tendons to 0.7 × 270 = 189 ksi, and allowing 15,000 psi for losses provides an effective tendon force, f_{se}, of 174,000 × 0.153 = 26.6 k for each ½ in. diameter 270 ksi strand (A_{ps} = 0.153 sq in.)

$\rho_p = A_{ps} / bd$

end span $\rho_p = \dfrac{17.5}{26.6} \times 0.153/(12 \times d)$

Use average $d = \dfrac{3.25 + 3.75}{2} = 3.50$ in.

end span $\rho_p = \dfrac{17.5}{26.6} \times \dfrac{0.153}{12 \times 3.50} = 0.00240$

$f_{ps} = 174,000 + 10,000 + \dfrac{4000}{300 \times 0.00240}$

$f_{ps} = 189,556$ psi

interior span $\rho_p = \dfrac{9.72}{26.6} \times \dfrac{0.153}{12 \times 3.88} = 0.00120$

$f_{ps} = 174,000 + 10,000 + \dfrac{4000}{300 \times 0.00120}$

$f_{ps} = 195,100$ psi

$F_{ps} = \dfrac{189.56}{174} \times 17.5 = 19.06$ k/ft @ end spans

$F_{ps} = \dfrac{195.1}{174} \times 9.72 = 10.90$ k/ft @ interior spans

7.2.10.2 Calculate Design Capacity

Capacity at Exterior Midspan Section:

$A_s = 0.002(5.5)12 = 0.132$ sq in. Use #4 @ 18 in. $= 0.133$ sq in

$F_{ps} = 19.06$ k @ end span

$A_s f_y = 0.133 \times 60 = 7.98$ k

$T_u = 19.06 + 7.98 = 27.04$ k @ end span

Depth of Compression Block:

$a = \dfrac{T_u}{0.85 f'_c b}$

$a = \dfrac{27.04}{0.85 \times 4 \times 12} = 0.66$ in.

$\left(d - \dfrac{a}{2}\right) = 3.75 - 0.66/2 = 3.42$ in.

$M_u = \phi T_u \left(d - \dfrac{a}{2}\right) = 0.9 \times 27.04 \times \dfrac{3.42}{12}$

$M_u = 6.94$ ft-k

M_u provided $= 6.94$ ft-k > 4.49 ft-k required OK with minimum steel

Capacity at First Interior Support:

$A_s = 0.133$ #4 @ 18 in. centers

$T_u = 27.04$ k as for exterior midspan

Depth of Compression Block:

$a = \dfrac{27.04}{0.85 \times 4 \times 12} = 0.66$ in

$\left(d - \dfrac{a}{2}\right) = 3.33 - 0.25 = 2.92$ in.

$M_u = \phi T_u \left(d - \dfrac{a}{2}\right) = 0.9(27.04)\dfrac{2.92}{12}$

$M_u = 5.92$ ft-k

Moment provided = 5.92 ft-k > Moment required = 4.62 ft-k OK

Capacity at Typical Interior Midspan:

Use #4 @ 18 in. o/c $A_s = 0.133$ sq in./ft

$F_{ps} = 10.90$ k

$A_s f_y = 0.133 \times 60 = 7.98$ k

$T_u = 10.90 + 7.98 = 18.88$ k

$a = \dfrac{18.88}{0.85 \times 4 \times 12} = 0.46$ in.

$\left(d - \dfrac{a}{2}\right) = 4.50 - 0.46/2 = 4.27$ in.

$M_u = \phi T_u \left(d - \dfrac{a}{2}\right) = 0.9 \times 18.88 \times 4.27/12$

$M_u = 6.04$ ft-k

M_u provided $= 6.04$ ft-k > 3.58 ft-k required OK

Capacity at Typical Interior Support:

Use #4 @ 18 in. o/c $A_s = 0.133$ sq in./ft

$\left(d - \dfrac{a}{2}\right) = 3.25 - \dfrac{0.46}{2} = 3.02$ in.

$M_u = \phi T_u \left(d - \dfrac{a}{2}\right) = 0.9 \times 18.88 \times 3.02/12$

$M_u = 4.27$ ft-k

M_u provided $= 4.27$ ft-k $> M_u$ required $= 3.76$ ft-k OK

Limit load capacity of typical interior span assuming full moment redistribution –
(Note: ACI 318 limits amount of redistribution to 20%)

M_u required = 0.163 (18 – 14/12)²/8 = 5.77 ft-k

M_n provided = 4.27 + 6.04 = 10.31 ft-k > 5.77 ft-k

Capacity provided is more than adequate. Critical sections have been checked above using ACI Code minimums for non-prestressed reinforcement. All of the sections have been found to be adequate with no redistribution. Note that bonded reinforcement must be lapped in accordance with Chapter 12 of ACI 318 since the reinforcement is used in meeting the capacity requirements.

7.2.11 Check Shear Capacity at Grid 2

$$V_u = \frac{w\left(L - b/2\right)}{2} + \frac{M_u}{L}$$

$$V_u = \frac{0.163(18 - 14/12)}{2} + \frac{4.78}{18} = 1.64 \text{ k}$$

$$v_c = 2\sqrt{f'_c} = 0.126 \text{ ksi}$$

$$\phi V_n = \phi v_c bd = 0.75 \times 0.126 \times 12 \times 3.25$$

$$= 3.69 > 1.64 \text{ OK}$$

Shear strength is more than adequate without checking further since V_u need not be taken as less than $2\sqrt{f'_c}$ under the provisions of ACI 318-02. In cases where applied shear stresses are higher (rare for one-way slabs) permissible shear stress may be evaluated by use of Eqn. (11-9) or Eqns. (11-10) and (11-12) of ACI 318-02.

7.2.12 Deflection

Computed deflections need to satisfy Section 9.5.4 of ACI 318-02. For parking structure slabs, which typically do not have any partition walls, immediate live load deflection must be limited to L /360.

7.2.13 Transfer Condition

The stresses in members need to be checked at transfer condition according to Section 18.4.1 of ACI 318-02.

7.3 DESIGN OF A TWO-WAY SLAB

Design typical transverse strip as described in Fig. 7. 4 and 7.5 drawings and calculations below.

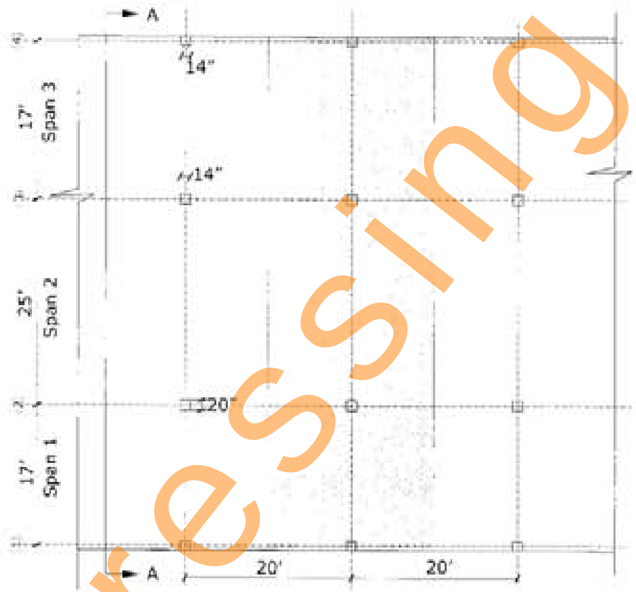

Fig. 7.4 Partial Plan of a Two-Way Flat Slab

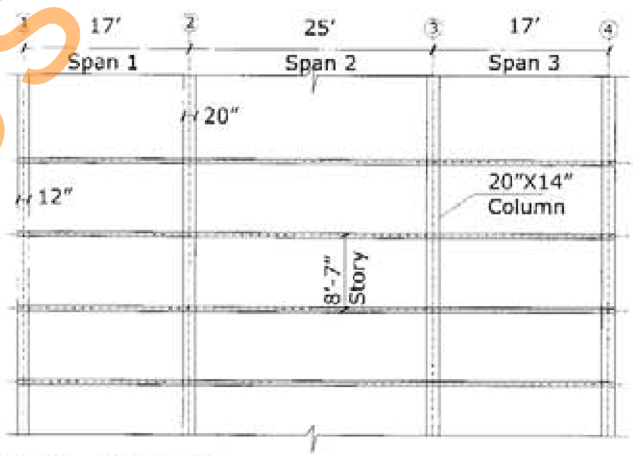

Fig. 7.5 Section A-A

7.3.1 Given Information

f'_c = 4000 psi

w = 150 pcf (slab and column)

f_y = 60,000 psi

f_{pu} = 270,000 psi

Live load = 40 psf

Partition load = 15 psf

7.3.2 Determine Slab Thickness

From Table 9.3 (See Chapter 9) for a two-way slab
Span / depth = L / 45

$$\text{Longitudinal} = \frac{20 \times 12}{45} = 5.3 \text{ in.}$$

$$\text{Transverse} = \frac{25 \times 12}{45} = 6.7 \text{ in.}$$

Use 6- ½ in. thick slab

7.3.3 Determine Loads

7.3.3.1 Dead Loads

Self weight	6.5/12 × 150	= 81 psf
Partitions		= <u>15 psf</u>
Total dead load	81 + 15	= 96 psf

7.3.3.2 Live Loads

Reduce live loads in accordance with IBC 2000, Section 1607.9.2

Span 2:

$$\text{Live load} = 40 \times \left(1 - \frac{0.08(20 \times 25 - 150)}{100}\right) = 29 \text{ psf}$$

Factored dead load	= 96 × 1.2 = 115 psf
Factored live load	= 29 × 1.6 = 47 psf
Total load	= 125 psf, unfactored
	= 162 psf, factored

Spans 1 and 3:

$$\text{Live load} = 40 \times \left(1 - \frac{0.08(20 \times 17 - 150)}{100}\right) = 34 \text{ psf}$$

Factored dead load	= 1.2 × 96 = 115 psf
Factored live load	= 1.6 × 34 = 55 psf
Total load	= 130 psf, unfactored
	= 170 psf, factored

7.3.4 Design Procedure

Assume a set of loads to be balanced by parabolic tendons. Analyze an equivalent frame subjected to the net downward loads, according to the principles of ACI 318-02, Section 13.7. Check flexural stresses at critical sections and revise load balancing tendon forces as required to obtain new flexural tension stresses in accordance with ACI 318-02, Sections 18.3.3 and 18.4.

When final forces are determined, obtain frame moments for factored dead and live loads. Calculate secondary moments induced in the frame by post-tensioning forces, and combine with factored load moments to obtain design moments. Provide minimum mild-steel reinforcement in accordance with ACI 318-02 Section 18.9. Check flexural capacity and increase mild steel if required by strength criteria. Investigate shear strength, including shear due to vertical load and due to moment transfer by torsion; compare total to allowable values calculated in accordance with ACI 318-02 Section 11.12.2.

7.3.5 Load Balancing

Arbitrarily, assume the tendons will balance 80% of the slab weight (0.8 × 0.081 = 0.065 ksf) in the controlling span (Span 2), with a parabolic tendon profile of maximum permissible sag, for the initial estimate of the required prestress force F_e.

Maximum tendon sag in Span 2:

$$a_{int} = 6.5 - 1 - 1 = 4.5 \text{ in.}$$

$$F_e = \frac{w_{bal} L^2}{8a} = \frac{0.065 \times 25^2}{8 \times 4.5/12} = 13.5 \text{ k/ft}$$

Assuming ½ in. diameter, 270 ksi strand tendons and 14 ksi long term losses, effective force per tendon is:

0.153 × (0.7 × 270 − 14) = 26.8 k

For a 20 foot bay, 20 × 13.5/26.8 = 10.1 tendons, say use 10 tendons.

Then:

F_e = 10 × 26.8/20 = 13.4 k/ft

F/A = 13.4/(6.5 × 12) = 0.172 ksi

7.3.6 Tendon Profile

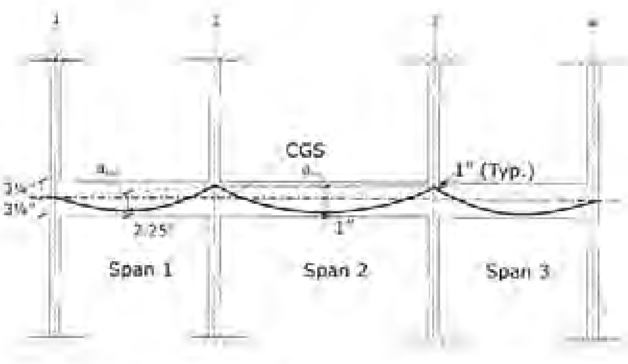

Fig. 7.6 Initial Tendon Profile Layout (Measured to CGS)

Establish Tendon Profile –
Actual balanced load in Span 2:

$$w_{bal} = \frac{8F_e a}{L^2} = \frac{8 \times 13.4 \times \left(4.5/12\right)}{25^2}$$

$$w_{bal} = 0.064 \text{ ksf}$$

Adjust tendon profile in Span 1 and 3 to balance the same load as in Span 2:

$$a = \frac{w_{bal}L^2}{8F_e} = \frac{0.064 \times 17^2}{8 \times 13.4} \times 12 = 2.1 \text{ in.}$$

Midspan CGS = (3.25 + 5.5)/2 − 2.1 = 2.275 in. say 2.25 in.

Actual sag in Spans 1 and 3 = (3.25 + 5.5)/2 − 2.25 = 2.125 in.

Actual balanced load in Spans 1 and 3:

$$w_{bal} = \frac{8F_e a}{L^2} = \frac{8 \times 13.4 \times \left(2.125/12\right)}{17^2}$$

$$w_{bal} = 0.066 \text{ ksf}$$

Net Load Causing Bending –

Span 2:

$$W_{net} = 0.125 - 0.064 = 0.061 \text{ ksf}$$

For Spans 1 and 3:

$$W_{net} = 0.130 - 0.066 = 0.064 \text{ ksf}$$

7.3.7 Equivalent Frame properties

See ACI 318-02, Section 13.7 for detailed discussion on equivalent frame properties.

7.3.7.1 Column Stiffness

The basic stiffness of columns, including the effects of "infinite" stiffness at joints may be calculated by classical methods or by simplified methods which are in close agreement. The following formula for "approximate" stiffness is taken from *Equivalent Frames of Reinforced Concrete* by Cross and Morgan.[7.5]

$$K_c = \frac{EI}{L'}\left[1 + 3\left(\frac{L}{L'}\right)^2\right]$$

Where I is taken at the column, L is center to center height and L′ is clear height.

Carryover factors are approximated by $\frac{1}{2}(1 + 3h)$ where h is the length of infinite I.

A simpler approximation is shown by Rice and Hoffman in *Structural Design Guide to the ACI Building Code.*[7.6]

$$K_c = \frac{4EI}{L - 2h}$$

The approximate formulae give results within five percent of the "exact" values, and considering the nature of assumptions necessary for design of the highly complex two way flat plate, these formulae are completely adequate. Refer to Rice and Hoffman[7.6] for a comparison of approximate and classical methods.

Exterior column:

$$I = \frac{14 \times 12^3}{12} = 2016 \text{ in}^4$$

$$\frac{E_{col}}{E_{slab}} = 1.0$$

$$K_c = \frac{4 \times 1.0 \times 2016}{103 - 2 \times 6.5} = 90 \times 2 = 180 \text{ in}^3 \text{(Joint total)}$$

Torsional stiffness of slab in column line, K_t, is calculated as follows:

$$C = \left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$

$$C = \left(1 - 0.63\frac{6.5}{12}\right)\frac{(6.5)^3 \times 12}{3} = 724 \text{ in}^4$$

$$K_t = \frac{9 \times C \times E}{L_2 \times (1 - c_2/L_2)^3}$$

Where:

c_2 = width of column

L_2 = tributary width

$$K_t = \frac{9 \times 724 \times 1}{20 \times 12(1 - 1.17/20)^3} = 32.5 \text{ in}^3$$

$$\sum K_t = 2 \times 32.5 = 65 \text{ in.}^3 \text{ (Joint total)}$$

Equivalent column stiffness is then obtained:

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_t} + \frac{1}{\sum K_c}$$

$$K_{ec} = (1/65 + 1/180)^{-1} = 48 \text{ in}^3$$

Interior column = 14 × 20

$$I = \frac{14 \times (20)^3}{12} = 9333 \text{ in}^4$$

$$K_c = \frac{4 \times 1 \times 9333}{103 - 2 \times 6.5} = 415 \text{ in}^3$$

$$\sum K_c = 2 \times 415 = 830 \text{ in}^3 \text{ (joint total)}$$

$$C = \left(1 - 0.63 \times {}^{6.5}\!/_{20}\right) \times \frac{(6.5)^3 \times 20}{3} = 1456 \text{ in}^4$$

$$K_t = \frac{9 \times 1456 \times 1}{240\left(1 - {}^{1.17}\!/_{20}\right)^3} = 65 \text{ in}^3$$

$$\sum K_t = 2 \times 65 = 130 \text{ in}^3 \text{ (joint total)}$$

$$K_{ec} = \left(\frac{1}{130} + \frac{1}{830}\right)^{-1} = 112 \text{ in}^3$$

7.3.7.2 Slab Stiffness

See Rice and Hoffman[7.6] for a full discussion on the calculation of equivalent slab stiffness.

width of slab-beam = 20/2 + 20/2 = 20 ft

$$K_s = \frac{4EI}{L_1 - c_1 / 2}$$

Where:

L_1 = centerline span

c_1 = column depth

At exterior column:

$$K_s = \frac{4 \times 1 \times 20 \times (6.5)^3}{12 \times 17 - 12/2} = 111 \text{ in}^3$$

At interior column spans 1 and 3:

$$K_s = \frac{4 \times 1 \times 20 \times (6.5)^3}{(12 \times 17 - 20/2)} = 113 \text{ in}^3$$

At interior column (span 2):

$$K_s = \frac{4 \times 1 \times 20 \times (6.5)^3}{(12 \times 25 - 20/2)} = 76 \text{ in}^3$$

7.3.7.3 Distribution Factors for Moment Distribution

Slab distribution factor at exterior joint:

= 111/(111 + 48) = 0.70

At interior joints for spans 1, 3:

= 113/(113 + 76 + 112) = 0.37

Span 2:

= 76/301 = 0.25

7.3.8 Moment Distribution - Net Loads

Since the nonprismatic section causes small effects on fixed end moments and carry over factors, fixed end moments will be calculated from $wl^2/12$ and carry over factors taken as 1/2.

Span 1, 3 net load FEM = 0.064 × 17^2/12 = 1.54 ft-k

Span 2 FEM = 0.061 × 25^2/12 = 3.18 ft-k

Table 7.10 - Moment Distribution – Net Loads

	1	2	
DF	0.70	0.37	0.25
FEM	-1.54	-1.54	-3.18
Dist.	+1.08	-0.61	+0.41
Carry-over	+0.31	-0.54	-0.21
Dist.	-0.22	+0.12	-0.08
Sum	-0.37	-2.57	-3.06

Moments shown have units in ft-k

7.3.9 Check Net Tensile Stresses

7.3.9.1 At Face of Column 2

Moment at column face = center line moment + Vc/3

$$-M_{max} = -3.06 + \frac{1}{3}\left(\frac{0.061 \times 25}{2}\right)\frac{20}{12}$$

$$-M_{max} = -2.64 \text{ ft-k}$$

$$S = bh^2/6 = 12 \times 6.5^2/6 = 84.5 \text{ in}^3$$

$$f_{t,b} = -f_{pc} \pm \frac{M_{net}}{S_{t,b}} = -0.172 \pm \frac{12 \times 2.64}{84.5}$$

$$f_{t,b} = +0.203, -0.547 \text{ (ksi)}$$

$$\left.\begin{array}{l}\text{Allowable tension} = 7.5\sqrt{f'_c} = 0.474 \text{ ksi} \\ (6\sqrt{f'_c} = .380 \text{ ksi in ACI 318-2005})\end{array}\right| \begin{array}{l} < 0.203 \text{ ksi} \\ \text{OK}\end{array}$$

Allowable compression under total load =

$0.6f'_c$ = 0.6 × 4000 = 2.4 ksi > 0.547 ksi OK

Allowable compression under sustained loads =

0.45 × 4000 = 1.8 ksi > 0.547 ksi OK

7.3.9.2 At Midspan of Span 2

$+M_{max}$ = (0.061 × 25^2/8) – 3.18 = +1.59 ft-k

$$f_{t,b} = -f_{pc} \mp \frac{M_{net}}{S_{t,b}} = -0.172 \mp \frac{12 \times 1.59}{84.5}$$

$$f_{t,b} = -0.398, +0.054 \text{ ksi}$$

Compression at top 0.398 ksi < 1.8 ksi allowable sustained load < 2.4 ksi allowable OK
Tension at bottom 0.054 ksi < 0.474 (0.380) ksi OK

When tensile stress exceeds $2\sqrt{f'_c}$ in positive moment areas, the total tensile force N_c must be carried by bonded reinforcement, ACI 318-02 Section 18.9.3.2.

$2\sqrt{4000} = 0.126\,\text{ksi} > 0.054\,\text{ksi}$, therefore, positive bonded reinforcement is not required. When it is, the calculation for the required amount of bonded reinforcement is done as follows.

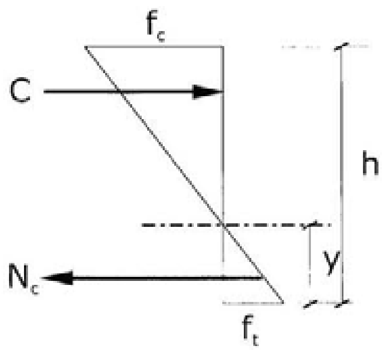

Fig. 7.7 Stress Distribution for Bonded Reinforcement

$$y = \frac{f_t}{f_t + f_c} h$$

$$N_c = \frac{12y\, f_t}{2} \quad \text{k/ft}$$

$$A_s = \frac{N_c}{0.5 f_y} \quad \text{in}^2/\text{ft}$$

Determine minimum bar lengths for this reinforcement in accordance with 18.9.4 (Note that conformance to Chapter 12 is also required).

Calculate deflections under total loads using usual elastic methods and gross cross section properties according to section 9.5.4 of ACI 318-02. Limit computed deflections to those specified in Table 9.5(b) of ACI 318-02.

This completes the service load portion of the design.

7.3.10 Flexural Capacity

7.3.10.1 Calculation of Design Moments

Design moments for statically indeterminate post-tensioned members are determined by combining frame moments due to factored dead and live loads with secondary moments induced into the frame by the tendons. The load balancing approach directly includes both primary and secondary effects, so that for service conditions only "net loads" need be considered.

At design load, the balanced load moments are used to determine secondary moments by subtracting the primary moment, which is simply F × e, at each support. For multistory buildings where typical vertical load design is combined with varying moments due to lateral loading, an efficient design approach would be to analyze the equivalent frame under each case of dead, live, balanced, and lateral loads, and combine these cases for each design condition with appropriate load factors. For this example, the balanced load moments are determined by moment distribution as follows:

For span 1 and 3 balanced load:

FEM = 0.066 × $17^2/12$ = 1.59 ft-k

For span 2 balanced load:

FEM = 0.064 × $25^2/12$ = 3.33 ft-k

Table 7.11 - Moment Distribution Balanced Loads:

	1	2	
DF	0.70	0.37	0.25
FEM	+1.59	+1.59	+3.33
Dist.	-1.11	+0.64	-0.44
Carry-over	-0.32	+0.56	+0.22
Dist.	+0.22	-0.13	+0.09
Sum	+0.38	+2.66	+3.20

Moments shown have units in ft-k

Since balancing moment includes both primary and secondary moment, secondary moments can be calculated from the following relationship:

$$M_{bal} = M_1 + M_2 \text{ then } M_2 = M_{bal} - M_1$$

The primary moment, M_1, equals F × e at each point. ("e" is the distance between the CGS and CGC)

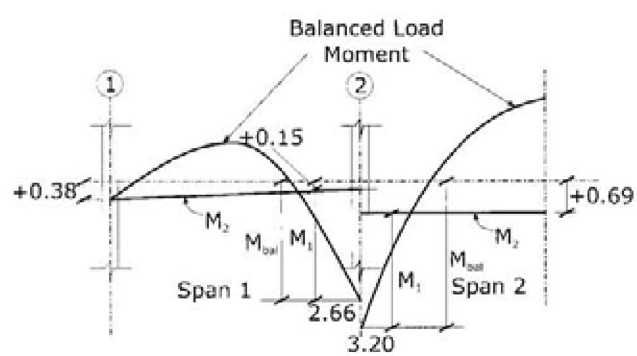

Fig. 7.8 Calculation of Secondary Moments

The secondary moment, M_2 are:
At Exterior Column –

$M_2 = 0.38 - 13.4 \times 0/12 = 0.38$ ft-k

At Interior Column –
Span 1 and 3:

$M_2 = 2.66 - 13.4 \times (3.25 - 1.0)/12 = 0.15$ ft-k

Span 2:

$M_2 = 3.20 - 13.4 \times 2.25/12 = 0.69$ ft-k

Factored Load Moments:

1.2 dead + 1.6 live

Spans 1 and 3:

FEM = $0.170 \times 17^2/12 = 4.09$ ft-k

Span 2:

FEM = $0.162 \times 25^2/12 = 8.44$ ft-k

Table 7.12 - Moment Distribution of Factored Loads:

	1	2	
DF	0.70	0.37	0.25
FEM	-4.09	-4.09	-8.44
Dist.	+2.86	-1.61	+1.09
Carry-over	+0.81	-1.43	-0.55
Dist.	-0.57	+0.33	-0.22
Sum	-0.99	-6.80	-8.12

Moments shown have units in ft-k

Combine Factored Load and Secondary Moments to Obtain Design Moments:

Table 7.13 - Design Moments at Face of Column

	1		2
Factored Moment	-0.99	-6.80	-8.12
Secondary Moment	+0.38	+0.15	+0.69
Moment at Col. C/L	-0.61	-6.65	-7.43
Moment reduction to Col. Face	+0.48	+0.8	+1.13
Design Moment at Col. Face	-0.13	-5.85	-6.30

Moments shown have units in ft-k

Calculate Design Midspan Moments –

Span 1:

$$V_{ext} = \frac{0.170 \times 17}{2} - \frac{(6.65 - 0.61)}{17}$$

$$= 1.45 - 0.36 = 1.09 \text{ k/ft}$$

$$V_{int} = 1.45 + 0.36 = 1.81 \text{ k/ft}$$

Point of zero shear and maximum moment:

$x = 1.09/.170 = 6.42$ ft from centerline of exterior column

End span positive moment:

$M_{max} = 0.5 \times 1.09 \times 6.42 - 0.61 = 2.89$ ft-k/ft

Span 2:

$V = 0.162 \times 25/2 = 2.03$ k/ft

$M_{max} = -7.43 + 0.5 \times 2.03 \times 12.5 = 5.26$ ft-k/ft

7.3.10.2 Flexural Strength

Capacity Check at Interior Support:
Section 18.9.3.3 of ACI 318-02 requires a minimum amount of bonded reinforcement in negative moment areas at column supports regardless of service load stress levels. More than the minimum may be required for flexural strength. The minimum amount is to help ensure flexural continuity and ductility, and to control cracking due to overload, temperature or shrinkage.

$A_s = 0.00075A_{cf}$

Where:

A_{cf} = larger cross-sectional area of the slab-beam strips of the two orthogonal directional equivalent frames intersection at a column of a two-way slab

$A_s = 0.00075 \times 6.5 \times (17 + 25)/2 \times 12 = 1.23$ in^2

Try 6- #4 Bars, spaced at 6 in. o.c. so that bars are placed within a width of column plus 1-½ slab thickness on either side of column.

Bar length = $2 \times (25 - 20/12)/6 + 20/12 = 9$ ft-5 in.

For average one-foot strip

$A_s = 6 \times .20/20 = 0.06$ in^2/ft

Initial check of flexural strength will be made considering this reinforcement.

Calculate design stress in tendon, use ACI 318-02 equation (18-5):

$$f_{ps} = f_{pe} + 10,000 + \frac{f'_c}{300\rho_p}$$

since we have 10- ½ in. tendons in a 20 ft bay, each with area = 0.153 in^2

$$\rho_p = \frac{A_{ps}}{bd} = \frac{10 \times 0.153}{20 \times 12 \times 5.5} = 0.00116$$

$$f_{se} = 0.7 \times 270 - 14 = 175 \text{ ksi}$$

$$f_{ps} = 175 + 10 + \frac{4}{0.00116 \times 300}$$

$$= 175 + 10 + 12 = 197 \text{ ksi}$$

f_{ps} shall not be taken greater than

$f_{py} = 0.85\ f_{pu} = 230$ ksi $>$ 197 ksi or

$f_{se} + 30 = 205$ ksi $>$ 197 ksi OK

$$F_{su} = \frac{197 \times 0.153 \times 10}{20} = 15.07 \text{ k/ft}$$

$$F_u = 60 \times 0.06 = 3.6 \text{ k/ft}$$

$$F = \text{Total Force} = 18.67 \text{ k/ft}$$

$$\text{depth of compression block } a = \frac{F}{0.85bf'_c}$$

$$a = \frac{18.67}{0.85 \times 12 \times 4} = 0.46 \text{ in.}$$

$$\varepsilon_t = (5.5 - 0.54) \times 0.003/(0.46/0.85) = 0.028$$

Assume reinforcing bars and tendons to be in the same layer:

$$(d - a/2) = (5.5 - 0.46/2)/12 = 0.44 \text{ ft}$$

Moment capacity at column centerline:

$$\phi M_n = 0.9 \times 0.44 \times 18.67 = 7.39 \text{ ft-k/ft} > 6.30 \text{ ft-k/ft OK}$$

Since there is excess negative moment capacity available, use moment redistribution to increase the negative moment and minimize the positive moment demand in Span 2. Note that the actual inelastic moment redistribution occurs at the positive moment section of Span 2.

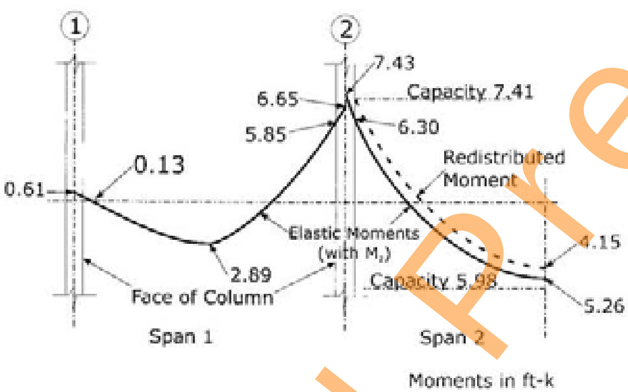

Fig. 7.9 Design Moments

Calculate available capacity at midspan and allowable inelastic moment redistribution at column. See ACI 318-02 Section 18.10.4.1 and 8.4.

Permissible change in negative moment =

$$1000\ \varepsilon_t = 1000\ (0.028) = 28\% > 20\% \text{ max}$$

Available increase in negative moment =

$$0.2 \times 6.30 = 1.26 \text{ ft-k/ft}$$

Actual increase in negative moment capacity =

Maximum capacity – Elastic negative moment

$$= 7.41 - 6.30 = 1.11 \text{ ft-k/ft} < 1.26 \text{ ft-k/ft available OK}$$

Minimum design positive moment in Span 2 =

$$5.26 - 1.11 = 4.15 \text{ ft-k/ft}$$

Capacity at midspan of Span 2:

$$A_{ps}f_{ps} = 15.10 \text{ k/ft}$$

$$a = \frac{15.10}{0.85 \times 12 \times 4} = 0.37 \text{ in.}$$

$$\left(d - \frac{a}{2}\right) = \left(5.5 - \frac{0.37}{2}\right)/12 = 0.44 \text{ ft}$$

Moment capacity at center of span:

$$\phi M_n = 0.9 \times 15.1 \times 0.44 = 5.98 \text{ ft-k/ft} > 4.15 \text{ ft-k/ft}$$

OK at midspan

$$\left(d - \frac{a}{2}\right) = \left((6.5 - 2.25) - \frac{0.37}{2}\right)/12 = 0.39 \text{ ft}$$

Check positive moment capacity in Span 1:

$$\phi M_n = 0.9 \times 15.1 \times 0.39 = 5.30 \text{ ft-k/ft} > 2.89 \text{ ft-k/ft}$$

OK at midspan

Exterior Columns:

$$A_{smin} = 0.00075 \times 20 \times 12 \times 6.5 = 1.17 \text{ in}^2$$

Use 6- #4 bars

$$A_s = 6 \times 0.20/20 = 0.06 \text{ in}^2\text{/ft}$$

$$A_sf_y = 0.06 \times 60 = 3.6 \text{ k/ft}$$

$$\rho_p = \frac{A_{ps}}{bd} = \frac{10 \times 0.153}{20 \times 12 \times 3.25} = 0.00196$$

$$f_{se} = 0.7 \times 270 - 14 = 175 \text{ ksi}$$

$$f_{ps} = 175 + 10 + \frac{4}{0.00196 \times 300}$$

$$= 175 + 10 + 7 = 192 \text{ ksi}$$

$$A_{ps}f_{ps} = 10 \times 0.153 \times 192/20 = 14.7 \text{ k/ft}$$

$$a = \frac{14.7 + 3.6}{0.85 \times 12 \times 4} = 0.45 \text{ in.}$$

Tendons :

$$\left(d - \frac{a}{2}\right) = \left(3.25 - \frac{0.45}{2}\right)/12 = 0.25 \text{ ft}$$

Rebar :

$$\left(d - \frac{a}{2}\right) = \left(5.5 - \frac{0.45}{2}\right)/12 = 0.44 \text{ ft}$$

$$\phi M_n = 0.9 \times (14.7 \times 0.25 + 3.6 \times 0.44) =$$

$$4.73 \text{ ft-k/ft} > 0.13 \text{ ft-k/ft OK}$$

This completes the flexural design check.

7.3.11 Shear Capacity

7.3.11.1 Shear and Moment Transfer at Exterior Column

Shear and moment transfer strength at exterior column:

$V_u = 0.170 \times 17/2 - (6.65 - 0.61)/17 = 1.09$ k/ft

Assume exterior skin is masonry and glass averaging 0.4 k/ft

Total slab shear at exterior column:

$V_u = (1.2 \times 0.4 + 1.09) \times 20 = 31.4$ k

Transferred moment $= 20 \times 0.61 = 12.2$ ft-k

(factored moment at exterior column centerline = 0.61 ft-k/ft)

Combined shear stress at inside face of critical transfer section:

Assume:

$d = 0.8 \times 6.5 = 5.2$ in.

$c_1 = 12$ in.

$c_2 = 14$ in.

$b_1 = c_1 + d/2 = 12 + 5.2/2 = 14.6$ in.

$b_2 = c_2 + d = 14 + 5.2 = 19.2$ in.

$A_c = (2b_1 + b_2)d = (2 \times 14.6 + 19.2)5.2 = 252$ in^2

$J_c/c = [2b_1d(b_1 + 2b_2) + d^3(2b_1 + b_2)/b_1]/6 =$ 1419 in^3

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\left(\frac{b_1}{b_2}\right)}} = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\left(\frac{14.6}{19.2}\right)}} = 0.37$$

$$v_u = \frac{V_u}{A_c} + \frac{\gamma_v M_u}{J/c}$$

$$v_u = \frac{31,400}{252} + \frac{0.37 \times (12.2 \times 1000 \times 12)}{1419} = 163 \text{ psi}$$

Permissible shear stress –

According to Equation (11-40) of ACI 318-02 which govern in this case:

$\phi v_n = \phi V_c / (b_o d)$

Where:

$v_c = 4\sqrt{f'_c} = 4\sqrt{4000} = 253$ psi

$\phi v_n = 0.75 \times 253 = 190$ psi > 163 psi OK

Check flexural moment transfer:

Although the flexural moment to be transferred is small, for illustrative purposes, calculate the capacity of the section of width equal to the width of the column plus 1.5 × slab thicknesses on each side. Assume that of the ten tendons required for the 20 foot bay width, three are anchored within the column cage and are bundled together across the building. This amount should be noted on the structural drawings. Besides providing flexural capacity, this prestress force will act directly on the critical section for shear and enhance shear strength. As previously shown, a minimum amount of mild steel is required at all columns. For this joint the area of rebar is:

$A_s = 0.00075 \times 6.5 \times 20 \times 12 = 1.17$ in^2

Use 6- # 4 bars, 5 ft long (including standard hook)

Calculate stress in tendons:

Effective slab width $= 14 + 2(1.5 \times 6.5) = 33.5$ in.

$$\rho_p = \frac{A_{ps}}{bd} = \frac{3 \times 0.153}{33.5 \times 3.25} = 0.0042$$

$$f_{ps} = 175 + 10 + \frac{4}{0.0042 \times 300}$$

$$= 175 + 10 + 3.2 = 188.2 \text{ ksi}$$

Prestress force in the critical width =

$3 \times 0.153 \times 188.2 = 86.4$ k

$A_s f_y = 6 \times 0.2 \times 60 = 72.0$ k

$A_{ps}f_{ps} + A_s f_y = 158.4$ k

$$a = \frac{158.4}{0.85 \times 4 \times 33.5} = 1.39 \text{ in.}$$

tendon $(d_p - a/2) = (3.25 - 1.39/2)/12 = 0.21$ ft

rebar $(d - a/2) = (5.5 - 1.39/2)/12 = 0.40$ ft

$\phi M_n = 0.9 \times (86.4 \times 0.21 + 72 \times 0.40) = 42.25$ ft-k

$$\gamma_f = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\left(\frac{b_1}{b_2}\right)}} = 0.63$$

$\gamma_f M_u = 0.63 \times 12.2 = 7.69$ ft-k << 42.25 ft-k OK

7.3.11.2 Shear and Moment Transfer at Interior Column

Shear and moment transferred at Interior Column:

Direct shear and moment to the left and right of interior columns is calculated in section 7.3.10.2 above.

Total direct shear $= (1.81 + 2.03)20 = 76.8$ k

Moment transfer $= 20(7.43 - 6.65) = 15.6$ ft-k

Combined shear stress at face of critical transfer section:

$d = 0.8 \times 6.5 = 5.2$ in.

$c_1 = 20$ in.

$c_2 = 14$ in.

$b_1 = c_1 + d = 25.2$ in.

$b_2 = c_2 + d = 19.2$ in.

$$A_c = 2(b_1 + b_2)d = 462 \text{ in}^2$$

$$J_c/c = [b_1 d(b_1 + 3b_2) + d^3]/3 = 3664 \text{ in}^3$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\left(\frac{b_1}{b_2}\right)}} = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\left(\frac{25.2}{19.2}\right)}} = 0.43$$

$$v_u = \frac{V_u}{A_c} + \frac{\gamma_v M_u}{J/c}$$

$$v_u = \frac{76,800}{462} + \frac{0.43 \times (15.6 \times 1000 \times 12)}{3664} = 188 \text{ psi}$$

Permissible shear stress –
For interior columns, calculate permissible shear stress by ACI 318-02 equation (11-36):

$$\phi v_c = \phi\left[\beta_p \sqrt{f_c'} + 0.3 f_{pc} + \frac{V_p}{b_o d}\right]$$

Where:

$$\beta_p = \left(\frac{\alpha_s d}{b_o} + 1.5\right) \text{ but not greater than 3.5}$$

$$b_o = 2\left[(20 + 5.2) + (14 + 5.2)\right] = 88.8 \text{ in.}$$

$$\alpha_s = 40 \text{ for interior columns}$$

$$d = 5.2 \text{ in.}$$

$$\beta_p = \frac{40 \times 5.2}{88.8} + 1.5 = 3.8 > 3.5; \text{ use } 3.5$$

V_p is the shear carried through the critical transfer section by the tendons. For thin slabs, the V_p term must be carefully evaluated, as field placing practices can have a great effect on the profile of the tendons through the critical section. Conservatively this term may be taken as zero.

$$\phi Vc = 0.75\left(3.5\sqrt{4000} + (0.3 \times 172)\right)$$

$$\phi Vc = 205 \text{ psi} > 188 \text{ psi OK}$$

Check moment transfer:

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\left(\frac{b_1}{b_2}\right)}} = 0.57$$

Moment transferred by flexure within width of column plus 1.5 times slab thickness on each side

$$M_{vf} = 0.57 \times 15.6 = 8.89 \text{ ft-k}$$

Effective slab width = 14 + 2(1.5 × 6.5) = 33.5 in.

Say $A_{ps}f_{ps}$ = 86.4 k (same as exterior column)

$$A_s = 0.00075 A_{cf}$$

$$= 0.00075 \times 6.5 \times (17 + 25)/2 \times 12 = 1.23 \text{ in}^2$$

Use 6- #4 bars (A_s = 1.2 in^2)

$$A_s f_y = 1.2 \times 60 = 72.0 \text{ k}$$

$$A_{ps} f_{ps} + A_s f_y = 86.4 + 72.0 = 158.4 \text{ k}$$

$$a = \frac{158.4}{0.85 \times 4 \times 33.5} = 1.39 \text{ in.}$$

$$\left(d - \frac{a}{2}\right) = \left(5.5 - \frac{1.39}{2}\right)/12 = 0.40 \text{ ft}$$

$$\phi M_n = 0.9 \times 158.4 \times 0.40 = 57 \text{ ft-k}$$

$$> 8.89 \text{ ft-k OK}$$

This completes the shear design check.

7.3.12 Distribution of Tendons

In accordance with ACI 318-02, Section 18.12.4, the 10 tendons per 20 ft bay in this design will be distributed in a group of 3 tendons directly through the Column with the remaining 7 tendons spaced at 2 ft 6 in. centers (about 4.6 times the slab thickness). Tendons in the direction perpendicular to the tendons designed in this example to be placed in a narrow band through and immediately adjacent to the columns.

7.4 DESIGN OF A SINGLE SPAN CAST-IN-PLACE T-BEAM

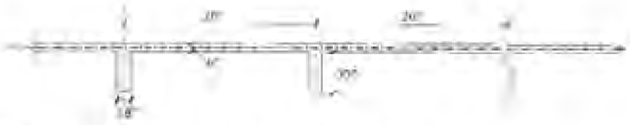

Fig. 7.10 Cross-Section of the Beam/Slab System

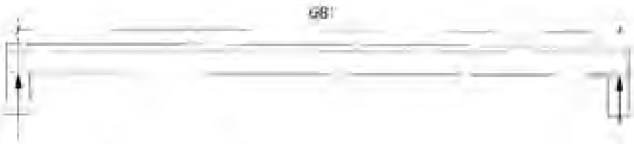

Fig. 7.11 Beam Elevation

7.4.1 Given Information

$f'_c = 5000$ psi

$f'_{ci} = 4000$ psi

$w = 150$ pcf

$f_y = 60$ ksi

Superimposed dead load = 10 psf
Live load = 40 psf
270 ksi low-relaxation strands
Tendons are bonded to the surrounding concrete.

7.4.2 Preliminary Design

7.4.2.1 Section Properties

Although not required, use ACI 318-02 Section 8.10.2 to determine the flange width:

68/4 = 17 ft

$(16 \times h) + b_w = 16 \times 6 + 18 = 114$ in.

= 9 ft-6 in. Controls

Beam spacing/2 = 20/2 = 10 ft

From Fig. 7.10 above, one can obtain the following cross-sectional properties:

$A = 1224$ in^2

$I_{gross} = 140{,}300$ in^4

$y_b = 25.06$ in.

$y_t = 10.94$ in.

$S_b = 5600$ in^3

$S_t = 12{,}820$ in^3

7.4.3 Flexural Design

7.4.3.1 External Moment and Stress Computations

Table 7.14 below summarizes external moment, bottom fiber stress, and top fiber stress computations due to self-weight and external loadings.

7.4.3.2 Compute Number of Required Post-Tensioning Tendons

Assuming a bottom concrete cover at mid-span to the center of gravity of the prestressing steel equal to 3.75 in., the eccentricity at mid-span can be calculated as follows:

$e = y_b - \text{cover} = 25.06 - 3.75 = 21.3$ in.

Design the beam per ACI 318-02 as a Class U member assuming an allowable tensile stress at the bottom fiber to be less than or equal to $6\sqrt{f'_c} = 424$ psi.

Compute the required force in the post-tensioning tendons assuming that tension at service loading conditions controls.

Table 7.14 – Summary of Service Load Moments and Stresses Due to External Loads

Load Type	Load w k/ft	Moment $w\ell^2/8$, ft-k	Top Stress ksi	Bottom Stress ksi
Slab (0.5 × 20 × 0.15)	1.5	867	-0.81	+1.86
Beam (1.5 × 2.5 × 0.15)	0.56	324	-0.30	+0.69
Superimposed (20 × 0.010)	0.20	116	-0.11	+0.25
Live Load (20 × 0.040)	0.80	462	-0.43	+0.99
Totals		1769	-1.66	+3.79

$$f_t = -\frac{F}{A} - \frac{Fe}{S_b} + \frac{M_t}{S_b}$$

Where:

M_t = 1769 ft-k (Table 7.14)

$$424 = -\frac{F}{1224} - \frac{F \times 21.3}{5600} + \frac{1769 \times 1000 \times 12}{5600}$$

F = 728,634 lbs

Compute the required number of ½ in. diameter tendons based on an assumed initial effective force of 0.7 × 270 = 189 ksi. Assume total losses of 14 ksi.

Final effective force, f_{se} = 189 – 14 = 175 ksi.

Approximate number of strands required:

$$N = \frac{728,634}{175,000 \times 0.153} = 27.21$$

Try 28- ½ in. diameter low-relaxation strands

A_{ps} = 28 × 0.153 = 4.28 in^2

7.4.3.3 Check Allowable Mid-Span Stresses at Critical Load Stages

	Stresses, ksi	
	Top	Bottom
Dead Load, beam and slab	-1.12	+2.55
Post-Tensioning initial		
F/A_g = 189 × 4.28/1224	-0.66	-0.66
Fe/S_t = 189 × 4.28 × 21.3/12,820	+1.34	
Fe/S_b = 189 × 4.28 × 21.3/5600		-3.08
1. At transfer =	-0.44	-1.19
Dead Load, beam, slab and permanent dead load	-1.23	+2.80
Post-Tensioning final		
F/A_g = 175 × 4.28/1224	-0.61	-0.61
Fe/S_t = 175 × 4.28 × 21.3/12,820	+1.30	
Fe/S_b = 175 × 4.28 × 21.3/5600		-2.85
2. Under permanent loads	-0.54	-0.66
Live load	-0.43	+0.99
3. Under full service loads	-0.97	+0.33

Maximum allowable tensile stress for Class U member =

$6\sqrt{f'_c}$ = 424 psi > 330 psi OK

7.4.3.4 Check Flexural Strength

Compute factored moment, M_u:

$$M_u = 1.2M_{DL} + 1.6M_{LL}$$
$$= 1.2 \times (867 + 324 + 116) + 1.6 \times 462$$
$$= 2308 \text{ ft-k}$$

Compute nominal moment, ϕM_n, assuming rectangular section behavior. In order to compute ϕM_n, the stress in the bonded post-tensioning tendons at nominal strength, f_{ps}, must be calculated per Eq. (18-3) in Section 18.7 of ACI 318-02:

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p}(\omega - \omega')\right]\right\}$$

(ACI 318-02 Eq. 18-3)

Assume $A_s = A_s'$ therefore:

$\omega = \omega' = 0$

Where:

γ_p = 0.28 (f_{py}/f_{pu} = 0.9)

β_1 = 0.8

ρ_p = 4.28/(114 × 32.25) = 0.0012

$$f_{ps} = 270\left(1 - \frac{0.28}{0.8}\left(0.0012 \times \frac{270}{5}\right)\right) = 264 \text{ ksi}$$

From force equilibrium:

$$a = \frac{4.28 \times 264}{0.85 \times 5 \times 114} = 2.33 \text{ in.} < 6 \text{ in.}$$

ϕM_n = 0.9 × 4.28 × 264 × (32.25 – 2.33/2)/12

ϕM_n = 2634 ft-k

ϕM_n = 2634 ft-k > 2308 ft-k OK

7.4.3.5 Check Reinforcement Limits

Check maximum reinforcing:
Check maximum reinforcing requirements per Section 18.8.1 of ACI 318-02 for ductile behavior.

$$c = \frac{a}{\beta_1} = \frac{2.33}{0.8} = 2.91 \text{ in.}$$

$$\therefore \varepsilon_t = \frac{0.003(32.25 - 2.91)}{2.91} = 0.03 > 0.005$$

$\Rightarrow$ Tension Controlled OK

Check minimum reinforcing:
In order for the amount of prestressed and nonprestressed reinforcement to be adequate, the section must develop a factored load of at least 1.2 M_{cr}.

$$M_{cr} = F\left(e + \frac{S_b}{A}\right) + f_r\,S_b$$

$$f_r = 7.5\sqrt{5000} = 530 \text{ psi}$$

$$M_{cr} = 175 \times 4.28\left(21.3 + \frac{5600}{1224}\right)$$

$$+\ 0.53 \times 5600 = 22,348 \text{ in.-k}$$

$$M_{cr} = 1862 \text{ ft-k}$$

$$1.2\ M_{cr} = 1.2 \times 1862 = 2234 < 2634 \text{ ft-k OK}$$

7.4.4 Shear Design

$$w_u = 1.2 \times (1.5 + 0.56 + 0.20) + 1.6 \times 0.8 = 4 \text{ k/ft}$$

$$V_u = w_u\,(L/2 - x)$$

$$V_u = 4\,(68/2 - 1.5) = 130 \text{ k}$$

Per Eq. (11-9) of Section 11.4 of ACI 318-02, compute V_c as follows:

$$V_c \geq 2\sqrt{f'_c}\,b_w d$$

$$V_c = \left(0.6\sqrt{f'_c} + 700\frac{V_u d}{M_u}\right) b_w d$$

$$V_c \leq 5\sqrt{f'_c}\,b_w d$$

Where:

$$\frac{V_u d}{M_u} = \frac{d(L - 2x)}{x(L - x)} \leq 1.0$$

$$\frac{V_u d}{M_u} = \frac{32.25 \times (68 - 2 \times 1.5)}{12 \times 1.5 \times (68 - 1.5)} = 1.75$$

$$\frac{V_u d}{M_u} = 1.0$$

Max limit of $5\sqrt{f'_c}\,b_w d$ governed

$$V_c = 5\sqrt{5000} \times 18 \times 32.25 = 205 \text{ k} > 130 \text{ k OK}$$

ϕ for shear per Section 9.3.2.3 of ACI 318-02 is = 0.75.

$$\frac{\phi V_c}{2} = \frac{0.75 \times 205}{2} = 77 \text{ k} < 130 \text{ k}$$

According to Section 11.5.5.1 of ACI 318-02 minimum shear reinforcement is required.

Per Eq. (11-13) of ACI 318-02:

$$A_v/s = 0.75\sqrt{5000}(18)/60,000 = 0.016 \text{ in.}$$

Per Eq. (11-14) of ACI 318-02:

$$\frac{A_v}{s} = \frac{4.28 \times 270}{80 \times 60 \times 32.25}\sqrt{\frac{32.25}{18}} = 0.010 \text{ in.}$$

A_v/s is the smaller of the above two values.

Using # 4 Grade 60 stirrups:

$$A_v = 0.40 \text{ in}^2$$

The required spacing of stirrups:

$$s = 0.4/0.01 = 40 \text{ in.}$$

Per Section 11.5.4.1 of ACI 318-02, s shall not exceed 0.75h = 27 in., nor 24 in. for prestressed concrete members. Use # 4 stirrups at 24 in. o/c throughout beam length.

7.4.5 Check Deflection

Live load deflection can be calculated as follows:

$$\delta_{LL} = \frac{5\ w_{LL}\ L^4}{384\ E\ I_{gross}}$$

$$\delta_{LL} = \frac{5 \times 20 \times 40 \times (68 \times 12)^4}{384 \times 12 \times 4.3 \times 10^6 \times 140,300}$$

$$= 0.64 \text{ in.} < L/360 \quad \text{OK}$$

7.5 ANALYSIS OF A TWO SPAN T-BEAM

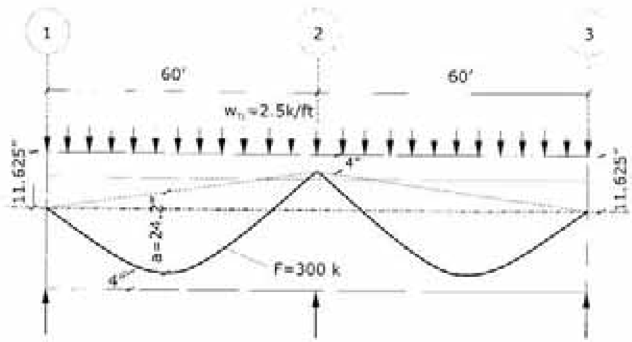

Fig. 7.12 Uniformly Loaded Two Span Continuous Beam

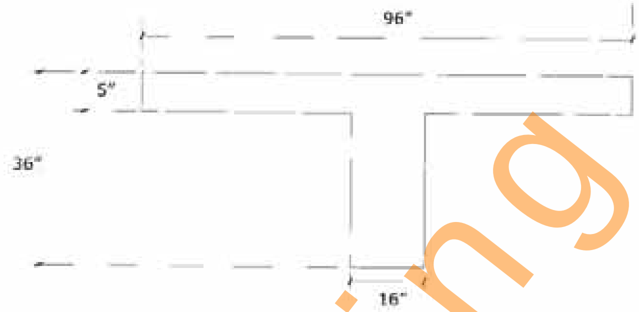

Fig. 7.13 Cross-Sectional Dimensions of the Two Span Beam

7.5.1 Given Information

For the two-span continuous unit shown in Fig. 7.12, having the cross-section shown in Fig. 7.13, determine:

- Equivalent loading
- Secondary moments
- Maximum flexural tensile stress at the bottom of the beam

Assume that the force in the post-tensioning tendons, F = 300 k, and that the tendons are placed at the center of gravity of the beam at both exterior ends. Furthermore, assume that the tendons are placed 4 inches from the extreme top and bottom of the beam.

Each span is 60 ft long, with an external total dead load of 2.5 k/ft applied along both spans, as shown in Fig. 7.12.

7.5.2 Cross-Sectional Properties

From Fig. 7.13, the cross-sectional properties can be computed as follows:

A = 976 in^2

I_{gross} = 119,756 in^4

y_b = 24.35 in.

y_t = 11.65 in.

S_b = 4918 in^3

S_t = 10,282 in^3

7.5.3 Equivalent Load and Reactions on the Tendon *FBD*

In order to compute equivalent loads and reactions, offset, a, must be first calculated (Fig. 7.12):

$$a = \frac{y_1 + y_2}{2} = \frac{(24.35 - 4) + (36 - 4 - 4)}{2}$$

$$a = 24.2 \text{ in.}$$

Equivalent load, w_p, is given by the equation below:

$$w_p = \frac{8\,F\,a}{L^2} = \frac{8 \times 300 \times 24.2}{12 \times 60^2} = 1.34 \text{ k/ft}$$

Remove tendon, and apply on it w_p as shown in Fig. 7.14. Compute the reactions P_1 and P_2 due w_p.

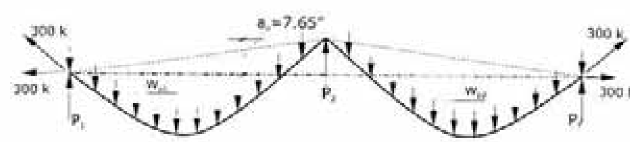

Fig. 7.14 Equivalent Loads Acting on Tendon

$$P_1 = w_p \frac{L_1}{2} - F\frac{a_o}{L_1}$$

$$P_1 = 1.34\left(\frac{60}{2}\right) - \frac{300 \times 7.65}{12 \times 60} = 37 \text{ k}$$

$$P_2 = w_p\left(\frac{L_1 + L_2}{2}\right) + F\left(\frac{a_o}{L_1} + \frac{a_o}{L_2}\right)$$

$$P_2 = 1.34\left(\frac{60 + 60}{2}\right) + 300\,\frac{7.65}{12 \times 60}$$

$$+\,300\,\frac{7.65}{12 \times 60} = 86.8 \text{ k}$$

7.5.4 Replace Tendon with Equivalent Loads

Applying the equivalent load, w_p, and its reactions, P_1 and P_2, on the beam, beam reactions R_1 and R_2 due to post-tensioning can be calculated (see Fig. 7.15). Reactions R_1 and R_2 will be used to draw the secondary moment diagram.

Note how each reaction has two components: one equilibrating the concentrated load and another equilibrating the uniform balanced load.

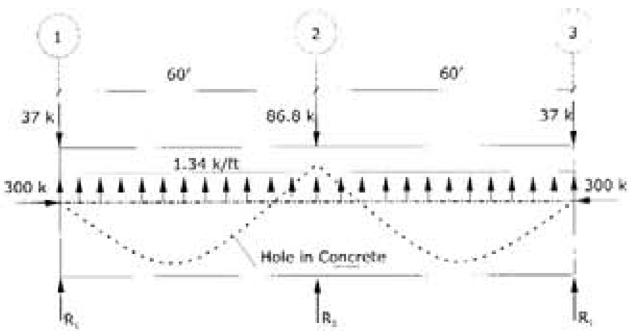

Fig. 7.15 Equivalent Loads Applied on the Beam Concrete

$$R_1 = P_1 - \frac{3}{8} w_{p1} L_1 = 37 - \frac{3}{8} 1.34 \times 60$$

$$R_1 = 6.85 \text{ k}$$

$$R_2 = P_2 - \frac{5}{4} w_{p1} \frac{L_1}{2} - \frac{5}{4} w_{p2} \frac{L_2}{2}$$

$$R_2 = 86.8 - 1.25 \times 1.34 \times 60$$

$$R_2 = -13.70 \text{ k}$$

7.5.5 Place Tendon Back Into Beam

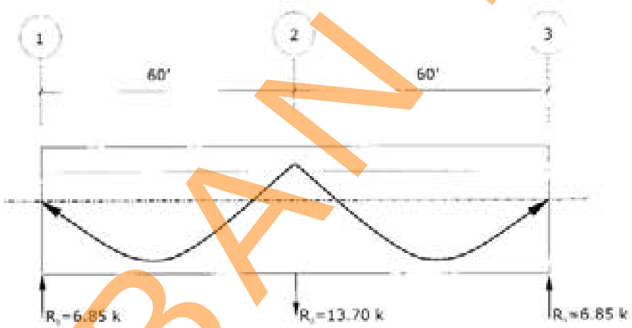

Fig. 7.16 Tendon Placed Back Into Beam

The secondary moment diagram can be drawn along the two span continuous beam using the reactions R_1 and R_2 shown in Fig 7.16.

$$M_2 = R_1 (L_1)$$

$$M_2 = 6.85 \ (60) = 411 \text{ ft-k}$$

The secondary moment diagram is shown in Fig. 7.17.

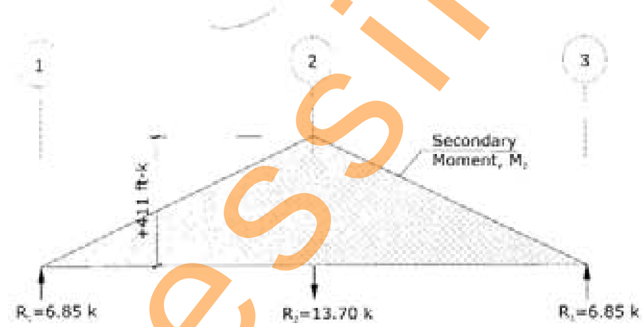

Fig. 7.17 Secondary Moment Diagram

An alternate way to compute the secondary moment M_2 is to subtract the primary moment, F_e, from the balanced moment, M_{bal} as follows:

$$M_2 = M_{bal} - Fe$$

$$M_2 = \frac{1.34\,(60)^2}{8} - 300\,\frac{7.65}{12}$$

$$M_2 = 411 \text{ ft-k}$$

7.5.6 Maximum Flexural Tensile Stress at Bottom of Beam

In order to compute the net load on the beam, the total load will be applied, as shown in Fig. 7.18, counteracted by the equivalent load. This results in a net *FBD* shown in Fig. 7.19.

In order to compute the maximum tensile stress at the bottom fiber, the location and magnitude of the maximum positive moment must first be determined. This can be accomplished using simple statics by drawing the shear and bending moment diagrams. The bending moment diagram due to net load is shown in Fig. 7.20.

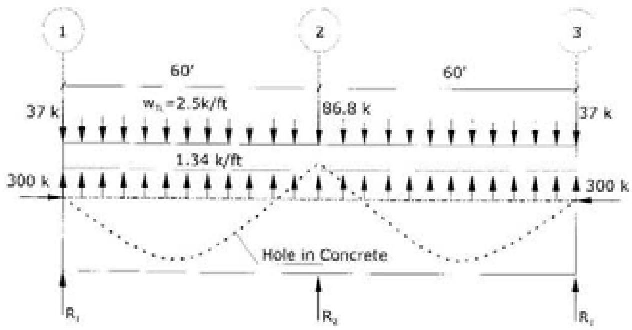

Fig. 7.18 Equivalent Load *FBD*

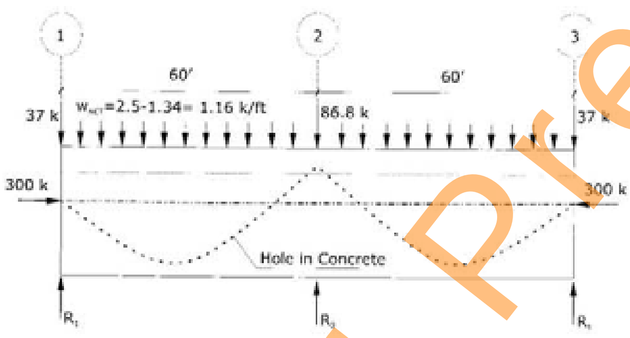

Fig. 7.19 Net Load *FBD*

For a two-span uniformly loaded beam, the maximum positive and negative moments, as well as their locations, are given by the following:

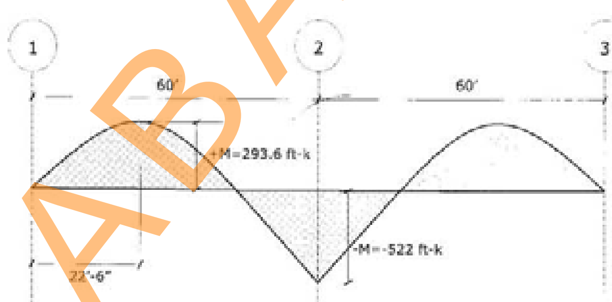

Fig. 7.20 Moment Diagram from Net Load

Maximum positive moment is calculated as:

$$M = \frac{9\,wL^2}{128} = \frac{9 \times 1.16 \times 60^2}{128}$$

$$M = 293.6 \text{ ft-k}$$

Maximum negative moment is calculated as:

$$M = \frac{wL^2}{8} = \frac{(1.16) \times 60^2}{8}$$

$$M = 522 \text{ ft-k}$$

$$x = \frac{3}{8}L = \frac{3}{8}(60)$$

$$x = 22.5 \text{ ft}$$

The maximum flexural tensile stress at the bottom fiber occurs at point of maximum positive net moment (i.e., $M = 293$ ft-k @ $x = 22.5$ ft).

$$f_t = -\frac{F}{A} + \frac{M}{S_b}$$

$$f_t = -\frac{300}{976} + \frac{293.6\,(12)}{4918} = 0.409 \text{ ksi}$$

7.6 ANCHORAGE ZONE DESIGN

Design the anchorage zone reinforcement for a wide shallow beam shown in Fig. 7.21 for a using the strut-and-tie approach.

7.6.1 Given Information

$f_{pu} = 270$ ksi

Strands: ½ in. diameter

$A_{ps} = 0.153$ in²

$f_{pj} = 0.8\, f_{pu} = 216$ ksi $\Leftrightarrow$ $Pj = 216 \times 0.153 = 33.05$ k

$f_{pi} = 0.7\, f_{pu} = 189$ ksi $\Leftrightarrow$ $Pi = 189 \times 0.153 = 28.9$ k

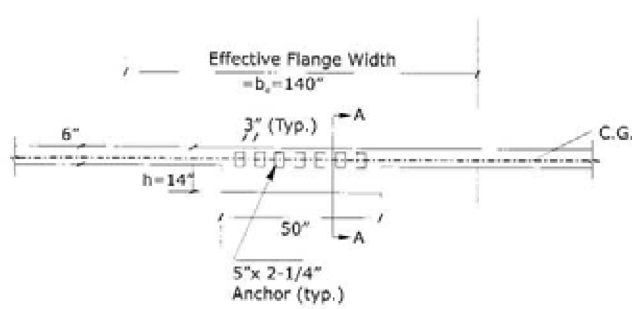

Fig. 7.21 Cross-Section at End Anchorage

7.6.2 Required

Find the required bursting reinforcement using the strut-and-tie model approach.

7.6.3 Solution

The following is assumed:

1. The resultant prestressing force on the slab and on the stem is proportional to the ratio of the cross-sectional areas of slab and stem.
2. Because the anchorage plate is placed at the centroidal axis of the T-beam, the stress distribution along the depth of the plate shall be assumed uniform.
3. Internal resisting forces shall be assumed located at the center of gravity of the slab and that of the stem.
4. The location of the tension chord is at $x = h/2$ (see Fig. 7.22).
5. The cross-sectional area of the bearing plate being subjected to internal force is proportional to the ratio of cross-sectional areas of each of the slab and stem, to the total area of the beam cross-section.

7.6.3.1 Compute Internal Forces on Slab and Stem

Per assumption No. 1:

The cross-sectional area of slab = 6 in. × 140 in. = 840 in^2

The cross-sectional area of stem = 8 in. × 50 in. = 400 in^2

$$F_{slab} = \frac{840}{840 + 400} F = 0.68\ F$$

$$F_{stem} = \frac{400}{840 + 400} F = 0.32\ F$$

7.6.3.2 Compute the Area of the Anchorage Plate Resisting Each Force

Per assumption No. 5:

The area of the bearing plate which force F_{slab} bears on = 0.68 × (5 in. × 2.25 in.) = 3.4 in. × 2.25 in.

The area of the bearing plate which force F_{stem} bears on = 0.32 × (5 in. × 2.25 in.) = 1.6 in. × 2.25 in.

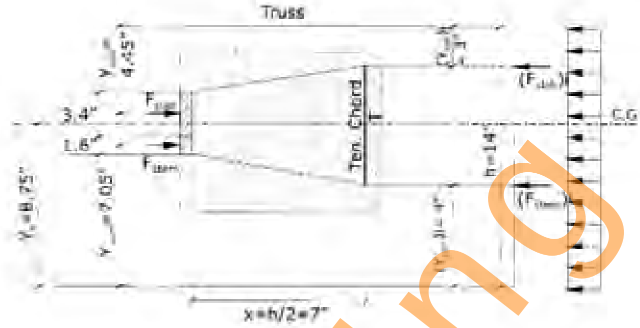

Fig. 7.22 Section AA - Strut-and-Tie Model for Shallow Beam

7.6.3.3 Determine the Location of Tension Chord, T (Fig. 7.22)

As per assumption No. 4:

$$x = \frac{h}{2} = \frac{14}{2} = 7 \text{ in.}$$

7.6.3.4 Find the Location of F_{stem}

$$y_{stem} = (y_b - L_p / 2) + \frac{1.6}{2}$$

where L_p = Plate length = 5 in.

$$y_{stem} = 8.75 - \frac{5}{2} + \frac{1.6}{2} = 7.05 \text{ in.}$$

Find the location of F_{slab} from the extreme top fiber:

$$y_{slab} = 5.25 - \frac{5}{2} + \frac{3.4}{2} = 4.45 \text{ in.}$$

7.6.3.5 Locate Internal Forces $(F_{slab})_i$ and $(F_{stem})_i$

Per assumption No. 3:

$(F_{slab})_i$ = internal force of the slab portion (Fig. 7.22)

$$(y_{slab})_i = \frac{h_{slab}}{2} = \frac{6}{2} = 3 \text{ in.}$$

$$(y_{stem})_i = \frac{8}{2} = 4 \text{ in.}$$

7.6.3.6 Compute Truss Forces

Consider the section through the truss shown in Fig. 7.23.

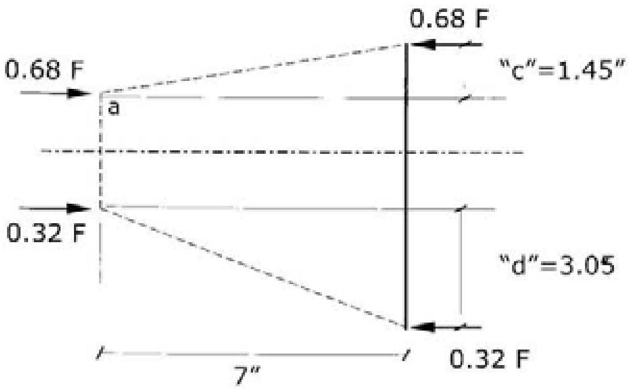

Fig. 7.23 Section Through Truss

Dimensions "c" and "d" shown in Fig. 7.23 can be determined using the calculated values of $(y_{slab})_i$, y_{slab}, $(y_{stem})_i$ and y_{stem} in steps (4) and (5).

Considering the top *FBD*. Taking moment about point "a" (Fig. 7.24).

$$0.68F \times 1.45 = T_{burst} \times 7 \text{ in.}$$

$$T_{burst} = 0.14\ F$$

Note that considering the bottom *FBD* should render the same results.

$$0.32F \times 3.05 = T_{burst} \times 7 \text{ in.}$$

$$T_{burst} = 0.14F$$

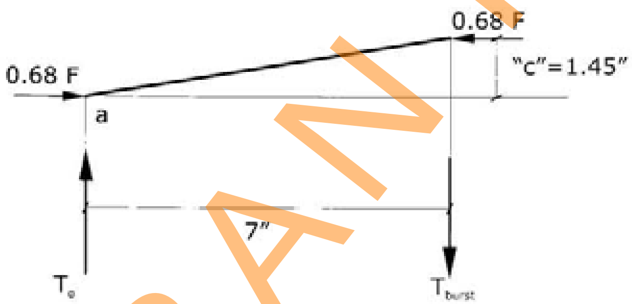

Fig. 7.24 Free Body Diagram of the Truss

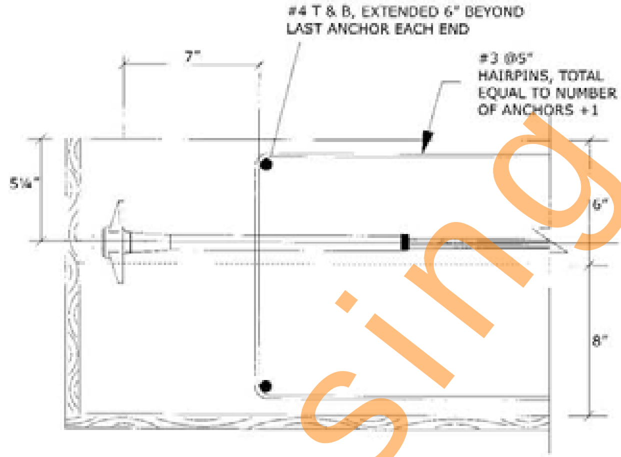

Fig. 7.25 Bursting Reinforcement for the Monostrand Shallow Beam

Compute T_{burst} using F_i

$$T_{burst} = 0.14 \times 28.9 = 4.07 \text{ k}$$

$$0.85\ f_y A_s > 1.2\ T_{burst}$$

(ϕ = 0.85; load factor = 1.2)

$$A_s \geq \frac{1.2 \times 4.07}{0.85 \times 60} = 0.10 \text{ in}^2 \text{ per tendon}$$

Use #3 hairpin stirrups at every anchor. See Fig. 7.25 for anchorage details.

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NOTATION

a Depth of the Whitney stress block, in.

a_{end} Perpendicular offset measured from the lowest point of the CGS of the post-tensioning tendons to a line connecting the highest points of the tendons above the supports for an end span, in.

a_{int} Perpendicular offset measured from the lowest point of the CGS of the post-tensioning tendons to a line connecting the highest points of the tendons above the supports for an interior span, in.

A Gross cross-sectional area of the slab or beam, in^2

A_c Area of concrete section resisting shear transfer, in^2

A_{cf} Larger gross cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, in^2

A_{ps} Cross-sectional area of the post-tensioning steel, in^2

A_s Area of longitudinal non-prestressed bonded reinforcement, in^2

A_v Area of non-prestressed shear reinforcement, in^2

A' Floor or roof area supported by a member subject to LL reduction, ft^2

b Width of the slab section, in.

b_o Perimeter of critical section for shear, in.

b_w Web thickness of a T-Beam, in.

b_1 Width of the critical section measured in the direction of the span for which the moments are determined (see Section 11.12.1.2 of ACI 318-02), in.

b_2 Width of the critical section measured in the direction perpendicular to b_1, in.

CGC Center of gravity of the concrete section considered

CGS Center of gravity of the strand

c_{AB} Distance measured from the front face of the critical section AB to the centroidal axis of the critical section C-C, in.

d Distance from extreme compressive fiber to the centroid of the non-prestressed bonded reinforcement, in.

d_p Distance from extreme compressive fiber to the centroid of the post-tensioning tendons, in.

DL Dead load, psf

e Eccentricity measured from the center of gravity of the tendon to the center of gravity of the section, in.

e_o Eccentricity at mid-span for a simply supported beam, in.

f Actual stress on the extreme fiber of a slab cross-section, psi

f_{cs} Calculated compressive stress at the extreme fiber of a slab, psi

f_{pc} Average value of the compressive stress in concrete (after allowance for prestress losses) at centroid of cross-section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, in the two directions, psi

f_{pi} Initial stress in the tendons immediately after wedge release, psi

f_{PJ} Jacking stress of the post-tensioning tendons, psi

f_{ps} Stress in the post-tensioning tendons at nominal strength, psi

f_{pu} Ultimate strength of the tendon, psi

f_{py} Yield strength of the tendon, psi

f_r Concrete modulus of rupture, psi

f_{se} Final stress in the tendons after immediate and long term losses have taken place, psi

f_{ts} Calculated tensile stress at the extreme fiber of a slab, psi

f_y Yield strength of non-prestressed bonded reinforcement, ksi

f'_c Concrete compressive strength, psi

f'_{ci} Initial concrete compressive strength, psi

Allowable tensile stress per ACI 318-02, psi (or ksi)

F Effective force in the post-tensioning tendons (i.e., after losses), lb (or kip)

F_{end} Required post-tensioning force, calculated using the equivalent load, in an end span, lb (or kip)

F_{int} Required post-tensioning force, calculated using the equivalent load, in an interior span, lb (or kip)

F_J Post-tensioning force at the jacking end of the tendon, lbs (or kip)

F_{pc} Required post-tensioning force, calculated using average compression, lb (or kip)

F_x The force at any point x along the tendons after frictional losses, lb (or kip)

h Slab thickness, in.

I_{eff} Effective moment of inertia, in^4

I_{gross} Gross moment of inertia, assuming uncracked section, in^4

J_c Property of assumed critical section analogous to polar moment of inertia, in^4

L Length of a slab panel, ft

L_x Curvilinear length of the tendon from the anchorage to distance x, ft

LL Live load, psf

m Distance between the centers of gravity of two adjacent slabs with different slab thicknesses, in.

M_b Moment due to the post-tensioning balanced load, ft-k (or in.-lb)

M_{cr} Cracking moment on the basis of the modulus of rupture, f_r, ft-k(or in.-lb)

M_n Nominal moment, ft-k (or in.-lb)

M_p Equivalent moment resulting from a tendon profile at a location where the slab thickness changes, k/ft (or in.-lb)

M_t Total service moment due to the effect of dead load, superimposed dead load, and live load, ft-k (or in.-lb)

M_u Factored moment at the section, ft-k (or in.-lb)

M_{ub} Unbalanced moment due to post-tensioning tendons, ft-k (or in.-lb)

M_{unb} Unbalanced factored moment transferred to the slab and contributing to the factored shear stress in two-way slabs, ft-k (or in.-lb)

M_1 Primary moment due to post-tensioning force, F, multiplied by the corresponding eccentricity, e, ft-k (or in.-lb)

M_2 Secondary moment due to the restraint of supports in continuous post-tensioned structures, ft-k (or in.-lb)

N Number of 7-wire strands in a beam

N_c Tensile force in concrete due to unfactored dead plus live loads, lb (or kip)

r Reduction of LL, %

S Section modulus of the slab = I_{gross}/y, in^3

S_t Top section modulus of a T-beam = I_{gross}/y_t, in^3

S_b Bottom section modulus of a T-beam = I_{gross}/y_b, in^3

s Non-prestressed reinforcement spacing, in.

Shear stress at design (factored) loads, psi

V_c Nominal shear strength of concrete, lb (or kip)

V_p Vertical component of the post-tensioning force effective in resisting shear, lb (or kip)

V_u Factored shear force at the critical section, lb (or kip)

w Uniform load, psf

w_b=w_p Balanced portion of the total dead load, psf

w_{tl} Total dead plus live load, psf

y Distance from centroidal axis of gross section to extreme fiber = $h/2$, in.

b_c Ratio of the long side or a rectangular column to the length of its short side

f Strength reduction factor per ACI 318-02

g_f Fraction of the unbalanced moment transferred by flexure at slab-column connections

g_v Fraction of the unbalanced moment transferred by eccentricity of shear at slab-column connections

SEISMIC DESIGN OF POST-TENSIONED CONCRETE STRUCTURES

8.1 INTRODUCTION

The seismic advantages of using post-tensioned concrete structures are many, including reduced seismic forces and overturning moments, reduced cracking of members subject to seismic load, and reduced residual displacements of unbonded post-tensioned wall and frame systems.

This chapter reviews the basics of seismic design for post-tensioned concrete structures with examples and references for practicing engineers new to the subject. First, the review covers the role post-tensioning plays in seismic-resisting structures, and how seismic forces are developed for post-tensioned members. Next, this chapter discusses how post-tensioned members are designed to resist seismic forces and displacements. Finally, unbonded post-tensioned wall and frame structures are reviewed.

The building code requirements presented throughout this chapter refer to the International Building Code (IBC)[8.1] for minimum regulations for building systems using prescriptive and performance-based provisions.

8.2 ROLE OF POST-TENSIONING IN SEISMIC DESIGN

The seismic advantages of using post-tensioned concrete structures include:

- Reduced seismic forces and overturning moments, which result from the reduced floor height and weight compared with non-prestressed concrete members. This is especially true of the flooring system where the weight of a post-tensioned floor slab can be up to 30% less than a non-prestressed concrete slab.
- Reduced cracking of members subject to seismic load, which reduces the amount of post-earthquake repairs. In particular, post-tensioned slabs resist diaphragm loads as an elastic system with less cracking than non-prestressed concrete structures.
- Reduced residual displacements of unbonded post-tensioned wall and frame systems. This is because the tendons, which are designed not to yield, will tend to return to an undeformed configuration upon dissipation of seismic forces.

While the primary purpose of building structures is to support gravity loads, these systems must also resist loads and deformations induced by seismic events. The horizontal component of seismic response requires analysis and design considerations separate from the gravity load design. In this situation, the vertical component is typically accounted for by increasing or decreasing the dead load, whichever produces the maximum effect. It is common for building structures to have frames designed to resist gravity loads only in conjunction with frames and/or walls that resist both gravity and lateral loads. Post-tensioning can be a part of both the gravity and the lateral-force resisting systems. However, in regions of high seismic risk, building codes limit the contribution of prestressed concrete in lateral-force resisting elements because of a perceived reduction in energy dissipation and ductility capacity over non-prestressed concrete components.[8.1]

8.2.1 Strength and Ductility of Post-Tensioned Concrete Members

The available ductility of a prestressed or a reinforced concrete beam is limited to the compression capacity of the concrete and the tension capacity of the steel. For unconfined concrete, the nominal upper limit for the compression capacity specified in building codes is given in terms of the ultimate compressive strain in the concrete, equal to 0.003. While this upper limit is satisfactory for determining the flexural strength, higher limits should be used for estimating the rotational capacities of plastic hinges.[8.2,8.3] Furthermore, the crushing strength and strain of the concrete can be increased with transverse, or confinement, reinforcement. This type of reinforcement uses closed hoops or ties to prevent the loss of strength after spalling occurs. Building codes specify a prescriptive amount of transverse hoop reinforcement within plastic hinge regions. In addition to confining the concrete, these transverse ties also prevent premature buckling of the longitudinal reinforcement.

8.2.1.1 Unbonded Tendons

The Code limits the contribution of prestressed reinforcement to 25% in the determination of strength for either positive or negative moments at nonlinear action locations.[8.1]

Assessment of the ductility requires analysis of the section at two separate limit states; initial yielding of the reinforcement and ultimate stress in the reinforcement. At first yield, the strain of the non-prestressed reinforcement has been reached, and the section is in a cracked configuration. The effect of prestress at this limit state is to increase both the maximum compressive strain and depth of the neutral axis. As a result, the yield curvature is increased with prestressed tendons. Since the tendons are not bonded to the adjacent concrete, the prestressing steel is not subject to the same strains as the concrete. In calculating the moment capacity, the stress in the prestressing steel is calculated using the ACI Code equations in Ref. 8.4. A more accurate assessment of prestressed steel stress at ultimate can be found in Ref. 8.5.

At the ultimate limit state, the maximum compressive stress is reached in the top fiber. Thus the maximum, or ultimate curvature is the compressive strain capacity divided by the neutral axis depth. The effect of prestress is to increase the neutral axis depth, and thus reduce the ultimate curvature/ rotation capacity of the plastic hinges.

This effect is recognized in building structures where the amount of prestress on the section of a frame member is limited to 720 psi (5 MPa), or $f'_c/6$ whichever is less, to account for the reduction on the ultimate curvature.[8.1]

8.2.1.2 Bonded Tendons

Bonded tendons are analyzed in a similar fashion to non-prestressed reinforcement where tension and compression strains in the concrete are assumed to be the same as the adjacent concrete fibers. Prior to grouting the ducts, the prestressing steel is free to move relative to the surrounding concrete. A moment-curvature analysis can be used to develop the moment capacity of the prestressed section, where the strain history of the tendon is required to evaluate the stress in the prestressing steel at ultimate. This strain history is necessary because there is no definite yield plateau, or yield point, to define prestress steel stress at ultimate. In lieu of a moment-curvature analysis, prescriptive equations can be used to assess the stress in the prestressing steel at ultimate in Ref. 8.6.

8.2.2 Post-Tensioned Gravity Frames

Post-tensioning is most commonly used in floor framing to resist gravity loads where it is not part of the seismic load-resisting system. In this role, post-tensioning improves the seismic performance for the following reasons:

- As discussed in Chapter 9, post-tensioning can reduce the floor weight by 30% over non-prestressed concrete floor systems. This reduction in weight results in reduced design forces to the lateral load resisting system.
- Post-tensioning can reduce floor thickness and floor to floor height; this reduces overturning moments and the associated member sizes of the lateral-force resisting system.
- Post-tensioning reduces cracking and related post-earthquake repairs.

8.2.3 Post-Tensioned Lateral Force Resisting Systems

Recent research on precast lateral-force resisting systems has shown that post-tensioning can play a primary role in seismic-resisting elements.[8.7,8.8] Post-tensioning improves the seismic performance of this system for the following reasons:

- Residual displacements after an earthquake are reduced. Because the unbonded post-tensioning rarely exceeds its elastic limit, the structure returns to its undeformed position after a seismic event.
- Under large member deformation, unbonded tendons absorb, store, and release substantial amounts of energy elastically; however they do not dissipate much energy in the form of heat or member degradation. This elastic response results in a substantial decrease in associated damage to the member.
- Using displacement-based design methods, a significant reduction in lateral base shear forces can result when compared with non-prestressed concrete systems.

8.3 POST-TENSIONED MEMBER DESIGN

Modern seismic design philosophy is predicated on the development of stable plastic hinges at preselected locations to develop a ductile response without a significant loss of strength. American building codes require that post-tensioned building members emulate reinforced concrete members to resist seismic forces. In particular, post-tensioned members designed to yield during a seismic event are required to have the same strength and ductility as their reinforced concrete counterparts. Further, these members must have essentially the same damping characteristics. Due to concerns over reduced energy dissipation of post-tensioned members and reduced ductility capacity, modern building codes restrict post-tensioning to a limited role in elements designed to develop plastic hinges under seismic loading. For moment-resisting frames in building structures, plastic hinges are designed to occur in the beams in order to spread inelastic deformations along the height of the building and reduce localized inelastic curvature demands and inter-story drift.

Post-tensioned members in gravity frames are subject to deformations that could result in nonlinear behavior during a seismic event. Therefore, post-tensioned beams and slabs that are part of these gravity frames are designed to withstand the deformations expected to occur during a seismic event, even though these elements are not included in the lateral resisting force calculations.

8.3.1 Beams

8.3.1.1 Flexure

To ensure that prescriptive detailing will result in a structure that can achieve the ductile capacity implied by response modification factors, the Code limits the contribution of post-tensioning to the flexural moment capacity to 25% of the overall resistance to seismic moment demands in beams.[8.1] Therefore, the flexural design and analysis requirements for non-prestressed reinforced concrete members apply, as listed in the following:

1. The effective overhang for T-beams is the lesser of:
   a. Eight times the slab thickness
   b. One-half the clear distance to the face of the next web
   c. The overall effective flange width cannot be greater than one-quarter of the supporting beam span
2. The effective overhang for spandrels or edge beams (beams with a slab on one side only) shall be the lesser of:
   a. Six times the slab thickness
   b. One-half the clear distance to the face of the next web
   c. One-twelfth the span length of the supporting beam

Minimum flexural strength under positive bending must be at least 50% of the negative moment capacity, and vice versa if applicable. Without this requirement, beams with non-prestressed reinforcement, or tendons with fully bonded strands, will develop residual curvature that will increase on repeating cycles. Shear transfer at the interface will also be resisted by dowel action alone as the concrete contact area deteriorates. This type of shear resistance can result in substantial displacements at the beam-column interface, and lead to premature rupture of the longitudinal reinforcement and/or prestressing steel due to large flexural strains.

Post-tensioning is designed for gravity loads, and any additional moment resistance required for seismic loading is resisted by non-prestressed reinforcement. For moment-resisting frames, the requirement for the positive moment capacity to be at least 50% of the negative bending capacity results in a large amount of bottom reinforcement that may not be entirely necessary. The moment due to prestressing is a secondary moment and it must be countered by the non-prestressed reinforcement in the bottom of the beam, in addition to the negative moments at the face of the column induced by the seismic and dead load moments. Unbonded post-tensioning remains in the elastic range, and it will compress the non-prestressed reinforcement in the negative bending region back to zero strain even when the centroid of the prestressed reinforcement is above the neutral axis.

8.3.1.2 Shear

The benefits of post-tensioning on the shear performance of concrete members are substantial, as was discussed previously in this chapter. The precompression provided by prestressing reduces the principal tensile stress in the region of maximum shear stress, which reduces cracking and hence reduces the amount of shear reinforcement required. Draped post-tensioning tendons also contribute to the shear resistance with the vertical component of the prestressed reinforcement. The Code requires that this vertical component not be used in the shear induced by seismic loads, because seismic forces are reversible. Hence, if the shear induced by seismic forces is added to the dead load, shears are increased.

8.3.2 Slabs

Post-tensioned slabs serve two purposes in the resistance of seismic forces. They resist in-plane seismic forces and distribute these forces horizontally to the lateral-force resisting systems. Also, as the structure displaces horizontally, the slab and supporting columns deform, which introduces additional forces, moment and shears, into the gravity load supporting system. Therefore, the slab must be designed to resist seismic forces resulting from lateral displacement.

8.3.2.1 Displacement Compatibility

Slabs and columns that are not part of the lateral force resisting systems are subject to seismic forces as a result of displacement compatibility. If continuity is specified between the column and the slab, these forces can be significant when large lateral drift ratios are sustained. Experiments have shown that unbonded post-tensioned slabs can sustain significant inelastic lateral drifts with a ductile and stable response.[8.9] However, building codes require that connections of the slab to vertical load resisting elements also resist seismic forces in the inelastic range. This inelastic behavior is ensured by amplifying demands by the overstrength factor Ω_0. Post-tensioning, designed to resist gravity loads, often provides this resistance without any added cost to the structure.[8.6]

In lieu of detailed finite element (continuum) modeling, the equivalent frame method can be used to develop seismic moments and associated shears acting on the slab. The width of the slab effective in resisting seismic moments is the width of the column plus one and one half (1.5) the thickness of the slab or drop panels on each side of the column.[8.6] These moments are then added to gravity loads using the appropriate load combinations, as stated in Section 8.3. Note that the slab width effective in resisting the seismic moment is different from the width that would be used for frame stiffness.

Parking structures deserve special attention, as short, relatively stiff columns are inevitably used to support inclined ramps and floor levels that meet the columns at different elevations.[8.10] If the moments and shear forces are too high, shear reinforcement, drop panels, or capitals are used. However, experimental and analytical studies have shown that capitals should not be used to improve the shear capacity for seismic applications.[8.11]

In lieu of strengthening, seismic moments acting on the slabs can be reduced with a flexural pin (also referred to as a column-release detail) at the interface between the slab and the ends of the column. Typical pin details consist of a single dowel or reinforcement bar extending through the slab into the columns. Although the intent is to release the moment, this connection develops some flexural resistance under lateral drifts, which must be accounted for. If a groove is used around the edge of the column, the effect of the reduced section on the shear capacity of the slab must be evaluated for other load cases.

8.3.2.2 Diaphragms

Post-tensioned slabs have demonstrated ample performance as a diaphragm for distribution of seismic forces.[8.12] Conservatively, diaphragms can be analyzed as beams spanning between lateral force resisting elements.[8.10,8.13] A more detailed approach using finite-element continuum models can be found in Ref. 8.12 and using the strut-and-tie models in Ref. 8.13. These more detailed methods should be used for diaphragms with irregularities, such as large perforations, and non-rectangular/square shapes.

Parking structure ramps are an example of areas that require special attention with respect to diaphragm forces. These elements need to be designed to span between the main floor diaphragms and often require additional non-prestressed chord reinforcement as well as drag reinforcement at the intersection of the main floor diaphragms.

8.3.2.3 Chord Forces

As an elastic member, post-tensioned diaphragms resist bending and shear forces. Similar to the displacement compatibility problem, post-tensioning designed for gravity load support is often sufficient to counter any tensile stress that develops as a result of flexure. This is shown in the design example later in this chapter.

For seismic design categories D, E or F, F_p is the total force the roof or floor diaphragm must resist. This force, using Eq. (16-63) from the IBC-2003, is:

$$F_p = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

Where:

F_i = The design force applied to Level i

F_{px} = The diaphragm design force

w_i = The weight tributary to Level i

w_{px} = The weight tributary to the diaphragm at level x

The force determined from the above equation need not exceed $0.4S_{DS}I_E w_{px}$ but shall not be less than $0.2S_{DS}I_E w_{px}$, where I_E is the importance factor, S_{DS} is the short period site design response acceleration, and w_{px} is the weight of the diaphragm.[8.1] If prestress reinforcement is insufficient to counter this flexural tension, chord reinforcement is required.

8.3.2.4 Drag Forces

Many design professionals utilize the precompression force in resisting drag and chord forces developed in the diaphragm. There is no consensus on what percentage of the residual precompression should be used for design. Many design firms determine the residual precompression available for seismic design by re-analyzing the vertical design using a load combination of $1.2DL + f_1 LL + 1.0E$. (E is zero in the vertical load design but a vertical seismic component needs to be added to the dead load.) A variety of factors influence the development of the precompressive forces in a diaphragm. Framing elements exist throughout a structure that provide slab restraint, thereby creating an additional pathway for precompressive loss that is not accounted for in most analyses. Another area of design that requires clarification is the effective width of slab to be used in determining the effective precompressive force to be utilized. A 45 degree distribution is assumed from the end of the shear wall, in each direction, for determining the resisting precompressive force. The following example illustrates the design process for drag members utilizing these concepts.

8.3.2.5 Design Example

The overstrength seismic load over the slab area is determined by:

$$w = \frac{\text{Seismic Force} \times \Omega_o}{\text{width} \times \text{length}}$$

$$w = \frac{2150 \times 2.5}{220 \times 150} = 163 \text{ psf}$$

For the tension zone area the force becomes:

Total Drag Force = 17,500 ft^2 × 163 psf = 2853 k

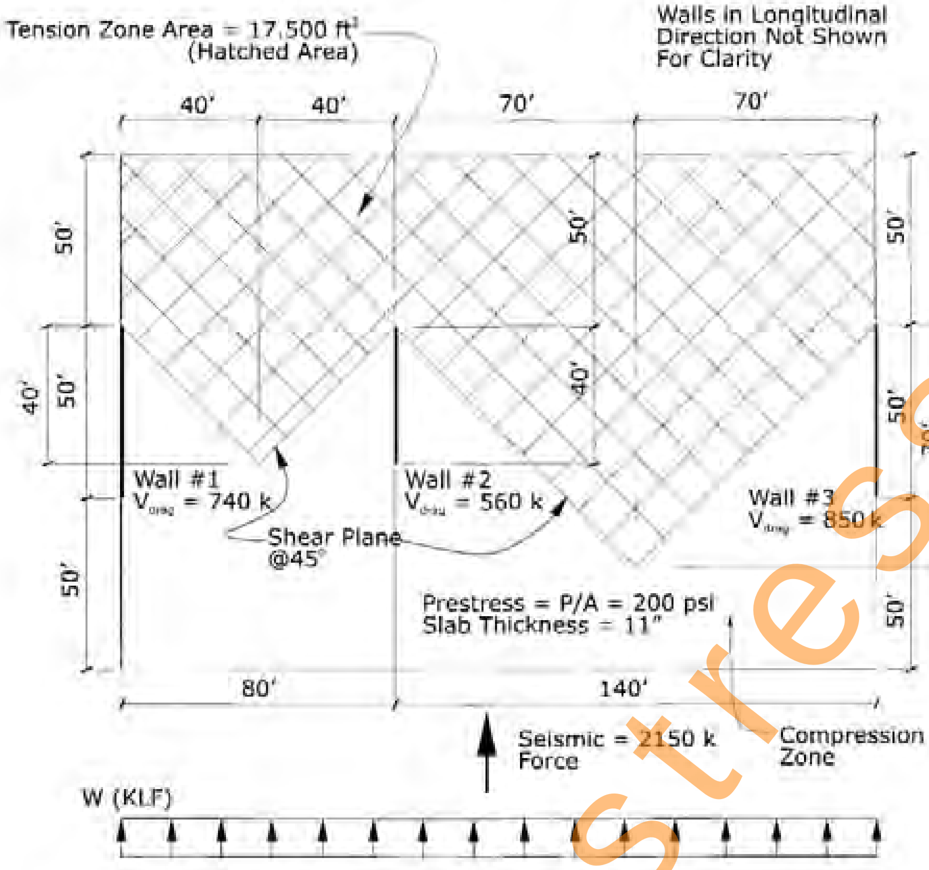

Fig. 8.1 Floor Plan Showing Diaphragm Forces in a Prestressed Slab

The overstrength force shown above may not govern for design. When the vertical distribution of the overstrength seismic forces is used in IBC Eq. (16-63), there is a cap of $0.4S_{DS}I_E w_{px}$. This cap on the diaphragm force will govern the design of both drag and shear transfer between the top of the wall and the diaphragm for many structures.

The capacity of the total shear plane is determined as follows:

$$\phi V_n = \phi 2\sqrt{f_c'} \times \sum d$$

$$\phi V_n = (0.85)(2)\sqrt{4000}(40 + 40 + 70 + 70)(12)(11)$$

$$\phi V_n = 3122 \text{ k} \geq 2853 \text{ k}$$

No Drag Reinforcement Required

If drag reinforcement had been required based on the diaphragm capacity, the residual precompression could have been used to resist the remaining net tension. After re-analyzing the vertical design using a load combination of $1.2DL + f_1LL + 1.0E$, it was determined that the residual precompression that could be used in seismic design was approximately 40% of the total average precompression.

The overall resistance is determined as follows:

Effective Prestress Force = 0.4 (Slab Thickness) (Effective Width) (Average Pre-compression)

Effective Prestress Force = 0.4 (11) (220 × 12) (0.200) = 2323 k

As can be seen from the results above, the impact of pre-compression on the seismic performance of the diaphragm can be dramatic.

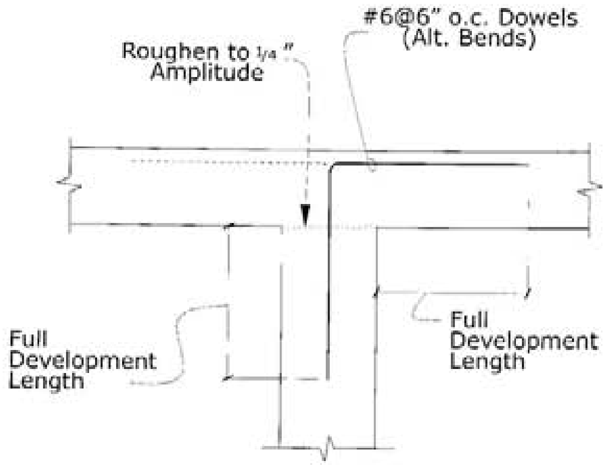

Fig. 8.2 Seismic Dowels at Slab Wall Connection

Had there been net tension at the shear interface, the tensile force would have been divided by the appropriate strength reduction factor times the reinforcement yield strength as shown in the following equation:

$$A_{REQ} = \frac{T_{NET}}{\phi f_y}$$

The drag reinforcement would have been distributed between the various shear walls based on their seismic shear coefficients.

The next step in the load path is to connect the diaphragm to the walls with dowels.

Dowel Design – Wall No. 2

Wall No. 2 – V = 560 k

$$F_{DOWEL} = \Omega_o V_{WALL\,\#2} = (2.5)(560) = 1400 \text{ k}$$

$$A_{vf} = \frac{F_{DOWEL}}{\phi \mu f_y L} = \frac{1400}{(0.85)(1)(60)(40)} = 0.69 \text{ in}^2/\text{ft}$$

∴ USE #6 @ 6 in. o.c. DOWELS

The overstrength seismic force is used in this example, but as mentioned above the governing force may be that obtained from IBC Eq. (16-63).

The overstrength seismic load does not apply to chord design. Therefore, the seismic force of 2150 k is distributed over the length of the diaphragm and the diaphragm is analyzed as a deep beam with supports at the shear wall locations. For this example, the shear walls have similar rigidities and the support points are modeled at pins. Where rigidities are vastly different between the various shear walls of a structure, it is more accurate to utilize spring supports in the direction of the seismic force when modeling the diaphragm. When modeling diaphragms, shear deformation should be included in the analysis. The resulting moment diagram is as follows:

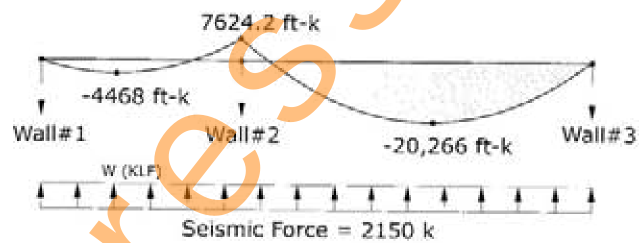

Fig. 8.3 Moment Diagram of Diaphragm Due to Lateral Loads

Units are in ft-k. As can be seen from the diagram in Fig. 8.3, the maximum moment is 20,266 ft-k.

The stress in the diaphragm at the extreme fibers is determined by:

$$\sigma = -\frac{P}{A} \pm \frac{Mc}{I}$$

$$\frac{P}{A} = 40\% \text{ of } 200 \text{ psi} = 0.08 \text{ ksi}$$

$$\sigma = -.08 - \frac{(20,266)(12)\left(\frac{1800}{2}\right)}{\frac{1}{12}(11)(1800)^3}$$

$$\sigma = -0.08 - 0.041 \text{ ksi}$$

$$\sigma = -0.121 \text{ ksi}, -0.039 \text{ ksi}$$

∴ Diaphragm remains in compression
& no chord reinforcement required

8.4 SEISMIC PERFORMANCE OF UNBONDED P/T WALL AND FRAME SYSTEMS

Currently, building codes require precast lateral-force resisting systems to emulate cast-in-place reinforced concrete, unless it can be shown by experiment that the proposed system has the same ductility and energy dissipation as an equivalent reinforced concrete system. Structural testing has shown that unbonded post-tensioned wall and frame systems can resist severe seismic forces and displacements from a design level event.[8.7] In this system, precast frame and wall elements are connected using unbonded post-tensioning and bonded non-prestressed reinforcement, or structural steel. The following is a short summary of the seismic performance of these systems. For more detailed information on the seismic design of unbonded post-tensioned wall or frame systems, see Ref. 8.8.

8.4.1 Wall Systems

Unbonded post-tensioned wall systems are designed to sustain all inelastic deformations at the base of the wall. Resistance is supplied by unbonded post-tensioning provided through the center of the wall. Precast wall elements are composed of 8 to 12 ft wide, side-by-side segments. These segments are connected with non-prestressed steel fixtures designed to yield and dissipate energy under differential movement on the vertical interface, sustained by the wall system, as the top of the building translates. Because the post-tensioning remains elastic under seismic loading, this system will return to its undeformed configuration after removal of the lateral forces.

8.4.2 Frame Systems

All inelastic deformations are designed to occur at the beam-column interface. The post-tensioning is designed to remain elastic, while yielding of the non-prestressed reinforcement provides all energy dissipation. By confining the deformations to the joints between the structural precast elements, damage to the system is confined to these regions. This reduces post-earthquake repairs to the structural frame. As with the wall system, the post-tensioning is designed to remain elastic, and therefore tends to return to its undeformed configuration after removal of the seismic forces.

With the hybrid post-tensioned frame system, the post-tensioning is centered in the beam and non-prestressed reinforcement is provided at the top and bottom of the beam. To ensure adequate bearing of the beam on the column, a thin, fiber-reinforced grout pad is poured between the beam-column interface.

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POST-TENSIONED CONCRETE FLOORS

9.1 FLOOR FRAMING SYSTEMS

In buildings, a floor system is a horizontal platform spanning between vertical supports such as columns and walls. It provides a support surface for gravity loads, and acts as a diaphragm for transferring seismic and wind loads to lateral load-resisting elements (such as shear walls, frames, and trusses). Slabs, beams, column capitals, drop panels, band beams, and any other horizontal structural elements that transfer load to supports are parts of the floor system. Commonly used floor systems include flat plates, flat slabs, beam and slab systems, waffle slabs, and joist and beam systems. These floor systems are described in more detail in Section 9.2.

The floor system is typically the costliest structural element in any given building. For low-rise buildings with few floors, the floor system represents the majority of the structural cost. As the number of floors increase, the cost of foundations, vertical elements carrying gravity loads, and lateral load-resisting systems become a larger percentage of the total structural cost. However, even in tall buildings the cost of the floor system often dominates the economics for the structure. The general relationship between the costs of the various structural elements as a function of the number of floors is shown in Fig. 9.1. More information on design differences and selecting the appropriate floor system can be found in Section 9.2.

Post-tensioned tendons are used as primary tensile reinforcement in slabs, beams, girders, and joists. The force and profile of the tendons in these members is designed to produce loads that counteract (i.e., balance) a portion of the gravity loads, which results in the reduction of flexural

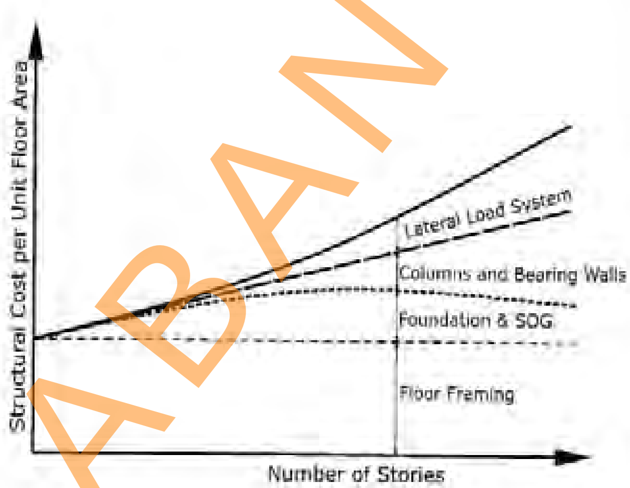


Fig. 9.1 Components of Total Structural Cost

tensile stresses and deflection. For the portion of the gravity load that is balanced, the member cross-section is under uniform axial compressive stress and has no net deflection. Deflection and bending stresses in the framing members are produced only from the unbalanced portion of the loads.

Post-tensioned tendons in floor systems can be either unbonded or bonded. In the United States, the majority of post-tensioning used in buildings is unbonded. The wide usage of unbonded post-tensioned tendons has been influenced by economics, performance, and extensive laboratory testing programs performed at various universities for more than 40 years. For the tendon size used in building structures, unbonded tendons are less expensive per pound than bonded tendons and are more compatible with the member shapes and rapid construction schedules required for most building construction.

9.1.1 Transfer Slabs and Beams

Transfer slabs (also commonly called "podium slabs") are a special type of floor system that transfers loads from elements above the slab to load-carrying members below the slab, Fig. 9.2. High loads on post-tensioned concrete podium slabs from wood-frame or steel superstructures, landscaping, and water features are common. Typically, column spacing above transfer slabs is smaller than for those below. For example, many wood-framed structures are built over parking floors, requiring a transfer of loads from bearing walls and posts to columns with larger spacing below. Beams that support discontinuous columns above and transfer their loads to other columns and/or walls below are called transfer beams, Fig. 9.3. Post-tensioning of transfer slabs and beams often results in cost savings and superior performance when compared with other framing alternatives.

Transfer slabs and beams require careful consideration of concrete flexural stresses at the time the tendons are stressed. In most cast-in-place post-tensioned floor systems the majority of the dead load is present at the time of initial tendon stressing. In that case all of the tendons can usually be stressed at one time without inducing excessive "reversal" stresses in the concrete due to high tendon balanced loads. This is often not true in transfer slabs and beams where a significant amount of the total dead load may not be present initially. In such cases the tendons may have to be stressed in stages (a process called "staged post-tensioning") as the construction of upper floors is completed. In stage stressing it is advisable to achieve the required prestress force at each stage by stressing some of the tendons to their full permissible anchor force, rather than by stressing all of the tendons to a reduced percentage of their full permissible anchor force. This avoids multiple penetrations of the wedge teeth into the strand at one location and results in improved anchorage performance.

Fig. 9.2 Post-Tensioned Podium Slab
Courtesy of Seneca Structural Engineering, Inc.

9.1.2 Penetrations and Openings

Despite significant projected initial cost savings, post-tensioned floor systems are occasionally rejected by inexperienced designers, contractors, or owners because of the perception that adding future penetrations and openings is difficult, expensive, or impossible. In fact, openings and penetrations in existing post-tensioned floor systems are commonly installed. Most experienced post-tensioning firms have the expertise to execute them, and several independent firms specialize in such work. Small penetrations are commonly drilled through slabs between known tendon locations; large openings that must interrupt tendons are regularly installed economically by knowledgeable contractors. For example, a company may lease several floors in an existing building and require an interior stair to connect the floors. Such large openings are installed successfully in many buildings.

If concerns exist about future penetrations, several things can be done during initial construction to simplify them and minimize their cost. Because the locations of beams, girders, and joists are permanently obvious, the locations of slab tendons are the primary concern in penetrating the floor system. Slab tendons can be permanently located with the use of chalk lines snapped on forms (remaining visible on the slab soffit), or with the installation of small pin inserts marking the tendon locations. Further, one-way and uniformly distributed two-way slab tendons can be spaced at regular intervals, as wide as possible, measured from easily observable known locations. Even if none of these procedures is performed initially, tendons can be reliably located in existing slabs using non-destructive methods such as pachometers, x-rays or ground-penetrating radar adapted for use with concrete.

While the installation of openings and penetrations in existing post-tensioned slabs requires more care and expertise than is required in a non-prestressed floor system, where the cutting of reinforcing steel is often accepted without concern or repair, the cost of installing such penetrations is generally a small fraction of the initial cost savings inherent in the use of a post-tensioned floor system, and should not influence the selection.

9.1.3 Other Applications

For other applications of post-tensioning in building construction, see Chapters 2 and 8. Chapter 10 and Ref. 9.1 address the use of post-tensioning in parking structures.

Fig. 9.3 Post-Tensioned Transfer Beam
Courtesy of Ken Bondy

9.2 PLANNING AND DESIGN OF POST-TENSIONED FLOOR SYSTEMS

In the design of a floor system for a building, the project team must select the appropriate framing system, identify the structural and material properties, and set the design parameters. The following outline identifies a step-by-step approach used in the design of post-tensioned floor systems.

1. **Conceptual Design Phase**
   a. Planning
      Architectural criteria, shape, spans, occupancy, exposure, durability criteria
   b. General design objectives
      Governing codes, criteria in excess of code minimums
2. **Predesign Phase**
   a. Determine loading
   b. Determine floor system (see 9.2.1)

   c. Coordinate structural geometry
   d. Determine member sizes
3. **Material and Cross-Sectional Properties**
   a. Calculate cross-sectional properties
   b. Determine material properties
   c. Unbonded versus bonded tendon systems
   d. Select standard or encapsulated
4. **Set Design Parameters**
   a. Cover requirements for reinforcement
   b. Fire rating, wearing surface requirements
   c. Maximum tendon drape profile
   d. Target load balancing
   e. Average compression limits
   f. Allowable stresses
   g. Load combinations
   h. Standard reinforcement and placement detailing requirements for post-tensioning
5. **Design And Analysis**
   a. Calculate post-tensioning force using load balancing
   b. Calculate post-tensioning force using allowable average compression
   c. Select larger post-tensioning force from a. and b. above
   d. Calculate member moments due to gravity loads
   e. Compute secondary moments
   f. Calculate moment redistribution
   g. Check service stresses
   h. Calculate minimum non-prestressed reinforcement
   i. Calculate temperature and shrinkage reinforcement (one-way slabs)
   j. Check ultimate strength and supplement with non-prestressed reinforcement, if required
   k. Check punching shear
   l. Check deflection
6. **Layout of Reinforcement**
   a. Tendon layout
   b. Non-prestressed reinforcement layout
   c. Openings, corners

Commercially available computer programs can perform many of the tasks in Item 5. See Chapter 5 and Refs. 9.2-9.6 for detailed information regarding the planning and design of post-tensioned concrete floor systems.

9.2.1 Types of Floor Systems

A variety of post-tensioned floor systems are used in buildings; the most common are identified in this section. Floor systems are typically classified as one-way or two-way. In one-way systems floor loads are transferred to supports by members that resist bending in one direction only. Reactions from one-way members are transferred progressively to orthogonal one-way members until the loads reach vertical supports. In two-way systems floor loads are transferred to vertical supports by members that resist bending in both orthogonal directions. This is shown in Fig 9.5. Code criteria for one-way and two-way members are significantly different (see Chapter 5).

Tables 9.1 and 9.2 summarize the different one-way and two-way floor systems most commonly used in post-tensioned concrete buildings. These tables present a graphical depiction of each type of floor system with typical post-tensioned tendon arrangements. In addition, the tables provide typical spans and loadings commonly used in practice for each framing system. It should be noted, however, that the spans and loadings shown are the common range of application and are not intended to limit the use of a particular floor system. Pertinent comments about each system are also included.

The economics of post-tensioned floor systems involves interactions and tradeoffs between the three major cost factors involved in every floor: concrete material, reinforcement, and forming. In flat plates, for example, it is common to construct thicker slabs that have a flat soffit. This reduces the cost of forming while the cost of concrete and reinforcement increase slightly. Voided systems such as one-and two-way joist systems reduce the amount of concrete and reinforcement but generally increase the cost of forming. The number of re-uses of form material is also an important cost factor affecting the selection of every floor framing system.

Fig. 9.4 The Metropolitan Tower in Seattle
Courtesy of Cary Kopczynski & Company, Inc.

Table 9.1 - One-Way Framing Systems

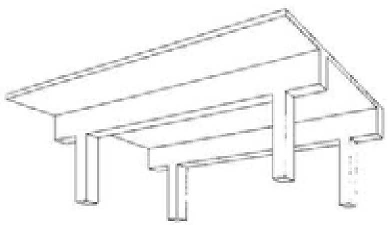
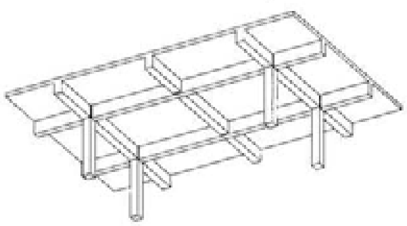
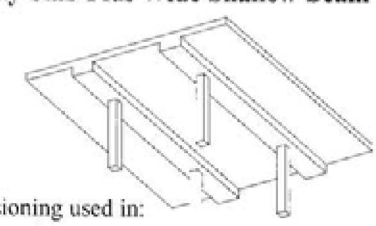
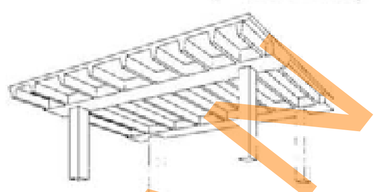
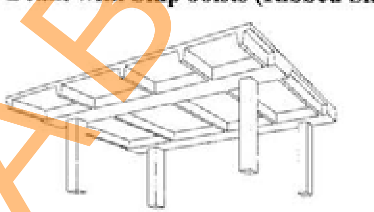
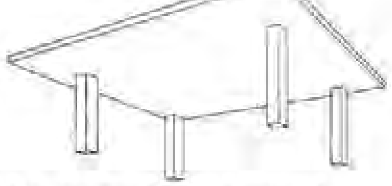
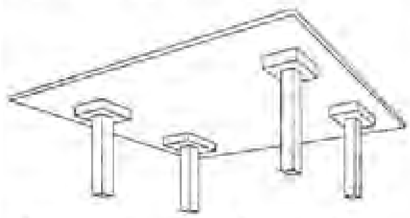
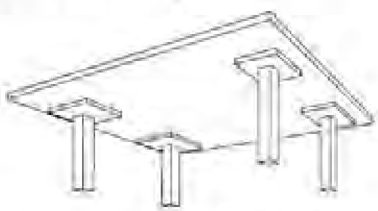
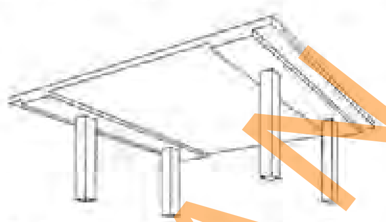
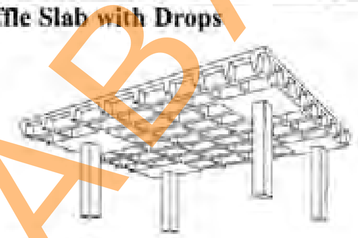
FLOOR SYSTEM AND LAYOUT OF POST-TENSIONING TENDONS	Typical span range (column center-to-center)	TYPICAL LOADING	COMMENTS
One-Way Slab and Beam  Post-tensioning used in: Beams, slabs (main and temperature)	Beams = 50 to 65 ft (15 to 20 m) Slabs = 15 to 30 ft (4.5 to 9 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Commonly used in parking structures but has also been used effectively in office buildings with long spans • Specialized forming systems have been designed for this system (steel beam forms and large-panel slab forms)
Slab, Beam, and Girder System  Post-tensioning used in: Slabs (main and temperature), beams and girders	Slabs = 15 to 20 ft (4.5 to 6 m) Beams = 50 to 65 ft (15 to 20 m) Girders = 30 to 40 ft (10 to 12 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Generally more economical than spanning a very thick slab between beams located on column lines • Commonly used in parking structures at "turn-around" aisles and in other occupancies with short-direction spans of 30 ft and more
One-Way Slab Plus Wide Shallow Beam  Post-tensioning used in: a. Beams only b. Slab only c. Beams and slab	Beams = 25 to 40 ft (8 to 12 m) Slabs = 18 to 25 ft (5.5 to 7.5 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Effective for column layouts with short span in one directions and long span in orthogonal direction • Normally beams span long direction, slab spans short direction • Used primarily where structural depth is limited
Wide Beam with Joists (Ribbed Slab)  Post-tensioning used in: Beams and joists *Note: Beams and joists should have the same depths.*	Slabs = Typically about 3 ft (1 m) Beams = 20 to 35 ft (6 to 11 m) Joists = 35 to 65 ft (11 to 20 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Effective for column layouts with short span in one directions and long span in orthogonal direction • Normally beams span short direction and joists spans long direction • Minimized structural depth
Wide Beam with Skip Joists (Ribbed Slab)  Post-tensioning used in: Beams and joists, occasionaly in slab *Note: Beams and joists should have the same depths.*	Slabs = Typically about 3 to 12 ft (1 to 4 m) Beams = 20 to 35 ft (6 to 11 m) Joists = 35 to 55 ft (11 to 17 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Spreads joists as far as possible without increasing cost of slab • Often allows more eficient use of post-tensioning in joists (force per foot of width

Table 9.2 - Two-Way Framing Systems

FLOOR SYSTEM AND LAYOUT OF POST-TENSIONING TENDONS	Typical span range (column center-to-center)	TYPICAL LOADING	COMMENTS
Flat Plate  Bands in one direction, and uniformly distributed tendons in the other	20 to 30 ft (6 to 9 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Lowest formwork cost • Flexibility in column arrangement • Flat ceiling • Greatest flexibility in under-ceiling services layout • Most efficient if bay size is approximately square • Load path easy to visualize • Punching shear strength can be increased using studrails, shearheads, or conventional shear reinforcement
Flat Slab with Column Capitals  Bands in one direction, and uniformly distributed tendons in the other	25 to 35 ft (8 to 11 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Effective system for increasing punching shear capacity if architectrual considerations permit • Small caps have minor effect on flexural behavior
Flat Slab with Drop Panels  Bands in one direction, and uniformly distributed tendons in the other	30 to 40 ft (9 to 12 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Larger drop panels can be effective in reducing flexural reinforcement • Normally used for longer spans
Slab with Slab Band  Bands in one direction, and uniformly distributed tendons in the other	25 to 40 ft (8 to 14 m)	Light: Up to 100 psf (5 kN/m^2) to Medium: 100 to 200 psf (5 - 10 kN/m^2)	• Can be very effective in panels with rectangular aspect ratios • Two-way behavior must be justified to avoid more restrictive one-way code requirements
Waffle Slab with Drops  Ribs both ways *Note: Ideally the "drops" and "ribs" have the same depth.*	30 to 60 ft (9 to 18 m)	Medium: 100 to 200 psf (5 - 10 kN/m^2) to Heavy: over 200 psf (10 kN/m^2)	• Very effective for heavy loading and relatively long spans • Most efficient if bay size is approximately square

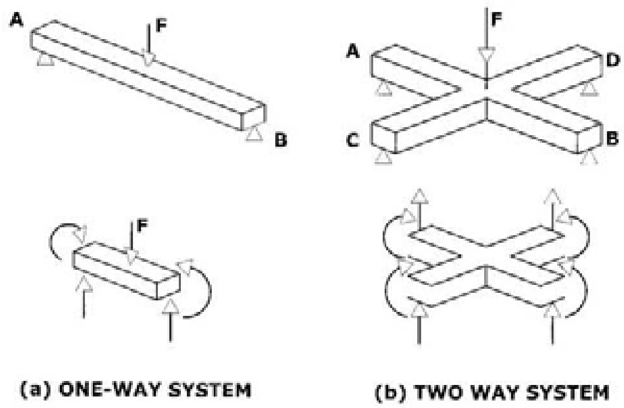


Fig. 9.5 One-Way and Two-Way Systems

Cost savings in voided systems such as one- and two-way joist systems can be realized by maintaining the same depth in supporting beams in one-way systems and filled panels around columns in two-way systems. Table 9.3 lists the span/depth ratios that have been found to give economical and satisfactory structural performance for each structure type listed. See Chapter 2 for applications of different types of floor systems in actual building structures.

Table 9.3 - Suggested Span/Depth Ratios*

Floor System	Span/Depth Ratio
One-way slabs	48
Two-way slabs	45
Two-way slab with drop panel (minimum drop panel at least L/6 each way)	50
Two-way slab with two-way beams	55
Two-way waffle slab (5 ft × 5 ft grid)	35
Beams, b ≈ h/3	20
Beams, b ≈ 3h	30
One-way joists	40

*These values apply for members with LL/DL ratios < 1.0

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POST-TENSIONED PARKING STRUCTURES

10.1 INTRODUCTION

Millions of square feet of cast-in-place post-tensioned parking structures are built in North America every year. An independent survey of parking garage construction in the United States for the year 2000 found that cast-in-place post-tensioned systems were the most popular structural alternative.[10.1] The survey showed that post-tensioned structures had a significant lead in market share in terms of both number of structures built, and volume (number of spaces). Other countries are experiencing similar trends.

Many post-tensioned parking structures built today are constructed as free-standing structures. They can also be constructed as part of hotels, condominiums, apartment and office buildings, and other facilities. Attached parking structures require careful consideration of not only the garage design requirements, but also the various design requirements of the basic facility.

The PTI publication *Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures*[10.2] is an excellent source of information on post-tensioned parking structures. The publication provides a comprehensive reference for the design, construction, and maintenance practices that will ensure long-term durability and minimize life-cycle costs of free-standing, cast-in-place post-tensioned concrete parking structures.

This chapter complements the previous chapter on post-tensioning in buildings, but will focus on post-tensioned parking structures. The design fundamentals are the same, however this chapter highlights considerations specific to parking structures.

The use of post-tensioning in parking structures offers several advantages; namely:[10.3]

- **Initial and Life-Cycle Cost Savings** – Economic analysis and competitive bids show that cast-in-place post-tensioned structural systems often provide initial cost savings when compared to other framing systems. When initial costs are close to those of other systems, life-cycle costs often show savings for the post-tensioned systems.
- **Low Maintenance Costs** – Properly designed, detailed, and constructed post-tensioned floors are relatively crack free. Reduced cracking, encapsulation of post-tensioned reinforcement, low water/cement ratio, air entrained concrete, and concrete sealers make it possible to achieve floor systems with minimal maintenance in even the most aggressive environments. Walker[10.4] rated the durability of cast-in-place post-tensioned systems as best when compared with systems that utilize structural steel, pre-topped and site-topped precast concrete, and non-prestressed cast-in-place concrete. Precast floors, particularly pre-topped precast floors, require periodic maintenance of the joints between precast units. Loss of parking spaces during maintenance work results in loss of parking revenue and inconvenience to users.
- **Crack Control and Watertightness** – Cast-in-place post-tensioned structural systems eliminate closely spaced joints and help promote watertightness by placing the slab in bi-axial compression, thereby controlling and counteracting shrinkage and flexural cracks. This is a very important advantage over other systems in exposed construction because post-tensioning would, in effect, prevent water and deicing chemicals from leaking through cracks, which eventually can result in deterioration and costly repairs.
- **Smooth Riding Surfaces** – Eliminating closely spaced joints results in a superior riding surface. Differential deflections across joints are not an issue in cast-in-place post-tensioned floors.
- **Lighting and Security** – The wide beam spacing and flat surfaces provided by long-span post-tensioned parking structures enhance lighting and improve patron security, compared to precast long-span double-tee systems. A flat plate structure (either post-tensioned or rebar) utilizing studrails instead of drop caps is truly the easiest to light.
- **Fire Resistance** – Cast-in-place post-tensioned members allow for varying concrete thicknesses to provide fire ratings that meet differing code requirements. The thin-stemmed elements commonly used in other structural systems provide less concrete cover and lower fire resistance ratings.
- **Functional Flexibility** – Cast-in-place post-tensioned construction allows long, column-free spans and is adaptable to other functional requirements of parking structures. Post-tensioned structures can easily accommodate irregular floor plans, slopes to drains, straight or curved ramps, and warped surfaces that provide smooth transitions between ramps and level floors. There is no functional compromise as is often the case with other structural systems. Another advantage of cast-in-place post-tensioned construction is vertical/horizontal expansion. For precast construction, there must be room for a sizable crane with a reach capable of placing the structural members.
- **Aesthetics** – The architectural appeal of cast-in-place concrete is widely recognized. Post-tensioning can further enhance the clean lines and sleek look by allowing thinner concrete members. Curvilinear shapes and forms are economically feasible with

Fig. 10.1 Freestanding Parking Structure
Courtesy FBA Inc., Structural Engineers

cast-in-place structures that provide the necessary flexibility for creative architectural features.

- **Reduced Structural Depth** – Post-tensioning can reduce structural depths by one-fifth or more in comparison to other non-prestressed systems. Structural depth reductions are often essential to meet building height restrictions. In the case of underground parking garages, structural depth reduction is desirable to reduce excavation, soil retention system costs, and dewatering costs in sites with high water tables.

- **Deflection and Vibration Control** – The precompression imparted by post-tensioning results in increased stiffness because the entire concrete section is effective. Also, because of their draped configuration, post-tensioning tendons carry a significant portion of the dead load directly to the columns (the net load, which is the load that produces deflections, is therefore reduced). The precompression, the draped tendons, and the monolithic nature of cast-in-place garages significantly reduce deflection problems. The monolithic construction, continuity, and rigid connections between slabs, beams and columns reduce vibrations. When vibration control is critical, a short-span flat-plate structure should be considered.

- **Lateral Load Compatibility** – Monolithic connections between slabs, beams, and columns provide rigid frame action to resist wind and moderate seismic loads. This frame action may eliminate the need for shear walls in low seismic regions, which results in cost savings and more open and efficient parking structures with enhanced patron security.

- **Seismic Performance** – Post-tensioned structures performed well in the 1971 San Fernando, the 1989 Loma Prieta, and the 1994 Northridge earthquakes.[10.5] Research has shown that unbonded post-tensioning improves the behavior of moment frames under seismic loads.[10.6] Code provisions developed by the Building Seismic Safety Council allow the use of post-tensioning in ductile moment-resisting frames.[10.7]

- **Structural Integrity** – Research and experience have shown that post-tensioned structures inherently provide structural integrity under abnormal and catastrophic loading. Well-detailed cast-in-place post-tensioned structures have significantly higher structural integrity, redundancy, and resistance to catastrophic loading than other systems, especially precast systems.[10.3,10.8,10.9] This is typically provided by the continuity of the structure over intermediate supports.

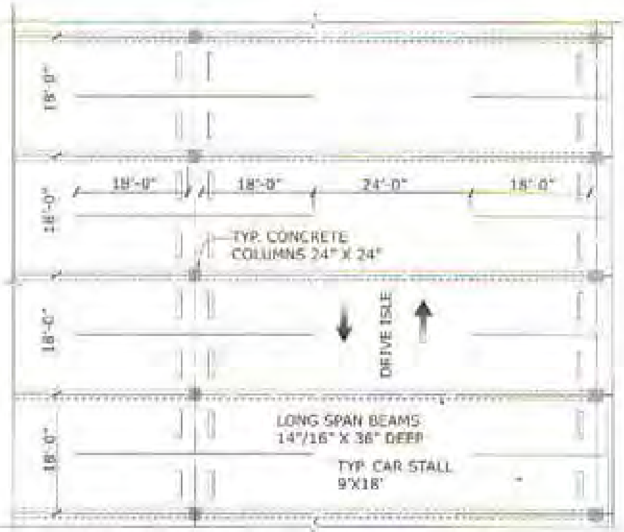

Fig. 10.2 Layout for a Long-Span Parking Structure

- **Construction Advantages** – One of the most important advantages of cast-in-place construction is that the builder has more control over the project's cost and schedule. Many of the problems associated with having to rely on other suppliers are eliminated. Both the time and cost of hauling prefabricated pieces from the plant to the job site and the need for heavy lifting are eliminated. Large panel flying forms, modular forming systems, and ever-improving concrete technology continue to enhance the construction speed and economy of cast-in-place post-tensioned structures.

Specific information regarding durability of parking structures can be found in Chapter 19, "Durability," and maintenance of parking structures can be found in the PTI publication *Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures.*[10.2]

10.2 APPLICATIONS OF POST-TENSIONING IN PARKING STRUCTURES

The primary application for post-tensioning in parking structures is in floor systems; however, post-tensioning is also utilized for other components such as barrier cable systems. Parking structures that utilize post-tensioned floor systems can be classified as either freestanding or those constructed as part of office, housing or hotel buildings.

10.2.1 Freestanding Parking Structures

Freestanding parking structures are those that are independent of the buildings that they serve. The only design parameters are those that are related to building site geometry and individual parking requirements. Freestanding parking structures are classified as either long-span or short-span.

10.2.1.1 Long-Span Parking Structures

Long-span parking structures have no interior columns encroaching on the drive lanes and parking stalls. The column spacing parallel to the length of the structure is typically based upon the economies of the selected structural system. For this type of application, the use of a one-way beam-and-slab floor system is the preferred choice.

The introduction of post-tensioned concrete led to significant enhancements in the functional design of long-span parking facilities; namely:

- **Reduction in the number of columns** – The reduction of the number of columns is obvious when compared to a flat plate or flat slab structural system.
- **Increased visibility across the structure** – Increased visibility across the structure is achieved by using fewer columns and barrier cables on ramps. Precast

(a) Precast Sections - Sky Harbor Parking Garage (b) P/T Addition - Sky Harbor Parking Garage

Fig. 10.3 Lighting Comparison between a Precast and Post-Tensioned Structure structures require deep structural perimeter beams to support tees.

- **Increased ceiling height** – The ceiling is perceived as the underside of the slab in a beam-and-slab floor system while the ceiling height is perceived as the bottom of the tees in a precast floor system. With the same floor-to-floor heights, this can be greater than a two-foot increase in ceiling height.
- **Improved lighting** – The combination of the beam spacing and increased ceiling height allows more open space and enhances the overall lighting design. Additionally, painting the ceilings and beams white boosts the safety and brightness of the facility.

10.2.1.2 Short-Span Parking Structures

A short-span parking structure is defined as one in which columns are located on either side of the drive lane encroaching upon the parking spaces. The column layout must work in conjunction with the parking space layout to render a functional parking facility. With 90-degree stalls, this will typically work out to one column every three spaces. This type of application utilizes post-tensioned two-way floor systems. Because of the high shear forces that occur near the columns, column capitals or shear reinforcing within the slab are used as part of the floor system to satisfy punching shear requirements. This type of post-tensioned floor system offers greater design creativity, because the exterior façade does not depend upon a perimeter beam.

From an economic standpoint, the cost of a short-span structure is likely to be less per square foot as compared to a long-span structure because the forming and reinforcement is less costly. However, in many situations more square feet are required for the same number of parking spaces because the parking space layout is not as efficient.

It is then necessary as part of the economic evaluation to estimate the total costs of the facility (as opposed to the cost per square foot), taking into account the effect these differences might have.

Fig. 10.4 Layout for a Short-Span Parking Structure

10.2.2 Parking Facilities as Part of Other Building Occupancy

Parking facilities are frequently built underneath occupancies including offices, commercial spaces, hotels, and apartments and condominiums. The use of transfer floors and beams, also known as podium floors, is common in such cases where the column layout above and below is not aligned. The primary purpose of transfer floors and beams is to properly distribute the forces from the columns located above to the supporting columns located below. Transfer floors allow for an efficient parking layout because the column grid above need not necessarily align with the column grid below. Transfer (podium) floors are common when wood structures are constructed above parking facilities. Further information about transfer floors can be found in Chapter 9.

10.2.2.1 Above-Ground Parking Facilities

Parking facilities built below buildings but above ground can be effectively constructed using post-tensioned floor systems. The type of floor system is dependent upon the column support geometry of various occupancies in the building. When the column grids are the same as the structure above, the same type of floor system may be used.

The functional layout of the parking area is economically critical when the column spacing remains the same. Many times the column layout will not lend itself to efficient parking stall layout.

10.2.2.2 Underground Parking Garages

Post-tensioned underground parking garages can have challenging detailing that is critical to the overall performance of the structural members. In addition to the points made in Section 10.4, the following items need to be considered during design:

- Restraint to shortening created by the foundation walls
- Access to anchorages of tendons for stressing

Some of the commonly used details that reduce the stresses caused by restraint are discussed in Chapter 6. Fig. 6.4(a) illustrates a detail that is used to reduce restraint to shortening at the foundation walls. The detail consists of non-prestressed reinforcement inside a splice sleeve, which is the only element that connects the slab to the wall, and also allows the movement of the slab with respect to the wall. The quantity and size of the non-prestressed reinforcement is determined through design and will depend upon several factors, including the amount of lateral loads due to earth pressure into or out of the slab and required slab diaphragm action.

Fig. 10.5 Post-Tensioned Podium Slab

In order to facilitate the placement of the rams that stress the post-tensioning tendons, a three-foot-wide closure strip is built around the perimeter of the foundation. Fig. 6.15 illustrates the access needed for stressing the tendons near foundation walls. Additionally, closure strips are used to allow movement of the post-tensioned floor system with respect to the foundation wall. When closure strips are used, the slab will need to be shored at the perimeter until it is permanently connected to the foundation walls. See Chapter 6 for more information on closure strips.

10.2.3 Barrier Cable

Post-tensioned barrier cable systems used in parking structures provide a high impact railing on ramp slabs and allow for an open feel to the interior of the parking facility. Barrier cable systems with high-strength steel strands can be used instead of solid structural members to provide pedestrian barriers, vehicle control or restraints. The versatility of barrier cables allows the designer to meet the pedestrian protection requirement as well as improve the aesthetic appearance of the parking facility. Barrier cables may be installed either vertically or horizontally. Barrier cables installed horizontally need to be evaluated for the ladder effect according to the governing building code. Barrier cables are typically used as:

- Ramp separators
- Parapet railings
- Hand rails
- Perimeter confinement
- Architectural façades

Barrier cable systems are easy to install and provide an economical and flexible method to meet the needs of the project. For long-term protection against corrosion, the steel strands are protected with plastic, epoxy, or are simply galvanized.

Fig. 10.6 Barrier Cable

10.3 ECONOMICS OF PARKING STRUCTURES[10.2]

The key to the economical design of a parking structure is a thorough understanding of the cost structure. The total price of a parking structure is affected by many factors, some of which have wide cost latitude. A 1992 survey[10.3] reported that the cost per car of cast-in-place parking structures could vary from \$5,000 to \$15,000. Overall cost efficiency can only be attained through a clear knowledge of the cost impact of each decision, be it a structural issue such as column size or an architectural issue such as barrier cable versus railing.

Private owners as well as city and state agencies often solicit "design/construct" parking projects. This approach shifts the burden of an economic study to the prospective contractors and their consulting teams. In addition to providing a cost estimate, the contractor is required to make decisions regarding the structural system, dimensions, and materials. The PTI publication *Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures*[10.2] provides a perspective for the cost structure by establishing a reference basis for cost comparison. It recognizes the demand for an extension beyond the traditional cost estimate and is tailored to meet the needs and interests of developers, contractors, and consultants.

To develop this reference basis, the PTI publication[10.2] analyzes in depth costs for a typical parking structure. The significance of each cost item is examined with respect to alternate choices and the overall cost. This provides a basis for evaluating the absolute cost of each item as well as its relationship to the cost of other items. The way that variations in each significant item affect the total cost is also discussed. This knowledge of cost variation is central to the proper selection of options such as the structural system and façade.

More information regarding the selection of specific structural systems can be found in Chapter 5.

10.4 ADDITIONAL DESIGN REQUIREMENTS FOR POST-TENSIONED PARKING STRUCTURES

Chapter 5 presented a step-by-step procedure for the selection, analysis, and design of post-tensioned floor systems. The same procedure may be used for parking structures, with modifications and slight adjustments made to the following activities that are specific to parking structures: design loadings, material properties, cover requirements for reinforcement, average compression limits, allowable stresses, drainage, and floor surface treatments.

The design engineer should assess the corrosive environment, if any, surrounding the parking structure to be designed. Depending on the geographical location of the facility, one or more of the above activities might be impacted.

10.4.1 Design Loading[10.2]

Most building codes and local standards require a live load of 50 psf (2.4 kN/m^2) for parking structures; however, allowance for live load reductions vary. The use of live load reduction is usually justified because actual automobile weight in fully loaded parking garages will seldom exceed 25 to 30 psf (1.2 to 1.4 kN/m^2). The IBC requires parking structures to be designed for an unreduced 40 psf (1.9 kN/m^2) live load. Many building codes also require that the floor be designed for a 2000 lb concentrated wheel load. However, this requirement rarely controls except for thin slabs.

Snow loads are another form of gravity loads that are typically applied on roofs of parking facilities. When snow loads are present, they should be clearly defined by the respective building code; however, it is not always clear how snow loads should be combined with parking live loads when designing roof members. While this may be logical for the design of building roofs, it is usually not appropriate for design of parking garage roofs in which both cars and snow can be present at the same time.

In the absence of specific code requirements, it is suggested that the roof of parking structures be designed for the following load combinations:

Serviceability Design:

$$1.0D + 1.0L + 1.0S \tag{5.1}$$

Ultimate Strength Design:

$$1.2D + 1.0L + 1.6S \tag{5.2}$$

Where:

D = Dead Load

L = Live Load

S = Snow load

Roof areas subject to snow drifting or sliding accumulation should be designed for appropriate increased loadings as defined by the governing codes. An increase in localized loading is recommended around areas adjacent to snow chutes where piling may occur with snow clearing.

For a comprehensive treatment of load combinations, reference to Section 9.2 of the ACI Code.[10.12]

10.4.2 Material Properties

10.4.2.1 Concrete Properties

Concrete in parking structures, regardless of the structural system used, requires greater quality control than concrete used in buildings because of exposure to various weather factors, such as snow, deicing salts, and corrosion. Proper construction practices and concrete placement techniques are critical to the durability of post-tensioned parking structures, because concrete is the primary defense against corrosion. One way to improve durability is to design for a dense concrete mix, which typically results in low permeability.

The following parameters should be considered when specifying concrete properties for post-tensioned parking structures:

- **Concrete Strength** – Typical 28-day concrete strengths (f'_c) range from 4000 to 6000 psi (27.6 MPa to 41.4 MPa), similar to concrete used in post-tensioned building construction.
- **Concrete Weight** – Most post-tensioned parking structures in the United States are constructed with normal weight rather than lightweight concrete. Lightweight concrete has greater elastic shortening and creep for the same average compression than normal weight concrete because of its reduced modulus of elasticity.[10.2]
- **Water-Cement Ratio** – Water-cement ratios are typically between 0.40 and 0.45.[10.2]
- **Admixtures** – Admixtures include water-reducing and retarding agents; water-reducing and accelerating admixtures; water-reducing and high-range admixtures; and water-reducing, high-range and retarding admixtures. Typical ranges used in portland cement concrete vary from 2 to 6 oz. per 100 pounds of cement (130 to 390 ml per 100 kg).[10.14] In no case should admixtures containing calcium chloride or other chemicals known to be deleterious to prestressing steel be used in post-tensioned concrete.
- **Mineral Additives** – Mineral additives are used to increase the density of concrete and reduce its permeability. In addition, these additives reduce the maximum temperature developed during cement hydration, control bleeding, and increase long-term strength. Mineral additives include fly ash conforming to ASTM C618, ground granulated blast furnace slag conforming to ASTM C989, and silica fume conforming to ASTM C1240.[10.14]
- **Concrete Curing** – Proper curing techniques are essential to the long-term performance of concrete. Water curing or application of liquid membranes can give satisfactory results when used properly.

10.4.2.2 Post-Tensioning Tendons

Post-tensioned tendons in parking structures can be either unbonded or bonded, similar to the tendons used in building structures. See Chapter 3 for discussions on the differences. The majority of post-tensioned parking structures constructed in the United States since the mid-1960s have utilized unbonded tendons.

Unbonded post-tensioning systems typically consist of ½ in., 270 ksi, 7-wire, low-relaxation strands coated with corrosion-inhibiting grease and encased in an extruded plastic sheathing. For added corrosion protection, the strands and anchorages may be fully encapsulated. Refer to Chapter 19 for a discussion of durability requirements for parking structures and when post-tensioning tendons should be fully encapsulated.

10.4.2.3 Non-Prestressed Reinforcement

Non-prestressed reinforcement is the first element to be attacked by corrosion in post-tensioned parking structures. This is due to the fact that even though non-prestressed reinforcement and post-tensioning tendons have similar concrete covers, the tendons are protected by plastic sheathing and corrosion-inhibiting coating. Because the corrosive agents migrate from the slab surface through the concrete slab, one would expect the top non-prestressed reinforcement to be most susceptible to corrosion.

Epoxy-coated bars should be considered for top non-prestressed reinforcement in parking structures located within certain durability zones. The use of epoxy-coated non-prestressed reinforcement in beams, columns, and for bottom bars in slabs should be in accordance with the designer's practice. A corrosion inhibitor can be introduced into the concrete for added protection.

10.4.3 Cover Requirements for Reinforcement

It is recommended that the floors of parking structures be considered exposed to weather for the purpose of setting the concrete cover. The minimum concrete cover should be set to the outermost layer of the non-prestressed reinforcement, to the extreme area of sheathing closest to the surface for tendons, or to anchorages on post-tensioning elements. Table 10.1 summarizes the minimum cover requirements per the ACI Code[10.12] for post-tensioned cast-in-place concrete.

Table 10.1 - Recommended Concrete Cover Requirements for Post-Tensioned Parking Structures*

DESIGN ELEMENT	DURABILITY ZONE					
	I		II/CC-I		III/CC-II	
	American	SI	American	SI	American	SI
Concrete Cover:						
Slab Top	1"	25 mm	1- ½"	40 mm	2"	50 mm
Slab Bottom [(a)]	¾"	20 mm	¾"	20 mm	1"	25 mm
Beam [(a)]	1- ½"	40 mm	1- ½"	40 mm	1- ½"	40 mm
Columns [(a)]	1- ½"	40 mm	1- ½"	40 mm	1- ½"	40 mm

Notes: (a) The licensed design professional may choose to use epoxy-coated non-prestressed reinforcement in accordance with experience and local standard practice.

* This table addresses the minimum cover from a durability point of view only. Fire resistance (restraint and unrestraint conditions) or other considerations may require larger covers than those noted above.

The structural drawings should clearly identify all concrete cover requirements and should take the reinforcement placing sequence and concrete cover hierarchy into account. In particular, cover requirements and placement of intersecting tendons over columns should be clearly defined to maintain proper cover and achieve maximum structural efficiency. The drawings should explicitly account for the fact that slab and joist reinforcement is placed over beam reinforcement and that, where applicable, beam reinforcement may need to be placed over that of girders. Different top cover should thus be specified for slabs, beams, and girders.[10.2]

Ramped floors used in parking structures create unique situations that must be addressed to ensure that concrete cover is not compromised. Figs. 10.7 and 10.8 show examples of specifying concrete cover for some typical ramp conditions that should be detailed on the drawings.

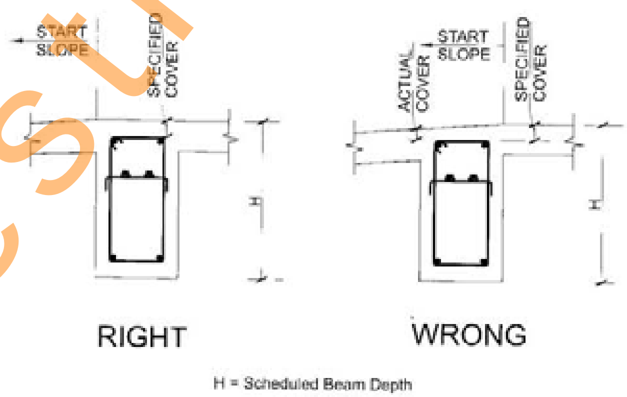

Fig. 10.7 Concrete Cover for Beam Reinforcement at the Start of the Ramp

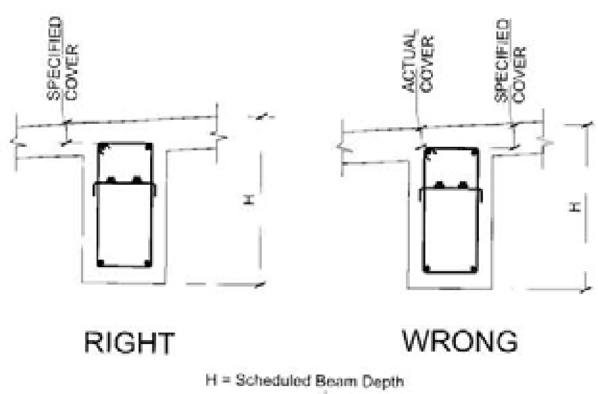

Fig. 10.8 Concrete Cover for Beam Reinforcement in the Ramp

10.4.4 Minimum Average Compression Limits

PTI recommends that the minimum average compression in parking structures be higher than that in building structures. However, with higher average compression, the potential for restraint to shortening cracking increases.

To minimize cracking and address durability considerations, recommendations for minimum average compression levels for floor systems (both slabs and beams) based upon durability zones are:[10.2]

- Zone I — 150 psi (1.0 MPa)
- Zone II, III, CC-I, CC-II — 200 psi (1.4 MPa)

When unbonded tendons are used for temperature and shrinkage reinforcement in one-way floor systems, in accordance with ACI 318-02, Section 7.12.3, the tendons shall provide a minimum average compression of 100 psi (0.7 MPa).

10.4.5 Allowable Stresses

The ACI Code[10.12] specifies allowable stresses in concrete structures. Refer to Chapter 5 for a detailed discussion on allowable stresses. Post-tensioned parking structures follow essentially the same requirements as buildings.

Performance (durability) of a post-tensioned parking structure is a primary concern for the designer. Concrete cracking should be mitigated as much as possible for overall long-term durability. In the design of the post-tensioned floor system, the designer can control structural cracking by limiting the allowable tensile stresses to below the modulus of rupture of the concrete, $7.5\sqrt{f'_c}$. For two-way floor systems (Class U) or one-way floor systems designed as Class U, the allowable stresses per ACI Code[10.12] are already limited to the concrete modulus of rupture. One-way floor systems designed as Class T members have an allowable tensile stress range of $7.5\sqrt{f'_c}$ to $12\sqrt{f'_c}$ per ACI Code.[10.12] The design of post-tensioned members allows the designer flexibility in limiting tensile stresses. Essentially the designer can limit the tensile stresses anywhere within the structure that is desirable. Depending upon the structure's durability exposure, the designer may limit the tensile stresses in the negative moment region to below $6\sqrt{f'_c}$.

10.4.6 Drainage[10.2]

Proper drainage is essential for the durability of parking structures and is normally a low cost item. Drainage design requires attention to three areas: proper slopes, proper drain catchment area size, and proper drainpipe size and location.

Parking structure floors should not be level, even when there is no exposure to the elements. Vehicles and wind will carry moisture into covered levels and overflow can run down ramps to lower floors. Slopes in two directions are essential to attain positive drainage and avoid ponding. The absolute minimum slope should be ⅛ in. per ft or about 1%. However, the preferred slope is ¼ in. per ft or about 2%.[10.2] The flexibility of post-tensioned floor systems in setting slopes in two directions provides an advantage over precast floor systems.

Drain catchment area size for floors with one or two percent slopes should not exceed 4500 sq. ft (420 m^2). Catchment area can be larger for floors that have greater slopes. Catchment areas should be located so that water does not have to cross expansion joints or turn corners to reach the drain.

Drains should be located at the low point of the catchment area and adequately sized or oversized for the design storm. Normally, circular or square drains are used and are recessed slightly below the adjacent floor surface. Segmented or continuous trench drains may be required at roof levels or at the bottom of ramps. Note that continuous trench drains create isolation joints and the separation should be treated like any other isolation joint.

It is extremely important that the designer select a drain layout which allows overflow to adjacent drains (secondary drainage) in case one drain location is not functioning. It is not uncommon on roof levels to provide an overflow drain as a secondary line of defense to ensure that water cannot build up on slab areas.

10.4.7 Floor Surface Treatments[10.2]

Surface treatments can further protect parking structures floors against corrosion. The PTI publication *Design, Construction and Maintenance of Cast-in-Place Post-Tensioned Concrete Parking Structures*[10.2] offers detailed information about surface treatments. The three primary floor surface treatments are concrete surface sealers, traffic-bearing membranes, and concrete wearing surfaces.

A concrete surface sealer is a cost-effective way to enhance the durability of parking structure floor surfaces. The objective of the surface sealer is reduction of moisture and salt penetration into the concrete slab. Surface sealers are generally used when the structure can be exposed to deicing salts, freezing and thawing conditions, or is near a body of salt water. While sealers can enhance the durability characteristics of the concrete, they do not have crack-bridging capabilities and thus do not provide protection against infiltration through cracks.

Elastomeric deck-coating and membrane systems prevent the intrusion of moisture and salt into the concrete. The multi-layer elastomeric polyurethane material provides a protective barrier against corrosion and freezing and thawing deterioration. Traffic-bearing membranes provide better waterproofing protection than concrete surface sealers in addition to having the elastomeric properties to bridge small cracks. Periodic inspection will be required to ensure the integrity of system.

A concrete wearing surface can be applied over a waterproofing membrane to protect the membrane from wear and tear and ultraviolet rays, thus providing for an even longer service life. The membrane is covered with a protection board, a drainage blanket and either a concrete topping slab or concrete pavers. After applying the concrete wearing surface, minor leaks are nearly impossible to locate for repair, and membrane replacement is complicated and time consuming because the wearing surface must be removed. As a result, these systems are typically only used in parking areas subjected to severe wear and tear such as truck traffic, plaza decks and areas over occupied space where long service life is an important factor.

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POST-TENSIONED SLABS-ON-GROUND

11.1 INTRODUCTION

Since the early 1950s, post-tensioned reinforcing has been effectively used throughout the United States to construct many types of ground-supported slabs. This includes foundations for residential and light commercial construction, warehouse floors in commercial and industrial applications, concrete sport courts (such as tennis courts and basketball courts), parking lot and highway pavements, airport runways, and heavily reinforced mat foundations used to support high-rise structures.

Ground-supported concrete slabs can have forces acting on them from several sources. A slab can have applied loading from a structure it is designed to support. A slab constructed in active soils (expansive or compressible) can have loads applied as a result of the shrinking and swelling of the supporting soil as it is wetted and dried. And finally, all ground-supported slabs suffer to some degree from a restraint-to-shortening action caused by the friction between the slab and the soil that supports it. All of these loads can, to varying degrees, be anticipated and calculated. Post-tensioned reinforcing can then be applied to increase the tensile capacity of a concrete member to the point of effectively counteracting the various anticipated loads.

11.1.1 Advantages of Post-Tensioned Slabs-On-Ground

In ground-supported slabs, the use of post-tensioning tendons puts the concrete into compression before applied loads can deform the slab and before applied tensile stresses cause the slab to crack. Using post-tensioned tendons in ground-supported slabs can reduce or eliminate the need for control joints and also reduces or eliminates restraint-to-shortening cracks. It can also produce a slab with a higher flexural capacity than a slab of the same thickness that is reinforced with non-prestressed reinforcing.

In residential and light commercial foundations built on active soils, the designer can use post-tensioned reinforcing to increase the stiffness of the foundation to resist the movement of the supporting soil, or to span over any deformations in the supporting soil as it shrinks and swells. Post-tensioned slab-on-ground foundations are an economical solution for building in active soils throughout the United States. Properly designed foundations that utilize post-tensioned tendons as their primary, or only, means of reinforcing require fewer or shallower stiffening ribs, or can have a reduced slab thickness, when compared to an equivalent non-prestressed foundation. In areas where removal of expansive sub-grade material prior to foundation construction is common, the PTI Method can be used to predict the soil/structure interaction so that an economical foundation can be constructed on the native soils, avoiding costly export and import operations.

11.2 FOUNDATIONS FOR RESIDENTIAL AND LIGHT COMMERCIAL CONSTRUCTION

Currently, more than 50% of the post-tensioned reinforcing used in the United States goes into the reinforcing of foundations used to support residential and light commercial structures. This includes production and custom single family housing, multifamily housing, assisted living facilities, and many types of small commercial buildings such as restaurants or convenience stores. In some parts of the country (most notably Texas, California, Arizona, Nevada, Colorado, Louisiana, Florida, and Georgia) post-tensioned foundations are the predominant type of construction used to build housing in expansive clays, or other types of active soils.

11.2.1 History and Development

Following World War II, ground-supported reinforced concrete slabs, occasionally referred to as "floating slabs," were utilized for the foundations of residential and light commercial construction. The use of these ground-supported foundations became quite common in the early 1950s, with the design established substantially by trial and error. This method of design for ground-supported foundations persisted until the Federal Housing Administration authorized a technical study to establish design criteria for residential ground-supported foundations. The Building Research Advisory Board, a division of Engineering and Industry Research, National Research Council, prepared publications for the National Academy of Sciences. These publications became known as the Building Research Advisory Board (BRAB) Reports, and were addressed to the Federal Housing Administration. The latest BRAB Report is No. 33,[11.1] dated 1968.

The BRAB Reports were the first attempt to combine consideration of the soil characteristics of an area with the anticipated cycle and range of moisture content due to climate, and also with the structure to be supported by the foundation. During the mid 1960s, post-tensioned concrete foundations first began to be used to support residential buildings in various parts of Louisiana, Texas, and California. Generally these early post-tensioned foundations were built on expansive soils where the performance of some non-prestressed concrete foundations had been less than satisfactory. In Texas and Louisiana, the early post-tensioned foundations consisted of a slab with a grid of stiffening ribs in both directions. This type of foundation became known as the "ribbed foundation." In California, where soils were generally less expansive than those in Texas and Louisiana, the early post-tensioned foundations consisted of a slab with an edge beam at the entire perimeter, but no interior stiffening ribs. This type of foundation became known as the "California slab" or the "California foundation."

In March of 1967, the first three post-tensioned ground-supported foundations approved by the Federal Housing Administration were installed in Houston, Texas. These structures were available for inspection and were found to be performing very satisfactorily. In January 1968, a report was published on tests of a 20-ft by 40-ft prestressed residential ground-supported foundation. Based on the information from this test and previous experience with completed construction, the first general approval for the use of post-tensioned ground-supported foundations throughout the United States was issued in June of 1968 by the Department of Housing and Urban Development, Central Offices, Washington, D.C. The only requirement placed on the use of this method of reinforcement was that a rational design be provided by a licensed design professional.

Subsequent to this general approval in June of 1968, the use of ground-supported post-tensioned concrete foundations supporting residential and light commercial buildings increased dramatically, to the point where it is now the largest single application for post-tensioned tendons in the United States. At the time of this publication, more than half of all post-tensioned tendons sold in the United States are used in residential ground-supported foundations. During the calendar year 2003 alone (the most recent year for which PTI tonnage statistics were available at the time of this publication) slightly over 72,000 tons of tendons were used in residential ground-supported foundations, representing about 375,000 individual homes.

Initial design techniques for post-tensioned ground-supported foundations, both California and ribbed foundations, were empirical and somewhat arbitrary. Building code requirements provided little guidance for engineers designing these foundations because existing post-tensioned concrete codes were developed for elevated structures where the applied loads and the responses to them can be determined with reasonable accuracy. Very little research had been done on the complex effects of soil volume changes and their resultant soil loadings on these types of foundations. Thus engineers relied upon the observable performance of existing post-tensioned foundations and developed simplified but rational design techniques to model the complex effects of differential soil movement. Most of the early design methods involved the assumption of a complete loss of support under the foundation for an arbitrary distance (in some cases specified by the geotechnical engineer). The post-tensioned foundation was designed to "span" this arbitrary distance at any point throughout the foundation, supporting all required superimposed loads, without exceeding commonly used stress or deflection limits for elevated structures. This design method, called "spanability," has been widely used, primarily in California and Nevada, on soils with Expansion Indices (EI) up to 130.

In 1976, the Post-Tensioning Institute initiated a research effort at Texas A & M University for development of new procedures for soils investigation and rational design tech-

Fig. 11.1 Ribbed Foundation

niques for post-tensioned ribbed foundations supported on expansive or compressible soils. This research included review of past design and construction practices and the performance of post-tensioned ground-supported foundations, as well as extensive computer studies of post-tensioned foundations supported on expansive or compressible soils. The Texas A & M research study resulted in the publication in 1980 of *Design and Construction of Post-Tensioned Slabs-on-Ground* (First Edition), which included detailed recommendations for soils investigations, materials, plans and specifications, installation, field procedures, and structural design procedures for ribbed foundations, including design examples.

Since 1980, the structural design procedures presented in the first edition of *Design and Construction of Post-Tensioned Slabs-on-Ground* (generally known as the "PTI Method") have become the standard design method for post-tensioned ribbed foundations and have been extensively applied with good results. In 1988, the ribbed foundation design method was added to the Uniform Building Code Standards and to the Standard Building Code, as an acceptable method for designing ground-supported ribbed foundations on expansive soils.

The second edition of *Design and Construction of Post-Tensioned Slabs-on-Ground*, published in 1996, contained extensive editorial modifications and clarifications. The design procedures were expanded to include uniform thickness foundations, which have strength and stiffness equivalent to that of the ribbed foundation. This edition of the manual was incorporated into the International Building Codes and the NFPA 5000 code.

Published in 2005, the third edition of *Design of Post-Tensioned Slabs-On-Ground*[11.2] contained a major change in the method for calculating geotechnical parameters, changes and updates to some of the structural equations, and extensive editorial modifications and clarifications. To clarify intent and distinguish between "recommendations" and "requirements," this document is accompanied by two documents titled *Building Code Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils*[11.3] and *Building Code Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils*.[11.4] These two documents are written in mandatory code language and have been adopted into the 2006 International Building Code.

11.2.2 Selection of a Design Procedure

PTI's *Design of Post-Tensioned Slabs-On-Ground*[11.2] contains three distinct and separate design procedures. The selection of a procedure is determined by the type of soil that will support the foundation. The procedures are applicable for foundations supported on expansive, compressible, or stable soils.

Sites for which the expansive soil design is applicable should have an expansion index (EI) greater than 20 when determined in accordance with ASTM D4829. Alternately, the soils should meet all of the following criteria:

- Plasticity Index (PI) of 15 or greater when determined in accordance with ASTM D4318
- More than 10% of the soil particles pass a No. 200 sieve (75 μm), determined in accordance with ASTM D422

Fig. 11.2 Uniform Thickness Foundation

- More than 10% of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D422

Sites for which the compressible soil design procedure is applicable will be sites in which the predominant geotechnical effect could be settlement under the imposed loads of the structure or fill. A definition of compressible soils is provided as follows:

- Soil in which the consolidation pressure is greater than the preconsolidation pressure as found on the e-log P relationship developed from consolidation testing, provided that the average applied pressure taken over the entire area of the foundation is 500 psf or smaller.
- If the applied average pressure does not exceed the preconsolidation pressure for a depth within 0.85 the width of the entire foundation, it is unlikely that the site is compressible.

Sites in which the soils are considered to be stable, or non-active, due to expansive clay activity or compressibility can generally utilize the stable soil design procedure.

11.2.2.1 Expansive Soil Design

The procedure for designing shallow foundations on expansive clay soil sites begins by using a rational means to evaluate the soil support parameters; predicting soil movement based on both local climatic factors and the properties of the on-site soil. Based on these predicted movements, specific structural design formulas and procedures are then presented to determine the moments, shears, and deflections in a ribbed foundation. The resulting ribbed foundation can then be converted (if desired for constructability purposes) into a slab of uniform thickness based on providing the same gross moment of inertia as the ribbed slab section. The complete design procedure detailed in this document has been incorporated into the current editions of the International Building Codes, and the NFPA 5000 Building Code. Ground-supported slabs designed using the procedures in this document are specifically excluded from the provisions of ACI 318-05.

The first step in designing a ground-supported foundation is to determine the properties of the supporting soils. The procedure provides the ability to model soil conditions by incorporating extensive databases and research from the USDA Natural Resources Conservation Service, National Soil Survey Center, and allows for flexibility in evaluating vertical moisture barriers, planter areas, and variable soil suction values controlling the suction conditions at the surface of the soil profile. In the case of expansive clay soils, the geotechnical engineer will use the procedure to evaluate the edge moisture variation distance (e_m) and the differential soil movement (y_m). The edge moisture variation distance and the differential soil movement are graphically illustrated in Fig. 11.3, and are based on the anticipated deformation of a section of unrestrained soil when subjected to wetting and drying conditions.

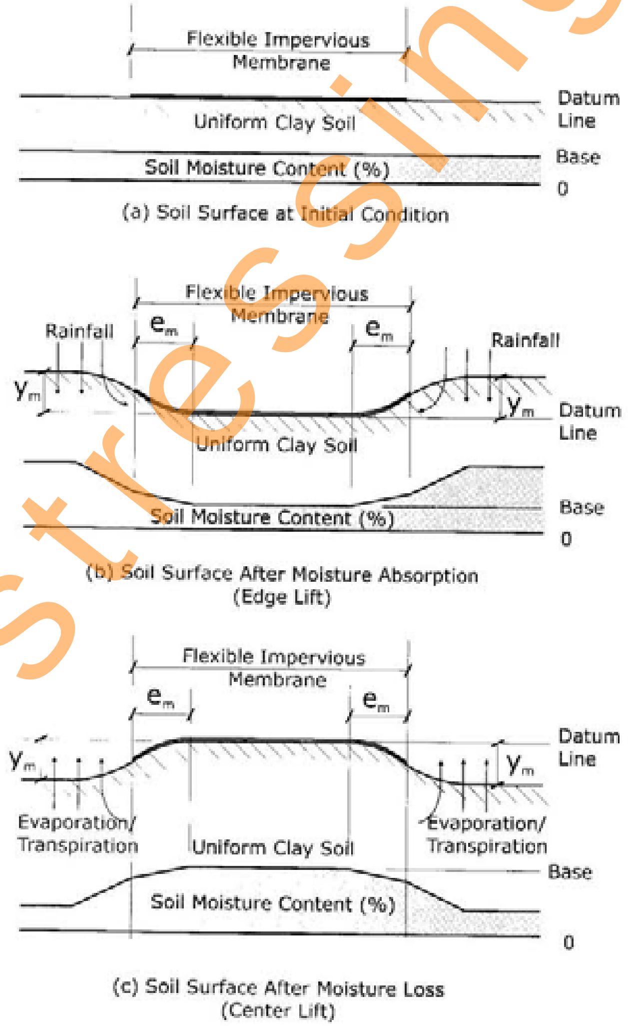

Fig. 11.3 Determination of e_m & y_m

The edge moisture variation distance (e_m) is the distance beneath the edge of a shallow foundation within which moisture will change due to wetting or drying influences around the perimeter of the foundation. In an edge lift condition, the moisture in the soil is higher at the edges than in the center. In a center lift condition, the moisture is higher in the center than at the edges. The edge moisture variation distance is dependent to a large degree on the climate, using the Thornthwaite Moisture Index (I_m) as a value that represents the long-term climatic condition of the soil in a given region. The other major contributing factor is known as the unsaturated diffusion coefficient, which is calculated from actual site soil properties.

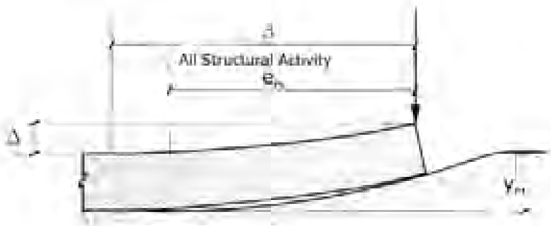

Fig. 11.4 Edge Lift

Differential soil movement (y_m) is an estimate of the change in soil surface elevation over a distance between the edge of the slab and the calculated e_m distance. The amount of differential soil movement to be anticipated can depend on a number of conditions, including the type and amount of clay mineral in the soil, depth and uniformity of the clay layers, the initial wetness, and the depth of the active zone, which is a measurement below the ground surface to which a change in moisture content can be expected.

Once these soil parameters are determined by the geotechnical engineer, the structural designer can use the design procedure to design a shallow post-tensioned foundation that will satisfactorily perform in the site-specific soils. The design procedure is based on a working stress, or serviceability method. Moments, shears, and differential deflections under the action of applied service loads (including the soil loading, dead loads superimposed by the supported structure, and anticipated live loads) are predicted using equations developed from empirical data and a computer study of a plate on an elastic foundation. Concrete stresses caused by these moments and shears are limited to specific allowable values. Differential deflections in the slab are limited to acceptable values by providing a minimum foundation stiffness that is compatible with the type of superstructure construction being utilized. Structural framing systems that can accept more movement without creating serviceability issues are allowed a higher degree of slab deflections.

An analysis is performed on the foundation for each of the two bending conditions, edge lift (Fig. 11.4), which occurs when moisture increases at the edge of the slab, and center lift (Fig. 11.5), which occurs when moisture evaporates from the edges of the slab.

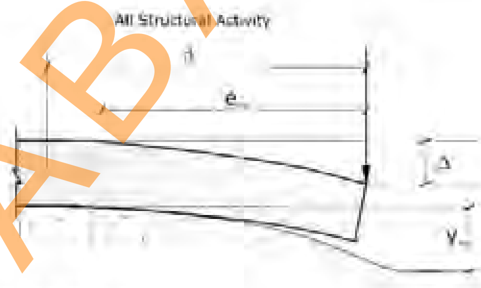

Fig. 11.5 Center Lift

11.2.2.2 Compressible Soil Design Summary

The design procedure for foundations on compressible soils is similar to the expansive soils procedure, except that differential deflection equations are used and the primary bending deformation is typically similar to that shown in Fig. 11.5. On compressible soil sites the geotechnical engineer simply provides the estimated total settlement in the center and differential settlement between center and edges of the structure based on applied average structural loads and anticipated depths and thicknesses of fill.

Because compressible soils may have local settlement areas, it is often recommended that in addition to the settlement analysis, an analysis should also be performed using the procedures for center lift expansive clay. If the geotechnical engineer determines that loose granular soil or shallow compressible fine grained soil is present below the foundation footprint, the foundation can be analyzed for compliance with shear, moment, and deflection in accordance with the expansive soil procedure for center lift assuming e_m = 5.0 ft and y_m taken as the estimated differential settlement.

11.2.2.3 Stable Soil Design Summary

Sites that are considered to be non-active due to expansive clay activity or compressibility may utilize any type of foundation that can rely on non-active subgrade support. Such sites could include outcropping rock, properly consolidated sand or gravel deposits, or non-expansive soil which has proper density. These types of foundations need only be checked for bearing, and lightly reinforced against temperature and shrinkage cracking.

Post-tensioned tendons can be used as an economical means of temperature and shrinkage reinforcing in this type of slab, and are typically employed to minimize joints and cracks in the slab. The recommended design procedure for these slabs is to provide a minimum prestress force of 0.050A, after all losses including the effects of subgrade friction. In areas where excessive shrinkage cracking is anticipated, cracking can be mitigated by increasing the minimum prestress force to 0.10A. This type of slab is typically constructed as a 4 in. or 5 in. thick slab without stiffening ribs. Edge turn downs or other types of footings are designed to carry the loads from the supported structure.

Further discussion on the design of this type of slab, also referred to as a BRAB Type II Slab, can be found in Section 11.3.1.

11.3. POST-TENSIONED SLABS USED IN COMMERCIAL AND INDUSTRIAL APPLICATIONS

The primary reason for the economy of post-tensioned industrial floors is the reduced thickness of the concrete slab permitted because of the compressive stress induced in the concrete by the post-tensioned tendons. Additional savings in industrial floor costs are provided by the elimination of most slab joints and reduced construction time. Elimination of joints and cracks reduces both slab and vehicle (fork lift) maintenance costs, further reducing life cycle costs of post-tensioned floors. For floors of prefabricated metal buildings the post-tensioning can also serve as a tie for the horizontal reaction from the building columns. This eliminates the need for reinforcing bars in the slab to dissipate the column reaction, and eliminates the possibility of slab cracking associated with such details.

The BRAB reports discussed herein established four basic slab types, designated as follows:

- Type I - Unreinforced
- Type II - Lightly reinforced against temperature and shrinkage cracking
- Type III - Reinforced for loads
- Type IV - Structural (not directly supported on the ground)

Commercial and industrial applications can include slabs in the Type I and Type II categories, where post-tensioning is utilized to eliminate joints and/or to control shrinkage and temperature cracking; however, most commercial and industrial floors that are subjected to any sort of heavy loading or traffic patterns would fall under the Type III designation.

11.3.1 Design of Type II Slabs

The most common type of cracking found in slabs-on-ground is caused by restraint-to-shortening (RTS). Concrete slabs will shorten due to the effects of shrinkage and axial creep as the concrete ages. In ground-supported slabs this shortening is restrained to some degree by friction between the slab and the supporting soil. Shortening can also be restrained by the "keying" action of any beams or footings that lock the slab into the soil. The main goal of this design procedure is to provide enough compression in the slab to either overcome the restraint-to-shortening caused by slab-subgrade friction, or to substantially close any RTS cracks that may develop prior to stressing the tendons.

A complete discussion of RTS cracking can be found in Ref. 11.5.

An extensive review of technical literature was performed in order to determine the value of the coefficient of friction that might be expected to be effective during tendon stress-

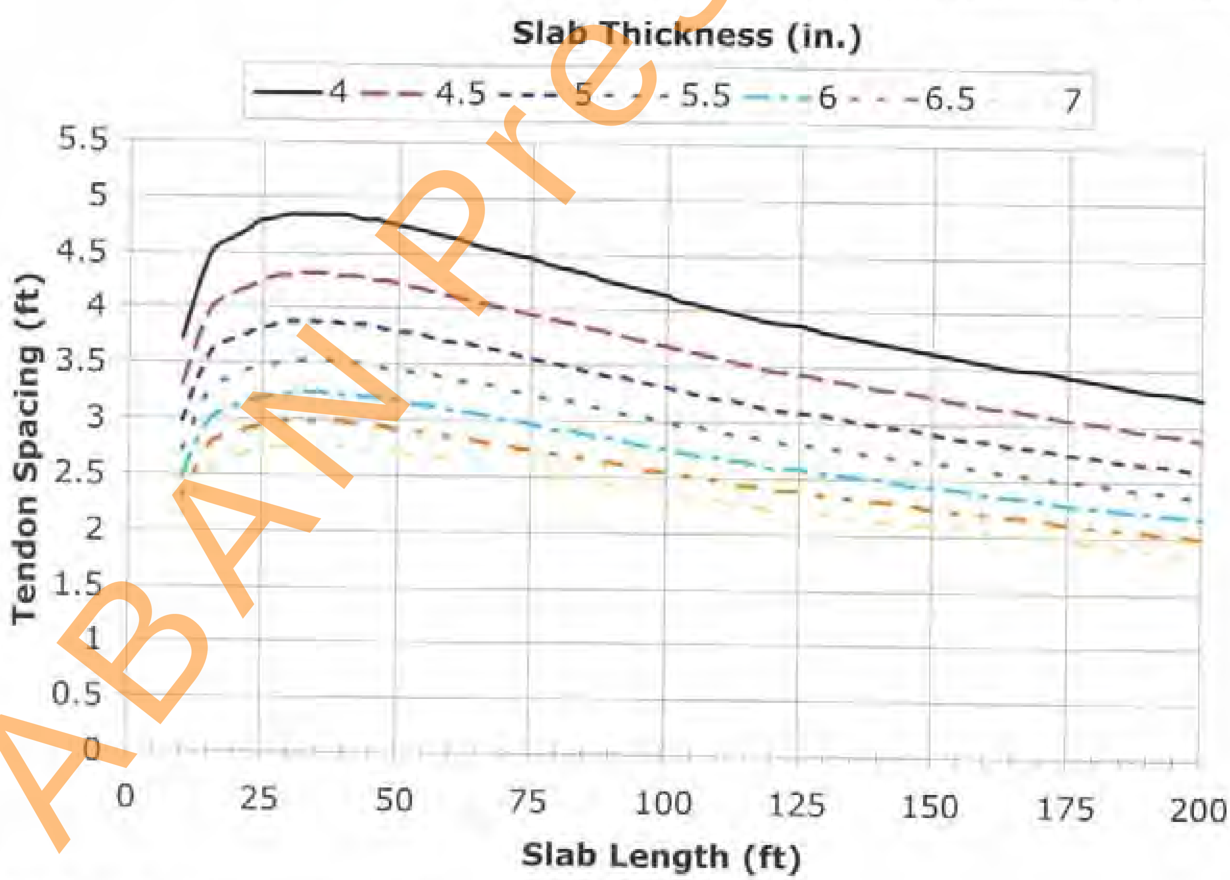

Fig. 11.6 Tendon Spacing (prestress force = 0.10A, coefficient of friction = 0.75)

ing. This information can be found in PTI's *Design of Post-Tensioned Slabs-On-Ground*[11.4] along with a table of anticipated friction coefficients relative to the type of sub-grade medium on which the slab is constructed. Once the coefficient of friction (μ) is determined, the tendon spacing can be determined as follows:

$$\text{tendon spacing} = \frac{F_e}{(f_p \times (12H)) - \left[W_{slab} \times \frac{L_s}{2} \times \mu\right]}$$

Where:

- F_e = final effective force per tendon (lb)
- f_p = desired minimum average residual compressive stress (psi)
- H = slab thickness (in.)
- W_{slab} = weight of slab (lb/ft²)
- L = total slab length (ft)
- μ = coefficient of friction between slab and subgrade

Using slabs with a unit concrete density of 150 lb/ft³, the weight of the slab can be calculated as:

$$W_{slab} = \frac{H}{12} \times 150$$

The recommended design procedure for Type II slabs is to provide a minimum prestress force of 0.050A, after all losses including the effects of subgrade friction. In areas where excessive shrinkage cracking is anticipated, cracking can be mitigated by increasing the minimum prestress force to 0.10A. Fig. 11.6 shows a plot of tendon spacings in slabs of various thicknesses and lengths, with a minimum prestress force of 0.10A and a coefficient of friction of 0.75 (typically used for slabs cast on a sand base).

11.3.2 Design of Type III Slabs

Many non-prestressed reinforced industrial and commercial floor slab designs are developed solely on the basis of experience or rules of thumb for determination of slab thickness and reinforcement requirements. Alternate post-tensioned designs may be developed for such floors by equating the tensile stress due to load-induced bending moments in the reinforced concrete slab to the net tensile stress in the post-tensioned slab. The recommended design procedure for Type III slabs is to provide a minimum prestress force of 0.10A, after all losses including the effects of subgrade friction.

Taking the minimum recommendations for prestress force into account, the required thickness of a post-tensioned slab can be determined based on equivalency to the non-prestressed slab. The flexural (bending) capacity of a slab with non-prestressed reinforcement placed at its center of gravity (CGC or middle of the slab) is reached when the applied bending moment produces the maximum allowable flexural tensile stress f at an extreme fiber. In a prestressed slab, however, in order to produce the same flexural tensile stress f at that same extreme fiber, the applied moment must first overcome the precompression stress P/A. This effectively increases the allowable flexural tensile stress in the post-tensioned slab to $f_{allow} = f + P/A$. The allowable flexural stress f can be conservatively estimated as $2\sqrt{f'c}$. The required equivalent thickness of a post-tensioned slab with tendons located at the midpoint of the slab can be calculated as follows:

$$H = \sqrt{\frac{f \times 2(H_{nonps})^2}{f_{allow}} \times \frac{1}{2}}$$

Where:

- H = thickness of the prestressed slab (in.)
- H_{nonps} = thickness of the non-prestressed slab (in.)
- f = allowable flexural tensile stress in the non prestressed slab (psi)
- f_{allow} = allowable flexural tensile stress in the prestressed slab (psi)

Utilizing this conversion theory, sample equivalent slab thicknesses are presented in the table below.

Table 11.1 - Equivalent Thicknesses For Prestressed Slab-On-Ground

Non-Prestressed Thickness	Prestressed Thickness	Required Prestressing Force
5 in.	4 in.	100 psi
6 in.	4 in.	125 psi
6 in.	4.5 in.	100 psi
7 in.	5 in.	100 psi
8 in.	5.5 in.	125 psi

A more precise design analysis for floor slabs with heavy loadings can be obtained based on either Westergaard's Theory[11.6] or on the approach developed by Panak and Rauhut[11.7] which considers the interaction of the slab and subgrade stiffness and wheel and uniform loading. The Panak and Rauhut article includes charts which can be readily adapted to post-tensioned slab designs by adding the precompression due to post-tensioning to the allowable tensile stress in the reinforced concrete designs to obtain a modified design tensile stress. The same procedure can be applied to design of post-tensioned slabs using the Westergaard Theory.

11.4 POST-TENSIONED SPORT COURTS

Tennis courts, basketball courts, and smaller multi-purpose courts can be designed and constructed using a variety of methods and materials. Factors to consider in selecting construction methods are the preference of the players who will be using the court, the climate, construction costs, maintenance costs, and repair costs. The United States Tennis Association (USTA) asserts that a primary factor in the type of construction selected is the amount of supervision to be provided at the court facility. For courts with limited supervision constructed in areas such as parks, schools, or other public areas, consideration should be given to the advantages of a strong durable playing surface and one that can take some abuse without damage.

Concrete can provide a sturdy, durable playing surface that will withstand abuse and is less susceptible to the effects of climate (heat, cold, and rain) than an asphalt court. When properly designed and constructed, a concrete court will need very little care and maintenance, and will have an extremely long useful life. Concrete courts can also be used right after a rain because of their good drainage characteristics. In addition, they offer a non-discoloring surface, controllable ball skid length, and they can receive many types of playing-surface topping systems.

The American Sports Builders Association states in its *Tennis Court Construction Guidelines*[11.8] that post-tensioned concrete slabs are the preferred method of concrete court construction. Post-tensioned concrete courts provide the additional advantages of reducing or eliminating control joints, and resisting and controlling cracking due to the compression induced into the concrete by the post-tensioned tendons. A post-tensioned court will also typically have a higher flexural capacity than a slab of the same thickness reinforced with non-prestressed reinforcing steel such as rebar or heavy mesh.

Slabs used in the construction of sport courts should be designed as Type II slabs described in Section 11.3.1 above. For additional assurance against the development of cracking, the recommended design procedure for sport courts is to provide a minimum prestress force of 0.10A, after all losses including the effects of subgrade friction. Ref. 11.9 provides additional design and construction guidance on other important considerations such as detailing to prevent restraint-to-shortening that can be caused by edge beams, fence post footings, and other elements common to sport court construction.

11.5 OTHER TYPES OF POST-TENSIONED SLABS-ON-GROUND

Other types of post-tensioned slabs-on-ground include parking lots, highway pavements, airport runway construction, and heavy mat foundations supporting high-rise structures. Most of the general advantages described in Section 11.1.1 apply to these applications. The elimination of the majority of the control joints in parking lots and other pavement applications increases the durability of the paving and reduces maintenance and other related costs.

The use of post-tensioned concrete pavements dates back to the 1950s with the construction of a taxiway at the International Airport in San Antonio. A summary of many of the highway and airport runway paving projects that have occurred around the United States can be found in a report published in 2003 by the Center for Transportation Research, University of Texas at Austin (CTR Report No. 0-4035-1).[11.10] This report documents several full-scale highway projects that have been undertaken dating back to 1973 in Pennsylvania and 1977 in Arizona and Mississippi. More recent projects were constructed during the 1980s in various locations in Texas. These projects utilized cast-in-place concrete with unbonded post-tensioned tendons, and reports generally indicate excellent performance.

More recently the FHWA has sponsored several highway construction projects that utilized precast pavement segments with longitudinal and transverse post-tensioning used to connect the segments and form one continuous pavement section from expansion joint to expansion joint. Completed projects include one outside of Austin, Texas in 2002 and one in Southern California in 2004.

Both of these projects incorporated precast prestressed (offsite) pavement segments, and site-installed post-tensioned reinforcing consisting of 0.60 in. diameter single-strand bonded tendons in the longitudinal and transverse directions. Among the intent of these projects is to develop precast/post-tensioned pavement as a method of rapid pavement replacement in locations where long-term traffic disruption or detour is not feasible or desirable.

Post-tensioned pavements have been utilized to construct sections of runways at several airports throughout the United States. Examples exist at two airports in Illinois. In 1980 at the O'Hare International Airport two segments of post-tensioned pavement were placed on the east end on Runway 27L. Each segment was 400 ft long, 150 ft wide, and 8 to 9 in. thick. This project utilized bonded post-tensioning in both longitudinal and transverse directions. A recent condition survey conducted in January 2001 indicated no existence of cracks or other distresses. After 22 years of service under stringent conditions (50,000 to 60,000 departures annually over the life of the pavement),

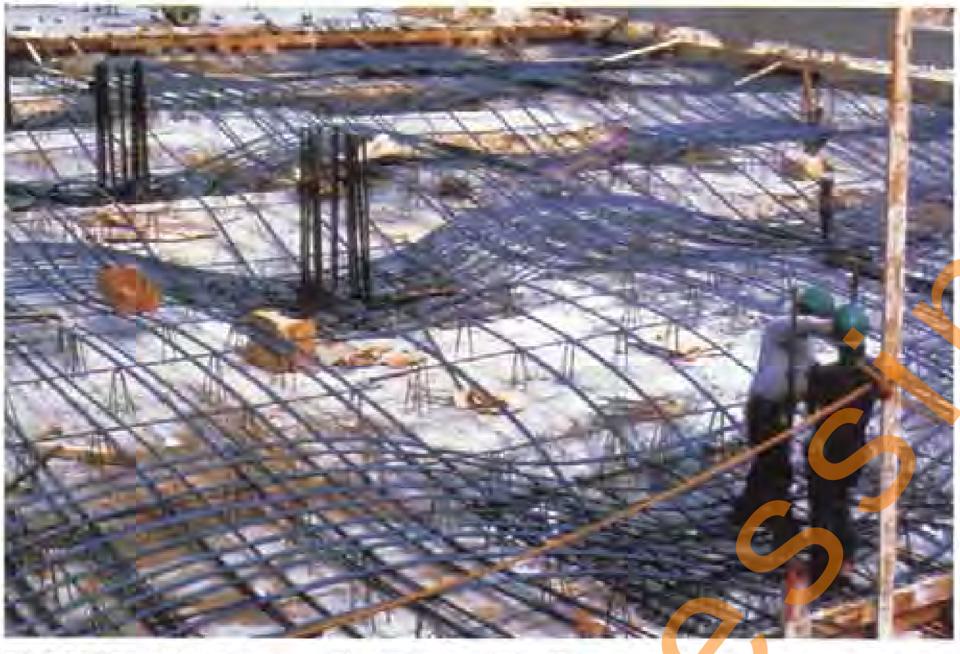

Fig. 11.7 Heavy Mat Foundation with Tendons Profiled to Balance and Distribute Column Loads

the PCP was reported to be in outstanding condition. Except for a problem caused by snowplows in a joint, there has been no maintenance at all on the slabs.

The second project was constructed in 1993 at the Greater Rockford Airport in Illinois and is part of the primary taxiway serving the United Parcel Service air to ground terminal at the airport. This project utilized bonded post-tensioning in the longitudinal direction only and incorporated shrinkage-compensating cement. The 10-year study published by the Federal Aviation Administration indicates that the pavement is in excellent condition and has not required any maintenance. The only distress noted in the post-tensioned pavement is a small longitudinal crack at a location where the cargo planes turn off of the taxiway, which probably could have been avoided by adding some transverse reinforcing in the pavement design.

Post-tensioned reinforcing has also been used to construct heavy mat foundations designed as large floating slabs supporting high-rise or other heavy structures. In this type of design the post-tensioned tendons can be placed in a parabolic curve with the low points occurring under the columns which support the elevated structure. The tendons are designed to help balance the point loads imposed by the columns and distribute the loads uniformly throughout the entire slab.

11.6 MATERIALS

The most commonly used system in post-tensioned slab-on-ground construction consists of ½ in. diameter 270 ksi single-strand unbonded tendons. The use of a standard (non-aggressive environment) system is typical unless the concrete will be exposed to, or in direct contact with, chlorides from deicing chemicals, salt water, brackish water, sea water, spray from these sources, or soils containing high levels of chlorides. These conditions would warrant the use of an encapsulated (aggressive environment) system. Additional information on post-tensioning materials used in slab-on-ground construction can be found in PTI's *Specification for Unbonded Single Strand Tendons*.[11.1]

Foundations for residential and light commercial construction, slabs in light commercial applications, and sport courts will typically utilize concrete with 28-day compressive strengths in the range of 2500 psi to 3000 psi, subject to the durability requirements contained in the International Residential Code. Heavier slabs used in industrial applications or pavements will utilize concrete strengths determined to be applicable to their specific use and environment.

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BRIDGES

12.1 INTRODUCTION

This chapter describes the principal applications of post-tensioning in bridge design and construction, identifies the primary technical issues associated with these applications, and gives guidance on how these issues can best be addressed to create post-tensioned bridges that are functional, cost-effective, durable, and aesthetically pleasing.

The chapter is intended primarily for practicing engineers who wish to gain a broad understanding of the current state-of-the-art for post-tensioned bridges and to apply this knowledge on design projects. It will also be of interest to engineers who work for entities that own and manage bridges, and who wish to expand their knowledge of post-tensioned bridges as a basis for making informed choices on design alternatives. Finally, it will be a useful resource to professors and students of structural engineering in support of teaching and learning activities at the graduate or advanced undergraduate level.

It is expected that readers of this chapter will be knowledgeable in the fundamental aspects of behavior, design, and detailing of post-tensioned concrete structures (which is covered in Chapter 5 of this manual). In addition, it is assumed that readers will be familiar with the basic concepts of bridge engineering, including functional requirements, geometry, and loadings. No knowledge of any specific design standard is assumed. It is expected, however, that readers will be familiar with the general philosophy of modern design standards regarding acceptable structural behavior at serviceability limit states and ultimate limit state.

The chapter is divided into the following sections:

The benefits of post-tensioning in bridge design and construction are described in Section 12.2. Post-tensioning enables the casting of concrete to be de-coupled from the stressing of tendons, which allows the benefits of prestressing to be brought to the widest possible range of applications.

The history of post-tensioned bridges is briefly reviewed in Section 12.3. Post-tensioning has been well established in bridge design and construction for over fifty years. During this time, post-tensioning has enabled the creation of innovative structural systems, and has made possible an evolution towards the true mechanized construction of concrete bridges.

Design concepts are discussed in general terms in Section 12.4. The most important elements of design concepts are the longitudinal structural system, cross-sections, method of construction, and tendon layouts. Design concepts form the basis for final design and have a profound impact on the quality of the finished product.

This section is followed by a description of the most important types of post-tensioned concrete bridges currently in use. Section 12.5 describes girder bridges, the most common type of post-tensioned bridge. Sections 12.6 through 12.8 present slab bridges, frame bridges and arch bridges respectively. Section 12.9 briefly discusses other types of bridges. All of these sections describe possibilities for longitudinal structural systems, cross-sections, methods of construction, and tendon layouts.

Section 12.10 describes other important applications of post-tensioning in bridges, including piers, concrete deck slabs for steel girder bridges, and rapid rehabilitation of existing bridges.

The future of post-tensioned bridges is explored in Section 12.11, which discusses new challenges and new opportunities that are expected in the foreseeable future.

An extensive set of references is included at the end of this chapter, which will be useful to readers who wish to study topics covered in this chapter in greater detail. Particularly rich sources of information among the references listed are the classic works by Menn,[12.1] which primarily covers cast-in-place bridges, and Podolny and Muller,[12.2] which primarily covers precast concrete bridges.

12.2 BENEFITS OF POST-TENSIONING IN BRIDGE DESIGN AND CONSTRUCTION

Post-tensioning makes possible the cost-effective construction of high-quality bridges over a wide range of conditions. The primary benefits of post-tensioning are as follows:

- Post-tensioning enables concrete bridges to be designed and built cost-effectively for practically all span lengths.
- Post-tensioning enables bridges to be designed and built cost-effectively for any highway alignment.
- Post-tensioned structures have high intrinsic durability.
- Post-tensioning enables concrete bridges to be built quickly, with minimum impact on the human and natural environment.
- Post-tensioning creates rich opportunities for aesthetic expression.

These benefits are due in part to the fact that post-tensioning is a method of prestressing. Prestressed concrete members are put into a state of pre-compression by an equal and opposite force in steel tendons that are attached to concrete through mechanical anchorage or bond. This pre-compression force can be dimensioned to eliminate tensile stresses in concrete under service conditions, thus preventing the formation of cracks at this level of demand. By applying the pre-compression force eccentrically, upward deflections can be induced in members, which can further improve behavior under service conditions. The control of cracking and deflections under service conditions made

Fig. 12.1 Confederation Bridge, Canada

possible by prestressing allows concrete structures to be made lighter and more slender for a given span and loading than would otherwise have been possible with reinforced concrete alone.

Although these properties are desirable, they are not in themselves sufficient to produce the benefits listed previously. It is possible, for example, to prestress concrete members by stressing steel strands against rigid abutments, placing concrete around this steel, and transferring the force in the steel from the abutments to the concrete once it has hardened. This method, called pre-tensioning, is used primarily to produce linear concrete members such as I-beams that are then transported to site and erected. Pre-tensioning cannot be used to connect two precast concrete components, nor to connect precast components to cast-in-place concrete. The spans and geometry of structural systems that use pre-tensioning alone are thus relatively limited.

Post-tensioning, on the other hand, involves casting ducts rather than stressed tendons into concrete. After the concrete has hardened, tendons are installed into the ducts and stressed against mechanical anchorage devices cast into the structure at the ends of the ducts. The casting of concrete is thus effectively decoupled from the stressing of prestressing steel, allowing these two operations to be separated from each other in both space and time. It is this decoupling of concrete casting and stressing of tendons, in addition to the intrinsic advantages provided by prestressing, that are primarily responsible for the benefits of post-tensioning listed at the beginning of this section. These benefits are discussed in further detail:

1. **Post-tensioning enables concrete bridges to be designed and built cost-effectively for practically all span lengths:** This proposition is illustrated by the Confederation Bridge, shown in Fig. 12.1. This bridge, which links the Canadian provinces of New Brunswick and Prince Edward Island, has spans of 250 m. These spans are among the longest of any girder-type bridge, regardless of material. The entire bridge, including the substructure, was assembled from large precast concrete components, which were made continuous to produce a structural system with sufficient integrity to withstand severe design loads. Post-tensioning enabled these important connections to be made over water, quickly and effectively, independent of activities in the casting yard.
2. **Post-tensioning enables bridges to be designed and built cost-effectively for any highway alignment:** The adaptability of post-tensioned bridges to challenging alignments is dramatically demonstrated by the structure shown in Fig. 12.2. This bridge, built along Switzerland's Gotthard Highway in the 1970s, accommodates the tight curves required of a steep mountain highway, while minimizing the impact of the road on the land below. Post-tensioning enabled the prestressing steel to follow the sharp curves of the highway alignment, which made possible a cost-effective and visually elegant solution.

3. **Post-tensioned structures have high intrinsic durability:** All other conditions being equal, it is generally agreed that uncracked concrete is more durable than cracked concrete. Because prestressing can limit tensile stresses in concrete to levels that will prevent the formation of cracks under service conditions, it is an effective means of enhancing the durability of concrete structures. This beneficial effect can be maximized through the use of post-tensioning, which allows designers to limit tensile stresses in any given direction, and in any given structural component. This is a considerable improvement over structures where prestressing is provided only by pre-tensioning, for which tensile stresses can be limited only in those components that are actually pre-tensioned, and only in the direction of pre-tensioning. For example, if post-tensioning is not used, it is not possible to prestress the deck slab of bridge superstructures consisting of precast, pre-tensioned I girders and cast-in-place deck. The deck slab, arguably the most vulnerable component with regard to durability, is thus susceptible to cracking under service conditions. When post-tensioning is used, however, it is relatively easy to prestress structural components in any number of orthogonal directions, and hence to eliminate computed tensile stress under service conditions. On the Confederation Bridge, shown in Fig. 12.1, post-tensioning enabled the deck slab to be prestressed both longitudinally and transversely. The project requirements specified a design life of 100 years that would be difficult to achieve without post-tensioning of the deck slab.
4. **Post-tensioning enables concrete bridges to be built quickly, with minimum impact on the human and natural environment:** Post-tensioning is a fast and effective means of connecting concrete components together into a continuous structure. This enables large structural systems to be assembled from relatively small elements. The use of small precast segments minimizes the need for large construction equipment and facilitates erection on sites that are tightly constrained by existing structures and other facilities. When used in conjunction with the precast segmental method of construction with match-cast joints (described in greater detail in Sections 12.5.7 and 12.5.8), these elements can be quickly made structurally continuous. Superstructure erection rates of more than one span per week are not uncommon. These concepts are illustrated by the structure shown in Fig. 12.3. This bridge replaced an existing viaduct on its original alignment in a densely populated neighborhood. The use of precast concrete segments assembled with post-tensioning allowed the total construction time to be minimized, which effectively minimized disruption to motorists and neighborhood residents.

Fig. 12.2 Cast-in-Place Post-Tensioned Bridge, Switzerland
Courtesy of *BBR International*

Fig. 12.3 BQE Connector Ramp, Brooklyn, NY

Fig. 12.4 Sunniberg Bridge, Switzerland

Fig. 12.5 Bridge Over the Marne at Esbly, France

5. **Post-tensioning creates rich opportunities for aesthetic expression**: Post-tensioning through the decoupling of casting and stressing, gives designers far greater freedom to shape the visible form of structures than pre-tensioning. Pre-tensioned members are typically constrained to straight, linear elements. In general, the opportunities for visual expression offered by post-tensioning are limited only by economic factors, and not by technical feasibility. The visual possibilities created by post-tensioning are exemplified by the bridge shown in Fig. 12.4. This structure, built in the mountains of Switzerland, is a multiple-span post-tensioned concrete cable-stayed bridge on a curved alignment. The bold yet balanced form of this bridge shows how, in the hands of a gifted designer, post-tensioning can be used to endow structures with rich aesthetic significance.

12.3 HISTORICAL OVERVIEW

This section presents a brief summary of the history of post-tensioned bridges. The focus is on major developments in structural systems and methods of construction that produced significant increases in the value, broadly defined, created by post-tensioned bridges. Because the developments in bridge technology that will be considered in this section remain in current use, they are of interest for more than purely historical reasons, and will be discussed in greater detail from a technical perspective in subsequent sections of this chapter. In the current section, these developments will be described within their historical context to gain insight into the needs and opportunities that led to their creation.

Reconstruction of European bridges destroyed in World War II provided the primary impetus for the initial stage of development of post-tensioned bridges. Had an adequate supply of steel been available, it is likely that girder bridges spanning between 25 (82 ft) and 100 m (328 ft) would have been rebuilt in this material. Steel was in extremely short supply, however, as the war had also destroyed much of Europe's steelmaking capacity. The challenge was therefore to find ways to design and build concrete bridges cost-effectively in the 25 to 100 m (82 ft to 328 ft) range of spans.

A concerted effort to create a practical means of post-tensioning concrete had been underway since the 1920s. Among the developers of post-tensioning technology active during this period, the most influential was the French engineer and builder Eugene Freyssinet. The body of knowledge and experience produced by the theoretical and practical work of Freyssinet and his contemporaries before and during the war provided the basis for the successful post-war development of cost-effective post-tensioned systems for medium span bridges, which satisfied Europe's urgent need to rebuild in a time of scarce materials.

A remarkable example of post-tensioned bridge construction from the early post-war years is Freyssinet's bridge over the Marne at Esbly, France (Fig. 12.5), completed in 1949. This work is visionary in its use of post-tensioning

Fig. 12.6 Weinland Bridge, Switzerland

Fig. 12.7 Nibelungen Bridge, Germany

not only as a means of enabling the structure to carry load more efficiently, but also as a means of rationalizing its construction. The span is 74 meters and the ratio of span to mid-span depth exceeds 40:1. These proportions convey a visible slenderness that remains impressive to this day. The bridge was assembled from small precast concrete elements, post-tensioned together into larger units for erection, and post-tensioned again to make the entire structure continuous. The chosen construction method minimized site labor and falsework. Freyssinet designed and built the bridge at Esbly as one of a set of five identical bridges crossing the Marne, which reduced the cost of precasting through re-use of formwork and repetition of operations. Taken together, these bridges are arguably the first significant example of industrialized construction of concrete bridges made possible by post-tensioning.

Notwithstanding the achievements of Freyssinet's Marne bridges, the opportunities for rationalizing construction offered by post-tensioning were not fully exploited in the 1940s and 1950s. Post-tensioned bridges of this era were generally built using methods identical to those used for reinforced concrete bridges. The Weinland Bridge, built in Switzerland in 1958, is an example of this type of bridge (Fig. 12.6). As with reinforced concrete bridges, this structure was built of concrete that was cast in place into single-use formwork supported on single-use falsework. Reinforcing steel was assembled into the formwork on site bar by bar. Post-tensioning tendons were bonded to concrete after stressing by injecting grout into the post-tensioning ducts.

As wages increased in industrialized countries following World War II, this type of labor-intensive construction became expensive. In their search for ways to reduce the cost of labor in post-tensioned concrete bridges, designers recognized the need to approach the problem from a perspective in which design, detailing, and construction methods were tightly integrated. Thus began the second stage in the evolution of post-tensioned concrete bridges, which was driven by the challenge of minimizing construction labor, and which culminated in the intensively mechanized methods of construction in use today.

Fig. 12.8 Bridge Over the Inn at Kufstein, Austria

(The classical post-tensioned concrete girder bridge, cast-in-place on falsework, remains to this day a viable option on design projects where bridge geometry and conditions under the bridge enable the use of simple, standardized falsework and where speed of construction is not critical. Using reasonable means, this type of bridge can be built to the same standards of quality as bridges built using more modern methods of construction.)

Engineers in the 1950s recognized that a significant proportion of the total labor required to build cast-in-place girder bridges on falsework went into erection and dismantling of the falsework itself. The first major challenge in reducing the cost of labor in post-tensioned bridges was therefore to develop methods of construction that did not require falsework. German engineer Ulrich Finsterwalder pioneered the method of cantilever construction, by which a bridge superstructure is built out from both sides of a given pier, segment by segment. An early application of this method on a major river crossing is the Nibelungen Bridge over the Rhine at Worms, Germany (Fig. 12.7). This structure, with a main span of 114 meters, demonstrated the feasibility of post-tensioned concrete for long-span bridge construction.

As shown in Fig. 12.7, formwork was suspended from a movable structure, called a traveler that was attached to the tip of the previously completed portion of the cantilever. Construction of a given segment consisted of installing reinforcing steel and post-tensioning ducts into the forms, placing and curing concrete, installing and stressing longitudinal tendons in the top slab to resist the cantilever moment, and advancing the traveler into position for casting the next segment. In addition to the benefits of eliminating falsework, therefore, the cantilever method enabled construction to proceed according to a cyclic procedure, which enabled a more rational use of labor on site.

The cantilever method of construction has proven itself to be a remarkably versatile method of construction. From its origins in the 1950s in cast-in-place construction, it was successfully adapted to use with precast concrete segments in the 1960s, and to the construction of cable-stayed bridges in the 1970s. Cantilever construction has also been used for the construction of concrete arches with the help of temporary towers and stays. It remains the preferred method of construction for long-span concrete bridges.

The second important means of eliminating falsework, developed in the 1960s by the German engineers Fritz Leonhardt and Willi Baur, was the method of incremental launching. Using this method, bridge superstructures are built segment by segment at grade behind one of the abutments, following a cyclic procedure that consists of casting a segment, post-tensioning it to the previously completed structure, and then pushing the entire assembly thus created toward the opposite abutment. Although less versatile than cantilever construction, incremental launching has

Fig. 12.9 Oleron Viaduct, France
Courtesy of International Bridge Technologies, Inc.

Fig. 12.10 Seven Mile Bridge, Florida
Courtesy of Figg Engineering Group

had many successful applications, primarily on the construction of viaducts with a large number of relatively short spans. The Bridge over the Inn River at Kufstein, Austria (Fig. 12.8), completed in (1965), is an early application of the method.

Having developed effective methods of eliminating falsework, engineers turned their attention to other ways of reducing the cost of labor associated with the construction of post-tensioned bridges. It had been recognized as early as Freyssinet's Marne bridges that an intensive use of precast concrete eliminated much of the labor on site. Although reinforcing steel and post-tensioning ducts still had to be placed, these operations were rationalized through the off-site production of similar precast elements. The time and effort required to connect precast concrete elements using cast-in-place closure pours or grouted joints, however, proved to be prohibitive.

This problem was solved by the precast segmental method of construction using match-cast joints. This method, which was pioneered by the French engineer Jean Muller in the 1960s, was based on the use of a "perfect" joint between a given pair of precast concrete segments, which allows precast segments to be assembled on site without the need for cast-in-place closure joints. Segments are produced following a cyclic process, by which a given segment is cast against the previously cast segment that would be its mate in the completed structure. The segments thus produced are separated after casting for storage and transportation to site, and then assembled on site by post-tensioning. Since connecting a pair of segments requires only the application of epoxy adhesive to the joint faces and the stressing of tendons to provide axial compression across the joint, erection can proceed at a much faster rate than previously required for other types of joints.

The Oleron viaduct, completed in France in 1966 and shown in Fig. 12.9, is the first major bridge to be built using this method. As with practically all precast segmental bridges of the 1960s and 1970s, it was built using the cantilever method. An overhead launching truss was used to transport segments to the tips of the cantilevers and hold them in place until they were made continuous with the structure. The use of the launching truss allowed the superstructure to be built entirely from above, independent of the conditions below the bridge. The bridge demonstrates a carefully planned integration of design and construction, and achieved economy by minimizing labor through an intensive mechanization of both precasting and erection.

In cantilever construction, a cycle of installing and stressing tendons occurs for every single segment or every pair of segments. By lengthening this cycle to cover an entire span, site labor can be further reduced and speed of construction increased. This is accomplished by the span-by-span method of precast segmental construction, the most recent major advance in the evolution of post-tensioned bridge technology. As the name implies, all of the segments for a given span are erected on a temporary girder or truss, and then stressed together by tendons extending from pier to pier. The Seven Mile Bridge, built in Florida in 1982, is one of the first examples of a precast segmental bridge built span by span.

Longitudinal prestressing for span-by-span precast segmental bridges normally consists of external, unbonded tendons located inside the cavity of a single-cell box girder. External prestressing was used the 1960s as a means of rehabilitating and strengthening existing bridges, and was first applied to the construction of new bridges by Jean Muller, the inventor of the span-by-span method of precast segmental construction.

In recent years, owners and users of bridges have become increasingly concerned about the impact of bridges on the natural and human environment. Post-tensioned superstructures, which can be built quickly and without touching the land or water below the bridge are already a relatively low impact structural system. It is anticipated that the next major challenge that will drive the evolution of post-tensioned bridges will be to minimize the impact of construction even further. The issues related to this challenge are discussed in greater detail in Section 12.11.

12.4 DESIGN CONCEPTS

12.4.1 Introduction

Sections 12.5 through 12.9 of this chapter deal with technical aspects of post-tensioned concrete bridges. The focus is on knowledge that is relevant to the creation and rapid validation of concepts during the preliminary stage of the design process, rather than on the refinement of concepts into final designs. The primary topics covered are longitudinal structural systems, cross-sections, methods of construction, and arrangement of post-tensioning tendons. This section emphasizes design concepts because of their significant impact on the cost and quality of the final product, and because they require the application of skill and knowledge that goes beyond simple checks of capacity and demand.

Bridges are bid and built on the basis of a final design, which is a complete description of the structure and all of its components. Final designs are supported by a complete set of calculations that demonstrate that the bridge is in full conformance with the project requirements. The production of a final design is based on a design concept, which is an outline of a proposed solution to the problem defined by the project requirements. Concepts define the primary characteristics of bridges, to a level of detail sufficient to assess feasibility. They generally include a description of the visible form, the structural system, and the method of construction. Concepts are deemed to be feasible when they can be developed, using reasonable means, into final designs that satisfy all applicable design criteria.

Design concepts should be sufficiently detailed to allow the development of final designs to be limited to the dimensioning of components such as reinforcing and prestressing steel, and the fine-tuning of details. Given that the process of bringing a design from concept to completion involves a considerable investment of time and money, it is important to minimize changes to the primary features of the bridge during the final design phase.

Final design clearly has a significant effect on safety and serviceability of bridges. It is obvious, for example, that providing an insufficient area of longitudinal prestressing steel can lead to collapse. Final design has a much smaller impact, however, on many other important aspects of the quality of bridges. Although the final dimensions of post-tensioning tendons do affect construction cost, choices made in the concept phase (e.g. arrangement of spans) generally have a far greater impact. Some aspects of quality, such as aesthetic impact and speed of construction, are hardly affected at all by decisions made in the final design phase. It is important, therefore, to develop sound design concepts from the outset, because the impact of a poor concept on quality cannot normally be corrected in the final design phase.

Final design primarily involves the calculation of demand under design loads and capacity of structural components. Although these calculations require care as well as an understanding of structural behavior, they are generally straightforward to perform. The creation of concepts, however, requires a much broader mastery of bridge engineering, including detailed knowledge of current design practice, methods of construction, and the long-term performance of bridges in service, as well as an appreciation of the aesthetic impact of bridges. Rapid validation of feasibility requires, in addition to this, a sound understanding of simplified methods of structural analysis. A strong focus on concepts is therefore also justified from the perspective of the skills and knowledge required of designers.

12.4.2 Elements of Design Concepts

As a minimum, design concepts for post-tensioned bridges should define the following characteristics of the proposed solution:

1. **Visible form** – Rough sketches, two-dimensional engineering drawings, three-dimensional renderings, and scale models can be used to depict the visual characteristics of the concept.
2. **Longitudinal structural system** – Definition of the longitudinal structural system begins with the type of structure, the most common of which are beam-based systems, slabs, frames, arches, and cable-stayed bridges. Primary dimensions of the overall structure, including the arrangement of spans, are also defined. A description of the arrangement of bearings, internal hinges, and expansion joints should also be included.
3. **Cross-sections** – Concepts generally include complete dimensions for typical cross-sections in the superstructure and piers. Unlike steel structures, concrete dimensions are generally not calculated in the final design phase, but rather are defined in the concept phase. Concrete dimensions are often determined by detailing, e.g. providing adequate room for tendon ducts embedded in the cross-section, rather than by capacity and demand. In addition, the cost saved by fine-tuning concrete dimensions is often canceled out by the additional cost associated with changes to formwork and details.

4. **Foundations** – Concepts should define the type of foundation to be used. Foundations for post-tensioned bridges generally do not differ significantly from foundations used for bridges in other materials. For this reason, foundations will not be discussed further in this chapter.
5. **Method of construction** – Construction methods are an important element of the design concept for most post-tensioned bridges. Although the choice of the actual methods used to build a given bridge rests with the contractor, it is important that designers give serious consideration to construction methods in the development of design concepts, for three main reasons. First and foremost, designers need to demonstrate that bridges can be built using reasonable means. Second, the method of construction has important consequences for other elements of the design concept, especially the arrangement of tendons. It would be extremely impractical, if not impossible, to design a segmental bridge without reference to how it would be built. Finally, many methods of post-tensioned bridge construction require the structure to resist important loads before construction is complete. Designers need to decide whether or not these effects will be resisted by permanent structural components or using temporary measures.
6. **Arrangement of post-tensioning tendons** – Design concepts should include a description of the longitudinal layout of tendons and important post-tensioning details such as anchorage and deviation.
7. **Other characteristics** – Other characteristics should be included as required by the design criteria specific to the project, to ensure that there is an adequate basis upon which to validate feasibility and proceed with final design. On projects where special requirements regarding durability have been defined, for example, the concept should include a description of measures that will be taken to satisfy these requirements.

The elements of design concepts listed above are mutually related. Decisions made regarding any one of these elements generally have implications for the options available for the remaining elements. The relative importance of the characteristics defining design concepts must be determined by designers based on the design requirements and constraints specific to the given project. In this regard, special care should be devoted to balancing short-term and long-term requirements, which can occur, for instance, when speed of construction is given greater importance than the quality of the finished product. Although it is often faster to build simply supported spans, this generally produces a finished product that is less durable than a continuous structure.

12.4.3 Economy

As a minimum, bridges must be safe and serviceable, i.e., they must not collapse and they must be capable of performing their required function over a specified service life. Design requirements related to safety and serviceability are prescribed in detail in design codes and standards. Satisfying these requirements is all that is required. There is no perceived benefit arising from providing safety or serviceability in addition to specified levels.

After safety and serviceability, economy is usually considered to be the most important design requirement. Economy is a relative criterion, in the sense that design standards do not prescribe minimum acceptable values of cost. Rather, engineers are expected to balance cost with a number of other design requirements such as durability, speed of construction, and aesthetic quality. In this regard, designers should be mindful of the following issues:

1. Careful dimensioning of components in final design is too often regarded as the sole means to economical bridges. Just as important, however, if not more so, is the choice of sound design concepts. Menn[12.1] showed that the impact of material quantities on construction cost is relatively insignificant. For example, he observed that prestressing steel accounts, on average, for approximately 11 percent of the total construction cost of post-tensioned bridges. Assuming that a highly refined analysis in final design could reduce the quantity of prestressing steel by 15 percent relative to a simplified analysis, the total construction cost could be reduced by only 1.7 percent. It is evident that the impact of choices made in defining the design concept, such as the choice of structural system, arrangement of spans, and method of construction can be many times greater than this.
2. The pursuit of the lowest possible construction cost on a given project can inhibit the creation of economic value through innovation. As demonstrated in Section 12.2, the history of post-tensioned bridges is marked by innovations in structural systems and methods of construction that brought about major savings in materials and labor. Major developments such as these usually require contractors to work in new and unfamiliar ways, which entails additional risk relative to systems and methods that have long records of use. Faced with the cost associated with this risk, innovative solutions are often abandoned in favor of tried and true solutions, in spite of the clear benefits of the new approach. This is especially true on small projects, where there is little opportunity for contractors to benefit from the learning effect gained through repetition of identical operations.

3. It is particularly important to strike a suitable balance between economy and aesthetic quality. The relation between economy and aesthetics defies simplistic treatment. Although focusing exclusively on economy usually leads to unattractive structures, the bridges of gifted designers such as Robert Maillart are compelling evidence that the most economical solution can be endowed with rich aesthetic significance. Maillart's Salginatobel Bridge [Fig 12.11(a)], is perhaps the most important example of such a bridge. In recent years, there have been several cases in which large sums of money were spent on bridges to create a unique visual effect. London's Millennium Bridge, shown in Fig. 12.11(b), is an example of this type of bridge. Its construction cost was approximately $30,000 per sq. m, of which approximately one third was due to a retrofit required to correct a problem with dynamic behavior. When an aesthetic impact requires expenditures that exceed the cost of bridges that can perform the same function by factors in excess of 10, serious consideration must be given to the issue of how this expenditure best serves the public good.

12.4.4 Description of Sections

Sections 12.5 through 12.10 give an overview of design concepts for the following bridge types: girder bridges, slab bridges, frame bridges, arches, cable-stayed bridges, and other types of bridges. Each of these sections discusses longitudinal structural systems, cross-sections, methods of construction, and tendon layouts.

Fig. 12.11 (a) Salginatobel Bridge (above)
(b) Millennium Bridge (below)

12.5 DESIGN CONCEPTS FOR GIRDER BRIDGES

The term "girder bridge" is used to denote any type of bridge for which: (1) structural response in the longitudinal direction can be reasonably represented using a single beam with strength and stiffness in bending, shear, axial force, and torsion, and (2) foundation reactions due to permanent loads have only a negligible horizontal component.

Girder bridges are the most common type of post-tensioned bridges. They can be built cost-effectively for spans ranging from 25 m (82 ft) to over 300 m (1000 ft). Within this range, they can accommodate practically all types of bridge alignments, geometric constraints, and geotechnical conditions. They are adaptable to a variety of different methods of construction and can be built equally well using traditional labor-intensive methods or using the most modern mechanized methods.

This section reviews issues that arise in the development of design concepts for girder bridges. It begins with a discussion of the longitudinal structural system, focusing on span lengths, geometrical constraints, and the arrangement of bearings and joints. Suitable cross-sections for girder bridges are then presented, with particular emphasis on the single-cell box section. This is followed by an introduction to the most common methods used to construct post-tensioned girder bridges. The implications of method on the choice of structural system, cross-section, and tendon layout are discussed.

12.5.1 Longitudinal Structural System

The longitudinal structural system is primarily defined by the three-dimensional geometry of the girder, the arrangement of spans, and the arrangement of bearings, joints, and internal hinges. This section deals primarily with the longitudinal structural system of the completed bridge. Issues related to the longitudinal structural system of the incomplete structure during construction are discussed in greater detail in the subsections dealing with specific methods of construction.

The structural system and the method of construction are closely related. Choices made for one can limit the options for the other. In general, designers should first create a structural system that suits the long-term project requirements and then develop a method of construction to build this system quickly and cost-effectively, rather than decide first on a construction method and then develop a structural system that suits the method. Allowing the method of construction to drive the design of the structural system can lead to compromises on durability, aesthetics, and other characteristics of the design concept that will ultimately impair the long-term success of the project.

12.5.1.1 Arrangement of Spans

Alignment and profile are usually defined before the actual bridge design process begins, based on the functional requirements of the highway or railway to which the bridge belongs. Given this basic geometrical information, the task of bridge designers is therefore to develop a suitable arrangement of spans based on the following considerations:

1. Available locations for piers and abutments, as determined by clearance requirements below the bridge for traffic or navigation, as well as underground utilities. In some cases, it can be cost-effective to relocate underground utilities prior to construction to improve the arrangement of spans.
2. Geotechnical conditions affect the type and cost of foundations. For a multiple-span bridge of fixed length between abutments, increasing the span length tends to increase superstructure cost and decrease substructure cost. Total construction cost can be more or less sensitive to changes in typical span length depending on the specific conditions of a given project.
3. Curvature in plan tends to decrease the most cost effective span relative to a straight bridge over identical conditions.
4. Visual issues should also be considered in determining the arrangement of spans, especially when there are prominent visible features or other structures in the vicinity of the bridge.
5. Regularity of spans and details generally has a positive impact on construction cost. On long multiple-span viaducts, it is generally more economical to work with spans of identical length rather than to optimize the length of each span. Bridges with a regular arrangement of structural components and details also tend to have better seismic behavior than bridges with highly irregular characteristics.

Post-tensioned girder bridges have a strong and consistent record of cost-effective construction for spans between 30 and 250 m. Beyond this upper value, cable-stayed bridges have usually been more economical. There are, however, exceptions. The Raftsundet Bridge (Fig. 12.12), built in Norway in 1998, is a post-tensioned girder bridge with a main span of 298 m. At the lower end, formwork for girder bridges is generally difficult to install and strip for spans less than 25 m. It is usually preferable to design spans of less than 25 m (82 ft) as slab bridges.

12.5.1.2 Arrangement of Bearings and Joints

It is generally acknowledged that the durability of bridges is enhanced by keeping the number of expansion joints in the deck to a minimum. Doing so minimizes the paths that can be used by water and deicing chemicals to travel from the top of the deck slab to the ends of girders, bearings, and tops of piers, and thus reduces the likelihood of deterioration of components due to freeze-thaw action and corrosion of reinforcement. Post-tensioned girders are well adapted to systems with minimum numbers of expansion joints, because post-tensioning is an ideal means to connect components built at different times into monolithic units.

The first issue to be addressed in design is the load path linking superstructure to substructure for horizontal force in the longitudinal direction. Three possibilities are shown in Fig. 12.13:

1. **Fixed system** – A fixed bearing is provided at one of the abutments to restrain longitudinal displacement of the superstructure and to transfer horizontal force from superstructure to substructure. The location of this bearing is called the neutral point of the system, i.e., the point in the superstructure that does not displace under the action of creep, shrinkage, and temperature. Fig. 12.13 shows that shortening of the girder induces bending in the piers, which increases with increasing distance away from the neutral point. Fixed systems are best suited to relatively short bridges.
2. **Flexible system** – The abutments provide no longitudinal restraint to the superstructure. Horizontal force is translated from superstructure to the piers. Longitudinal displacement of the superstructure is restrained by the stiffness of the piers. The neutral point shifts to a location within the span. Under the action of creep, shrinkage, and temperature, the superstructure displaces inward toward this point.
3. **Isolated system** – Special bearings are provided to: (a) allow the superstructure to displace relative to the piers under the action of creep, shrinkage, and temperature, (b) transfer wind load and other non-seismic forces from superstructure to substructure, and (c) minimize seismic demand on piers by tuning the natural period of the structure and by providing damping. Isolated systems generally work best on bridges with relatively stiff (i.e. short) piers. Although isolation can be an effective way of solving the challenges related to long bridges, the design of bridges using such concepts is beyond the scope of this chapter. Readers are referred to the book by Priestley, Seible, and Calvi[12.17] for further information on this topic.

The remainder of this discussion pertains to flexible systems.

The primary challenge is to ensure adequate behavior as the girder shortens under the action of creep, shrinkage, and temperature, as well as under longitudinal seismic action. The focus in both cases is on the piers, which must bend in response to displacements imposed at their upper ends (in the case of shortening) and inertial forces caused by the accelerating mass of the superstructure (in the case of earthquake). In the former case, bending effects are reduced by increasing the flexibility of the connection between girder and pier; they can be eliminated altogether by providing expansion bearings which allow the girder to move without displacing the top of the pier. Bending effects produced by seismic action are reduced by spreading the inertial force over as many piers as possible, i.e., by ensuring that the maximum number of piers is connected by fixed bearings or monolithic connections to the girder.

For low to moderate seismic action, the design procedure generally begins with a calculation of the maximum number of piers that can be connected to the superstructure to produce adequate behavior under shortening of the girder. This arrangement of piers is then checked under longitudinal seismic action. If the response of the piers under earthquake is adequate, then the concept is valid. If seismic response is not adequate, then the distance between expansion joints must be reduced.

It is excessively conservative to insist that piers remain crack-free under the action of girder shortening. Instead, piers should be allowed to crack and crack widths checked against a suitable limiting value. The following procedure is suggested for calculations at a given pier (Fig. 12.14):

1. Calculate axial force due to permanent load, N, which is taken here to be constant.
2. Calculate displacement at the top of the pier, δ, produced by shortening of the girder.
3. Determine the tensile strain in the outer layer of reinforcement, $\varepsilon_{s,max}$, corresponding to the maximum acceptable crack width. For preliminary calculations, a value of 0.0012 can be used.
4. Calculate the moment at the base of the column, M_{base}, corresponding to N and $\varepsilon_{s,max}$.
4. Calculate the cracking moment, M_{cr}, and the distance z_{cr} from the base of the column to M_{cr}. Above z_{cr}, the concrete is uncracked; below z_{cr}, the concrete is cracked.
6. Draw the curvature diagram, ϕ, for the pier. Curvature in the uncracked region can be taken to be $M/(EI)$, where E is modulus of elasticity and I is moment of inertia. In the cracked region, curvature can be calculated from a plane-sections analysis.
7. Calculate the tip displacement, δ_1, corresponding to the curvature diagram using the method of virtual work:

$$\delta_1 = \int (M \cdot \phi)dz$$

8. If δ_1 is greater than δ, then the pier can undergo the required tip displacement with cracking within acceptable limits.

Fig. 12.12 Raftsundet Bridge, Norway

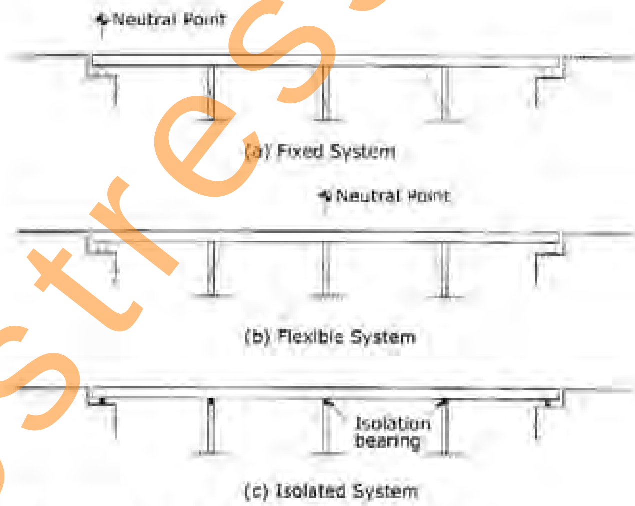


Fig. 12.13 Fixed, Flexible, and Isolated Systems

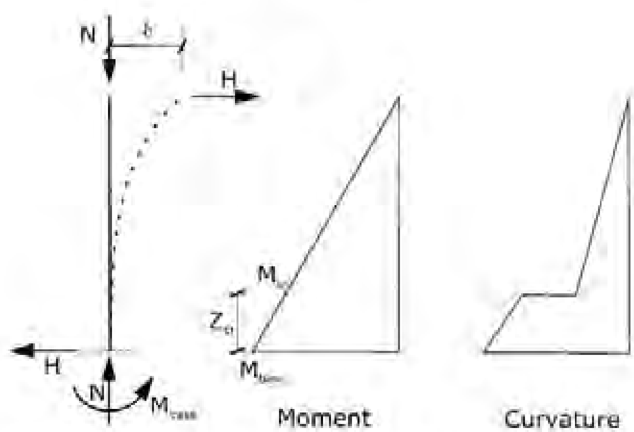


Fig. 12.14 Response of Piers to Shortening of Superstructure

It is acceptable to consider the effect of creep in pier concrete when calculating the response of the pier to the long-term component of superstructure displacement (i.e., due to creep and shrinkage). Additional discussion of concepts for flexible systems is given by Menn.[12.1]

12.5.1.3 Curved Bridges

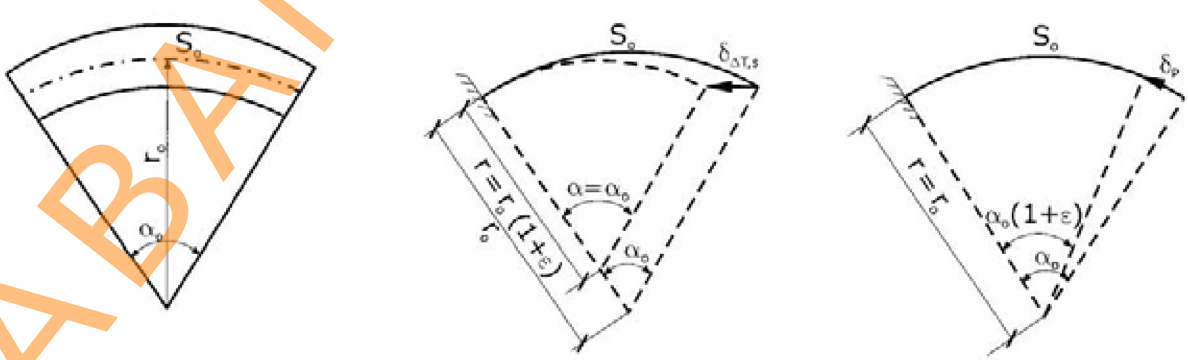
Post-tensioned girders are well adapted for use on curved alignments. The following issues should be considered in developing concepts for curved post-tensioned girder bridges:

1. In straight girders, vertical load can produce torsion only when it is applied eccentrically to the shear center. In curved girders, however, vertical load applied through the shear center can produce significant torsional moments. The longitudinal structural system must therefore provide adequate means of resisting these effects.
2. The effect of curvature on overall structural behavior can be estimated on the basis of the ratio of span length to radius of curvature, L/r, where L is arc length. The behavior of a curved girder approaches that of a straight girder with decreasing values of this ratio.
3. It is not necessary to provide torsional restraint at every support. In fact, for bridges that subtend a relatively large angle, sufficient strength and stiffness can be obtained using only torsionally free supports. An example of such a system is shown in Fig. 12.15.
4. In straight girders, shortening of the superstructure is induced by prestressing, creep, shrinkage, and temperature. Structural displacements due to prestressing and creep on the one hand, and shrinkage and temperature on the other, are fundamentally different. The axial component of prestressing force produces a purely axial strain. Since creep strain is produced by the sustained stress due to prestress, it will also be purely axial. Fig. 12.16(c) shows the displacements produced by these two actions in curved girders. The structure gets shorter, and retracts along its length. The radius of the girder does not change. Shrinkage and temperature, however, are volumetric deformations of the girder. Each dimension of the girder (length, width, and height) is reduced by the given strain. Consequently, the radius of curvature is also reduced by the same strain. The resulting deformed shape of the girder (assuming full fixity at the end) is shown in Fig. 12.16(b).

The volumetric nature of shrinkage and temperature deformations must be considered in the layout of bearings and expansion joints. To ensure that expansion joints function properly, it is advisable to constrain displacements at the abutments and other joint locations to be parallel to the original curved axis of the girder. If guided bearings are used at piers between expansion joints, the guides can be set tangent to the net direction of total displacement, taking into account the parallel setting of the guides at joint locations.

Other issues related to curved girders (choice of cross-section and post-tensioning details) will be discussed in relevant subsections.

Fig. 12.15 Stability of curved girders on torsionally free supports



(a) Basic Geometry

(b) Deformation Due to Shrinkage and Temperature

(c) Deformation Due to Prestressing and Creep

Fig. 12.16 Displacements in Curved Girders

12.5.2 Cross-Sections: General Considerations

The primary characteristics of the superstructure cross-section have a significant impact on all aspects of the performance and quality of bridges. The choice of the type of cross-section is thus a crucial decision in the development of design concepts. It should be based on an understanding of the requirements and constraints governing application to the given project as well as on knowledge of the advantages and disadvantages of a broad range of common cross-section types.

The main features of the single-cell box section, which is the most common type of cross-section used for post-tensioned girder bridges, are described in Section 12.5.3. Section 12.5.4 discusses other less common but important types of cross-section. The current section discusses two cross-section properties that are of interest for all types of cross-sections, the span to depth ratio and the efficiency index.

12.5.2.1 Initial Dimensioning: Span to Depth Ratio

Width and depth are the two most important dimensions of cross-sections. These values are normally selected early in the design process. The overall width of the cross-section is controlled by the dimensions of the deck slab, which is in turn defined by the functional requirements of the highway or railway carried by the bridge. Bridge designers are normally provided with the number and width of traffic lanes, sidewalks, shoulders, and barriers for highway bridges, and the number and spacing of tracks for railway bridges.

Depth of the cross-section is usually determined on the basis of conventional span to depth ratios. These ratios are representative of a large body of previously built bridges that have performed well. Although it would be possible to use a cross-section depth thus derived as a starting value for a process by which the depth is actually optimized, it is more common to lock in on a depth of cross-section at the concept phase and not to change it for the remainder of the design process. The economic benefits of working with more generously proportioned cross-sections that facilitate forming, reinforcing, placement of concrete, and stripping of forms have until now tended to outweigh any savings arising from optimizing the depth of the section.

For constant depth girders, the span to depth ratio of the longitudinal structural system can vary between 12:1 and 35:1.[12.1] More realistically, though, the span to depth ratio tends to lie between 17:1 and 22:1. Below 17:1, girders tend to look heavy. Above 22:1, the consumption of longitudinal prestressing steel tends to increase sharply and becomes difficult to accommodate within the section.

For variable depth girders, Menn[12.1] proposes a ratio of span to minimum depth at mid-span of 50:1 and a ratio of span to maximum depth at the support of 17:1. This combination of ratios gives girders a visually pleasing profile in which the haunching is clearly visible. The upper bridge in Fig. 12.17 follows these "classical" proportions. When the span to depth ratio at mid-span is significantly lower than 50:1, the visual impact of haunching tends to become less significant. This situation is illustrated in the lower bridge in Fig. 12.17.

Span to depth ratios are always guidelines and should not be treated as rigid rules. The Raftsundet Bridge, discussed previously and shown in Fig. 12.12, has a ratio of main span to depth at mid-span of 90:1. It is reasonable that a higher ratio was used for such a long-span bridge, where there is a large premium to pay for additional dead load at mid-span and relatively little to gain in terms of structural performance for added bending stiffness at this location.

For multiple-span bridges with spans of variable length, the span used to define the governing span to depth ratio

Fig. 12.17 Visual Impact of Haunching

must be chosen carefully. In this regard, visual considerations, efficient use of materials, and ease of construction should be considered. In the case of viaducts with many spans of identical length and a single main long-span opening, it is generally preferable, both economically and aesthetically, to define the span-to depth ratio relative to the approach span length to haunch the main span and the two sides to provide the additional depth required for the larger spans.

Span to depth ratios can also be used to assist in dimensioning of individual structural components.

12.5.2.2 Efficiency Index (Bending)

Another useful parameter for designers is the efficiency index of the cross-section, defined as $\rho = I/(Az_{top}z_{bot})$, where I is moment of inertia, A is area, and z_{top} and z_{bot} are distances from the centroid of the section to the extreme top and bottom fibers of the section, respectively.

Values of ρ for T-sections are typically around 0.4. For single-cell box girders, values around 0.6 are typical.

This efficiency index is a purely geometrical parameter that essentially compares the capacity of the uncracked concrete section to resist bending stress to the amount of concrete used. It actually says little about the efficiency with which the structure carries load, which depends on demand, reinforcement, as well as the properties of the cross-section. Nevertheless, the parameter can be a useful measure for comparing alternative cross-sections in the concept phase.

12.5.3 Cross-Sections: Single-Cell Box

The single-cell box girder is the most efficient and versatile type of cross-section currently used for post-tensioned girder bridges. As shown in Fig. 12.18, single-cell box sections consist of a top slab, two webs, and a bottom slab. The dimensions of these components and their arrangement into a cohesive unit are based on considerations that include the longitudinal structural function of the entire section, the transverse structural function of individual components of the section, detailing, and construction. These issues will be discussed further in this section.

Fig. 12.18 Single-Cell Box Girder

12.5.3.1 Advantages

Single-cell box sections offer the following advantages:

1. **Structural performance** – Single-cell boxes are strong and stiff in bending, shear, and torsion. Wide top and bottom slabs provide significant capacity for both positive and negative moments. This generally makes boxes a more suitable choice than T-sections for continuous girders, since the negative bending capacity of T-sections is limited by the lack of a proper bottom flange. Because boxes enable a closed shear flow to be established, they provide behavior in torsion that is far better than that of T-sections, which must resist torsion primarily by warping.
2. **Ease of construction** – Single-cell sections are generally easier to build than multiple-cell sections, since there are only two webs to reinforce, form, and post-tension.
3. **Ease of analysis** – The longitudinal structural response of single-cell box girders can normally be calculated on the basis of a model consisting of a single beam, with moment of inertia, area, and torsional constant equal to the respective properties of the box girder. Sectional forces (M, V, and T) obtained from the beam model can then be used to calculate forces in individual cross-section components, using the simple statical relationships described in Section 12.5.3.2. For box girders and T-girders with more than one web, a much more elaborate analytical procedure is required to calculate forces in individual cross-section components.
4. **Durability** – Single-cell boxes have a smaller exposed surface area than multiple-T sections, including those produced with precast concrete I-girders. Since single-cell boxes have only two webs, the transverse spans of the deck slab are relatively long and are generally post-tensioned transversely. This further enhances the intrinsic durability of single-cell boxes.
5. **Compact substructure components** – Because the two webs are located centrally rather than near the edges of the deck slab, single-cell box girders can be supported on relatively narrow single-shaft piers, eliminating the need for "tomahawk" type piers or multiple-column bents. The use of compact pier shafts often makes possible the use of more compact foundations.

These qualities give single-cell box girders remarkable adaptability, which enables them to be used in practically any application over the range of possible highway geometries and spans.

12.5.3.2 Structural Behavior

The components of single-cell box sections (top slab, webs, and bottom slab) have more than one structural function. Each component contributes to the strength and stiffness of the overall cross-section, which is the primary load carrying element in the longitudinal direction. This is referred to as the longitudinal structural function of a given cross-section component. In addition, the top slab, webs, and bottom slab have a transverse structural function, which requires them to carry wheel loads from their point of application in the top slab to the webs, where these loads can then be resisted by the entire cross-section.

12.5.3.2.1 Longitudinal Structural Response

As stated previously, the longitudinal response of single-cell box girders is modeled using a single beam. A given load is resolved into a pure symmetric component and a pure antisymmetric component, as shown in Fig. 12.19. The symmetric component is equated to a single load Q and is applied to the beam model. Sectional forces (M and V for straight beams) can be calculated using the familiar methods of structural analysis. The antisymmetric component is equated to a single torsional load mt and is applied to the beam model. For straight beams, this type of load produces only torsion, T.

Forces in individual cross-section components can be calculated from M, V, and T using simple statical relations. Assuming the girder is uncracked, stresses due to M are obtained from the familiar relation $\sigma = (M_z)/I$, where z is the distance from the neutral axis to the location under consideration and I is the moment of inertia.

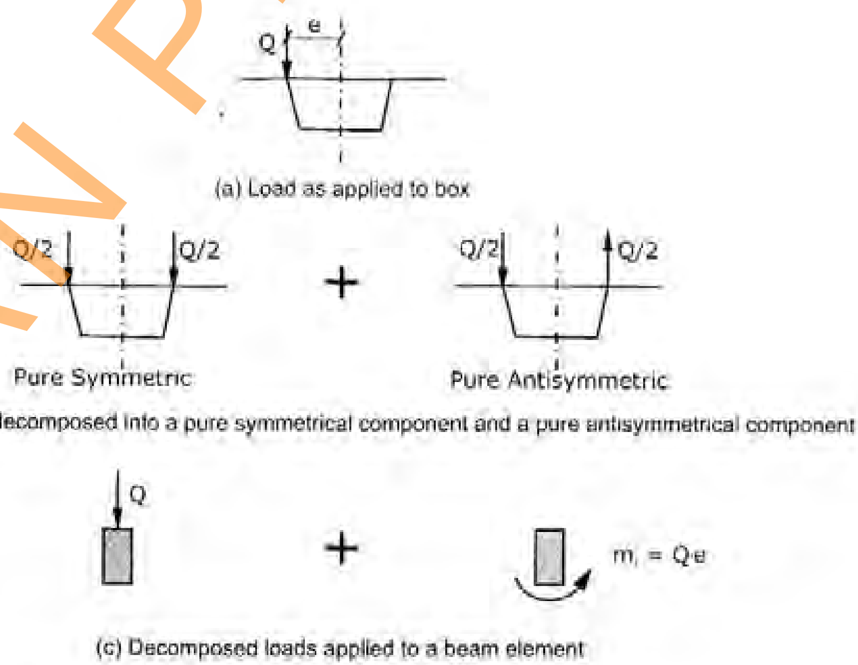
The shear force produced in a given web and at a given location by the symmetrical component of load is called V_{symm}. Because the load and the section are symmetric, V_{symm} is simply equal to half the total shear force due to Q applied symmetrically, i.e., $V_{symm} = V_Q/2$.

The shear force produced in a given web by the anti-symmetrical component of load is called V_{anti}. It is produced by the torsional moment T at the location under consideration. Torsional moment T creates a closed shear flow v around the box defined by the webs, bottom slab, and the portion of the deck slab between the webs. The shear flow is given by the following equation:

$$v = \frac{T}{2A_0}, \text{ where}$$

A_0 is the area enclosed by the middle surface of the box formed by the bottom slab, webs, and central portion of the deck slab. Shear force in the web, V_{anti}, is obtained by multiplying v by the depth of the web, h:

$$V_{anti} = v \times h = \frac{T}{2A_0} \times h$$



(a) Load as applied to box

(b) Load on box decomposed into a pure symmetrical component and a pure antisymmetrical component

(c) Decomposed loads applied to a beam element

Fig. 12.19 Distribution of Live Load in Single-Cell Boxes

(Shear in the central portion of the top slab and in the bottom slab due to the anti-symmetric component of load can be obtained in a similar manner.)

The total shear force due to the combined action of the symmetric and anti-symmetric components is thus given by the following equation:

$$V_{web} = V_{symm} \pm V_{anti}$$

12.5.3.2.2 Transverse Structural Response

Loads also produce a structural response in the transverse direction which must be considered. Transverse bending due to dead load is relatively straightforward to calculate. If dead load is constant along the length of the girder, then transverse bending due to dead load will be constant along the girder away from discontinuities such as diaphragms. Away from discontinuities, therefore, transverse moments due to dead load can be calculated for a unit slice cut from the girder, considering the top slab, webs, and bottom slab as one-way slabs.

Transverse bending due to concentrated loads (produced, for example, by the wheels of the applicable live load model) requires a more detailed calculation. Concentrated loads on slabs produce curvature in both the transverse and longitudinal directions, and thus create both transverse and longitudinal bending moments in the slab. Consequently, the transverse bending moments due to concentrated load will not be constant along the length of the girder but will likewise vary longitudinally. It is therefore no longer possible to calculate transverse moments directly on a model consisting of a slice cut transversely from the girder.

Although three-dimensional computer models can be used to calculate transverse bending moments, the conventional approach is to use a simpler procedure based on the use of influence surfaces.[12.1] [Influence surfaces are the two-dimensional analog of influence lines. They are used to calculate a specific force quantity S (e.g. transverse bending moment) at one specific location (x_0, y_0) on an elastic plate of given geometry and support conditions, for a unit load applied perpendicular to the plate at any given location (x, y).] Pucher[12.3] and Homberg[12.4] have published influence surfaces for a variety of load and support cases, including most of the cases of interest in bridge design. Alternatively, designers can generate their own influence surfaces using a finite-element program.

Three cases can be considered for the calculation of transverse bending moments using influence surfaces: (a) deck slab cantilevers, (b) the interior portion of the deck slab between webs, and (c) the webs and bottom slab.

a. **Deck slab cantilevers**: Calculation of transverse bending moments in the cantilever portions of the top slab is straightforward. Influence surfaces are used to compute maximum negative moment at the fixed end (labeled A in Fig. 12.20), as well as maximum positive and negative moments at the midpoint of the cantilever (labeled B and C in Fig. 12.20). Menn[12.1] has proposed the simple envelope of transverse bending moments shown in Fig. 12.20. Straight-line interpolation has been used in this diagram.

b. **Interior portion of the deck slab**: The calculation of transverse bending moments in the deck slab between the webs is based on two sets of influence surfaces from available published sources. The first is for infinitely long plates spanning between two parallel, fully fixed edges; the second is for infinitely long plates with two simply supported (hinged) edges. From the influence surface for the fully fixed case, one can calculate maximum negative moment in the slab at the web and maximum positive moment at mid-span. These moments are labeled D and E, respectively, in Fig. 12.21, which is an approximate envelope of maximum bending moments in the inte-

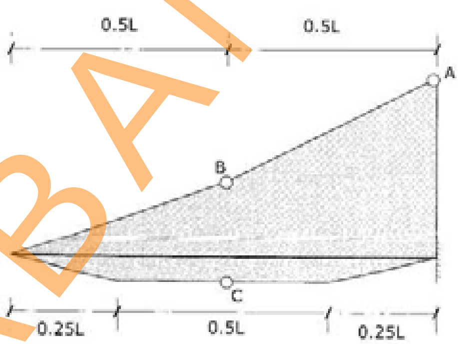


Fig. 12.20 Envelope of Transverse Bending Moments in a Deck Slab Cantilever – Ordinates A, B, and C are Obtained from Influence Surfaces (adapted from Ref. 12.1)

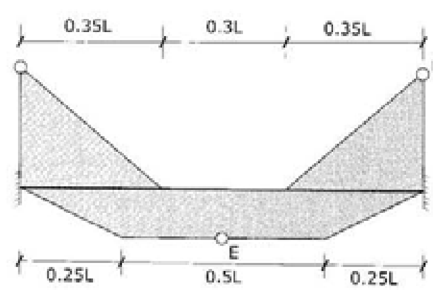


Fig. 12.21 Envelope of Transverse Bending Moments a Slab Spanning Between Two Fixed Edges – Ordinates D and E are obtained from influence surfaces (adapted from Ref. 12.1)

rior portion of the deck slab due to live load applied between the webs.

The assumption of a rigid connection at the ends of the interior deck span, however, is not correct. Due to the flexibility of the webs, there will be some rotation of the joint between the deck slab and the webs when live load is applied to the deck. Moment D in Fig. 12.21 is thus an upper bound for negative moment in the deck slab at the webs due to loads applied between the webs. Moment E in the same Figure, however, is not an upper bound, since flexibility of the webs will increase the maximum positive moment in the span. The upper bound for positive moment at this location can be calculated using the second influence surface, corresponding to hinged supports along the parallel edges of the plate. The envelope is shown in Fig. 12.22. The maximum positive moment at mid-span is labeled E′.

For conventionally proportioned cross-sections, dimensioning of reinforcement for negative moment in the slab at the webs is generally governed by moment A (Fig. 12.22), due to load applied to the cantilever portion of the deck slab. Reinforcement for positive moment at mid-span will be governed by a moment that lies between E and E′ (Fig. 12.21 and Fig. 12.22). It is conservative to estimate reinforcement on the basis of E′, the upper bound value. A more refined estimate can be obtained by distributing the fixed-end moment corresponding to E between top slab and web. The procedure is as follows:

1. Assume that the load producing E is the same as the load producing E′.
2. Draw the approximate moment diagram for the hinged-hinged case [Fig. 12.23(a)]. This diagram can be taken from Fig. 12.22.
3. Draw the approximate moment diagram for the fixed-fixed case [Fig. 12.23(b)]. Since this diagram is produced by the same arrangement of load as the moment diagram for the hinged case, it can be visualized as an upward shift of Diagram (a). The fixed-end moment, F, is not equal to D in Fig. 12.21 but is rather equal to (E′ - E).
4. Distribute the fixed-end moment F to the frame consisting of the interior portion of the top slab, the webs, and the bottom slab. The resulting moment at the joint between the top slab and the webs is called F′. The moment diagram in the top slab shifts downward from Diagram (b) to yield Diagram (c) in Fig. 12.23. The refined estimate of moment at mid-span, E″, is equal to E′-F′.

c. **Webs and bottom slab:** Moments in the webs and bottom slab produced by loads applied to the top slab can be calculated by distributing fixed-end moments at the intersection between top slab and webs. For live load, the two primary cases to be considered are load applied to the cantilevers (producing fixed-end moment A in Fig. 12.20, and load applied to the interior portion of the deck slab (producing fixed-end moment D in Fig. 12.21. Moment distribution is performed for a frame corresponding to the transverse slice taken from the girder. The results obtained will be conservative for live load, since the calculation ignores any longitudinal distribution of live load moments in the longitudinal direction in the webs and bottom slab.

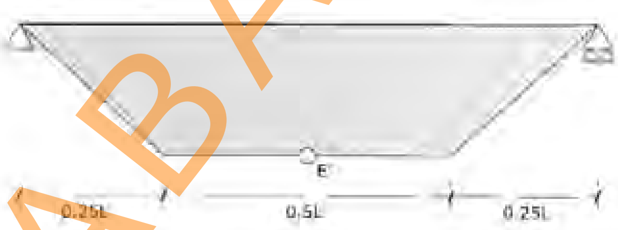


Fig. 12.22 Envelope of Transverse Bending Moments in a Slab Spanning Between Two Hinged Edges – Ordinate E′ is obtained from an influence surface (adapted from Ref. 12.1)

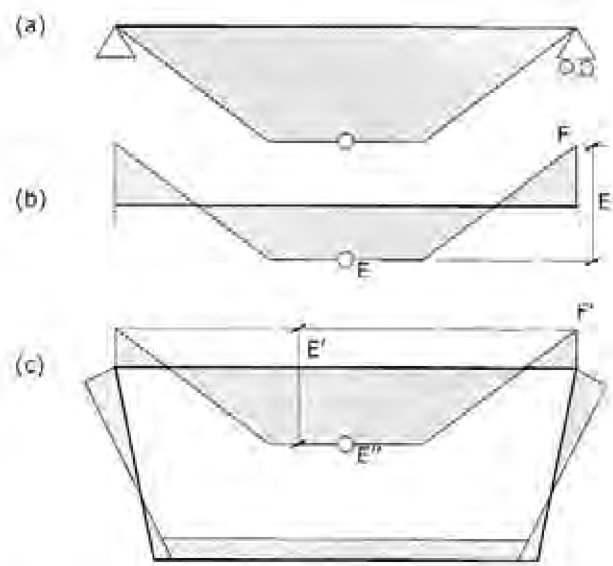


Fig. 12.23 Refined Estimate of Mid-Span Moment E′

12.5.3.2.3 Local Longitudinal Response

The longitudinal bending moments in the deck slab produced by wheel loads must also be considered, since the previously described procedure for calculating transverse moments is based on a state of equilibrium that relies on both transverse and longitudinal bending. Published influence surfaces are also available for the most commonly occurring cases in bridge design.[12.3,12.4] In general, the largest local longitudinal bending moments in the deck slab occur near diaphragms and other features that strengthen the longitudinal load path relative to the transverse path.

12.5.3.3 Dimensions

This section gives guidance for the selection of the primary dimensions of single-cell box sections. The symbols used refer to Fig. 12.24.

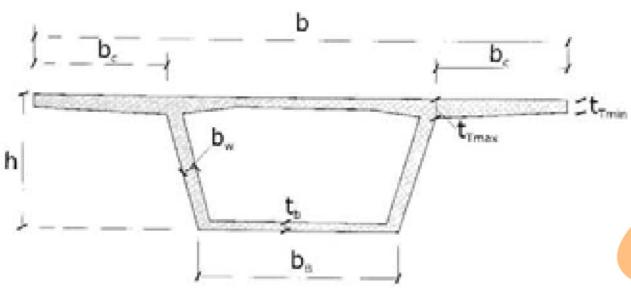
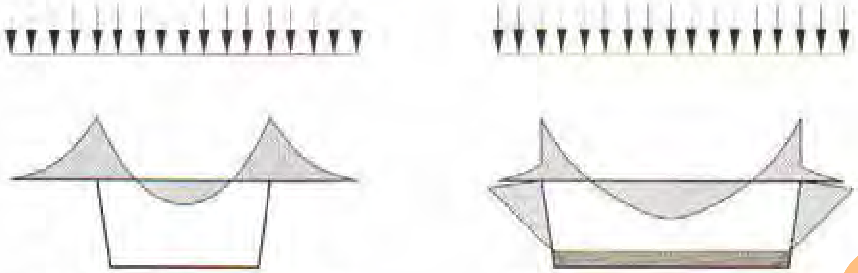


Fig. 12.24 Basic Dimensions for a Single-Cell Box Section

1. **Overall depth of cross-section,** h: The span to depth ratios described in Section 12.5.2.1 are valid for single-cell boxes. In addition, there is a practical lower limit of about 1.8 m (6 ft) to overall depth of the section. The interior cavity of the box must have sufficient height to allow workers to strip formwork and perform other duties while standing reasonably erect. (Due to concerns with durability, the use of lost forms is not recommended for the interior of box girders.) Assuming a span to depth ratio of 17:1, the minimum depth of 1.8 m (6 ft) corresponds to a minimum practical span of about 30 m (100 ft) for single-cell box girders.
2. **Width of top slab,** b: As stated in Section 12.5.2.1, the width of the top slab is determined by the functional requirements of the highway or railway carried by the bridge. Box girders with top slab width less than about 15 m (50 ft) can generally be designed and built without undue difficulty. Above this value, special attention needs to be paid to the thickness of the deck slab. The effect of roadway width on concepts for single-cell boxes is discussed further in Section 12.5.3.5.
3. **Intersection of top slab and webs**: The location of this point can be expressed quantitatively by the two dimensions b (width of top slab) and b_c (length of deck slab cantilever). It is usually possible to locate the intersection of the top slab and webs to minimize transverse bending in the webs due to dead load, i.e., roughly at the quarter points of the deck slab. The effect of the location of the slab/web intersection on transverse bending due to dead load is illustrated in Fig. 12.25. For highway bridges, it is generally advisable to locate the intersection of top slab and webs in this way. For railway bridges, it is generally preferable to center the webs with respect to the tracks.
4. **Width of bottom slab,** b_B : A number of factors, no single one of which governs in all cases, needs to be considered in determining the width of the bottom slab. For superstructures supported on bearings, b_B needs to be wide enough to ensure stability of the superstructure against tipping under torsional action. Particularly for long-span bridges, the bottom slab must be sufficiently wide to provide sufficient negative moment resistance. For externally post-tensioned box girders, the interior of the box must be wide enough to accommodate the tendons in a single row in the central portion of the span, preferably leaving a gap to allow workers and inspectors to walk inside the box girder without stepping on tendons. Finally, the width of the bottom slab should be chosen with due regard to its impact on the overall visible characteristics of the bridge, especially when viewed from below.
5. **Thickness of the top slab,** t_{Tmin} **and** t_{Tmax}: Minimum thickness of deck slab is usually governed by the dimensional requirements of the anchorages for transverse post-tensioning tendons and minimum clear cover. A value of 225 mm (9 in.) is commonly used. Maximum thickness, t_{Tmax}, is generally governed by requirements for strength and stiffness of the deck slab cantilever. The span to depth ratio of the cantilever, b_c / t_{Tmax} generally does not exceed 10:1. (Higher span to depth ratios are, however, possible. On the Felsenau Bridge in Switzerland (Fig. 12.26), for example, b_c / t_{Tmax} is approximately 14:1. Such a slender deck slab cantilever was made possible through the use of partial prestressing, by which cracking under live load at serviceability limit states is permitted provided the deck slab remains in compression under permanent load.) Dimension t_{Tmax} is also affected by dimensional requirements for the anchorages of longitudinal tendons in cantilever constructed bridges. This issue will be discussed further in Section 12.5.6.



(a) Balanced transverse span
(no bending in webs)

(b)Unbalanced transverse span
(bending in webs)

Fig. 12.25 Transverse Bending Due to Dead Load of Deck Slab

6. **Thickness of web,** b_w: For box girders with external, unbonded tendons, b_w can be as low as 250 mm (10 in.). When internal bonded longitudinal tendons are draped within the webs, b_w must provide adequate clear cover to the ducts as well as sufficient spacing between ducts to allow for proper placement and vibration of concrete. Web thickness is generally kept constant along the span, to facilitate formwork and detailing.

7. **Thickness of bottom slab,** t_B: The lower limit of this dimension is generally taken to be 200 mm (8 in.). The bottom slab must be made thicker if it contains longitudinal tendons or if required for negative moment resistance. Dimension t_B can normally be varied along the span without undue difficulty.

12.5.3.4 Detailing Issues

In general, the details of single-cell box sections should reflect a high degree of uniformity and repetition. Given the high cost of labor in industrialized countries, it is usually more cost-effective to keep details constant rather than to minimize materials by fine-tuning dimensions and quantities of reinforcement, which usually entails an increase in labor. This is particularly important for the dimensions of the concrete section described in Section 12.5.3.3.

Fig. 12.26 Felsenau Bridge

The top slab of single cell boxes is usually post-tensioned transversely. Given the relatively long transverse spans used for single-cell sections, transverse post-tensioning allows the thickness of the slab to be minimized. In addition, transverse post-tensioning is usually dimensioned to ensure that transverse tensile stresses at the top of the deck slab are less than the cracking tensile stress, which significantly enhances durability and increases stiffness relative to a conventionally reinforced slab.

Post-tensioning tendons for deck slabs usually consist of four-strand units in flat ducts. They are arranged at constant spacing along the length of the girder or, in the case of segmental construction, at constant spacing within a given segment. Forces induced by these tendons can be calculated using the approach described in Section 12.5.3.2. Equivalent loads due to transverse post-tensioning can be calculated from the deviation and anchor forces of the tendons. These are then applied to a slice extracted from the superstructure. Fixed-end moments at the joint between top slab and webs can be distributed as discussed in Section 12.5.3.2.

Reinforcement in the webs must be provided for longitudinal shear and torsion, as well as for transverse bending. It is conservative to calculate the area of steel required for longitudinal demand separately from the area of steel required for transverse demand and to provide the total of these two areas. Menn[12.1] describes a model for calculating a combined resistance of webs in shear and transverse bending, in which capacity is based on an interaction diagram. This generally leads to reduced web reinforcement compared to the sum of the two areas of reinforcement calculated for both types of demand separately.

For long-span girders, the width of the web can be minimized by post-tensioning the webs vertically.

The method of construction has an important impact on structural details in box girders. Sections 12.5.5 through 12.5.9, which present the most common methods of construction used for post-tensioned girder bridges, discuss specific detailing issues arising from the methods of construction at greater length.

12.5.3.5 Issues Related To Width of Roadway

As stated in Section 12.5.3.3, the thickness of the deck slab at its junction with the webs is determined primarily by the length of the transverse spans. As the width of the bridge increases, so too must the maximum thickness of the deck slab. The primary question for designers is how this additional thickness affects longitudinal behavior, i.e., does it contribute needed capacity in positive bending or does it only contribute additional dead load? For some long-span bridges, the additional thickness can indeed be fully utilized at ultimate limit state. For short-span bridges, however, it is likely that only a small fraction of the bending capacity provided by the additional thickness of deck slab will be required.

The challenge of wide bridges is therefore to limit the thickness of the deck slab to ensure that it can be used efficiently as a compression flange for positive longitudinal moment. Several options are available for designers faced with this situation.

It is often possible to split bridges that carry traffic in two directions into two separate parallel bridges, each carrying traffic in one direction. In such cases, it is recommended to keep the two superstructures completely separate unless transverse post-tensioning can be provided to link the two deck slabs together, essentially limiting transverse tensile stress on the top surface of the deck at all locations. In most cases, however, this is not possible. The most common way of joining two parallel box girders is therefore to use a cast-in-place concrete longitudinal closure pour that is conventionally reinforced. This strip will crack under service conditions and thus presents a vulnerability with regard to durability. This vulnerability can be eliminated by keeping the two structures separate. To ensure that the gap between the two structures is accessible for inspection and maintenance, it should be at least 300 mm (12 in.) wide over the entire length of the bridge.

Another option is to build a single cross-section but to reduce the width of the deck slab through the use of transverse ribs. Such a solution was used on the Tsable River Bridge in British Columbia, Canada (Fig. 12.27). The ribs are dimensioned to provide adequate resistance in transverse bending without significantly increasing dead load. The disadvantage of ribs is that the formwork for the deck slab is more complicated than formwork for sections without ribs. When ribs are not used, formwork need only be lowered a minimal amount to allow it to be moved forward

Fig. 12.27 Tsable River Bridge
Courtesy of ND LEA Consultants Ltd.

to be used again. When ribs are used, the formwork for the deck slab must be lowered to clear the ribs before it can be advanced. The method for stripping the interior formwork needs to be considered in cases where the girder is shallow.

Other types of section, such as multiple-cell box girders and single-cell boxes with struts, can also be used for wide roadways. These are discussed in Section 12.5.4.

The width of roadway can change along the length of a bridge. When the difference between the maximum and minimum width is less than the total length of the two deck slab cantilevers, the width of deck slab can be varied by adjusting the length of the cantilevers, keeping the dimensions of the box proper constant. For the cross-section shown in Fig. 12.28, the length of cantilever bc can be varied from its maximum value of bc,max to almost zero. It is far easier to move the bulkheads forming the edge of the deck slab than to change the formwork for the interior of the box.

12.5.3.6 Diaphragms

At specific locations along the length of single-cell box girder bridges, the interior cavity is partially or completely filled, creating a solid transverse wall called a diaphragm. The primary structural function of diaphragms is to provide a means of getting load into and out of the box cross-section. They are always provided at the longitudinal supports of the superstructure (piers and abutments).

The primary tool used to visualize the flow of forces and to calculate the required reinforcement in diaphragms is the truss model, which is also referred to as the strut-and-tie model. Truss models represent the flow of forces in concrete components using trusses composed of steel members carrying only tension and concrete members carrying only compression. The article by Schlaich, Schäfer, and Jennewein[12.5] provides a practical introduction to the use of truss models in design.

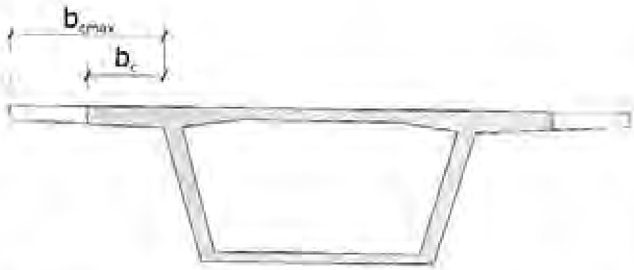


Fig. 12.28 Varying Width of Roadway by Adjusting Length of Deck Slab Cantilevers

The use of truss models in the design of diaphragms is demonstrated for the transfer of vertical reaction from the webs to a pier. For visual and economic reasons, the piers of single-cell box girder bridges are generally narrower than the width of the bottom slab. Vertical reactions cannot, therefore, be transferred directly from webs to pier. Fig. 12.29 shows one possible load path that transfers the force as required. Steel must be provided to resist tensile forces T_1 and T_2. Depending on the size of these forces, and requirements for crack control at serviceability limit states, reinforcing steel or post-tensioning steel (usually post-tensioning bars) can be used. The anchorage of the steel provided for T_1 at the bottom of the webs is of critical importance. If reinforcing steel is used, the use of headed bars should be seriously considered.

Similar truss models should be developed for the transfer of other forces into the pier, including torsional moments due to eccentric live load and horizontal force due to seismic action. For precast segmental bridges built using the span-by-span method, another important case is the transfer of force from the anchorage of longitudinal tendons into the box cross-section, which will be discussed further in Section 12.5.8.

Diaphragms should normally be provided with an access opening to allow workmen and inspectors to pass from one span to another inside the box. Truss models developed for the load cases described above must therefore leave an area free of compression and tension members through which the access opening can be provided. In the model shown in Fig. 12.29, for example, such an opening could be provided along the centerline of the bridge, between members T_2 and C_2.

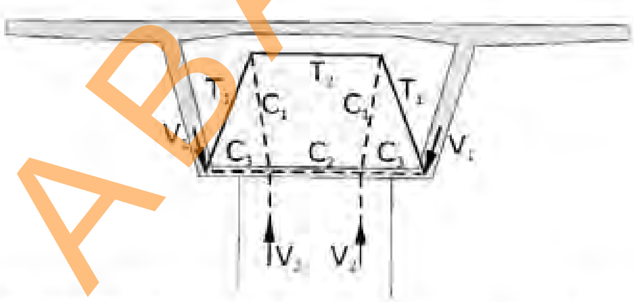


Fig. 12.29 Transfer of Vertical Reactions from Box Girder to Pier

It is usually not necessary to provide diaphragms within a given span, except in cases where internal hinges have been provided or where external unbonded tendons need to be deviated. The latter case is discussed in greater detail in Section 12.5.8. Interior diaphragms complicate the installation and removal of formwork, add weight, and usually perform no useful structural function except in the case of very tightly curved bridges.

12.5.4 Cross-Sections: Other Types

Notwithstanding the adaptability and efficiency of single-cell box girders, other types of cross-section have been used effectively for girder bridges, including variations on the single-cell box, T-sections, and U-sections. These types of cross-section are discussed briefly in this section.

12.5.4.1 Strutted Single-Box Sections

Struts can be used to increase the width of single-cell box girders without significantly increasing the thickness of the deck slab. The cross-section used for the Bang Na Expressway in Thailand, shown in Fig. 12.30, is one example of how this can be done. Post-tensioned concrete struts support the central portion of the deck slab and transfer load down to the intersection of the inclined webs and the bottom slab. One pair of struts was used per precast concrete segment. The total width of the cross-section is 27.2 m (89.2 ft).

Struts can also be located outside the webs. In this case, they transfer load from the deck slab cantilevers to the intersection of the webs and bottom slab. The Kochertal Bridge in Germany (Fig. 12.31) is an example of a bridge using this type of section.

To reduce weight, the webs of box-girder bridges can be transformed from solid walls into trusses. This type of cross-section was used as early as 1959 for the Mangfall Bridge in Germany (Fig. 12.32). The labor required to build such a concept, however, is considerable. More recently, precast concrete and steel tubes have been used to reduce the labor required to construct this type of bridge. The Boulonnais Viaduct, also shown in Fig. 12.32, is an example of this type of structure. Open-web sections can be built cost-effectively using the precast segmental method of construction.

12.5.4.2 Multiple-Cell Box Sections

Multiple-cell box sections have been used since the 1950s for post-tensioned girder bridges. At that time, increasing the number of webs was a means of reducing the transverse spans of the deck slab and thus reducing its thickness.

Fig. 12.30 Bang Na Expressway (Thailand) Cross-Section
Courtesy of International Bridge Technologies, Inc.

Fig. 12.31 Kochertal Bridge, Germany

Multiple-cell box girders require more labor to build than comparable single-cell boxes. Each web must be reinforced, formed, post-tensioned, and stripped. The total width of the webs provided tends to be greater than the minimum required for strength, since the width of an individual web is determined primarily by the size of the tendon ducts it contains. In addition, the analysis of multiple-cell boxes is more complicated than the analysis of single-cell boxes, since the distribution of live load to the individual webs cannot be determined by simple statical relationships alone.

In recent years, the use of multiple-cell box girders has gradually decreased in favor of single-cell boxes. Their use is restricted primarily to bridges that are cast in place on falsework, and for very wide bridges built using the segmental method of construction.

12.5.4.3 T-Sections

As the name implies, multiple-T sections consist of a top slab and two or more webs. A typical double-T section is shown in Fig. 12.33. T-sections can be economical relative to box girders because they do not use a bottom flange. This eliminates the need to install and strip interior formwork, which reduces labor relative to box sections.

The cost-effective range of spans for post-tensioned T-girders lies between 25 and 35 m. Below 25 m, the labor required to form the webs outweighs the savings in materials relative to solid slab bridges. Above 35 m, the limited capacity of T-sections to resist negative moment, due to the absence of a bottom slab, becomes significant relative to the more efficient box sections. Although this range of spans appears small, it accounts for a large percentage of total bridge inventory. T-sections can therefore be a serious option in many cases.

In the United States and Canada, superstructures consisting of precast, pre-tensioned concrete I-girders and a cast-in-place concrete deck slab are strongly competitive in the 25 (82) to 35 m (115 ft) span range. For this reason, post-tensioned T-girders have not seen widespread use in recent years. In evaluating the overall benefits of these two systems, it is important to consider the intrinsic advantages of post-tensioned T-girders, which include enhanced durability of the deck slab that is post-tensioned in two directions as well as the greater range of aesthetic possibilities offered by the T-section.

Although post-tensioned T-girders have traditionally been cast in place on falsework and prestressed using internal bonded tendons, the system is adaptable to precast segmental construction with external unbonded tendons draped between the webs.

T-sections are not suitable for spans with significant curvature due to their lack of strength and stiffness in torsion.

12.5.4.4 Precast-I Sections

Post-tensioning has been used to extend the range of precast, pre-tensioned I-girders. Precast girders can be post-tensioned together on site to create spans greater than the maximum length of girder that can be trucked to site and lifted by crane. The cross-section of the completed bridge is essentially identical to that of a conventional bridge with precast I-girders and cast-in-place deck slab. Spans ranging from 45 m (150 ft) to 100 m (325 ft) have been built using spliced I-girders.

Fig. 12.34 illustrates the erection of a spliced I-girder bridge. Temporary brackets are attached to the ends of the central girder that will allow it to be suspended on the two cantilever girders extending out from the piers. This eliminates the need for temporary falsework in this span. Joints between girders are generally made of cast-in-place con-

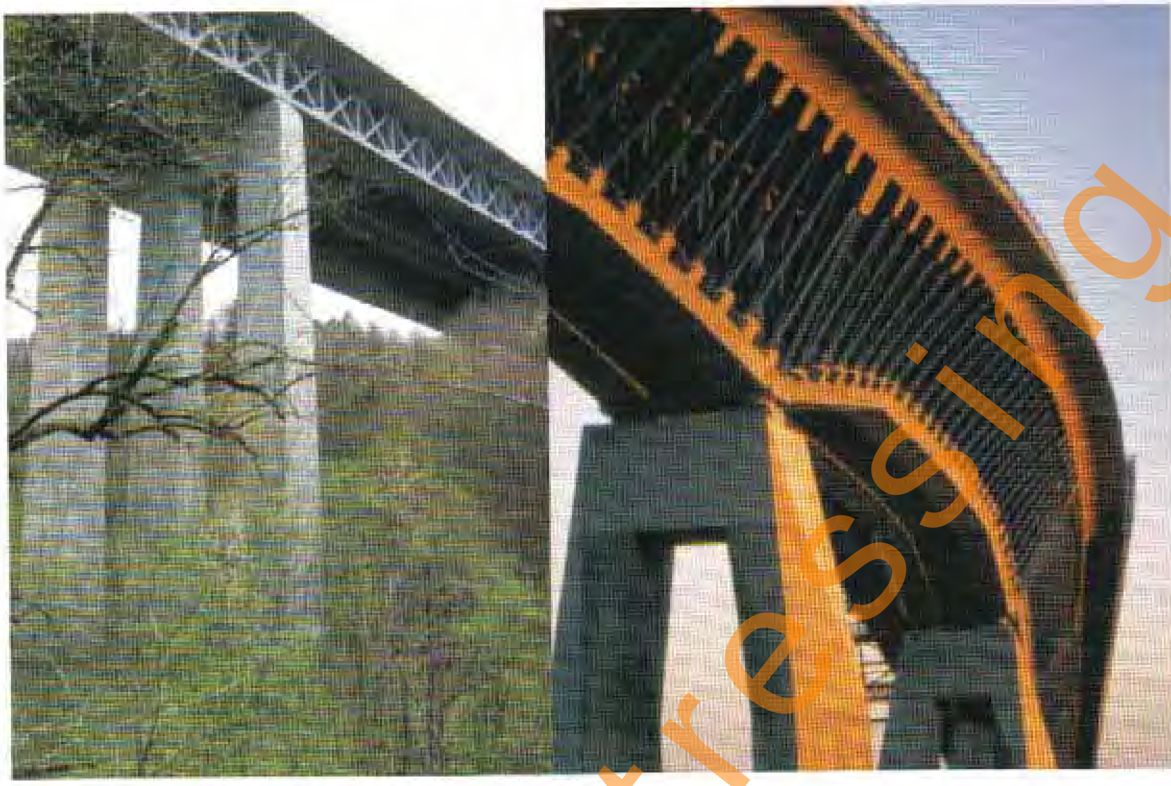


Fig. 12.32 Open-Web Boxes: Mangfall Bridge (left) and Boulonnais Viaduct (right)

crete, and range in length from 150 to 600 mm (6 -24 in.). Post-tensioning ducts must be spliced at these joints. Once the concrete in the joints has reached sufficient strength, the girders are post-tensioned together. The deck slab can then be constructed. It is also possible to apply a second increment of post-tensioning to the system after the deck slab is complete, to provide longitudinal pre-compression in all components of the cross-section. A comprehensive discussion of the design and construction of spliced girder bridges has been prepared by the PCI.[12.6]

The primary advantage of spliced I-girder bridges is economy. This type of bridge benefits from a well established precast concrete industry in the US and Canada that can furnish precast I-girders at competitive prices, as well as a good supply of bridge contractors who are experienced in the construction of conventional precast I-girder bridges. The main disadvantages of this system, however, relate to speed of construction (related to curing time of cast-in-place joints between girders and of the deck slab itself), durability (deck slabs are generally not prestressed in both directions), and limited aesthetic possibilities.

12.5.4.5 U-Sections

Post-tensioned bridges with U-shaped cross-section have provided a viable option for crossings where the available clearance under the bridge is insufficient for more conventional solutions such as box girders or I-girders. They do so

Fig. 12.33 Double-T Section

Fig. 12.34 Spliced I-Girder Bridge

Fig. 12.35 U-Shaped Cross-Section
Courtesy of Michael Goodman

by using the concrete barriers on either side of the deck as the primary longitudinal structural elements. (In conventional bridges, these barriers would not participate in the longitudinal structural system, but would contribute only dead load.) The resulting superstructure has an effective depth below the top of roadway equal to the deck slab plus any wearing surface.

Because the barriers have a crucial longitudinal function, they can no longer be replaced in service. Designers of this type of bridge must therefore pay greater attention to the durability of these components. These U-shaped sections are suitable only for relatively narrow bridges (usually two lanes plus shoulders), since the required depth of deck slab must increase with spacing between the barriers to maintain an acceptable transverse span to depth ratio.

Additional information on post-tensioned bridges with U-shaped cross-sections is given by Hitec.[12]

12.5.5 Methods of Construction: Cast-In-Place on Falsework

12.5.5.1 Description of Method

The cast-in-place on falsework method of construction has figured prominently in the history of post-tensioned bridges since the earliest days and continues to be a viable option in many situations to this day. It consists of the following main steps:

1. Erect falsework supported on the ground and construct formwork
2. Place reinforcing steel and post-tensioning ducts
3. Place and cure concrete
4. Install and stress tendons
5. Strip formwork and strike falsework

With the exception of installing the post-tensioning ducts and stressing the tendons, the method differs little from the classical methods of reinforced concrete construction.

The primary advantages of cast in place construction on falsework derive from the method's intrinsic simplicity. They include:

- For the most part, the method requires only rudimentary construction skills that can be easily found in all parts of the world, and requires little heavy equipment and no specialized machinery. It can therefore be applied wherever falsework can be erected and concrete batched.
- The method is adaptable to a wide range of geometrical conditions, including tightly curved alignments and skewed supports. It is especially well suited to bridges that must accommodate major changes in geometry, such as an increase in the number of traffic lanes.
- The geometric adaptability of cast-in-place construction on falsework creates an extremely wide range of aesthetic possibilities.

The method does, however, have significant drawbacks that can limit its range of application. The most important of these are as follows:

- The method is highly labor intensive. In industrialized countries, therefore, it can be uncompetitive against other more mechanized methods of construction.
- Because all work is done on site, the method is relatively slow. In cases where speed of construction is important, it is unlikely to be the preferred method.
- It is poorly suited to situations where falsework cannot be erected, or erected only with difficulty. Such cases include long spans over water and inner-city construction.
- Finally, the method has all of the difficulties associated with casting concrete in place in extreme weather conditions.

12.5.5.2 Longitudinal Structural System

This method can be used to build bridges covering a wide range of longitudinal structural systems, and spans ranging from 25 m (82 ft) to in excess of 100 m (328 ft). Most multiple-span bridges built using this method are designed to be continuous over the piers. It is generally simple and economical to provide monolithic connections between superstructure and piers, which can be beneficial for bridges built in regions of high seismic risk.

Internal hinges can be incorporated with relative ease. These have been used to break long bridges into manageable units (see Section 12.5.1.2) without disrupting the arrangement of tendons and other details at the piers.

12.5.5.3 Cross-Sections

With the exception of the spliced precast I-girder section, all of the cross-sections described in Sections 12.5.3 and 12.5.4 can be used for cast-in-place bridges built on falsework. Although the single-cell box has been the most important cross-section for this method of construction, many bridges have been built using multiple-cell boxes and T-sections.

12.5.5.4 Tendon Layout and Details

The longitudinal prestressing steel generally consists of internal bonded tendons draped within the webs. The width of the webs and the arrangement of the ducts within the webs must therefore be carefully detailed to ensure that there is sufficient clear cover and space between ducts to enable good placement and vibration of concrete. For curved bridges, tendon ducts in the webs must be detailed to provide sufficient resistance against pull-out.

Although tendon profiles have traditionally been drawn as parabolic curves, the tendons can also be laid out along circular curves. Generally speaking, the centroid of prestressing steel should be as high as possible over the supports and as low as possible at mid-span. A reverse curve must be provided over the supports of continuous spans. The length of the reverse curve can be taken to be one fifteenth the length of the span[12.1] provided the radius of the tendon is greater than the acceptable minimum value for the size of tendon used (Fig. 12.36).

For short bridges, it is generally possible to lay out the tendons without intermediate anchors. For longer superstructures, the loss of prestress due to friction becomes significant and it is preferable to provide intermediate anchors. When intermediate anchors are used, it is usually necessary to place concrete in several stages, with a vertical construction joint between stages. The intermediate anchors can then be provided at these joints. Two possibilities exist for tendon layouts with intermediate anchors. The first uses tendon couplers, and is shown in Fig. 12.37. To prevent cracking, it is important that a significant percentage of the total prestressing steel be continuous at any given cross-section; coupling all of the tendons at a given section is not recommended. It is usually necessary to thicken the webs near the vertical construction joint to accommodate the couplers, which require spirals as do conventional tendon anchorages.

Tendons should be coupled as near as practical to the point of zero bending moment.

The second possibility, which uses tendons that overlap at the supports, is shown in Fig. 12.38. Because the tendons are laid span by span, this arrangement is readily adaptable to spans of varying length, which likewise require the total area of prestressing steel to vary from span to span. It is also necessary to thicken the webs near the construction joint. To simplify formwork, web thickness is generally kept constant from the pier to the intermediate anchors.

Coupled tendons and overlapping tendons can be combined as required to provide a layout of tendons that satisfies all design requirements.

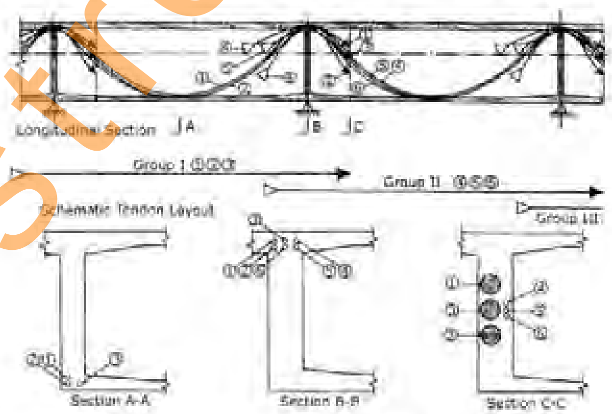


Fig. 12.38 Tendon Layout with Overlapped Tendons (adapted from Ref. 12.1)

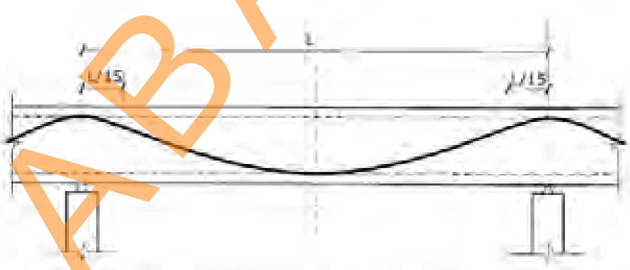


Fig. 12.36 Reverse Curvature of Draped Tendons

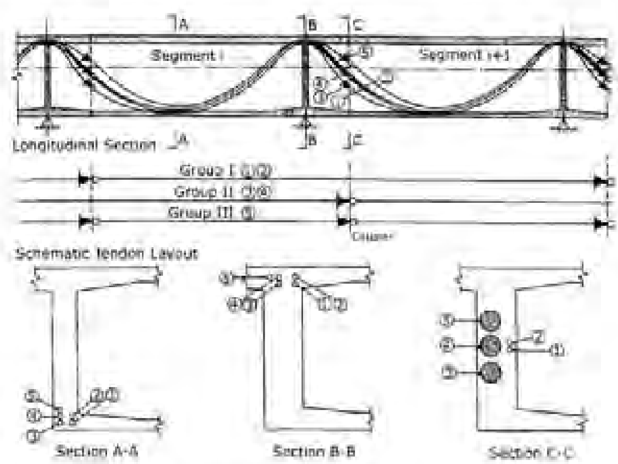


Fig. 12.37 Tendon Layout with Couplers (adapted from Ref. 12.1)

Fig. 12.39 Bras de la Plaine Bridge, La Réunion
Courtesy of International Bridge Technologies, Inc.

Fig. 12.40 Felsenau Bridge, Switzerland

Fig. 12.41 Cantilever Construction: Travelers and Formwork for Segments
Courtesy of VSL

12.5.5.5 Other Details

It is common to provide a horizontal construction joint at the top of the webs. This allows interior formwork for the webs to be installed and stripped without interference from above, and reduces to a minimum the formwork that needs to be stripped while working inside the cavity of the box.

12.5.6 Methods of Construction: Cast-In-Place Cantilever Construction

12.5.6.1 Description of Method

As its name implies, cantilever construction is a means of building bridge superstructures as cantilevers, by extending them outward progressively from the fixed end by adding segments to the free end. After the cantilevers have reached their final length, they are incorporated into a continuous structural system by connecting them to adjacent portions of the superstructure. The primary advantage of cantilever construction is that it enables bridge girders to be built without falsework. An important challenge for designers arising from this method relates to the change in structural system (from statically determinate cantilever to an indeterminate continuous system) that the girder must undergo.

The simplest application of the method is to the construction of single-span bridges from two half spans, each built as cantilevers extending from a fixed foundation. After the cantilevers meet at mid-span, they are connected to each other to provide the state of continuity required for the completed structure. The Bras de la Plaine Bridge in the French island territory of La Réunion (Fig. 12.39) is an example of such a bridge.

More commonly, though, cantilever construction is used to build multiple-span bridges. The method begins with construction of the piers. The superstructure is built outward from the tops of the piers on both sides, maintaining a close balance of the cantilever moments in the girders on either side of a given pier (Fig. 12.40). After both cantilevers reach the midpoint of a given span, they are connected together.

Superstructures built by cantilever construction can be assembled from segments that are cast in place using movable formwork suspended from the tip of the cantilever, or from precast concrete segments. This section deals with the former method of construction. The precast segmental method of cantilever construction will be discussed in Section 12.5.7.

Formwork for cast-in-place cantilever construction is shown in Fig. 12.41, which shows two sets of forms, one on the tip of each cantilever. The formwork is suspended from a steel frame called a traveler, which is attached to the completed portion of the superstructure. After a given segment is complete, travelers and formwork are advanced

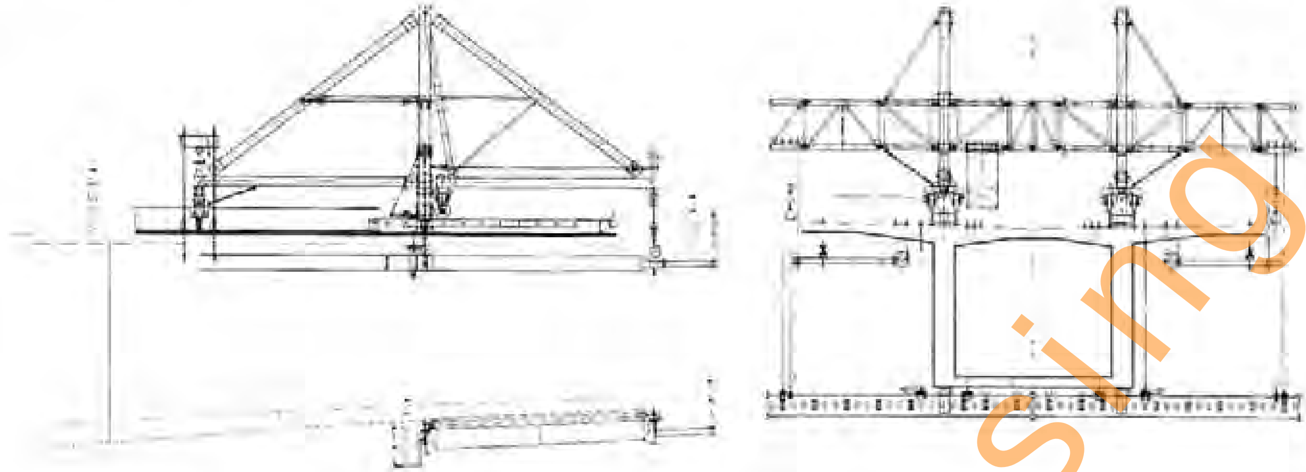


Fig. 12.42 Details of Traveler and Segment Formwork
Courtesy of VSL

into place to construct the next segment. Details of a traveler and segment formwork are shown in Fig. 12.42. Many variations on this basic theme are possible.

Because cantilever construction eliminates the need for falsework, it effectively frees the construction process from the need to make any contact with the ground or water below a given span. The method is thus highly adaptable to a wide range of applications. Cast-in-place cantilever construction has been a cost-effective means of building spans ranging from 60 m (200 ft) to over 300 m (1000 ft). It has been used to cross major bodies of water, deep mountain canyons, and densely populated urban areas. It can be used effectively to build bridges on alignments that are curved in plan.

The only specialized pieces of equipment required by the cast-in-place method of cantilever construction are the travelers and formwork. Heavy lifting equipment is generally not required. In most cases, a tower crane at each pier provides sufficient lifting capacity to erect travelers and formwork, as well as to transport reinforcing steel, prestressing strands, and concrete. Because construction of the superstructure remains essentially cast in place, the method can be used with success in most parts of the world.

The primary disadvantage of the method, compared to precast segmental cantilever construction, is that it is relatively slow. The duration of the construction cycle depends to a large extent on the time required for curing the concrete in the segments. Cycle times for cast-in-place cantilever construction are generally within the range of 3 to 4 days.

12.5.6.2 Longitudinal Structural System

The longitudinal structural system of cantilever constructed bridges changes significantly during construction. Initially, it consists of a set of independent double cantilever beams connected at their center to a pier such that bending moments created by unbalance of the cantilevers can be transferred to a foundation. This system is referred to as the "cantilever" system (upper diagram in Fig. 12.43). Once they have been constructed to their final length, adjacent cantilevers are connected at mid-span by a closure segment, which transforms the structural system into a continuous girder system. Depending on the project requirements, the moment connection between girder and pier that existed during construction can be released. The system created after construction of the closure segments and the release of bending restraint at the connection between girder and pier (if required) is called the "continuous system" (lower diagram in Fig. 12.43).

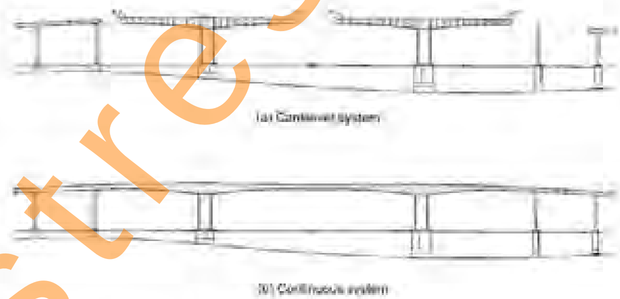


Fig. 12.43 Cantilever System (above)
and Continuous System (below)

It follows that the cantilever system must resist the entire dead load of the girder. Superimposed dead loads, such as the weight of barriers and wearing surface as well as live load, are carried by the continuous system. (There will be some redistribution of dead load of the girder to the continuous system due to creep in concrete, which is discussed later in this subsection.) Because the load that must be carried by the cantilever system is substantial, the behavior of the cantilever system must be given specific consideration in the development of design concepts and not merely treated as an afterthought after design is complete.

Fig. 12.44 Arrangement of Spans

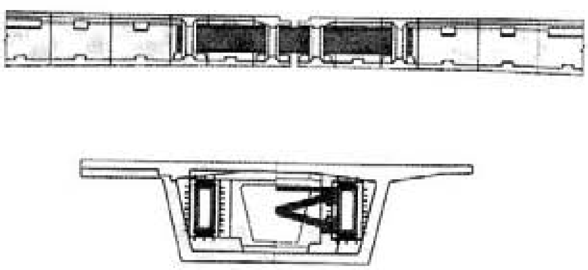


Fig. 12.45 Closure Segment Allowing Axial Displacement and Maintaining Continuity for Shear, Moment, and Torsion
Courtesy of International Bridge Technologies, Inc.

Because most of the dead load is carried by the cantilever system, the arrangement of spans must be chosen to ensure that there is no uplift at the ends of side spans under live load. As shown in Fig. 12.44, if the length of the side spans L_s is half the length of the main span L_m, then the dead load reactions at the end piers will be zero. If live load is applied to the main span only, then there will be uplift of the girder at the end piers. To ensure that there is sufficient dead load reaction at the end piers to overcome uplift under live load, it is common to make L_s greater than half of L_m. This implies that the side spans cannot be built entirely by balanced cantilever construction. It is common to build the side spans in cantilever construction out to a length of $L_m / 2$ and then to build the remainder on falsework.

The closure segment is designed to provide a specific degree of continuity between the two adjacent cantilever girders. In its simplest form, the closure segment is a monolithic cast-in-place concrete segment that provides continuity for all sectional forces (axial force, shear, bending, and torsion). This type of closure segment is commonly used where the cantilever constructed portion of the bridge is relatively short and expansion joints in the girder can be provided outside the limits of cantilever construction. When the length of cantilever constructed superstructure is sufficient to require intermediate expansion joints, mid-span closure segments that allow relative axial displacement of the superstructure can be used. In such cases, it is recommended to maintain continuity for shear, bending, and torsion at mid-span. This can be accomplished through the use of a pair of steel beams that pass through the closure segment and are supported such that they can transfer a couple of forces to the girder on both sides of the expansion joint (Fig. 12.45).

If the properties of concrete did not change over time, the calculation of the total sectional forces due to permanent load would be a simple matter of adding the forces due to dead load, calculated on the cantilever system, to the forces due to superimposed dead load, calculated on the continuous system. Under constant stress, however, concrete undergoes gradually increasing strain, which can be regarded as an apparent reduction in modulus of elasticity over time. This phenomenon, called creep, causes a change in sectional forces due to dead load related to the change in structural system from the cantilever system to the continuous system.

The effect of creep on bending moments due to dead load is illustrated in Fig. 12.46. The moment diagram produced by dead load applied to the cantilever system [labeled (a) in Fig. 12.46] is called $M_{D,cant}$. These are the moments in the girder at the time the closure segment is constructed. The fictitious moment diagram corresponding to dead load applied to the continuous system [labeled (b) in Fig. 12.46] is called $M_{D,continuous}$. The mathematical relation between the two diagrams is described by the following relation:

$$M_{D,continuous} = A + Bx + M_{D,cant}$$

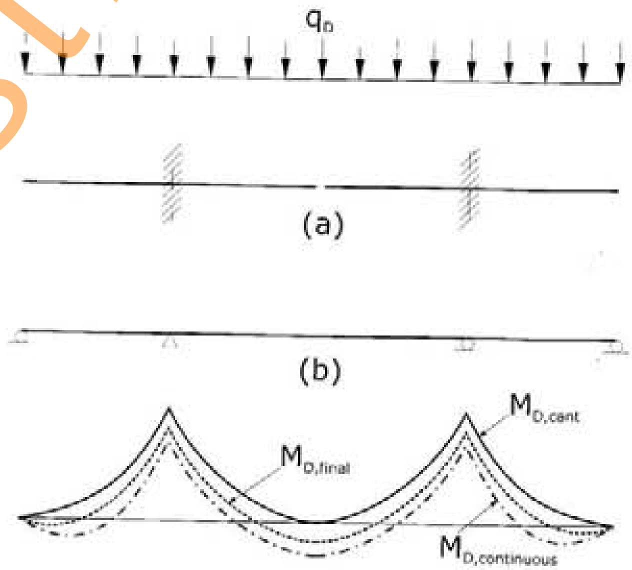


Fig. 12.46 Redistribution of Bending Moments Due to Creep

where A and B are constants and x is the horizontal coordinate measured along the length of the girder. The effect of creep can be visualized as a shift of the $M_{D,cant}$ diagram towards the $M_{D,continuous}$ diagram. The final bending moment diagram, $M_{D,final}$, is located between $M_{D,cant}$ and $M_{D,continuous}$.

Specialized structural analysis software with nonlinear capabilities can be used to calculate $M_{D,final}$. In the concept phase of design, however, $M_{D,final}$ can be estimated using the following equation:

$$M_{D,final} \approx M_{D,cant} + 0.8\,(M_{D,continuous} - M_{D,cant})$$

(The preceding discussion applies to the calculation of the effects of all permanent actions applied in the cantilever system. In general, these include dead load of the girder plus prestress applied on the cantilever system.)

Superstructure segments should be of equal length. Available travelers are generally adapted to the construction of segments between 3.5 m (11 ft) and 5.0 m (16 ft). It is generally not possible to place concrete in equal increments at both cantilever ends, and hence to eliminate the unbalanced moment applied to the piers entirely. To minimize this moment, it is common to set the length of the first segment on one side of the pier to be half the length of the standard segments.

Bending moments in the piers resulting from unequal bending moments in the cantilever girders they support can be significant, especially for long-span bridges. They can be resisted by the piers on their own or with the help of temporary supports. When these unbalanced moments are resisted by the piers on their own, it is important to maintain the flexibility of the piers under longitudinal shortening of the superstructure due to prestressing, creep, and shrinkage. Piers with two legs, as shown in Fig. 12.40, are commonly used to accomplish this goal. The spacing of the legs is dimensioned to provide sufficient bending capacity to resist the unbalanced cantilever moments from the superstructure and the thickness of the legs is selected to maximize the flexibility of the legs to control cracking in the pier as the superstructure shortens.

12.5.6.3 Cross-Sections

Boxes are the only practical cross-sections for cantilever construction, due to the large negative moments that are produced by dead load in the cantilever system. Although double-cell boxes have been used for wide bridges, the single-cell box is preferred. To work with three webs rather than two, it is necessary to increase the number of frames in the travelers, the number of tendons, and the labor required to place reinforcing steel in the webs.

It is most common for superstructures built using the cast-in-place cantilever method to have variable depth. Varying the depth of the cross-section can be easily accomplished using current traveler and formwork technology, and minimizes demand on the girder by reducing dead load in the span. The span to depth ratios proposed in Section 12.5.2.1 for variable depth girders can be used.

It is common to increase the thickness of the bottom slab close to the piers to ensure that the compressive stress block at ultimate limit state in bending lies completely within the bottom slab. Longitudinal reinforcement has been provided in the bottom slab to enable its thickness to be minimized.

Dimensions of cross-section components need to be selected to accommodate the ducts and anchorages of the longitudinal tendons that will be used. These requirements will be discussed in the following section.

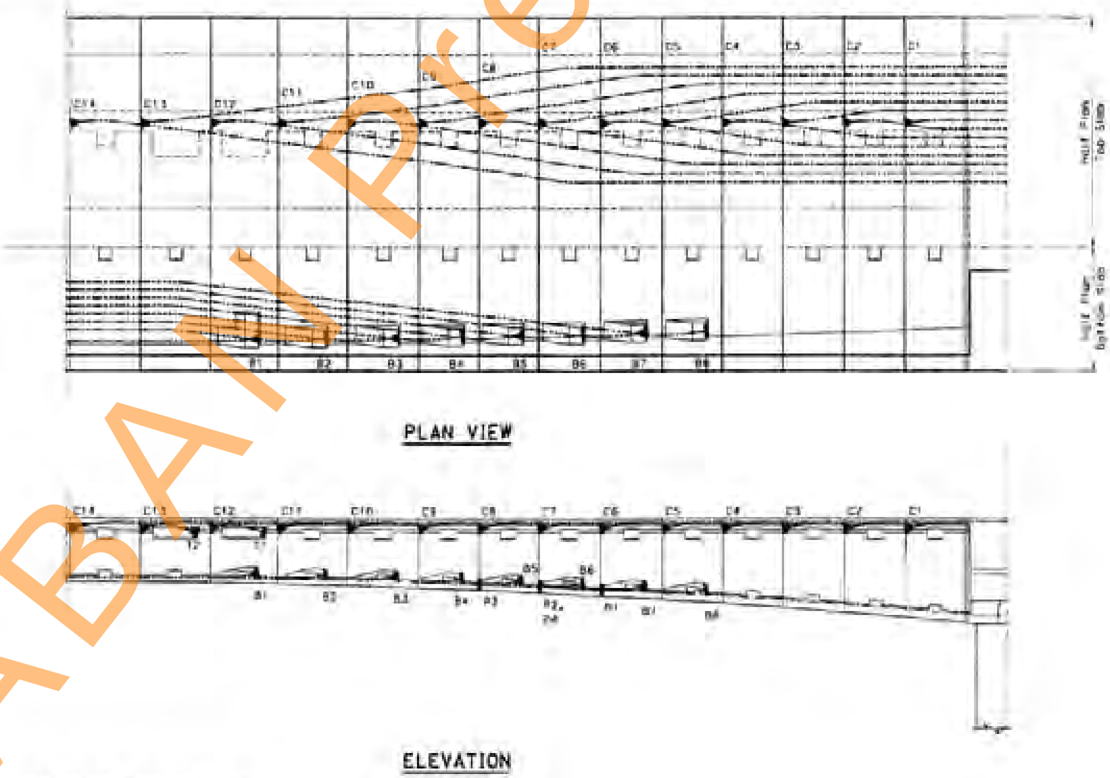


Fig. 12.47 Longitudinal Tendon Layout
Courtesy of International Bridge Technologies, Inc.

12.5.6.4 Tendon Layout and Details

Longitudinal tendons used in cantilever constructed superstructures can be divided into two groups: cantilever tendons and continuity tendons.

Cantilever tendons are provided to resist dead load bending moments in the cantilever system and to provide resistance for all other applicable loadings in the completed structure. They are located in the top slab. The total prestressing force provided by cantilever tendons is varied along the length of the girder according to bending demand, thus providing a highly economical use of post-tensioning. This is accomplished by adding tendons in more or less equal increments as each pair of segments (one at each cantilever end) is constructed. Cantilever tendons are identified by the prefix "C" in Fig. 12.47. As shown in the figure, the cantilever tendons are spread over the width of the top slab and are brought inward to the web to be anchored. The top slab must therefore be thick enough to accommodate the ducts and anchorages of the longitudinal tendons.

Continuity tendons are installed after the closure segment has been constructed. They provide positive moment resistance at mid-span and generally supplement the capacity of the cantilever tendons. They are generally provided in the bottom slab, and are arranged in equal increments moving back from mid-span. Because the segment faces are no longer accessible when the continuity tendons are installed and stressed, they must be anchored in built-outs provided at the junction of the bottom slab and the web. These tendons are identified by the prefix "B" in Fig. 12.47.

If required, additional continuity tendons can be draped in the webs of the box girder or provided as external, unbonded tendons inside the cavity of the box girder.

12.5.6.5 Other Details and Analytical Considerations

The intersection between girder and pier must be properly detailed to ensure a clean flow of forces "around the corner" to transfer unbalanced moment from girder to pier. As shown in Fig. 12.48, when a triangular diaphragm is provided, the problem can be solved using a simple two-dimensional flow of forces. When a pair of vertical diaphragms is provided (as would likely be the case when double-leg piers are used), however, a simple two-dimensional model is not feasible. In such cases, it is necessary to create a three-dimensional flow of forces in which the diagonal force shown in Fig. 12.48 is provided in the webs of the box girder. Guidance on this issue is given in Ref. 12.1.

The forms for segments of cantilever constructed girders must be carefully set to ensure that the final profile of the bridge, after all long-term effects have taken place, is in accordance with the project requirements. This requires careful calculation of long-term deflections of the girder due to creep. Nonlinear structural analysis software is normally used for this purpose. These calculations are normally not required in the concept phase of design.

12.5.7 Methods of Construction: Precast Segmental Cantilever Construction

The precast segmental method of cantilever construction is similar in many important regards to the cast-in-place method. Instead of casting the segments at the tips of the cantilevers, the segments are precast, lifted to the cantilever tips, and attached to the previously completed portion of the structure using post-tensioning. Only the significant differences between the two methods will be discussed in this section.

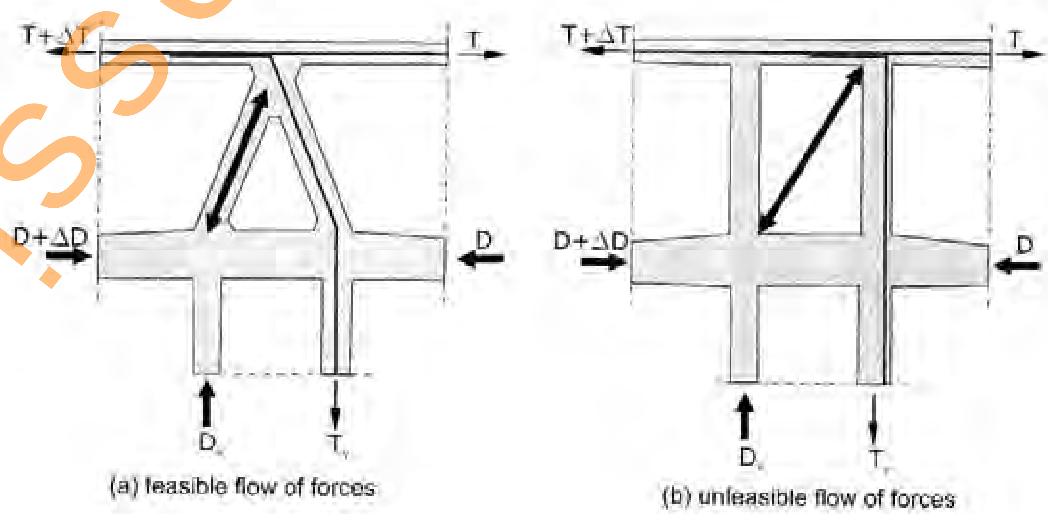


Fig. 12.48 Transfer of Moment from Girder to Pier

Fig. 12.49 Formwork for Match-Casting of Precast Concrete Segments
Courtesy of Hilliard Bond

12.5.7.1 Description of Method

The cantilever construction using precast concrete segments is made possible by the method of match-casting. Match-casting is a means of producing precast concrete segments that can be erected and assembled together on site with no cast-in-place concrete or grout required between the segments. This is accomplished by placing concrete for a given segment against the segment that will be its mate in the completed structure.

The equipment used for match casting is shown in Fig. 12.49. The previously cast segment (called the match-cast segment) is visible in the foreground of the left-hand photograph. Behind this segment is the formwork in which the adjacent segment will be cast. The segment to be cast is called the wet-cast segment. Concrete for the wet-cast segment is placed against the match-cast segment. In this way, the match-cast segment serves as formwork for one face of the wet-cast segment. Once the concrete in the wet-cast segment has hardened, the match-cast segment is moved into storage and the previous wet-cast segment is moved out of the formwork to become the match-cast segment for the next segment to be cast.

By casting one segment against another, a joint that provides perfect contact of both mating concrete services is obtained. In addition, by adjusting the position of the match-cast segment relative to the wet-cast segment, it is possible to build in specific angle breaks between adjacent segments. The accumulation of these angle breaks between segments allows a superstructure of any given geometry to be produced.

Fig. 12.50 Erection Truss for Cantilever Construction

Precast segmental cantilever constructed bridges are normally erected using one of the following methods:

1. Where access is available for cranes below the bridge, and when the height of piers permits, segments can be hoisted by crane to the tips of the cantilevers and stressed into place.
2. A lifting device (commonly called a beam and winch) can be mounted to the tips of the cantilevers and used to hoist segments into place. Erection by beam and winch is a valid option when conditions under the bridge do not permit the use of cranes.
3. An erection truss (Fig. 12.50) can be used to enable segments to be erected without any contact with the ground below the bridge. Segments are transported by trailer up the completed portion of the bridge to the truss, which then transports the segments to the tip of the cantilever. After a given double cantilever has been erected and the closure segment constructed, the truss is then launched into position to erect the next double cantilever.

Before attaching a given segment to the previously completed portion of the superstructure, the match-cast joint faces are coated with epoxy adhesive, which ensures that the joint is properly sealed.

Precast segmental cantilever constructed bridges are cost-effective for spans ranging from 45 m (150 ft) to over 100 m (328 ft). Below 45 m (150 ft), the span-by-span method (Section 12.5.8) is often faster and more economical. Above 100 m, the size and weight of segments generally exceeds applicable limits for transportation over highway. In general, only if segments can be transported from the precasting facility to the erection site entirely by barge can the use of precast segmental cantilever construction be considered for spans above 100 m (328 ft).

Within the 45 (150 ft) to 100 m (328 ft) range of spans, the primary advantages of the method include many of the advantages cited previously for cast-in-place cantilever construction. In addition, the precast segmental method offers speed of construction, high intrinsic durability due to 100 percent use of precast concrete, and, when the number of segments is sufficiently large, lower construction cost.

12.5.7.2 Longitudinal Structural System

Most of the issues discussed in Section 12.5.6 for cast-in-place cantilever construction are applicable to precast segmental cantilever construction.

The mid-span closure segment is constructed of cast-in-place concrete.

The equation given in Section 12.5.6 for the calculation of the final value of bending moments due to loads applied to the cantilever system should be adapted for precast concrete, which generally has a lower creep coefficient than cast-in-place concrete. The applicable equation is as follows:

$$M_{D,final} \approx M_{D,cant} + 0.7\ (M_{D,continuous} - M_{D,cant})$$

12.5.7.3 Cross-Sections

The single-cell box is the preferred cross-section for precast segmental cantilever construction. Formwork for box segments with more than two webs is prohibitively complex. The American Segmental Bridge Institute[12.8] has produced a set of drawings for standard segments for precast segmental construction. The standards included for cantilever construction cover the spans ranging from 30 m (100 ft) to 60 m (200 ft). Within this range of spans, constant-depth girders are generally feasible and cost-effective. If additional bending capacity for negative moment is required when using constant-depth sections, the thickness of the bottom slab can be increased from within the box near the piers.

12.5.7.4 Tendon Layout and Details

Tendon layouts and details are generally similar to those described in Section 12.5.6 for cast-in-place cantilever construction.

12.5.7.5 Other Details and Analytical Considerations

The pier segment, i.e., the superstructure segment located immediately on top of a given pier, is generally precast. Although it is possible to produce a monolithic connection between superstructure and pier when a precast pier segment is used, this generally requires a cast-in-place pour which slows down erection. Pier segments of precast segmental bridges are thus generally supported by bearings. Initial adjustment of the position and orientation of the pier segment is crucial, since any errors in angle will be magnified as the cantilever is extended. To accomplish this, the pier segment is generally erected onto temporary jacks, which can be used to correct the geometry of the segment. Pier segments supported by bearings in the completed structure are generally locked in place temporarily to the pier using temporary tendons and blocks, to ensure the transfer of moment between girder and pier.

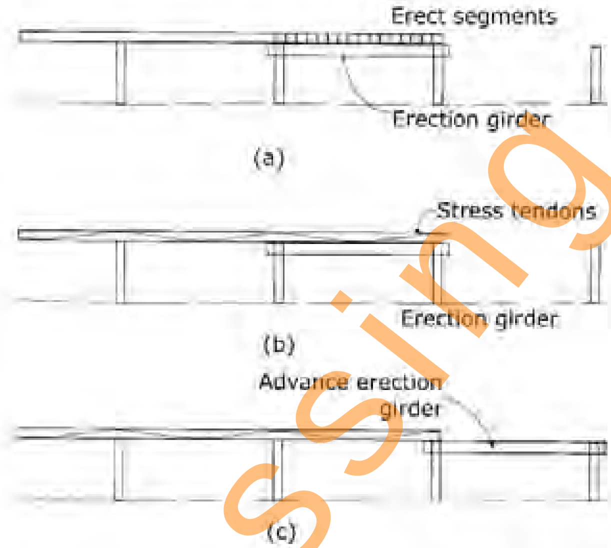


Fig. 12.51 Span-by-Span Method of Precast Segmental Construction

12.5.8 Methods of Construction: Precast Segmental Span-By-Span Construction

12.5.8.1 Description of Method

The span-by-span method of precast segmental construction consists of the following primary steps: (a) erecting the segments for an entire span onto a temporary erection girder spanning between a pair of adjacent permanent piers, (b) installing and stressing longitudinal tendons enabling the segments to span on their own, and (c) advancing the erection girder into place to erect the adjacent span. These steps are shown schematically in Fig. 12.51. Since there is only one cycle of stressing and grouting of tendons per span, the method can be significantly faster than precast segmental cantilever construction, which requires one such cycle per pair of segments.

The most common use of span-by-span construction is to build long viaducts with spans of similar length. For this type of bridge, the method has been used most often for spans ranging from 30 m (100 ft) to 45 m (150 ft). The lower limit of this range corresponds to a span to depth ratio of about 17 and a depth of cross-section of about 1.8 m. Both of these values can be considered to be effective minima (see Sections 12.5.2.1 and 12.5.3.3). As spans increase, there is a significant increase in the cost of the erection girder. In contrast to launching trusses used in cantilever construction, which must support at most two segments at a given time, erection girders for the span-by-span method must support all of the segments in a given span simultaneously. Although span-by-span construction has been used for spans of up to 55 m (180 ft), the cost of the erection girder has generally made other methods more cost effective for spans greater than 45 m (150 ft).

Fig. 12.52 Under-Slung Erection Girder

Fig. 12.53 Overhead Erection Girder

Erection girders can support the segments from below or above. Fig. 12.52 shows an example of the use of under-slung erection girders supported on temporary frames provided at the permanent pier locations. The girders are located outboard of the webs of the box girder segments, which allows them to support the segments by the deck slab cantilevers. This maximizes clearance under the erection girders and allows them to be launched forward with relative ease. Overhead erection girders, such as the one shown in Fig. 12.53, have also been used. With this type of equipment, segments are suspended from the girder before they are post-tensioned. Overhead girders allow the superstructure to be erected entirely from the previously completed portion of the bridge, without any need for heavy equipment on the ground.

Regardless of the type of erection girder used, the span-by-span method involves supporting segments during construction on members that are straight between piers. For bridges on curved alignments, the feasible length of span decreases with decreasing radius of curvature. This proposition is illustrated in Fig. 12.54 for under-slung girders supporting segments under the deck slab cantilevers (the system shown in Fig. 12.52). It can be seen that increasing the span length for a given radius increases the distance a from the web of the box girder to the erection girder. For a given radius, as span length increases, dimension a will become prohibitively large, either requiring excessive amounts of transverse reinforcement in the bottom face of

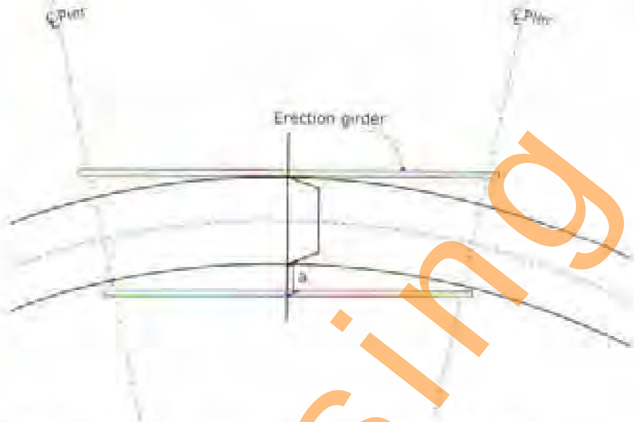


Fig. 12.54 Curved Superstructure Supported on Straight Erection Girders

the deck slab, or exceeding the length of the deck slab cantilevers altogether.

12.5.8.2 Longitudinal Structural System

Superstructures built using the span-by-span segmental method can be designed as continuous girders or as a series of simply supported spans. For highway bridges in areas where deicing salt is applied to bridge decks, simply supported spans are not recommended. For very long viaducts, intermediate expansion joints separating continuous portions of the superstructure are generally provided at the centerlines of the piers.

Bearings are provided at all piers to support superstructures built using the span-by-span method. Eliminating monolithic connections between superstructure and piers speeds construction.

12.5.8.3 Cross-Sections

The single-cell box cross-section is the most common type of cross-section used for the span-by-span method. Strutted box sections (such as the one shown in Fig. 12.30) have also been used. Double-T sections would be theoretically possible for short span applications, but they have not been used in any significant application to date.

The webs of boxes used for span-by-span construction are generally thinner than those used for other types of construction, partly because the span-by-span method is rarely used for spans greater than 45 m, and also because external, unbonded tendons are used to post-tension span-by-span bridges longitudinally. There is thus no need to dimension the webs to accommodate tendon ducts.

The bottom slab of span-by-span bridges is usually thin relative to boxes used for other types of construction, since the method of construction biases the dead load moment diagram towards positive moment.

12.5.8.4 Tendon Layout and Details

External unbonded tendons are used for the longitudinal post-tensioning of bridges built using the span-by-span method. The use of external tendons results in thinner cross-sections and faster construction. Although internal tendons have been used with the span-by-span method, any reduction of prestressing steel gained through their use is usually more than offset by additional complications during erection on site.

The basic tendon layout is shown in Fig. 12.55 in standard orthogonal views and in Fig. 12.56 in a cut-away perspective view. Each tendon extends the length of one span. Tendons from adjacent spans overlap within the pier segment and are anchored in the exterior faces of the diaphragm. They are deviated downward as they leave the pier diaphragm and enter the cavity of the box girder. As they approach the bottom slab, the tendons are deviated again, either back up to the opposite pier segment or to a horizontal profile. Segments with partial diaphragms, called deviation segments, are provided for this purpose. Deviation segments can be designed and detailed to deviate several tendons (Fig. 12.55) or individual tendons (Fig. 12.56). When all of the tendons in a given span are deviated at a single deviation segment, it is generally located between the quarter and third point of the span.

All remaining segments are essentially identical and are called intermediate segments. The tendons pass through the box cavity of these segments without making contact. They can therefore be detailed very simply.

12.5.8.5 Other Details and Analytical Considerations

This section briefly discusses the following issues requiring consideration in design that are specific to span-by-span bridges: (1) Detailing of pier segments, (2) Detailing of deviation segments, (3) Detailing of closure pours, and (4) Behavior of girders prestressed with external unbonded tendons at ultimate limit state.

1. **Detailing of pier segments**: Pier segments must be detailed to ensure adequate behavior under the load cases normally considered for pier diaphragms, which are described in Section 12.5.3.6. In addition, they must resist the forces imposed by the longitudinal tendons anchors. After construction is complete, the forces applied from the anchors on one side of the diaphragm are balanced by forces applied from the other side, as shown in Fig. 12.57 (only forces produced by the tendon anchors are shown).

   During construction, however, the diaphragm must resist forces applied by tendons anchored on one side only. The corresponding flow of forces is depicted in the truss model shown in Fig. 12.58. The load applied by the tendon anchors must spread into the entire cross-section. This requires tensile reinforcement in the diaphragm face opposite the anchors. The flow of forces shown is a two-dimensional simplification of a three-dimensional condition, showing only the effect of prestressing force. Designers must account for the complete three-dimensional spreading of forces as well as the simultaneous presence of dead load in establishing the flow of forces in the diaphragm.

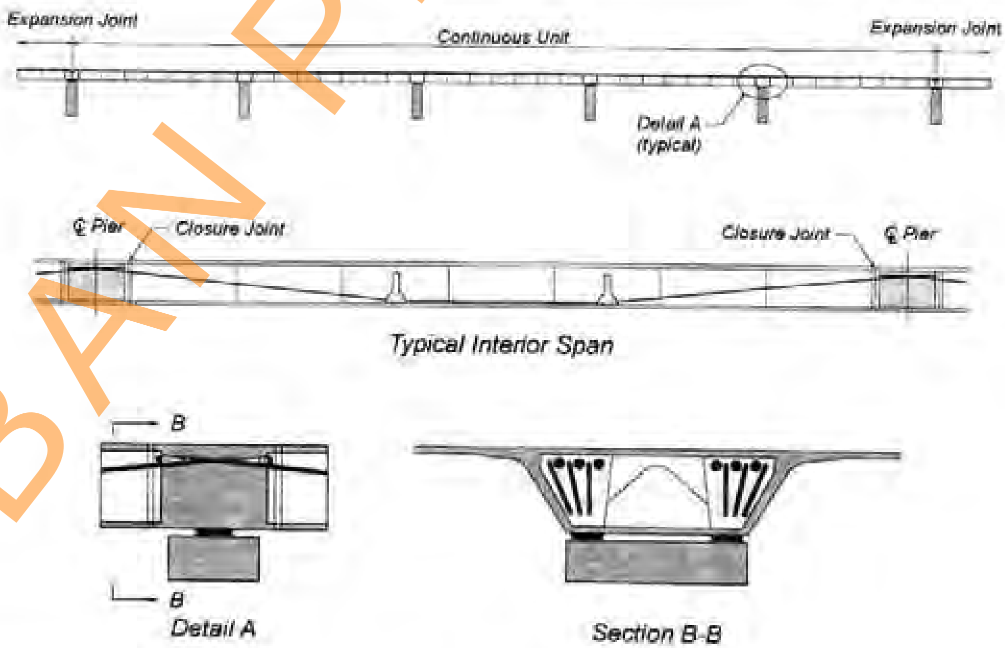


Fig. 12.55 Layout of Tendons for Span-by-Span Construction: Longitudinal Section
Courtesy of Corven Engineering, Inc.

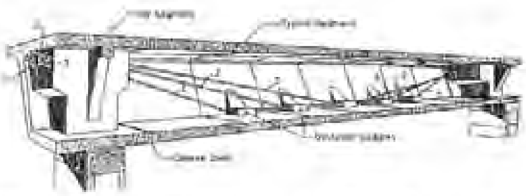


Fig. 12.56 Layout of Tendons for Span-by-Span Construction: Three-Dimensional View
Courtesy of American Segmental Bridge Institute

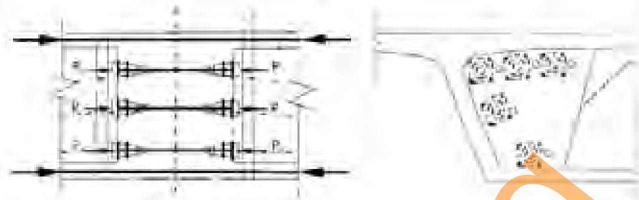


Fig. 12.57 Flow of Forces in Pier Segments: Balanced Condition After Tendons on Both Sides Have Been Stressed

A similar situation occurs at segments where expansion joints will be provided. This type of segment is provided at abutments and over piers where intermediate expansion joints are required. The balanced force condition shown in Fig. 12.57 does not occur at these segments, since longitudinal tendons are anchored only on one side of the diaphragm. The flow of forces shown in Fig. 12.58 must thus be considered in the final state of the bridge after construction.

2. **Detailing of deviation segments**: The flow of forces in deviation segments is shown in simplified fashion in Fig. 12.59. The upward component of tendon force, D, must be transferred from the interior of the box to the webs. This is accomplished for the case shown through partial diaphragms, which allow the force to be brought to the junction of top slab and web. There, it is brought down as a tensile force in the webs. Tensile reinforcement is required in the top slab to resist the horizontal force T_{Deck}.

   Tendons passing through a deviation segment cannot be detailed with a concentrated angle break as shown in Fig. 12.59, but must rather follow a smooth curve that respects guidelines for the minimum radius of bend for external tendons. It is common to embed steel pipes, that have been previously bent to the required radius, into deviation segments for this purpose. Alternatively, flared trumpets (commonly referred to as diabolos) can be used (Fig. 12.60). The advantage of diabolos is that they accommodate a range of angles of incidence of tendons. For bridges with spans of varying length and geometry, it is possible through the use of diabolos to detail all deviation segments identically, in spite of the lack of repetition on tendon geometry. The primary disadvantage of diabolos is that they require more space than bent pipes and hence larger transverse diaphragms.

3. **Closure pours**: Cast-in-place closure pours are provided at both ends of the spans of continuous girders, immediately adjacent to the pier segments. They are a means of adjusting the geometry of a given span and enable pier segments to be precast without the need to match-cast them with segments in the span. If the length of closure pours is no greater than the typical spacing of transverse reinforcement in the segments, they do not need to be reinforced.

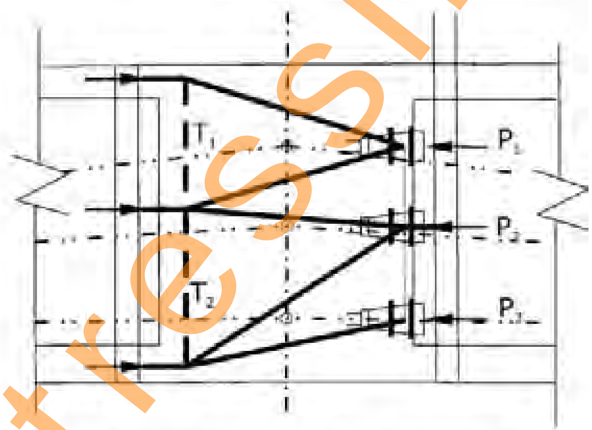


Fig. 12.58 Flow of Forces in Pier Segments: Unbalanced Condition After Tendons from Only One Side Have Been Stressed

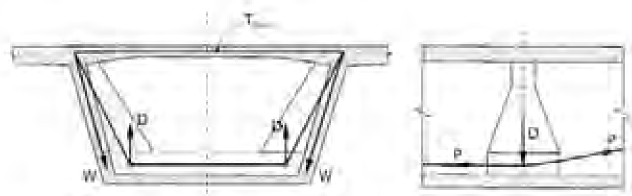


Fig. 12.59 Flow of Forces in Deviation Segments

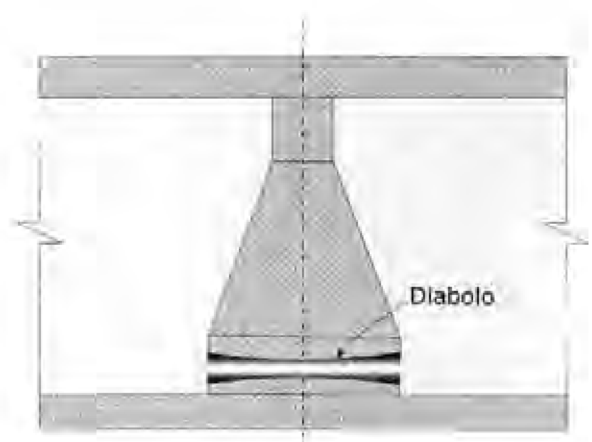


Fig. 12.60 Diabolo

4. **Ultimate limit state of girders with unbonded tendons**: The behavior of girders prestressed with unbonded tendons differs fundamentally from that of girders with bonded tendons. In the latter case, ultimate bending capacity can be calculated simply on the basis of the principle of plane sections, since the increase in strain in prestressing steel, $\Delta\varepsilon_p$, is equivalent to the strain in the concrete at the same location in the cross-section, ε_{cp}.

For girders with unbonded tendons, $\Delta\varepsilon_p$ is effectively constant along the length of a given tendon, and hence is not equal to ε_{cp} at any given location along the span. The increase in strain in prestressing steel can be calculated as the change in length of the unbonded tendon divided by its original length:

$$\Delta\varepsilon_p = \Delta l_p / l_p$$

where the change in length in prestressing steel is equal to the integral of strain in the concrete at the level of prestressing steel over the entire length of the tendon:

$$\Delta l_p = \int \varepsilon_{cp} dx$$

The calculation is iterative, i.e., it is necessary to assume a value of to get started and to correct its value until a satisfactory degree of convergence has been obtained.

For preliminary calculations, it is always conservative to assume that is zero, i.e., there is no increase in strain in the unbonded tendons at ultimate limit state.

12.5.9 Methods of Construction: Incremental Launching

12.5.9.1 Description of Method

Launching is a method of bridge construction by which the superstructure is built at grade behind one of the abutments and then slid longitudinally into its final position. Incremental launching is a special implementation of this method by which the superstructure is built in segments according to a cyclic procedure. The first two cycles of this procedure are illustrated in Fig. 12.61. A typical cycle (Steps 3 and 4) consists of constructing a segment in a casting yard behind the abutment, connecting it to the previously constructed portion of the superstructure by post-tensioning, and moving the entire assembly forward to permit another segment to be constructed. This cycle is repeated until the superstructure is complete and has reached its final position.

Launching significantly reduces (and often eliminates entirely) the need for falsework and construction equipment between the abutments. The method thus has the potential not only to reduce construction costs but also to minimize the impact of construction on activities below

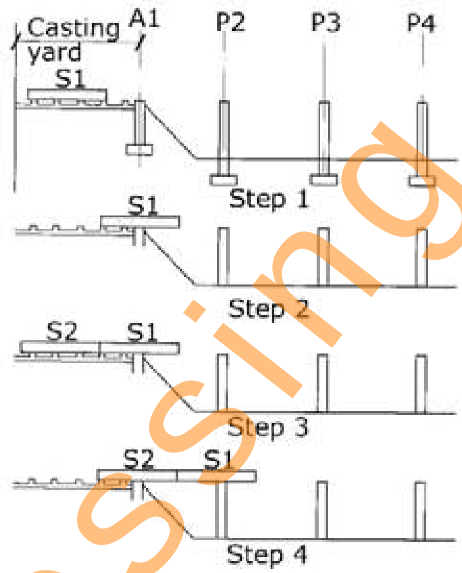


Fig. 12.61 Incremental Launching

the bridge. Due to the cost of the casting yard and other specialized equipment required for launching, the method is generally used only for bridges of considerable length.

Incremental launching has been used to build concrete bridges with spans of up to about 60 m, but more commonly with spans averaging about 40 m. The difficulty and cost of building incrementally launched bridges increases sharply with span length due to the large cantilever moments that the girder must resist during construction. These moments are discussed further in Section 12.5.9.2.

The range of allowable bridge geometries that can be accommodated by incremental launching is, however, limited since the girder must displace as a rigid body along the axis of the structure in its final position. All segments produced must therefore be identical. Allowable geometries can be defined mathematically by considering the production of segments when the casting yard has infinitesimal length dx. The superstructure must therefore follow one of three basic types of curves, according to the shape of segments and orientation of the casting yard (Fig. 12.62):

- Case 1 – If the segments produced are rectangular, then the geometry of the superstructure is defined by a straight line.
- Case 2 – If the segments are wedge-shaped, then the superstructure will follow a path of constant radius, i.e., a circle.
- Case 3 – The circle defined in Case 2 lies in a plane. The casting yard can produce wedge-shaped segments lying in that plane (as described in Case 2) or at a constant angle α to that plane, in which case the superstructure will follow a path with constant slope

α and constant radius R, i.e., a helix. The longitudinal axis of the helix need not be vertical. A bridge that followed a crest curve in profile, for example, could be defined by a helix with horizontal longitudinal axis.

(Cases 1 and 2 are actually degenerate cases of Case 3. When angle α is zero, the helix becomes a circle. When the radius of the circle is infinite, it becomes a straight line.)

12.5.9.2 Longitudinal Structural System

The longitudinal structural system changes radically during construction. For this reason, the structural system during launching and the final structural system will be discussed separately.

1. **During launching** – As the girder is pushed away from the casting yard towards the opposite abutment, it must resist the combined effect of dead load and prestressing on a constantly changing structural system. The highest demand on the structural system is experienced by the forward portion of the girder. Large negative moments are produced in the girder as it extends in cantilever fashion out from a pier. These moments are greatest immediately before the tip of the girder touches down at a given pier. In addition, large positive moments are produced in the forward span immediately upon touchdown.

   Although the remainder of the girder experiences continually varying bending moments due to dead load during launching, the peaks are not as pronounced as those developed by the forward portion. Peak negative and positive bending moments are approximately equal to those corresponding to a single-span girder with both ends fixed, i.e., $(q_D L^2)/12$ and $(q_D L^2)/24$ respectively, where q_D is dead load.

   This behavior can be represented graphically by a moment envelope such as the one shown in Fig. 12.63. The maximum peak negative moment and positive moment in the forward portion are labeled A and B, respectively. The lower peak negative and positive moments in the other portions of the girder are also visible. They are essentially constant along the length of the girder.

   The implications of this moment envelope on choice of cross-section and post-tensioning concept are discussed in the following two subsections.

2. **After completion of construction** – Following completion of launching, the final dead load moment diagram is generally close to the moment diagram due to dead load for an identical structure built using a conventional method. All loads applied after launching is complete (superimposed dead load, live load, wind load) are carried in the same way as a conventionally built structure.

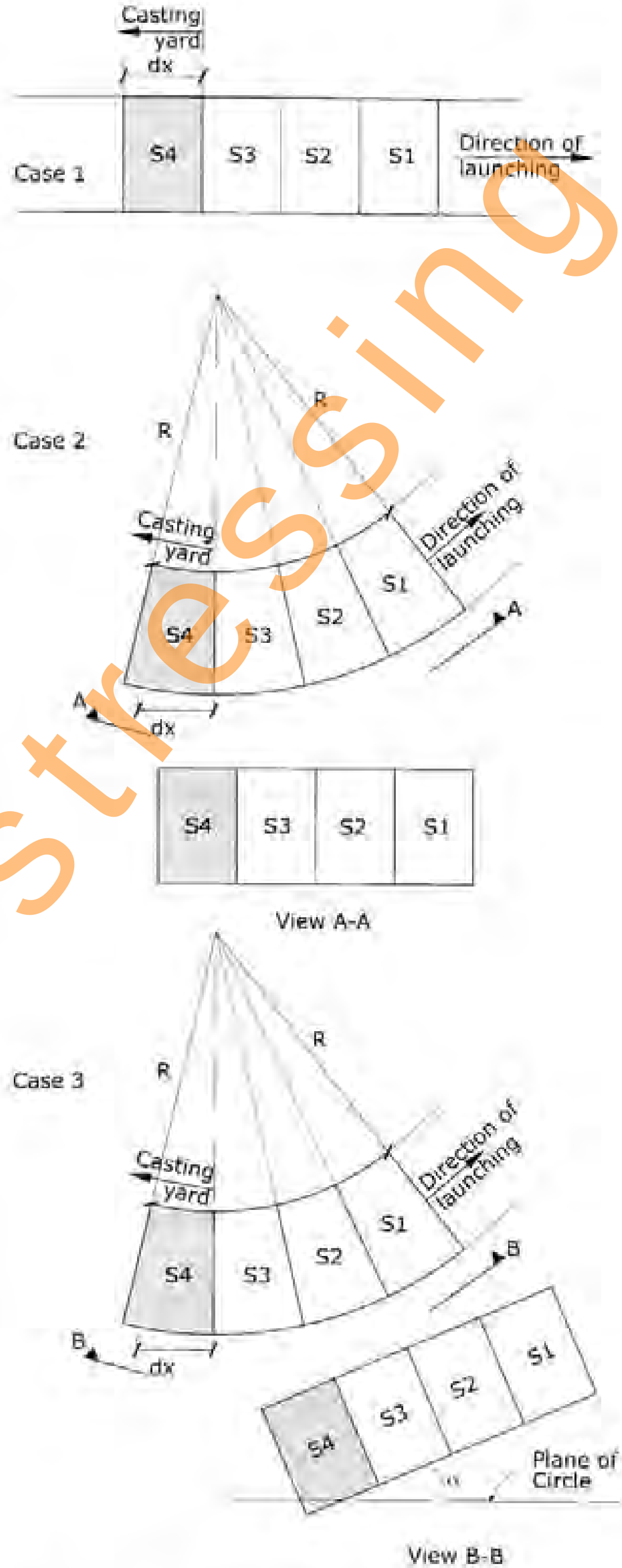


Fig. 12.62 Allowable Geometry for Incrementally Launched Bridges

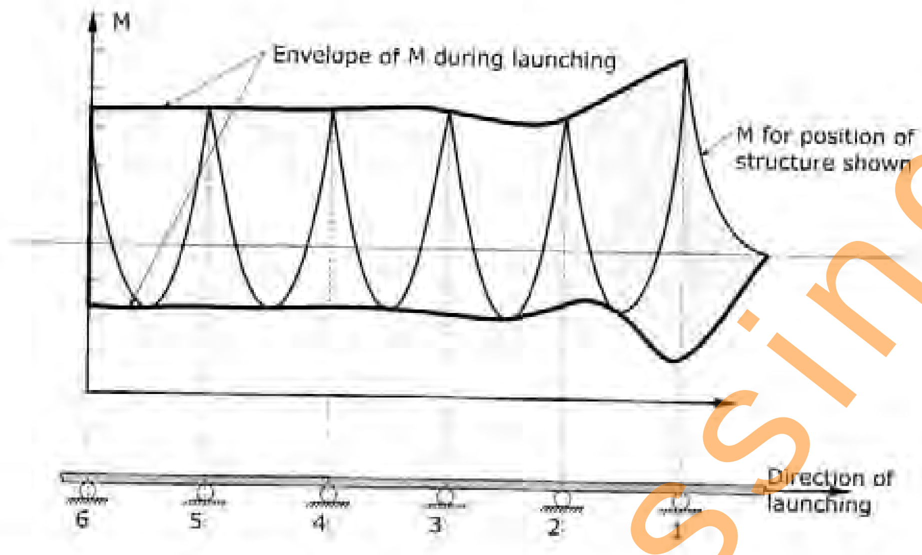


Fig. 12.63 Envelope of Moments in Girder Due to Launching

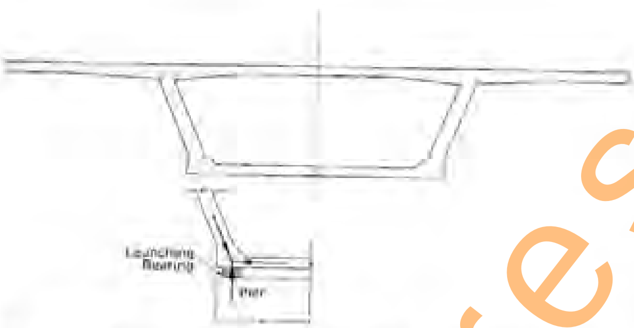


Fig. 12.64 - Cross-section for Incrementally Launched Bridges

Structural response to these loads can thus be calculated using normal tools and methods of analysis.

The final structural system must be buildable by incremental launching. Superstructures launched from one end must therefore be continuous from end to end, with expansion joints provided only at the abutments. The girder must be supported on bearings at all piers.

12.5.9.3 Cross-Sections

Single-cell boxes are the most commonly used cross-sections for incrementally launched bridges. Because they have a bottom slab, single-cell boxes can efficiently resist both positive and negative bending moments. This is important since, as discussed in Section 12.5.9.2, significant positive and negative bending moments due to dead-load will be present at every location in the superstructure during launching.

Box-girders with two or more cells have been successfully launched. This type of cross-section, however, should only be considered for very wide bridges. Generally speaking, the use of more than two webs creates difficulties in detailing prestressing steel and formwork, and requires additional launching equipment. These drawbacks tend to counteract benefits that might be obtained through the use of more than two webs.

A typical cross-section for incrementally launched bridges is shown in Fig. 12.64. The section is essentially identical to those used for cast-in-place construction on falsework (Section 12.5.5) except for the detail at the intersection of bottom slab and web. This detail is required to ensure the transfer of dead load reaction from the webs of the box girder to the piers. In bridges build using other methods, the location of these reactions does not change. Consequently, bearings can be located inboard of the webs and the transfer of forces can be accommodated by diaphragms (see Section 12.5.3.6). In incrementally launched bridges, however, dead load reaction must be transferred at all locations along the length of the bridge. Since it is obviously not feasible to provide diaphragms everywhere, the transfer of force must be accomplished more directly, with temporary bearings provided directly under the webs. To accommodate this load path, an enlargement of the joint between webs and bottom slab is required.

Cross-sections used for incrementally launched bridges tend to be deeper than those built by other methods of construction. Whereas a span to depth ratio of 20:1 is often used for conventionally built girders, a ratio of 15:1 is generally used for incrementally launched bridges.[12.9] Slenderness is reduced for incrementally launched bridges to give the cross-section sufficient capacity to resist high positive and negative bending moments due to dead load during launching.

Fig. 12.65 Launching Nose
Courtesy of Norbert Luksch

12.5.9.4 Tendon Layout and Details

In incrementally launched bridges, post-tensioning must perform two primary functions. As with any bridge, it must resist forces due to dead load, live load, and other loads in the completed structure. In addition, however, it must resist forces due to dead load during launching. Because the forces corresponding to these two functions have radically different distributions, it is usually not practical to provide a single arrangement of tendons to perform both functions. Instead, two separate post-tensioning concepts are generally provided, each adapted to the needs of one of the two primary functions.

In most cases, a concentric prestress is provided to resist the forces produced by dead load during launching. Tendons are straight over their entire length. This follows directly from the fact that at a given location in the superstructure, significant positive and negative bending moments must be resisted. It is thus not possible to arrange tendons in a way that produces bending moments due to prestress that balance these moments for all possible arrangements of the girder during launching.

Once launching is completed, the final dead load moment diagram is similar to the diagram that would be produced in a conventionally constructed girder. As stated previously, the additional loads applied after launching is complete (superimposed dead load, live load, etc.) produce forces identical to those produced in a conventionally constructed girder. Because the bending moments thus produced tend to be negative near the supports and positive in the span, it is practical to resist these moments using draped tendons, which can produce moments that counteract the moments produced by load.

In the "classical" implementation of incremental launching, all tendons were internal and bonded. The concentric prestress provided to resist forces produced by launching was therefore left in place after the girder was in its final position. This arrangement of tendons was supplemented by draped tendons provided in the webs. The two groups of tendons acted together in the completed structure to provide capacity at serviceability limit states and at ultimate limit state.

In recent years, several bridges have been built with a combination of internal bonded tendons and external unbonded tendons. On the approaches of the Normandie Bridge,[12.10] for example, concentric prestress was provided by two groups of external tendons, the first draped conventionally and the second draped in a reverse profile. The deviation forces produced by one group of tendons were therefore cancelled out by the deviation forces produced by the other group, thus leaving a pure concentric prestress. After completion of launching, the reverse draped tendons were detensioned and reinstalled with a conventional profile, leaving 100 percent of tendons in a draped condition. Although the labor required for these operations was significant, it produced a very efficient arrangement of tendons for both launching and the completed structure. Additional examples of the use of external tendons in incrementally launched bridges are given by Rosignoli.[12.11]

Particular attention must be paid to detailing of tendons as they cross the joints between segments. Tendon couplers and/or overlapping tendons have been used at these locations. Examples of commonly used details are given in Refs. 12.9 and 12.11.

12.5.9.5 Other Details and Analytical Considerations

To reduce the peak negative and positive moments in the girder during launching, a number of measures can be considered by designers, including the use of:

- **A launching nose** – This is a steel girder or truss that is attached to the front of the girder. It reduces both peak negative and positive moments by reducing the dead load of the forward portion of the girder. An example is shown in Fig. 12.65.
- **Temporary piers** – When used, they are provided halfway between the permanent piers, thus reducing the span during construction by half. This reduces moments due to dead load during launching at all locations along the girder.
- **Temporary stays** – The use of temporary stays and a temporary tower can provide additional resistance to negative moment by significantly increasing the flexural lever arm.

Additional information on these measures and other equipment required for incremental launching is given in Refs. 12.9 and 12.11.

12.6 DESIGN CONCEPTS FOR SLAB BRIDGES

Solid slabs are relatively inefficient structural systems. The efficiency index of solid slabs, as defined in Section 12.5.2.2, is only 0.33 compared to 0.6 for single-cell boxes. In spite of this, however, slabs can be cost-effective solutions for short-span bridges because they require significantly less labor to build than box girders or T-girders. At its most simple, the formwork for slab bridges need consist only of a bottom form and side forms. The placement of reinforcing steel and ducts for post-tensioning tendons is likewise simple and straightforward.

The maximum practical span to depth ratio for slab bridges is about 25:1. As stated in Section 12.6.2, the maximum cost-effective depth for slab bridges is about 0.8 m. This corresponds to an effective maximum span of 20 m.

Slab bridges are most often cast in place on conventional falsework. They are generally not suitable, therefore, for minimum-impact construction from the perspective of time or use of land under the bridge. Although it is possible to conceive of a precast segmental slab system, this method of construction has not yet been used.

12.6.1 Longitudinal Structural System

A number of longitudinal structural systems can be used for slab bridges. Single-span bridges can be simply supported or incorporated into rigid frames (see Section 12.7). Multiple-span slabs can be supported on bearings or can be monolithically connected to the piers. According to Menn,[12.3] the suggested maximum span of 20 m (65 ft) can be increased by 10 to 20 percent for slabs that are continuous over several supports or for slabs incorporated into frames.

Slabs can be designed to span over sharply skewed crossings, which are often required for grade separations. Additional information on the behavior and design of skew slab bridges is given by Menn.[12.3] Slab bridges need not be supported on piers that are continuous in the transverse direction. With adequate transverse prestressing, supports can be spaced relatively widely in the transverse direction.

12.6.2 Cross-Sections

As stated previously, the simplest cross-section for slab bridges is a simple rectangular slab. Dead load can be reduced to some extent by tapering the free edges of the slab.

The maximum practical depth of slab is approximately 0.8 m. For greater depths, the cost of the additional labor required to form and reinforce a multiple T-girder is generally more than offset by savings in concrete and post-tensioning.

The weight of slabs has been reduced through the use of longitudinal cylindrical lost forms. This type of cross-section is called a voided slab. Because the interior surfaces created by the lost forms are not accessible for inspection, and due to the difficulty of fixing the void forms in place during casting of concrete, voided slabs are not recommended.

Fig. 12.66 Single-Span Frame Bridge

12.6.3 Tendon Layout and Details

Equilibrium of slab bridges requires bending moments in two orthogonal directions. The "exact" calculation of these moments is a complex undertaking, and the layout and dimensioning of tendons to satisfy stress limits at serviceability limit states at every point on the slab for every possible position of live load is even more so. For this reason, it is suggested that tendons be arranged and dimensioned according to the following principles:

1. Arrange tendons in bands of parallel units oriented in the direction of primary principal stress.
2. Provide a band of tendons along each free edge of the slab.
3. Use reasonable redistribution of moments at ultimate limit state to make efficient use of tendons laid out in bands. The strip method[12.12] can be used to perform these calculations.

12.7 DESIGN CONCEPTS FOR FRAME BRIDGES

Frame bridges are systems that require significant horizontal reactions at the piers to maintain equilibrium, but which do not carry load in a state of pure compression as do arches. They are most often cast in place on falsework, but could conceivably be built using more mechanized methods to eliminate the need for temporary support from the ground.

Because horizontal reactions must be developed by the foundations, frame bridges are generally suitable only where good soil or rock is present.

12.7.1 Longitudinal Structural System

The two primary types of structural system used for frame bridges are single-span frames and inclined-leg frames.

A single-span frame bridge is shown in Fig. 12.66. The structural system is a simple haunched portal frame. This type of bridge can be cost-effective for spans in excess of 70 m. For example, one of the earliest post-tensioned bridges, Freyssinet's bridge over the Marne at Esbly, was a single-span frame with a span of 74 m (243 ft) (Fig. 12.5). Because of the restraint provided by the legs, large negative moments are developed at the ends of the girder, which make haunching the girder a sensible decision. The span to depth ratios suggested in Section 12.5.2.1 can be used for this type of bridge.

Single-span frame bridges are often used for grade separations. The relatively shallow depth of the superstructure helps to reduce the length of approaches for a given underclearance, and the absence of a central pier enhances highway safety.

An inclined-leg frame is shown in Fig. 12.67. This type of bridge is suitable for a wide range of spans. It is closer to arches in its behavior than the single-span frames discussed previously, in that the shape of the frame is closer to the true pressure line due to dead load. When used for long-spans, it is often more cost-effective and preferable aesthetically to design the crossing as an arch.

Fig. 12.67 Inclined-Leg Frame

Fig. 12.68 Change of Cross-Section

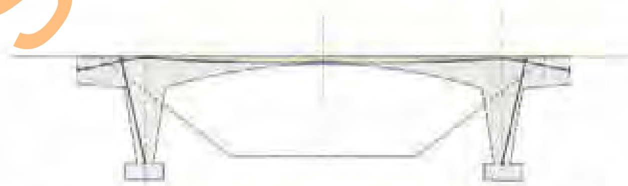


Fig. 12.69 Longitudinal Tendon Layout for Single-Span Frames

Fig. 12.70 Arch Bridge

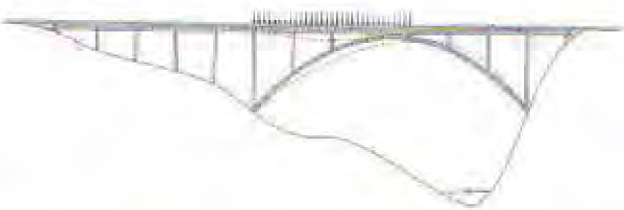


Fig. 12.71 Deflection of Arch Bridges Under Partial Live Load

Fig. 12.72 Cantilever Construction of Arch Bridges
Courtesy of Kiewit Pacific Company

12.7.2 Cross-Sections

Box girders, T-girders, and solid slabs can be used for frame bridges, depending on span length. The depth limitations proposed previously for these types of section apply. Solid sections should thus not be used for depths above 0.8 m, and boxes should not be used for depths less than about 1.8 m. It is possible to change from one type of section to another within the span. Fig. 12.68 shows a transition from a two-cell box to a three-web T-girder in a frame bridge.

12.7.3 Tendon Layouts and Details

A typical tendon layout for single-span frame bridges is shown in Fig. 12.69. For short-span bridges, it may be possible to eliminate the tendons in the legs and to use mild reinforcement instead. When the legs are post-tensioned, it is generally necessary to use dead-end anchors at the lower ends of the tendons.

Truss models should be used to investigate the flow of forces in the corners of frame bridges, and to proportion reinforcement accordingly.

12.8 DESIGN CONCEPTS FOR ARCH BRIDGES

Arches are efficient, visually expressive structural systems. Although few significant arch bridges have been built in recent years, they remain a viable option for long-span bridges crossing steep valleys. Although there is no specific range of spans in which arches can be said to be the most cost-effective option, there is no practical limit to their feasible range of spans. Arches spanning up to 390 m (1280 ft) have been built in concrete. Although arches do not normally need to be post-tensioned to resist loads in the completed structure, post-tensioning is often used to assist in their construction. In addition, the girder supporting the roadway of arch bridges is often post-tensioned as a conventional continuous girder. This section will deal only with those aspects of arch bridge design that relate directly to post-tensioning. A more complete discussion of concepts for arch bridges is provided in Ref. 12.1.

12.8.1 Longitudinal Structural System

Classical arch bridges consist of the arch proper, a girder that carries the roadway, and spandrel columns that carry load from the girder to the arch (Fig. 12.70). In arch bridges, the systems provided to carry dead load and live load are fundamentally different.

1. **Dead load** – The girder carries load essentially as a continuous beam supported on the spandrel columns, which can be assumed not to deflect in the vertical direction. The arch is generally laid out to coincide with the pressure line, i.e., the shape that allows it to carry load in a state of pure compression.
2. **Live load** – The critical cases are caused by live load applied to only part of the arch span. As shown in Fig. 12.71, when load is applied to only one half of the arch span, the arch and girder both deflect asymmetrically. Assuming that axial deformation in the spandrel columns can be neglected, it follows that the vertical deflections of the arch must be equivalent to the vertical deflections of the girder at matching locations. Partial live load thus induces bending in both arch and girder. We can define the total bending moment on the system M which must be shared between girder and arch, i.e.,

$$M = M_G + M_A$$

Moments in the girder and in the arch can be estimated by considering the relative bending stiffness of both components:[12.1]

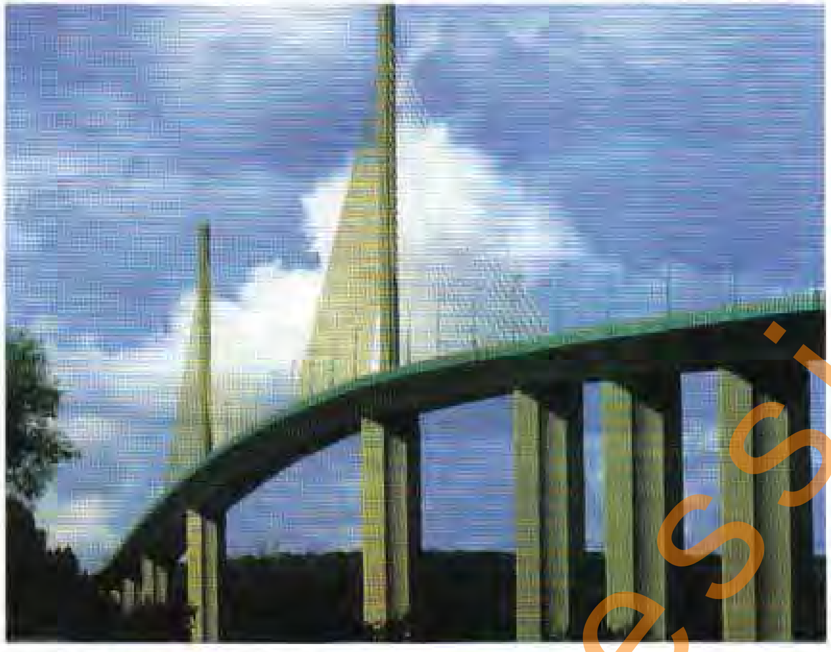


Fig. 12.73 Cable-Stayed Bridge[12.16]

$$M_G = M\frac{I_G}{I_G + I_A} \quad \text{and} \quad M_A = \frac{I_A}{I_G + I_A}$$

These moments must be considered in the calculation of total demand on both structural components.

12.8.2 Cross-Sections

The same options are available for cross-sections of the girder of arch bridges as exist for girders on conventional piers. Depending on the span between spandrel columns and the stiffness required to reduce bending moments in the arch, solid slabs, T-sections, and box girders can be used.

Solid slabs, twin solid ribs, and hollow boxes are common cross-sections for arches.

12.8.3 Methods of Construction

Modern arches are rarely built on falsework. Instead, they are built by cantilever construction using temporary post-tensioning, stays, and possibly towers as shown in Fig. 12.72. Once the arch has been closed at mid-span, the spandrel columns and girder are constructed. Although several options are possible for constructing the girder, a span-by-span method (either cast in place or precast) is generally most suitable. Girders for arch bridges have also been built by incremental launching.

12.8.4 Tendon Layouts and Details

Tendons in the girders are generally arranged in a similar manner to tendons in comparable continuous girders on conventional piers.

12.9 DESIGN CONCEPTS FOR OTHER TYPES OF BRIDGES

Post-tensioning has been used for other important types of bridges, including cable-stayed bridges and stress ribbon bridges. The complexity of both types of bridge makes it impossible to give a complete summary of the important issues governing the development of design concepts within the scope of this chapter. Rather, readers are referred here to references that can serve as an introduction to both types of bridge.

Cable-stayed bridges (Fig. 12.73) are the most cost-effective solution for long-span post-tensioned bridges. They combine high structural efficiency, relative ease of construction, and a bold visual aspect. The classic text by Gimsing[12.13] is a comprehensive reference on this type of bridge and is recommended as a starting point for designers. Chapter 13 of this Manual gives detailed guidance on the behavior and technology of cable stays as used in bridges.

Stress ribbon bridges (Fig. 12.74) are used primarily for pedestrian bridges. They are essentially suspension bridges with main cables stiffened by concrete panels that carry pedestrian traffic. This type of structure is highly efficient and can be used for relatively long spans, provided there is good subsurface material present for anchoring the main cables.

The book by Strasky[12.14] is a comprehensive reference on this type of bridge.

Fig. 12.74 Stress Ribbon Bridge
Courtesy of Charles M. Redfield, Consulting Engineer

12.10 SPECIAL APPLICATIONS OF POST-TENSIONING IN BRIDGES

Post-tensioning can also be used in applications other than the construction of new bridge superstructures. Two of the most important of these applications are precast concrete segmental bridge piers and precast concrete deck panels.

12.10.1 Precast Concrete Segmental Bridge Piers

Precast segmental construction offers a reduction of labor, elimination of falsework, and increase in speed of construction. These benefits, which were discussed in previous sections in the context of bridge superstructures, can also be extended to bridge piers.

A typical application is shown in Fig. 12.75. The geometry of the lowest segment must be carefully adjusted after it has been erected onto the foundation. After the alignment of the first segment has been completed, a concrete or grouted closure pour is usually provided between segment and foundation. The remaining segments can then be stacked up on top of each other. Temporary post-tensioning is usually provided to ensure stability of the pier and to guarantee a positive clamping force over the freshly epoxied joint faces.

Loop anchors are normally provided in the foundations, which allows strands to be pushed into one end of a duct from the top of the pier. The strands will descend to the foundation, curve around the loop anchor, and rise to the top of the matching duct. Both ends of this tendon can then be stressed from the top.

Given that the available plastic strain in prestressing steel is considerably less than that of conventional reinforcing steel, special measures are required when post-tensioned piers are used in regions of high seismic acceleration. The use of external, unbonded tendons has been proposed as an effective means of post-tensioning piers to ensure good seismic behavior.

12.10.2 Precast Concrete Deck Panels

The use of precast concrete deck panels can significantly increase the speed of deck slab construction, since the need to cure large expanses of concrete on site has been eliminated. Precast concrete deck panels can be used on a wide variety of supporting structures, including steel I-girders and trusses.

There is an increasing demand from the public for minimum impact construction. For deck slab replacements, this implies that work must be done only at night, with full traffic capacity restored during peak demand periods during the day. The use of precast concrete allows large sections of deck slab to be erected in short intervals of time. Reducing the use of cast-in-place concrete as a means of joining these panels is of critical importance.

The use of post-tensioning enables panels to be joined using small grouted or epoxy joints. The only steel continuous across the joint is prestressing steel. Fig. 12.76 illustrates one possible application of large precast concrete panels with post-tensioned joints.

Fig. 12.75 Erection of Segments for a Precast Segmental Concrete Bridge Pier

Fig. 12.76 Precast, Post-Tensioned Deck Panels for Rehabilitation
Courtesy of The Jacques-Cartier and Champlain Bridges Inc.

On projects that incorporate large precast concrete panels joined by post-tensioning, care must be taken to ensure that the panels are free to shorten during stressing operations, to ensure that the force applied to prestressing strands is actually pre-compressing the concrete and not leaking into other structural components. When precast post-tensioned concrete panels are made composite with a supporting steel structure, the loss of prestress in the concrete due to creep and shrinkage must be given careful consideration.

12.11 THE FUTURE

Post-tensioning will continue to be an important means of building and renewing bridges in the twenty-first century. The challenges that led to its rapid growth in the second half of the twentieth century remain current:

- Labor costs continue to rise in the industrialized world. As the living standard rises in other countries on the way to an industrialized economy, it will be necessary to find ways of minimizing the cost of labor even further.
- As the world becomes increasingly urbanized, and as the public's demand for minimum impact construction increases, the need to build quickly and unobtrusively will grow.

In addition, new challenges can be seen in the horizon, including the need to build in an environmentally sustainable manner.

It is anticipated that recent developments in materials technology, including high-strength concrete, ultra high-strength concrete, fiber reinforced concrete, and advanced composite materials will provide opportunities for using post-tensioning in new ways to address these challenges. It is likely that bridge construction will become more intensively mechanized, possibly through the development of lower cost and/or standardized systems for casting and erection of precast segments. This will allow the benefits of precast segmental construction to be extended to much shorter bridges and smaller projects.

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STAY CABLES

13.1 INTRODUCTION

Since the time of Verantius and his chain cables, the design of cable-stayed bridges has captured the imagination of bridge engineers around the world. Experiments with stayed bridges in the 1800s were triumphs of engineering, but they were short-lived. Chain stay designs revealed the efficiency of tensile members, but they failed the service demands of fatigue. Therefore, cable-stayed bridge design was largely dormant after the 1800s. It took the advent of wire rope technology to overcome the limitations of forged chains and to open up new opportunities for structural design.

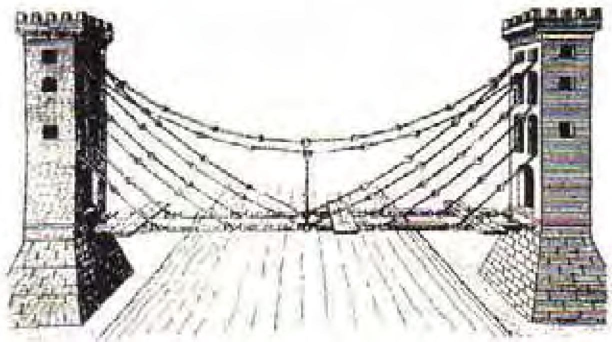

Fig. 13.1 Early Bridge With Chain Stays

Roebling was one of the first to use wire ropes for stays. He used stay cables to stiffen his famous Brooklyn Bridge over the East River in New York. This landmark structure, still in use after more than a hundred years, is one of the better known examples of successful early stay application.

Cable-stayed bridge design became popular in Europe, after World War II. It offered cost-effective solutions for rebuilding major river crossings destroyed during the war. Today, cable-stayed bridges are built for efficiency and also for aesthetics. The profile of a cable-stayed roadway ribbon, passed from shore to shore on the slender fingers of cable stays, has become a symbol of modern bridge engineering and architecture.

Cable-stayed bridges are cost effective because of the inherent material efficiency of the high strength stay tension elements and because cable-supported bridge decks can bridge long spans without intermediate piers. The normally shallow bridge decks structurally are "beams on elastic foundations" with the stays providing the supports.

Early designs used cable systems developed for steel suspension bridges, such as those made from wire rope, locked coil, and stranded bridge cables. These types of materials now are seldom used. They have been replaced by cables evolved from the post-tensioning technology for prestressing concrete. High-strength prestressing steel wires, strands, and bars are the prevalent materials for modern cable stays. Their anchoring as well as stressing features are adaptations of those used for post-tensioning purposes.

Modern stay cable systems are proprietary developments of the major post-tensioning system suppliers. Their basic components—wires, strands, bars, and anchoring devices—satisfy special stay cable performance requirements for fatigue resistance and environmental protection, which are more demanding than those for normal post-tensioning applications.

Fig. 13.2 Modern Cable Stayed Bridge
Courtesy of VSL Mexico

This chapter provides basic information on cable stays. It is not intended as a comprehensive design resource. For design requirements and additional information see the PTI *Recommendations for Stay Cable Design, Testing and Installation.*^-[1]

13.2 ENGINEEREVG OF STAY CABLE STRUCTURES

13.2.1 Design Elements and Responsibility

The design of cable stays covers five essential elements.

1. Design of cable stays for static loads and fatigue
2. Design against stay vibration
3. Design of anchorage details
4. Design of corrosión protection features
5. Design of cable erection procedure

The design responsibility for elements 1 to 4 is typically shared between the bridge Design Engineer and the stay supplier. The responsibility for element 5 is normally also shared with the contractor.

The licensed design professional is normally expected to perform and be responsible for all design-related aspects of the overall structure. Included are such stay-related Ítems as:

- Specifying design and performance requirements for the cables which are applicable for the particular structure; often the requirements follow the PTI *Recommendations for Stay Cable Design, Testing and Installation*[13A]
- Detailing cable arrangement and basic anchorage provisions; this includes such constructability considerations as space requirements for anchorages, installation and stressing equipment; it also includes structural feasibility to remove and replace cables if this is a design objective
- Determining cable sizes and forces
- Specifying anchorage placement and assembly tolerances
- Design of connection details to support the stay anchorage and to

ensure the force transfer from the anchorage into the main structure; for steel structures this includes the design of force transfer members into the main structure; for concrete structures this includes the design of confinement and bursting reinforcement in the anchorage zone

The *supplier* of stay cables is typically responsible for the cable hardware, including:

- Design and testing of the cable system and its components to meet the design and performance requirements of the contract documents
- Quality control and quality assurance of hardware components
- Furnishing to the licensed design professional detailed shop drawings for all cable components, testing records and quality control documentation
- Cable hardware performance in accordance with the contract documents

The *contractor* normally carnes the prime responsibility for the cable installation. However, cable installation and stressing require careful planning and monitoring, and involve all parties. The contractor may subcontract the cable installation to a specialty contractor, who often is also the supplier.

13.2.2 Construction Engineering for Cable-Stayed Structures

Stay cable erection engineering for construction stages and associated geometry control is an essential task for successful construction. The requirements for an engineered erection program are described in the PTI Recommendations.[13'] The basic elements of an engineered construction program include:

- Establishing permissible construction equipment loads
- Developing a system of temporary works for installation of stays and deck elements
- Developing an erection cycle for girder and stays which controls strength requirements for intermedíate construction stages
- Developing a program for stay erection which ensures that the completed structure achieves proper deck profile and acceptable cable forces
- Developing procedures to adjust cable forces if necessary to obtain the correct geometry profile of the bridge deck at the end of construction

13.3 STAY CABLE DESIGN

13.3.1 Design Methodology

The Third Edition of the PTI *Recommendations for Stay Cable Design, Testing and Installation*[^2] uses the conventional allowable stress design approach for designing stay cables. Fatigue was considered by providing allowable stress ranges for various numbers of load cycles. Section 13.3.2 outlines the allowable stress requirements.

The Fourth Edition of the PTI *Recommendations for Stay Cable Design, Testing and Installation*[nA] is based on the *AASHTO LRFD Bridge Design Specification.*[13A] It uses the strength design methodology and applies it to the design of stay cables. The limit states for strength and fatigue were calibrated to correspond to the earlier allowable stress design approach. Section 13.3.3 outlines the LRFD requirements.

13.3.2 Allowable Stress Design

The allowable stress design method is useful for preliminary designs and for applications not subjected to the AASHTO LRFD specifications requirements. The allowable stresses given below are those from Third Edition of the PTI Recommendations:[13.2]

1. 0.45 f'_s for AASHTO Group I loading (including any permanent locked-in erection loads) upon completion of construction with no consideration of future superimposed dead load (wearing surface) and without considering creep and shrinkage effects.
2. 0.45 f'_s for AASHTO Group I loading (including any permanent "locked-in" erection loads) plus future superimposed dead load and effects of creep and shrinkage.
3. 0.50 f'_s for all other AASHTO Group loadings (to be used in lieu of AASHTO allowable over-stress factors).
4. 0.56 f'_s during construction or cable-stay replacement.
5. Allowable fatigue stress ranges are given in Section 3 of the Third Edition of the PTI Recommendations.[13.2]
6. Flexural stresses in excess of 20.7 MPa (3 ksi) shall be added to the axial stresses due to live load plus impact.

13.3.3 LRFD Design

13.3.3.1 General LRFD Information

The LRFD methodology is expressed by the basic Eq. (13.1), which applies to all limit states, including limit states for:

- Service
- Strength
- Fatigue and Fracture
- Extreme Conditions

$$\eta\sum(\gamma_i Q_i) \leq \phi R_n = R_r \qquad (13.1)$$

Where:

Q_i = force effect of direct axial plus bending for load case i

R_n = nominal resistance (strength)

R_r = factored resistance (strength)

ϕ = resistance factor (material strength reduction factor)

γ = load factor

η = modification factor

The strength and fatigue limit states for stay cables are discussed in Sections 13.3.3.2 and 13.3.3.3. The PTI[13.1] test requires that a stay cable withstand 2 million stress cycles with an upper stress of 45% f'_s and stress ranges of:

strand 8.5% f'_s for f'_s = 1860 MPa

wire 11.7% f'_s for f'_s = 1655 MPa

bar 10% f'_s for f'_s = 1035 MPa

After the fatigue test a cable must support at least 92% of AUTS, but it should not be not less than 95% MUTS. For more information see Refs.13.1 and 13.3.

Where:

AUTS = actual ultimate tensile strength measured as force

MUTS = minimum ultimate tensile strength measured as force ($A_s \times f'_s$)

13.3.3.2 Strength Limit State

The strength limit state shall ensure that a structure, its members, and components have the strength and stability to resist the specified, as defined by AASHTO, load combinations expected during their design life.

The PTI[13.1] recommends for use in Eq. (13.1) the following values for factors γ, ϕ, and η:

Typical Load Factors:

Structural Dead Load γ = 1.25

Live Load γ = 1.75

The AASHTO LRFD Specification[13.4] provides more information on load factors.

Strength reduction factors recommended by PTI:[13.1]

Strength	$\phi = 0.65$
Fatigue	$\phi = 0.95$
Replacement	$\phi = 0.8$
Loss of cable	$\phi = 0.9$
Extreme events	$\phi = 0.95$

Modification factor:

$$\eta = \eta_D \times \eta_R \times \eta_I$$

AASHTO requires $\eta \geq 0.95$. The values specified for ductility (η_D), redundancy (η_R), and operational significance (η_I) vary between 0.95 for ductile and redundant members to $\eta = 1.05$ for non redundant and non ductile members; for fatigue $\eta = 1.0$. See AASHTO[13.4] for additional information.

Combined force effect:

The values for "Q" in Eq. (13.1) include the combined effects of axial forces and bending. For the static strength limit state the $\phi = 0.65$ factor applies. For establishing fatigue limits it is appropriate to superimpose bending fiber stresses with axial stresses (see Section 13.3.4.2).

Bending stress-reducing features are part of proprietary cable stay systems. Neoprene collars, typically placed where the cable exits the support structure, function as flexible supports and reduce critical bending stresses at the anchorage locations.

13.3.3.3 Fatigue Limit State

The fatigue limit is the constant stress range below which fatigue life is assumed to be infinite. It is conventionally taken as the stress range—at a specified upper stress—which is sustained for two million cycles.

The Wöhler curve and the Smith diagram graphically define the fatigue properties of specific materials. Figs. 13.3 and 13.4 show typical curves.

Such curves are useful to evaluate the suitability of a particular material for fatigue-sensitive applications. It should be noted that the fatigue strength of wires, strands, and bars (all conforming to applicable ASTM specifications) may vary considerably, depending on the source of the material. Therefore, proper material selection is important.

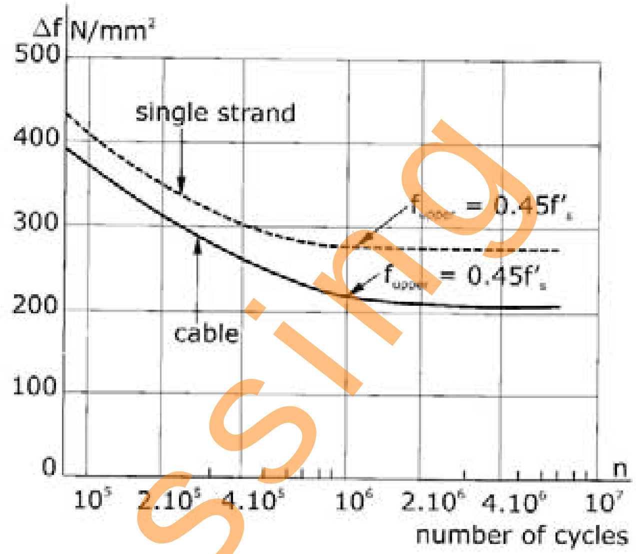

Fig. 13.3 Wöhler Curve for Typical 15mm Strand
Courtesy of Freyssinet International

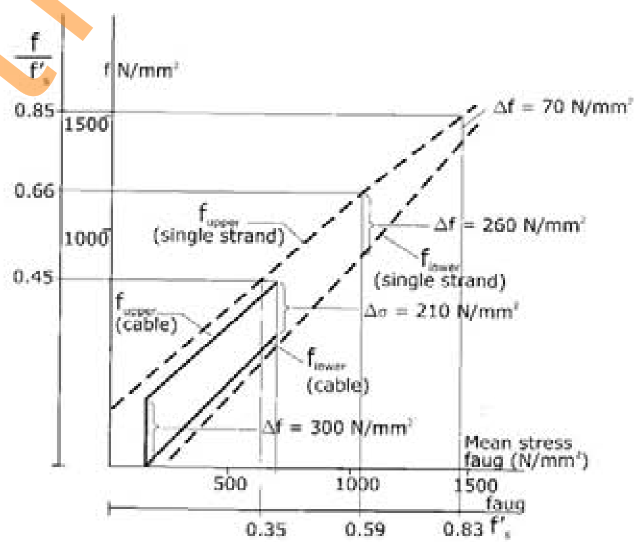

Fig. 13.4 Smith Diagram for Typical 15mm Strand and Stay Cable
Courtesy of Freyssinet International

The Fourth Edition of the PTI *Recommendations for Stay Cable Design, Testing and Installation*[13.1] provides the following equations for determining fatigue limit state stress ranges. The equations are based on a variety of individual strand fatigue tests and on full size stay cable tests. The full size fatigue tests were performed with an upper stress of $0.45f'_s$ and for different types of anchorages. Fatigue stress range values for assembled cables are smaller than for individual strands because of group effects:

$$\gamma(\Delta f) \leq \phi(\Delta f)_n \qquad (13.2)$$

$$G = \frac{1}{2}(\Delta f)_{TH} \qquad (13.3)$$

$$B = \left(\frac{A}{N}\right)^{1/3} \qquad (13.4)$$

$$\text{if} \quad \gamma(\Delta f) \leq G \quad \text{then } (\Delta f)_n = G \qquad (13.5)$$

$$\text{else if} \quad \gamma(\Delta f) \leq B \quad \text{then } (\Delta f)_n = B \qquad (13.6)$$

$$\text{if} \quad (\Delta f)_n \neq G \text{ or } B \text{ then } \gamma(\Delta f) = \text{too high} \qquad (13.7)$$

With: (in consistent metric units)

- ϕ = 1
- γ = 0.75
- A = from Table 13.1
- N = 365 days × 75 years × 1 cycle × ADTT
- ADTT = average daily truck traffic in one direction in one lane
- $(\Delta f)_{TH}$ = from Table 13.1

Where:

- Δf = stress range due to passage of the fatigue load
- $(\Delta f)_n$ = nominal fatigue resistance
- $(\Delta f)_{TH}$ = constant amplitude fatigue threshold

Table 13.1 - Fatigue Threshold Guide Values

	$A \times 10^{10}$ [(MPa)^3]	$(\Delta f)_{TH}$ [(MPa)^3]
Parallel strands	39.3	110
Parallel wires	74	145
Uncoupled bars	39.3	110
Epoxy coupled bars	7.2	48

13.3.4 Cable Forces

Stay cables are subject to axial tension over the length of the cable. The axial cable forces and stresses are calculated assuming the use of pin joints, which allow unrestricted rotations. In reality, however, cable anchorages are normally fixed against rotation, which induces bending moments at cable ends. The bending stresses superimpose on the axial stresses and affect the fatigue resistance of the cable.

Cable bending moments are caused by variations in axial cable forces due to load and temperature changes. Resulting length changes affect the cable sag and the angle at which the cable enters the fixed anchorage. Likewise, deflections of the structure to which a cable is attached produce bending stresses in the cable. Bending stresses also result from cable oscillations due to wind and other dynamic causes.

13.3.4.1 Cable Tension

In practice cable forces can be determined by statics for an equivalent straight tension member along the chord. Because the cable sag reduces the longitudinal cable stiffness, an equivalent elastic modulus, first proposed by Ernst, is used to model actual cable stiffness:

$$E_{eq} = \frac{E}{\left[1 + \frac{(\gamma \ell)^2}{12 f_{axial}^3} E\right]} \qquad (13.8)$$

Where: (in consistent dimensional units, metric or U.S.)

- E = modulus of elasticity of cable material ($E_{strand} = 195 \times 10^3$ N/mm^2)
- E_{eq} = equivalent modulus of elasticity for a straight chord member
- ℓ = horizontal projection of cable span (mm)
- f_{axial} = axial unit stress in cable (N/mm^2)
- γ = specific weight of cable per unit volume ($\gamma_{steel} = 87 \times 10^{-6}$ N/mm^3)

With forces along the chord known from statics, the actual cable forces (F) at the anchorages can be determined. The equations below are based on simple parabolic geometry and force relationships.

$$f = \frac{w\ell^2}{8F} \qquad (13.9)$$

$$H = \frac{w\ell^2}{8f_v} \quad \text{with} \quad f_v = \frac{f}{\cos\theta} \qquad (13.10)$$

$$V_A = H\tan\theta + \frac{w\ell}{2} \qquad (13.11)$$

$$V_B = H\tan\theta - \frac{w\ell}{2} \quad \downarrow \tag{13.12}$$

$$F = \sqrt{H^2 + V^2} \tag{13.13}$$

$$\tan\theta = \frac{h}{\ell} \tag{13.14}$$

$$\tan\beta = \frac{V}{H} \tag{13.15}$$

$$\alpha = \beta - \theta \tag{13.16}$$

Length and length variations can be determined from the following equations:

$$s \cong \bar{s} + \frac{8f^2}{3\bar{S}} \tag{13.17}$$

$$\Delta s_{\Delta t} \cong s\,\varepsilon\,\Delta t \tag{13.18}$$

$$\Delta s_{el} \cong \frac{H}{EA}\,\bar{s}\left[1 + \frac{16}{3}\left(\frac{f}{\bar{s}}\right)^2\right] \tag{13.19}$$

$$y = \frac{4f_y}{\ell^2}\left(\ell x - x^2\right) \tag{13.20}$$

Where: (in consistent dimensional units, metric or U.S.)

A = cross-sectional area of cable
E = modulus of elasticity of cable material
F = cable force, tangential at anchorages [subscripts (A) and (B) indicate locations]
$\bar{F}$ = force along chord
H = horizontal cable reaction force
ℓ = horizontal cable projection
s = cable length
$\bar{S}$ = length of chord
Δs_{el} = elastic elongation of cable under cable force
$\Delta s_{\Delta t}$ = cable elongation due to temperature change
V = vertical cable reaction [subscripts (A) and (B) indicate location]
f = cable sag at mid-span normal to chord
f_{axial} = axial tensile unit stress in cable
f_y = vertical cable sag at mid-span
h = vertical cable projection
w = uniform weight per unit length along horizontal span length
x = variable horizontal distance from anchorage
y = vertical dimension from chord to cable at distance x from anchorage
α = angle between chord and cable tangent at anchorage
β = angle between horizontal and cable tangent at anchorage
θ = angle between horizontal and chord
ε = thermal expansion coefficient for cable material

Above equations are derived for a parabolic cable. Catenary equations may also be used, but the two are essentially the same above 10% MUTS.

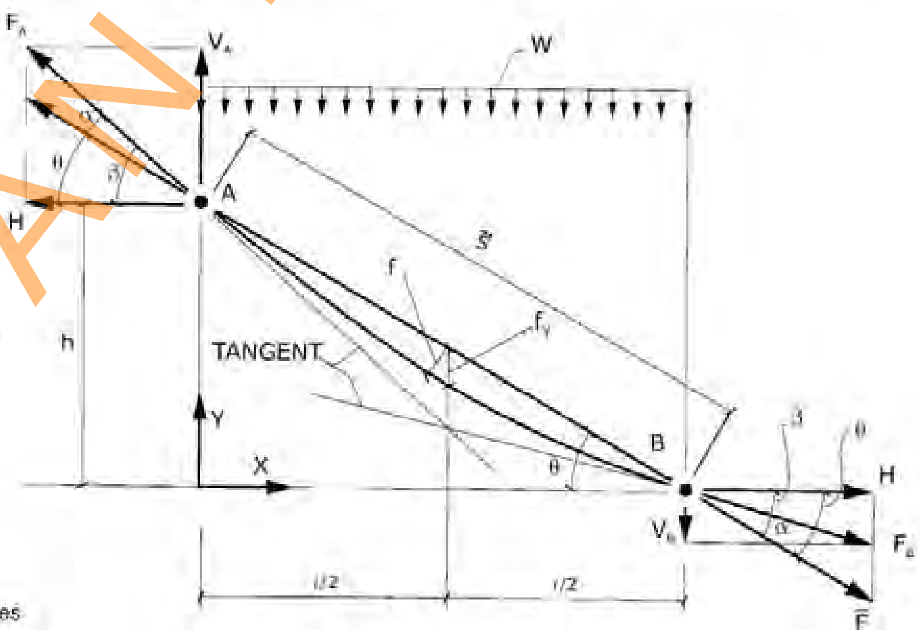

Fig. 13.5 Cable Forces

13.3.4.2 Cable Bending

Bending moments in cables depend on the angle α between the fixed anchorage and cable tangent at that point. The angle varies with cable length, cable dead weight, cable force, and deflection of supporting structure under load changes. Angles between 0.001 and 0.01 radians are common. Cycling of cable forces also produces cycling of angle change.

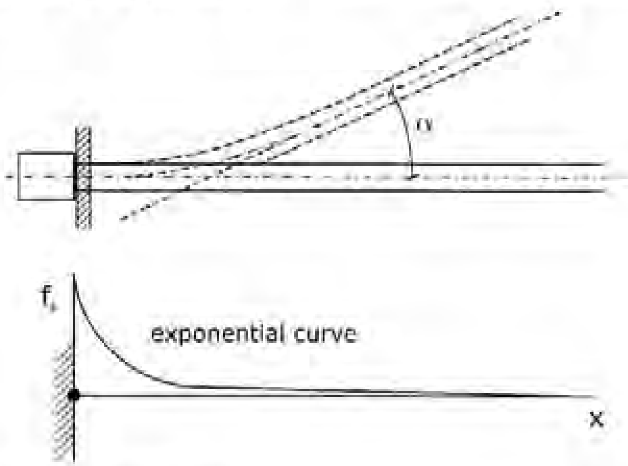

Fig. 13.6 Cable Bending
Courtesy of Freyssinet International

The bending moment at a section (x) is given by the equation:

$$M = \alpha e^{-kx}\sqrt{EIF}$$

$$\text{with} \quad k = \sqrt{\frac{F}{EI}} \tag{13.21}$$

The bending moment has its maximum value at the anchorage (x = 0) and reduces exponentially with increasing x.

For a circular cross-section the bending moment causes flexural fiber stresses of:

$$f_M = 2\alpha\sqrt{E f_{axial}}\, e^{-kx} \tag{13.22}$$

The maximum flexural stress at the anchorage is:

$$f_{Mmax} = 2\alpha\sqrt{E f_{axial}} \tag{13.23}$$

Where: (in consistent dimensional units, metric or U.S.)

- E = modulus of elasticity of cable material
- F = cable force, tangential at anchorages
- I = moment of inertia of cable or tension element
- f_M = unit fiber stress from bending moment
- f_{axial} = unit stress in strand from axial cable force
- x = horizontal distance from anchorage
- α = angle between tangent and chord in radians

It is interesting to note that for a circular cable and for a given axial cable stress the bending stress is only a function of the angle α and is independent of the cable diameter. However, there is a difference between grouted cables and those that are left ungrouted.

If a cable is grouted, uncoated tension elements are bonded together and the cable acts as a unit. In that case the cable is relatively stiff, the curvature (ρ = 1/r) is small, and the bending moment is large. Such cable bends as a unit around its neutral axis and maximum bending stresses occur only in the tensile elements farthest away from the neutral axis, in effect increasing their axial stress component.

If the cable is left ungrouted, on the other hand, the cable is relatively flexible, the curvature (ρ = 1/r) is larger, and the bending moment smaller. However, each tensile element bends about its own axis and each tensile element has identical fiber stresses without changing their axial stress components.

13.3.5 Stay Vibrations

Stay cables are sensitive to wind-induced vibrations, which become more critical as cable length increases. Longer cables gain less from the intrinsic damping afforded by the anchorage details and from mass damping.

Cable vibrations are complex phenomena. Analyses and wind tunnel tests are helpful tools for studying vibration susceptibility of cables and for designing vibration-suppressing measures. But it is not entirely possible to accurately predict and model the multitude of random variables that in certain combinations may cause vibrations. It is therefore prudent to anticipate that some corrective measures may be required during erection or after completion of construction.

Cable vibrations can be separated into several basic types. To identify them can be difficult since several types may occur simultaneously.

13.3.5.1 Natural Frequency of Cables Under Tension

The natural frequency of cables depends on the cable tension, length, mass per unit length, and on the vibration mode. Mode shapes have been observed to abruptly change back and forth because of small changes in wind velocity and direction.

From the idealized Eq. (13.24) it can be seen that the natural frequencies increase linearly with the mode. Placing stiffening ropes at mid-span or one-third points doubles or triples the natural frequency, which is an effective way to suppress cable vibrations.

$$N_n = \frac{n}{2\ell}\sqrt{\frac{f}{m}} \qquad (13.24)$$

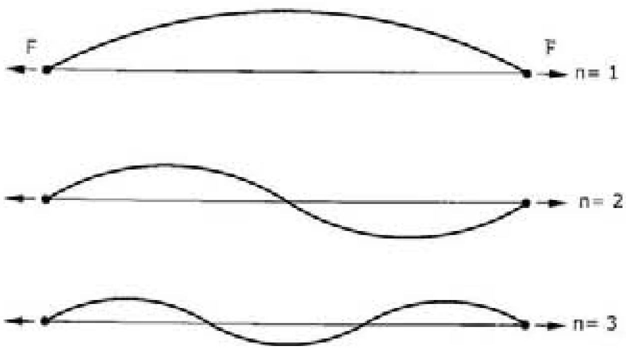

Fig. 13.7 Cable Vibration Modes

Where: (in consistent dimensional units, metric or U.S.)

- ℓ = cable span length (m)
- N = natural frequency (Hz, s^{-1})
- $\bar{F}$ = cable tension along chord (N, kg.m/s^2)
- m = cable mass per unit length (kg/m)
- n = vibration mode (1, 2, 3, ... n)

13.3.5.2 Wind and Rain Induced Resonance

Cable vibrations can be excited by such dynamic wind effects as:

- air flow turbulence (buffeting)
- vortex shedding in wake behind the cable
- fluid elastic interaction between neighboring cables (wake galloping)
- interaction between rain, wind, and cable
- dynamic forces acting on other parts of a structure, e.g. pylon or bridge deck

Forces due to vortex shedding are independent of the cable motion and will not lead to aero-elastic instability but only to vibrations of limited amplitude. On the other hand, the dynamic forces acting during self-excited base structure vibrations are a feedback of the motion of the cable itself. The ensuing aero-elastic instability leads to vibrations of large amplitudes. Such vibrations start suddenly when the wind speed reaches a certain critical value.

For rain or wind induced vibrations of circular cables the following tentative limit for the mass damping parameter has been proposed:[13.1]

$$\left(\frac{m\xi}{\rho d^2}\right) \geq 10 \qquad (13.25)$$

where: (in consistent dimensional units, metric or U.S.)

- m = cable mass per unit length (kg/m)
- ξ = damping ratio-to-critical. Values between 0.0005 and 0.005 have been measured
- ρ = air density (kg/m^3)
- d = cable diameter (m)

13.3.5.3 Cable Galloping

Galloping-type instability on some cable-stayed bridges has been observed. Such wind-induced oscillations can occur under certain wind velocities when the circular inclined stay cables present an elliptical profile to the air flow. Large amplitude vibrations of similar magnitude also can be caused by wind-driven periodic deformations of the cable-supported structure (Section 13.3.5.4).

$$v_{crit} = CNd\sqrt{\frac{m\xi}{\rho d^2}} \qquad (13.26)$$

Eq. (13.26) is an attempt to analytically predict the critical wind velocity (v_{crit}) that causes galloping of a single cable or a group of cables:[13.1]

Where:

- C = constant, for circular cables a value of 40 is recommended;[13.1]

other notations as in Sections 13.3.5.1 and 13.3.5.2

13.3.5.4 Base-Excited Vibrations

Periodic deformations of structures—pylons or bridge decks—may be caused by vortex shedding, buffeting, or flutter. Periodic archorage vibrations may excite stay cable motions, even if the periodic deformations of the structure remain small.

13.3.5.5 Vibration-Suppressing Measures

A common defense against cable stay vibrations is the use of stiffening ropes. Such ropes are placed normal to the basic stay system, tying stays to each other or to the deck, (see Fig. 13.8-10).

Hydraulic dampers and other proprietary visco-elastic damping devices normally are effective against cable vibrations, (see Fig. 13.11). Also, neoprene or visco-elastic collars at anchorages act as dampers, and simultaneously reduce bending stresses in the cables.

Helixes or other surface contours on the cable sheathing disrupt airflow and the formation of rain rivulets. They can be effective in controlling rain or wind induced vibrations (see Fig. 13.12).

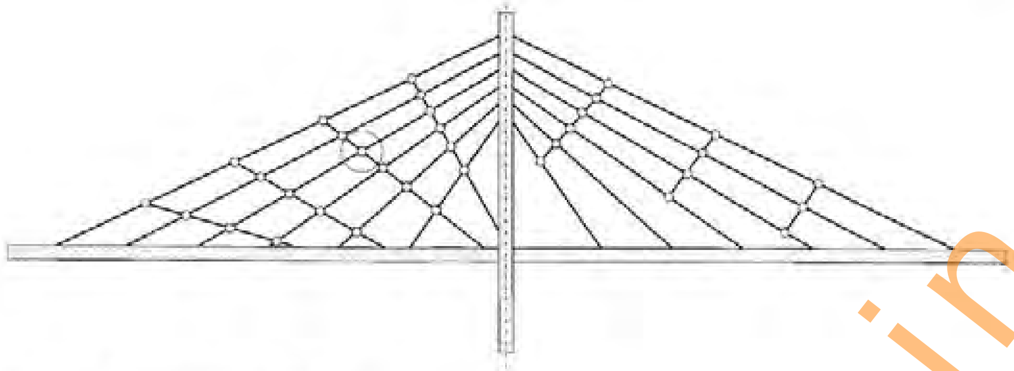

Fig. 13.8 Stiffening Ropes, Arrangement Options
Courtesy of DYWIDAG-Systems International, USA

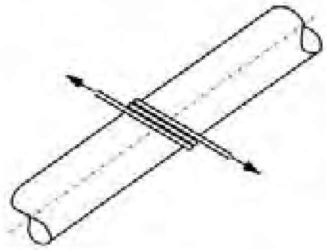

Fig. 13.9 Temporary Stiffening Rope

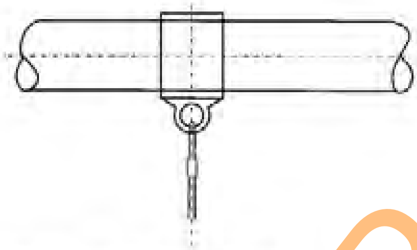

Fig. 13.10 Permanent Stiffening Ropes

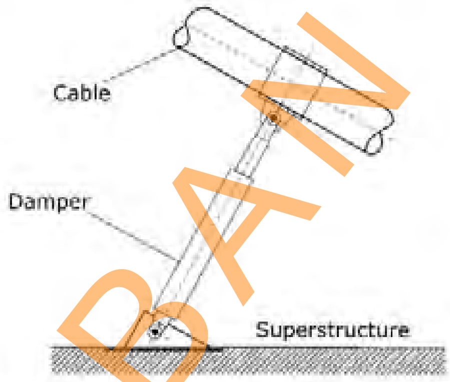

Fig. 13.11 Shock Absorber
Courtesy of DYWIDAG-Systems International, USA

Fig. 13.12 Surface Contours on Cable Sheathing, Tagun Bridge, Portugal
Courtesy of Freyssinet International

13.4 MATERIALS FOR STAY CABLES

Strand, wire, and bar materials used in contemporary stays are similar to those used in post-tensioned construction. However, more severe exposure to fatigue and corrosion places additional requirements on material production and quality control.

This section provides general information on stay cable materials. For additional information see the PTI *Recommendations for Stay Cable Design, Testing and Installation*,[13.1] which sets the standards for stay cable material requirements. The PTI publication also includes information on corrosion inhibiting compounds, stay pipe material, and strand sheathing materials. Supplier's cable stay brochures provide additional information.

13.4.1 Stay Cable Materials

13.4.1.1 ASTM A421 Wire Cables

Wires used for stay cables conform to the ASTM A421 *Standard Specification for Uncoated Stress-Relieved Wire for Prestressed Concrete*. Wire diameters between 6.35 (0.250 in.) and 7.01 mm (0.276 in.) are normally used. Their nominal ultimate tensile strength varies between 1655 MPa (240 ksi) to 1620 MPa (235 ksi) respectively.

Presently only a few manufacturers produce ASTM A421 wires. U.S. manufacturers have stopped making this material because it has been replaced by strands for both post-tensioning and stay cable applications in the United States.

The BBR wire cables typically consist of 7 mm wires with a nominal tensile strength of 1670 MPa (242 ksi). Cables with 19 to 421 wires are available.

Wire cables are normally preassembled into compact, parallel wire cables, which have good fatigue properties because side pressures and fretting between crossing wires are avoided.

The individual wires are normally button-headed and attached to a template plate, which bears against the filler material in the bond socket. The bond socket is filled with a special BBR-HIAM compound, a mixture of steel balls, epoxy resin, and zinc dust (see Fig. 13.14). This anchorage has superior fatigue properties because it avoids stress raisers in the wires.

13.4.1.2 ASTM A416 Strand Cables

Strands conforming to ASTM A416 *Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete* are the most competitive material for typical highway bridge stay cables. Most stay cable systems use grade 1860 MPa (270 ksi) strand with a diameter of 15.2 mm (0.6 in.), but other strand sizes can also be used. Cables with up to 127 strands are available as standard products.

For anchoring purposes, strands are attached with wedges to a wedge plate, which is part of an anchorage assembly similar to those used for post-tensioning tendons. Typically, the strand-wedge connection is designed to develop at least 95% of the actual ultimate strength of the strand.

Special fatigue considerations for stays require that suitable strand must, in addition to the ASTM A416 requirements, also pass the one-pin test requirement. The assembled cable must also pass the full size fatigue test requirements of the PTI Recommendations.[13.1] The PTI *Acceptance Standards for Post-Tensioning System*[13.3] provides additional information on strand-wedge interaction and fatigue testing.

13.4.1.2.1 Sheathed and Coated Strand Cables

Some cable stay systems use sheathed and coated ASTM A416 strands, which are similar to those used for unbonded single-strand post-tensioning tendons.

The plastic sheath is seamlessly extruded over the coated strand. It consists of either high-density polyethylene (HDPE) or polypropylene. An additional variation is how tightly the sheathing is extruded over the coated strand. In some systems the plastic sheath allows the strand to move freely during stressing, as in conventional unbonded post-tensioned construction. More common is a tightly extruded sheath that will not allow the strand to move inside the sheathing, which is desirable for installation purposes.

The coating material applied to the strand during the extrusion process is either a special corrosion-inhibiting, petroleum-based or wax-based compound. Moisture can travel or condense in the interstices between inner and outer strand wires. It is, therefore, desirable to have all interstices between the 7-strand wires completely filled. This can be accomplished by a special coating procedure in which the wire lay is temporarily opened up, allowing the coating material to enter the space between the wires.

13.4.1.2.2 Galvanized Strand Cables

Galvanized strands are used in some countries for stay cable bridges for enhanced corrosion protection. Galvanized strand has not been used for major cable stay bridges in the United States because there are no ASTM specifications for galvanized post-tensioning strand and because domestic manufacturers presently do not produce a suitable material.

13.4.1.2.3 Epoxy Coated Strand Cables

Epoxy coated strand conforming to ASTM A882 *Standard Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand* is a proprietary product of Insteel Industries, Inc. and its licensees. The material essentially is ASTM A416 strand with an epoxy corrosion barrier applied to it. The type suitable for cable stay application has interstices between inner and outer strand wires that are filled with epoxy. Tight tolerances on epoxy thickness must be maintained since epoxy thickness is critical for proper wedge function.

The epoxy initially participates in carrying stresses, which over time creep out and transfer to the steel strand. This increases steel strains in cable stays with essentially constant forces. The strains can be assumed to amount to about 0.025% of the cable length over the life of the structure. Post-tensioning cables with fixed lengths, on the other hand, would experience relaxation values of up to 2.5 times that of normal low-relaxation strands.[13-1]

To anchor epoxy coated strand, special wedges are required that bite through the epoxy into the strand. During installation, wedge seating is critical and requires special equipment and procedures to ensure proper seating and to avoid damaging the corrosion protective epoxy coating.

Epoxy coated strand has been used for bond socket anchorages (Fig. 13.16). For such applications silicon sand is applied to the epoxy surface during the manufacturing process. The sand provides improved bond properties.

13.4.1.3 ASTM A722 High-Strength Bar Cables

High-strength bars conforming to ASTM A722 *Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete* are primarily used for single bar stays of small bridges and other stayed structures. Multiple bar stays, however, have also been used for major stay applications.

Single bar stays are well suited for smaller stay forces. The bars normally are anchored with threaded anchor nuts, which develop the strength of the bars. At non-stressed anchorages, bars may be directly threaded into the bearing plate if bending stresses can be avoided.

As for strand and wire cables, bar anchorage details must provide reliable corrosion protection. Single bar stays can be pregrouted or grouted in place. As for other systems, end details must allow movements during stressing without compromising reliable corrosion protection.

Single bar stays have installation advantages because of their relatively small size and stiffness. Bars come in lengths up to 18 meters (60 ft). Longer stays require couplers, which are epoxied to prevent accidental unscrewing during installation. The epoxy also reduces stress raisers in the threaded connection and improves fatigue resistance. Tapered couplers further improve fatigue performance.

13.4.2 Stay Cable Pipe

Stay cables typically are placed inside steel or plastic pipe for corrosion protection purposes. High-density polyethylene (HDPE) pipe is more prevalent due to its light weight and corrosion protection properties. Typically HDPE joints are fused to provide reliable seamless enclosures.

Half shells with longitudinal sliding quick joints also have been used. They can be placed over the erected cable, which may be useful for large and long cables or for repair work. Fig. 13.13.

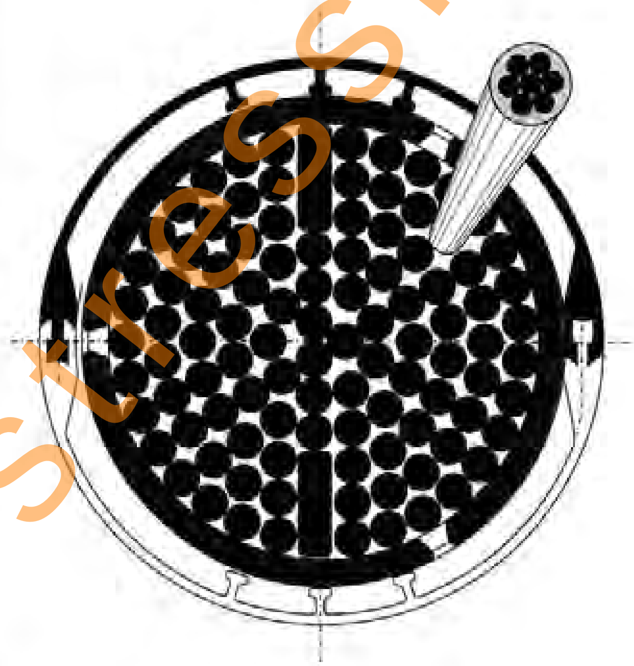

Fig. 13.13 Half Shell Cable Pipe

HDPE must be UV stabilized. Black HDPE with an addition of 2% carbon black is the conventional material. Such pipes have been wrapped with colored PVF (polyvinyl fluoride) tape for temperature control and additional protection. Colored UV stabilized HDPE is now on the market. However, some colors, such as shiny red, are not suitable for long-term UV exposure. White and colored coextruded pipe also have been manufactured. The outer 1.5 to 2 mm material can then be any color, while the inner portion consists of black HDPE. (See index for additional information on plastics.)

Cable pipe must have provisions for expansion and contraction under temperature change and for cable adjustment during construction. This is especially important for HDPE pipe, which has a thermal expansion coefficient about seven times that of steel or grout. In addition, the thrust from the pipe dead weight must be supported, which is normally done from the upper anchor to avoid duct buckling.

In the United States, past stay cable installations had the space between the cable and the pipe filled with cement grout similar to that used in post-tensioning applications. (For corrosion protection see Section 13.4.4.) The grout mass improves the mass damping of modest-length stays, but this advantage is not significant for longer stays. Current practice is tending toward ungrouted stays.

13.4.3 Stay Cable Anchorages

13.4.3.1 Anchorage Design Requirements

Anchorages for stay cables typically are proprietary developments of cable suppliers, and therefore are normally not fully detailed on contract drawings. The design drawings, however, must provide adequate space for anchorages and must provide access for cable installation and jacking equipment. Suppliers provide pertinent design information in their product literature. In addition, at the time of shop drawing approval, the engineer obtains from the suppliers detailed shop drawings and test information for review.

Design, detailing and installation of anchorages and their components are critical for cable performance and for the secure transfer of cable forces into the structure. The cables are inherently redundant—they consist of many tension elements, which are uni-axially stressed and are relatively ductile—their anchorages do not have such redundancy built in. Anchorages, therefore, require adequate strength reserves at cable ultimate tensile strength to compensate for the lack of redundancy and more complex stresses.

The full sized cable test required by the PTI *Recommendations for Stay Cable Design, Testing and Installation*[13.1] provides assurance that the anchorage can at least sustain 92% of AUTS (actual ultimate tensile strength, measured as force). The PTI *Acceptance Standards for Post-Tensioning Systems*[13.3] provides additional information on component requirements, which are also applicable to cable stay anchorages.

The design of anchorages covers the following essential topics:

- force transfer into the structure
- method of anchoring the tension elements for static and fatigue forces
- provisions to limit flexural stresses in the tensioning elements at the anchorage to acceptable values
- vibration damping provisions
- corrosion protection features
- installation features
- tensioning and retensioning methods
- replacement provisions

Suppliers solve the above outlined design elements in different ways. However, most systems have the basic parts described as follows.

13.4.3.1.1 Anchor Heads

Anchor heads transfer forces from the individual tension elements (wires, strands, bars) into the bearing plates. For a wire cable the anchor head consists either of a bond socket as shown in Fig. 13.14, or of a thick high-strength steel disk with closely spaced holes for anchoring wires with button heads. For a strand cable the anchor head is equivalent to the wedge plate of a post-tensioning tendon anchorage. For single bar stays the equivalent to the anchor head is the anchor nut, which normally has a spherical bearing surface to limit bending moments.

13.4.3.1.2 Ring Nuts

Ring nuts normally are provided at stressing end anchorages. The ring nut permits fine adjustment of stay length and forces. For a strand anchorage the ring nut permits fine adjustments without unseating and reseating wedges. The same function can be performed by shim packs under the anchor head or a combination of both ring nut and shims. The bar anchor nut also performs the ring nut function.

13.4.3.1.3 Bearing Plates

Bearing plates transfer the stay forces into the supporting structures. The design of bearing plates for concrete applications is governed by the design requirements for post-tensioning tendon bearing plates as specified by AASHTO[13.4] or by PTI.[13.3] Confinement reinforcement in front of the bearing plate is normally necessary. Care must be taken that adequate access for placing and consolidation of concrete is available and not restricted by reinforcing bar congestion. See PTI Publication *Anchorage Zone Design*.[13.5]

13.4.3.1.4 Transition Tubes

Transition tubes allow the tension elements to flare from a tight cable bundle to a wider spacing required for anchoring purposes. The transition tube encloses the flared cable bundle and provides a corrosion protective enclosure. The transition tube serves no structural function unless it is designed as a bond socket.

13.4.3.1.5 Deviators

Deviators of different designs are used to resist lateral forces due to angle changes of the tension elements at the front end of the transition tube. Their design and placement is governed by fatigue considerations. Details, which reduce fatigue effects, are important elements in the qualification testing of stay cable systems.

13.4.3.1.6 Recess Tubes

Recess tubes provide openings in the structure to accommodate cable installation. They are sized to allow free cable movements during force adjustments and to allow passage of the anchor head in case the stay must be replaced.

13.4.3.1.7 Neoprene Collars

Neoprene collars or dampers are normally provided at the front end of recess tubes. The neoprene provides a flexible lateral support for the cable, which reduces critical bending stresses at the cable anchorage. The neoprene also acts as vibration damper and seals the recess tube.

13.4.3.1.8 Caps

Caps enclose the exposed anchor heads. The cap is typically bolted to the bearing plate to effectively seal the anchorage assembly against water ingress. For further protection, the cap often is filled with a suitable corrosion-inhibiting compound. Seals must provide tight and long lasting fit.

13.4.3.2 Wire Stay Cable Anchorages

The most common anchorage for wire systems is the BBR HIAM anchorage shown in Fig. 13.14. The HIAM principle also has been used to anchor strand cables.

A relatively long cylindrical anchor head has a conical cavity in which the wires are bonded with a special HIAM compound of epoxy, steel balls, and pure zinc dust. The wires are button-headed and fixed to a template plate for spacing and alignment purposes.

The HIAM anchorage principle is similar to the traditional anchoring of wire ropes in zinc-filled sockets. Within the conical socket length, which produces some wedge action, the wires are flared so each wire is fully surrounded by the HIAM compound. The HIAM anchorage is designed to develop the full ultimate tensile strength of the wires at optimal fatigue characteristics.

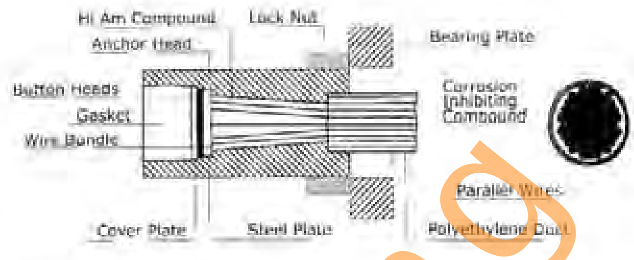

Fig. 13.14 HIAM Anchor Head
Courtesy of BBR International

The anchor head is threaded on the outside and a ring nut transfers the cable force to the bearing plate. During stressing the relatively long threaded anchor head allows for cable length adjustments through tightening or loosening of the ring nut. Additional length adjustments can be accomplished by placing or removing shims under the ring nut.

The stressing end anchor head has, in addition, an internally threaded recess for the attachment of a short, temporary cable extension for stressing purposes. The stressing rod or stressing cable extends through the jack and bears against the jack piston.

13.4.3.3 Strand Stay Anchorages

Strand anchorages have two essential components unique for strands:

- the strand-wedge connection
- the anchor head (which is equivalent to the wedge plate in post-tensioning systems)

The strand-wedge connection is critical for the ultimate and fatigue strength of the strands and of the cable. The wedges cause notches in the strand wires and produce side pressures; two effects which tend to reduce the fatigue strength of strands. Ref. 13.3 provides additional information on the strand-wedge interaction and testing.

The anchor head (or wedge plate) is subjected to very complex stresses and is not readily assessable by routine analy-

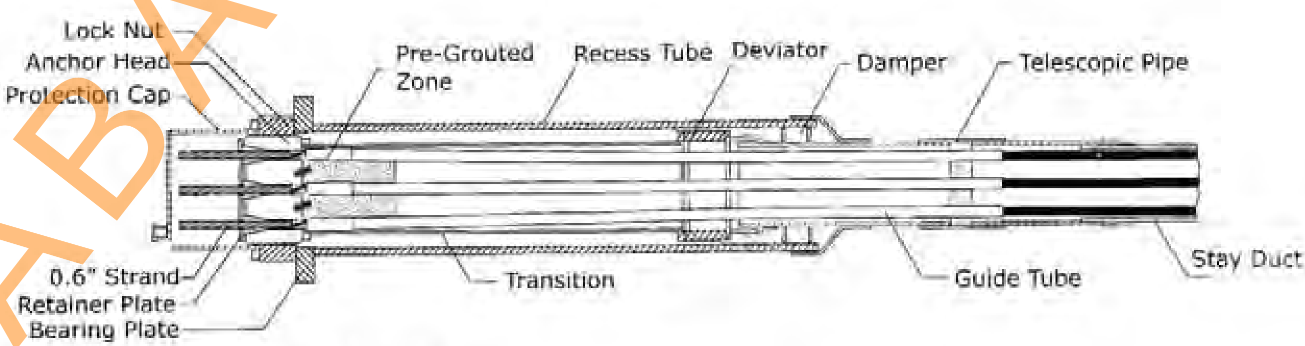

Fig. 13.15 Typical Strand Stay Cable Anchorage
Courtesy of BBR International

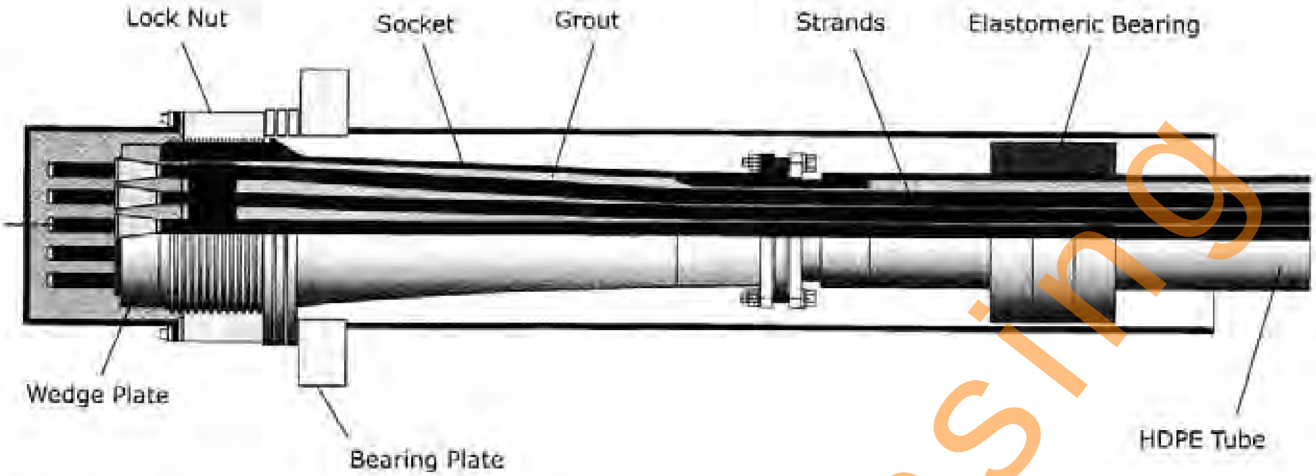

Fig. 13.16 Strand Stay Cable Anchorage With Bond Socket
Courtesy of DYWIDAG-Systems International, USA

sis, which makes testing the anchor head a critical element of stay cable design.

The 92% AUTS full sized stay cable system test required by the PTI Recommendations[13.1] serves as a proof test for the anchorage components but does not establish their actual ultimate capacity. Test equipment limitations normally do not permit establishing the failure load of an anchor head. However, experience with tendon wedge plates allows the conclusion that an anchor head has more than adequate capacity against overloads, manufacturing and material defects if it supports the 92% AUTS test force without showing significant permanent deflection of its top surface.

The simple deflection criterion described above is valid for steel anchorages with reliable ductility properties. A material with limited ductility may show no deflection but may be on the verge of failure. Therefore, less expensive ductile iron, used for small tendon wedge plates, is unsuitable for cable stay anchorages. In spite of its name, ductile iron is brittle in comparison to steel. For additional information on anchorages, see Ref. 13.3.

Fig. 13.15 shows a strand anchorage where static and dynamic loads are carried by the strand-wedge connection. Special high fatigue wedges and special plastic sleeves help improve fatigue resistance of the strand-wedge connection. This system is economical and practical for sheathed and coated strands, where individual strands can be replaced.

Fig. 13.16 shows a strand cable bond socket. The bond socket reduces by about 50% the live load carried by the strand-wedge connections and enhances its fatigue resistance. The load-carrying transition is filled after stressing with a mixture of sand and cement grout or with a suitable epoxy for field installation. A threaded bond socket and lock nut arrangement allows fine adjustments after grouting. The bond socket must be designed and manufactured to resist fatigue loads. The system does not allow replacement of individual strands.

13.4.3.4 Bar Stay Anchorages

Anchor nuts with spherical surfaces bear directly against matching surfaces in bearing plates. The swivel capability prevents bending stresses in bars if bars and bearing plates are not perfectly aligned. Such bending stresses should be avoided because they reduce the ultimate static and fatigue strength of the relatively stiff bars. Alternatively, a set of two washers with matching spherical surfaces placed between nuts and bearing plates allows rotational adjustment.

The threaded anchor nuts permit easy and controlled length adjustments during stressing operations.

At fixed anchorages, provided bending movements can be avoided, bars can be threaded directly into compact single-bar bearing plates. This feature can be advantageous for erection purposes and for limited clearance.

Multiple bar anchorages may require flaring of bars because of relatively large space requirements for anchor nut and stressing jack chair clearances. Flaring of bars complicates otherwise simple bar anchorage details.

13.4.4 Stay Cable Corrosion Protection

The surface roughening effect of corrosion causes stress raisers in high-strength prestressing steel and can significantly reduce its fatigue resistance. This is especially the case for cold-drawn wires that normally have relatively smooth surfaces.

The effectiveness of cable corrosion protection, therefore, determines its service life at least as much as its loading history. The Fourth Edition of the PTI *Recommenda-*

tions[13.1] provides detailed performance and test criteria for corrosion protection. It requires at least two independent self-sufficient corrosion protective barriers in order to qualify.

The corrosion protection of the cable in the free length is typically provided by a steel or plastic pipe surrounding the cable (Section 13.4.2). The pipe provides the outer corrosion protective barrier against the weather and protects the cable against other environmental influences.

The space between the outer pipe and the tension elements may be grouted or left open, depending on system preferences or contract requirements. Grouting for corrosion protection has been found effective for normal bonded post-tensioning applications. However, there is some concern that high grout pressures during cable grouting may force water out of the grout into the interstices between the strand wires and that wire corrosion will reduce the fatigue resistance of the strands. For this reason, and because of the additional grout weight, cable grouting is not common outside the United States, and is not being used for new structures in the U.S.

Coating of individual tension elements provides additional corrosion protection. Epoxy coated strand is discussed in Section 13.4.1.2.3. Coated sheathed strand is discussed in Section 13.4.1.2.1. Galvanized cable materials in combination with other protective methods enhance corrosion protection (see Section 13.4.1.2.2). Various combinations of these fundamental solutions are used for enhanced performance.

Corrosion protection of anchorage regions requires special attention. In these regions the tension elements penetrate their normal corrosion protection barriers and are especially vulnerable. Also, overlapping covers and expansion joints in ducts require special design and installation attention. They must be reliably and redundantly sealed to prevent moisture penetration to the cable tension elements.

Additional information on strand, anchorages, plastics, and corrosion protection compounds is located in the index.

13.5 STAY INSTALLATION

Stay installation techniques vary with the type of stay system. Stays may be factory fabricated, preassembled on site, or installed in place. The suitable method depends on the special conditions of the project and is normally a matter of contractor choice. Selection is based on the following variables:

- cable stay system
- cable size and length
- access
- construction schedule

13.5.1 Installation of Wire Stays

Button headed wire systems require precise prefabrication, which is best performed in a factory. Limited length adjustments during installation and stressing require tight fabrication tolerances and accurate geometry control.

Preassembled wire cables are normally installed as a unit. Completely preassembled cables, including ducts, can be pulled directly off reels into jacking position. Installation techniques vary from simple crane-assisted installation to elaborate winch systems.

Fig. 13.17 Preassembled Wire Cable

13.5.2 Installation of Strand Stays

Strand stay cables may be either preassembled and then installed as a unit or they may be installed in place one strand at a time. The latter method is especially suited for large and long cables. This method allows the duct to be supported and held in place by the first installed and tensioned strand. The advantages of the "one-strand-at-a-time" installation method have made strands the material of choice for modern highway bridges.

Provided adequate access and working platforms are available, individual strands can be pushed from the tower anchorage down into the duct. Frequently, however, access on the top of the pylons is restricted, requiring installation from the bottom. Each strand is then pulled into the duct with a lead wire.

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The one-strand-at-a-time installation permits perfectly parallel strand alignment, without strands crossing each other. To accomplish this, the wedge holes in top and bottom wedge plate are numbered in identical sequence from the top down. With top and bottom plate aligned, strand installation starts with the top strand. Subsequent strands are then inserted below already tightened strands, making intertwining impossible as long as the prenumbered sequence is maintained. The stressing procedure for such strand-by-strand installation is described in Section 13.6.3.3.

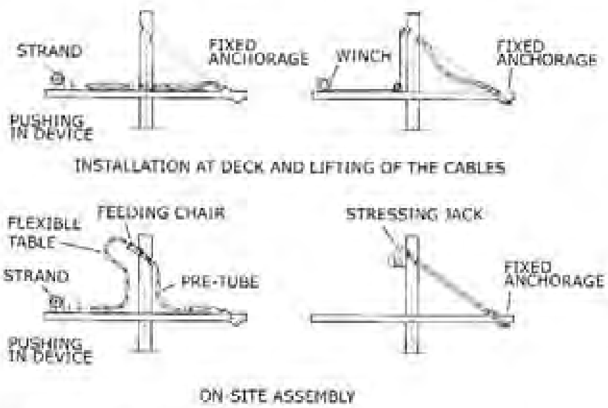

Fig. 13.18 Strand Cable Installation Methods
Courtesy of DYWIDAG-Systems International, USA

Fig. 13.19 Cable Erection With Spreader Bars
Courtesy of DYWIDAG-Systems International, USA

13.5.3 Installation of Bar Stays

Relatively stiff bar stays are normally preassembled in the factory or on site, depending on length limitations set by transportation needs. Short bar stays can also be pre-grouted inside corrosion protective pipes.

Bar stays are lifted into place by cranes that are equipped with suitable spreader bar arrangements to prevent permanent bar deflections. If a small bridge is built on falsework then it is possible to place the relatively stiff bar stays into exact position on shoring towers prior to placing concrete around their anchorages. This procedure avoids cumbersome geometry control, which is necessary if anchorages are cast into concrete and stays are installed later.

13.6 STRESSING OF STAY CABLES

13.6.1 General Information on Stay Cable Stressing

Erection procedures of cable-stayed structures typically require stressing stays in stages to predetermined tension forces while simultaneously maintaining planned elevations and camber of the supported structure. Whether priority is given to target cable forces or elevations depends on the project conditions. For instance, bridges with slender decks normally favor targeting of deck elevations, while stiff decks favor targeting of cable forces. After a structure is completely erected, it is often necessary to adjust cable forces to optimize the interdependent requirements for cable forces, stresses in the structure, and geometry.

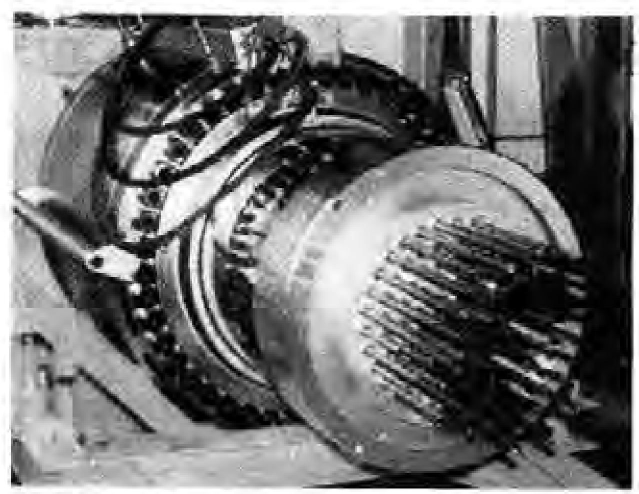

Fig. 13.20 Large Stressing Jack
Courtesy of DYWIDAG-Systems International, USA

The hydraulic jacking equipment for stressing cable stays can be quite large, heavy, sophisticated, and expensive. It is, therefore, desirable to choose designs that allow the various stressing operations to proceed with a minimum number of stressing units, while limiting equipment movement between stressing points.

13.6.2 Stressing of Wire Stay Cables

Tensioning of wire cables requires large jacks, which can simultaneously stress the whole wire cable. For tensioning purposes stressing rods or short stressing cables are attached to the anchorage and bear against the jacking piston. (See also Section 13.4.3.2.)

Cable length adjustments are accomplished by moving the anchor head to which the fixed length button headed wires are attached. A ring nut threaded onto the anchor head allows length adjustment, which is limited by the thread length on the anchor head. Additional length adjustments can be accomplished by placing or removing shims under the ring nut.

As for all stressing procedures, suitable centering devices are needed to properly align cable anchorage hardware with the jack and to maintain their alignments during stressing and seating.

13.6.3 Stressing of Strand Stay Cables

13.6.3.1 General Information

Wedge anchored strand cables installed as a unit are not length sensitive during fabrication because the potential for cable shortening during stressing is unlimited. During the stressing operation each jack stroke reduces the length of the cable until the final cable length and related cable forces are achieved.

During each jacking cycle the wedges are first released and are reset at the end of the stroke. The last stressing cycle is often planned to ensure that the stroke will be larger than the length of the wedge. Lengthening of cables to reduce cable forces should not be performed by releasing previously set wedges. Such procedures are accomplished by adjusting ring nuts or shim packs, as discussed above for wire cables.

Strand anchoring requires that the wedge clamping force develops enough friction between the strand and the wedge to support the force in the strand. If later the strand force increases because of live loads or other reasons, then the increased strand force tends to pull the wedge deeper into the wedge pocket, increasing the clamping force in the process. If a wedge is frozen it can not set further and a force increase may cause the strand to slip through the wedge.

Experience shows that in order to prevent long term strand slippage it is desirable to initially set wedges with forces between 55% and 65% MUTS. This requirement is automatically met when post-tensioned tendons are anchored off at the usual 70% MUTS. However, stay cables are stressed initially to less than 35% MUTS and at times the cable forces may be as small as 10 or 20% MUTS.

For stay cables, therefore, separate wedge blocking operations are often necessary. The wedge blocking force has to be adequate to ensure a secure strand-wedge connection without reducing the fatigue strength of the strands more than necessary.

Hydraulic blocking devices normally block one strand at a time and transfer their reaction force to the anchorage rather than the strand. Blocking more than one strand at a time is problematic.

The above considerations are equally valid for stressing a complete cable with a large jack or individual strands with a monojack.

13.6.3.2 Stressing With Large Jacks

Cables are often stressed as units, which requires large jacks with adequate stroke to accomplish the stressing task in a reasonable time. Stressing a cable as a unit ensures that all strands will have equal force. If only fine adjustment of a complete cable is required then the required stroke can be reduced, which makes the jacks shorter and lighter.

13.6.3.3 Stressing With Single-Strand Jacks

Light, specially equipped single-strand jacks are now used for stressing individual strands as they are installed.

The Freyssinet company introduced this stressing method. Freyssinet uses a patented arrangement of load cells and electronics to monitor and control the stresses in individual

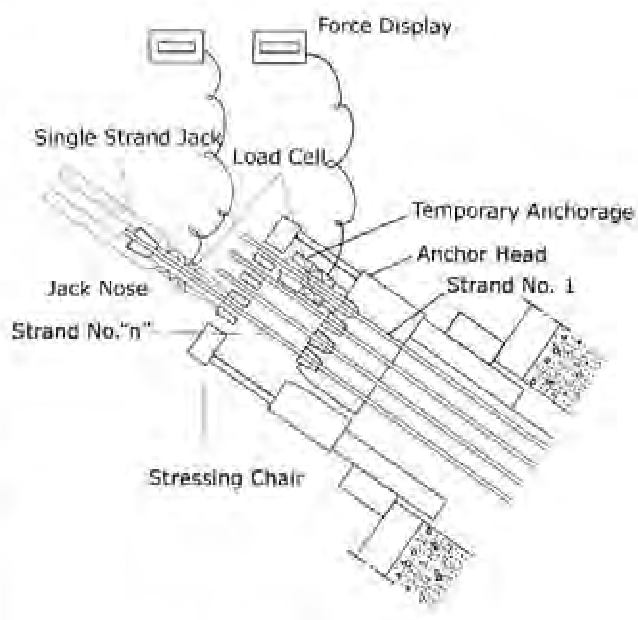

Fig. 13.21 Single-Strand Jacking
Courtesy of Freyssinet International

strands as they are installed and stressed. Fine adjustment of the cable bundle using large-capacity jacks may be necessary to lower forces or elongate cables, depending on project conditions.

In principle, the one-strand-at-a-time installation and stressing method requires the following steps:

1. Predetermine by suitable analysis the force to be jacked into the first strand, called the reference strand. This analysis takes into account the fact that the force in the first strand gradually reduces as subsequent strands are installed and stressed. However, the initial force in the first strand must also be within acceptable limits.
2. After the first strand has been installed, stressed and anchored off at the predetermined force, its force is monitored throughout the subsequent strand installations and stressing cycles.
3. Each subsequently installed strand is then stressed to the same force that is simultaneously measured in the first strand. This procedure ensures that all installed and stressed strands will have equal forces at each strand installation step.

13.6.4 Stressing of Bar Stay Cables

Bar stays are normally stressed with small, standard single-bar jacks. The bar jacks have special features to tighten or loosen the anchor nuts as the stressing progresses. This feature makes it easy to shorten or lengthen bars and to perform fine adjustments.

For multiple bar stays the stressing procedure is similar in principle to the one explained above for stressing multiple strand stays with a monostrand jack. The individual bars can also be stressed in several cycles until the forces in the bars are identical. Stressing multiple bar stays as a unit is not feasible because of the relatively large space requirements for the anchor nuts.

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13.2 *Recommendations for Stay Cable Design, Testing and Installation*, 3rd Edition, Post-Tensioning Institute, Phoenix, AZ, 1993.

13.3 *Acceptance Standards for Post-Tensioning Systems*, Post-Tensioning Institute, Phoenix, AZ, 1998.

13.4 *AASHTO LRFD Bridge Design Specifications*, SI Edition, American Association of State Highway Transportation Officials, Washington, D.C., 1999.

13.5 *Anchorage Zone Design*, Post-Tensioning Institute, Phoenix, AZ, 2000.

NOTATION

Forces and Loads:

F	Cable force
$\bar{F}$	Cable force along chord
H	Horizontal cable reaction force
V	Vertical cable reaction force
AUTS	Actual ultimate tensile strength of cables or its tension elements, measured as force
MUTS	Minimum ultimate tensile strength of cables or its tension elements, measured as force ($A_s \times f'_s$)
m	Cable mass per unit length
w	Uniform weight per unit length along horizontal span length

Cross-Sectional Properties:

A	Cross-sectional area of cable (or A_s)
E	Modulus of elasticity
E_{eq}	Equivalent modulus of elasticity
I	Moment of inertia

Linear Dimensions:

d	Cable diameter
f	Cable sag at mid-span normal to chord
h	Vertical cable projection
ℓ	Horizontal cable projection
$\bar{\ell}$	Cable span length
s	Cable length
$\bar{S}$	Length of chord
x	Horizontal distance from anchorage
y	Vertical dimension from chord to cable at distance x from anchorage

Stresses:

f'_s	Minimum nominal ultimate tensile stress of tension elements, $f'_s = MUTS/A_s$
f_{axial}	Nominal axial tensile unit stress
f_v	Vertical cable sag at mid-span
f_M	Unit fiber stress from bending moment
f_{axial}	Unit stress from axial cable force
f_{axial}	Axial unit stress in cable
Δf	Fatigue stress range

Angles:

α	Angle between chord and cable tangent at anchorage
β	Angle between horizontal and cable tangent at anchorage
θ	Angle between horizontal and chord

Miscellaneous:

ADTT	Average daily truck traffic in one direction
C	Vibration constant for circular cables
N	Natural frequency (hertz, s^{-1})
Δs_{el}	Elastic elongation of cable under cable force
$\Delta s_{\Delta t}$	Cable elongation due to temperature change
γ	Specific weight of cable per unit volume (N/mm^3)
ε	Thermal expansion coefficient for cable material
ρ	Air density (kg/m^3)
ξ	Damping ratio-to-critical

LRFD Notations:

Q_i	Force effect of direct axial plus bending for load case i
R_n	Nominal resistance (strength)
R_r	Factored resistance (strength)
ϕ	Resistance factor (material strength reduction factor)
γ	Load factor
η	Modification factor
$(\Delta f)_n$	Nominal fatigue resistance
$(\Delta f)_{TH}$	Constant amplitude fatigue threshold

STORAGE STRUCTURES

14.1 INTRODUCTION

Storage structures fulfill an important role in society and our day-to-day lives. Environmental structures are used for potable water storage and wastewater treatment. Tanks, bins and silos are also used to store a wide variety of industrial and agricultural products, such as: low temperature liquefied gases, liquid chemicals, oil and solids including grain, cement and clinker (silos).

Post-tensioned concrete storage structures are economical and have a proven performance and durability record. The history and development of P/T tanks dates back to the 1920s. Although most structures have been replaced with larger structures, some tanks from that era are still in use today. The early attempts to prestress cylindrical shells were generally unsuccessful. Similar to what was being done to farm silos, mild-steel bars and turnbuckles were used to impart compressive forces to the concrete. However, creep soon relieved all the tension in the circumferential bands and allowed vertical cracks to open up, making these early tanks generally unsuitable for water storage.

After the end of World War II and Freyssinet's discovery of the need to use high tensile strength wire, wire-wrapped post-tensioned concrete tanks started gaining popularity. These early wire-wrapped tanks used shotcrete to cover the critical wire wrapping, which was often inadequate to protect the wire from corrosion.

In the early 1970s, an alternative method of post-tensioning using internal tendons was introduced and has since gained widespread acceptance. This approach utilizes the same internal tendon technology (bonded and unbonded) used for bridges, buildings and parking garages in corrosive environments.

For the past three decades, post-tensioning has been used in many different ways in the construction of concrete storage structures, including:

- Post-tensioning of membrane floor and foundation slabs
- Longitudinal post-tensioning of straight walls
- Circumferential post-tensioning of curved walls
- Vertical post-tensioning of walls
- Post-tensioning of flat roofs
- Post-tensioning of structural shells, such as egg-shaped digesters
- Support of shell structures, such as dome roofs

14.2 ADVANTAGES

Post-tensioning offers the following advantages when used in storage structures:

1. **Versatility** – Post-Tensioned storage structures can be of any shape or geometry needed to fit the site, storage needs or the treatment process: cylindrical for potable water, clarifiers, digesters; rectangular with rounded corners for potable water, with diagonal sides when needed; rectangular for aeration basins; oval for oxidation ditches; and egg-shaped for digesters and domes for the tops of circular tanks and nuclear containment vessels.

2. **Structural Efficiency** – As in other applications, post-tensioned concrete can be structurally more efficient, and therefore less costly as compared to non-prestressed concrete. Post-tensioning facilitates the construction of highly efficient larger-capacity tanks (one-half million gallons or more),[14.1] with thinner concrete wall, roof, and floor sections. Because of the high strength materials used, post-tensioning allows greater span-to-depth ratios on roof slabs, resulting in either longer spans and fewer columns, or thinner structural members for a given column spacing. The thickness of floor slabs, walls and foundations can typically be reduced when using post-tensioning. In comparison, non-prestressed concrete tanks require high percentages of steel and thicker cross-sections to maintain serviceability criteria within acceptable code limitations, which often limits their economical use to tanks of less than one-half million gallons (MG).

3. **Water-tightness** – With proper design and construction, P/T concrete tanks can be made watertight. It has been shown that less than one gallon per year of water is lost through the walls and floor of a typical five MG water tank due the permeability of concrete. When cracking and honeycombing are eliminated, concrete water tanks are, for all practical purposes, water-tight.

   With the combination of reduced restraint to shrinkage and shortening due to the application of compressive forces by prestressing, shrinkage cracks can be prevented and the concrete can be made essentially crack-free, which ensures water-tightness. Floor joints, which are virtually impossible to monitor for leakage after the tank has been filled, can be eliminated in most cases. Other construction joints—such as the wall/floor joint and joints between wall segments can be observed for water-tightness during service and any seepage can be eliminated, assuring intended water-tightness.

4. **Construction** – Post-Tensioned concrete tanks are economical and straightforward to construct. They can be constructed using either cast-in-place or pre-cast concrete. They can be built by most experienced concrete contractors using local labor and materials. Specialized equipment such as that used to construct wire-wound prestressed structures is not needed. Required post-tensioning materials and equipment are readily available and easily transportable to even the most remote project sites.

   With the structural efficiency of post-tensioning, less reinforcing steel and concrete is needed to build a storage structure. This speeds placement and reduces labor costs. Reducing steel congestion also improves concrete consolidation and overall installation quality.

   Circular precast post-tensioned tanks are constructed using arc wall segments, and straight panels for rectangular tanks. The wall panels are precast off-site and tied together in place using post-tensioning. Since the wall panels can be manufactured at the same time while the floor is being built rapid construction is possible.

5. **Durability** – P/T storage structures are virtually maintenance free. An internal coating is not required because of the crack-free nature of post-tensioned concrete and the low permeability of concrete. As a result, the high cost of recoating that is periodically required for steel tanks is eliminated. These costs can be as much as a third of the initial construction cost of a new steel tank, and can be required every 25 to 35 years depending on the extent of annual maintenance.

   Most post-tensioned storage structures are typically designed with three levels of corrosion protection. The first level of corrosion protection of internally post-tensioned tanks is the precompressed concrete surrounding the tendons. Vertical and horizontal stressing significantly reduces concrete tensile stresses on the exterior face of the tank, thereby eliminating flexural, temperature, and shrinkage related cracking. With good quality, low permeability concrete, this cover serves as an effective barrier to the ingress of water, oxygen and corrosion-causing contaminants such as chlorides. Plastic ducts in the case of bonded tendons and high density polypropylene or polyethylene sheathing in the case of unbonded tendons, provide the second level of corrosion protection. The third level of protection is provided by the corrosion-inhibiting coating (in the case of unbonded tendons) or the grout (in the case of bonded tendons). This "triple corrosion protection" system keeps the primary containment reinforcement protected from the corrosive elements and ensures long-term durability and performance.

6. **Economy** – Because of these and other advantages, post-tensioned storage structures are extremely economical and cost competitive—both in terms of initial construction and lifecycle costs.

Fig. 14.1 Circular Storage Tank
Courtesy of VSL

14.3 APPLICATIONS

As noted above, post-tensioned concrete storage structures are well suited to store a wide range of materials. The versatility, economy, water-tightness, and long-term durability of post-tensioned concrete make it an ideal method of reinforcing storage and containment structures.

14.3.1 Liquid Storage Tanks

Water storage tanks are certainly the most common liquid storage structures. Water tanks on or in the ground are usually cylindrical, or rectangular with rounded corners. The roof is either flat, supported by columns, or is domed and therefore spans the vessel without supports. Because the pressure on the walls of the vessel is proportional to the water head, relatively low walls are preferred, usually in the 24–30 ft range.

14.3.2 Wastewater Treatment Tanks

Application of post-tensioning in wastewater treatment tanks include: sedimentation tanks, aeration tanks and sludge tanks. Sedimentation tanks are generally circular and aeration tanks rectangular, while sludge tanks are cylindrical or egg-shaped. To protect the public and surrounding water supplies, wastewater treatment facilities have strict standards for water-tightness, which makes post-tensioning particularly well suited for these applications.

14.3.3 Low-Temperature Liquefied Gas Tanks

Liquefied Natural Gas (LNG) and Liquefied Petroleum Gas (LPG) tanks are used for storing liquefied gases. These gases have to be cooled in order to liquefy them. LPG is typically stored at around 23° F (–5° C) and LNG at –265° F (–165° C). LPG must be stored under pressure, whereas LNG can be stored at atmospheric pressure on account of the very low temperatures.

A liquefied gas storage tank has to fulfill two functions: it must be leak proof in order to store the liquefied gas without leakage, and it must minimize the heat absorption of the gas as much as possible. It has been found that concrete tanks with a suitable lining are very well suited to these requirements. The lining is subjected to wide temperature fluctuations and is often made of a nickel steel sheet. Thermal insulation is incorporated between the lining and the concrete wall. Another alternative is to provide a steel sheet in the concrete wall and to insulate the external side of the concrete wall.

In some cases of LNG storage, the concrete and post-tensioning tendons may be subjected to very low temperatures. The materials used must be capable of withstanding these temperatures without damage.

Fig. 14.2 Rectangular Wastewater Tank
Courtesy of Jorgensen & Close Associates, Inc.

14.3.4 Solid Storage (Silos)

Silos are used to store solid, dry materials and are generally cylindrical in shape. Examples of materials stored in silos include cement, clinker, mining ores, coal, flour and grain. Silos are usually filled from the top and emptied from discharge chutes located at the bottom where stored material exits into transporting equipment.

Silos are large structures with very tall walls that are constructed using slip forms. Because of their high vertical walls, circular post-tensioning is ideal as the primary reinforcement and results in a crack-free slender structure. Internal, bonded post-tensioning is typically used. The post-tensioning strands are usually placed inside the ducts following the slip-forming operation to help maximize the speed of the slip-forming operation.

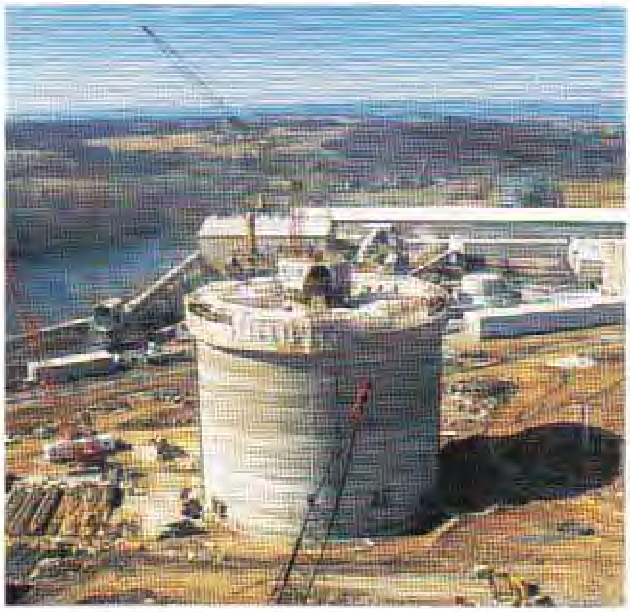

Fig. 14.3 Post-Tensioned Silo
Courtesy of VSL

14.3.5 Nuclear Reactor Containment Structures

Initial use of prestressed concrete pressure vessels for nuclear reactors in the United States commenced in the early 1960s. By 1978, approximately 60 nuclear containment vessels were built, ranging from 100 to 140 ft (30 to 43 m) in diameter, 150 to 210 ft (46 to 64 m) high, and 3 to 4 ft (0.9 to 1.2 m) thick. Primarily two types of post-tensioned prestressed concrete reactor structures have been used.

In the first type of concrete reactor, the complete pressure circuit-embracing reactor and heat exchangers are placed within one concrete vessel (a prestressed concrete reactor vessel). The Fort St. Vrain prestressed concrete reactor vessel near Denver, CO, was the first of its type in the United States. The vessel is an approximate hexagonal prism, 106 ft high and 61 ft across the sides. The internal cavity is 75 ft in height and 31 ft in diameter (Fig. 14.4).

The second type of post-tensioned concrete reactor structure is called a containment vessel. In this type of design, the reactor is contained in a steel pressure vessel connected by external ducts to heat exchangers. The complete system is then surrounded by a larger, more voluminous containment structure. Fig. 14.5 shows construction of the post-tensioned containment structure of the Palisades Nuclear Plant near South Haven, MI.

The safety of a nuclear power station is the first consideration in every reactor structure design. The purpose of a concrete containment structure is to prevent the release of radioactive materials into the atmosphere in the event of a catastrophic accident. The modes of failure of a pressure vessel must be predictable. Most available design specifications require that the interior face of the concrete be under compression when subjected to accidental elevated pressure values. These requirements can be attained with post-tensioned concrete.

Design guidelines were developed by ACI/ASME in the publication *Boiler and Pressure Vessel Code*, Section III, Division 2. FIP[14.10] has a report on the design and construction of such structures. Halligan[14.11] developed several structural details related to the use of post-tensioning in nuclear containment structures.

A preliminary design of these structures calls for two important considerations: the internal pressure must be balanced by the post-tensioning with an appropriate factor of safety; and penetrations must be carefully evaluated, analyzed, and reinforced.

Post-tensioned tendons in nuclear containment vessels are typically placed circumferentially and anchored against buttresses that are staggered over the height of the vessel. By circumferentially staggering the buttresses, the overall effect of frictional losses in the tendons can be evenly distributed.

Nuclear containment structures can be built using bonded or unbonded tendons. The use of unbonded tendons has the

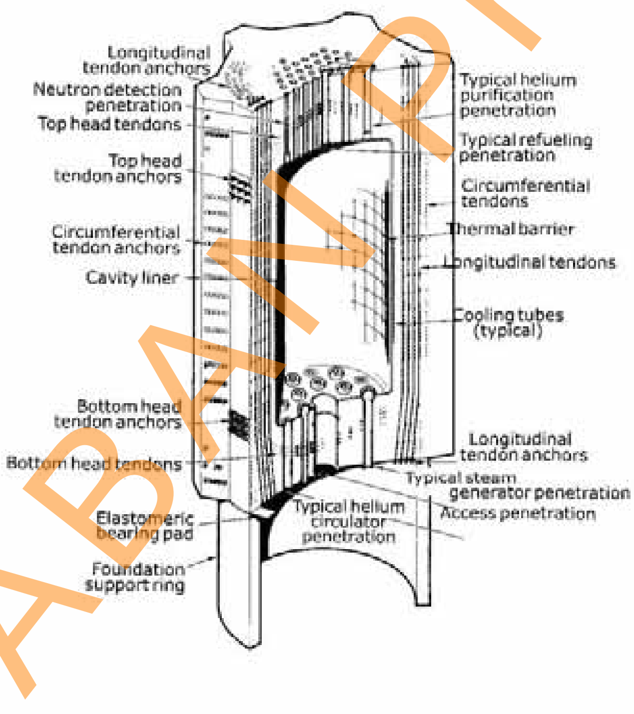

Fig. 14.4 Details of the Fort St. Vrain Reactor Vessel

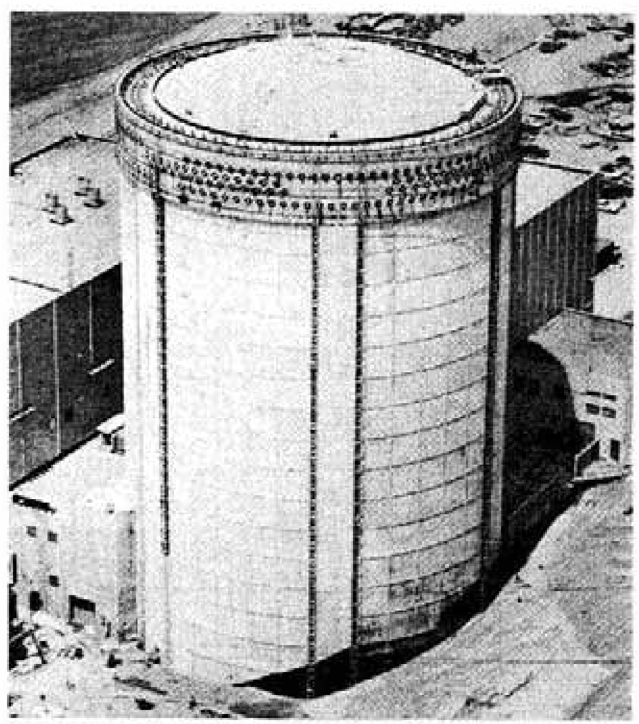

Fig. 14.5 Palisades Nuclear Plant

advantages of easy removal, monitoring, testing, and if needed, replacement. Testing is carried out to confirm that the tendons have not deteriorated and the design parameters are still being met. Additionally, the sample strands and grease samples can be examined for radioactive contamination.

14.3.6 Water Towers

Depending upon local hydraulic conditions it is sometimes desirable to construct water tanks as high-level water towers. While uncommon in the USA, P/T concrete water towers are the norm in Europe and Asia. P/T concrete water towers usually consist of a cylindrical shaft and a conical or domed tank roof.

High-level tanks can be constructed in various ways: on falsework supported on the ground around the tower shaft; on scaffolding suspended from the tower shaft; on falsework close to the ground with the tank pushed upward as the tower shaft is constructed beneath it; and on falsework close to the ground with the tank pulled up from the previously erected tower shaft.

14.4 SHAPES OF STORAGE STRUCTURES

14.4.1 Circular Tanks

Circular storage structures are very effective in sustaining tensile stresses, providing the necessary pre-compression for maximum durability and water-tightness. Furthermore, they can be easily prestressed by placing tendons in the vertical and circumferential directions. They are mostly used for potable water storage, clarifiers and digesters and can be constructed above ground, partially buried, or completely buried. Capacities range from 100,000 to 25 million gallons. Until recently, most P/T concrete tanks were circular.

Because of the numerous advantages realized using internal post-tensioning, the majority of circular prestressed concrete water tanks built worldwide use internal post-tensioning with either bonded or unbonded tendons. External wire winding has been almost completely abandoned in Europe in favor of internal post-tensioning because of long-term durability concerns.[14.7]

In a post-tensioned storage structure, the circumferential post-tensioning of walls is provided by means of individual tendons. Internal circumferential tendons are usually anchored at "pilasters" (also known as buttresses). A minimum of two and a maximum of six pilasters are typically used. Since the primary source of friction loss in multi-strand tendons is their curvature and not their length, adding more pilasters does not significantly reduce the number of circumferential tendons required, even on very large tanks. Pilasters can be eliminated through the use of a "W" circumferential tendon arrangement where the tendons are tensioned at the top of the wall, (Fig. 14.6).

14.4.2 Rectangular Tanks with Roofs

Rectangular storage structures are used when site geometry limitations dictate their use or when they are required by the owner for functional requirements. Sometimes, the base slab of rectangular storage structures consists of a mat foundation to resist the buoyancy pressure exerted by groundwater on the tank. Walls of rectangular storage structures are typically post-tensioned both vertically and horizontally.

Rectangular post-tensioned tanks can be designed with or without rounded corners. Rectangular tanks built with rounded corners can result in a very efficient design; by rounding the corners, the same pilaster details that are common on circular tanks can be used. Rounded corners not only prevent wall cracking, but can help prevent roof cracking, as the roof can expand and contract freely over the self-supported rounded corners.

14.4.3 Open-Top Rectangular Tanks

Post-tensioned concrete is particularly well suited to very large, open-top, rectangular wastewater treatment struc-

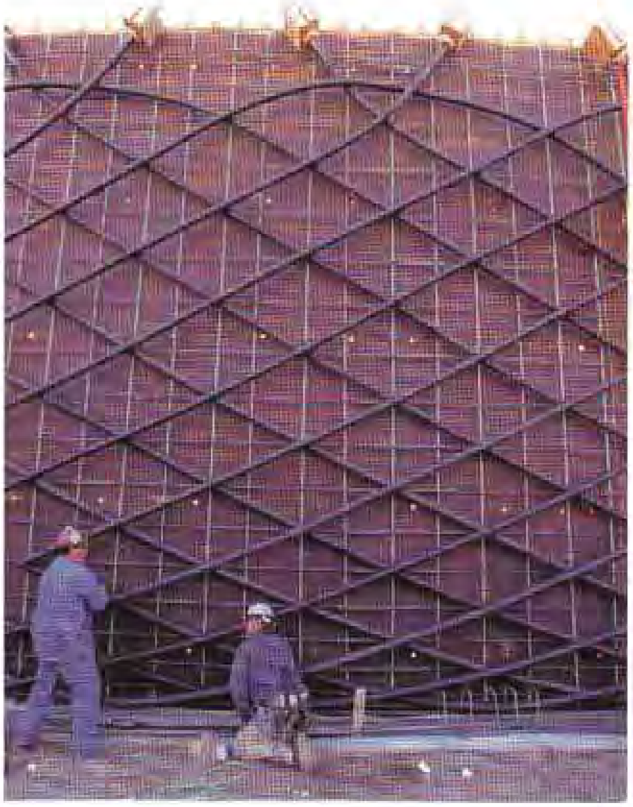

Fig. 14.6 "W" Circumferential Tendon Arrangement
Courtesy of VSL

tures. These structures are commonly used as aeration basins and sequencing batch reactors. These structures are sometimes greater than 300 ft in length. Nevertheless, the floors of these tanks can be placed monolithically without any construction, expansion/contraction, or "crack control" joints. Cracking can be controlled through the use of special anti-friction detailing and prestressing forces applied in stages as the concrete gains strength. Similarly, the walls can be placed in very long segments and post-tensioning is combined with special details to prevent vertical shrinkage cracks. This approach provides for improved liquid-tightness.

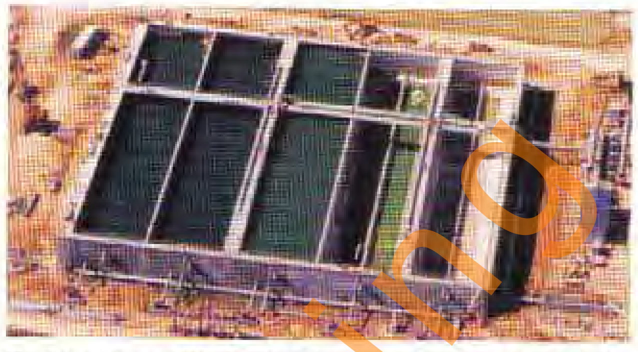

Fig. 14.7 Bayport Wastewater Treatment Plant in Pasadena, TX
Courtesy of Jorgensen & Close Associates, Inc.

14.4.4 Doubly Curved Tanks (Egg-Shaped Digesters)

Egg-Shaped Digesters (ESD) are often used for sludge digesters because their shape contributes to a more efficient digestion process. The use of ESDs has proven to be advantageous for the sludge digestion process and is gradually becoming more common. The curved surface causes the deposits to sink to the bottom of the cone, where they can be easily and continuously removed. Conversely, a crust of light particles forms on the surface of the sludge. Because their surface area is smaller than that of circular tanks, crust removal becomes easier and more efficient.

Post-tensioning tendons are used vertically and horizontally in this type of tank. Vertical tendons are typically looped, anchored in lower lifts and foundations, and extend through subsequent lifts. Horizontal tendons are often stressed at blockouts, hence buttresses are not needed.

Construction of digester tanks (ESDs) usually progresses in horizontal rings, where post-tensioning is applied in the circumferential and vertical directions.

Fig. 14.8 Egg-Shaped Digester
Courtesy of DYWIDAG-Systems International, USA

14.5 ANALYSIS AND DESIGN

14.5.1 Design Concepts, Circular Tanks

The basic theories of circular prestressing are the same as those used to analyze and design linear post-tensioned concrete structures (e.g. buildings and bridges). Therefore, fundamental principles used for linear structures are also applicable to the design of circular structures. The main construction differences between linear and circular prestressing are found in the techniques used in applying the prestressing force and in anchoring the tendons.

Linear structures can be easily stressed from one or both ends, and are anchored against concrete end blocks. Tendons in circular structures are stressed from both ends and are anchored against uniformly spaced buttresses or blockouts. Sometimes, the circumferential tendon anchorages are staggered 60 or 90 degrees along the circumferential direction, by bypassing one buttress to be anchored at the adjacent buttresses. The basic premise in the design of circular structures is the use of circumferential post-tensioning to resist hoop tensile stresses caused by the stored material, and vertical post-tensioning to resist vertical bending moments. In effect, the circular structure can be visualized to be comprised of horizontal ring segments precompressed as required to resist anticipated tensile stresses.

In addition, other effects can be present and should be considered. These can include: dynamic effects due to seismic activity, end restraints, shrinkage, creep, temperature differentials, and even swelling in the presence of water. These effects might lead to a more complex state of stress, necessitating the use of more detailed plate and shell analysis techniques available in finite element computer programs. For example, the greater the restraint at the bottom of a circular tank, the higher the resulting flexural stresses in the vertical direction, and the lower the hoop stresses in the circumferential direction near the base of the wall.

In general, proper details are critical in order to achieve an efficient design. Reducing the restraint effects will reduce shrinkage cracking, between the roof and the wall and enhance durability of the design. The design of storage structures should reflect the following overall durability, operational, and maintenance factors:[148]

- Meet or exceed the applicable code requirements
- Minimize or eliminate construction joints and have reasonable redundancy against water loss
- Maximize protection of the prestressing and reinforcing steels from corrosion
- Account for constructability, considering the requirement for high-quality workmanship
- Facilitate operation and ease of access, ventilation, and lighting for safe periodic cleaning
- Incorporate features for the safety of personnel performing periodic cleaning

Proper detailing and construction practices are important considerations for storage structures. Experienced detailing of base joints, walls, buttresses, wall-roof joints, floor joints, and columns are very important to the simplicity of construction and overall performance of the structure. (Refer to Section 14.6 for more information.)

14.5.2 Bonded vs. Unbonded

Both bonded and unbonded post-tensioning systems are used in the construction of storage and containment structures. Both systems if designed, detailed, and constructed according to good practice and current codes and specifications should provide durable structures meeting serviceability and strength requirements.

Unbonded tendons are used extensively in floor and roof slabs because of their efficiency. They can also be used in tank walls, particularly for smaller circular and rectangular structures. Additional corrosion protection can be provided for unbonded tendons by encapsulation in accordance with PTI Specifications.

Bonded post-tensioning is typically used for horizontal or circumferential reinforcement, and is the most commonly used system for circumferential reinforcement of circular tanks. Horizontal (circumferential) tendons will typically consist of multiple strands, while vertical tendons are often comprised of either multi-strand tendons or post-tensioned bars. Strands are usually used for the vertical reinforcement of taller walls, but the selection of bars versus strands is typically an economical decision.

Round corrugated duct is normally used for horizontal tendons, while vertical multi-strand tendons can be placed inside round or flat corrugated ducts. Flat corrugated plastic ducts are sometimes used in floor or roof slabs, and in the circumferential post-tensioning of precast concrete tanks. Plastic duct with watertight fittings are normally used in liquid-containing structures to provide a corrosion-resistant barrier that significantly enhances the long-term durability of the structure.

14.5.3 Wall Design

The walls of P/T tanks are often slender as compared to similar non-prestressed tanks. Reinforced concrete walls that are designed in accordance with PCA/ACI guidelines can be 16 in. thick or more. The same tank designed with post-tensioning can be 10 or 11 in. thick if cast-in-place, and 7 or 8 in. thick if precast.

The wall design should take into account the interaction between the wall, floor and roof. The designer should also consider the various stages of loading during construction, service (e.g. tank empty or full), wind, seismic and other possible overloads. At a minimum, it is recommended that a storage and containment structure wall be analyzed for three different stages of loading:

- During stressing operations
- After placement of the roof
- After application of all loads (water and backfill, if any, separately)

Circumferential P/T is used to resist hoop tensile stresses, which are directly affected by the degree of base restraint. In circular tanks, there is a complex interaction between the hoop stresses, vertical bending moments and radial shear forces depending on the base constraint conditions. To ensure water-tightness, the wall should be designed for a residual compression under full hydrostatic load. The amount of residual compression varies from a minimum of 100 psi for buried or warm-climate tanks up to 400 psi at the tops of open-top tanks in cold climates.

If buttresses are used to anchor horizontal or circumferential tendons, a minimum of two and a maximum of six buttresses should be used depending on code and design requirements. The number of buttresses depends on the tank diameter, height, tendon size, the friction coefficient, and the cost of labor and materials. Fewer buttresses will reduce formwork cost, but usually increase the number of strands or tendons. In most cases, buttresses are staggered to average out frictional losses. The number of stressing locations is usually a trade-off between buttress and tendon anchorage cost and the

Fig. 14.9 Typical Distribution of Tendons in a Tank Wall
Courtesy of Jorgensen & Close Associates, Inc.

effective prestressing force (which determines the required number of tendons) after allowance for friction losses.

Vertical post-tensioning (bars or strand) is used to resist flexure due to the end restraints and internal and external loads. Vertical tendons are typically designed to provide an average minimum residual compression of 125–200 psi.

14.5.4 Foundations and Floors

Tank foundations and floor slabs are usually cast monolithically. Post-tensioning is commonly used to cast very large tank floor/foundations with few or no joints, since joints may present a constant maintenance issue. Tanks well over 400 ft in one direction have been successfully constructed this way. In non-prestressed floors, shrinkage cracks can be very difficult to control when fresh concrete is cast against the ground, or tied into previously placed concrete floor sections.

Post-tensioning can also be effectively used to minimize or eliminate temperature and shrinkage cracking. With proper design details and construction techniques—such as the use of a low-friction membrane on a smooth, stable subgrade and two-stage stressing—cracking in floor slabs can be virtually eliminated.

Post-tensioning can be used to reduce the floor slab thickness. Five-inch-thick floors are the typical minimum for post-tensioned floors compared to six inches or more using non-prestressed reinforcement.

There are several types of foundations that can be used in conjunction with storage structures:

- Slab-on-ground
- Mat foundations resisting uplift loads from groundwater
- Two-way slabs supported on piles or drilled piers designed to carry the weight of the structure and the material within

In typical circular tanks, concrete used for the base slab is placed in one continuous concrete placing operation. Experience has shown that leakage generally occurs at the construction joints where consolidation of the concrete around water-stops is difficult.[14.15] With large concrete placements, it is recommended that one or more of the following techniques be considered to help minimize shrinkage cracking:[14.15,14.16]

- Moist cure the floor slab.
- Reduce slab-subgrade friction by using one or more layers of plastic between the subgrade and the slab, or by providing some other surface treatment/finish.

Fig. 14.10 10 MG Potable Water Tank, Arvada, CO
Courtesy of Jorgensen & Close Associates, Inc.

- Apply two-stage stressing by partially stressing the tendons at an early stage after concrete placement to help prevent early-age shrinkage cracking.
- Use additives/admixtures to lower water content of concrete.
- Use expansive cement in the concrete.

Base slabs are usually post-tensioned orthogonally in two directions and are typically designed for a minimum residual pre-compression of 200 psi.

14.5.5 Roofs

Storage structure roofs are often constructed of two-way P/T flat slabs. Post-tensioned tank roofs have many of the same structural advantages as post-tensioned elevated slabs in buildings. In addition, the structural integrity and crack mitigation of a post-tensioned roof structure provides a watertight, low maintenance surface that allows the structure to be buried or used for recreational or landscape purposes. Tendon layout is usually banded in one direction and uniformly spaced in the other.

Two-way P/T flat-plate roofs can be designed to facilitate the structure to be buried at a nominal additional cost. Flat roofs of buried post-tensioned tanks can and have been used as parks and sport courts.

14.6 CONSTRUCTION – KEY DETAILS AND PRACTICES

14.6.1 Base Joint

Base joint details are critical to the design of post-tensioned tanks because they affect the magnitudes of hoop stresses and vertical bending moments near the base of the wall. Fixed, hinged, or free bases are used depending on the various designer's preferences. The fixed-base design, shown in Fig. 14.11, creates a rigid connection between the wall and floor slab, and is most commonly used in regions of high seismicity and open-top rectangular tanks. A concrete closure strip is cast at the intersection of the wall and

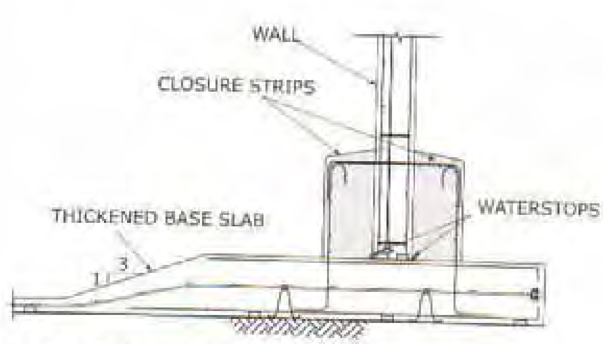

Fig. 14.11 Fixed-Base Design Commonly Used in High Seismic Areas

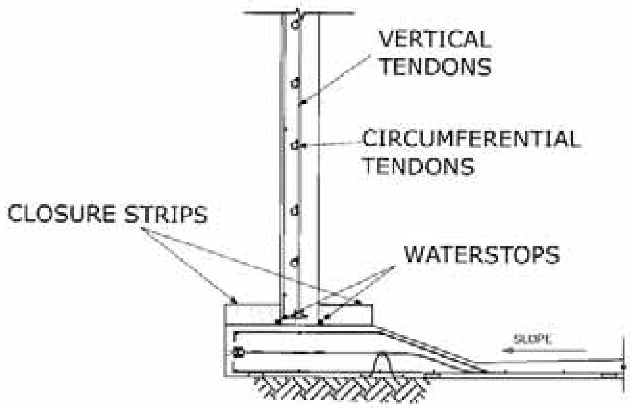

Fig. 14.12 Hinged-Based Detail Allows for Rotation of the Wall

floor slab to restrain the base of the wall against rotation and translation.

The hinged-base design, shown in Fig. 14.12, allows for rotation of the tank wall. Hinged tanks can be used in seismic zones provided that post-tensioning tendons penetrate the slab every 2 ft (0.6 m) along the tank perimeter.[14.15] Tank closure strips consist of a thickened section of the floor slab, approximately 5 ft (1.5 m) wide, adjacent to the wall. The closure strip must be properly designed to ensure that sufficient rotational and translational capacities are provided for, against external moment and shear forces. Shear at the base of fixed walls due to hydrostatic load tends to cause radial tension in the closure strip. Radial bars in the closure strip must be provided and extended for a sufficient distance into the floor slab to ensure that these radial tension forces are transmitted to the foundation.[14.15] Special attention must be paid to the joint between the closure strip and the wall. Once hydrostatic pressure is applied, the joint is subjected to combined tensile hoop stresses and flexural bending moments. Some portions of

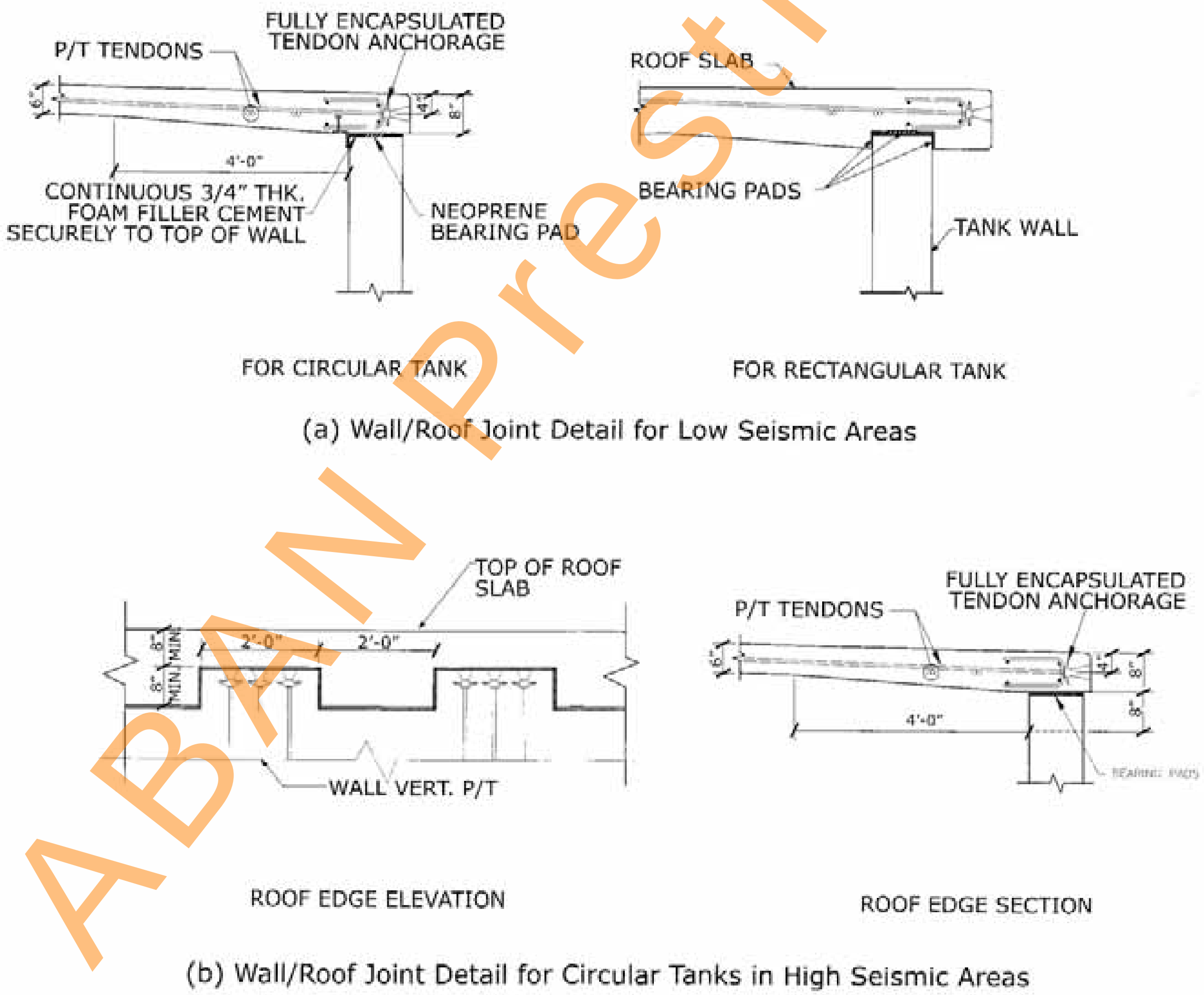

Fig. 14.13 Wall-Roof Joint Detail

Fig. 14.14 Tendon Intersection at a Buttress

this joint might be in tension, while other portions might be under compression. The hinged base design, shown in Fig. 14.12, allows for rotation, but not radial translation of the tank wall. Hinged-base tanks can be used in higher seismic zones provided that uplift does not occur. Slots in the floor are filled with grout, or curbs are cast, after the wall is post-tensioned to transfer the base shear to the floor.

14.6.2 Wall-Roof Joints

Wall-roof joints are designed to allow the wall and roof elements to move independently of each other, thereby helping to prevent restraint cracking. For rectangular tanks with square corners, the roof must be able to move freely in the longitudinal direction in order to avoid restraint cracking. Fig. 14.13 shows a simple detail that allows the roof concrete to shrink but still provides transverse support at the top of the wall.

14.6.3 Buttresses

Circumferential prestressing tendons are stressed at buttresses (pilasters) placed at equal spacing around the tank. Buttresses act as anchoring blocks for the post-tensioning tendon anchor plates, so their design and detailing requirements are extremely important. Fig. 14.14 shows a detail of circumferential tendons that cross at a buttress. The figure also shows how non-prestressed reinforcement is used to reinforce the buttress-to-wall region.

14.7 APPLICABLE STANDARDS

14.7.1 American Water Works Association

- AWWA D115 Tendon Prestressed Concrete Water Tanks[14.3]

14.7.2 American Concrete Institute

- ACI 313 Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials[14.21]
- ACI 349 Code Requirements for Nuclear Safety Related Concrete Structures[14.22]
- ACI 350 Code Requirements for Environmental Engineering Concrete Structures[14.4]
- ACI 373 Design and Construction of Circular Prestressed Concrete Structures With Circumferential Tendons[14.5]

14.8 SUMMARY

In summary, post-tensioned concrete is very well suited to many types of storage and containment structures. The versatility of post-tensioned concrete can accommodate anything from simple rectangular tanks to doubly curved egg-shaped digesters. The virtually crack-free properties of post-tensioned concrete and the redundant corrosion protection of the prestressing steel provide for improved long-term durability. These factors, combined with its inherent structural efficiency and low maintenance, make post-tensioned concrete an extremely economical choice for any storage structure.

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ROCK AND SOIL ANCHORS

15.1 INTRODUCTION

15.1.1 Description – What is a Prestressed Ground Anchor

A ground anchor is a system that mobilizes and transfers a resisting force from the ground to a structural element such as a wall or slab. Anchors transmit forces to soil or rock through a tensile member assembly that consists of prestressing steel, an anchorage, grout, coating, sheathing, and couplers (if used). Ground anchors are also referred to as "tiebacks" because they tie the load back to rock or soil material.

15.1.2 History

Ground anchors were introduced in the 1930s and first used to anchor structures in rock. One of the first recorded uses of permanent ground anchors occurred at the Cheurfas Dam, Algeria, in 1933.[15.1] In 1958 in Germany, Bauer developed the pressure-injected soil tieback.[15.2] By the 1960s, anchors were commonly used for all types of major structures in Europe.

The scarcity of land available for construction, particularly in urban areas, required excavation and retention techniques that could be accomplished with a minimum of disturbance to land owners or the traveling public. This led to the successful, widespread use of ground anchors in Europe. Anchored walls substantially reduced the amount of land required for construction while also reducing the project cost.

Of equal importance was the assurance that this technique was safe and could be easily controlled in construction. Design and construction codes for anchors were developed based on field experience and long-term observations. These codes helped assure engineers of the safety of each anchor by requiring testing to beyond proposed design loads.

Temporary ground anchor systems were introduced in the United States several decades ago to support excavations during construction. This temporary support system gained widespread acceptance because of economic and safety aspects. The first permanent soil anchor in the United States was installed in Detroit, Michigan, in 1961; but up until the late 1970s, ground anchors were primarily used in the United States to support temporary excavations.

At first, engineers in the United States were reluctant to accept permanent ground anchors because of a perceived lack of design and construction experience and documented design techniques. In 1980, PTI published the first edition of the document titled *Recommendations for Prestressed Rock and Soil Anchors*. These recommendations provided detailed guidance for the design, installation and testing of prestressed rock and soil anchors. The First Edition was followed by the Second Edition in 1986, the Third Edition in 1996 and most recently by the Fourth Edition in 2004.[15.3] These guidelines represent the current state of practice and provide practical guidance on grouted prestressed rock and soil anchors.

Since the early applications, anchor technology and acceptance has continued to grow. Today, engineers specify permanent tiebacks for a variety of applications. Over the last 40 years, significant advances in anchor technology have been achieved. State-of-the-art developments in recent years have focused on refinement of corrosion protection systems, load transfer of anchor forces into the ground, the form of tension members, and grouting methods.[15.4]

Working capacities of anchor/tiebacks in soils and weak rock have gradually increased because of changes in anchor dimensions, installation techniques, and increased understanding of load transfer mechanisms. This has resulted in increased anchor capacities and more versatile systems (e.g. fully removable anchors) that better accommodate some of the constraints imposed by urban construction.

Some of the significant enhancements of the technology that have provided these increased capacities include:[15.4]

- Increased bore diameter
- Use of end-of-casing pressure grouting
- Use of post-grouting systems with grouting repeatability
- Availability of better grouting equipment
- Development of more powerful drilling rigs to allow high production and reduction of breakdowns
- Availability of more experienced and qualified personnel who have a better understanding of construction techniques and load transfer mechanisms
- Development of fixed-length geometries that optimize efficiency of load transfer

15.1.3 Advantages & Benefits

In civil engineering works such as a gravity wall or an uplift slab, substantial weight is often required to maintain stability. Anchors are economical if the weight can be replaced efficiently by transmitting tensile forces to the ground. Prestressed ground anchors have many advantages over alternative construction methods. This economy typically stems from the following benefits:

- Reduced construction time
- Conservation of land and materials through reduced excavation
- Improved structural stability
- Improved safety
- Adaptability and flexibility, particularly for structure repair, rehabilitation or strengthening
- In the case of transportation applications, reduced disruption to traffic

Fig. 15.1 Conventional Diagonal Bracing
Courtesy of Schnabel Foundation Company

Fig. 15.2 Temporary Tieback Shoring
Courtesy of Schnabel Foundation Company

In highway cut sections, the use of permanent ground anchors can lead to substantial benefits in both economy and safety due to elimination of temporary support systems.

Excavation walls can be economically stabilized using temporary anchors. Safety is improved by eliminating cramped excavations cluttered with delicate bracing, and reducing the time and area required for standard construction methods as shown by the examples in Figs. 15.1 and 15.2. In these types of applications, anchors may need to extend beyond the site perimeter. Sometimes it is necessary, for environmental and legal reasons, to extract the tendons at the end of the project so they do not interfere with future earthwork on adjacent sites.

Special benefits may be realized in urban areas where adjoining facilities must be supported during construction. Often the area saved by utilizing permanent ground anchors can eliminate the need for underpinning nearby structures.

15.1.4 Components of Ground Anchors

The fundamental hardware components of ground anchors are essentially the same as those used in post-tensioned construction. Ground anchors consist of two main components as shown in Fig. 15.3: an anchorage, and a tension member. Each of these components must satisfy various requirements that depend upon the specific application, the forces to be transmitted, the material anchored to (i.e., rock or soil), and the life of the anchor. These factors influence the makeup of the specific ground anchor and the methods of construction and installation.

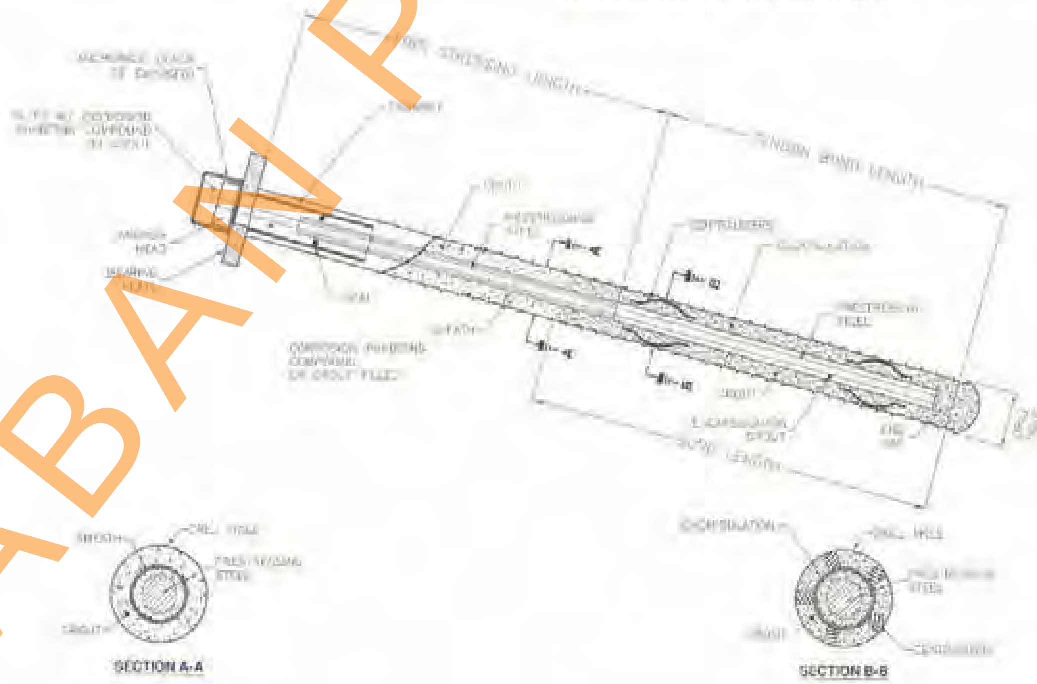

Fig. 15.3 Typical Components of an Anchor

15.1.5 Anchorage

Anchorages are the means by which the prestressing force is permanently transmitted from the tension member to the supported structure. Anchorages must be capable of developing 95% of the minimum ultimate tensile strength (MUTS) of the tension member.

15.1.6 Tension Member

Tension members primarily consist of prestressing steel that allows force transmission to the anchorage. The prestressing steel may be single or multiple strands, wires, or bars. Couplers are sometimes used with bar tendons to make up the full length of the tension member. If couplers are used, they must be capable of developing 95% of the minimum ultimate tensile strength (MUTS) of the tension member.

The total length of the tension member can be classified as either free (i.e. unbonded) or bonded.

15.1.6.1 Free Stressing (Unbonded) Length

The free stressing length is the length of the tendon that is not bonded to the surrounding ground or grout during stressing. Stressing lengths are the segments of tension members that are free to elongate elastically during stressing and are not bonded to the surrounding medium. The minimum stressing length should not be less than 15 ft (4.6 m) for strand tendons and 10 ft (3 m) for bar tendons to prevent significant reductions at transfer due to anchorage loss or movement.

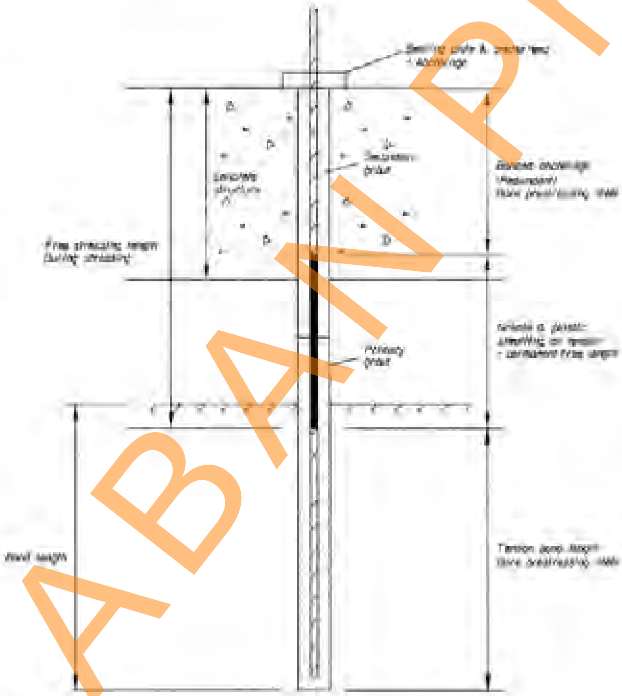

Fig. 15.4 Partially Bonded Anchor

15.1.6.2 Bond Length

Bond length is the length of the grout body that transmits the applied tensile load to the surrounding soil or rock. The force is transmitted from the tension member to the cement grout and the surrounding strata by bond stresses. Bond lengths are determined from geotechnical properties of the strata and the size of the drill hole. Typically, bond lengths should be greater than 10 ft (3 m). Bond length should be distinguished from the tendon bond length, which is defined as the length of the prestressing steel that is bonded to the grout.

15.1.6.3 Anchors

Anchors may be unbonded, fully or partially bonded.

An unbonded anchor is an anchor in which the free stressing length remains permanently unbonded to the surrounding ground or structure. Unbonded free stressing lengths allow a more flexible performance of the anchor and the averaging of structure strains resulting in less load change in individual anchors.

A fully bonded anchor is an anchor in which the free stressing length without a bond breaker is surrounded by grout, after stressing, and so is bonded to the surrounding structure or ground. Fully bonded free stressing lengths force the anchor to strain with the structure.

A partially bonded anchor is an anchor in which a partially bonded free length provides a redundant load transfer at the anchorage while at the same time leaving a certain amount of unbonded free length (Fig. 15.4).

Partially bonded free stressing lengths can be designed by terminating the bond breaker at some depth below the anchor head and limiting the primary grout to a level below the top of the bond breaker. This upper bond length is then bonded to the structure by secondary grout.

Generally, the free length should remain unbonded after stressing, except to satisfy specific structural requirements. Typically, fully bonded and partially bonded free lengths are only applicable in massive concrete structures, such as dams and diaphragm wall T panels. Fully and partially bonded anchors require that grouting be accomplished in two stages—the first to grout the bond zone and the second to grout the free length after the anchor has been stressed and tested.

15.2 APPLICATIONS

15.2.1 Retaining Walls

Perhaps the most common use of ground anchors is in the construction of retaining walls. When temporary tied-back sheeting is required, a permanently anchored wall can substantially reduce the cost of the finished wall. Construction time for completion of the wall can also be significantly reduced, typically by at least 25% when permanent tiebacks are used.[15.5] Anchored walls do not require footings, which may eliminate the need for wide construction easements. In many cases, only a narrow temporary surface easement is necessary for the construction of permanent tied-back walls. Permanent tied-back walls may completely eliminate the need for foundation piles, large footings, and backfill. In addition, the quantity of excavation and concrete are reduced. The savings resulting from elimination or reduction of these items far exceeds the cost of installing the permanent ground anchors in most situations.

Typical benefits of anchored walls over conventional wall construction include:

- Incorporation of the temporary excavation support system into the permanent facility
- Reduction of the quantity of lateral excavation
- Elimination of wall footings and related excavation and concrete work
- Elimination of foundation piles
- Reduction in the wall thickness resulting in a reduced quantity of reinforced concrete
- Elimination of backfill
- Reduction of disturbance due to construction
- Reduction in total temporary right-of-way acquisition
- Improvement in safety through construction zones as work area is reduced and only open a short time
- Greater adaptability if unexpected subsurface conditions are encountered

Figs. 15.5 and 15.6 show a comparison between permanent anchor wall and conventional retaining wall construction in soil and in rock.

15.2.2 Resist Hydrostatic Uplift

Ground anchors can also be used to resist hydrostatic uplift forces. Watertight structures located below the water table will "float" out of the ground unless the structure has sufficient weight or is tied-down. Normally, thick concrete mats have been used to offset this uplift force. Usually it takes one foot of concrete to offset every 2.4 feet of unbalanced hydrostatic head.[15.5] Permanent anchors normally are a cost-effective means of anchoring a structure against uplift, particularly as concrete costs rise.

15.2.3 Resist Unbalanced Lateral Pressures

Ground anchors are used to support unbalanced earth pressures, which result when a structure is constructed on a sloping site or into a hillside. If permanent anchors were not used, temporary sheeting and tiebacks would still be required in order to build at this type of site. In addition, the entire structure and its foundation would have to be designed to resist the unbalanced lateral pressures.

When a permanent tied-back wall is used instead of temporary sheeting and tiebacks, the anchors are designed to support the unbalanced lateral pressures and are structurally connected to the permanent wall. Extra costs for this type of anchored wall, when compared to conventional construction, result from additional monitoring or instrumentation, the corrosion protection of the anchors, and the addition of extra anchors to account for long-term loading conditions.[15.5] These costs normally are much less than the cost of designing a structure and of constructing a foundation to support large lateral forces.

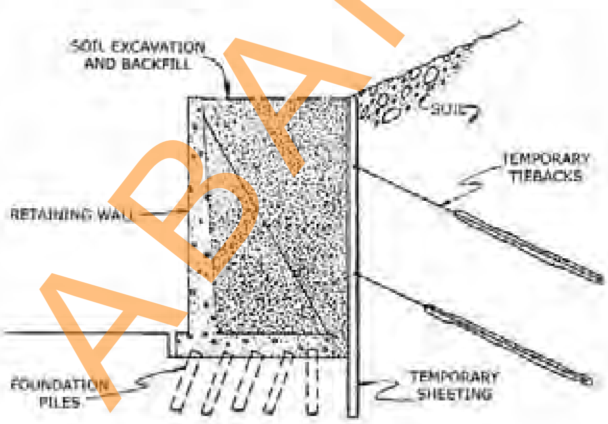

Fig. 15.5 Permanently Anchored Wall[15.4]

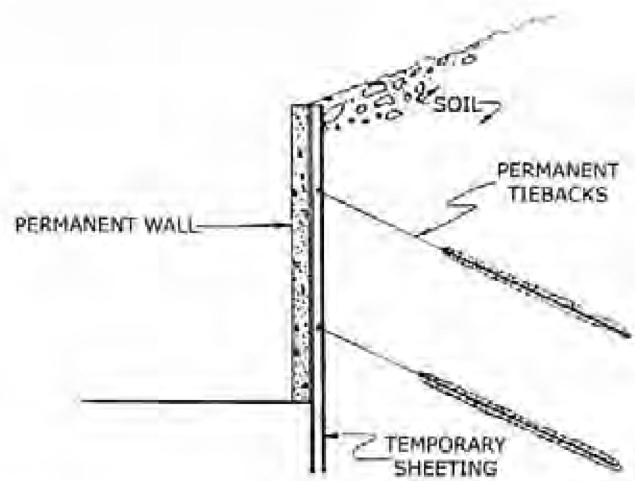

Fig. 15.6 Conventional Retaining Wall[15.4]

15.2.4 Wall Stabilization

Permanent anchors are used to stabilize or repair existing walls. These walls typically are overturning or founded on soil or rock that is part of a slide. Normally, an existing wall can be stabilized with permanent anchors at a fraction of the cost of constructing a new wall—without disturbing the rock or soil behind the wall.

Many types of existing structures can be stabilized or strengthened at low cost with anchors. Prior to the development of anchoring techniques these structures were replaced, underpinned, or buttressed. Obviously, these alternatives can be very expensive and in transportation applications can cause long-term traffic disruptions on the adjacent highway. Many transportation agencies have produced innovative anchor designs to permit highway widening beneath existing structures and to salvage and reuse retaining walls that have undergone movement.

15.2.5 Waterfront Walls

Ground anchors are used to support walls constructed along the shores of oceans, rivers and lakes. They offer several advantages over conventional alternatives. For example, deadmen, excavating, tie-rods, and backfilling are all eliminated when permanent anchors are used. Permanent tiebacks are installed without disturbing the existing ground surface, thus allowing normal operations behind the wall to be maintained. If pile-supported deadmen anchors or uninterrupted access are required, permanent anchors will normally be an economical support system for waterfront walls.

15.2.6 Tiered Walls

Tiered anchored walls are more stable and often more economical than a single vertical wall constructed at the same site. The tiered walls are less expensive than a single wall because they are designed for lower earth pressures and use shorter tiebacks than a single vertical wall. Tiered walls also have multiple failure surfaces because the failure surface for the lower wall intersects the wall above.

15.2.7 Tower Tiedowns

Elevated structures subjected to large lateral loads can be anchored with tiedowns. Ground anchors can be used to resist tower uplift forces, which usually result from hydrostatic or lateral loads. Commonly called tiedowns, these anchors are efficient and have an economic advantage over dead weight solutions as the weight of the underlying ground is mobilized for the uplift reaction. Tiedowns must be designed for two possible failure mechanisms: individual capacity to resist uplift pressures, and group dimensions (length and spacing) adequate to envelop a mass of ground with sufficient weight to resist uplift force.

Anchor tiedowns are an economical means of anchoring towers when footings cannot be embedded deep enough to develop adequate pullout resistances or when large amounts of rock must be excavated for the footings, or when the terrain makes it difficult for large drilling or excavating equipment to reach the site.

15.2.8 Dam Anchors

Federal regulations are requiring that many existing dams undergo safety inspections. If it is determined that the dam has marginal resistance to sliding or overturning, permanent ground anchors are used to anchor a dam and increase its resistance to sliding and overturning. High capacity anchors are often needed to stabilize locks and dams. With the advances in anchor capacity noted above, design loads for dam anchors typically range from 400 to more than 2000 kips. These high capacities often make ground anchor tiedowns the most economical means of increasing the stability of an existing dam. Fig. 15.7 shows a typical dam anchor tiedown.

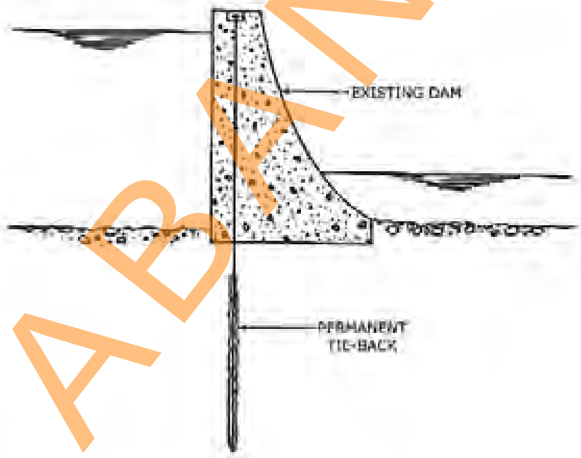

Fig. 15.7 Typical Dam Anchor Tiedown

15.3 ANCHOR DESIGN

An anchor design should include the following steps:

- Evaluation of the feasibility of anchors
- Selection of an anchor system
- Estimation of anchor capacity
- Determination of bond/free stressing length
- Selection of corrosion protection

More detailed guidance on anchor design can be found in the PTI Recommendations.[15.7]

15.3.1 Anchor Feasibility

The Design Engineer must determine whether anchors can be economically used at a particular site based on the ability to install the anchors and the ability to develop anchor capacity. The presence of utilities or other underground facilities may control whether anchors can be installed.

Before any numerical design is performed, the engineer should establish permissible limits of underground construction and determine if permanent underground easements can be obtained. These costs and any relocation costs necessitated by anchor use should be included in the preliminary cost estimates for the anchor design. Frequently, these underground easements can be avoided by establishing limiting criteria on which the anchor design will be based; i.e., total horizontal anchor length limited to a certain value by adjacent properties, or minimum angles of anchor inclination specified to avoid shallow utilities. All utilities or substructures should be clearly shown on the cross sections taken for the site. Careful evaluation of site conditions early in the design process is essential for minimizing utility relocation costs and for avoiding surprises after contract letting.

15.3.2 Anchor System Selection

Many different anchor types can usually fulfill the needs of a particular project. To achieve maximum economy, the objective of the designer is to specify only those parameters that are necessary for the long-term stability of the anchor system. The final selection of the anchor details should be left to the contractor.

Anchor system performance is ensured by testing each installed anchor to well above the design load. In determining which parameters to specify, the designer must consider various possible failure mechanisms.

The failure mechanisms are:[15.8]

- **Failure of the Steel Tendon** – As the anchor is loaded, the steel tendon is stressed in tension. If the load that is applied is greater than the structural capacity of the tendon, failure is inevitable. Therefore, a factor of safety must be used with respect to structural failure of the steel. It is recommended that the tendon load not exceed 0.6 of the ultimate tensile strength for final design and 0.8 of the ultimate tensile strength for temporary loading conditions such as testing.
- **Failure of the Ground Mass** – Related to shallow soil anchors, this failure mechanism is characterized by uplift of a mass of soil in front of the bond zone followed by pullout of the bond zone. A shear surface develops in the soil mass ahead of the anchor as increasing stresses cause complete mobilization of resistance in the anchor bond zone. The failure surface simulates a passive earth pressure failure. Practically, soil mass failure is not a factor for anchors embedded more than 15 feet below ground.

  The likely plane of failure for shallow installations in sound bedrock is along a cone generated at about a 45-degree angle from the anchor bond zone. In fractured or bedded rock, the cone shape and size varies with the distribution of bedding and cleavage planes and the grout take in fissures. Rock mass failure seldom occurs in anchors embedded more than 15 ft below ground because the bond strength between the rock and grout or the grout and tendon is much less than the intact rock strength.
- **Failure of the Soil-Grout Bond** – Anchor systems installed deep into the ground beyond external failure surfaces mobilize skin friction between the bond zone and the ground. In general, this bond is dependent on the normal stress acting on the bond zone grout and the adhesion and/or friction mobilized between the ground and grout.
- **Failure of Grout-Tendon Bond** – The bond between the grout and steel tendon must not be exceeded if the full strength of the supporting ground is to be mobilized. The failure mechanism of the grout tendon bond involves three components: adhesion, friction, and mechanical interlock.

15.3.3 Restressable Anchor Systems

Generally, anchor loads normally do not need to be adjusted during their service life. Restressable systems may be required when a significant portion of the lock-off load may be lost or gained due to movement of the ground and/or the structure.

Load adjustment of strand anchors is typically accomplished by lifting the anchor head in its entirety and installing or removing shims. An option is to provide a ring nut around the anchor head, which allows the position of the anchor head to be adjusted. The load in bar tendons is adjusted by turning the anchor nut. In order for anchors to be restressable, the free length of the anchor must not be bonded to the surrounding grout.

15.3.4 Destressable and Removable Anchor Systems

After an anchor has fulfilled its design intent, special conditions may require its destressing or even removal. Destressing may be accomplished by use of a wedge plate that allows destressing, by unthreading of the nut on a bar tendon, or by the controlled application of heat to the prestressing steel.

Removable anchors may be required in some situations, including locations where easements cannot be obtained. When removal of the anchor tendon is necessary, consideration should be given to removing only the free stressing length. Removal of anchor tendons has traditionally proved difficult and expensive and can be justified only in rare cases.

15.3.5 Anchor Capacity

In most instances, project design may consider either a large number of low capacity anchors or a smaller number of high capacity anchors. The final choice should take into consideration the design economics of the overall structure as well as feasibility.

The overall stability of an anchored structure should be determined by an experienced engineer. This analysis must consider the system's factor of safety, anchor spacing, minimum free length, ability to withstand the applied anchor loads, group action, soil and rock profile, soil and rock strength, groundwater conditions, the geometry of the structure or site and the consequences of the failure of a single anchor.

The design objective for anchors shall be to arrive at safe, economical systems that meet the acceptance criteria during initial tensioning and that perform satisfactorily throughout the life of the structure. A unique aspect of prestressed anchors compared to other structural elements is that the load-carrying capacity of each anchor is verified by load testing after installation and prior to being placed in service. In the design of anchors, consideration must be given to the specific site conditions, corrosion protection, construction means, methods and materials, and the performance requirements. Construction means, methods and materials can have a significant impact on the load-carrying capacity of anchors.

15.3.5.1 Anchor Safety Factors

The design load for an anchor is the maximum anticipated load to be resisted by an anchor during its service life. The design load is determined by the design engineer using standard design procedures, which incorporate uncertainty and risk associated with the work. Any factor of safety included in the design loads should be defined so that it is clearly understood and not duplicated. The engineer should not compound various factors of safety when designing an anchored structure.

The designer should choose a separate safety factor for each potential failure mechanism. The safety factor on the tendon at the design load shall not be less than 1.67. Therefore, tendons must be designed so that the design load is not more than 60 percent of the specified minimum tensile strength of the prestressing steel. For permanent anchors, a minimum safety factor of 2.0 should be applied to the ground/grout interface.

15.3.6 Bond/Free Stressing Length

Anchors develop resistance by stressing the soil around the bond zone. This zone must be designed so that:

1. The length is sufficient to develop the resistance necessary to prevent pullout.
2. The resistance is developed behind the potential failure surface.
3. The forces acting on the soil mass behind the wall and the anchor force are in equilibrium.

The mechanism by which an anchored wall system resists soil forces is complex because the soil, wall, bond length, and free length interact to resist earth pressures developed during and after construction while limiting deformations of the wall to acceptable values.

15.3.6.1 Bond Length

The design guidelines for estimating the load transfer capacity in the bond length are based on field experience and on full-scale tests of anchors. They are based on the assumption that load is uniformly transferred from the ground anchor over the tendon bond length. However, theoretical and experimental data show that the bond stresses are not uniformly distributed along the bond length during loading (Fig. 15.8).

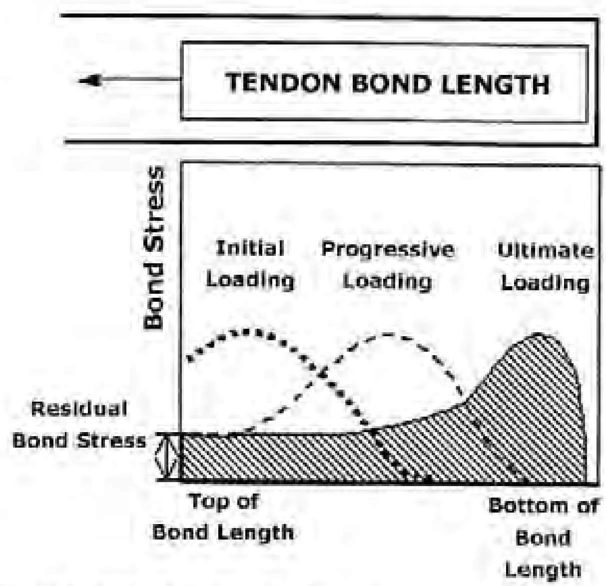

Fig. 15.8 Anchor Load Transfer Concepts

For anchor bond lengths in tension, the bond stress distribution is uniform at low loads as the anchor resists the applied load near the top of the bond length. Little or no load reaches the bottom. As additional load is applied to the anchor, the strain at the grout-to-soil interface may exceed the peak strain of the soil or the ultimate bond stress at the interface. When this occurs, the bond stress along this length reduces to a residual value and the peak bond stress moves down the bond length.

15.3.6.1.1 Rock Anchors

For conventional rock anchors installed in competent rock, the bond stresses are typically concentrated at the top of the bond length. The maximum strain in the tendon bond length occurs at the top of the tendon bond length and may cause local load redistribution within the rock or the displacement of a small cone of rock. When this occurs, the peak stress position moves down the tendon bond length.

When selecting the elevation of the top of the bond length, the designer must consider the resistance to pullout of the rock mass, which also governs anchor length. The shape of the volume of rock mobilized by the anchor depends on the orientation and frequency of jointing and bedding planes.

15.3.6.1.2 Soil Anchors

Actual bond length dimensions for specific design loads are dependent upon installation techniques and should be determined by the contractor. The average ultimate bond stress is dependent on the following variables:

1. Method of drilling, flushing and cleaning of the drill hole
2. Soil properties:
   - Permeability
   - Density
   - Angle of internal friction
   - Shear strength
   - Degree of consolidation
   - Changes of soil properties within the bond zone
   - Grain size distribution
3. Overburden pressure
4. Hole diameter
5. Grouting methods, pressures and mix designs
6. Number of post-grouting cycles
7. Tendon configuration

Normally, the bond length for soil anchors is in the range of 20-40 ft (6-12 m). Bond lengths greater than 50 ft (15 m) in soil are not efficient, unless special provisions are taken to transfer load throughout the bond zone. Minimum bond lengths of 15 ft (4.5 m) are recommended for all types of soil. In general, cohesive soils will require longer bond lengths than non-cohesive soils.

15.3.6.2 Free Stressing Length

The free stressing length for rock and soil anchors shall not be less than 4.5 m (15 ft) for strand tendons and 3.0 m (10 ft) for bar tendons. Longer free lengths may be required:

- To locate the bond length a minimum of 5 ft (1.5 m) or 20 percent of the wall height, whichever is greater, beyond the critical failure plane
- To locate the bond zone in the appropriate ground and at a sufficient depth to provide the necessary soil overburden pressure
- To ensure overall stability of the anchor/structure system

To accommodate long-term movements the free length may:

- Be grouted together with the bond length (one stage)
- Be grouted in a separate operation (two stages)
- Remain ungrouted (temporary anchors only)

15.3.7 Corrosion Protection

The continued performance of permanent ground anchors depends on their ability to withstand corrosive attacks from the environment. The corrosion protection systems must be designed and constructed to provide reliable and acceptable anchors for temporary and permanent structures. Selection of the corrosion protection class should be based on the service life of the structure, aggressivity of the environment, consequences of tendon failure and incremental in-place costs.

15.3.7.1 Temporary Versus Permanent Anchors

The design of temporary and permanent anchors differs primarily in the approach to corrosion protection. Service life is used to distinguish between a temporary and a permanent anchor. For corrosion protection considerations, permanent anchors have a service life greater than 24 months. Temporary anchors will generally require less extensive corrosion protection than will permanent anchor systems. Permanent anchors often require a larger drill hole diameter as compared to temporary anchors to accommodate added corrosion protection requirements.

The class of corrosion protection system for a project is to be selected using the principles outlined in the flow chart shown in Fig. 15.9. For permanent ground anchors, aggressive conditions shall be assumed if the aggressivity of the ground has not been quantified by testing. The principles of protection are the same for a bar or strand tendons, but the details may vary. The corrosion protection system must be compatible with the tendon, drilling method, tendon insertion method and grouting methods selected.

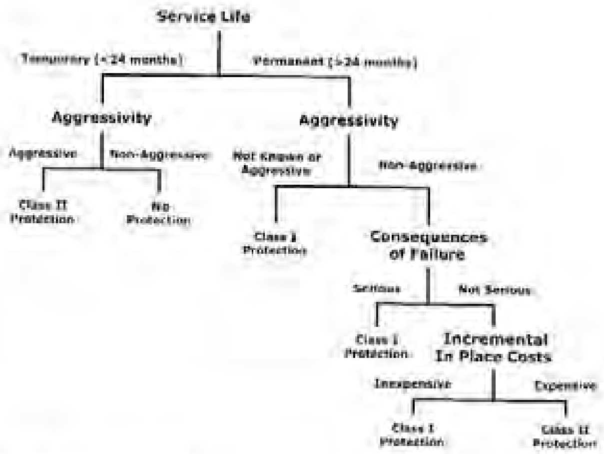

Fig. 15.9 Corrosion Protection Decision Tree

There are two classes of corrosion protection:

1. **Class I Protection** a Class I Protection system encases the prestressing steel inside a plastic encapsulation filled with either grout or corrosion inhibiting compound. Class I Protection is often referred to as an encapsulated tendon or double corrosion protected tendon.
2. **Class II Protection** – a Class II Protection system encases the prestressing steel over the free length and relies on the cement grout to protect the prestressing steel along the bond length. Class II Protection is often referred to as a grout protected tendon or a single corrosion protected tendon.

Table 15.1 outlines the requirements for each class of protection.

A Class II Protection without an anchorage cover shall be required for temporary anchors, when the environment is aggressive or when the service life exceeds 24 months.

Table 15.1 - Corrosion Protection Requirements

CLASS	CORROSION PROTECTION REQUIREMENTS		
	ANCHORAGE	FREE STRESSING LENGTH	TENDON BOND LENGTH
I Encapsulated tendon	• Trumpet • Cover if exposed	• Corrosion-inhibiting compound-filled sheath encased in grout, or • Grout-filled sheath, or • Grout-encased epoxy-coated strand in a successfully water-pressure tested drill hole	• Grout-filled encapsulation or • Epoxy-coated strand tendon in a successfully water pressure tested drill hole
II Grout protected tendon	• Trumpet • Cover if exposed	• Corrosion-inhibiting compound-filled sheath encased in grout, or • Heat shrink sleeve, or • Grout-encased epoxy-coated bar tendon, or • Polyester resin for fully bonded bar tendons in sound rock with non-aggressive groundwater	• Grout • Polyester resin in sound rock with non-aggressive ground water

15.4 CONSTRUCTION

The cost of the installation of an anchor is generally controlled by:

- Time to set up and start drilling
- Drilling time through structure or materials (wall or fill)
- Drilling time for free length of anchor
- Drilling time for fixed length of anchor
- Tendon material costs
- Grouting time and materials costs
- Anchor head material
- Stressing time

Anchors can be either shop or field fabricated in accordance with the approved drawings and schedules using personnel trained and qualified for this work.

15.4.1 Drilling

Drilling methods may involve, amongst others, rotary, percussion, rotary/percussive or auger drilling; or percussive or vibratory driven casing. Each drill hole shall be drilled at the location and to the length, inclination and diameter shown on the approved drawings and schedules.

Drilling methods should be left to the discretion of the contractor, whenever possible. The contractor is responsible for using a drilling method to establish a stable hole of adequate dimensions, within the tolerances specified. However, the designer or owner must specify what is not permissible. The owner should identify special concerns such as noise, vibrations, hole alignment, and damage to existing structures in the project specifications.

15.4.2 Tendon Insertion

Tendons should be placed in accordance with the approved drawings and details, and with the recommendations of the tendon manufacturer or specialist anchor contractor. Field personnel must inspect each anchor tendon during its installation into the drill hole or casing. Damage to the corrosion protection system shall be repaired, or the tendon replaced if not repairable.

Installation methods that might damage the encapsulation must be avoided. The rate of placement of the tendon into the hole must be controlled such that the sheath, coating, any grout socks and grout tubes are not damaged during installation of the tendon. Anchor tendons should not be subjected to sharp bends, which could damage the strand or encapsulation. A mechanical means (i.e. an uncoiler) may be needed to accomplish controlled placement of long strand tendons.

15.4.3 Grouting

The details of grouting operations for rock and soil anchors must be addressed with care, because the result of the process is not directly observable in place. Further guidance on the details of materials, mix design, testing, QA/QC, and construction may be found in Refs. 15.3 and 15.8.

The owner typically determines the level of quality assurance on any given project. The level of grout testing depends on the service life of the anchor, prior experience with the grout mix, and the type of specification for the project.

15.4.4 Installation of Anchorage

The bearing plate must be installed perpendicular to the tendon, within ±3 degrees (±0.05 rad) and centered on the drill hole, without bending or kinking of the prestressing steel elements. Wedge holes and wedges shall be free of rust, grout, and dirt. Special care should be taken to provide the continuity of corrosion protection in the vicinity of the anchorage.

15.5 STRESSING, LOAD TESTING AND ACCEPTANCE

Stressing and testing are required for every anchor to fulfill the following two functions:

1. To demonstrate that the anchor meets the acceptance criteria
2. To stress and lock-off the tendon at its specified load

Stressing and recording should be carried out by experienced personnel under the control of a suitably qualified supervisor, preferably provided by a specialist anchor contractor/supplier or an engineering agency fully experienced with the procedures.

Stressing equipment must be capable of stressing the whole tendon preferably in one stroke to the specified Test Load (TL). Hydraulic jacks must be calibrated together with the production and reference gauges against a load cell or test machine, whose calibration is traceable to NIST.

No tendon shall be stressed at any time beyond 80% of the specified minimum tendon strength, F_{pu}.

15.5.1 Testing

The three classes of tests are:

1. Preproduction Tests
2. Performance Tests
3. Proof Tests

Preproduction tests are based as a minimum on the principles of the performance test but may be more rigorous in detail. They will feature bond zone geometries likely to cause grout-ground failure within the safe operating limits of the other interfaces (e.g. grout-steel) or components (e.g. tendon to 80 percent of F_{pu}). Such special tests may be undertaken to demonstrate or investigate, in advance of the production anchors, the quality and adequacy of the design, the materials and the construction.

Due to cost and time considerations, preproduction tests are usually specified only in extraordinary circumstances. The number of tests will vary based on the size of the project and the number of anchors to be installed. Typically one to three tests will be performed for each significantly different ground condition.

Performance tests are conducted on selected production anchors constructed under methods and conditions identical to those foreseen for the overall project. The first two or three anchors on a project are usually performance tested. The Design Professional may determine that the number of performance tests needs to be increased, especially when the anchors are being used for permanent applications, when creep susceptibility is suspected, or when varying ground conditions are encountered; but normally the total number of performance tests need not exceed 5% of the total number of anchors.

Performance tests are used to determine:

- Whether the anchor has sufficient load carrying capacity
- Whether the apparent free tendon length has been satisfactorily established
- The magnitude of the residual movement
- Whether the rate of creep stabilizes within the specified limits

A performance test is conducted by cyclically and incrementally loading and unloading the anchor.

Proof tests are carried out on all production anchors not subjected to a performance test. This test is intended to quickly and economically determine:

- Whether the anchor has sufficient load-carrying capacity
- Whether the apparent free tendon length has been satisfactorily established
- Whether the rate of creep stabilizes within the specified limits

Proof tests are conducted by incrementally loading the anchor in 0.25 Design Load (DL) increments up to the full test load of 1.33 DL.

15.5.2 Supplementary Extended Creep Tests

It is recommended that at least two extended tests be made on permanent anchors in soils having a Plasticity Index greater than 20. Extended creep tests normally are not performed on rock anchors because they do not exhibit time-dependent movements. However, anchors installed in very decomposed or argillaceous rocks may exhibit significant creep behavior.

Every anchor should be tested in accordance with the Proof or performance test procedures. If the anchor is installed in ground that may be susceptible to appreciable creep, then the performance test procedures shall be extended.

15.6 ACCEPTANCE

Ground anchors must satisfy three groups of acceptance criteria:

- Creep
- Movement
- Lock-off load

The responsible engineer should evaluate the test data and determine whether the anchor is acceptable. Fig. 15.10 shows the decision process that should be followed in determining the acceptance of anchors.

15.7 SUMMARY

The uses of prestressed rock and soil anchors are diverse and continue to grow. As project space constraints become tighter, material prices rise, and the need for more rapid construction techniques grows, ground anchors become an even more economical design alternative. With increasing capacities, enhanced corrosion protection systems, and proper design and construction techniques, ground anchors can be used reliably in many different structural applications.

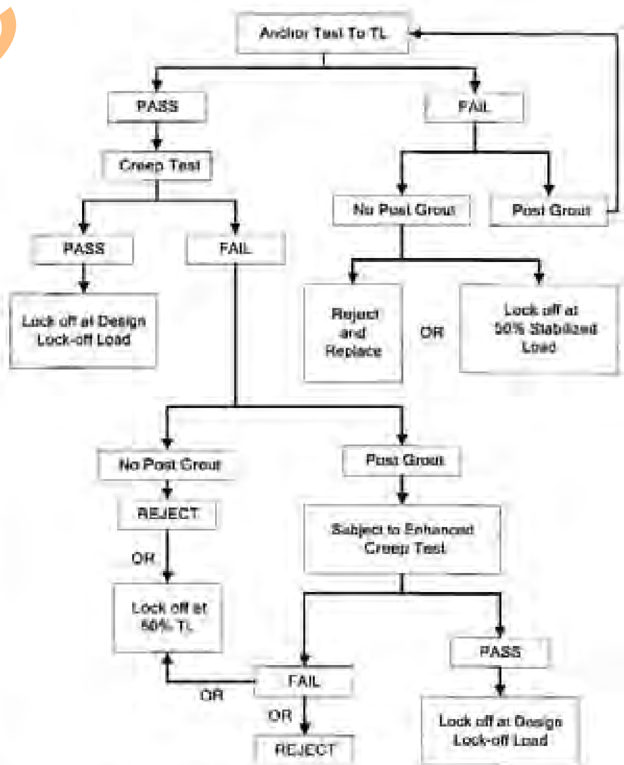

Fig. 15.10 Decision Diagram for Acceptability Testing of Anchors

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15.4 Barley, A. D., Bruce, D., Bruce, M., and Lang, C., "High Capacity and Fully Removable Soil Anchors," *PTI Journal*, Vol. 1, No. 2, August 2003, pp. 3-8.

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DESIGN OF PRESTRESSED BARRIER CABLE SYSTEMS

16.1 INTRODUCTION

The selection and design of a vehicle barrier system is an important element in the structural design of every parking garage. Some type of barrier system must be erected at the perimeter of the structure and at the open edges of the ramps to prevent automobiles and pedestrians from falling from the open sides. Due to its high tensile strength and low relaxation properties, prestressing steel is an ideal material for constructing these barriers.

The design of barriers that resist vehicle impact loads differs from the design of typical pedestrian barriers. Pedestrian barriers, handrails, cable guards, etc. are typically designed by a project's architect to meet applicable code requirements for pedestrian protection and accessibility. The materials and methods used to construct barriers meant solely for pedestrian protection will typically not be the same as those used to construct vehicle barriers, due to the higher strength demands placed on vehicle barrier systems.

The International Building Code-2003[161] (IBC) gives requirements for vehicle impact resistance and states that the barrier system must nave anchorage or attachments capable of transmitting the resulting loads to the structure. Because the vehicle impact loads are transmitted to the structure, it is important that the structural designer consider the vehicle barrier system in the structure's overall design.

One option for vehicle barrier systems is the use of prestressed 7-wire steel strand conforming to the Post-Tensioning Institute's *Specification for Seven-Wire Steel Strand Barrier Cable Applications.*^2 Steel strands conforming to this specification are capable of resisting the high lateral loads produced by the impact of a moving vehicle and are economical and flexible in meeting the geometric layout of a specific project.

Fig. 16.2 shows a typical cost comparison between various types of vehicle barrier systems and illustrates why 7-wire steel strand barrier cable systems are a popular choice for garages of all types of construction.

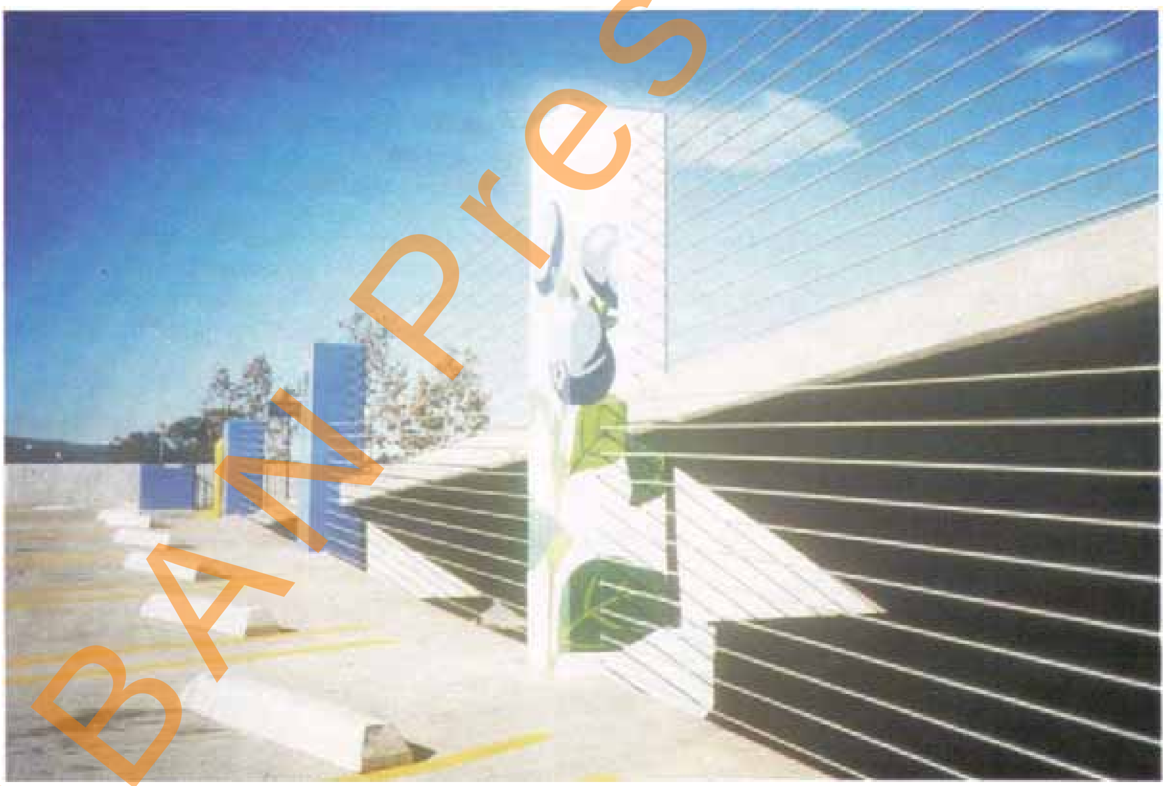


Fig. 16.1 Examples of Vehicle Barrier Cable Systems

Fig. 16.2 Cost of Exterior Barrier Systems

This chart uses the following configuration in its cost comparisons:

- Barrier cable system consists of 11 cables
- Masonry and cast-in-place spandrels are 42 in. high and built on the slab
- Precast spandrels are 60 in. high and extend over the edge of the slab

16.2 BUILDING CODE REQUIREMENTS

IBC outlines requirements for parking garage barrier systems in Section 406.2. This section includes requirements for the barrier system to meet two (2) distinct objectives:

- Pedestrian protection (Section 406.2.3)
- Automobile restraint (Section 406.2.4)

While the Licensed Design Professional will typically only be concerned with the barriers that are necessary for automobile restraint, the locations requiring vehicle barriers will, in most cases, require pedestrian protection as well. Given this condition, it is logical to design a single barrier system that meets both requirements, as discussed in the following sections.

16.2.1 Pedestrian Protection

Barrier systems for pedestrian protection are required at exterior and interior vertical openings where vehicles are parked or moved, and along open-sided walking áreas or ramps, when the vertical distance to the ground or surface below exceeds 30 in. (762 mm). The pedestrian barrier system (guard) must meet the physical requirements of IBC Section 1003.2.12.

This section states that the guard must form a protective barrier not less than 42 in. (1067 mm) high, "measured vertically from the leading edge of the tread or adjacent walking surface." Openings in the guard must be limited such that a 4 in. (102 mm) diameter sphere cannot pass through any opening up to a height of 34 in. (864 mm). Above a height of 34 in. (864 mm) a sphere of 8 in. (203 mm) cannot pass through the opening(s).

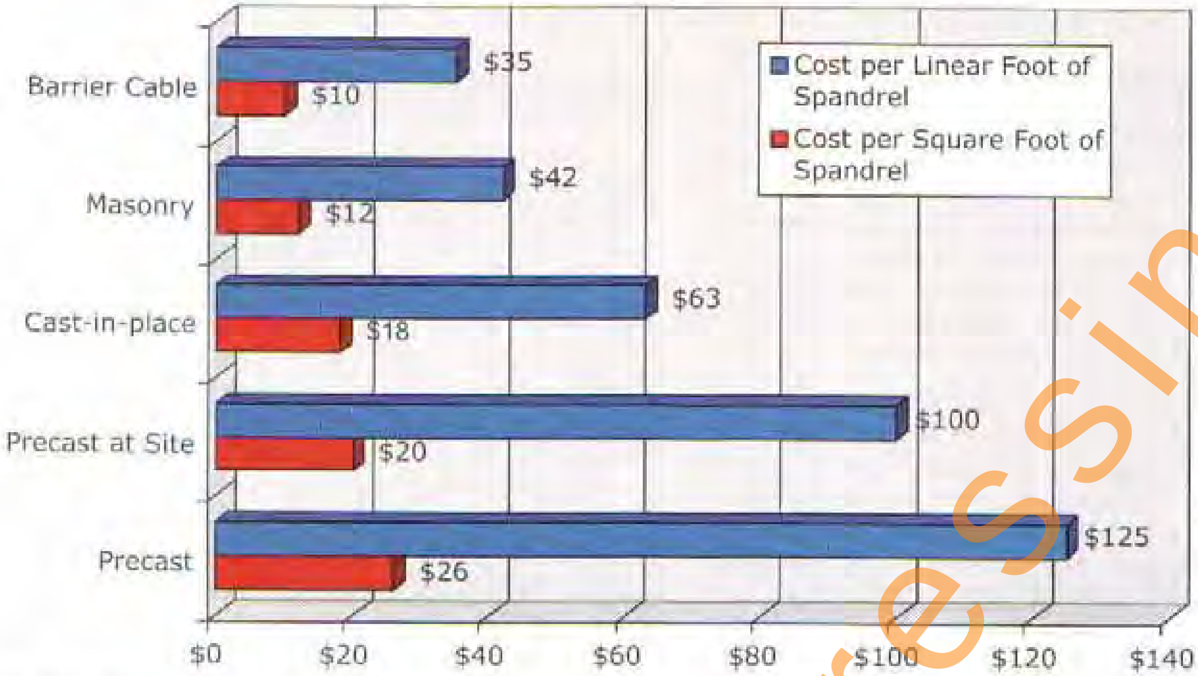
This section also outlines the minimum loading requirements for guards for pedestrian protection; however these loads are not discussed herein as they represent only a small fraction of the load capacity required for vehicle barriers.

16.2.2 Automobile Restraint

IBC Section 406.2.4 requires vehicle barriers not less than 24 in. (607 mm) in height to be placed at the ends of drive lañes and at the end of parking spaces where the difference in adjacent floor elevation is greater than 12 in. (305 mm). Vehicle barriers of all types must meet the physical requirements of IBC Section 1607.7.

This section states that barriers for garages designed for passenger cars are to be designed to resist a single (unfactored) load of 6000 Ibs (26.70 kN) applied horizontally in any direction to the system. For design purposes, the code assumes the load to act at a minimum height of 18 in. (457 mm) above the floor surface on an área not to exceed 1 sq ft (0.09 m^2).

Barriers for garages that accommodate trucks and buses are to be designed in accordance with an approved method that contains provisions for larger vehicles. Depending on



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Barriers for garages that accommodate trucks and buses are to be designed in accordance with an approved method that contains provisions for larger vehicles. Depending on the interpretation of the building official, this may include levels or areas of the garage subject to truck traffic such as delivery areas and loading zones. The traffic patterns in these areas should be carefully considered as it can be common for these larger delivery trucks to impact barriers when backing up in close quarters.

Another very important (and often overlooked) element of the code is the requirement that the barrier system have anchorages or attachments capable of transmitting the loads resulting from a vehicle impact to the structure. Prestressed barrier cable systems are typically anchored to supporting columns or walls in the garage. It is critical that the designer calculate the stresses that will result from a vehicle impact to ensure that these connecting elements have the capacity to resist this force. This is particularly important when the connecting column is a short "stub" column (sometimes used on the top level of a parking garage—see Fig. 16.3) that most likely will not have the capacity to handle these loads unless additional reinforcing is added.

The following sections will present equations and design examples that illustrate a design procedure for calculating the forces and deflections that result from a vehicle impact. Note that local building code requirements may be considerably more stringent than the IBC requirements, particularly with regard to vehicle impact loads; therefore larger values may need to be used in these equations.

16.3 DESIGN CONSIDERATIONS

The primary design consideration is to provide protection by resisting the impact of a vehicle without a failure of the barrier cable system. It is important to recognize that failure of the barrier cable system can occur in several different modes:

- Failure of the anchorage system (either due to the anchorage, or anchorage assembly, itself pulling out of the column, or due to the cable pulling out of the anchorage)
- Failure of the system to limit deflection of the cable on impact to a value that still provides protection
- Failure of the cable or group of cables to resist the impact without breaking
- Failure of the connecting column(s) or wall(s)

16.3.1 Failure of the Anchorage System

The connection of the anchoring system to the column(s) or wall(s) is explained in Section 3 of PTI's *Specification for Seven-Wire Steel Strand Barrier Cable Applications*. Appropriate material reports and test data should be used to calculate the ultimate pull-out and shear strength of *all* component parts being used. When utilizing brackets that are attached to the columns via weld plates or anchor bolts, this includes checking the capacity of these attachments as well as the capacity of the anchorage used to attach the cable to the bracket.

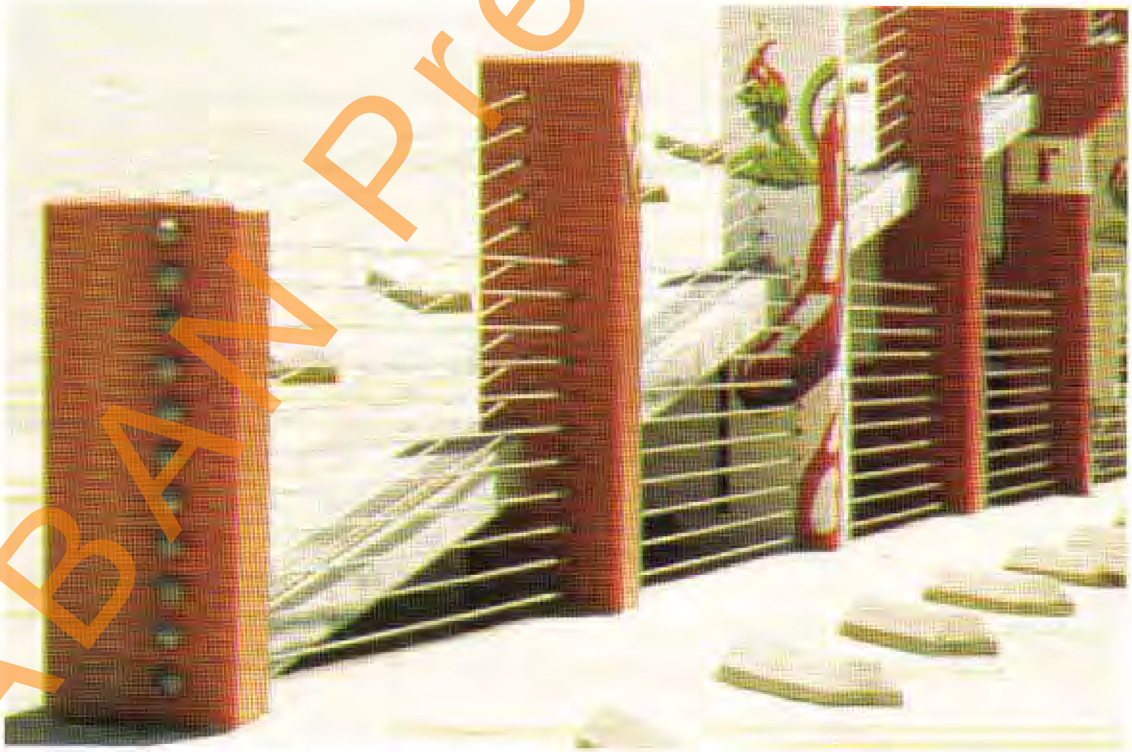


Fig. 16.3 Stub Column Used to Anchor Barrier Cables

Fig. 16.4 Backstressing of Barrier Cables

Failure caused by the cable pulling out of the anchoring device can be avoided by following the material requirements listed in Section 3,[16.2] and by strict enforcement of the installation requirements detailed in Section 5;[16.2] specifically, all wedge-type anchorage devices (the most commonly used in this type of system) *must* be backstressed to a force equal to 80% of the Minimum Ultimate Tensile Strength (MUTS) of the cable. These wedge-type anchorage systems are typically the same type of anchorage used to anchor single-strand unbonded tendons (see Chapter 3); however, barrier cable systems are typically tensioned to a relatively low jacking force that is not adequate to properly seat the wedges and form the mechanical connection that the system relies on. Back-stressing ensures that the wedges are seated and the proper (permanent) connection is made.

Procedures for backstressing are outlined in Section 5 of the Specification,[16.2] and calculations for accommodating the loss of force due to seating loss are discussed later in Section 16.5.

16.3.2 Limiting Deflection

Prestressing steel strand elongates under a load as follows:

$$\Delta = \frac{TL}{AE} \tag{16.1}$$

Where:

T = the applied load in pounds

L = the length of the strand in inches

A = area of prestressing steel in sq in.

E = modulus of elasticity of the steel

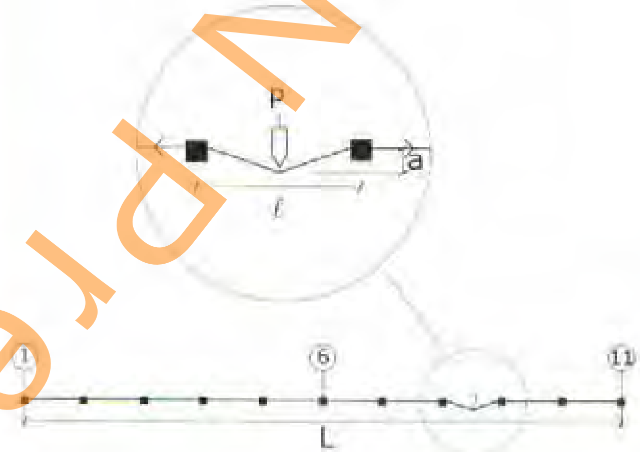


Fig. 16.5 Deflection Under a Point Load

Using the model shown in Fig. 16.5 and ignoring any applied prestressing force, other than that necessary to remove the slack in the cable, total deflection in a barrier cable strand a can be calculated as:

$$a = \left(\frac{P\ell^2 L}{8AE}\right)^{1/3} \tag{16.2}$$

When barrier cables are spaced to meet the pedestrian protection requirements discussed earlier, a typical assumption will be that the 6000 lb point load imposed by the IBC is resisted equally by three cables.

Using 7-wire galvanized post-tensioning strand with the following properties:

- Cross Sectional Area of Steel (A) = 0.153 in^2
- Modulus of Elasticity (E) = 28,500,000 lbs/in^2

and a barrier cable system with a total length (L) of 200 ft and a column-to-column span (ℓ) of 20 ft, the deflection of one cable under a 2000 lb load (P) would be:

$$a = \left(\frac{2000 \times 20^2 \times 200}{8 \times 0.153 \times 28,500,000}\right)^{1/3} = 1.66 \text{ ft}$$

Typically, maximum allowable deflection should be limited to 18 in. in order to prevent the front wheels of an impacting vehicle from traveling over the edge of the slab. However, there are instances when it is important to limit deflection to a lower value. This includes instances when the barrier cable system is placed in front of architectural masonry walls or precast panels that are not specifically designed to handle impact loads. In this case it would be important to limit deflection so that an impacting vehicle would be stopped before it impacted the wall (and sent debris to the ground below). It may also be necessary to limit deflection to the point where a vehicle will not impact cars in opposing stalls or the edge of the slab at adjacent ramps.

When calculated deflection exceeds allowable deflection, the designer has two options:

- Increase the pretensioning force in the barrier cable strands (explained in the next section)

or

- Add intermediate anchorage devices which shorten the effective length L

Using the previous example and adding intermediate anchorage devices on either side of column line 6 (Fig. 16.6) will shorten the effective total length (L) to 100 ft, which reduces deflection (a) to 1.32 ft.

The need for intermediate anchorage devices should be determined by the design professional and their locations should be clearly shown in the contract documents.

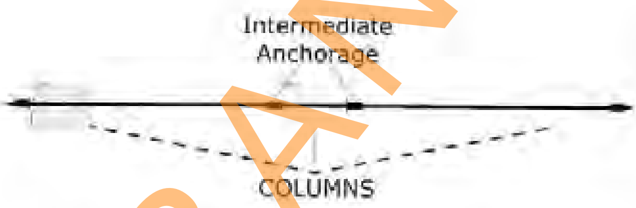


Fig. 16.6 Intermediate Anchorage Devices Used to Limit Deflection

16.3.3 Calculating the Capacity of the Barrier Cable System

The load on a cable during a vehicle impact is not constant in value, but actually changes with deflection; therefore the method originally presented by Presswalla[6] termed the "energy method" is the more rational approach to barrier cable design. This method is based on energy principles in which the kinetic energy of a moving vehicle is converted to cable force and deflection. Using this method, the designer can accurately calculate both the tension in a cable and the deflection in a cable resulting from the impact of a vehicle of a given mass traveling at a given velocity.

The method consists of two steps: first, determining the tension (T) in a cable upon impact, and then determining the resulting deflection (a). Fig. 16.7 illustrates a cable of total length L strung between two points A and B. The cable is assumed to be supported by frictionless bearing points at C and D.

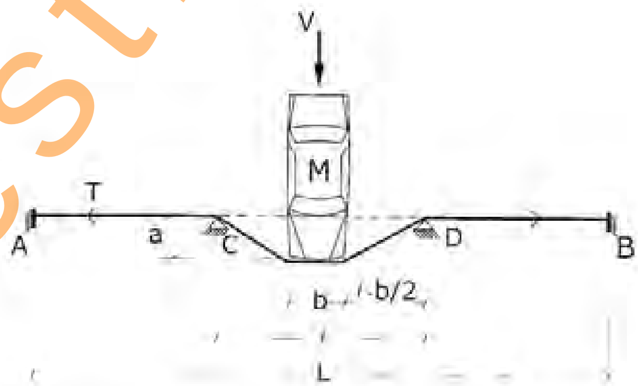


Fig. 16.7 Barrier Cable Deflection on Impact

This figure represents a typical straight run of barrier cable, anchored only at its end points (A and B). The run may have several spans in its length but the designer should use the longest span ℓ to calculate tension in the cable as follows:

$$T = \sqrt{\left(\frac{EA}{L}\right)\left(\frac{MV^2}{N}\right) + F_e^2} \qquad (16.3)$$

Where N is the number of cables resisting the impact, V is the velocity of the vehicle in ft/sec, and M is the mass of the vehicle calculated as:

$$M = \frac{\text{Vehicle Weight}}{g} \qquad (16.4)$$

The designer starts by choosing an initial value of prestressing force F_e in order to calculate T. The resulting value is then used in the following equation to calculate the deflection of the cable (a) upon impact:

$$a = \sqrt{\left[\left(\frac{T - F_e}{2AE}\right)L + \ell - b\right]\left[\left(\frac{T - F_e}{2AE}\right)L\right]} \qquad (16.5)$$

If calculated deflection exceeds allowable deflection, the designer can either choose a higher value for F_e and recalculate the resulting force T and deflection a, or add intermediate anchors as described in Section 16.3.2 and then recalculate T and a using the smaller value of L.

As a check against the applicable building code requirements, the following equation can be used to ensure the system meets the 6000 lb point load requirement:

$$T = \frac{(6000 / N) \times \ell}{4a} \qquad (16.6)$$

Note that a static load of 6000 lbs is roughly equivalent to a 5000 lb vehicle impacting the cables at a velocity of 5 mph (14.67 ft/sec).

In either equation, the resulting value of T is the total tension present in each resisting cable upon vehicle impact. This value should be compared with the yield strength of the cables (not the breaking strength) to determine the factor of safety against yielding. If prestressing steel strand conforming to ASTM A416 is used, yield is calculated as 90% of Minimum Ultimate Tensile Strength.

16.3.4 Column Design

The connecting column or wall must be designed to resist the total lateral load that is transmitted to it. This load includes *both* the initial prestressing force that is present in each cable, plus the added force generated by a vehicle impact. This is calculated by using the highest value obtained for T (from Eq. [16.3] or Eq. [16.6]) and multiplying it by the number of cables used to resist the impact N, then adding the product of the remaining cables multiplied by the final effective prestressing force (F_e) that has been applied.

Each of the connecting columns should be evaluated to ensure they are able to withstand this total lateral force. This is usually not a controlling factor on a typical column that spans from floor to floor. However, examples in the next section will show that total prestressing force plus the force generated upon impact can be well over 40,000 lbs which can be a factor when using short stub columns on the top level of a garage.

Since these short columns do not connect above, they do not have the same capacity as the other "typical" columns. This condition should be carefully evaluated to determine the need for additional reinforcing at the anchoring column(s).

Another factor to consider when using these stub columns is the ability of any intermediate stub columns to resist the same lateral impact load as the rest of the barrier system. In other words, the columns themselves must be able to resist the lateral forces of a vehicle impact without failure. Failure of intermediate stub column(s) upon impact would greatly increase the effective length of ℓ, thereby increasing the deflection (a) to the point of not providing adequate protection. It is particularly important to evaluate this condition when using small steel columns that are simply anchored to the floor.

16.4 PRESTRESSING TO ELIMINATE CABLE SAG

A minimum amount of prestressing force must be applied to the cables to eliminate sagging of the cables under their own weight. PTI's *Specification for Seven-Wire Steel Strand Barrier Cable Applications* in Section 5.4 requires all cables to be stressed to a minimum force (F_e) of 2 kips (8.9 kN) for 18 ft (5.5 m) spans in order to limit sagging to an acceptable value.

Excessive sag in the cables is not visually appealing and may not satisfy applicable code requirements in that it may allow the passage of a 4 in. diameter sphere even though cable spacing is less than 4 in. For example, if cable spacing is 3.5 in. but there is 1 in. of sag in the cables, a 4 in. diameter sphere will pass through the opening if enough force is applied to overcome the weight of the cable. Conversely, if cable spacing is 3.5 in. and sag is reduced to ⅛ in., a 4 in. diameter sphere will not pass through unless enough force is applied to start elongating the steel itself.

Cable sag is a function of the weight of the cable itself and the spacing of its supports (ℓ). The following equation (Ref. 16.4) can be used to calculate sag in barrier cable due to self-weight (w):

$$s_{\text{inches}} = \frac{\ell^2 w}{8F_e} \qquad (16.7)$$

Using the weight of galvanized PC strand, and based on the recommendation of using a prestressing force of 2000 lbs for an 18 ft span, allowable sag calculates to approximately ⅛ in. It is recommended that this ratio (⅛ in. per 18 ft or .007 in./ft) be used in calculating the maximum allowable sag in spans longer than 18 ft.

Using this ratio and solving for F_e, the equation for calculating the minimum prestressing force required to reduce sag to an acceptable level is:

$$F_e = \left|\frac{\ell^2 w}{s/12}\right| / 8 \tag{16.8}$$

Replacing (s) with the ratio of .007l, the equation becomes:

$$F_e = \left|\frac{\ell^2 w}{(.007 \times l)/12}\right| / 8 \tag{16.9}$$

The chart included as Appendix A of the *Specification for Seven-Wire Steel Strand Barrier Cable Applications* lists approximate values for the weight of various types of strand used as barrier cables. Use these values in the above equation to ensure that the prestressing force being used will eliminate sag to an acceptable value. If the force obtained in Eq. (16.9) is higher than the value used in the design equations the designer has two options:

1. Use the higher prestressing force obtained in Eq. (16.9) and re-run the design equations, finding a new value for T, a, and for the total load being applied to the anchoring columns.
2. Add some type of intermediate spacers or supports to reduce the value of ℓ in Eq. (16.9).

Adding intermediate spacers or supports increases material and labor costs, but may be necessary in garages with very long spans or where it is not desirable to increase the loading on the anchoring columns (or brackets). Note that since these spacers do not provide any lateral resistance they only reduce the value of ℓ as used in Eq. (16.9).

16.5 CALCULATING JACKING FORCE

As previously stated, the relatively low prestressing force (F_e) that is applied to the cable to reduce deflection and eliminate sag is not enough to properly seat the wedges into the anchorage devices typically used with 7-wire steel strand barrier cable systems. These wedge-type anchorage devices are designed to form a mechanical (not a friction) connection between the cable, the wedge, and the anchor, when the wedges are fully seated. It takes an applied force equivalent to 80% of the Minimum Ultimate Tensile Strength (MUTS) of the cable to fully seat the wedges. When using ½ in. grade 250 PC strand, this force is 30,600 lbs, much higher than the prestressing force (F_e) that is being applied to the cables.

In order to apply the required seating force, the installer must backstress the cable at all anchors to the required 80% of MUTS. This involves stressing each cable to a force equivalent to the calculated final effective force (F_e) plus an additional force needed to compensate for the seating loss that will occur (during backstressing), then removing the stressing jack and using it on the other side of the anchoring assembly to apply the required 80% of MUTS and fully seat the wedges.

The actual technique used for backstressing will depend on the particular job site conditions and is explained in more detail in Section 5.4 of the *Specification for Seven-Wire Steel Strand Barrier Cable Applications*.

It is appropriate to assume that the anchorage will experience the full seating loss only upon backstressing. This value is typically ⅜ in. but can vary depending on the particular anchorage system being supplied. The following equation can be used to calculate the jacking force (F_{pj}) that will be required to maintain the required final effective force (F_e) that was determined in the design equations:

$$F_{pj} = F_e + \frac{\text{Seating Loss} \times A \times E}{12 \times L} \tag{16.10}$$

Because seating loss can vary according to the type of anchorage system and/or stressing equipment being used, it is typically not appropriate for the designer to specify a required jacking force, but instead should specify the required final effective force (F_e) and then require the barrier cable material supplier to calculate the required jacking force according to materials and equipment being supplied to the project. The supplier should submit this jacking force to the designer as part of the submittal package. The supplier should also supply a stressing equipment calibration chart with the stressing equipment that is being supplied to show the gauge force that corresponds to the required jacking force.

16.6 DURABILITY AND CORROSION PROTECTION

The long-term durability of a vehicle barrier cable system is dependent upon proper installation of the system and on the type of protective coating(s) specified by the designer.

The system should be installed according to the minimum standards listed in Section 5 of the *Specification for Seven-Wire Steel Strand Barrier Cable Applications*. Any additional requirements given by the supplier should also be followed.

Corrosion protection of the anchorage system and the attachment devices should be considered with respect to the environment in which they are being installed. Electroplating, galvanizing, epoxy coating, and plastic encapsulation are some of the processes typically used to prevent corrosion.

The strand used must also have a corrosion-preventative coating applied to it prior to being installed. Coatings used include galvanizing (hot-dip or electro-plating), plastic (applied by a continuous seamless extrusion process), and epoxy (applied by a seamless thermal process). Combinations of these coatings, such as plastic coated galvanized strand, have also been used.

The designer should carefully evaluate the environmental and service conditions that the system will be subjected to throughout its expected service life. Considerations include:

- **Ultraviolet degradation** – the coating on cables installed at the perimeter of a garage may be subject to considerably more ultraviolet degradation than those installed at interior ramps
- **Salt spray** – including spray from deicing salts, seawater, and salt-laden air
- **Temperature extremes** – high-density polyethylene is typically more durable during installation in normal weather conditions, but low-density polyethylene may perform better in extreme cold weather conditions
- **Subjectivity to impact** – cables installed at the ends of parking bays may be more subject to light (but repetitive) vehicle impacts than those installed at ramps

Plastic coatings may provide a desired architectural appeal (due to color selection), but should not be relied on as the sole method of corrosion protection in environments or service conditions in which they will crack or split, leaving the exposed strand unprotected.

Durability and corrosion protection of the entire system should be addressed by the designer, with requirements clearly listed in the construction documents. The minimum requirements listed in the PTI Specification should be followed.

16.7 DESIGN EXAMPLES

16.7.1 Meeting Pedestrian Requirements

All of the following design examples use 11 cables, spaced at 4 in. center to center, with the center of the first cable positioned 3.5 in. above the floor. Using ½ in. cable results in a 3.5 in. spacing between each cable, and the height of the center of the top cable is 43.5 in.

Assuming the cables have enough prestressing force applied to them to limit sag and deflection, this will meet the requirements relating to opening sizes and total overall height.

16.7.2 Number of Cables Resisting Impact

The International Building Code requirements state that a barrier system must resist an impact load located a minimum of 18 in. from the floor and centered on a 1 sq ft area. Given this criterion and the cable spacing cited above, all of the following examples assume that a total of three (3) cables will resist the vehicle impact.

16.7.3 Example 1:

Total length of cable (L)	= 180 ft
Span length (ℓ)	= 18 ft
Cable grade	= 250 ksi
Yield strength	= 90% MUTS
Cross sectional area (A)	= 0.153 sq in.
Modulus of elasticity (E)	= 28,500,000
Weight of vehicle	= 5,000 lbs
Width of vehicle	= 6 ft
Impact velocity	= 5 mph

The mass of the vehicle is determined by Eq. (16.4):

$$M = \frac{5000}{32.2} = 156 \text{ lb-sec}^2/\text{ft}$$

Select an initial pretensioning force (F_e) of 3000 lbs and calculate the tension upon impact using Eq. (16.3):

$$T = \sqrt{\left(\frac{0.153 \times 28.5 \times 10^6}{180}\right)\left(\frac{156 \times 7.34^2}{3}\right) + 3000^2}$$

$$T = 8768 \text{ lbs}$$

Using this pretensioning force, deflection is calculated using Eq. (16.5):

$$a = \sqrt{\frac{5768 \times 180}{8.72 \times 10^6} + (18-6)\frac{5768 \times 180}{8.72 \times 10^6}}$$

$$a = 1.2 \text{ ft}$$

The yield strength of the cable is 34,425 lbs, which results in a factor of safety against yielding in this example of 3.93. Total load transmitted to the end (anchoring) columns would be:

$$(8768 \times 3) + (3000 \times 8) = 50{,}304 \text{ lbs}$$

Use Eq. (16.6) to check conformance with building code requirements:

$$T = \frac{(6000/3) \times 18}{4 \times 1.2} = 7500 \text{ lbs}$$

Because this result is lower than the value obtained for T using the energy method, no additional steps are required.

To determine the required jacking force, assume the supplier has given a value of ⅜ in. for expected seating loss and use Eq. (16.10) as follows:

$$F_{pj} = 3000 + \frac{0.375 \times 0.153 \times 28.5 \times 10^6}{12 \times 180}$$

$$F_{pj} = 3757 \text{ lbs}$$

16.7.4 Example 2

All of the above data is the same, except span length (ℓ) is increased to 27 ft and the total length (L) is increased to 270 ft.

Using the same initial prestressing force (F_e) of 3000 lbs and solving for T:

$$T = \sqrt{\left(\frac{0.153 \times 28.5 \times 10^6}{270}\right)\left(\frac{156 \times 7.34^2}{3}\right) + 3000^2}$$

$$T = 7366 \text{ lbs}$$

Using this force, deflection is calculated as:

$$a = \sqrt{\frac{4366 \times 270}{8.72 \times 10^6} + (27 - 6)\left|\frac{4366 \times 270}{8.72 \times 10^6}\right.}$$

$$a = 1.69 \text{ ft}$$

In order to limit deflection to 1.5 ft, use a higher initial prestressing force of 5000 lbs and recalculate the resulting tension (T) and deflection (a):

$$T = \sqrt{\left(\frac{0.153 \times 28.5 \times 10^6}{270}\right)\left(\frac{156 \times 7.34^2}{3}\right) + 5000^2}$$

$$T = 8382 \text{ lbs}$$

and deflection is:

$$a = \sqrt{\frac{3382 \times 270}{8.72 \times 10^6} + (27 - 6)\left|\frac{3382 \times 270}{8.72 \times 10^6}\right.}$$

$$a = 1.49 \text{ ft}$$

This limits deflection to 18 in. but total lateral force transmitted to the anchoring columns increases due to the increase in prestressing force applied to the cables.

$(8382 \times 3) + (5000 \times 8) = 65{,}146 \text{ lbs}$

Because the span length exceeds 18 ft, Eq. (16.9) should be used to calculate the minimum amount of prestressing force required to reduce sag to an acceptable value in the 27 ft span.

$$F_e = \left|\frac{27^2 \times 0.544}{(.007 \times 27)/12}\right| / 8 = 3147 \text{ lbs}$$

Because the force being used (F_e = 5000 lbs) exceeds the force required to reduce sag, no further steps are required.

16.7.5 Example 3

This example will use the same spans given in Example 1, but will increase the vehicle weight to 17,000 lbs (delivery vehicle on a plaza level).

The mass of the vehicle is:

$$M = \frac{17{,}000}{32} = 531$$

Use an initial prestressing force of 3000 lbs and solve for T:

$$T = \sqrt{\left(\frac{0.153 \times 28.5 \times 10^6}{180}\right)\left(\frac{531 \times 7.34^2}{3}\right) + 3000^2}$$

$$T = 15{,}492 \text{ lbs}$$

Using this value the resulting deflection is:

$$a = \sqrt{\frac{12{,}486 \times 180}{8.72 \times 10^6} + (18 - 6)\left|\frac{12{,}486 \times 180}{8.72 \times 10^6}\right.}$$

$$a = 1.78 \text{ ft}$$

In this example, the prestressing force would have to be increased to 9000 lbs in order to limit deflection to 1.5 ft, which would increase the total load transmitted to the anchoring columns to 125,000 lbs. Deflection could also be limited to 1.5 ft by adding intermediate anchorages to reduce the total effective length (L) to 90 ft and increasing prestressing force to 4000 lbs. This would reduce the total load transmitted to the columns to 98,000 lbs, but the use of the shorter cable length decreases the factor safety against yielding to 1.6.

Another option would be to decrease cable spacing in the area of impact and use five cables pretensioned to 4000 lbs to resist the impact. This would limit deflection to 1.46 ft and would increase the factor of safety to 2.8.

REFERENCES

16.1 *International Building Code*, IBC 2003, International Code Council, Inc., Falls Church, VA, 2003.

16.2 *Specification for Seven-Wire Steel Strand Barrier Cable Applications*, Post-Tensioning Institute, Phoenix, AZ, 1988.

16.3 Presswalla, H., "Designing Prestressed Barrier Cables," *Concrete International*, May 1989, pp. 67-73.

16.4 Wagner, C. R., *Stream-Gaging Cableways*, USGS, Department of the Interior, Reston, VA, 1995.

NOTATION

A Cross sectional area of cable, in^2

a Cable deflection, ft

b Width of vehicle, ft

E Modulus of elasticity of cable, lb/in^2

F_e Final effective prestressing force, lbs

F_{pj} Jacking force, lbs

g Acceleration due to gravity (32.174 ft/sec^2)

L Total length of cable, anchor to anchor, ft

ℓ Span of cable between supports, ft

M Mass of vehicle, lb-sec^2/ft

N Number of cables resisting impact

P Applied load, lbs

s Sag in cable due to self weight, in.

T Cable tension on impact, lbs

V Velocity of vehicle, ft/sec

w Weight of cable per foot, lbs

PRESTRESSED CONCRETE UNDER DYNAMIC LOADS AND FATIGUE

17.1 INTRODUCTION

17.1.1 General

In the context of this chapter dynamic means that loading and structural response are changing with time. Dynamic effects on concrete structures fall into three categories:

- Load cycling under service loads, which mainly affect the fatigue resistance of structures
- Impact loads causing tensile stresses under service loads
- One-time dynamic events, subjecting structures to extreme loading conditions

The first category includes fatigue of prestressed concrete and its constituents under service load conditions. This reflects on the importance of fatigue considerations for the design of dynamically loaded structures. In post-tensioned concrete and cable-stayed bridges, fatigue and fracture loads relate to repetitive vehicular live loads and dynamic response caused by a design truck per Ref. 17.1. Ref. 17.1 defines a new fatigue limit state in the 1998 edition. In building design, fatigue can be caused by machinery, or in the case of parking structures, by vehicular traffic. Fatigue of prestressed concrete and its materials are discussed in this chapter.

The second category applies to members subjected to the impact of wheel load from moving vehicles, and the impact load on prestressed concrete piles when they are driven into soft soil. Dynamic effects due to moving vehicles may be attributed to two sources:[17.1]

- Localized dynamic effects caused by the interaction of wheel assembly on the riding surface discontinuities, such as deck joints, cracks, potholes, delaminations, etc.
- Dynamic response of the bridge as a whole to passing vehicles, which may be due to long undulations in the roadway pavement, such as those caused by settlement of fill, or resonant excitation as a result of similar frequencies between the vehicle and the bridge.

Reinforced concrete members are typically considered to be cracked under the application of service loads; post-tensioned members can be either designed to be cracked or uncracked, depending on the specified permissible tensile stresses.

Not much information is available on prestressed concrete subjected to extreme impact and blast loads. For additional information on structural dynamics in general, see Refs. 17.2, 17.3, 17.4, and 17.5.

It should be noted that the terms "full" and "partial" prestressing used in this chapter have been phased out of ACI 318-02, which is currently based on a unified sliding-scale definition ranging from uncracked to a cracked state. However, these terms are still being used by AASHTO LRFD Specifications, specifically with regard to fatigue strength, and hence will be used in this chapter.

17.1.2 Fatigue of Unbonded and Bonded Tendons

Prestressed concrete is the material of choice for structures subjected to dynamic loads. Such structures are subjected to load cycling, vibrations, and small impact loads. The stress cycling associated with such loads can cause the material to fatigue.

When post-tensioned concrete members with bonded tendons are designed to be uncracked under full service loads (i.e., the actual tensile stress is less than the modulus of rupture of the concrete), stress fluctuations along the length of the post-tensioning steel are small. Such members, therefore, provide exceptional fatigue resistance. Fatigue considerations become an issue mainly for members designed to crack under the application of full service loads. The tendon elements at crack locations could experience, under unfavorable conditions, potential fatigue. The amount and extent of fatigue will depend on several conditions, such as crack depth, range of stress changes, and the number of load cycles. (For a detailed discussion on the various classifications of concrete with respect to cracking, see Section 17.5.2.) Fatigue to end anchorages of bonded tendons is normally not a critical design consideration because of the additional support provided by bond along the entire length of the tendon. For couplers, on the other hand, fatigue considerations apply because of the concern of the discontinuity of the prestressing force. Hence, placing couplers at points of critical stress should be avoided.[17.22]

In general, stress changes in unbonded tendons are even smaller than those present in bonded tendons, regardless of whether the concrete section is uncracked or cracked. They become significant in the presence of specific variables which include: 1) steel quality (i.e., ductility, notch sensitivity); 2) pitch of outer wires; and 3) surface conditions (such as rust which would reduce the fatigue life).[17.21] However, lacking the redundancy provided by bond, their anchorages, and specifically speaking, the strand-wedge connections, become the most sensitive part of the tendon in regard to fatigue resistance.

Stress cycling, and the associated strain cycling, may cause fretting-fatigue damage, thus increasing strand fatigue where strand is in contact with other materials. Fretting is more likely to occur in external tendons, which are subjected to substantial stress changes at critical locations. These locations include deviators, entrance or exit points of strand flares, anchorages, and toes of wedges. Fretting damage is also possible between a failed strand wire and adjacent wires, or between peripheral wires and the central wire. Care is needed in detailing anchorages and deviator hardware to avoid fretting fatigue.

17.1.3 One-Time Dynamic Load Conditions

Prestressed concrete structures subjected to one-time accidental impact or blast loads have not been specifically investigated. However, it is likely that the positive characteristics of prestressed concrete would be beneficial under accidental impact and blast loads. Prestressing raises the threshold of loads that can be resisted without cracking. After cracking, their behavior is similar to that of reinforced concrete members, especially for bonded tendons. Post-tensioning enhances elastic recovery and improves structural integrity by providing continuity and redistribution of the inelastic moments.

Structures subjected to blast or impact loads are typically designed to survive one design loading event. Cracking of concrete and yielding of tendons and reinforcement are usually acceptable.

The damping of prestressed concrete is similar to that of reinforced concrete. The following damping values can be used for prestressed concrete members:[17.6]

- Uncracked concrete — less than 1%
- Non-visible microcracking — about 2%
- Considerable cracking — 3% to 5%

17.2 DYNAMIC LOADS

17.2.1 Types of Dynamic Loads

Dynamic loads may arise from many sources. They can be separated into four groups:

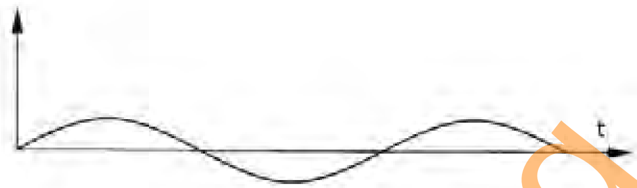
1. Repeated loads
2. Vibrations
3. Impact
4. Blast

The first three conditions are discussed in this chapter. Blast loads and blast design considerations are beyond the scope of this chapter.

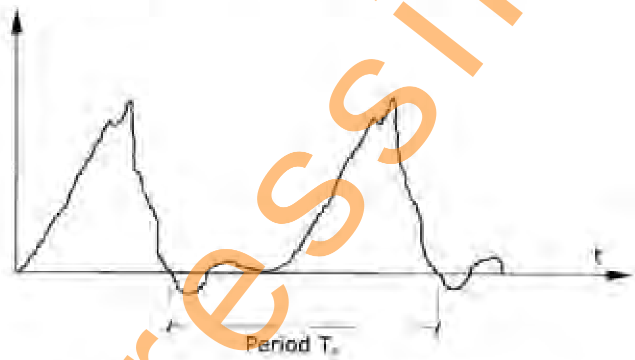
17.2.2 Repeated Loads

Repeated loads re-occur over the service life of the structure. They can be slow, fast, periodic or random. Their magnitudes may be constant, follow a certain pattern, or vary at random.

For design purposes, repeated loads are normally treated as static loads. If the loads repeat many times over the life of a structure, the structural materials could be subjected to fatigue. (See Section 17.4.)

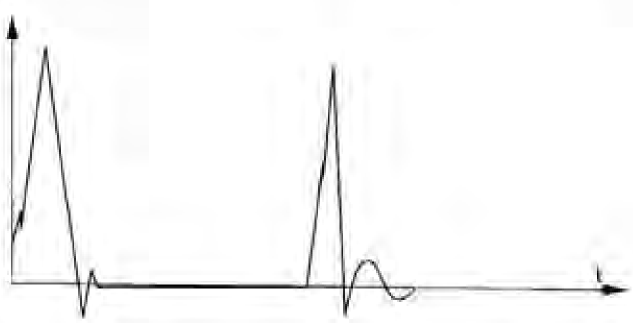


a) Harmonic vibration (rotating machines, wind, water flow)



b) Periodic vibration (machines, human motion, waves)

c) Transient vibration (earthquake, rail and road traffic)



d) Impact (traffic, machines, falling objects)

Fig. 17.1 Typical Vibration Patterns

Live load changes are the most common examples for repeated loads. To resist such dynamic loads, a fatigue life of 2×10^6 cycles is often required. In perspective, 2×10^6 cycles are equivalent to 55 load changes for each 24-hour period for a duration of 100 years. Setting the limit to 2×10^6 cycles is convenient because it approximately represents the fatigue limit of construction materials. (For definitions of fatigue life and fatigue limit see Section 17.4.2.)

Wind and wave loads are examples of repeated loads, which may accumulate 10×10^6 cycles (or more) over the anticipated life of structures, but at smaller load amplifications.

17.2.3 Vibrations

Vibrations typically are periodic, repetitive motions. They can also be one-time transitory events, like ground motions caused by earthquakes. Fig. 17.1 shows a few typical types of vibrations.

Vibrations can be associated with oscillating operating equipment such as compressors, for instance. Post-tensioning of compressor foundations has proven to counteract fatigue effects, resulting in reduced concrete cracking and deterioration.

Environmental loads, such as wind or water flow, can cause vibrations of structures or individual structural elements with low damping characteristics. Such vibrations are not limited to circular shapes. Wind-induced vibrations can occur in slender structures such as chimneys and towers. Cable vibrations are common in cable-stayed bridges; this is discussed in detail in Chapter 13.

The extent to which vibrations (other than earthquake vibrations) affect the performance of prestressed concrete structures is a function of user comfort and fatigue. Low amplitude vibrations can affect the comfort of the occupants. User comfort may become a consideration in cable-supported bridges under pedestrian traffic, thin long span slabs and slender beams. Fig. 17.2 provides information on human sensitivity to vertical vibratory motion.[17.5] Fatigue is discussed at length in Sections 17.4, 17.5, and 17.6.

Vibration frequencies are measured in cycles per second (Hz). Very large numbers of cycles accumulate, subjecting the materials to fatigue. However, strains and/or deformations caused by vibrations are typically small and the related stress changes are also small. Consequently, the materials are normally not subjected to severe fatigue.

It should be noted that under resonance conditions vibration amplitudes can become large and cause significant damage. Resonance occurs when the exciting load contains sufficient energy at a frequency that is close to the natural frequency of the structure.

Vibrations are classified by amplitudes and frequency. The dynamic characteristics of the responding engineering structures are their natural frequencies, and the mode shapes of their vibrations. For the response of structures to

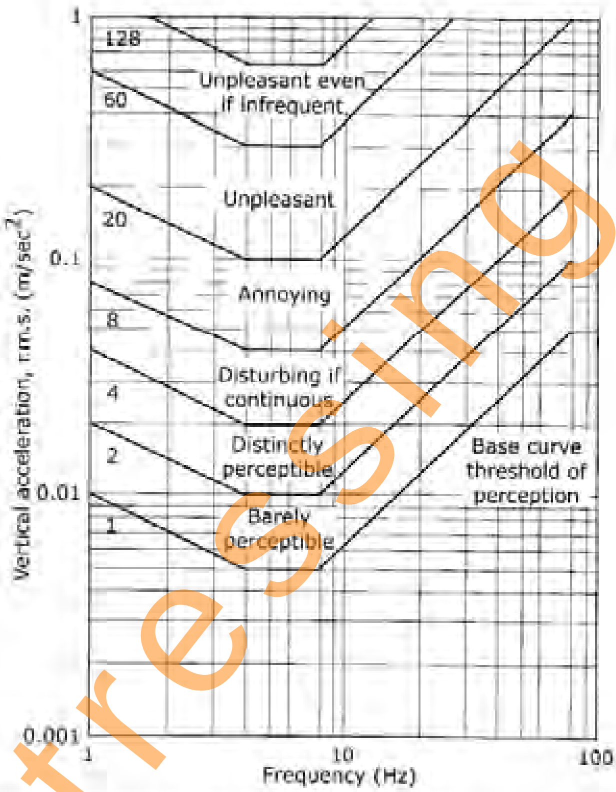


Fig. 17.2 Human Sensitivity to Vertical Motion[17.5]

vibrations see Section 17.3. Refs. 17.7, 17.8, 17.9, and 17.10 provide additional information on vibrations.

17.2.4 Impact

In this chapter impact phenomena are assumed to be non-destructive, making them essentially short-duration vibratory events. Destructive impacts and their structural responses are more related to blasts and are beyond the scope of this chapter.

Recurring impacts can be caused by a wide range of short-duration dynamic load pulses and typically have relatively low energy levels, such as hammer action from a forging press. Low energy impacts also occur if heavy trucks drive over damaged or misaligned expansion joints or other obstructions.

Fully prestressed concrete structures that are designed to be uncracked normally have low damping, and impact loads can cause high frequency impulses and vibrations. If relatively low energy impacts recur frequently it could cause local deterioration of concrete. However, it is not likely to cause tendon fatigue because tendon stress changes are small.

Most of the research has not been targeted at prestressed concrete structures. However, one can reason from relevant observations of non-prestressed members that prestressing

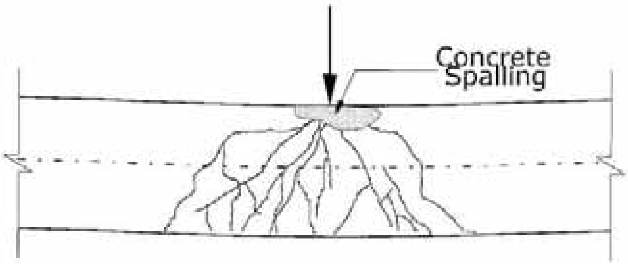


Fig. 17.3 Typical Failure Cone Under Impact Load

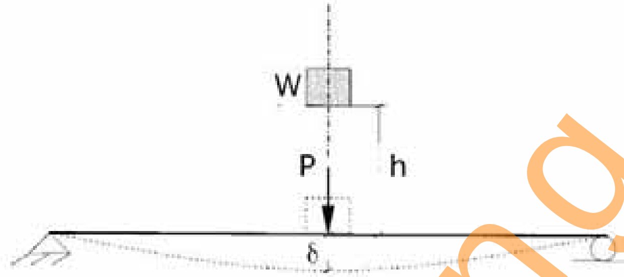


Fig. 17.4 Equivalent Static Load for Weight Falling on Elastic Beam

will be beneficial in reducing the destructive impact. The following practical examples of prestressed concrete members under severe fatigue conditions illustrate the beneficial effect of prestressing:

- Good long-term performance of millions of prestressed railroad ties has demonstrated the beneficial effect of prestressing on members subjected to large numbers of repetitive axle loads and related traffic impacts.[17.11]
- Prestressed concrete piles driven by hammer action into the ground are examples where the low damping of prestressed concrete is beneficial. Most of the impact energy remains available to drive the piles into the ground. The precompression of the concrete also prevents material deterioration and cracking during handling.

Prestressing conceivably could also increase the resistance of concrete members for withstanding impacts of falling objects (such as airplanes and bombs) by changing the angle of the otherwise 45° cone-shaped tension failures spreading from the impact point (Fig. 17.3).

For design purposes, impact loads are treated as static loads using appropriate impact amplification factors specified by design codes, or obtained from dynamic analysis. According to classical mechanics an amplification factor of 2 represents the condition where a load W is suddenly applied to an elastic beam causing a deflection δ (see Fig. 17.4).

For the condition shown in Fig. 17.4, one derives the dynamic amplification factor from the requirement that the potential energy of the weight W must be equal to the work performed by the equivalent static load P on the deformed beam:

$$W \cdot (h + \delta) = P \cdot \delta/2 \qquad (17.1)$$

With $\delta = \kappa\, \delta_o$ representing the deflection due to the equivalent static load P and δ_o being the deflection due to the weight, W, one obtains from Eq. (17.2):

$$\kappa = 1 + \sqrt{1 + 2 \cdot h/\delta_o} \qquad (17.2)$$

For h = 0 one obtains $\kappa = 2$ and hence:

$$P = 2 \cdot W \qquad (17.3)$$

Refs. 17.11, 17.12, 17.13, 17.14, and 17.15 provide additional information on impact.

AASHTO LRFD Specifications[17.1] refers to impact loads as dynamic allowance. The Specifications state that the "static effects of the design truck or tandem, other than centrifugal and braking forces, shall be increased by the percentage specified." The factor to be applied to the static load is given as (1 + IM/100). This allowance, per AASHTO Specifications, shall not be applied to pedestrian loads or to the design lane load.

17.3 DYNAMIC RESPONSE

17.3.1 Basics of Dynamic Structural Response

The subject of dynamic response described herein pertains to vibrations and non-destructive impacts. The latter one, as mentioned earlier, can be considered a special case of the vibration problem. The following considerations, however, do not apply to the structural response of large impact and blast effects.

For structural purposes, dynamic implies that external forces P(t) and the reacting internal forces—their stresses, strains, and deflections—vary according to a given time function f(t). The time-varying deflections are associated with accelerations of the mass points throughout the structure. The forces of the moving masses are then resisted by inertia forces, which cause dynamic stresses and strains, as illustrated in Fig. 17.5. As part of this process, a portion of the energy inherent in the moving masses is absorbed by damping.

At critical damping, a structure responds to any vibratory excitation by returning to the static condition without oscillation. The damping coefficient is defined as the ratio of the actual damping to the critical damping.

17.3.2 Dynamic Load Directly Applied to Structure

In a typical dynamic event, the dynamic load is directly applied to the structure. Dynamic equilibrium then requires that the applied external dynamic forces $P(t)$ are balanced by the sum of the inertia forces F_I, elastic forces F_E, and damping forces F_D:[17.4]

$$P(t) = F_I(t) + F_E(t) + F_D(t) \tag{17.4}$$

For the idealized single mass system shown in Fig. 17.6, Eq. (17.4) can be rewritten as shown by Eq. (17.5), which is the familiar differential equation for the horizontal vibratory motion of a single degree of freedom system that is excited by the force $P(t)$:

$$P(t) = M \cdot \ddot{v}(t) + C \cdot \dot{v}(t) + K \cdot v(t) \tag{17.5}$$

Where:

- M = mass
- $P(t)$ = external vibratory load (as a function of time)
- $v(t)$ = deflection (relative to base)
- $\dot{v}(t)$ = velocity
- $\ddot{v}(t)$ = acceleration
- C = damping coefficient (ratio of damping to critical damping)
- K = stiffness

17.3.3 Structure Responding to Ground Motion

The response of a structure to a seismic event differs from the above-described condition because the dynamic excitation is caused by the vibratory ground motion. The excitation is applied to the base of the structure; there are no external forces applied to the structure. Instead, the inertia forces of the structure's distributed mass and its elastic properties cause the structure to vibrate in response to the ground motion. With $P(t) = 0$ one obtains from Eq. (17.4):

$$-F_I(t) = F_D(t) + F_E(t) \tag{17.6}$$

For seismic vibrations the inertia forces depend on the relative accelerations of the masses to the structure's base plus the ground acceleration, i.e., $\ddot{V}_t = \ddot{V} + \ddot{V}_g$. For the idealized single degree of freedom systems shown in Fig. 17.7, Eq. (17.6) can then be written as follows:

$$-M \cdot \ddot{v}_g(t) = M \cdot \ddot{v}(t) + C \cdot \dot{v}(t) + K \cdot v(t) \tag{17.7}$$

Where:

- $\ddot{v}(t)$ = acceleration of mass relative to base
- $\ddot{v}_g(t)$ = ground acceleration
- $\ddot{V}_t$ = total acceleration of mass

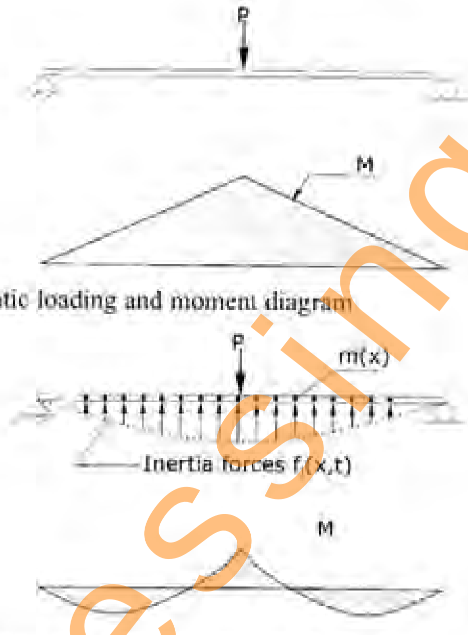


Fig. 17.5 - Difference Between Static and Dynamic Loading[17.4]

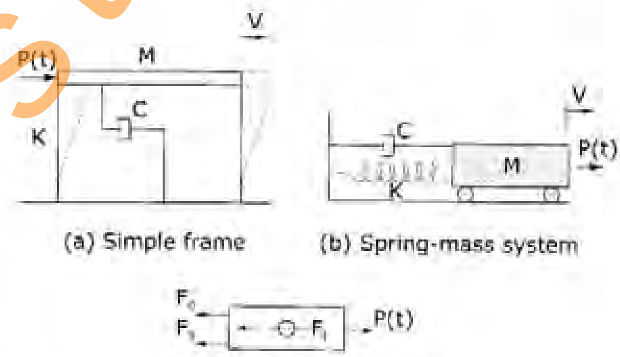


Fig. 17.6 Single Degree Freedom Vibratory System Excited by an External Force P(t)[17.4]

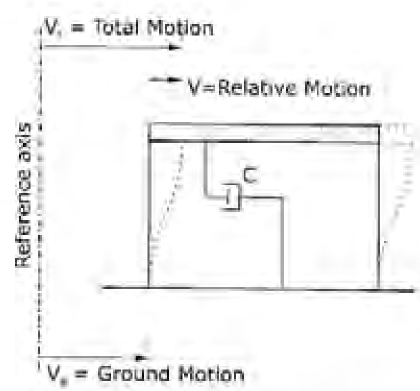


Fig. 17.7 Single Degree Freedom System Excited by Ground Motion[17.4]

It is interesting to note that Eqs. (17.5) and (17.7) are essentially identical. The negative sign on the left side indicates that when a structure is subjected to a vibratory ground motion the exciting force is the inertia force of the structure's mass.

For more information see Ref. 17.16, which contains contributions from a number of experts in this field.

17.3.4 Response Spectra

For design applications, one is interested in the response spectra of acceleration, velocity, and displacement of the vibratory event under investigation. In a response spectrum, the maximum values of the characteristic response parameter (i.e., acceleration, velocity, and displacement) for a single degree of freedom system are plotted as functions of the frequency of the responding structure and its damping coefficient. The three spectra are often shown together in a single tripartite logarithmic response spectra for various damping ratios. An example of such combined response spectra for an earthquake is shown in Fig. 17.8.

Knowing the natural frequency of a structure and its damping ratio, one obtains from the response spectra the acceleration, velocity, and displacement at the mass centroid of the structure. One can then calculate the shear forces, bending moments, and the related stresses in the structural members. For the particular case of determining a structure's earthquake resistance the maximum base shear force due to horizontal ground motion is of primary interest.

17.4 FATIGUE OF PRESTRESSED CONCRETE MATERIALS

17.4.1 Introduction

The fatigue resistance of prestressed concrete depends on the fatigue resistance of its constituents, namely concrete, tendons, and non-prestressed reinforcement. Structural failure occurs when the weakest constituent fails. Experience has shown that concrete fatigue in compression will not occur if the stresses are kept within allowable stress limits (see also Section 17.4.5.1).

As further discussed in Section 17.5, in a fully prestressed member (i.e., no crack opening and closing) tendon stress changes are small and well below the fatigue resistance of the tendons. In fact, the fatigue resistance of fully prestressed concrete is superior to reinforced concrete or structural steel.

Fatigue considerations are important for structures with bonded tendons in which concrete cracks open and close under fluctuating loads. In this chapter such members are considered to be only partially prestressed (see also Sections 17.5.4 and 17.5.6.3).

Fatigue considerations are also important for highly stressed tendons in cable-stayed structures, which are typ-

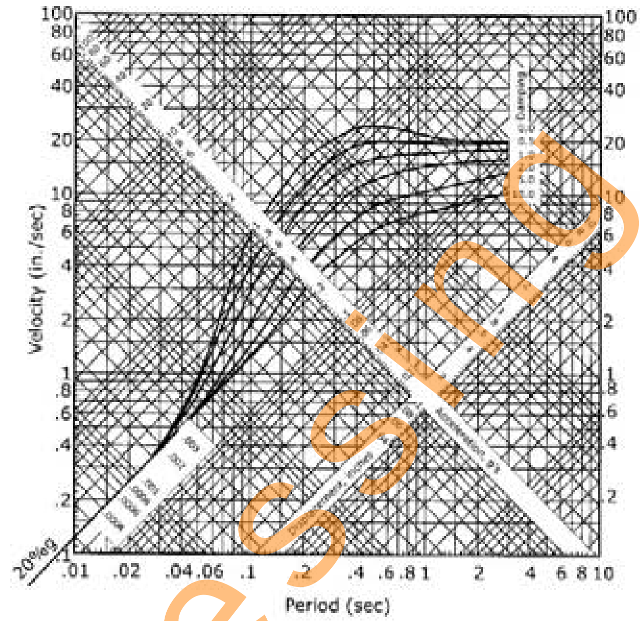


Fig. 17.8 Example of Combined Earthquake Response Spectra[17.16]

ically subjected to higher live load variations than other prestressed concrete structures. Fatigue requirements for cable stays are discussed in Chapter 13 and Ref. 17.17.

Fatigue properties of the prestressing steel can be obtained from the appropriate Wöhler, Goodman, and/or Smith diagrams. The fatigue-reducing effects of fretting also need to be considered.

17.4.2 Definitions

- Fatigue is the process of progressive and irreversible deterioration of a material subjected to repetitive stresses.
- Fatigue life is the number of cycles needed to fail a material under a given cyclic stress range.
- Fatigue limit or endurance limit is the stress range, which can be sustained for an indefinite number of cycles.
- Fatigue strength is the upper stress limit, which can normally be sustained for two million cycles.

17.4.3 Wöhler Diagram

A Wöhler diagram (Fig. 17.9) shows the fatigue life of a material for different cyclic stress ranges. It is also called S-N Diagram, where S stands for the stress range and N represents the number of stress cycles to failure.

The Wöhler diagram establishes the fatigue limit. That limit is reached where the Wöhler curve becomes asymptotic to the horizontal axis, indicating that with continued stress cycling further strength reduction becomes insignif-

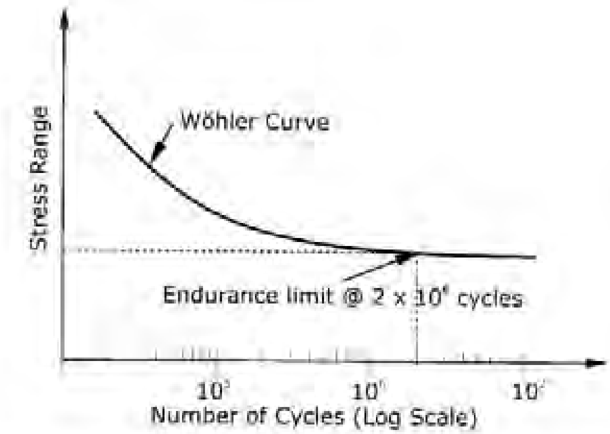


Fig. 17.9 Wöhler Diagram (S-N Diagram)

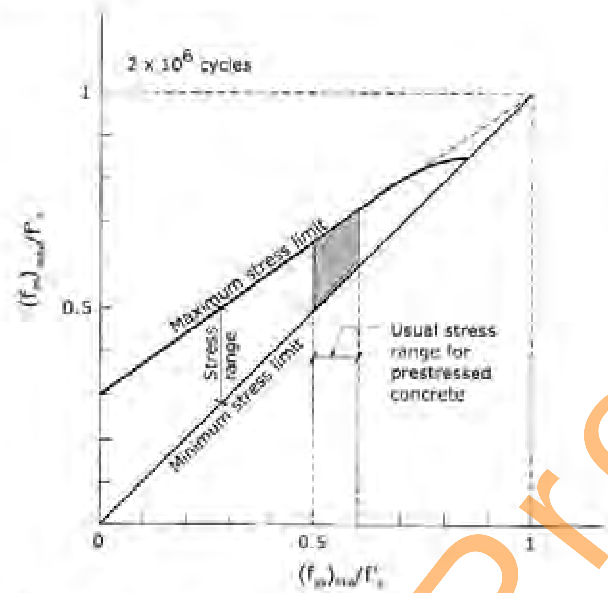


Fig. 17.10 Goodman Diagram

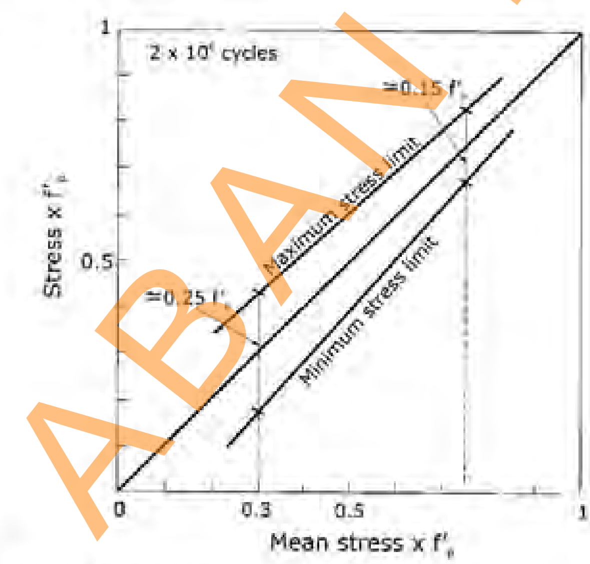


Fig. 17.11 Smith Diagram

icant. The number of stress cycles is shown on the horizontal axis on a logarithmic scale.

Typically, Wöhler diagrams are prepared for cyclic stress ranges between zero stress and a series of upper stresses. However, diagrams can also be prepared for constant lower stresses other than zero. For instance, a lower tendon stress of $0.60 \cdot f'_p$ is well suited for presenting fatigue data for prestressed concrete applications. For cable stays considerably lower stresses are applicable ($\approx 0.40 \cdot f'_p$) depending on design requirements.

Late 19th century Wöhler diagrams for typical construction steels indicated that for practical purposes fatigue limits approached at about two million cycles. These results are the historical basis for the general practice to establish the fatigue life of construction materials at two million cycles, even if no distinct fatigue limits have been established.

17.4.4 Goodman and Smith Diagrams

It is generally agreed that the relative magnitude of the stress change under load is the most important variable influencing the fatigue life of materials.

In the Goodman diagram (Fig. 17.10), the cyclic stress range, which causes fatigue failure at N cycles, is plotted on top of linearly increasing sustained lower stresses. The Smith diagram (Fig. 17.11), which is commonly used in Europe, shows the same test data plotted around the mean stress. The two diagrams normally are based on a fatigue life of two million cycles and do not establish fatigue limits. Based on the Wöhler curve, further strength deterioration with increasing cycles is assumed to be small and covered by safety margins applied to acceptable stress ranges and those included in the loading assumptions.

The Goodman diagram is well suited for prestressing applications. It allows the designer to select, for a given cyclic stress range, the permissible sustained tendon stress after all losses. That feature permits, if necessary, the use of smaller initial tendon stresses than the usual $0.7 \cdot f'_p$.

It should be noted that preparing a fatigue diagram requires testing of numerous samples under carefully controlled laboratory conditions. It is not feasible to produce such diagrams for each manufactured lot of prestressing steel. Instead, a manufacturer establishes representative fatigue data for his production material, which should cover the range of normal manufacturing tolerances.

It is also important to realize that the diagrams are for the prestressing steel material and do not take into account fatigue strength reducing effects due to: 1) stress raisers at strand-wedge or bar-couplers connections; 2) bending stresses; 3) side pressures at angle changes; and 4) fretting. (See Sections 17.4.6 and 17.4.8.)

The Wöhler and Smith diagrams for a particular strand and cable-stay system are shown in Ref. 17.17.

17.4.5 Concrete Fatigue

17.4.5.1 Concrete Fatigue in Compression

Concrete is very strong in compression and has favorable fatigue characteristics. It has been found that concrete can endure:[17.18]

- Long-term compressive stresses of about 90% of its short-term compressive cylinder strength
- Fluctuating compressive stresses between 0% and 50% for about ten million cycles; however, a fatigue limit for concrete has not been found.

It is generally accepted that fatigue failure due to compressive stresses will not occur if the stresses during normal operations are about 0.45 f'_c. Eq. (17.8), established by the ACI Committee 215 on Fatigue,[17.19] gives guide values for acceptable stress ranges up to 10 million cycles:

$$\Delta f = 0.4 \cdot f'_c - f_{min}/2 \qquad (17.8)$$

Where:

f_{min} = the minimum stress under cyclic load

17.4.5.2 Concrete Fatigue in Tension

Plain concrete is weak in tension under static loads and even more so under dynamic loads. This is especially true for flexural tensile stresses. This fundamental fact needs to be remembered when one evaluates prestressed concrete members for their fatigue resistance, especially so because the fatigue resistance of bonded tendons significantly reduces at crack locations, as discussed in Section 17.5.4.

17.4.6 Fatigue of Pretressing Steel

17.4.6.1 General Information

Prestressing materials (strands, wires, bars) are usually manufactured to meet minimum strength and ductility requirements discussed in Chapter 4. Materials meeting such requirements usually have adequate fatigue resistance for post-tensioning applications. However, it is noted that fatigue properties depend on numerous manufacturing variables, which cannot be quantified in generally applicable specifications. It is possible that similar materials may have significantly different fatigue properties.

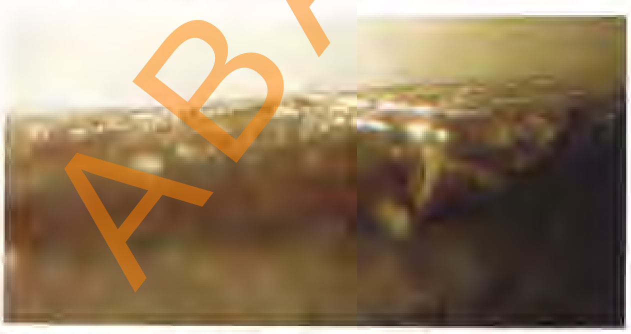


Fig. 17.12 Longitudinal Strand Profile (Amplified 1000 Times)

Differences in fatigue properties depend on numerous manufacturing variations affecting metallurgical, ductility and surface conditions. The effect of surface irregularities on fatigue resistance can be visualized by looking at Fig 17.12. The figure shows the greatly magnified surface irregularities of an apparently smooth prestressing steel wire. The surface irregularities cause stress raisers, which can lead to fatigue fractures. The extent of varying surface conditions has been established by comparable bond tests on strands from different manufacturers, rendering bond values varying by a factor of eight. Similarly, it is well known that light surface rust reduces the fatigue strength of prestressing strand.[17.20,17.21]

Goodman, Smith, and Wöhler (S-N) diagrams (see Section 17.4.4) for commercially available prestressing materials are useful for identifying qualitative differences between materials from different sources. For quantitative evaluation, one must keep in mind that the diagrams apply to laboratory conditions. In addition, permissible stress ranges require safety margins for fretting, side pressures, bending and anchorage effects. Such effects are unavoidable in real structures.

17.4.6.2 Fatigue Properties of Prestressing Materials

As mentioned above, fatigue properties of prestressing steels depend on numerous manufacturing variables. Therefore, only general guidance is given below. If more accurate information is needed one must refer to the particular manufacturer's fatigue data.

17.4.6.2.1 Fatigue Properties of Strands

Various researchers have performed fatigue tests on strand samples from different manufacturers. The laboratory tests are typically performed in air on short and straight test samples. Such tests do not take into account stress range or fatigue life reducing effects due to: 1) bending; 2) notches at strand-wedge connections; and 3) fretting effects at concrete crack locations. These effects must be covered by allowing for adequate safety margins. (Fatigue life reducing effects on bonded tendons are discussed in Section 17.4.8.2.)

Naaman[17.18] evaluated a large number of in-air strand fatigue tests. The test data are plotted in Fig. 17.13 and lead to Eq. (17.9). The equation permits the determination of fatigue life of a typical 270 ksi (1860 MPa) strand for a given stress range or vice versa:

$$\frac{\Delta f_p}{f'_p} \approx 0.87 - 0.123 \cdot \log N \qquad (17.9)$$

Where:

f'_p = nominal ultimate tensile strength of strand

Δf_p = cyclic stress range

N = number of stress cycles

At the time of initial stressing, tendons usually are anchored off at $0.70 \times f'_p$. After all losses due to friction, creep, shrinkage, and relaxation, the remaining sustained stress is normally less than $0.60 \times f'_p$. Therefore, of main practical interest is the cyclic stress range Δf_p that can be endured for the customary fatigue life of two million cycles, while maintaining a lower stress of about $0.6 \times f'_p$.

For such conditions one obtains from Eq. (17.9) an approximate stress range:

$$\Delta f_p \approx 0.10 f'_p \tag{17.10}$$

Paulson[17.22] evaluated different sets of in-air strand fatigue tests. The test data are plotted in the bi-logarithmic graph of Fig. 17.14 and lead to Eqs. (17.11) and (17.12):

$$\log N \approx 11.0 - 3.5 \cdot \log(\Delta f_p) \tag{17.11}$$

Eq. (17.11) is valid only for U.S. units. Expressing the above equation in the normalized format of Eq. (17.9) one obtains for the typical 270 ksi (1860 MPa) strand, which is valid for:

$$\frac{\Delta f_p}{f'_p} \approx \frac{1}{f'_p} 10^{(3.1429 - 0.2857 \log N)} \tag{17.12}$$

For the fatigue life of 2×10^6 cycles, one obtains from Eq. (17.12) $\Delta f_p \approx 0.08 f'_p$. Paulson's in air test data are also shown in Fig. 17.15, which permits to deduct the fretting effects on curved post-tensioned tendons at concrete crack locations, as further discussed in Section 17.4.8.2.

17.4.6.2.2 Fatigue Properties of Wires

Fatigue properties for individual wires are somewhat better than that of 7-wire strands. The wires are straight and avoid the fatigue strength reducing effects of stranding and inter-wire fretting. On the other hand, prestressing steel wire diameters are larger than those of the individual strand wires. This has a negative influence because of the more pronounced Poisson's effect. In the absence of comparable test data, it is reasonable to assume that the above equations are also useful for estimating the fatigue strength of prestressing steel wires.

17.4.6.2.3 Fatigue Properties of Bars

Available fatigue data for uncoupled prestressing bars suggests that Eqs. (17.9) and (17.10) also give reasonable estimates for their fatigue resistance, even if bars and strands have otherwise little in common. Fretting fatigue between bars and duct is unlikely because bars are typically straight, with each bar in its own duct.

It also should be realized that even greater metallurgical and manufacturing differences exist between different bar types than for strands. For instance, surface contours differ

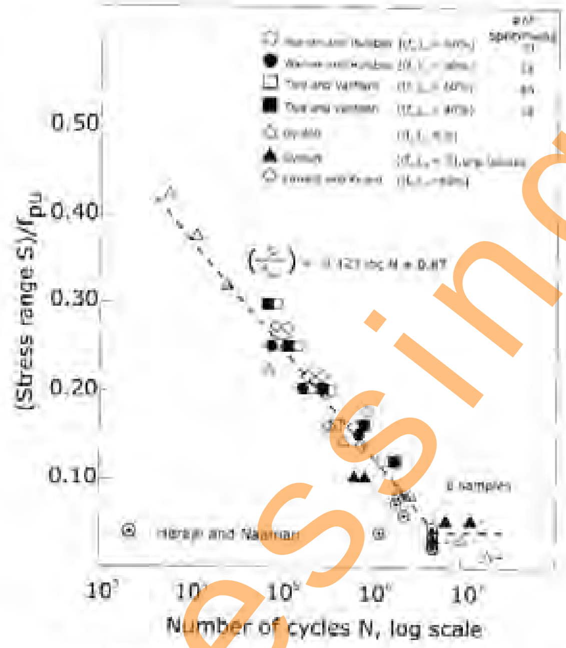


Fig. 17.13 Naaman, Typical S-N Curve for Prestressing Strand[17.18]

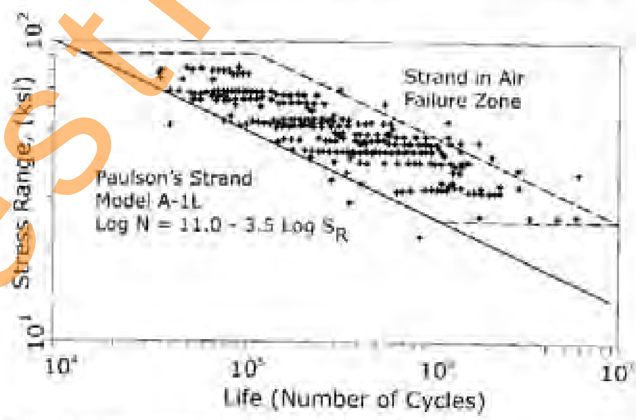


Fig. 17.14 Paulson's In-Air Fatigue Data for Strand[17.22]

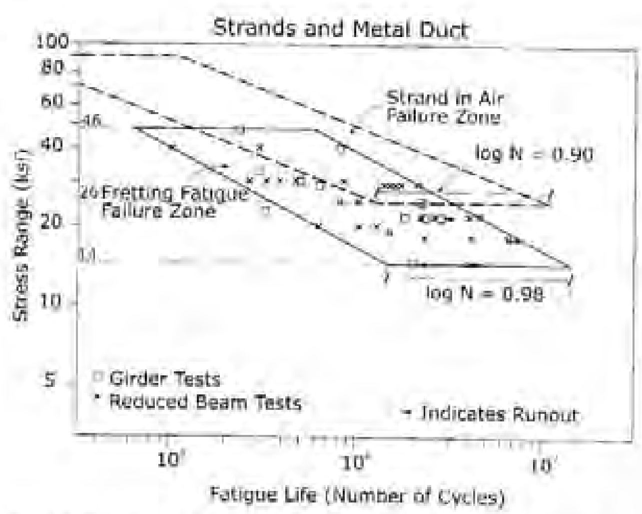


Fig. 17.15 Fatigue of Curved Post-Tensioned Tendons (Strands and Metal Duct)[17.22]

between hot rolled smooth and deformed bars and cold drawn bars. Also, bar diameters have an effect on their fatigue resistance and stress raisers at coupler locations and at anchorages reduce their fatigue resistance. Reliable supplier information is, therefore, essential when evaluating critical applications.

The ultimate tensile strength of prestressing bars is typically only 55% to 60% of that for strands. Therefore, for equal ratios of $\Delta f_p/f'_p$ sustainable stress ranges in bars (in terms of unit stresses) are significantly smaller than for strands. The smaller stress ranges for rolled bars reflect their rougher surface conditions compared to those for drawn wires. Also, Poisson's effect-related triaxial stresses increase with the diameter and have a negative impact on fatigue properties.

17.4.6.3 Fretting

Fretting is a friction-induced fatigue phenomenon. It occurs as stress changes cause small relative displacements (slip) between contacting surfaces. At the presence of lateral contact pressure, the slip changes produce a kind of rubbing effect, resulting in micro-cracks that can eventually lead to fatigue fractures. It has been shown that extremely small slip amplitudes—as low as 5×10^{-3} mm (2×10^{-4} inch)—degrade the fatigue resistance of carbon steel. As the slip increases, the fatigue life reduces until the slip reaches a critical value. Beyond this value fretting fatigue rapidly becomes less severe because the increasing slip amplitudes tend to wear away the previously formed fatigue-initiating surface cracks.[17.22,17.23]

The severity of the fretting problem increases with stress level, side pressure, hardness of contacting material, and number of cycles. Potential for fretting fatigue of prestressing strands exists at the following locations:

- Where strands enter wedges
- Where strands bear against wedge plate holes (depending on angle change of strand from duct to anchorage, i.e., more severe for short transitions)
- At deviation points and saddles (external tendons)
- In curved bonded tendons at crack locations (internal bonded tendons)
- Between a broken strand wire and adjacent stressed wires
- Between peripheral wire and central wire at crack locations (due to twisting and untwisting under load change and due to stress and strain differences between inner and outer wires)
- At contact points of crossing strands or wires

It should be noted that fretting could lead to significantly reduced fatigue life or stress ranges from those established under controlled laboratory conditions. For curved bonded tendons at cracks reduction of up to 50% may occur. This aspect is further discussed in Section 17.4.8.2.

The fretting mechanism is partly illustrated in Figs. 17.16 and 17.17 and explained as follows:[17.22]

- At the presence of moderate lateral forces (e.g., radial forces in curved tendons) extremely high contact pressures can develop at rough material interfaces, causing localized yielding.
- Cyclic displacements (slip) rapidly abrade the oxide film normally present on steel surfaces and small material particles can get dislodged (fretting debris). Without the protective oxide film, the highly laterally stressed material asperities fuse (cold welding).
- With each slip cycle, the cold welded contact areas are destroyed, and new ones are created. This process is the driving force for surface wear and crack initiation.
- Fretting fatigue is frequently accompanied by fretting debris. It includes corrosion products due to the dramatically accelerated corrosion at the exposed and very reactive contact surfaces.
- The crack propagation rate depends mostly on the magnitude of the fluctuating axial stress range, as in ordinary fatigue.

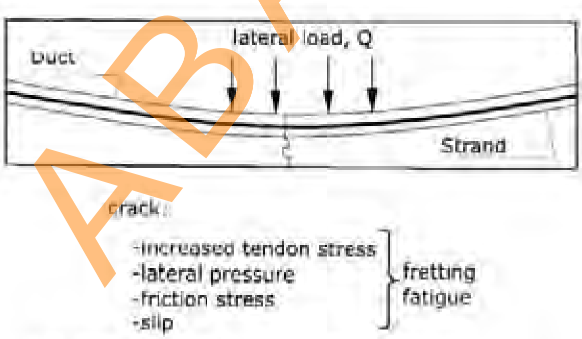


Fig. 17.16 Fretting in Post-Tensioned Beam[17.24]

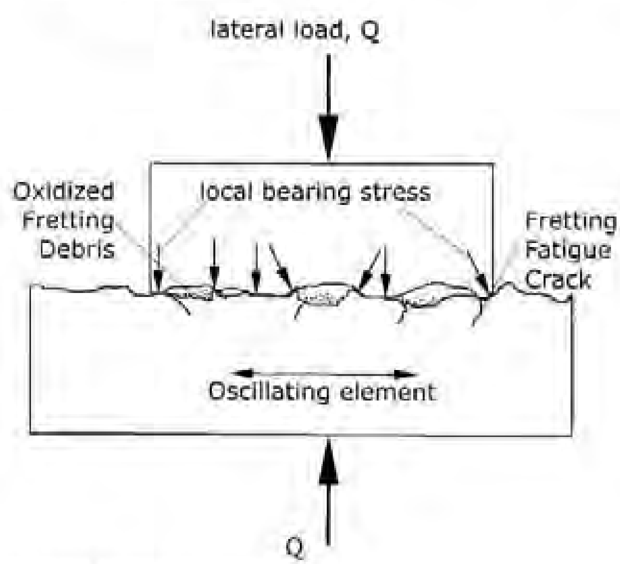


Fig. 17.17 Fretting Mechanism[17.22]

17.4.6.4 Galvanizing

Galvanizing has been shown to slightly improve the fatigue resistance of wires and strands. This may be due to the lubricating effect of the zinc coating, which reduces surface damage due to friction and fretting. Also, the galvanizing temperature partially stress relieves and anneals the steel, which has beneficial effects on the fatigue strength. Galvanizing also reduces the ultimate strength of the material, unless the wires are drawn through the final die after galvanizing.

17.4.7 Fatigue of Tendon Anchorage Hardware

17.4.7.1 Fatigue of Bonded Tendon Anchorages

Anchorages of bonded tendons typically are not subjected to significant fatigue. In most cases they are located at the member ends where changes in tendon forces are small. At frame corners and in segmental construction the anchors may be positioned at locations where live loads induce stress changes. However, in most cases the tendon force changes are induced by bond transfer rather than a sudden load transfer through the anchorage bearing plate. The bond provides redundancy; therefore, fatigue resistance of anchorage hardware is normally considered not to be critical, including their strand-wedge or bar-nut connections.

17.4.7.2 Fatigue of Unbonded Tendon Anchorages

The performance of unbonded tendons depends on the integrity of anchorages. The critical anchorage components are the strand-wedge and bar-nut connections. For multi-strand tendons the wedge plates are usually highly stressed and for fatigue-sensitive structures they are preferably made from ductile steel rather than ductile iron. Normally stress and force changes in unbonded tendons are even smaller than in bonded tendons because strain increases that are due to load changes average out over the length of the tendons. However, this may not be true at anchorage locations.

Notwithstanding the above, acceptance requirements for unbonded tendon systems traditionally require the tendons, including their anchorages, to endure 500,000 stress cycles between $0.60f'_p$ to $0.66f'_p$. In addition, it must be demonstrated that tendons can endure 50 stress cycles between $0.40f'_p$ to $0.80f'_p$. Most commercially available post-tensioning systems equally qualify to be used for bonded or unbonded tendons.

The PTI *Acceptance Standards for Post-Tensioning Systems*[17.2] provides additional information on static and dynamic testing of tendons and their components.

17.4.8 Fatigue of Tendons

17.4.8.1 General Information

Tendon fatigue depends only partly on the fatigue properties of the prestressing steel. Instead, tendon fatigue is largely controlled by project design choices in regard to:

- Selecting a design with bonded or unbonded tendons
- Designing a structure to be fully or partially prestressed

Stress changes in tendons in fully prestressed structures and in unbonded internal or external tendons are normally small and tendon fatigue is unlikely. Tendon fatigue is essentially limited to bonded tendons in partially prestressed members. Fatigue considerations for unbonded tendons are discussed in Sections 17.4.7.2 and 17.4.8.3; for fully and partially prestressed members see Sections 17.5 and 17.6.

17.4.8.2 Fatigue of Bonded Tendons – Uncracked Concrete (Fully Prestressed Condition)

As discussed in Section 17.5, fully prestressed herein means that the concrete is essentially uncracked, or existing cracks remain closed under prevailing fluctuating live loads.

Under uncracked conditions, stress increases in tendons due to live load fluctuations seldom exceed $\Delta f_p = 10$ ksi (69 MPa). For the common ASTM A416 strand with $f'_p = 270$ ksi (1860 MPa) that is equal to $\Delta f_p = 0.04f'_p << 0.10f'_p$. Because fretting fatigue is not a problem in uncracked members, this suggests a safety margin of 2.5 when measured against the stress range given by Eq. (17.10).

Similarly, for a typical prestressing bar with $f'_p = 150$ ksi (1034 MPa) the stress range would be $\Delta f_p = 0.07f'_p < 0.10f'_p$, suggesting a comfortable safety factor of 1.5 to cover for uncertainties associated with predicting fatigue resistance.

The following hypothetical example shows that even under extreme conditions tendon fatigue failure in uncracked fully prestressed concrete members is unlikely.

Consider a high strength concrete member with $f'_c = 6$ ksi (41 MPa) and $n = E_p / E_c = 6$. Assume further that the sustained compressive stress at the tendon level (in the precompressed tensile zone) is equal to the usually allowable value of $0.45f'_c$. In that case the cyclic tensile stress range, which produces zero compression at the tendon level, is $\Delta f_c = 0.45f'_c = 2.7$ ksi (18.5 MPa). Due to strain compatibility, the stress range in a fully bonded tendon will be $\Delta f_p = (n)\Delta f_c = 16$ ksi (111 MPa). For a typical strand tendon with $f'_p = 270$ ksi (1860 MPa) that stress range is equal to $\Delta f_p = 0.07f'_p$, providing a safety margin of 1.4 against the guide value of $0.10f'_p$ given by Eq. (17.10).

Section 5.5.3.1 of the AASHTO LRFD Specifications[17.1] states that "fatigue of the reinforcement need not be checked for fully prestressed concrete components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in Article 5.9.4.2.2b" of the Specifications.

17.4.8.3 Fatigue of Bonded Tendons – Cracked Concrete (Partially Prestressed Condition)

The fatigue life of bonded strand tendons at crack locations (i.e., cracks open under fluctuating live loads) has been shown to be significantly shorter than the fatigue life of straight strand samples tested in air in testing machines. This is caused by higher stress fluctuations at the cracks, and fretting effects, initiated by bond deterioration associated with crack opening. (See also Sections 17.4.6.3 and 17.5.4.)

Fretting becomes more severe as lateral pressures increase with increasing tendon curvature. Tests have shown that strand wires consistently fractured at or near crack locations. Fretting fatigue failures of strand wires were observed: 1) at interfaces with metal ducts; 2) between different strand layers; and 3) occasionally between strand wires (see Fig. 17.18). Using plastic ducts eliminates fractures at duct-strand interfaces but fretting fatigue failures still occur between strand layers and strand wires.[17.24] The benefit of plastic duct is primarily limited to preventing damage from metal duct corrosion and to providing improved corrosion protection to tendons.

Fig. 17.15 shows test data for tendon fatigue on precracked post-tensioned beams with grouted and curved tendons.[17.22] The figure also refers to the same in-air strand test fatigue data shown in Fig. 17.14. One can see that at 2×10^6 cycles the in-air sustainable stress range for strands is reduced by fretting by about 50%. At 10^5 cycles the reduction is about 25%. The different slopes of the failure zones for in-air and embedded curved tendons indicate that fretting fatigue depends more on the number of cycles than stress range.

Fig. 7.18 Strand Fretting Fatigue Fractures[17.22]

These observations and numerous other tests of precast beams show that at crack locations sustainable stress ranges in bonded tendons are significantly smaller than those on straight strand samples tested in air. Therefore, under cracked conditions the sustainable stress ranges obtained from Eqs. (17.9) and (17.10) must be reduced by 25% to 50%, depending on the number of anticipated load cycles. For the usual fatigue life of 2×10^6 one then obtains from Eq. (17.10) an approximate reduced stress range of:

$$\Delta f_p^* = 0.05 f'_p \tag{17.13}$$

Eq. (17.13) applies to curved tendons at cracked sections.

Bar tendons are usually straight. In the absence of side pressure, fretting fatigue does not occur. For deformed curved bars—with only the crest of the ribs in contact with the duct—no fretting fatigue was observed, as reported in Ref. 17.22. Instead, additional tensile bending stresses initiated fatigue fractures at the convex side of curved bars.

The ACI Fatigue Committee 21517.19 recommends a maximum tendon stress range of $\Delta f_p = 0.10 f'_p$ for strands, wires, and bars with minimum stress between $0.4f'_p$ and $0.6f'_p$. The ACI Guideway Committee 358[17.35] recommends for curved post-tensioned tendons a maximum stress range of $\Delta f_p = 0.04 f'_p$. However, this limit is reduced to $0.025f'_p$ at locations where the tendon curves.

17.4.8.4 Fatigue of Unbonded Tendons

Stress ranges due to repetitive live loads in unbonded internal and external tendons are significantly smaller than those in bonded tendons at locations of maximum moments. This is so because strains and stresses due to load changes average out over the length of unbonded tendons. This feature essentially precludes fatigue failure of internal and external unbonded tendons.

In addition, internal unbonded tendons are typically coated with a corrosion-protective compound, which also acts as a lubricant, preventing fretting at crack locations. The fatigue resistance of widely used plastic sheathed and coated mono-strands, therefore, is essentially the same as for strands fatigue tested in air and Eqs. (17.9) and (17.10) are applicable.

Therefore, concrete cracking has little effect on the fatigue resistance of unbonded tendons. However, other fatigue effects need to be considered such as: 1) fatigue of the anchorage hardware; 2) strand fatigue in the strand-wedge connections of anchorages and couplers; and 3) fretting at deviators with small radii and large side pressures.

For more information on anchorages and strand-wedge connections and their acceptance requirements see Section 4 of the PTI *Acceptance Standards for Post-Tensioning Systems.*[17.2]

17.5 FATIGUE OF PRESTRESSED CONCRETE MEMBERS

17.5.1 Introduction

As discussed in Section 17.4.8, it is well established that stress changes in tendons of fully prestressed members are small and well below the values that can be sustained over the expected life of the structure. However, tests have also shown that at cracks the fatigue life of bonded prestressing steel can be significantly less.[17.18,17.22]

Most design codes allow some tensile stresses under certain loading conditions. The question then arises if the calculated tensile stresses could lead to cracks in concrete and tendon fatigue.

17.5.2 Concrete Tensile Stresses

Plain concrete is weak in tension but strong in compression. Once cracking occurs, plain members lose their load-carrying capacity. On the other hand, it is common to rely on moderate concrete tensile strength in reinforced and prestressed concrete design for: 1) resisting shear; 2) resisting bursting stresses in tendon anchorage zones; and 3) load transfer between reinforcing bars in lap splices.

Most design codes allow some nominal flexural tensile stresses. This is mainly done for the convenience of elastic analysis, permitting the use of readily available section properties for uncracked sections. While the elastic analyses are essential for establishing the performance under service conditions, the importance of calculated tensile stresses is primarily to identify tension zones where crack distribution reinforcement needs to be placed.

Generally, AASHTO LRFD 1998 allows calculated tensile stresses under service loads, f_t, of $6\sqrt{f'_c}$ psi $\left(0.5\sqrt{f'_c}\text{ MPa}\right)$ at the extreme fibers of the precompressed tensile zone (under certain conditions even higher values are permitted). ACI 318-02, on the other hand, specifies f_t according to a new prestressed concrete member classification system, where the extreme fiber stress, f_t, at service loads in the precompressed tensile zone is as follows:

Class U: $f_t \le 7.5\sqrt{f'_c}$

Class T: $7.5\sqrt{f'_c} < f_t \le 12\sqrt{f'_c}$

Class C: $f_t > 12\sqrt{f'_c}$

Such allowable tensile stresses approach the cracking stress of concrete as measured by the modulus of rupture. This indicates that allowable tensile stresses must not be understood to ensure the flexural resistance of a section. Instead, the importance of calculated nominal stresses is to identify tensile zones possibly requiring supplementary reinforcement.

Modulus of rupture can be computed as follows:

Normal weight concrete (ACI 318-02 and AASHTO LRFD),

$7.5\sqrt{f'_c}$ psi $\left(0.62\sqrt{f'_c}\text{ MPa}\right)$

Sand-light weight concrete (AASHTO LRFD),

$6.25\sqrt{f'_c}$ psi $\left(0.52\sqrt{f'_c}\text{ MPa}\right)$

All-light weight concrete (AASHTO LRFD),

$5.4\sqrt{f'_c}$ psi $\left(0.45\sqrt{f'_c}\text{ MPa}\right)$

Light weight concrete (ACI 318-02),
(Refer to Sections 9.5.2.3[a] and 9.5.2.3[b] of the ACI 318-02 Code).

17.5.3 Supplementary Reinforcement

Cracked section analysis provides a rational means of determining the amount of supplementary reinforcement needed for crack control and fatigue resistance. However, such analyses per AASHTO LRFD Specification or Eurocode are cumbersome. ACI 318-02 requires that bonded reinforcement must resist the total tensile force in the uncracked concrete, using a steel stress of $0.5 \times f_{sy}$.

For other than flexural tensile stresses the supplementary reinforcement can be determined on the basis that the tensile force capacity of the reinforcement at yield should be equal to the tensile strength of the concrete in tension:

$$A_s \cdot f_{sy} = A_{ct} \cdot f_{ct} \tag{17.14}$$

$$\rho = \frac{A_s}{A_{ct}} = \frac{f_{ct}}{f_{sy}} \tag{17.15}$$

Where:

A_s = Reinforcement area

A_{ct} = Area of concrete in tension

f_{ct} = Concrete modulus of rupture

f_{sy} = Yield strength of reinforcement

ρ = Reinforcement ratio (0.7% to 1.0%)

17.5.4 Tendon Fatigue at Crack Locations

The reduction of tendon fatigue resistance at concrete crack locations is primarily caused by an increase in stress range in the tendon at the crack. This causes fretting-fatigue. In prestressed concrete structures fretting can be explained as follows:

- When concrete cracks, tendon forces increase to compensate for the lost concrete tension. Related tendon strains cause the neutral axis to shift toward the flexural compressive side. This increases the concrete compressive stresses. The stiffness of the member changes abruptly when the member cracks.

- The sudden change in tendon strain under cyclic loads tends to deteriorate the bond between tendons and surrounding grout. This further increases accumulated tendon strains and crack width. The neutral axis then shifts further toward the compressive side, increasing the compressive stresses further as required to establish equilibrium between internal compressive and tensile forces ($T = C$) and between internal and external moments ($M = T \times z$).
- With each succeeding load cycle bond deterioration continues along the tendon, causing cracks to widen further. The neutral axis shifts further toward the compressive side. As a result, the internal lever-arm z increases and the tendon force decreases. The reverse change in tendon strain enhances further bond deterioration.
- This process continues at a decreasing rate, until the bond in the tensile zones deteriorates completely.
- Cyclic concrete creep enhances crack widening and continues to do so after the bond has deteriorated. It is caused by time and increases with increasing compressive stresses as the depth of the compressive stress block decreases.
- Stress and strain changes caused by load fluctuations, in combination with bond deterioration, cause small relative displacements (slip) between prestressing strand and ducts, between strand layers and between contacting strand wires. At the presence of side pressure (due to tendon curvature) the slip causes fretting, which reduces the prestressing steel's fatigue life and strength.

The above considerations raise an interesting question on the fatigue life of tendons in high performance concrete (HPC) members. The expected higher bond strength could potentially reduce the unbonded length on either side of cracks, thus increasing the stress range in the prestressing steel and reducing the fatigue life.

17.5.4.1 Design Options

The above-outlined sequence of crack generation that can lead to tendon fatigue can be counteracted by design measures. Fatigue-reducing design options include:

Increasing the prestressing force to keep the decompression moment (i.e., the moment that causes zero tensile stress at the extreme tensile fiber) larger than the moment due to the fluctuating loads. This approach ensures that concrete remains essentially uncracked during service. Cracking is accepted under extreme loading conditions that are less frequent.

Providing adequate supplementary non-prestressed reinforcement in tensile zones. The reinforcement controls crack spacing, crack width, reduces stress changes in tendons, and reduces fretting.

Good performance of prestressed concrete structures subjected to fluctuating live loads, such as bridges, has established that applicable code requirements are adequate to ensure their fatigue resistance.

AASHTO LRFD discusses the fatigue limit state in depth per Section 5.5.3 of the Specifications. It stipulates that "in regions of compressive stress due to permanent loads and prestress in reinforced and partially prestressed concrete components, fatigue shall be considered only if this compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination."[17.1]

17.5.5 Fatigue Failure of Prestressed Structures

Most prestressed members have highly redundant tendon elements. Therefore, contrary to non-redundant steel members, structural failures of prestressed concrete members due to tendon fatigue are not sudden but are gradual over extended periods.

Isolated fatigue failures of individual strand wires are not detectable but usually are not critical because of redundancy. As the fatigue failure rate increases the member stiffness gradually reduces, cracks widen, and deflections increase, providing warning of approaching failure.

Structural failures of prestressed concrete members due to excessive compressive stresses are catastrophic without warning. Design codes include provisions for avoiding such failure modes.

17.5.6 Fully and Partially Prestressed Members

17.5.6.1 Definition

As mentioned earlier, the terms "full" and "partial" prestressing have been used in the current AASHTO LRFD Specifications. Fully prestressed refers to concrete members that remain uncracked under service loads. Partial prestressing[1.26] implies the use of several design alternatives, which are based on cracked section under service loads, due to the uncertainty of accurately predicting the tensile strength of concrete.

Because cracking in prestressed concrete can be very detrimental to fatigue, the following definitions are adopted for this discussion:

- Prestressed concrete members are considered to be fully prestressed if designed as uncracked members under service loads.
- Partially prestressed concrete members are members designed to crack under service loads.

The two conditions are further discussed in the next sections.

17.5.6.2 Fully Prestressed Members

In fully prestressed members the stress changes in the tendons are similar to those of uncracked concrete and well below the fatigue strength of the tendons. (See also Sections 17.4.6 and 17.4.8.)

Fully prestressed members are appropriate for structures subjected to significant and frequent fluctuating loads over long periods. For such structures the prestressing forces should be selected conservatively. The design must allow for the uncertainties inherent in calculated tensile stress and tensile strength of concrete in the precompressed tensile zone. (See also Section 17.5.2.)

Ref. 17.21 provides information on tendon force tolerances due to stressing procedures and friction losses. Ref. 17.27 provides information on prestress losses from creep and shrinkage.

Fully prestressed members are the most common application and are typically used in all structures. Partial prestressing is not recommended for bridges where the ratio of live load to dead load is relatively large and the structure is subjected to continuous load cycles. Buildings are rarely subjected to large enough load fluctuations during service to cause fatigue concerns.

17.5.6.3 Partially Prestressed Members

From the discussions in Section 17.5.2, it can be concluded that visible or invisible cracks can occur if calculated concrete tensile stresses occur under design loads. If these cracks open and close under fluctuating service loads, the member is considered to be partially prestressed. The service fatigue limit state may govern over the strength limit state in partially prestressed members.

Cracked section analysis is necessary to determine which limit state governs. The aim of partial prestress design is to limit:

- Stress range in the tensile steel elements
- Deflections
- Concrete crack width (acceptable values depend on fatigue considerations, deflection limits, and corrosive environment)

Partial prestressing is not widely used in buildings and bridges structures. However, the high allowable concrete tensile stresses permitted by most design codes could result in low levels of prestressing.

At the presence of bonded reinforcement for crack control, ACI 318-02 permits tensile stresses up to:

- Class U Members: $7.5\sqrt{f'_c}$ psi $\left(0.7\sqrt{f'_c}\text{ MPa}\right)$ (Uncracked)
- Class T Members: $12\sqrt{f'_c}$ psi $\left(1.0\sqrt{f'_c}\text{ MPa}\right)$ (Transitional)
- Class C Members: $>12\sqrt{f'_c}$ psi $\left(1.0\sqrt{f'_c}\text{ MPa}\right)$ (Cracked)

The AASHTO LRFD specification and the Swiss design code SIA 162 more consistently place no limits on calculated concrete tensile stresses of partially prestressed members. Instead, they rely solely on cracked section analysis and serviceability criteria such as fatigue and crack width.

As discussed earlier, calculated concrete tensile stresses must be understood—for the convenience of elastic analysis—to serve as guide values when more realistic cracked section analyses are needed. For instance, concrete tensile stresses of $3\sqrt{f'_c}$ psi $\left(0.25\sqrt{f'_c}\text{ MPa}\right)$ usually result in acceptable tendon stress increases under cracked conditions, meeting the reduced fatigue resistance requirement of Eq. (17.13).

17.6 SUMMARY

Fatigue of prestressed members has been extensively investigated by numerous researchers. Based on years of research and testing of many prestressed concrete beams in constant and random amplitude tests the following generally applicable design implications have evolved:[17.18,17.22,17.26,17.28,17.29,17.30,17.31]

1. Uncracked prestressed concrete members have superior fatigue performance. No fatigue failures of such members have been reported.
2. Flexural fatigue failures of prestressed concrete members originate at concrete cracks. They are caused by fatigue fractures in either the prestressing steel or non-prestressed reinforcement. No failures due to concrete fatigue have been reported when concrete was stressed within the limits of allowable stresses.
3. The loading sequence has a large influence on a structure's fatigue life. High crack-initiating loads, applied at the beginning of load cycling, significantly reduces the fatigue life of prestressing or reinforcing steel (i.e., early proof loading can be detrimental).
4. The fatigue life of prestressing steel is controlled by the cyclic stress range and crack width at critical crack locations.
5. Successive crack opening and closing under fluctuating loads causes fretting and abrasion in prestressing steel. Fretting can significantly reduce the fatigue life of prestressing steel.
6. Crack width increases under cyclic loading. The increase is caused by bond deterioration in the cracked region and cyclic creep of concrete.

7. Increasing the amount of non-prestressed reinforcement and reducing its spacing decreases crack spacing and crack width. This enhances fatigue of the member.
8. Cracked section analysis is required to determine tendon stress ranges for fatigue evaluation. It is assumed that prior cracking has occurred as a result of overloads or other means. Calculating concrete tensile stresses for uncracked sections and limiting calculated tensile stresses is unreliable for evaluating the fatigue life of dynamically loaded structures.
9. Shear design and web cracking due to principal tensile stresses require attention. Some test beams with bending-type failure modes under monotonic loading failed in shear under random amplitude fatigue loading.

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NOTATION

A_{ct}	Area of concrete in tension
A_s	Reinforcement area
C	Damping coefficient (ratio of damping to critical damping)
f_{ct}	Concrete modulus of rupture
f_{min}	Minimum stress under cyclic load
f'_p	Nominal ultimate tensile strength of strand
f_{sy}	Yield strength of reinforcement
K	Stiffness
M	Mass
N	Number of stress cycles
$P(t)$	External vibratory load (as a function of time)
Δ_{fp}	Cyclic stress range
$v(t)$	Deflection (relative to base)
$\dot{v}(t)$	Velocity
$\ddot{v}(t)$	Acceleration
$\ddot{v}(t)$	Acceleration of mass relative to base
$\ddot{v}_g(t)$	Ground acceleration
$\ddot{v}_t$	Total acceleration of mass
ρ	Reinforcement ratio (0.7% to 1.0%)

FIRE RESISTANCE

18.1 SCOPE

The purpose of this chapter is to present basic information on fire resistance and fire resistance ratings for post-tensioned structures. Code requirements pertaining to post-tensioned concrete structures subjected to specific fire endurance criteria are discussed. Finally, numerous references are presented for further information on the subject.

18.2 GENERAL

18.2.1 Effect of High Temperatures on Concrete and Steel Properties

Concrete and steel undergo certain changes when they are heated to temperatures reached in fires. When the temperature in concrete exceeds about 220°F, free water is boiled off. As the temperature continues to increase, hydrated cement begins to dehydrate. That process continues up to about 1800°F. Dehydration is accompanied by a decrease in strength and modulus of elasticity. Concrete that is in compression retains most of its strength up to about 1200°F, while the modulus of elasticity, E_c, diminishes significantly. At about 600°F and 1200°F, E_c is respectively reduced to around ⅔ and ⅓ of its room temperature value. Creep and stress relaxation of concrete increase significantly at high temperatures.

The strength of the post-tensioning steel decreases as temperature increases, and at about 800°F its strength is reduced to about half its room temperature strength. Stress relaxation of post-tensioning steel increases as temperature increases.

18.2.2 Behavior of Concrete Structures in Fires

Concrete has long been considered an excellent material for fire-resistant construction. Even after sustaining severe fires, most concrete structures can be repaired and reused. Concrete does not burn and concrete floors and walls provide excellent barriers for containing fires in the area of origin.

As noted below, building codes may require that floors in a given building qualify for a 2-hour fire resistance rating. This does not mean that a two-hour fire is expected. The "2-hour" requirement contains a factor of safety, generally in the range of 3, 4, or even higher. Indeed, most fires in residential, office, or institutional occupancies are equivalent to an ASTM E119[18.1] fire test (discussed below) of 20 to 40 minutes. Theoretically, the factor of safety in standard fire tests is 1.0, and load factors of 1.0 are used for dead and live loads.

After a fire has occurred some repairs are nearly always needed. Often repairs are merely cosmetic in nature, but after any severe fire, a structural engineer, preferably the engineer who designed the structure, should evaluate the damage and develop the necessary repairs. Discussions of methods of evaluating fire damage in concrete structures are included in Section 18.7 and Refs. 18.2 and 18.3.

18.3 CODE PROVISIONS

Building codes in the United States, such as IBC,[18.4] and Canada give requirements for various building elements in terms of fire resistance ratings. Such ratings are given in hours, e.g., 1 hour, 1½ hour, 2 hour etc., depending on the construction type and occupancy. The codes also require that fire resistance ratings be determined by one of three methods: 1) from results of standard fire tests; 2) from tabulated data included in the code; or 3) by calculation procedures included in the code.

18.3.1 Standard Fire Tests of Building Construction and Materials (ASTM E119)

The fire resistive properties of building components are measured according to this common standard. Performance is defined as the period of exposure to a standard fire before the first critical "end point" is reached.

The standard fire exposure is defined in terms of a time-temperature relationship. At 5 minutes the furnace atmosphere temperature is 1000°F, at 30 minutes 1550°F, at 1 hour 1700°F, at 2 hours 1850°F, and at 4 hours 2000°F.

The standard specifies minimum sizes of specimens to be exposed in fire tests. For floors and roofs, at least 180 sq ft must be exposed to fire from beneath, and neither dimension can be less than 12 ft. For tests of walls, the minimum specified area is 100 sq. ft with neither dimension less than 9 ft, while for beams the minimum length is 12 ft.

During fire tests of floors, roofs, beams, load-bearing walls, and columns, the maximum permissible superimposed load is applied. Floor and roof specimens are exposed to fire from beneath, beams from the bottom and sides, walls from one side, and columns from all sides.

End point criteria for floors and roofs are:

- Specimens must sustain the applied loading — collapse is an obvious end point.
- Holes, cracks or fissures through which flames gases hot enough to ignite cotton waste must not form.

The temperature of the unexposed surface must not rise an average of 250°F or a maximum of 325°F at any one point.

18.3.2 Tabulated Data

Codes contain lengthy tables describing assemblies qualifying for fire resistance ratings. For example, the IBC includes data pertinent to post-tensioned structures in Tables 719.1(1) and 719.1(3). For convenience, minimum slab thickness and cover requirements for post-tensioned slabs are summarized in Tables 18.1 and 18.2, respectively. Cover for post-tensioning tendons in thermally unrestrained slabs is determined by the elapsed time during a fire test until the tendons reach a critical temperature. For cold-drawn prestressing steel, that temperature is 800°F. Fire tests on thermally restrained slabs indicate that slabs with post-tensioned reinforcement behave the same way as non-prestressed concrete slabs having the same dimensions. The cover to the prestressing steel at the anchor should be at least ½ in. greater than that required by Table 18.2. Minimum cover to anchorages varies depending on their type (e.g., encapsulated, etc.).

Table 18.1 - Minimum Slab Thickness[18.4]

Aggregate Type	1 hr	2 hr	3 hr	4 hr
Siliceous	3.5 in.	5.0 in.	6.2 in.	7.0 in.
Carbonate	3.2 in.	4.6 in.	5.7 in.	6.6 in.
Sand-lightweight	2.7 in.	3.8 in.	4.6 in.	5.4 in.
Lightweight	2.5 in.	3.6 in.	4.4 in.	5.1 in.

It should be noted that Table 18.1 and 18.2 are provided herein for general reference only. The designer must check applicable local codes for fire resistance cover requirements.

For the selection of the required concrete fire cover to reinforcement in a continuous post-tensioned concrete slab, end spans are typically considered to be unrestrained and interior spans are considered restrained. In the case of two-way slabs, this criteria applies independently to the reinforcement in each orthogonal direction.

For unrestrained end bays in two-way slabs, it is worth clarifying an important point many design engineers find confusing. Tendons running parallel to the slab edge do not require an increased fire cover until they run into *their own* end bay. As an example, consider a rectangular building with only two bays in the short direction, and multiple bays in the long direction. Some design engineers might consider every portion of the deck an end span for the tendons running across the long bay direction. This interpretation is wrong. One should design the tendons running across the long bay direction as unrestrained *only in the end bays*, while tendons running across the two bays in the other direction should be designed as unrestrained (i.e., with an increased bottom cover). In other words, unrestrained end panels are those that only pertain to the direction being considered.

18.3.3 Calculation Procedures

Building codes permit the use of certain calculation procedures. For example, Section 720 of the IBC,[18.4] *Calculated Fire Resistance*, details the procedures. Although the section gives a few formulas, most of the information pro-

Table 18.2 - Required Concrete Cover Thickness for Post-Tensioned Slabs[18.4]

Restrained / Unrestrained	Type of P/T Structure	1 hr	2 hr	3 hr	4 hr
Unrestrained	Solid Slab(1)	---	1½ in.	2 in.	---
Unrestrained	Beams & Girders 8 in. Wide(3)	1¾ in.	2½ in.(2)	4½ in.	---
Unrestrained	Beams & Girders > 12 in. Wide(3)	1½ in.	---	2½ in.	3 in.
Restrained	Solid Slab(1)	---	¾ in.	1 in.	1¼ in.
Restrained	Beams & Girders 8 in. Wide(3)	---	1½ in.	2 in.	2½ in.
Restrained	Beams & Girders > 12 in. Wide(3)	---	1½ in.	1¾ in.	2 in.

(1) For solid slabs of siliceous aggregate concrete, increase tendon cover 20%

(2) Two layers of equal thickness with a ½ in. air space between

(3) For widths between 8 and 12 in., interpolation shall be permitted

vided is given in tabulated format. For example, Tables 720.2.3(2) and 720.2.3(4) summarize *calculated* cover for prestressed concrete floors or roof slabs and beams, respectively. It should be noted that the cover values shown in Table 18.2 are *prescriptive* values as per Section 719 of the IBC.[18.4] If, however, designers choose to compute fire resistance per Section 720 of the IBC, then they will *always* end up with more cover.

18.4 RATIONAL DESIGN PROCEDURES

18.4.1 General

Procedures discussed herein are based on data from ASTM E119 fire tests. Results are accurate predictions of the fire endurance that would occur if the assemblies were subjected to an ASTM E119 test.

As background for the discussion of rational design procedures for evaluating the behavior of structures during fires, three types of flexural members are briefly discussed.

18.4.1.1 Simply Supported Slabs or Beams

Consider a simply supported reinforced concrete slab subjected to fire from below. Assume that the ends of the slab are free to rotate and expand (i.e., unrestrained). Assume also that the reinforcement consists of straight bars located near the bottom of the slab. With the underside of the slab exposed to fire, the bottom will expand more than the top, and the slab will deflect. Also, the strength of the concrete and steel near the bottom of the slab will decrease as the temperature increases. When the moment capacity of the section is reduced to applied moment, flexural yielding, or in extreme conditions, collapse may occur. Such behavior has been clearly demonstrated in prestressed as well as reinforced concrete members.[18.5]

It is apparent from the above description that the moment capacity depends on the steel temperature, which in turn depends on the cover thickness. The required cover depends on the stress in the steel and the type of steel used. The stress in the steel depends on the load intensity the member is subjected to. For example, if the stress in the reinforcing steel has reached 50% of its initial yield strength, the critical temperature will be about 1120°F. However, if the steel stress is one-third of the yield strength, the critical temperature will be about 1220°F. The temperatures would be different for cold-drawn steel or high-strength alloy steel bars. Thus, it can be seen that if the load intensity is decreased the fire endurance will increase. When evaluating the strength vs. demand ratio, it is possible to estimate the increase in fire endurance due to a decrease in load intensity.

18.4.1.2 Continuous Slabs and Beams

Structures that are continuous or otherwise statically indeterminate undergo changes in moments when subjected to fire.[18.5] It should be noted that this is different from a simply supported member where the applied moment at a section remains constant during fire exposure.

Consider a two-span continuous slab with rocker-rollers at the outer supports. During fire exposure from beneath, the underside of the slab expands more than the top. This differential heating causes the ends of the slab to lift from the outer supports, thus increasing the reaction at the interior support. This action results in a redistribution of moments, i.e., the negative moment at the interior support increases while the positive moments decrease.

Assuming the fire intensity is higher at the slab soffit, the negative moment reinforcement remains cooler than that in the positive moment region because it is further away from the fire (i.e., increased fire protection). Therefore, the increase in negative moment is better accommodated due to the combination of cooler negative reinforcement and the redistribution of negative and positive moments. The resulting decrease in positive moment implies that the positive moment steel can withstand a higher temperature before failure will occur. This all amounts to the fact that the fire endurance of a continuous member is generally significantly longer than that of a simply supported member having the same cover and load intensity.

18.4.1.3 Members in Which Restraint to Thermal Expansion Occurs

If a fire occurs beneath a small interior portion of a large reinforced concrete slab, the heated portion will tend to expand and push against the surrounding part of the slab. In turn, the unheated part of the slab exerts compressive forces on the heated portion. The compressive force, or thrust, acts near the bottom of the slab when the fire initially occurs. As the fire progresses, the line of action of the thrust rises as the heated concrete softens.[18.6] If the surrounding slab is thick and heavily reinforced, the thrust forces that occur can be quite large, but considerably less than that calculated by use of elastic proprieties of concrete and steel together with appropriate coefficients of expansion. At high temperatures, creep and stress relaxation play an important role. Nevertheless, the thrust is typically enough to increase its basic fire endurance. In most fire tests of restrained assemblies, the fire endurance is determined by a temperature rise of the unexposed surface rather than by structural considerations, even though the steel temperatures often exceed 1500°F. Design procedures are discussed in Refs. 18.2 and 18.3.

18.5 ADDITIONAL INFORMATION

For a detailed discussion of calculation procedures for concrete cover based on fire rating requirements, see Refs. 18.2 and 18.3. Further information about fire resistance of prestressed concrete slabs in general can be found in Refs. 18.6 – 18.20.

18.6 POST-FIRE INVESTIGATIONS

Most post-tensioned structures damaged by fire have suffered little or no structural damage. Indeed most repairs have been cosmetic in nature. Nevertheless, any structure that has been exposed to a severe fire should be examined by a structural engineer, preferably the one who designed the structure, to determine the extent of damage. Because each fire-damaged structure is unique, there are no step-by-step procedures for such investigations.

The engineer's first responsibility is to ensure the safety of the structure. Questionable damaged areas should be shored or otherwise made safe.

The engineer should have the structural drawings, preferably the "as-builts," together with information on the type of tendons and anchors used, stressing data and information on the concrete, i.e., aggregate type and strength. A description of the nature and duration of the fire is often useful.

In most cases, fire-damaged portions of a concrete structure are visible. In certain situations, it is useful to tap the concrete with a hammer or use an impact-rebound hammer to outline the damaged area. Comparing the sound of the hammer or the rebound numbers obtained in damaged areas with those in undamaged areas often reveals distinct differences.

If there is extensive cracking or spalling that exposes tendons or reinforcing steel, extensive repairs or removal and replacement of the damaged areas might be needed. Similarly, if the deflection of slabs or beams is significant, it is likely that the prestressing force has been reduced. In this case, the engineer should determine if repairs are feasible.

It is possible to determine if the prestressing strands or wires have been heated above about 750°F. To do that, a short piece of wire must be removed, taking care not to heat the wire during removal. Certain laboratories can determine the micro-hardness of the wire by a technique described in Ref. 18.20. That reference also presents information relating the micro-hardness to the temperature to which the strand was heated.

Additional information on post-fire evaluations of reinforced and prestressed concrete is given in Refs. 18.2 and 18.3.

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DURABILITY

19.1 INTRODUCTION

Although the vast majority of all concrete structures have met, and continue to meet, their functional and performance requirements, there are numerous examples of structures that have not exhibited the desired durability or service life. This is particularly true of those in aggressive environments. In most cases, problems with durability have been attributed to poor design detailing, inadequate materials, and/or poor construction practices.

Post-tensioned structures are inherently durable due to the pre-compression that reduces or eliminates cracking. If designed and constructed properly, post-tensioned structures are particularly well-suited to aggressive environments such as coastal regions and areas where deicing chemicals are used. Post-tensioned slabs also have few joints to provide corrosive agents access to corbels, beams and columns.

Designing concrete structures to ensure adequate durability can be a complicated process. Service life depends on the structural design, detailing, mixture proportions, concrete production, concrete placement, construction methods and maintenance. Permeability is a major factor in the durability of concrete because water and deicing chemicals (or seawater) are major contributors to concrete degradation. The rate, extent and effect of fluid transport in concrete depend largely on the concrete pore structure (the pore size and distribution), the amount of cracking, and the microclimate at the concrete surface. Changes in use, loading and environment can also have a significant impact on durability.

The design of new concrete structures addresses service life by defining several critical concrete parameters. These include items such as the water to cementitious materials (w/cm) ratio, admixtures, reinforcement protection (concrete cover or use of coatings or corrosion inhibitors) and curing methods. The design engineer also checks serviceability criteria such as deflection and crack width. Other factors that promote durability such as appropriate joint details and drainage to minimize moisture accumulation are also addressed during design. Durability of all reinforced concrete structures can be increased with the use of low permeability concrete, proper curing, adequate concrete cover to reinforcement, reduced cracking, and measures to improve freeze-thaw resistance. This chapter focuses on the aspects that are unique in post-tensioned structures.

While ACI Building Code and Commentary 318-02[19.1] makes no specific service life requirements, serviceability is indirectly addressed through strength requirements and limitations on service load conditions. Examples of service load limitations include mid-span deflection of flexural members, maximum service level stresses in prestressed concrete, and requirements for a minimum amount of bonded reinforcement. The service load limitations are based on engineering experience rather than a rigorous analysis of the effects of these limitations on the service life of the structure, however. Other conditions affecting service life are applied to the concrete, the reinforcement material requirements and detailing. These include an upper limit on the w/cm ratio, a minimum entrained air content depending on exposure conditions, and minimum concrete cover over the reinforcement.

Omitting or improperly identifying the design parameters important to service life can compromise the durability of the structure. For example, a parking structure's exposure rating is considered to be either severe or moderate, based on its anticipated exposure to deicing chemicals. Because this decision affects the ACI Code requirements for w/cm ratio, it affects the price of the concrete. Improper selection of the exposure ratio can result in a lower initial cost but a more permeable concrete that results in faster chloride penetration and a shorter service life.[19.2]

Specific resources for each topic are cited throughout this chapter. The intent of the chapter is to present an overview of the durability issues facing various types of post-tensioned construction and to provide references (specifications and other documents) for details on durability design and construction.

19.2 DURABILITY IN BUILDINGS

19.2.1 General Considerations

Although post-tensioning can be used in many types of structural members, the majority of post-tensioning in the building market is used in slabs. Traditionally, unbonded single-strand systems have predominated in the United States. Bonded multi-strand systems have seen more widespread use in buildings in other parts of the world and have seen some limited usage in the United States in recent years. A thorough description of the differences between these two types of systems is provided in Sections 19.5 and 19.6 of this chapter. In all cases a multi-level protection system should be used to shield the prestressing steel from aggressive agents.

19.2.2 Past Performance

Apart from the immediate economy, a major concern of owners is the maintenance and the durability of the post-tensioning system selected. Due to shortcomings inherent in some early post-tensioning systems, coupled with a lack of specifications and poor workmanship, some buildings constructed with unbonded tendons have suffered from premature deterioration. From a statistical point of view, the bulk of the unbonded post-tensioned buildings in North America have performed well, and are providing their design-intended service. However, the geographically localized few that were constructed using poor quality materials, poor construction practice, and poor design practice have caught a disproportionate amount of attention. A good review of performance of post-tensioned building systems can be found in Aalami.[19.3]

19.2.3 Design Considerations

The unbonded tendons of today, following PTI's *Specification for Unbonded Single Strand Tendons*,[19.4] are designed to provide durability performance that is in line with other components used in building construction. Where the environment is aggressive, tendons specifically designed for aggressive environments must be used. Features described in the specification include the following:

- specially formulated corrosion-inhibiting coating
- minimum sheathing thickness
- watertight encapsulation of the strand over the entire length of the tendon (including watertight closure of the tendon ends)
- plastic or epoxy-coated anchorages

The specifications include not only the specific performance criteria for the materials used to protect the tendon, but also detailed recommendations for the installation and fabrication of the tendons. These include the following:

- handling during fabrication
- storage before shipping
- shipping
- handling and storage on-site
- sealing stressing pockets (finishing)

19.3 DURABILITY IN PARKING STRUCTURES

19.3.1 General Considerations

Like building structures, the majority of parking structures with post-tensioning utilize the unbonded single-strand systems in slab members. Many of the same durability concerns that exist for building structures also exist for parking structures; however the environment, including wide temperature variations and deicing salt exposure, is potentially more severe for parking structures.

Parking structures are typically very large in plan compared to most structures. In addition, both open parking structures (those with a large percentage of the façade open) and completely enclosed parking structures are exposed to seasonal and daily ambient temperature variations. They are thus subject to greater volume change effects than enclosed structures that are usually smaller in plan and have a more uniform temperature, humidity and moisture environment. Restraint of volume changes can cause cracking of floor slabs, beams, and columns, which if unprotected can allow rapid ingress of water and chlorides. Large cracks can lead to serviceability or structural concerns.

Parking structures, like highway bridges, can be subjected to severe temperature changes, exposure to the weather, and chloride penetration from deicing salts or ocean-generated airborne salts. The roof level is similar to a bridge deck in that it is exposed to weather including precipitation, ultraviolet and infrared rays, and solar heating as well as particulates and chemicals carried by wind and rain. The edges of an open parking structure may be subject to the same conditions as the roof, and other areas may be subjected to runoff from the roof.

All floors are subject to moisture in the form of water or snow carried in on the undersides of vehicles. In northern or mountain climates, this moisture may contain deicing salts. In coastal areas, salt spray, salty sand and high-humidity conditions combine to form a very aggressive environment. Unlike a bridge deck, the interior of a parking structure is not frequently rinsed by precipitation. Exposure to chlorides may be worsened by poor drainage layouts and clogged drains that result in ponding.[19.2]

19.3.2 Past Performance

As was mentioned in the previous section on building structures, a small percentage of post-tensioned parking structures have undergone premature deterioration and have received a disproportionate amount of negative publicity. Use of current specifications and documentation[19.2,19.4] will help to mitigate the problems of the past. The post-tensioned members used in parking structures are inherently durable, if properly constructed, because of their reduced cracking and the multiple layers of protection for the prestressing.

19.3.3 Design Considerations

The most common types of deterioration in post-tensioned parking structures are corrosion of the reinforcement, freezing and thawing damage, and cracking. These conditions frequently occur together and result in scaling of the driving surface, spalls and delaminations on both the driving surface and slab soffit, leakage of water through joints and cracks, and spalling of concrete on beams. Walls and columns suffer distress from leakage, splashing, wicking, and spray of salt-contaminated water. Concrete degradation, particularly in its advanced stages, is seldom due to a single mechanism. In harsher climates, there can be significant deterioration as a result of the combined effects of freezing and thawing and the corrosive effects of deicing salts.[19.2]

Two important factors affecting the durability of concrete structures are the actual environmental exposure conditions and the incorporation into the design of proven methods to mitigate chemical or physical attack, such as the use of appropriate cementitious materials and admixtures to resist chemical attack, and proper drainage to prevent the accumulation of water. Exposure conditions are generally handled through a specification that addresses the concrete mixture (for example, strength and w/cm ratio requirements) and details such as required concrete cover.

Detailed information about durability design of post-tensioned parking garage structures can be found in Ref. 19.2. The design considerations include the following:

- protection of the post-tensioning systems
- minimum prestress levels
- proper surface drainage
- protection of the non-prestressed reinforcement
- surface treatments and concrete additives
- construction practices

Details about protection of the post-tensioning systems are covered in later sections in this chapter on specific types of tendons. Minimum average prestress levels (P/A) help ensure crack control for the primary slab and beam post-tensioning. These values range from 150 psi to 200 psi depending on the environment (Zone). ACI 318-02[19.1] also has a minimum P/A value of 100 psi where unbonded tendons are used as temperature and shrinkage reinforcement.

Additional information can be found in Ref. 19.5.

19.4 DURABILITY IN BRIDGES

19.4.1 General Considerations

Post-tensioning in bridges exists in many forms: cast-in-place box girders; precast segmental box-girders; continuity tendons for pretensioned spliced girders; as additional live load capacity after deck placement in pretensioned girders; transverse tendons in box girder deck over hangs; and others. Nearly all multi-strand tendons in bridge applications are grouted. Internal tendons (those cast inside the concrete cross-section) are referred to as bonded tendons. External tendons (in contact with the section only at end anchorages and deviator points; typically inside the hollow portion of a box girder section) are considered to behave as unbonded tendons even though they may be grouted. The discussion in this section will refer to grouted multi-strand tendons. These systems depend on a multi-level protection system to provide maximum protection against aggressive agents.

Bridges may be subjected to extremely harsh environmental conditions as a result of deicing chemicals application, coastal proximity, and multiple temperature cycles.

19.4.2 Past Performance

Surveys of post-tensioned concrete bridges in the United States have indicated very good performance overall.[19.6,19.7] Potential durability concerns in grouted tendons came to the forefront beginning in the late '90s after discovery of grouting-related corrosion problems in the Niles Channel Bridge in Florida.[19.8] After a thorough investigation of additional Florida bridges with similar construction, other tendons were found with corrosion. The majority of these problems were grout related, and focused particularly on a lack of bleed resistance in the standard grouts being used throughout the United States. Additional occurrences of bleed voids in tendons in other parts of the country have been found, but with limited corrosion in the absence of Florida's harsh coastal environment. Two major steps taken to avoid similar problems in the future include the publication of PTI *Guide Specification for Grouting of Post-Tensioned Structures*[19.9] and the start of a Grouting Certification Training program by the American Segmental Bridge Institute. Materials and construction practices have significantly changed in recent years to conform to Ref. 19.9 and construction that follows these procedures should have excellent durability.

19.4.3 Design Considerations

A comprehensive document for durability design of all components of post-tensioned bridge systems is not readily available at this time. However, Ref. 19.9 contains very detailed information about grouting materials and proper construction practices for grouting. With the exception of grouting, the majority of the general durability considerations for buildings and parking decks discussed in the previous sections also apply to bridges. The multi-level protection system, including anchorage protection used in bridge structures, is discussed in greater detail in the section on bonded tendons.

19.5 UNBONDED TENDONS

19.5.1 Background and Types of Unbonded Tendons

Post-tensioning tendons are either considered unbonded or bonded. The traditional unbonded tendon is a greased 7-wire strand with an extruded plastic sheathing. This type of tendon is very common in building and parking garage construction. Grouted external bridge tendons are considered to be unbonded because they are in contact with the structure at discreet locations. This section will focus only on the standard greased and sheathed single-strand system. One of the key objectives of Ref. 19.4 is to ensure the durability of the post-tensioning system, including information for proper shipping and handling of the tendon bundles. The key components of this system are discussed in the following sections.

19.5.2 Potential Problem Areas

The protection system for standard unbonded tendons relies heavily on a multiple barrier system. The first level of protection is the concrete (including any corrosion-inhibiting admixtures and/or any membranes or sealers on the concrete surface). The precompressive force in post-tensioned systems limits cracking so that the concrete can become a very effective barrier for both the post-tensioning tendons and the mild reinforcement. The next level of protection for the strand is the sheathing, followed by the post-tensioning coating (grease). If care is taken in handling and placement not to breach the sheathing, the tendon is well protected along its free length. The biggest potential problem area lies in the anchorage zone and a number of precautions are taken to ensure an encapsulated system as discussed below.

Tendons that are fully encapsulated exhibit excellent corrosion resistance and help to provide a durable, low maintenance system. It is essential, however, that inspections are performed during construction to verify that all components of the encapsulation system are installed in accordance with the manufacturer's requirements. In addition, it is recommended that post-tensioning tendons be fabricated per PTI-approved certification programs.[19.10,19.11]

19.5.3 Anchorage Protection

The tendon sheathing is connected to all anchorages with a watertight seal, and plastic or epoxy-coated anchorages are required. A plastic cap covers the tendon tail and the wedges of the anchorage. This ensures complete encapsulation from end to end of the tendon. Design details that limit the access of direct runoff onto the anchorage area help reduce potential problems in the anchorage zone. The specification[19.4] gives detailed guidelines for a range of permissible length of strand projection from the wedges and minimum requirements for cover to the wedge cavity of the anchorages in various environments.

19.5.4 Sheathing

The integrity of the sheathing is important, as any breeches in the barrier would allow easy access points for moisture and chlorides. The specification[19.4] gives minimum sheathing thickness and discusses sheathing integrity as well as proper handling procedures.

19.5.5 Post-Tensioning Coating

A specially formulated corrosion-inhibiting coating is required. In aggressive environments, the entire length of the strand, and its projection beyond the anchor piece in the recess pocket, are covered with a corrosion-inhibiting coating. The specification[19.4] gives requirements for the performance of the strand coating and minimum coating coverage rates.

19.6 GROUTED TENDONS

19.6.1 Background and Types of Grouted Tendons

Grouted tendons encased in the concrete cross-section are considered bonded tendons. These are typically multi-strand tendons made up of bundles of 7-wire strand as opposed to the single-strand tendons typical in unbonded construction. Post-tensioning bars may also be used in some applications. Grouted external tendons, that is, those that are not encased in the concrete cross-section, are considered unbonded for design, but have very similar durability concerns as with bonded internal grouted tendons. This section will focus on all types of grouted tendons. Many of the same durability issues are found in these systems as in the other systems discussed previously, including concrete quality, reduced cracking, and proper anchorage protection. The primary difference is the grout injected into the tendons for corrosion protection. Ref. 19.9 covers grouting materials and construction in great detail.

19.6.2 Potential Problem Areas

The layers of protection for the strand include the surrounding concrete (in a bonded system), the duct and the grout. The main potential problem areas are poor grouting materials or poor grouting practices and inadequate anchorage protection. These points are discussed in depth in the next sections.

19.6.3 Anchorage Protection

End anchorages should be permanently sealed with a non-metallic end cap. The cap should contain a vent to ensure complete filling with grout while the tendon is being grouted. A non-shrink pour-back material should be used to fill the stressing pocket, and then covered by a membrane or sealant. Design details should ensure that runoff does not have direct access to the pour-back.

19.6.4 Duct

In non-aggressive environments a galvanized metallic duct is often used. In aggressive environments, a plastic duct is preferable as a watertight barrier. PP (polypropylene) or HDPE (high-density polyethylene) is used. Internal ducts are corrugated (PP or HDPE) and external ducts are smooth (HDPE). The connection from the duct to the anchorage should be watertight. In segmental construction, the joints should be sealed with epoxy on both faces. Duct couplers may also be used in conjunction with the epoxy.

19.6.5 Grout

The grout protects the strand as not only a barrier, but also by nature of its high pH environment. The high alkalinity forms a passive film on the steel that protects it from corrosion in the absence of chlorides. Chloride intrusion can break down the film in spots, leading to pitting corrosion of the strand. Voids in the grouted duct result in a loss of the protective environment and may end up as a collection point for moisture or outside contaminants such as saltwater. These voids can be formed by inadequate filling of the duct, or more commonly, from water separation (bleed) from the grout prior to set. Bleed water may remain trapped in the tendons or may reabsorb into the grout. The majority of durability problems found in grouted post-tensioned structures are related to bleeding of the grout.

Bleed is accentuated by the wicking effect of the 7-wire strand and also increases with vertical pressure head. Tall vertical members (such as bridge piers) represent a situation where a very bleed-resistant grout is needed. Bleed water will attempt to reach the highest point in the duct and may form significant voids as shown in Fig. 19.1. The typical grout mixes and procedures used in the past are not able to withstand bleeding, but grouts per Ref. 19.9 are designed to have minimal bleed. The current trend is toward prepackaged formulations that place the prequalification burden on the manufacturer. Testing of the grout during construction is still necessary to ensure that the field mix exhibits the desired properties.

Bleed-resistant grouts contain an admixture specifically designed for water retention, typically called a gelling agent, thixotropic agent, or stabilizer. Many of the grouts with water retentive admixtures will exhibit thixotropic properties; that is, they are fluid when agitated, but become gel-like at rest. High energy, colloidal type mixers are needed to get the full benefit from the admixture. Details about thixotropic grouts and bleed can be found in Refs. 19.12 and 19.13. These grouts can completely fill even very small areas between the strands as shown in Fig. 19.2. Expansive admixtures are not appropriate for counteracting bleed in grouts, although some expansive properties in the grout may be desirable to counteract settlement.

Optimum grouts combine bleed resistance, fluidity (assessed by the flow cone method as a measure of pumping ease), low permeability, and reasonable set time. The specification[19.9] covers materials and testing to achieve these properties. The specification also includes design and construction practices including mixing, pumping, venting, and post-grouting operations.

19.6.6 Temporary Corrosion Protection

The tendons should be grouted as soon as possible after stressing. In some cases, long periods of time may pass between stressing and grouting. The tendons should be kept completely sealed (with end caps in place) until grouting. In some cases a coating of emulsifiable oil is sprayed on the strand prior to inserting the strand into the duct as a temporary corrosion protection measure. The effects of these oils on corrosion protection and on strand bond after grouting is currently being researched. Preliminary findings and recommendations are available in Salcedo et. al.[19.14] The practice of flushing tendons with water prior to grouting is not recommended as the remaining water may mix with the grout and reduce bleed resistance or may become trapped and form voids.

Fig. 19.1 Bleed Water Lens

Fig. 19.2 Tendon Slice Showing Penetration of Grout Between Strands[19.12]

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INSPECTION

20.1 INTRODUCTION

Post-tensioned reinforcing is an engineered system that must be installed according to the requirements outlined in the project documents and applicable codes and standards. Detailed guidelines for the proper placement of post-tensioning tendons can be found in the following PTI manuals:

Single-Strand Unbonded Tendons

- Slabs-On-Ground: *Construction and Maintenance Procedures Manual For Post-Tensioned Slab-On-Ground Construction*[20.1]
- Elevated Structures: *Field Procedures Manual for Unbonded Single Strand Tendons*[20.2]

Bonded Systems (Multi-Strand and Bar)

- *Training and Certification of Field Personnel for Bonded Post-Tensioning—Student Manual*[20.3]

Proper field inspection by trained and qualified personnel helps to ensure that the engineer's requirements and instructions are carried out in the field. Inspection personnel should be certified under a program that is applicable to the type of construction being inspected. Additional information on the training and certification programs offered by the Post-Tensioning Institute can be found in Chapter 21.

20.2 CONSTRUCTION INSPECTION

Proper inspection during the various phases involved in post-tensioned construction helps to ensure that the requirements shown in the project documents are followed by the contractors in the field. Building codes in most jurisdictions require some level of inspection to verify proper tendon installation, stressing, grouting, and finishing of post-tensioned reinforcing.

The International Building Code (IBC)[20.4] defines Special Inspection in Section 1702 as an inspection that requires special expertise to ensure compliance with approved construction documents and referenced standards. Special Inspections are to be conducted by a qualified person who has demonstrated competence, to the satisfaction of the building official, for inspection of the particular type of construction or operation requiring special inspection. Special Inspections can be either "continuous" or

Table 20.1 - Inspection Requirements for Post-Tensioned Construction

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD	IBC REFERENCE
1. Inspection of reinforcing steel, including prestressing tendons, and placement		X	ACI 318: 3.5, 7.1–7.7	1903.5, 1907.1, 1907.7, 1914.4
2. Verifying use of required design mix		X	ACI 318: Ch. 4, 5.2–5.4	1904, 1905.2–1905.4, 1914.2, 1914.3
3. At the time fresh concrete is sampled to fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete	X		ASTM C 172 ASTM C 31 ACI 318: 5.6, 5.8	1905.6, 1914.10
4. Inspection of concrete and shotcrete placement for proper application techniques	X		ACI 318: 5.9, 5.10	1905.9, 1905.10, 1914.6, 1914.7, 1914.8
5. Inspection for maintenance of specified curing temperature and techniques		X	ACI 318: 5.11–5.13	1905.11, 1905.13, 1914.9
6. Inspection of prestressed concrete: a. Application of prestressing forces b. Grouting of bonded prestressing tendons in the seismic-force-resisting system	 X X		 ACI 318: 18.20 ACI 318: 18.18.4	
7. Verification of in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs		X	ACI 318: 6.2	1906.2

"periodic," as defined in the Code. Continuous inspections require full-time observation of the work being performed.

The IBC requires Special Inspection of post-tensioned concrete as shown in Table 20.1. Non-structural concrete slabs supported directly on the ground where the effective prestress is less than 150 psi (1.03 MPa) are exempt from the special inspection requirements listed in this section of the IBC, but many local building jurisdictions have similar inspection requirements that may apply to all post-tensioned foundations used in residential and light commercial construction, regardless of the level of prestressing force.

20.2.1 Tendon Installation

The quantity of tendons, tendon locations, and tendon profiles have been carefully calculated by the designer to impart the desired prestressing forces into the concrete member. It is critical that vertical deviations be kept to a minimum to ensure that the proper tendon profiles are achieved. Anchor placement, high points, and low points should all be checked in the field prior to concrete placement to verify proper installation within allowable tolerances. The tendons should follow their intended profile between high points and low points without excess wobble.

In post-tensioned concrete members, vertical deviations should be kept to within the following tolerances:[20.1]

- Slab depths to 8 in. (200 mm): +/– ¼ in. (6 mm)
- Slab or beam depths greater than 8 in. (200 mm) up to 24 in. (610 mm): +/– ⅜ in. (9 mm)
- Slab or beam depths over 24 in. (610 mm): +/– ½ in. (13 mm)

In ground-supported slabs used in residential and light commercial construction, where the profile of the tendons is flat, the placement tolerance for tendons is as follows:

- **For ribbed slabs** — the center of gravity of the prestressing steel should be kept within the middle ⅓ of the slab thickness for slabs ≥ 4.5 in. (114 mm) and within the middle ½ of the slab thickness for slabs ≤ 4.5 in. (114 mm).
- **For uniform thickness slabs** — the center of gravity of the prestressing steel should be kept within the middle ⅓ of the slab thickness, but should not exceed ±1 in.

Because the horizontal supporting surfaces of these slabs are not formed with wood or steel, subgrade preparation should be inspected prior to tendon installation to verify that dimensions are within the allowable tolerances found in PTI's *Design of Post-Tensioned Slabs-on-Ground.*[20.1]

Horizontal deviation of tendons is typically not as critical and a variance of +/– 12 in. (305 mm) is generally acceptable as long as excessive wobble is avoided and smooth transitions are made around obstructions with a maximum deviation of 1:6.

Note that none of these tolerances permits a reduction in allowable clear cover.

20.2.2 Concrete Placement

General procedures for the placement of concrete such as those published by the American Concrete Institute[20.3] are applicable to the construction of post-tensioned concrete. Calcium chloride, or other admixtures known to have a deleterious effect on steel, should not be used in concrete for post-tensioned construction.

Concrete should be placed in such a manner that tendon alignment and reinforcing steel positions remain unchanged. Special attention must be given to the vibration of concrete at tendon anchorages to ensure proper consolidation at these locations. Voids behind the bearing surface of the anchorage can cause undesirable concrete blowouts during stressing of the tendons.

Proper curing in accordance with ACI recommendations should be followed to ensure proper concrete strength is achieved.

20.2.3 Stressing

Proper inspection of tendon stressing operations helps to ensure that the desired prestressing forces have been applied. Tendon stressing should not begin until concrete has reached the proper strength as specified in the contract documents (typically about 3000 psi). Concrete strength should be verified with tests of properly constructed test cylinders or other approved methods. The use of concrete maturity meters can aid in the determination of concrete strength when they are properly calibrated against test cylinder results. The use of these devices can help verify concrete strengths for concrete exposed to "non-typical conditions" (such as a cantilevered slab exposed to cold winds) and can be an economical means of determining early strengths to facilitate stressing operations.

Stressing equipment should be operated by certified personnel who are experienced in post-tensioning stressing operations (see Chapter 21 for additional information). Stressing equipment should be accompanied by calibration records that are no more than six months old and should indicate the proper hydraulic gauge pressure to which the tendons are to be stressed to achieve the required final effective force.

Proper tendon stressing is monitored in two ways. First, the gauge reading on the hydraulic pump indicates the jacking force being applied at the tendon anchorage. Tendons should be stressed to the proper gauge pressure as indicated on the stressing equipment calibration records. Second, the elongation of the prestressing steel should be measured and compared to calculated values supplied by the post-tensioning supplier. Elongations should be measured to the nearest ⅛ in. and should fall within the allowable tolerances of +/– 7% for structural concrete and +/– 10% for slabs-on-ground unless more stringent tolerances are listed in the contract documents.

Accurate elongation measurements depend on proper marking and measurement procedures. The leading cause for elongations to be out of tolerance is a failure to follow these procedures. PTI's *Field Procedures Manual for Unbonded Single Strand Tendons*[20.2] provides instructions on proper marking and measuring.

20.2.4 Grouting

Grouting of bonded tendons should take place after the Licensed Design Professional has approved the tendon elongations, and within 40 days of installation of prestressing steel in very dry, non-aggressive environments. In aggressive environments with a very damp atmosphere, or in structures spanning over bodies of salt water, grouting should be completed within seven days of stressing of prestressing steel.

Grouting operations should follow the procedures outlined in an approved grouting plan that has been prepared and submitted by the contractor at least four weeks in advance of the scheduled grouting activities. The grout mix proportions contained in the approved grouting plan should be strictly adhered to in order to provide durable corrosion protection to the prestressing steel.

Proper grouting procedures and specific requirements for the items to be inspected during the grouting operation can be found in PTI's *Specification for Grouting of Post-Tensioned Structures*.[20.8]

20.2.5 Finishing

Finishing of the tendons involves removal of the stressing tail(s) and patching over the anchorage to provide long-term corrosion protection. For unbonded tendons, proper patching of the pocket former recess is critical for the long-term corrosion protection of the post-tensioned reinforcing.

Tendon tails can be removed using various methods, which should be approved in advance of the operation. The work should be inspected prior to patching to verify that the tendon tails have been cut back to a dimension that will result in the proper minimum concrete cover.

Patching should be done using an approved non-shrink material and mix design which will result in a durable patch that is free of voids and will preclude the intrusion of moisture that could result in corrosion of the anchorage or prestressing steel.

20.2.6 Special Inspection Requirements for Encapsulated (Aggressive Environment) Systems

Encapsulated systems are designed to provide a tendon that is watertight from end to end. There can be no exposed prestressing steel or anchorage components. During the inspection of the tendon installation, the sheathing should be checked to verify that all damaged areas, including pin holes, have been properly repaired. After the stressing tails have been removed they should be inspected to verify that they have been cut to a length to allow proper installation of the P/T coating filled protective end cap. The caps should be inspected for proper installation prior to patching the pocket former recesses.

20.3 POST CONSTRUCTION INSPECTION

Inspection of post-tensioned structures during service should be done on a periodic basis to assess the need for any preventative maintenance. The frequency of the inspections will depend on the durability features incorporated into the structure during its design and construction, and on the use and conditions to which the structure is exposed. Structures that are exposed to traffic, weather, deicing chemicals, salt water, or salt-laden air may warrant more frequent inspections. Inspections should focus on concrete deterioration or other conditions that could expose the prestressing steel or other tendon components to corrosion.

Structures with tendons that have been exposed, or tendons that appear corroded or broken, should be evaluated by an experienced engineer to determine if the structural integrity or capacity of the affected concrete member has been compromised.

REFERENCES

20.1 *Construction and Maintenance Procedures Manual for Post-Tensioned Slabs-on-Ground*, 2nd Edition, Post-Tensioning Institute, Phoenix, AZ, 1998.

20.2 *Field Procedures Manual for Unbonded Single Strand Tendons*, 3rd Edition, Post-Tensioning Institute, Phoenix, AZ, 2000.

20.3 *Training and Certification of Field Personnel for Bonded Post-Tensioning—Student Manual*, Post-Tensioning Institute, Phoenix, AZ, 2004.

20.4 *International Building Code*, IBC 2003, International Code Council, Inc., Falls Church, VA, 2003.

20.5 *Specification for Unbonded Single Strand Tendons*, 2nd Edition, Post-Tensioning Institute, Phoenix, AZ, 2000.

20.6 *Design of Post-Tensioned Slabs-on-Ground*, 3rd Edition, Post-Tensioning Institute, Phoenix, AZ, 2004.

20.7 *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-02, American Concrete Institute, Farmington Hills, MI, 2002.

20.8 *Specification for Grouting of Post-Tensioned Structures*, 2nd Edition, Post-Tensioning Institute, Phoenix, AZ, 2003.

POST-TENSIONING INSTITUTE CERTIFICATION PROGRAMS

21.1 INTRODUCTION

Cast-in-place post-tensioned concrete allows designers to realize the advantages of prestressed concrete while not being constrained to the various limitations inherent to precast, pre-tensioned concrete elements. Because post-tensioned concrete is typically site cast, the proper control of materials and processes plays an important role in producing a high quality finished product. PTI recognizes this need and addresses it through the implementation and use of several certification programs. These programs include certification of material producers and certification of field personnel, including installers and inspectors.

The Institute's material producer certification programs are designed to provide independent third-party verification of a plant's capabilities to produce material in accordance with applicable specifications. The programs include physical plant inspections, verification of production processes, and evaluation of material test results.

The installer and inspector certification programs include requirements for formal classroom training, achievement of minimum scores on closed book exams, and verification of on-the-job field experience. The inclusion of mandatory classroom training sets PTI's programs apart from other programs in the marketplace, and helps to ensure that installers have received the specialized training needed to maintain quality workmanship in the field.

Current model building codes include requirements for certification of material producers and installers by incorporating either PTI or ACI Standards Documents. The documents most often referenced are PTI's *Specification for Unbonded Single Strand Tendons*, and ACI's 423.6 Document (same title). The ACI 423.6 document is incorporated by reference into the ACI 318 *Building Code Requirements for Structural Concrete*. Both of these standards call for various levels of certification and list PTI's programs as meeting the specific requirements cited.

21.2 CERTIFICATION OF PLANTS PRODUCING PC STRAND FOR POST-TENSIONING APPLICATIONS

21.2.1 Scope and Applicability

Prestressing steel strand is typically specified to meet the requirements of ASTM A416/416M. This ASTM standard contains minimum physical requirements such as size, grade, strength properties, and relaxation characteristics. The PTI program for certification of the plants producing this material goes beyond these requirements to incorporate requirements for minimum testing intervals, standard reporting of the results and properties, and adds requirements for the testing of the corrosion potential of the strand.

This program provides independent verification of a plant's manufacturing capability to produce PC strand in accordance with the program requirements. This program is applicable to plants producing *uncoated* prestressed concrete strand used in all post-tensioning applications. For certification requirements for plants producing coated tendon systems for unbonded post-tensioning applications, see Section 21.3.

21.2.2 Program Overview

This program is based on the review of materials, test data, and manufacturing procedures during one scheduled inspection for each year that the plant is involved in the program. Between annual facility inspections, continuing certification is maintained subject to the random sampling and testing of finished production materials conducted twice per year on an unannounced schedule. Materials may be sampled at the production facility or at the materials' port of entry into the United States.

Each facility is inspected to evaluate the plant's quality control procedures, storage and protection methods, and marking, packaging, and handling techniques. Procedures must result in a finished product that meets all applicable specifications.

The program requires strict compliance with the physical material properties described in ASTM A416/416M including wire diameter, finished strand dimensions, yield strength, modulus of elasticity, breaking strength, and relaxation properties. Evaluation of a facility's in-house test data must indicate on-going compliance. Testing and traceability records are to be maintained at the facility for five years.

The program establishes a minimum testing interval of once per three years for conducting full 1000-hour relaxation tests on each size and type of strand, and requires annual 200-hour relaxation tests on each size and type of strand. Plants are required to conduct additional 1000-hour tests if they substantially change manufacturing processes or base materials.

Applicants must test each size and type of strand for its potential for hydrogen-induced stress corrosion once every three years. This test is performed according to ISO standards and results must be in compliance with standards contained in the program manual. Although not a part of the current ASTM specification for PC strand, this test is standard in many European countries and the Institute includes it as a certification requirement due to the influx of imported strand, which may be produced by methods that differ from North American production standards.

21.2.3 Verification of Compliance

Independent verification of compliance with the program is provided by a third-party certifying agency hired by the Post-Tensioning Institute. Certifying agencies are responsible for expeditiously conducting plant inspections and preparing an evaluation report that is forwarded to the Post-Tensioning Institute and to the inspected plant's designated representative. Initial and continuing certification under this program is based in part on the certifying agency's evaluation report.

The certifying agency is also responsible for the random sampling and testing of finished production materials and for the reporting of these results back to the manufacturer and the Institute.

21.3 CERTIFICATION OF PLANTS PRODUCING SINGLE-STRAND UNBONDED TENDONS

21.3.1 Scope and Applicability

Single-strand unbonded tendon systems are comprised of various components that are assembled to produce the finished product. The individual components can include prestressing steel, P/T coating, sheathing, anchors, wedges, and encapsulating features or parts. This program assigns responsibility to the fabricating facility to ensure that the components used are compatible with each other, and that the processes used result in a finished product that meets the requirements of PTI's *Specification for Unbonded Single Strand Tendons*.

21.3.2 Program Overview

PTI's Plant Certification Program is designed to ensure quality components, reliable fabrication processes, and responsible documentation. This industry-wide program provides professional independent verification that a supplier can produce unbonded single-strand tendons in compliance with the Post-Tensioning Institute's *Specification for Unbonded Single Strand Tendons*. This certification program also includes evaluation of calibration practices for jacks and gauges used for stressing unbonded single-strand tendons.

PTI Certification gives engineers, contractors and owners a proven way to prequalify single-strand unbonded post-tensioning suppliers throughout North America. The program involves detailed plant inspections and a review of the supplier's records, test data, fabrication procedures, material, equipment, and quality control program.

The PTI Certification process is thorough and rigorous. During the first year, the candidate supplier must pass one in-depth, pre-announced inspection and one unannounced plant inspection. To maintain its certification status, the supplier must pass two unannounced plant inspections each year thereafter.

During the inspections, the supplier is evaluated with regard to proper acceptance criteria for the strand and maintenance of adequate corrosion protection procedures during storage. It is required that the supplier maintain long-term records to allow identification of the strand supplier and suppliers' coil numbers for all tendons produced.

All anchorage systems must be accompanied by test data from an independent testing laboratory demonstrating that the assembled tendons comply with PTI specifications. Tendon sheathing must meet PTI tolerances for thickness and inside diameter, and must be free of all defects that may affect corrosion protection.

During tendon extrusion, the corrosion-protection coating must be applied over the entire tendon length and should completely fill the annular space between the helical shape of the strand and the sheathing.

For storage, handling, and shipping of fabricated tendons, the supplier must comply with all PTI specifications that minimize corrosion and damage to the materials.

Traceability of all tendon materials and components is central to the PTI Certification Program. For every project, the plant must keep records that ensure 100% traceability of the prestressing steel, anchors, wedges, couplers, sheathing, and corrosion-inhibiting coating. These records should be kept for a minimum of three years.

21.3.3 Verification of Compliance

Independent verification of compliance with the program is provided by a third-party certifying agency hired by the Post-Tensioning Institute. Certifying agencies are responsible for expeditiously conducting plant inspections and preparing an evaluation report that is forwarded to the Post-Tensioning Institute and to the inspected plant's designated representative. Initial and continuing certification under this program is based in part on the certifying agency's evaluation report.

21.3.4 Applicable Codes and Standards

Specific program requirements and grading criteria are detailed in the PTI's publication entitled *Manual for Certification of Plants Producing Unbonded Single-Strand Tendons*. This manual requires plants to be in compliance with the requirements specified in the latest edition of PTI's *Specification for Unbonded Single Strand Tendons*. Both this specification and the American Concrete Institute's *Specification for Unbonded Single-Strand Tendons and Commentary* (ACI 423.6-01/423.6R-01) require that tendons be fabricated in a plant certified by an externally audited quality assurance program. Both specifications specifically state that plants certified by the Post-Tensioning Institute meet this requirement.

21.3.5 Sample Guide Specification

To obtain the full benefit of the PTI Plant Certification Program, the following guide specification language can be used:

Material Quality Assurance

> The fabricator of the single-strand unbonded post-tensioning tendons shall be a fully certified Post-Tensioning Institute (PTI) Plant at the time of bidding and throughout the duration of the contract, as defined by the PTI Program for Certification of Plants Producing Unbonded Single-Strand Tendons.
>
> Upon request, the manufacturer shall provide a letter certifying that all products supplied to the job site meet the current PTI *Specifications for Unbonded Single Strand Tendons*.
>
> Verification of a plant's certification status can be obtained by contacting the Post-Tensioning Institute.

21.4 TRAINING AND CERTIFICATION OF FIELD PERSONNEL FOR UNBONDED POST-TENSIONING

21.4.1 Scope and Applicability

This program provides training and certification to all field personnel involved in the installation, stressing, finishing, and inspection of single-strand unbonded tendon systems utilized in all applications including slabs-on-ground, structural concrete members, and barrier cable applications.

This program includes training in field fundamentals, training for inspectors, and training that is specific to installers working on slabs-on-ground or structural concrete projects.

21.4.2 Field Fundamentals

The Field Fundamentals Certification is categorized as a Level 1 Certification and requires 16 hours of formal classroom training. The training encompasses the basic body of knowledge that should be possessed by all personnel involved with the installation and inspection of single-strand unbonded tendon systems. Topics covered in this training include the basic theories behind post-tensioned concrete, post-tensioning components, installation requirements, and special requirements for encapsulated systems. All of these topics are presented in a manner that relates them to both slabs-on-ground and structural concrete members. The training also includes an introduction to the installation of prestressed barrier cable systems.

At the conclusion of the training, applicants are required to take a closed book exam and achieve a minimum passing score before certification is issued.

21.4.3 Superstructure Ironworker

A Level 2 – Superstructure Ironworker Certification requires an additional 24 hours of formal classroom training and adds to the skills taught in the Field Fundamentals class through in-depth discussions of actual installation techniques. This program utilizes hands-on demonstrations involving stressing of tendons and the use of various tools unique to the installation of unbonded post-tensioning systems. This program is designed specifically for individuals who install unbonded post-tensioned reinforcing in structural concrete members.

Prior to issuance of certification under this program the applicant must achieve a minimum passing grade on a closed book exam and must present verifiable evidence indicating that they have a minimum of 500 hours of field experience in the installation of single-strand unbonded tendons.

21.4.4 Post-Tensioning Inspector

Applicants for this Level 2 – Post-Tensioning Inspector Certification are required to possess a current Level 1 – Field Fundamentals Certification and must attend an additional 16 hours of formal classroom training. This builds upon the concepts and applications learned in the Field Fundamentals class to give the post-tensioning inspector a solid knowledge base relating to the inspection of post-tensioned reinforcing in both structural concrete members and slab-on-ground foundations. This training encompasses plan reading skills, inspection techniques, reporting requirements, and addresses safety related issues.

Applicants must pass a closed book exam and must have a minimum of one year of inspection experience.

21.4.5 Slab-On-Ground Installer

This is a standalone specialty class designed specifically for individuals who install, stress, and finish unbonded tendon systems in residential and light commercial ground-supported slabs. Applicants must attend 12 hours of formal classroom training which includes plan reading skills, installation requirements, and hands-on demonstrations with stressing equipment.

Applicants must pass a closed book exam and present verifiable evidence of 500 hours of field experience installing single-strand unbonded tendons.

21.4.6 Sample Guide Specification

To obtain the full benefit of the PTI Field Certification Programs, the following guide specification language can be used to ensure that installers meet recommended certification requirements. It is recommended that inspection and quality control personnel on all types of projects utilizing single-strand unbonded tendons be certified under the Level 2 – Post-Tensioning Inspector program:

Quality Assurance (Structural Post-Tensioned Concrete)

> All individuals installing both prestressed and non-prestressed beam and/or slab reinforcing, or involved in the stressing operations, are to be certified as having successfully completed the requirements of the PTI Level 1 – Field Fundamentals Program. In addition, the installer's project supervisor and at least (one installer out of four) <insert ratio> are to be certified as having successfully completed the requirements of the PTI Level 2 – Superstructure Ironworker Program.
>
> Each stressing operation is to be performed by an individual who is certified as having successfully completed the requirements of the PTI Level 2 – Superstructure Ironworker Program.

PTI recommends that all of the personnel involved in the installation of reinforcing in post-tensioned structural members have at least a Level 1 certification, and that at least one installer out of four has a Level 2 certification. Note, however, that the wording does allow the specifier to insert a ratio that requires a higher number of Level 2 Certified installers if they choose. Note that this recommendation does not apply to site personnel working exclusively on non-prestressed members such as column cages and walls.

Quality Assurance (Post-Tensioned slabs-on-ground used in residential and light commercial construction)

> On ground-supported post-tensioned slabs for residential or light commercial construction, the personnel responsible for the installation of both prestressed and non-prestressed reinforcing shall be directly supervised by an on-site person who is certified as having completed the requirements of the PTI Level 1 – Field Fundamentals Program or PTI's Slab-on-Ground Installer program. In addition, at least one installer out of two is to be certified as having successfully completed the requirements of either the PTI Level 1 – Field Fundamentals Program or PTI's Slab-on-Ground Installer program.
>
> All individuals involved in the stressing operation are to be certified as having successfully completed the requirements of either PTI's Slab-on-Ground Installer program or PTI's Level 2 – Superstructure Ironworker Program.

PTI recommends that the on-site supervisor, and 50% of the field personnel working on a slab-on-ground project, be certified under either the Level 1 program or the Slab-on-Ground Installer program. Because stressing operations involve the use of special equipment at extremely high levels of force, the recommendation is that *all* personnel involved in that operation be certified under either the Level 2 program or the Slab-on-Ground Installer program, both of which incorporate hands-on training in the use of stressing equipment.

21.5 TRAINING AND CERTIFICATION OF FIELD PERSONNEL FOR BONDED POST-TENSIONING

21.5.1 Scope and Applicability

This training program is specifically aimed at field personnel involved in the installation, stressing, grouting, and inspection of grouted post-tensioning systems used in bridge and building construction, including multi-strand and bar post-tensioning systems. Certification of installers and inspectors under this program is a requirement on some state department of transportation projects.

21.5.2 Program Overview

This intensive course requires 24 hours of formal classroom training and covers the installation of various types of duct systems, various methods for installation of prestressing steel (including strand and bar), stressing equipment operation and procedures, grouting techniques, and operation of grouting equipment. The program incorporates live demonstrations using field production equipment and materials.

Certification under this program is issued in two categories. Level 1 Certification requires a passing grade on a closed book exam after completing the required training. Level 2 adds the requirement of providing verifiable evidence of 500 hours of field experience in each of the three categories; installation, stressing, and grouting.

The recommendation for requirements for certification of field installers is contained in the sample guide specification below. It is recommended that inspection and quality control personnel on each project be certified as meeting the Level 2 requirements of this program.

21.5.3 Sample Guide Specification

To obtain the full benefit of this PTI Field Certification Program, the following guide specification language can be used:

Quality Assurance (Bonded Post-Tensioned Reinforcing)

> All individuals installing, stressing, or grouting the bonded post-tensioned reinforcing system (which includes supports, ducts, grout tubes, prestressing steel, and anchorages) are to be certified as having successfully completed the requirements for Level 1 or Level 2 Certification in PTI's Training and Certification of Field Personnel for Bonded Post-Tensioning program.
>
> Each operation (installation, stressing, and grouting) must be directly overseen by an on-site supervisor who is certified as having completed the requirements for Level 2 Certification under PTI's Training and Certification of Field Personnel for Bonded Post-Tensioning program.

21.6 SUMMARY

PTI has been involved in the administration of certification programs since 1988 and its programs have been proven effective in raising the overall quality within the post-tensioning industry. Certification programs can be a useful and effective tool that can be used by an owner or specifier to pre-qualify suppliers and field personnel for their projects. By participating in certification programs, suppliers and field personnel demonstrate an extra level of dedication and are able to show proof of formal training that can assist in achieving the level of quality desired by a project owner.

APPENDICES

DESIGN AIDS

A.1 FRICTION LOSSES

The friction along a cable is calculated according to the formula:

$$T_x = T_o \, e^{-(\mu\alpha + kx)} \tag{1}$$

Or if $(\mu\alpha + kx)$ is not greater than 0.15, according to the following simplified formula:

$$T_x = \frac{T_o}{1 + \mu\alpha + kx} \tag{2}$$

Where:

T_x = prestressing force at point x
T_o = prestressing force at jacking end
e = base of Naperian logarithms
μ = curvature friction coefficient
α = total angular change in radians from jacking end to point x
k = wobble friction coefficient per ft of tendon
x = length of cable from jacking end to point x in feet

The nomogram (Fig. A.3) is provided for use with formula (1) above. It includes scales for T_x, T_a, T_o or f_{sx}, f_{sa}, f_{so}, and $(\mu\alpha + kx)$. The dimensionless value for the friction $(\mu\alpha + kx)$ can be taken from Figs. A.1 and A.2. When one of the three forces or stresses is known, the required values of the two others may be found by connecting the given force (stress) and the value $(\mu\alpha + kx)$ with a straight line and then extending it over the 4 scales.

Applicable code requirements must be observed, but in the absence of these, the values shown in Table A.1 may be applied for friction coefficients.

Table A.1 - Friction Coefficients

Type of Duct	Range of Values		Recommended for Calculations*	
	μ	k	μ	k
Flexible tubing non-galvanized	0.18 – 0.26	$5 - 10 \times 10^{-4}$/ft	0.22	7.5×10^{-4}/ft
Flexible tubing galvanized	0.14 – 0.22	$3 - 7 \times 10^{-4}$/ft	0.18	5.0×10^{-4}/ft
Rigid thin wall tubing non-galvanized	0.20 – 0.30	$1 - 5 \times 10^{-4}$/ft	0.25	3.0×10^{-4}/ft
Rigid thin wall tubing galvanized	0.16 – 0.24	$0 - 4 \times 10^{-4}$/ft	0.20	2.0×10^{-4}/ft
Greased and wrapped	0.05 – 0.15	$5 - 15 \times 10^{-4}$/ft	0.07	10×10^{-4}/ft

* Practice has shown that friction losses can vary from case to case. The recommended values given above are suggested for calculating the friction losses but in some instances the extreme values should be considered.

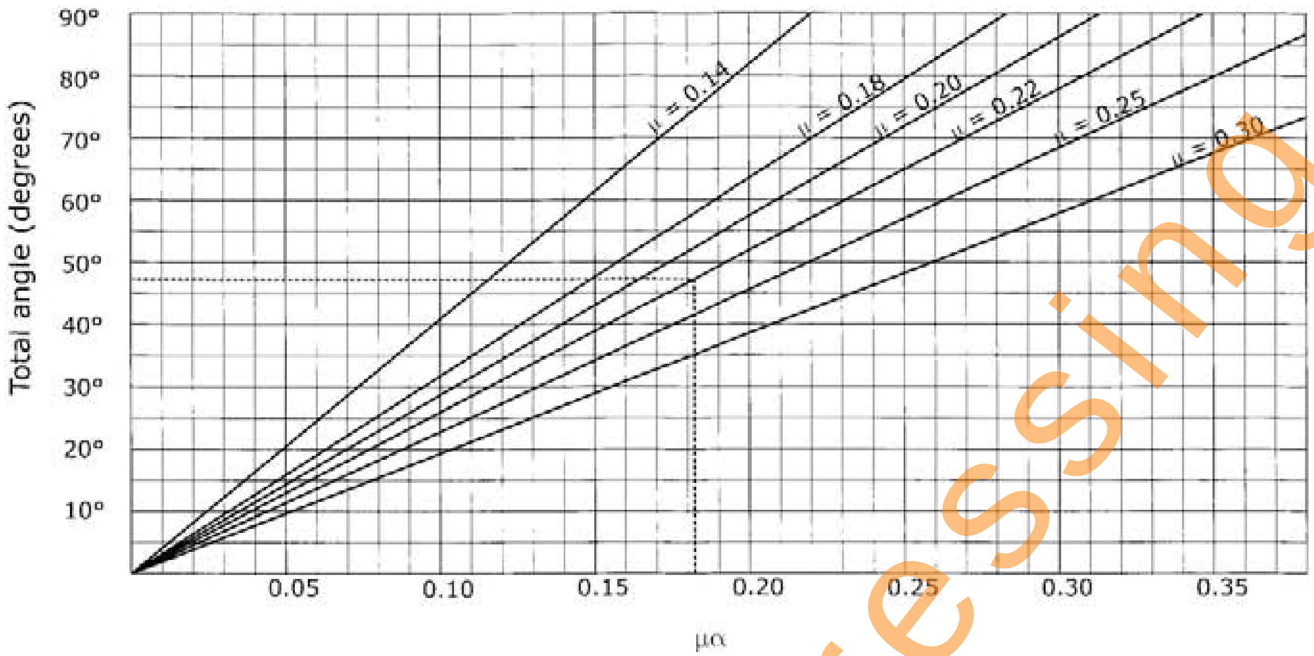


Fig. A.1 Diagram for Determining μα

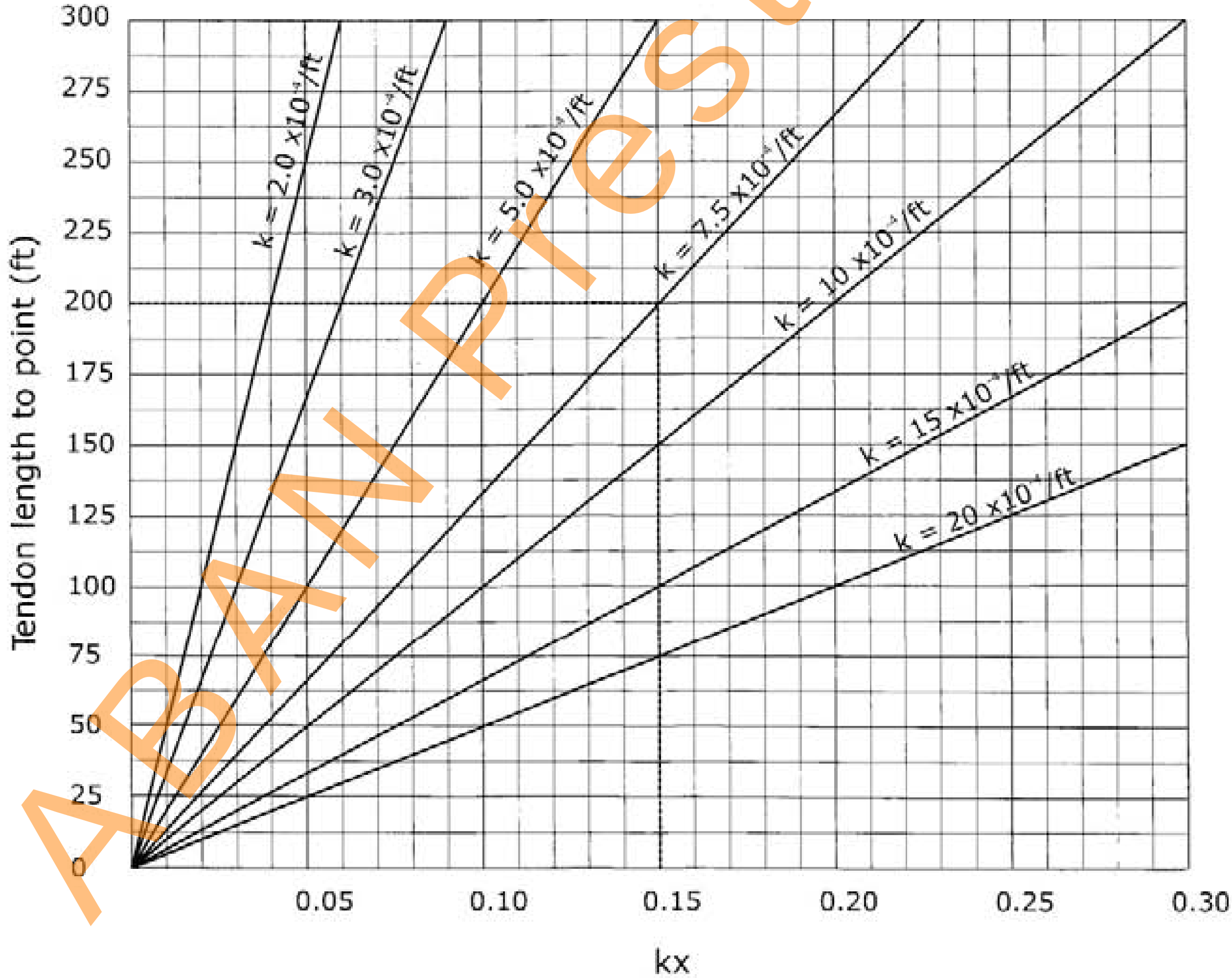


Fig. A.2 Diagram for Determining kx

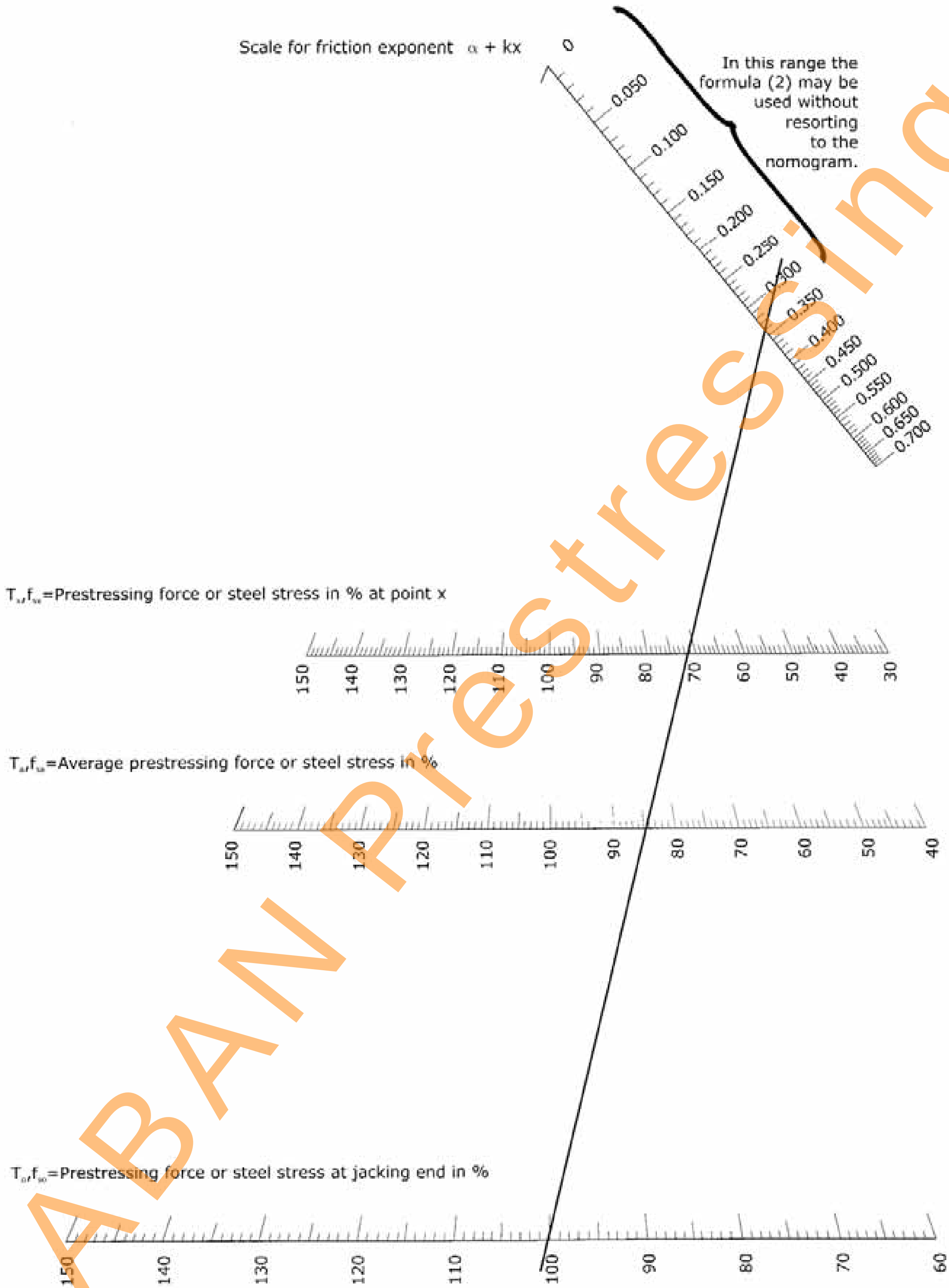
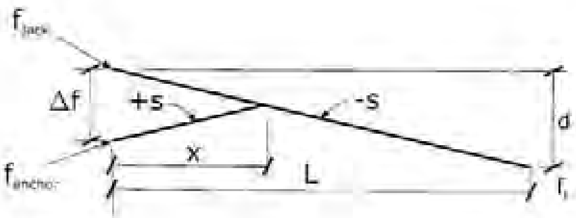


Fig. A.3 Nomogram for Determining the Prestressing Forces and Steel Stresses Along a Tendon

A.2 DERIVATION OF FORMULAS FOR CALCULATING THE EFFECTS OF ANCHOR SET

The effects of anchor set on tendon stresses may be calculated with sufficient accuracy for most conventional applications in accordance with the diagram and formulas presented below.



Δf = Change in stress due to anchor set, ksi
d = Friction loss in length, L, ksi
x = Length influenced by anchor set, ft
L = Length to point where loss is known, ft
ΔL = Anchor set, in.
E = Modulus of elasticity, ksi

$$E = \frac{\text{Unit Stress}}{\text{Unit Strain}} = \frac{f_{avg}}{\Delta L / x} = \frac{f_{avg}\, x}{\Delta L}$$

$$f_{avg} = \frac{E \Delta L}{x}$$

$$\frac{\Delta f}{2} = \frac{E \Delta L}{12\, x} \qquad \text{Units Correct}$$

$$\Delta L = \frac{P_{avg}\, x}{A E} = \frac{f_{avg}\, x}{E}$$

$$f_{avg} = \frac{E \Delta L}{12\, x}$$

$$\frac{\Delta f}{2} = \frac{E \Delta L}{12\, x} \qquad \text{Units Corrected}$$

$$\Delta f = \frac{E \Delta L}{6\, x} \qquad \Delta L \text{ \& x known}$$

By Similar Triangles:

$$\frac{x}{\Delta f / 2} = \frac{L}{d}$$

$$\Delta f = \frac{2 \times d}{L} \qquad \text{x known}$$

$$x = \frac{E(\Delta L)L}{6x \times 2d}$$

$$x^2 = \frac{E(\Delta L)L}{12\, d}$$

$$x = \sqrt{\frac{E(\Delta L)L}{12\, d}} \qquad \Delta L \text{ known}$$

Also from $\Delta f = \dfrac{E \Delta L}{6\, x}$ & $\Delta f = \dfrac{2 \times d}{L}$

$$x = \frac{E \Delta L}{6\, \Delta f} = x = \frac{L\, \Delta f}{2\, d}$$

$$\Delta f^2 = \frac{E \Delta L\, d}{3\, L}$$

$$\Delta f = \sqrt{\frac{E \Delta L\, d}{3\, L}} \qquad \Delta L \text{ known}$$

When measuring anchor set, the tendon elongation within the jack must be considered:

Assume the jacking force to be $0.8 f_{pu}$

$= 0.8 \times 270 = 216$ ksi

Anchor set is typically assumed to be ¼ in.

Assume the length of stressing jack to be 4 ft-0 in.

$E = 28.5 \times 10^3$ ksi

Elongation of tendon within the jack:

$$\Delta L_{jack} = \frac{216 \times 4 \times 12}{28.5 \times 10^3} = 0.36 \text{ in.}$$

Total elongation lost during anchor set:

= elongation within the jack + anchor set

= 0.36 in. + 0.25 in. = 0.61 in.

Table A.2 - Concrete Stresses (psi)

f'_c	$0.45f'_c$	$0.6f'_c$	$\sqrt{f'_c}$	$0.6\sqrt{f'_c}$	$2\sqrt{f'_c}$	$3\sqrt{f'_c}$	$3.5\sqrt{f'_c}$	$4\sqrt{f'_c}$	$5\sqrt{f'_c}$	$6\sqrt{f'_c}$	$12\sqrt{f'_c}$
3000	1350	1800	55	33	110	164	192	219	274	329	657
3500	1575	2100	59	35	118	177	207	237	296	355	710
4000	1800	2400	63	38	126	190	221	253	316	379	759
4500	2025	2700	67	40	134	201	235	268	335	402	805
5000	2250	3000	71	42	141	212	247	283	354	424	849
5500	2475	3300	74	44	148	222	260	297	371	445	890
6000	2700	3600	77	46	155	232	271	310	387	465	930
6500	2925	3900	81	48	161	242	281	322	403	484	967
7000	3150	4200	84	50	167	251	293	335	418	502	1004
7500	3375	4500	87	52	173	260	303	346	433	519	1039
8000	3600	4800	89	54	179	268	313	358	447	537	1073

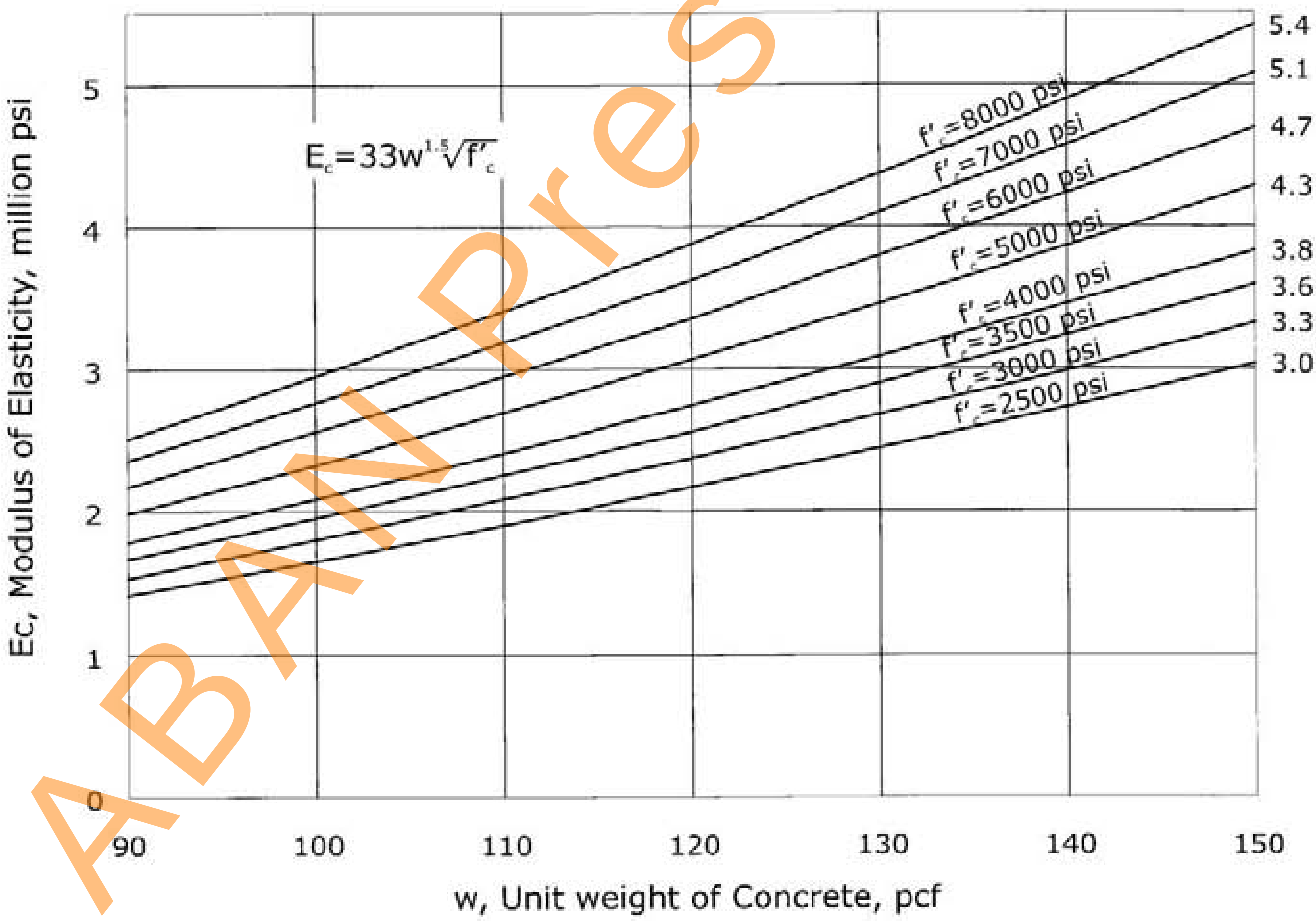


Fig. A.4 Concrete Modulus of Elasticity as Affected by Unit Weight and Strength

Table A.3 - Properties of Seven-Wire Strand, f_{pu} = 270 ksi

Nominal Diameter, in.	⅜"	7/16"	½"	0.60"
Area, sq in.	0.085	0.115	0.153	0.217
Weight, plf	0.29	0.39	0.52	0.74
$0.7f_{pu}A_{ps}$, kips	16.1	21.7	28.9	41.0
$0.8f_{pu}A_{ps}$, kips	18.4	24.8	33.0	46.9
$f_{pu}A_{ps}$, kips	23.0	31.0	41.3	58.6

Table A.4 - Properties of Prestressing Wire

Diameter	0.192"	0.196"	0.25"	0.276"
Area, sq in.	0.0289	0.0302	0.0491	0.0598
Weight, plf	0.098	0.10	0.17	0.20
Ult. strength, f_{pu} ksi	250	250	240	235
$0.7f_{pu}A_{ps}$, kips	5.05	5.28	8.25	9.84
$0.8f_{pu}A_{ps}$, kips	5.78	6.04	9.42	11.24
$f_{pu}A_{ps}$, kips	7.22	7.55	11.78	14.05

Table A.5 - Properties of Prestressing Bars

Nominal Diameter, in.	⅝"	1"	1"	1-¼"	1-¼"	1-⅜"
Area, sq in.	0.28	0.85	0.85	1.25	1.25	1.56
Weight, plf	0.98	3.01	3.01	4.39	4.39	5.56
Ult. strength, f_{pu} ksi	157	150	160	150	160	150
$0.66f_{pu}A_{ps}$, kips	28.7	84.4	90.0	123.8	132	154.4
$0.75f_{pu}A_{ps}$, kips	32.6	95.9	102.2	140.6	150	175.5
$f_{pu}A_{ps}$, kips	43.5	127.8	136.3	187.5	200	234

Table A.6 - Properties of Common Styles of Welded Wire Fabric

	Style Designation	Spacing of wires, in.		Size of wires, AS & W gage		Sectional area, sq in. per ft		Weight, lb per 100 sq ft
		Longit.	Trans.	Longit.	Trans.	Longit.	Trans.	
Two-Way Types	2 × 2-10/10	2	2	10	10	0.086	0.086	60
	2 × 2-14/14*	2	2	12	12	0.052	0.052	37
	2 × 2-12/12*	2	2	14	14	0.030	0.030	21
	3 × 3-8/8	3	3	8	8	0.082	0.082	58
	3 × 3-10/10	3	3	10	10	0.057	0.057	41
	3 × 3-12/12*	3	3	12	12	0.035	0.035	25
	3 × 3-14/14*	3	3	14	14	0.020	0.020	14
	4 × 4-4/4	4	4	4	4	0.120	0.120	85
	4 × 4-6/6	4	4	6	6	0.087	0.087	62
	4 × 4-8/8	4	4	8	8	0.062	0.062	44
	4 × 4-10/10	4	4	10	10	0.043	0.043	31
	4 × 4-12/12*	4	4	12	12	0.026	0.026	19
	6 × 6-0/0	6	6	0	0	0.148	0.148	107
	6 × 6-2/2	6	6	2	2	0.108	0.108	78
	6 × 6-4/4	6	6	4	4	0.080	0.080	58
	6 × 6-4/6	6	6	4	6	0.080	0.058	50
	6 × 6-6/6	6	6	6	6	0.058	0.058	42
	6 × 6-8/8	6	6	8	8	0.041	0.041	30
	6 × 6-10/10	6	6	12	12	0.029	0.029	21
One-Way Types	2 × 12-0/4	2	12	0	4	0.443	0.040	169
	2 × 12-2/6	2	12	2	6	0.325	0.029	124
	2 × 12-4/8	2	12	4	8	0.239	0.021	91
	2 × 12-6/10	2	12	6	10	0.174	0.014	66
	2 × 12-8/12	2	12	8	12	0.124	0.009	46
	3 × 12-0/4	3	12	0	4	0.295	0.040	119
	3 × 12-2/6	3	12	2	6	0.216	0.029	87
	3 × 12-4/8	3	12	4	7	0.159	0.021	64
	3 × 12-6/10	3	12	6	10	0.116	0.014	46
	3 × 12-8/12	3	12	8	12	0.082	0.009	32
	4 × 8-8/12	4	8	8	12	0.062	0.013	27
	4 × 8-10/12	4	8	10	12	0.043	0.013	20
	4 × 12-0/4	4	12	0	4	0.221	0.040	94
	4 × 12-2/6	4	12	2	6	0.162	0.029	69
	4 × 12-4/8	4	12	4	8	0.120	0.021	51
	4 × 12-6/10	4	12	6	10	0.087	0.014	36
	4 × 12-10/12	4	12	10	12	0.043	0.009	19
	6 × 12-00/4	6	12	00	4	0.172	0.040	78
	6 × 12-0/4	6	12	0	4	0.148	0.040	69
	6 × 12-2/2	6	12	2	2	0.108	0.054	59
	6 × 12-4/4	6	12	4	4	0.080	0.040	44
	6 × 12-6/6	6	12	6	6	0.058	0.029	32

*Usually furnished only in galvanized wire.

Table A.7 - Properties of ASTM Standard Reinforcing Bars

Bar Size Designation	Area in^2	Weight lbs/ft	Diameter in.*
#3	0.11	0.376	0.375
#4	0.20	0.668	0.500
#5	0.31	1.043	0.625
#6	0.44	1.502	0.750
#7	0.60	2.044	0.875
#8	0.79	2.670	1.000
#9	1.00	3.400	1.128
#10	1.27	4.303	1.270
#11	1.56	5.313	1.410
#14	2.25	7.650	1.693
#18	4.00	13.600	2.257

Current ASTM Specifications cover bar sizes #14 and #18 in A615 Grade 60 and in A706 only.

*Nominal dimensions

CONVERSION FACTORS

Table B.1 - Conversion Factors

Quantity	From English Units	To Metric Units	Multiply By
Length	mile	km	1.609347
	yard	m	0.9144
	foot	m	0.3048
	inch	mm	25.40
Area	square mile	km^2	2.590
	acre	m^2	4047
	square yard	m^2	0.8361
	square foot	m^2	0.092 90
	square inch	mm^2	645.2
Volume	cubic yard	m^3	0.7646
	cubic foot	m^3	0.02832
	100 board feet	m^3	0.2360
	gallon	L (1000 cm^3)	3.785
Mass	lb	kg	0.4536
	kip (1000 lb)	metric ton (1000kg)	0.4536
Mass/unit length	plf	kg/m	1.488
Mass/unit area	psf	kg/m^2	4.882
Mass density	pcf	kg/m^3	16.02
Force	lb	N	4.448
	kip	kN	4.448
Force/unit length	plf	N/m	14.59
	klf	kN/m	14.59
Pressure, stress, modules of elasticity	psf	Pa	47.88
	ksf	kPa	47.88
	psi	kPa	6.895
	ksi	MPa	6.895
Bending moment, torque	ft-lb	Nm	1.356
	ft-kip	kNm	1.356
Moment of mass	lb-ft	kgm	0.1383
Moment of Inertia	in^4	mm^4	416,200
Section modulus	in^3	mm^3	16,390
Velocity, speed	ft/s	m/s	0.3048
Acceleration	f/s^2	m/s^2	0.3048
Momentum	lb-ft/sec	kgm/s	0.1383
Angular momentum	lb-ft^2/s	kgm^2/s	0.04214
Plane angle	degree	rad	0.01745

INDEX