Post-Tensioning Manual

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INTRODUCTION

Modern development of prestressed concrete is attributed to Eugene Freyssinet of France who started using high strength steel wires for post-tensioning or prestressing concrete beams in 1928. 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, the Magnel system of post-tensioning was developed by Professor Magnel of Belgium. 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 war damaged bridges throughout much of Europe.

The early development of the prestressed concrete industry in the United States and Canada, which for practical purposes started in the early 1950's, was predominantly oriented towards factory production of precast-prestressed elements for highway bridges. There were, however, many notable exceptions utilizing post-tensioned prestressed concrete construction. In the 1960's the use of post-tensioned box girder bridges became predominant in California and other Western states. During the same period, the use of unbonded tendons for building floor systems became more widespread; and new applications emerged in the use of post-tensioned foundations for single and multi-family residences on expansive and compressible soils, and in the use of prestressed rock and soil anchors for a variety of tie-back and tie-down structural functions. The use of post-tensioned nuclear containments also began in the 1960's. As a result of these new markets, and a more widespread awareness of the advantages of post-tensioning among engineers, architects and owners, the use of post-tensioning increased more than 400 percent in the period 1965-1985. The economic and structural advantages provided by post-tensioning which have generated this remarkable increase in usage include: reduced structural depth; water-tight, virtually crack-free slabs; control of deflection; the esthetic potential of cast-in-place concrete; and, longer spans at an economical cost. In addition to these advantages, the increase in the use of post-tensioning is related to the development of the capability of post-tensioning materials fabricators to provide services and materials to meet a wide variety of job requirements. Companies which supply and install post-tensioning materials have a background of many years of successful experience from which they can provide assistance to architects, engineers, and contractors.

The continued rapid growth of the use of post-tensioned structural systems, as well as new developments in technology and changes in building codes in the past four years, have made necessary this fourth edition. The coverage of post-tensioning hardware or systems, specifications, and design, detailing and construction procedures contained in the third edition have all been up-dated to reflect current practices. New specifications are included for unbonded tendons, and up-dated recommendations are presented on rock and soil anchors. It is hoped that publication of the fourth edition of this manual will contribute to increased understanding and effective use of the powerful structural, economic and esthetic advantages afforded by post-tensioning.

Chapter I

Applications of Post-Tensioning

1.1 GENERAL

In this chapter, specific project examples are presented to illustrate the wide range of applications in which the use of post-tensioning has been found to be beneficial. In some cases, the descriptions of projects illustrated include discussions of typical construction procedures and details. The tendon types and sizes mentioned in the various examples are considered to be representative applications. However, in most cases, other types and sizes of tendons might have been used, and it is not to be inferred that the examples show the only appropriate post-tensioning solution for the job. See Section 2.1 for a brief discussion of the three basic types of post-tensioning tendons.

1.2 LOW-RISE BUILDINGS

1.2.1 The George Moscone Convention Center San Francisco, California

The Moscone Convention Center, located within walking distance of downtown San Francisco, occupies an entire city block and provides 261,000 square feet of column-free exhibition space with a maximum ceiling height of 37 feet. The roof structure was designed to support 3 feet of soil or 3-stories of light-frame buildings. The 300-foot wide by 800-foot long exhibition hall is spanned by eight pairs of concrete arches (Fig. 7.7) providing a column-free underground space which is the largest of its kind in the world.

The successful design and construction of these arches depended on the use of post-tensioned arch ties in three ways. First, the thrust of 6,000 tons could not be resisted by the abutting soil. Therefore, multi-strand tie cables were used to resist the force. Secondly, the ties were used to physically move one end of each arch 3-1/2 inches inward, thus forcing the crown up 4 inches. This procedure was carried out after the roof was loaded with 400 psf. With the addition of all dead and live loads, totalling 800 psf, the arch crown was expected to remain above its original position. Thirdly, the cables were curved vertically within the 6-foot thick mat foundation (convexed upward) to exert downward forces, thus helping counteract the large uplift from ground water. The ground water level is 10 feet above the finished floor of the exhibit hall. Post-tensioning was also used to insure safe transfer of the horizontal shear between the arch abutments and the mat. A system of inclined and looped strand tendons between abutment and mat was utilized for this purpose.

Fig. 1.1 — George Moscone Convention Center, San Francisco, California.

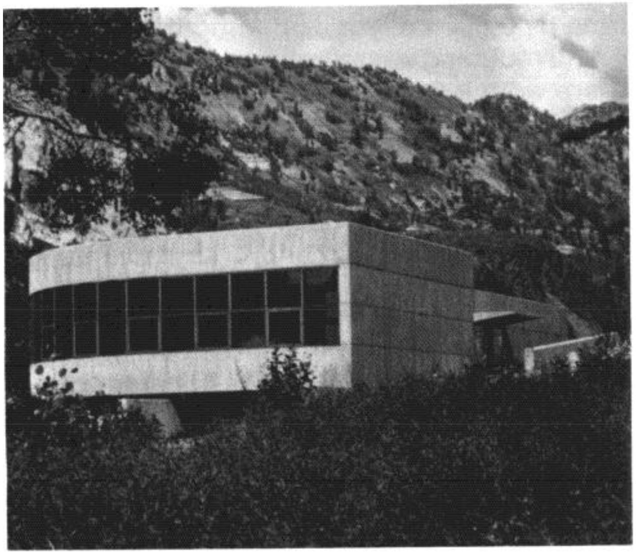


Fig 1 2 — Rothman Residence, Alta Utah

1.2.2 Rothman Residence Alta, Utah

The Rothman Residence, Fig. 7.2, containing 5,900 square feet of floor area, is possibly the first major application of post-tensioning to residential floor and roof construction.

The selection of the structural and architectural system was based on the following requirements:

1. Provide a system which could adapt to the curved shapes of the building.
2. Provide a structural system which could handle the exceptional design conditions such as large spans and immense snow loads.
3. Provide an exterior finish which could tolerate the severe mountain climate and be able to run directly into the ground on a sloping site.
4. Provide an exterior finish which would be compatible in color and spirit with the natural rock out-crops which surrounded the site.
5. Provide an interior finish directly with the structural system which could eliminate the need for extra interior finishes such as dropped ceilings.

Only exposed, poured in place concrete met all of the above requirements.

A cost estimate was made to compare a conventionally reinforced concrete system with a post-tensioned system. The post-tensioned system proved to be the most economical and was, therefore, selected for the project. Some of the reasons that economy was achieved by the selection of a post-tensioned prestressed system areas follows: (1) reduced slab thickness, (2) reduced dead loads which reduced the size of the spread footings, (3) reduced seismic mass which reduced the number and size of the shear walls, (4) elimination of extra reinforcement and beams around roof skylight openings, (5) ability of the post-tensioned prestressed system to handle the large spans with a flat slab design, and the bonus of crack control on the exposed interior slab bottoms.

1.2.3 Pace University Library Building Pleasantville

The Pace University Library, shown in fig. 7.3, was originally designed and bid as a structural steel frame building. The low bid for this design was $950,000.00. Redesign of the building utilizing post-tensioned flat plate construction for the floors and post-tensioned beams and one-way slabs for the roof resulted in a low bid of $700,000.00, a savings of 26 percent. A 7-1/2 inch thick post-tensioned flat plate with column capitals was used for the second floor stack room which had a design live load of 150 pounds per square foot. For the mezzanine reading area with a 60 pound per square foot livd load, a 7-1/2 inch thick flat plate was used without column capitals. The flat plate spans were generally 22 feet 6 inches in both directions. The roof consists of 6 inch thick one-way slabs spanning 22 feet 6 inches between 11 inch by 16 inch beams. Both beams and slabs were cast and post-tensioned along a 22 degree slope.

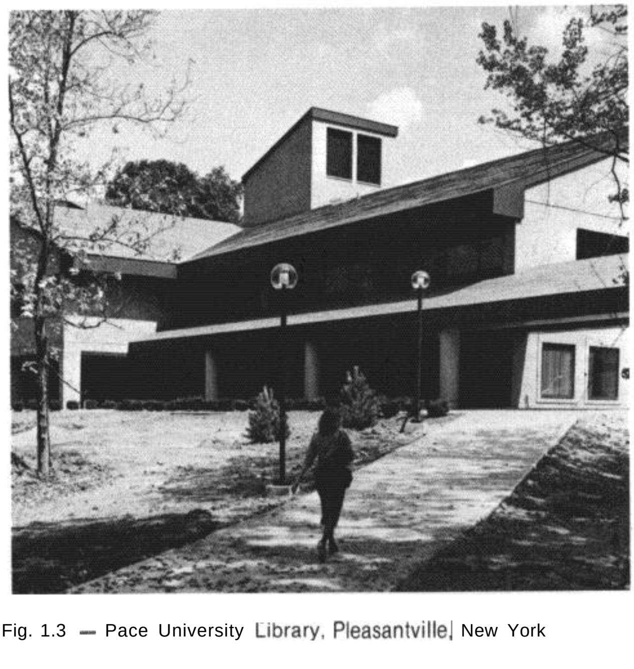


Fig. 1.3 — Pace University Library, Pleasantville, New York

1.2.4 Brown and Root Concourse Office Building Houston, Texas

The unique shape of the 600,000 square-foot three-story office building in Houston shown in Fig. 7.4 has been described as "hour-glass," "bow-tie," and "butterfly". By definition, the building consists of two triangles arranged point to point with a square-shaped core at their intersection. The bow-tie is sliced from end to end by a vaulted aluminum and plexiglass skylight, 500' long, which lights the main interior corridor of the building at all three floors. Pedestrian bridges, six in number, cross thecorridor, which alsofunctionsasan atrium, at the second and third floors. At each end of the bow-tie, six circular pods house mechanical and support facilities for the building, freeing the interior for open plan space for 2,300 employees. The core area contains escalators and serves as a lobby and employee entrance.

The structure was redesigned from a conventional reinforced concrete to a post-tensioned concrete scheme, reducing volume of concrete by 2,800 cu. yds., building height by 16 in. and reducing construction cost by $500,000. There were a limited number of typical joists, beams and columns because the unusual building shape caused every joist and beam line in a delta (one of four triangles formed by bisecting each half of the building) to be different. This unusual and nontypical building configuration required comprehensive three-dimensional wind, gravity, and temperature computer analysis to predict the design forces and moments. The core or center of the "bow-tie" shaped building was designed as an independent framework containing heavily prestressed simple spans 75 ft. long with 25 foot cantilevers, and was supported by only eight columns. The building shape dictated each floor be separated into five parts — the core and four deltas-for construction. Each delta was divided into five pours, with construction of deltas averaging 19 days. Floor framing began 20 days into construction, and the entire structure was completed one week ahead of schedule in spite of the extra time required for preparing design alternates.

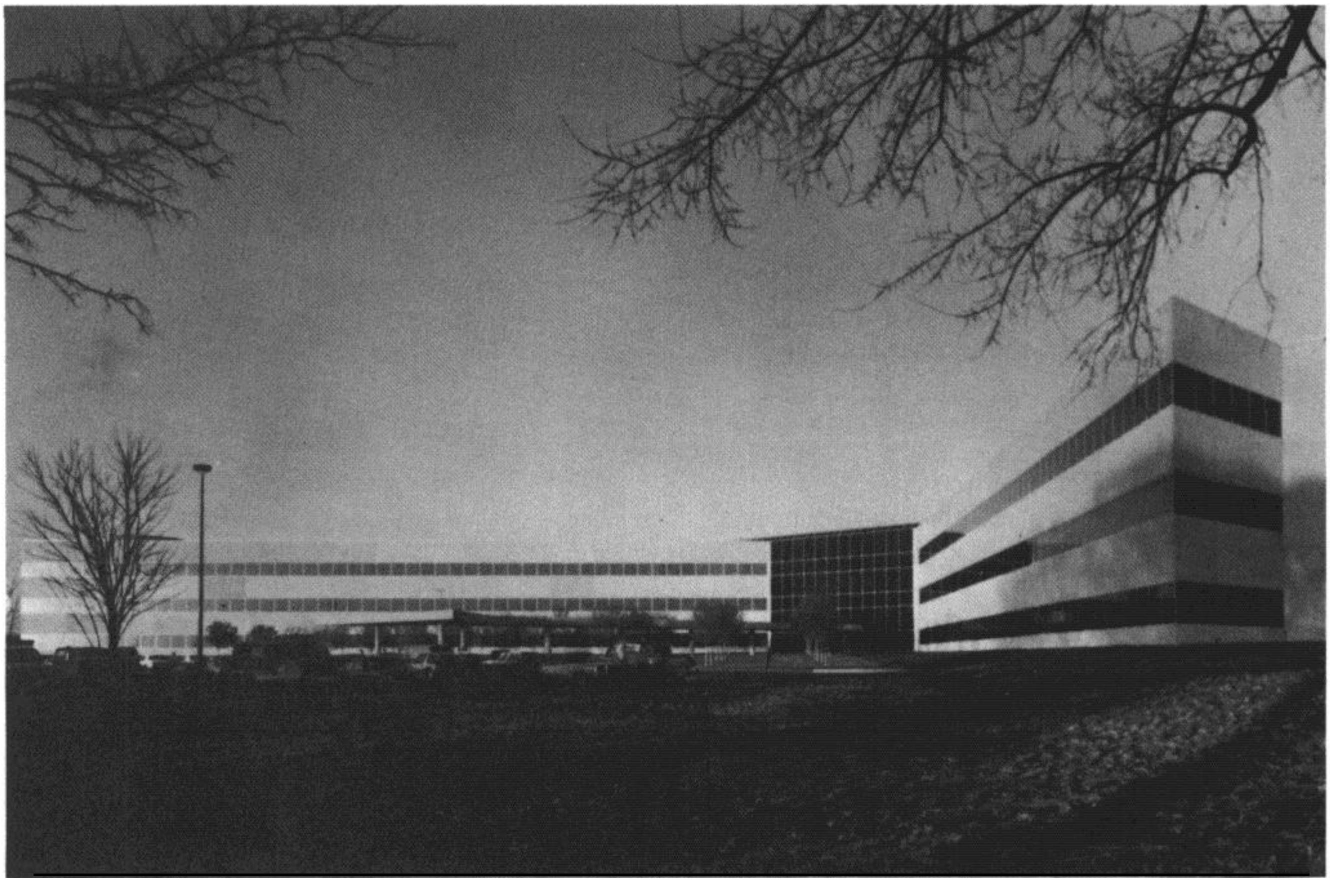


Fig. 1.4 Brown and Root Concourse Office Building, Houston, Texas.

1.3 MID-RISE BUILDINGS

1.3.1 Continental Park Plaza
El Segundo, California

The Continental Park Plaza project (Fig. 7.5) incorporates a six-story office building with approximately 520,000 square feet of lease-able area, and an adjacent 9-level parking structure for 1770 cars of approximately 500,000 square feet including the roof top health club and tennis courts. Nearly the entire 1.1 million square feet of elevated deck is post-tensioned. Because of its irregular shape and stepped-back terraced design, a flexible floor system was required which would be adaptable to each different floor and yet capable of being consistent so that economy and speed of construction would be maintained. Typical bay sizes were 32 feet by 32 feet, and because of a critical height limitation, alternate structural systems would have been unacceptable. Coordination with mechanical was also important, as the owner desired 9 foot ceilings. The floor system was designed for a live load of 75 pounds per square foot and additional dead load of 25 pounds per square foot to provide maximum flexibility for tenants. Typical balconies and terraces are cantilevered 12 feet 4 inches. All these design criteria were met by an 8-1/2 inch post-tensioned flat plate floor system. The parking structure utilizes a post-tensioned beam and slab system. Concrete work for the entire project was completed from foundations to topping out in less than 15 months.

1.3.2 Pacific Northwest Bell Office Building
Bellevue, Washington

With 476,000 square feet of floor area, the Pacific Northwest Bell Office Building shown in Fig. 7.6 includes computer space, administrative and programmer work space, and miscellaneous support spaces. The building form and massing was a direct result of the three distinct functions to be housed, i.e., computers, offices, and automobiles. Each function dictated unique requirements in terms of circulating. security,. and mode of operation. The challenge was to combine these three diverse components into one cohesive building. This was done by bridging the administrative wings over the computers and parking garage making a dramatic cohesive statement. Separation of the lower support structure from the upper administrative wings was enhanced by contrasting surface textures and matching surface colors on the lower wall elements.

Post-tensioned prestressed concrete was utilized extensively in three distinct areas:

- Precast, perimeter shear walls.
- Long-span perimeter spandrel beams.
- Long-span floor joints.

The precast architectural panels used for the principal building shear walls were prestressed horizontally in the precast plant and post-tensioned vertically in place to form a vertical diaphragm uniquely interconnected to a cast-in-place concrete frame. The two-way prestres-

Fig. 1.5 — Continental Park Plaza, El Segundo, California

sing of the wall panels permitted the use of increased wall shear stresses. Post-tensioned prestressed concrete spandrel beams located around the perimeter of the two office tower wings with clear span lengths to 85 feet, provided structural support for post-tensioned joists of up to 60 feet clear span. Post-tensioned prestressed concrete floor joists located at all office levels of the two "wing" towers as well as the lower levels of the parking garage, provided for clear span lengths to 60feet with minimal column obstructions. The post-tensioned slabs in the parking garage levels yielded maximum crack control and controlled joist deflections for clear spans up to 60 feet.

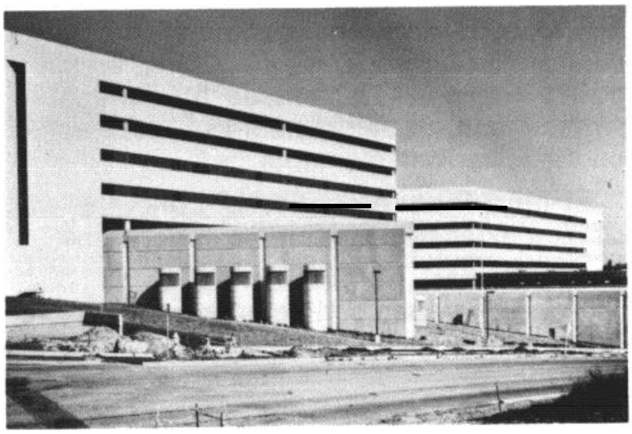


Fig. 1.6 — Pacific Northwest Bell Office Building, Bellevue, Washington.

1.3.3 Dallas Municipal Center
Dallas, Texas

This new municipal building for Dallas *(Fig. 7.7)* has been described as a Bold Symbol of the City's Image and as part of the new architecture in America. The structure is monumental, daring, but with simple and quality detailing; it is a forceful design for a growing and dynamic metroplex.

The most dramatic feature of the building is its 34 degree (2:3) pitch sloping north wall which cantilevers out over the plaza like a giant sculpture. The sloping face is stabilized by post-tensioning, both horizontally and vertically, 14 major bearing walls by a unique method. To simplify installation and control costs, and yet provide a uniform monolithically appearing face, the tendons were installed in tubes with loops at the lower ends and north ends allowing stressing to be done at the top or south ends. This eliminated the need for conventional anchorages at footings on the front face of the building. Vertical post-tensioning, in addition to overcoming theoverturning forces from the cantilevered north face, also allows stage-stressing which permits form removal from the overhanging face as the segments of bearing walls are stressed on the anchor face. A similar technique was used for the horizontal post-tensioning system of the walls.

Fig. 1.7 — Dallas Municipal Center, Dallas, Texas

The 770,000 square foot floor system of the office spaces between the cores was also supported by the core walls. There are approximately 600 post-tensioned "T" beams, 3 ft. deep, spaced on a 4 ft. 8 in. c-c module with diaphragms between the beams. The diaphragms, spaced 4 ft. 8 in. apart, form an exposed coffer ceiling which houses square light fixtures. The post-tensioned T-beams span up to 65 feet 4 inches and were formed with fiber-glass pansallowing up to 10 reuses.

1.3.4 Terracentre
Denver, Colorado

The unique trapezoidal plan of the fifteen story Terracentre (Fig. 7.8) was selected to fit the small lot size. The stepped-back form at the lower levels creates a pyramid effect and fits handsomely into the view plane of the Capitol Building. The exposed concrete interior and exterior surfaces feature a blend of exposed aggregate and board-form concrete. With a total leaseable space of 160,000 square feet, the office space ranges from 5,300 to 15,000 square feet per floor. The post-tensioned flat plate floor system of the Terracentre permitted the use of relatively long cantilevers for dramatic overhangs and column-free spaces.

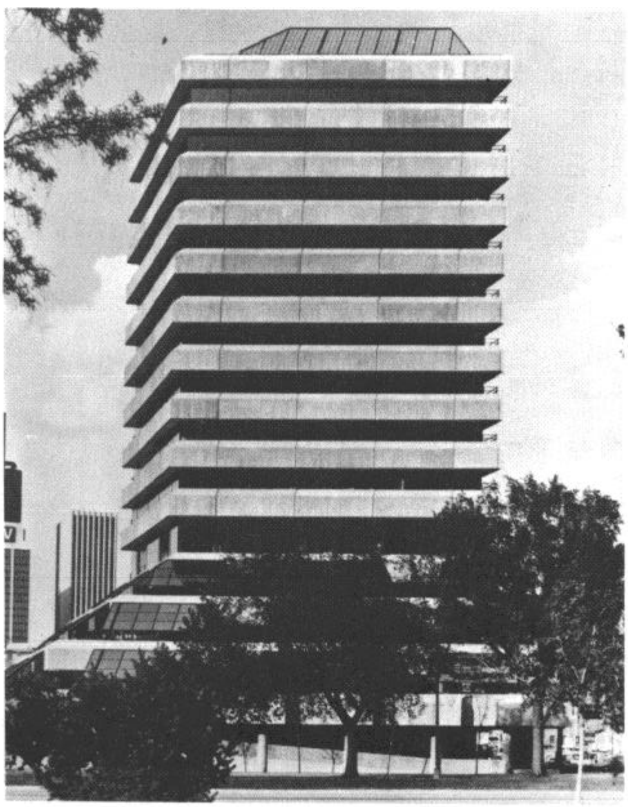


Fig 1 8 — Terracentre, Denver, Colorado

1.4 HIGH-RISE BUILDINGS

1.4.1 Riverplace: The Falls Condominium; The Pinnacle Housing Tower
Minneapolis, Minnesota

Riverplace *(Fig.* 7.9) includes a 24 story rental housing tower, and a 16 story luxury condominium, both constructed over a 4 level parking garage. The total project area incorporates 621,700 square feet.

The unique rental housing tower floor plan was achieved by using a column grid that was irregular in one direction. For ease of placing cables, a simple arrangement of banded cables over the "in-line" columns with uniformly spaced cables perpendicular to the bands was used. Spans of up to 32 feet were attained using an 8 inch post-tensioned flat plate.

To achieve the optimum view of both the downtown skyline and the adjacent riverfront, the tower was skewed at 45 degrees to the parking garage below. To preserve traffic flow patterns in the parking garage, two major columns were terminated at the plaza level above the garage on 108 inch deep post-tensioned transfer beams. The beams were post-tensioned in three stages as construction progressed to control excess camber and balance stresses.

The luxury condominium building form resulted in a very complex series of step-backs which occurred in some cases at midspan of the 28foot bays. Because architectural requirements of limited structural thicknesses precluded the possibility of cantilevering at these step-backs, a series of 1, 2 and 3 story intermediate columns were incorporated into the structure. To support these columns, the basic 8 inch post-tensioned slabs were either thickened to 10 inches or stiffened with shallow post-tensioned beams.

The underlying requirement for choosing a post-tensioned structure for this project was to provide a structure which could economically achieve the complex architectural form. The cost reductions provided by the structural system resulted from t.he considerations listed below:

1) Reduced Building Heights -- By using a minimum structural thickness with a post-tensioned flat plate, the overall building height was reduced resulting in substantial savings in building curtain walls, shear walls, columns, interior partions as well as mechanical and electrical risers.

2. Speed of Construction -- The flat plate post-tensioned slabs built with flying forms allowed construction to continue through the winter with a five day turn around cycle.
3. Structure Costs -- By using flat plate post-tensioned slabs with span to depth ratios of 45 to 48, the amount of concrete used on the project was kept to a minimum and dead load of the structure was reduced. This resulted in savings in colums and foundations.
4. Finishes--The crack-free flat slabs allowed ceiling finishes to be applied directly to the underside of slabs.
5. Fireproofing -- No additional fire proofing was necessary.
6. Repetition of Structural System --The use of a similar system throughout this project maintained continuity of the contractor's work and increased overall efficiency such that completion time and cost of the project were minimized in spite of the architectural complexities.

Fig. 1.9 — Riverplace, Minneapolis, Minnesota

1.4.2 Parklane Plaza Condominium
Houston, Texas

This 400,000 square-foot, 35-story structure (See *Fig.* 7.70) is square in shape with sides of 105 feet and rounded corners. These corners created cantilevers ranging in size from 18feet in one direction to 12 feet in the other. The structural deck was an 8-inch post-tensioned concrete flat plate spanning from a central core shear wall to the outside perimeter columns. The column arrangement, which was dictated by architectural considerations, defined a 42 foot x 30 foot interior bay. This large bay and the double cantilever at the building corners made the flat plate design most critical. The core area contains the stairs, elevators, and major mechanical chases, and is defined by 42'-4" square, 18-inch thick shear walls.

Because the architectural scheme dictated long spans, post-tensioned flat plate construction was the most economical system for this structure. By using a thin post-tensioned slab of lightweight concrete, cost savings were realized in the column and foundation mat systems as well as the floor slab itself. Detailed analysisand design were required to control stresses and deflections caused by the long interior and cantilever spans.

The two-way cantilevers created critical stresses at the building corners. These stresses were controlled by locating a conventionally reinforced diagonal slab beam spanning to columns on either side of the corner, and judiciously locating post-tensioning bands along the building perimeter. This design effectively reduced the cantilever to eight feet in the building diagonal. Diagonal top and bottom bar was placed over the slab beam to carry the cantilever load. The banded, post-tensioning tendons located at the slab edge and continuing to the corner, placed the slab in compression, resulting in a crack-free cantilever. Because of the unusual design details at the corner cantilevers, accurate perimeter insert placement for the precast window wall system and the many mechanical sleeves was extremely important. The contractor scheduled one floor per week with no exceptions for weather, holidays, or mechanical failures. To assure this performance, a full-scale mock-up was fabricated for a typical floor. All tendons, rebar, sleeves, and inserts were placed, and conflicts resolved prior to start of slab construction. As a result 35 floors were constructed without missing a cycle, with virtually no overtime, and without a single misplaced embedment.

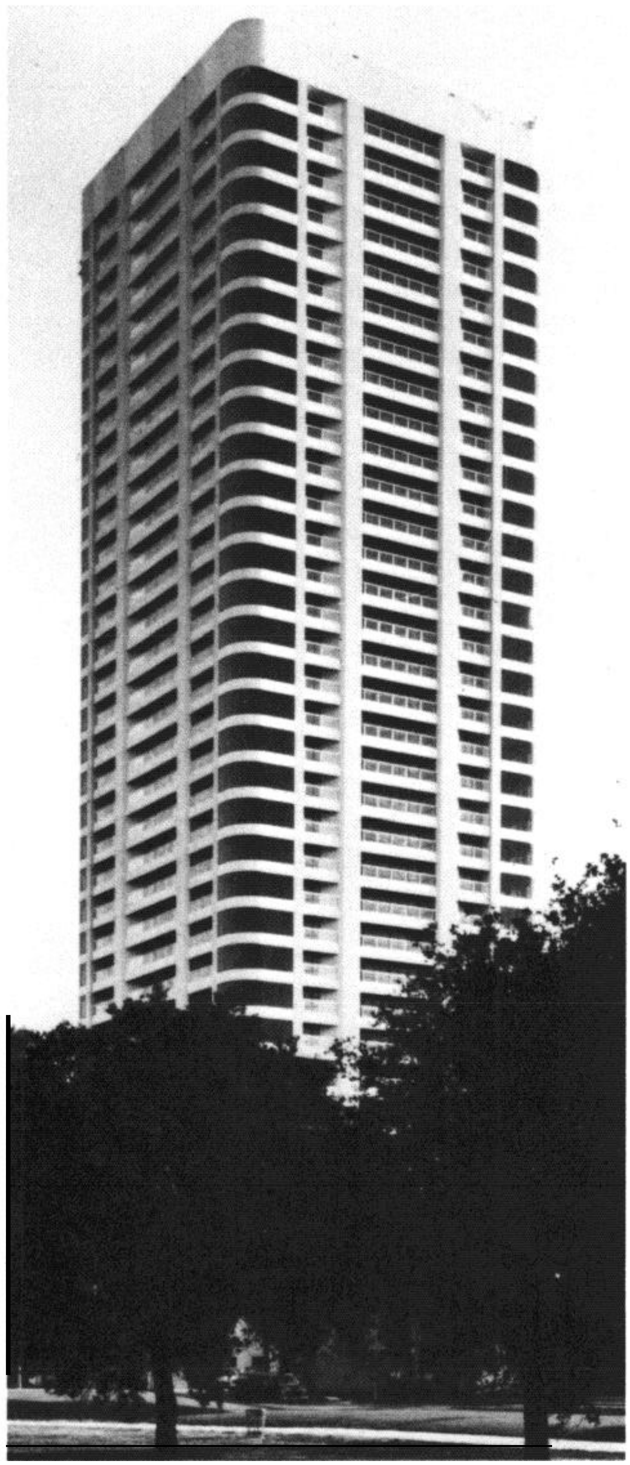


Fig. 1.10 — Parklane Plaza Condominium, Houston, Texas.

1.4.3 Western Canadian Place
Calgary, Alberta, Canada

Western Canadian Place, figs. 1.17 and 7.72, incorporates two towers, one of a 41 foot height and the other a 31 floor height. The office towers are interconnected from the fourth floor down, and enclose a three-story atrium and commercial base. There are two levels of underground parking. The total total floor area is 1,420,000 square feet. The structural framing system incorporates a wide, flat beam and slab system with longest beam spans of about 40 feet. The slab thickness varied between 7 and 8-1/4 inches, and the beam depths ranged from about 18 to 21 inches. The open, rounded corners were a special design requirement that was resolved through the use of post-tensioning.

After completion of the project, a change in tenant requirements made it desirable to provide stair openings to interconnect floors at approximately 10 separate floor levels. Stair openings of about 14 ft. by 14 ft. were easily cut into post-tensioned slabs without the use of extensive shoring, and in most cases without requiring any additional support beams. Interior designers were requested only to avoid beam locations and end spans when locating stair openings, and that did not unduly restrict their planning. Neither a cost nor a design restraint was created in locating stair openings, due to the fact that the floor system was post-tensioned.

In the two towers, there are 64 office and mechanical room floors, of which the first was poured in November of 1981 and the last was poured during November of 1982. Use of post-tensioning was a strong factor in ensuring fast construction, in spite of working through months of cold winter conditions. In the total project, there are 1,465,000 lbs. of 0.6 diameter post-tensioning strand.

Fig. 1.11 — Winter Construction of Western Canadian Place, Calgary, Alberta, Canada.

Fig. 1.12 -- Western Canadian Place. Calgary, Alberta, Canada.

1.4.4 Energy Centre
New Orleans, Louisiana

The Energy Centre shown in fig. 1.13 is a 39-story highrise tower located in downtown New Orleans. It consists of 32 office levels constructed over a 7 level parking garage. The structural system is a post-tensioned cast-in-place concrete frame with interiorcore shear walls. The total area of the project is approximately 1.1 million square feet. There are 816,800 square feet of construction in the tower with 25,525 square feet on each typical floor. The parking garage has space for approximately 630 cars with a mechanical mezzanine separating the tower and garage.

Special design features incorporated in the Energy Centre were as follows:

1) The design of the building required a very open layout with 42' clear spans, 9'0" ceiling heights, and a 2'3" ceiling plenum for mechanical HVAC, ceiling lights, and sprinklers, while maintaining a 12'6" floor to floor height. A post-tensioned lightweight concrete slab and beam structure was chosen as the most economical structural system to accomplish these goals.
2) The floor framing spans 42 feet using 14 inch deep post-tensioned, tapered haunched beams with a 5 inch post-tensioned slab spanning 20 feet between beams.
3) The building has a unique parallelogram-shape with twelve column-free, bay-window lease spaces on each floor requiring the slab to cantilever 10 feet. The slabs were to 8" in these areas to support the floor and the granite thickened exterior.
4) Control of deflections in the cantilevers was carefully investigated using two-way load balancing to avoid potential problems due to live load and long term effects.
5) Post-tensioned shrinkage and temperature reinforcement was used in lieu of conventional rebar reinforcement.
6) Building setbacks at the top of structure required 42' post-tensioned transfer girders supporting 6 levels.
7) The owner requested several floors to be designed for 125 psf live load to allow for leasing flexibility. This live load greatly exceeded the local code requirements, but did not revise the geometry of the floor framing. This factor was of primary importance for the constructabililty of the project using flying forms.
8) Due to poor soil conditions, the building is founded on 18 inch octogonal and 14 inch square prestressed piles driven 203 ft. and 170 ft., respectively. Design loads were determined by in-place load testing.

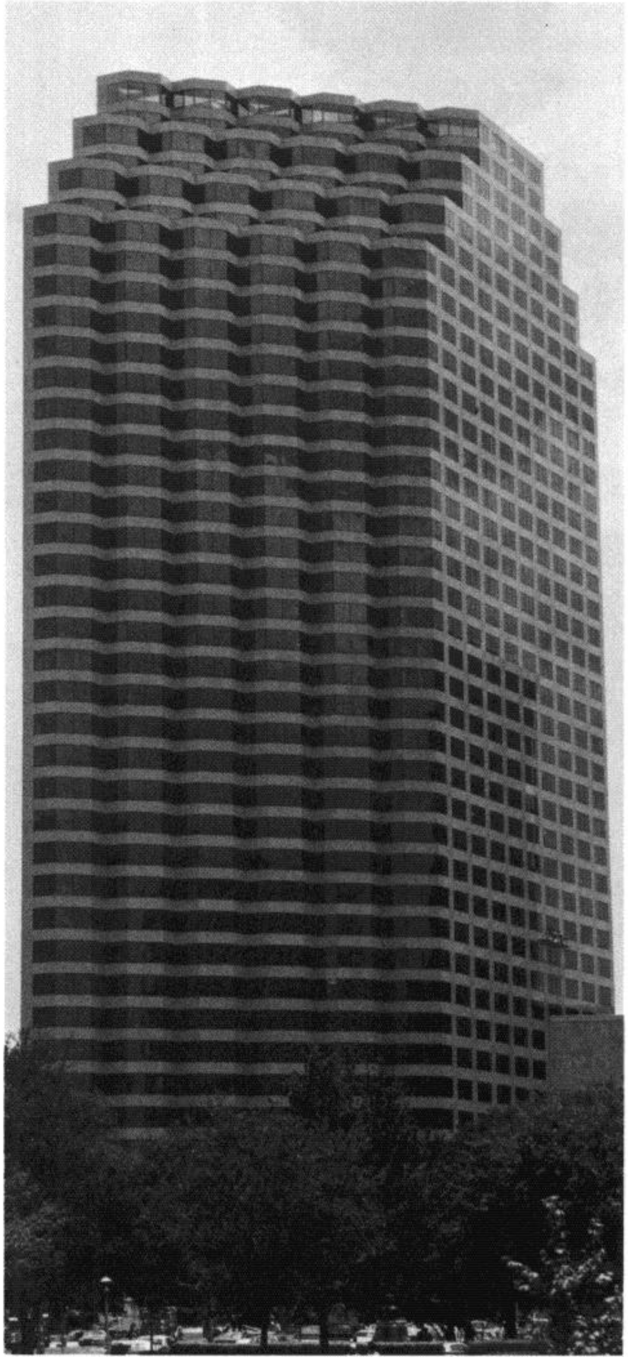


Fig. 1.13 — The Energy Centre, New Orleans, Louisiana.

1.4.5 Huron Plaza Office and Apartment Building Chicago, Illinois

The Huron Plaza Office and Apartment Building (fig. 7.74) is a 56-story highrise with an adjacent IO-story garage located just north of downtown Chicago. The structural system is a post-tensioned cast-in-place concrete frame which also forms the exterior of the building with exposed architectural concrete columns and spandrel beams. The typical floor is 70' x 130' yielding 9,100 sq. ft. for a total of 510,000 sq. ft. for the tower. The garage is 80' x 180' or 14,400 sq. ft. for a total of 144,000 sq. ft.

The design program called for 8 office floors with a "column-free" area around a compact center core below 48 apartment floors. Due to the narrow tower, it was necessary to provide three lines of coupled shear walls in order to maintain acceptable stiffness. These, however, could not be permitted within the 8 commercial floors at the lower part of the tower, where only the shaft could be accommodated. This was resolved by the use of the exterior columns as "outriggers" resulting in a moment diagram for the shear wall system, center core as shown in Fig. 7.75. These moments were readily handled by the shear wall system. The slabs were designed as a combination of one-way and banded flat plate post-tensioned slabs. The architectural layout did not lend itself to a straight and disciplined column layout. This, however, was overcome by deflecting the banded tendon runs in the transverse direction laterally to correspond to the column layout. The longitudinal tendons were generally uniformly spaced, except where deflected laterally around slab openings and mechanical obstructions.

Whereas the selection of post-tensioning was natural for the lo-story parking garage, the decision to use a post-tensioned design for the tower was based upon the need for a "column-free" floor plan for the eight commercial floors at the lower part of the tower, as well as the desire to place concrete shear walls on the dividing lines between apartments, which resulted in typical spans of 27'6". With weight at a premium, only a post-tensioned design could achieve these spans with a 7" thick slab.

In order to allow casting of two floors each week, three days per floor and working on Saturdays, a slight upgrading of the concretedesign strength was necessary, from f_c' = 4000 psi to 4600 psi. With the improved concrete strength, and the relatively low shear stresses for this design, it was possible to show that the post-tensioning greatly reduced the need for shoring and reshoring. This in turn allowed finishing work to follow closely the casting of the concrete frame.

A subtle advantage of post-tensioned construction for this project was the resulting flatness of the slabs, which made it easier for the finishing trades to install dry-walls, shower bases, bath tubs, and tiles, kitchen cabinets, and align trims and moldings. The crack-free exterior spandrel system is also a result of the compression introduced by the prestressing tendons.

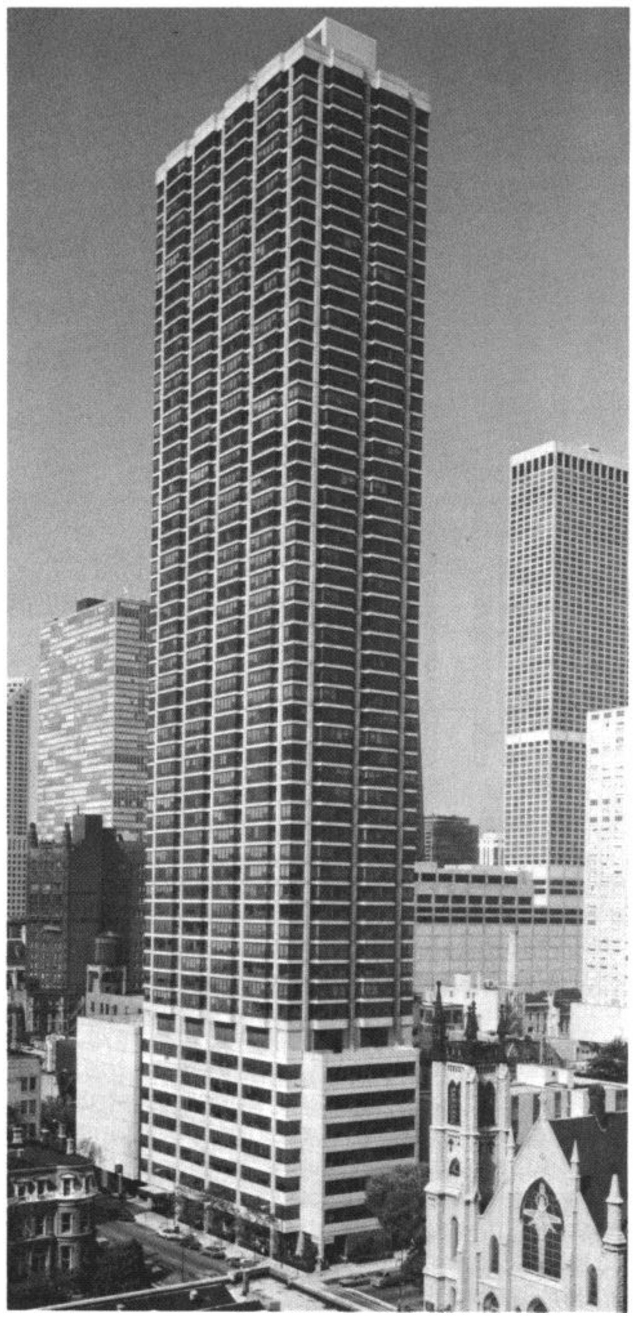


Fig. 1.14 — Huron Plaza Office and Apartment Building, Chicago, Illinois

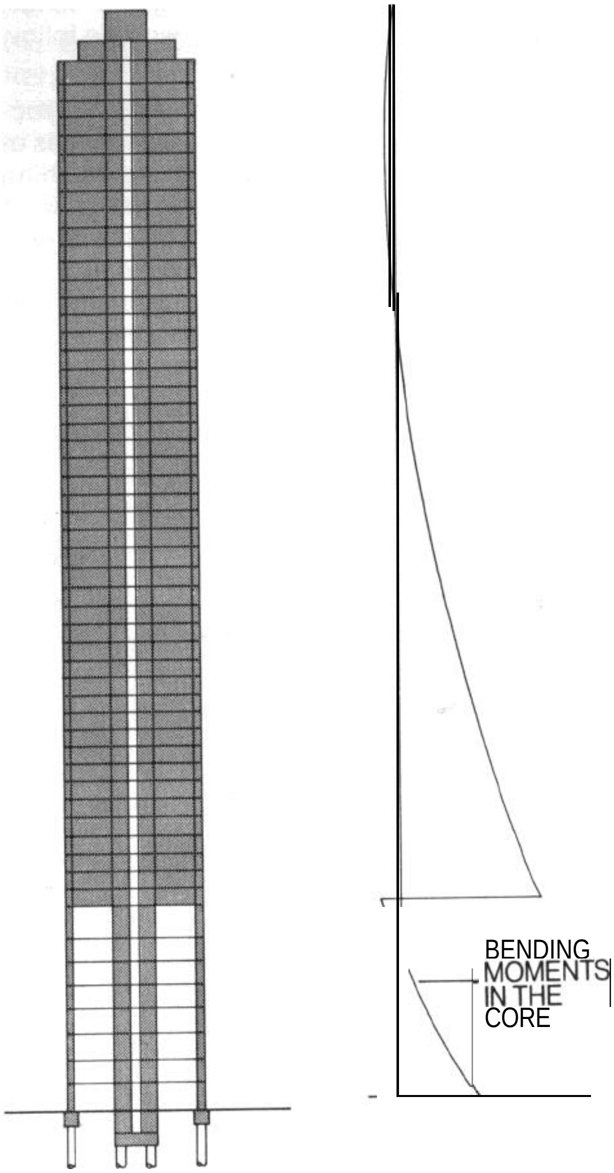


Fig. 1.15 — Moment Diagram for Shear Walls and Center Core, Huron Plaza Office and Apartment Building, Chicago, Illinois

1.5 PARKING STRUCTURES

Post-tensioned parking structures have been built economically with nearly all of the commonly used cast-in-place concrete structural framing systems. The size of post-tensioned parking structures ranges from small single story structures to the 5000 car capacity of the parking structure for the Superdome in New Orleans, described in Section 1.51. The examples presented in this section illustrate only a few of the many design options that have been used for post-tensioned parking structures.

1.5.1 Parking Structures for the New Orleans Superdome

The initial design decision relative to the parking structures for the New Orleans Superdome was the selection of a concrete framing system due to fire rating requirements. Various bay sizes and framing systems were then studied and detailed cost estimates were developed for structures with a rectangular bay size. The lowest estimate was for a 54 x 18-ft. bay post-tensioned structures using lightweight concrete (115 lb. per cu. ft.).

The basic framing system is an 18 ft. span with 5 in. slab spanning between 16 x 28 in. beams spanning 54ft. Expansion joints are provided on lines running east to west which divide the structures into units of eight bays at 18 ft. with the end units four bays at 18 ft. In the north to south directions the expansion joints divide the structures into segments of up to three bays at 54 ft. Joints through the slabs are terminated at the expansion joints with dual beams and columns. The joints through the beams are accommodated with beam brackets'on one side of the column and neoprene bearing pads to allow for movement due to temperature, shrinkage and creep.

The completed parking structures shown in **Fig.** 7.76 measure 378 x 864 ft. on the west side of the stadium, and 324 x 864 ft. on the east side. The three level parking structures accommodate a total of 5,000 automobiles.

Fig. 1 16 — Parking Structures for New Orleans Superdome

Fig. 1.17 -Transportation Center Parking Structure. Dayton, Ohio.

1.5.2 Transportation Center Parking Structure Dayton, Ohio

The 1,500 car parking structure in Dayton, Ohio shown in Fig. 7.77 incorporates 504,000 square feet in five parking levels with provisions for present and future transportation related stores and terminals at ground level. In addition, the top level of the east wing of the structure was designed as a heliport.

The supported parking decks consist of 6 in. post-tensioned slabs spanning 28 ft. between post-tensioned beams which are 16 in. wide by 36 in. deep. The beams span 61 ft. between reinforced concrete columns. Prime design requirements were that the parking decks be durable and waterproof to reduce maintenance and increase the life of the structure. Cast-in-place post-tensioned prestressed concrete was selected to fulfill these requirements. Another design requirement was that maximum usable floor space be provided. Again, cast-in-place post-tensioned prestressed concrete provided a solution because of the long spans attainable with minimum depth members. Transfer beams and 3-story high trusses were designed at several locations to eliminate columns below the first supported level to allow for vehicle traffic lanes and a four lane street. Post-tensioning of the transfer beams and truss bottom chords minimized deflections and permitted member proportions that conformed to architectural requirements.

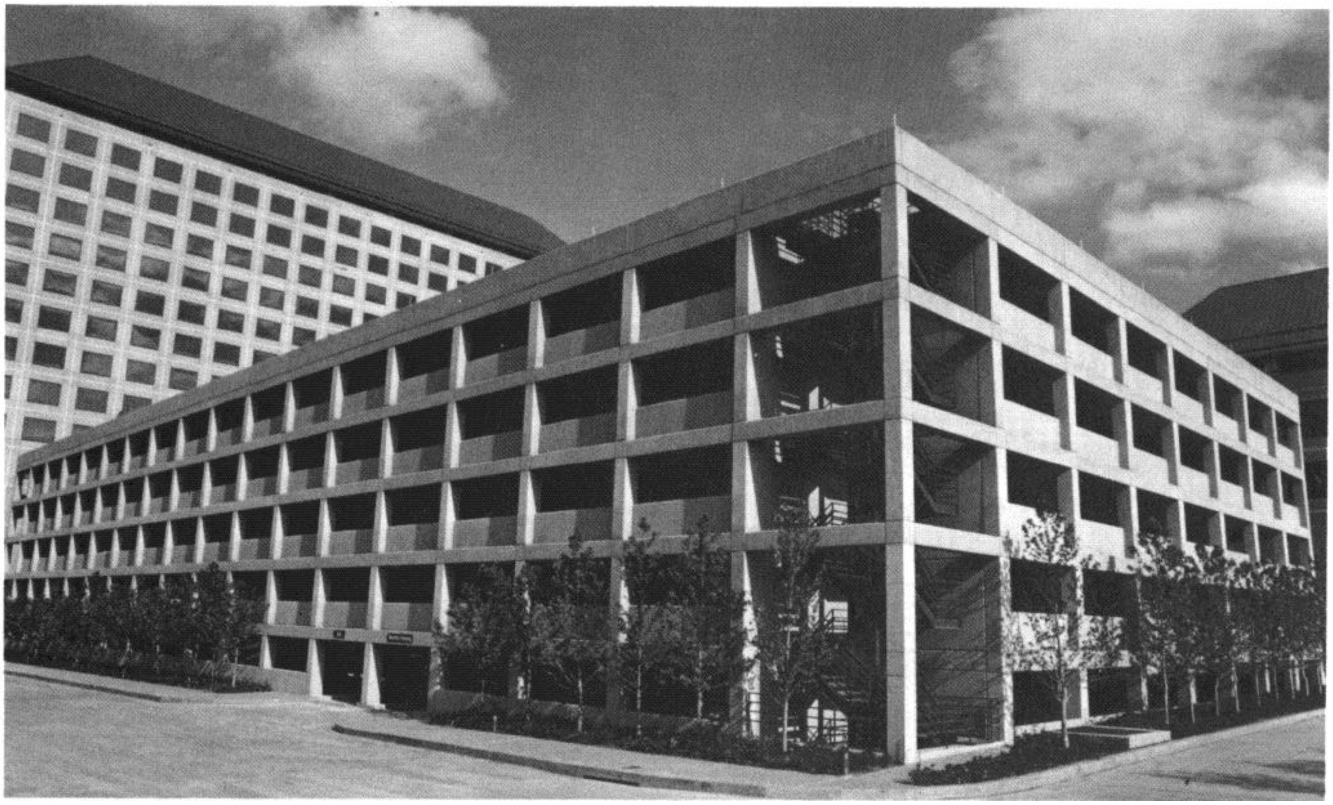


Fig. 1.18 -Williams Square West Garage in Las Colinas, Irving. Texas

1.5.3 Williams Square West Garage in Las Colinas, Irving, Texas

The Williams Square West Garage *(Fig. 7.78)* is an 8 level parking garage with 3 levels at or below grade and 5 levels above grade. The entire structure is cast-in-place concrete. The floor system consists of 4-1/2" post-tensioned slab spanning between 20" x 32" post-tensioned beams at 16'-0" on centers. The beams span 64'-0". The perimeter spandrel beams are also post-tensioned.

The significant feature of the project is the perimeter spandrel beam, column, and rail assembly. As shown in *Fig.* 7.79, the concrete in these exposed perimeter elements uses crushed granite aggregate while the concrete for the interior beams, slabs and columns uses a more economical crushed limestone aggregate. The concrete placement at the perimeter had to be performed in a sequence which would permit the maximum use of the more economical limestone aggregate concrete and insure that the granite aggregate made up the exposed surface. In order to maximize the color of the granite, these perimeter surfaces were sandblasted. To create a compatible compressive stress in the slab and spandrel beam, post-tensioing was installed in the spandrel beam.

The post-tensioned perimeter framing achieved the following desired results for the owner:

1. Allowed the use of a continuous smooth flowing monolithic cast-in-place concrete exterior as opposed to a segmented precast veneer. This consideration was critical to the Architect's aesthetic guidelines and was not achievable with comparative systems as economically as it was done with the post-tensioned solution.
2. Eliminated unsightly flexural, thermal, and shrinkage cracks in the exposed granite aggregate finish.
3. Eliminated the need for an additional construction trade on the site as the contractor handled all cast-in-place concrete construction.
4. Eliminated the expense of sealing and concealing metal connections with a precast spandrel or granite veneer solution.

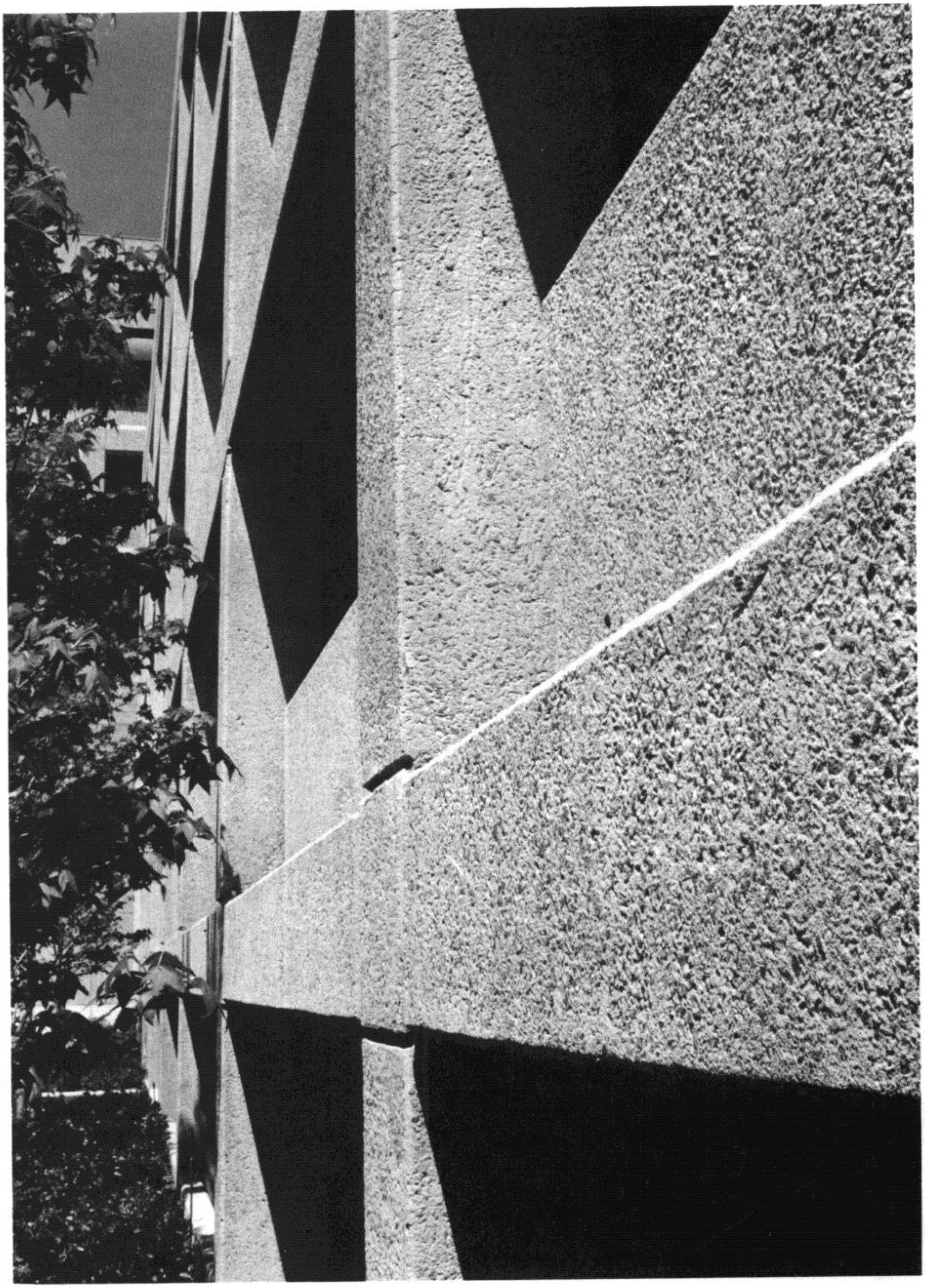


Fig. 1.19 — Crushed granite Aggregate Concrete Surface Treatment, Williams Square West Garage, Irving, Texas.

1.5.4 Port Columbus International Airport Parking Structure Columbus, Ohio

The parking and enplaning structure for the Port Columbus Airport Terminal Building, Figs. 7.20 and 1.27, is on prominent public display, Therefore, an architecturally pleasing appearance for both vertical and horizontal surfaces, along with the highest degree of watertightness were of major importance. To achieve these goals and to accommodate the intricate traffic pattern of the two on-grade drives below, the design requirements for this structure were established as follows:

1. Column spacing: 60' x 60' bays were selected to provide clear, open driving lanes for the Enplaning, Deplaning, and Parking Drives under the parking levels.
2. Floor construction: Parking levels; 4" top slab spanning between 30" deep, 16" wide tapered transverse ribs spaced at 6'8" on center.
3. Expansion joints: Three expansion joints at roughly equal spacing were located along the 810' length of the structure. The appearance of double columns or column brackets would have been objectionable; therefore, the expansion joints were formed by beam brackets at quarter points of the span.
4. Lateral stability: The stairs, elevators and helices separated from the Parking Structure by expansion joints to allow for a high degree of freedom for thermal movements. Therefore, lateral stability was provided by moment frames, involving all available columns in both directions.
5. Forming/Shoring system: To achieve a high quality finish, custom-made 30" deep x 5'4" wide x 30' (±) long fiberglass pans were used to form the slabs and ribs.

Fig 1.20 — Port Columbus International Airport Parking Structure, Columbus, Ohio

Fig 1.21 — View Underneath Port Columbus International Airport Parking Structure, Columbus, Ohio

Several framing systems were given preliminary consideration for the parking structure: exposed structural steel, precast concrete, conventionally reinforced and post-tensioned cast-in-place concrete. The following requirements determined the final selection:

- Fireproof construction
- Large bay sizes
- Highway loading on bridge
- High degree of watertightness
- Minimal maintenance requirements
- Aesthetically pleasing appearance

The structural system that best satisfied all these requirements was considered to be the post-tensioned cast-in-place concrete frame.

1.5.5 Spokane International Airport Parking Structure, Spokane, Washington

The cantilever spiral ramps are the main aesthetic design feature of the Spokane International Airport Parking Structure (fig, 1.22). The clean, smooth look is achieved by the use of the post-tensioned cantilever slab. To facilitate construction and to improve appearance, the tendons were sleeved through the wall so the core could be built without interruption. The cantilever moment is resisted by the core wall.

An 8 ft. clear ceiling height was maintained by utilizing post-tensioned joists with a total depth of 24-1/2 inches, The efficiency of the post-tensioned joists allowed a spacing of 56 inches. This created a more open feeling in the garage. The joists at the column lines were widened and reinforced to carry the seismic loads.

To achieve a higher quality slab, post-tensioning was used even though rebarwould have sufficed. Post-tensioning was provided at an added cost of only 17 cents per sq. ft., which compares favorably to other methods of increasing the life of the slab.

The simplicity of the post-tensioned forming system and its inherent economy in long span construction was the key to a very favorable cost per car. This project was a design/build competition which won over local precast and other structural systems.

Fig 1 22 — Spokane International Airport Parking Structure, Spokane, Washington

1.6 POST-TENSIONED ARCHITECTURAL PRECAST CONCRETE

1.6.1 Citizen's Bank Center, Richardson, Texas

The Citizen's Bank Center shown in *Fig. 7.23,* utilizes a facade of lightweight architectural precast concrete beam and column elements. The beams support the floor loads and, in conjunction with the precast corner columns, provide 60 percent of the lateral stability of the structure. The building, which is 110-ft. square in plan, also utilizes a slip-formed reinforced concrete core. To control the deflection of the 2-ft. by 5-ft. beams spanning 105-ft., the beams were both pretensioned and post-tensioned at the plant. The post-tensioning was performed after 28 days of plant curing, stressing the beams to the desired and calculated camber. The camber in the plant forms and the amount of post-tensioning and pretensioning was very carefully calculated to allow for deflections due to dead load, live load, creep and shrinkage. The vertical post-tensioning of the precast beams and columns at the site combined the units into a rigid frame for lateral stability, and restricted rotation of the beam ends which helped considerably with control of deflections. *Fig. 7.24* illustrates the assembly concept for the building.

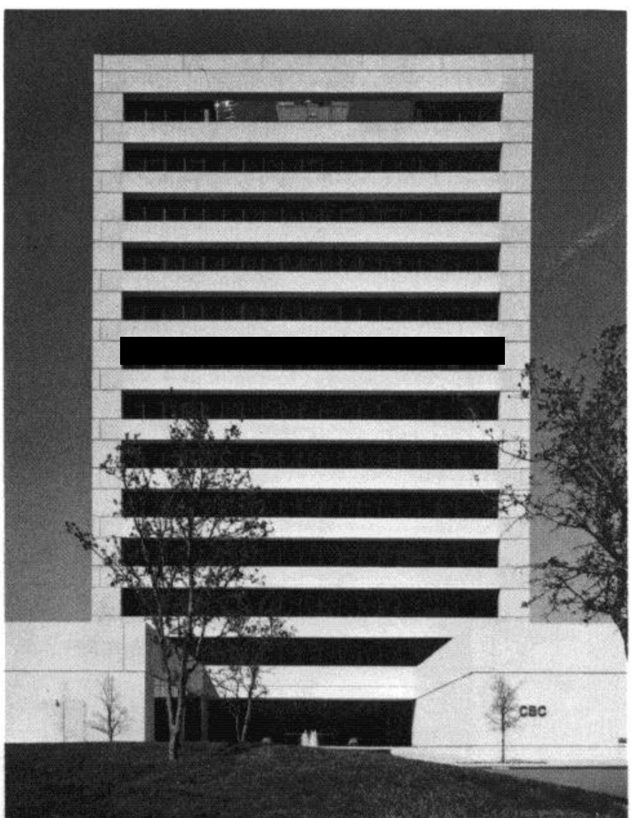


Fig 1.23 Citizen's Bank Center, Richardson Texas

Fig. 1.24 — Assembly of Exterior Framing. Citizen's Bank Center, Richardson, Texas.

1.6.2 Cecil Community College, Cecil County, Maryland

Cecil Community College was initially a "Campus in Miniature" with every service that a college should provide incorporated into the initial building. Phase 1 of the College is the first building of an ultimate campus complex of eleven buildings. In order to create large areas of open space without interference from the framing system, post-tensioned flat plate lift-slab floor and ceiling construction was utilized. Metal plates were anchored into the perimeter of the post-tensioned floor and ceiling slabs to which the architectural precast concrete panels were secured. Following lifting of theceiling and floor slabs, all panels were installed within a period of approximately two weeks. The flat panels, the largest of which measured 8-ft. by 29-ft., were post-tensioned in the precaster's yard to provide optimum structural integrity for panel lifting. Panels were cast on rough-sawn, random width oak using buff colored concrete. Both color and texture were uniformly maintained throughout all panels. The completed building is illustrated in *Fig. 1.25.*

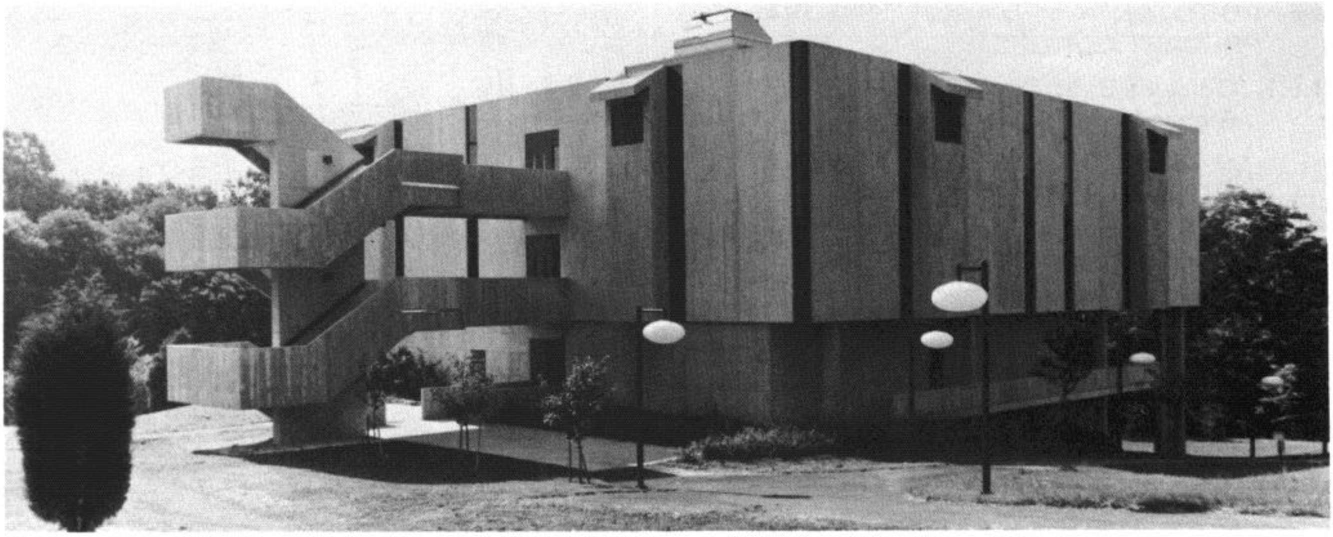


Fig. 1.25 — Cecil Community College, Cecil County, Maryland.

1.6.3 International Aviation Square, Montreal, Canada

The new headquarters for the International Civil Aviation organization (ICAO) shown in ***Fig. 7.26*** is 115-ft by 115-ft in plan with 27 stories plus a mechanical penthouse. The interior of the tower is completely column free, the entire structure is supported by the service core and eight exterior columns which also contain mechanical shafts. Between these columns span one story high architectural precast concrete Vierendeel trusses which support two floors each. A total of 44 Vierendeels, each 75-ft. 7-in. long by 16-ft., 8-in. frame the building starting at the sixth floor. Each Vierendeel truss was subdivided into twenty precast concrete elements: six I-shaped pieces as the vertical members of the truss and fourteen horizontal filler pieces. As illustrated in ***Fig. 7.27,*** segments were supported on scaffolding and post-tensioned together with four tendons, two per upper and lower chord of the truss, each tendon providing a design force fo 600 kips. The building is designed for earthquake Zone 2 with 75 percent of the loads carried by the interior core and 25 percent by the columns and Vierendeels.

Fig. 1 26 — International Civil Aviation Headquarters Building, Montreal, Canada

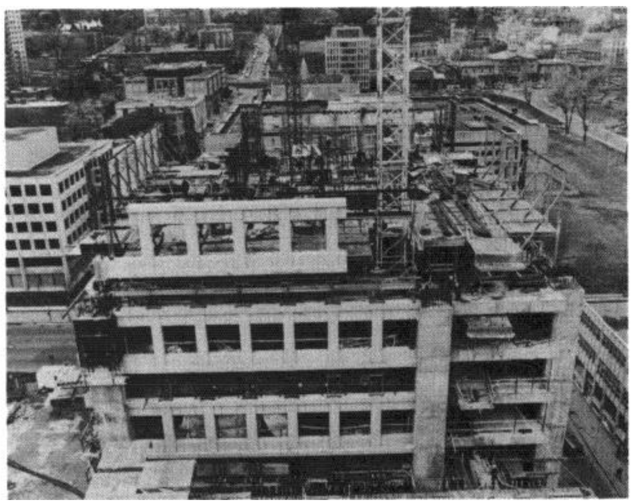


Fig. 1.27 — Erection of Architectural Precast Concrete

Fig. 1.28 — Calgary Exhibition and Stampede Grandstand, Alberta. Canada

1.7 GRANDSTANDS

1.7.1 Grandstand for Calgary Exhibition and Stampede, Alberta Canada

Rated as one of Canada's principal exhibitions and attracting nearly3 million people each year, the Calgary Exhibition and Stampede is a dynamic forum for the community. Design criteria for this new grandstand which accommodates 17,500 persons seated and 12,000 persons standing, included elimination of supporting columns which might interrupt sight lines to thestageand infield rodeo events, and an 11 month construction period. These design criteria were met with the results shown in *Fig.* 1.28 by use of a variety of precast elements connected into the final structural geometry by use of 370,000 linear ft. of 1/2 inch and 0.6 inch diameter post-tensioning strand. Completion of this attractive facility in the limited time period, which included the severe winter season experienced in Calgary, reflects the advantages of integrated precast-post-tensioned construction techniques. *Fig.* 7.29 illustrates the construction of the grandstand.

1.7.2 Hoolulu Park Grandstand, Hilo, Hawaii

The 2,200 seat grandstand for the Hoolulu Park shown in Fig. 7.30 is roofed with 14 precast barrel shells and 2 half-barrels at the ends. The shells tapered, front to back, give the roofline an esthetically pleasant curvature.

The roof shells cantilever 50 ft., from their supports giving a graceful silhouette to the structure. The shells were cast on the ground, lifted into place, then post-tensioned by tendons within the shell surface. Cast-in-place compression struts join the shells together. Because of the close proximity to active volcanoes, the structure had to be designed to withstand potential dynamic forces. A model study and computer analysis substantiated the post-tensioned prestressed concrete design chosen. The rear supporting columns are post-tensioned vertically.

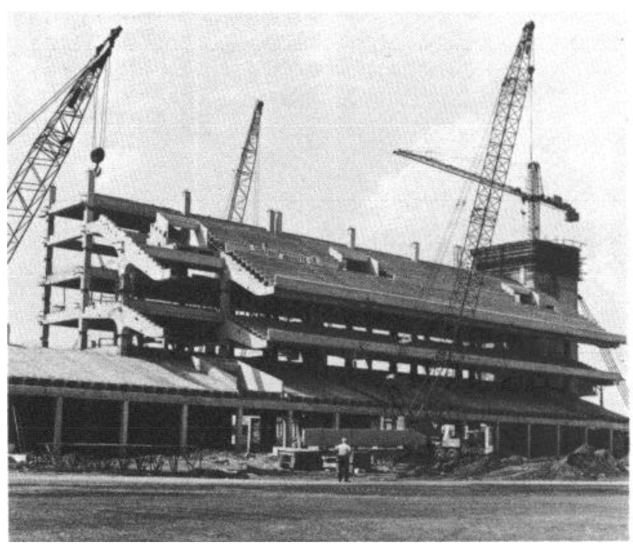


Fig. 1.29 — Construction View, Calgary Exhibition and Stampede Grandstand.

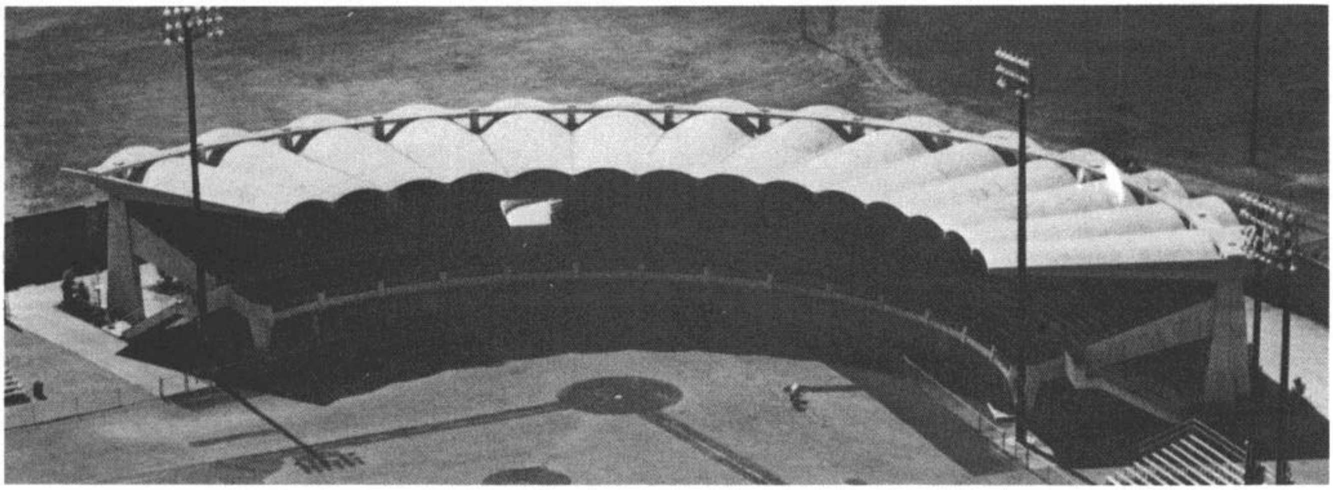


Fig. 1.30 — Hoolulu Park Grandstand, Hilo, Hawaii.

1.8 TENSION MEMBERS

1.8.1 Post-Tensioned Tension Rings

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. 7.37 shows the construction of the 360-ft. diameter domed fieldhouse 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. 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 1,130-ft. circumference. Eight tensioning locations were provided by notches formed on the outside of the ring. At each tensioning location, 7 tendons pass continuously through the ring, and 7 tendons are terminated for tensioning.

1.8.2 Post-Tensioned Tie-Beams

The Central University Libraryforthe University of California at La Jolla shown in ***Fig. 1.32,*** utilizes a structural frame of cast-in-place reinforced concrete with 24 post-tensioned concrete ties incorporated in the girders at the fourth, fifth and sixth floor levels. Each tie consists of three 44 wire grouted tendons. The average tendon length is 200 ft.

Post-tensioned tie-beams have the advantage of minimizing or eliminating movements in supported structural systems. This advantage is particularly significant in applications such as shells and other similar long span elements.

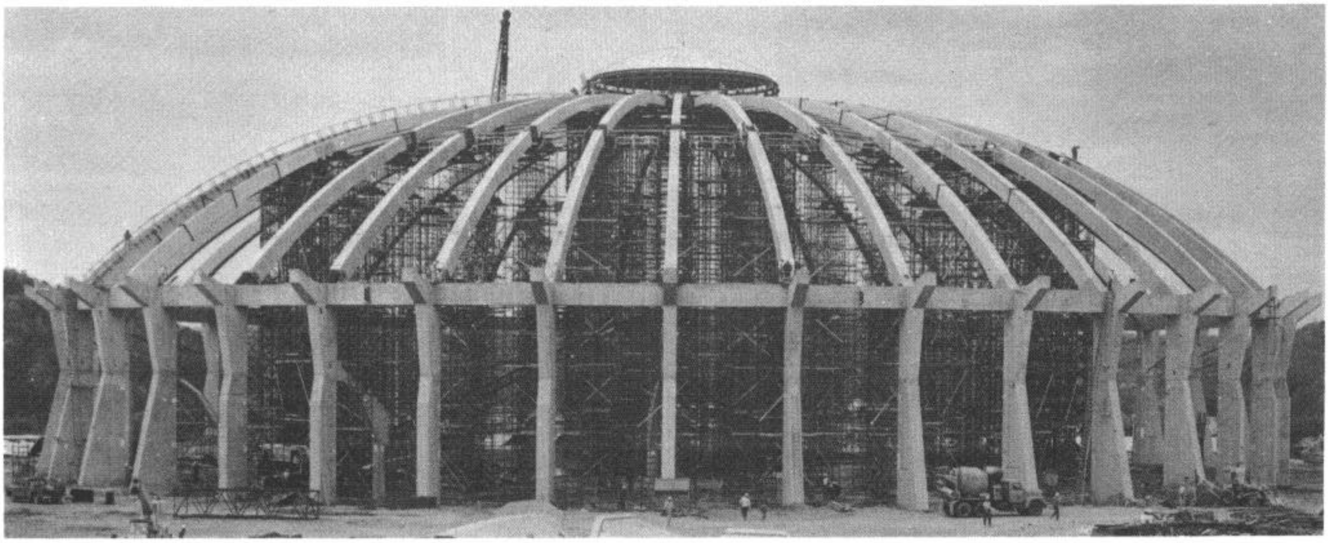


Fig. 1.31 -Construction of Domed Field House, University of West Virginia, Morgantown, West Virginia.

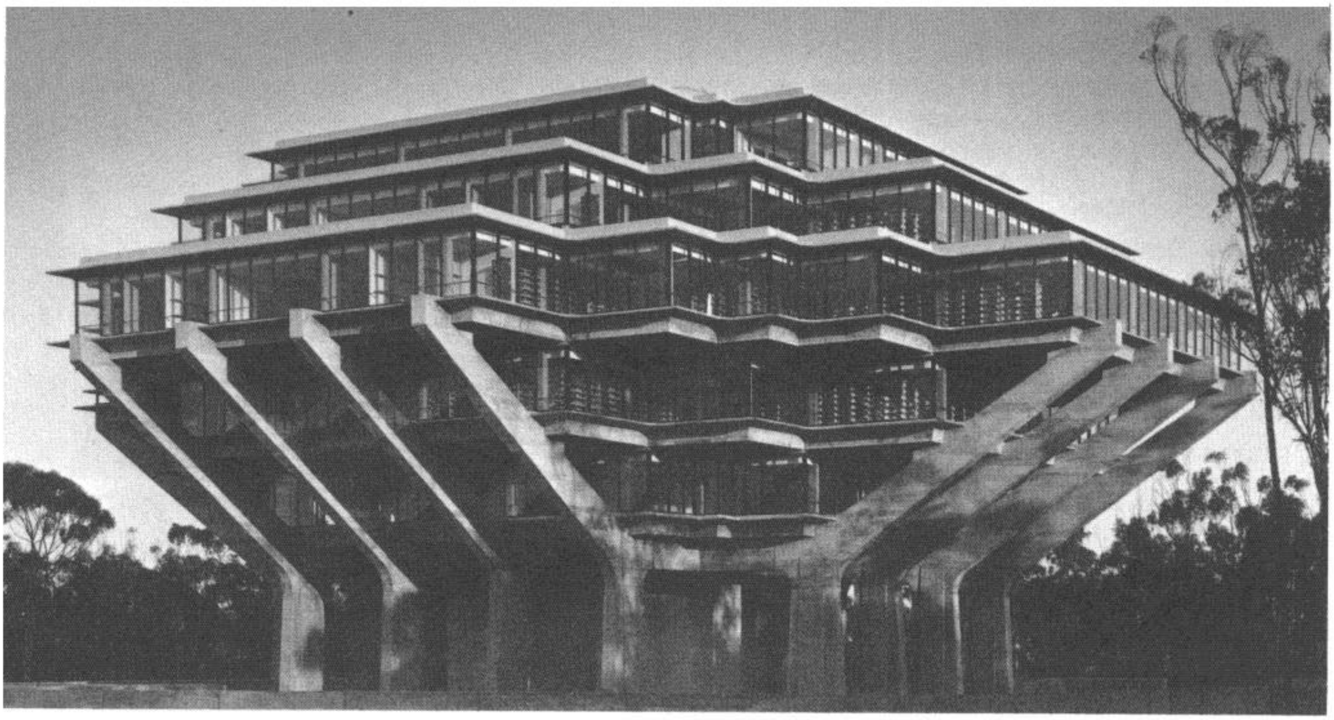


Fig. 1.32 -Central University Library, University of California, LaJolla, California

1.8.3 fainter Gate Anchorages

Post-tensioned ties have become the standard means of resisting the tremendous thrust from water forces on tainter gates. The tainter gate anchorages for the Wanapum Dam on the Columbia River are shown in Fig. 7.33. The use of post-tensioning in the anchor blocks reduces the mass and makes possible a much simpler design of the entire gate system. A schematic view showing the incorporation of post-tensioning in the tainter gate anchorage is shown in *Fig. 1.34.*

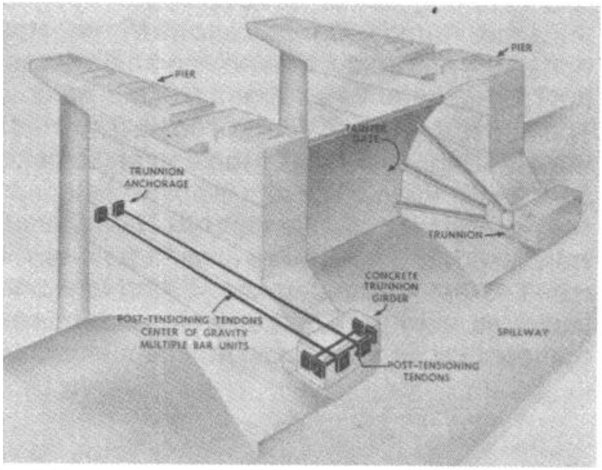


Fig. 1.34 — Schematic View of Post-Tensioning in Tainter Gate Anchorages.

Fig. 1.33 — Post-Tensioned Tainter Gate Anchorages, Wanapum Dam, Columbia River

1.9 PRESTRESSED ROCK AND SOIL ANCHORS

The first application of a prestressed anchor dates back to 1935 when the late Andre Coyne, a French engineer, used prestressed anchors to stabilize the Cheurfas Dam in Algeria. Until recently, the 1100 ton anchors used in the Cheurfas Dam were the largest ever installed in a structure. This project generated a number of new systems and applications in Europe. However, the widespread use of prestressed rock and soil anchors is a relatively recent development in North America.

A great variety of applications of prestressed rock and soil anchors has developed, including:

a. Tie-backs for foundation excavations

b. Retaining walls and revetments

c. Stabilization of rock slopes and underground excavations

d. Dam stabilization

e. Anchorage of fixed points and anchorage against upward water pressure.

The use of prestsressed tie-backs for foundation excavations is illustrated in Figs. 7.35 and 7.36. Recommended design and construction procedures for prestressed rock and soil anchors are presented in Chapter 4.

Fig. 1.36 — Drilled Concrete Shafts supported by Three Levels of Tie-Backs and Walers.

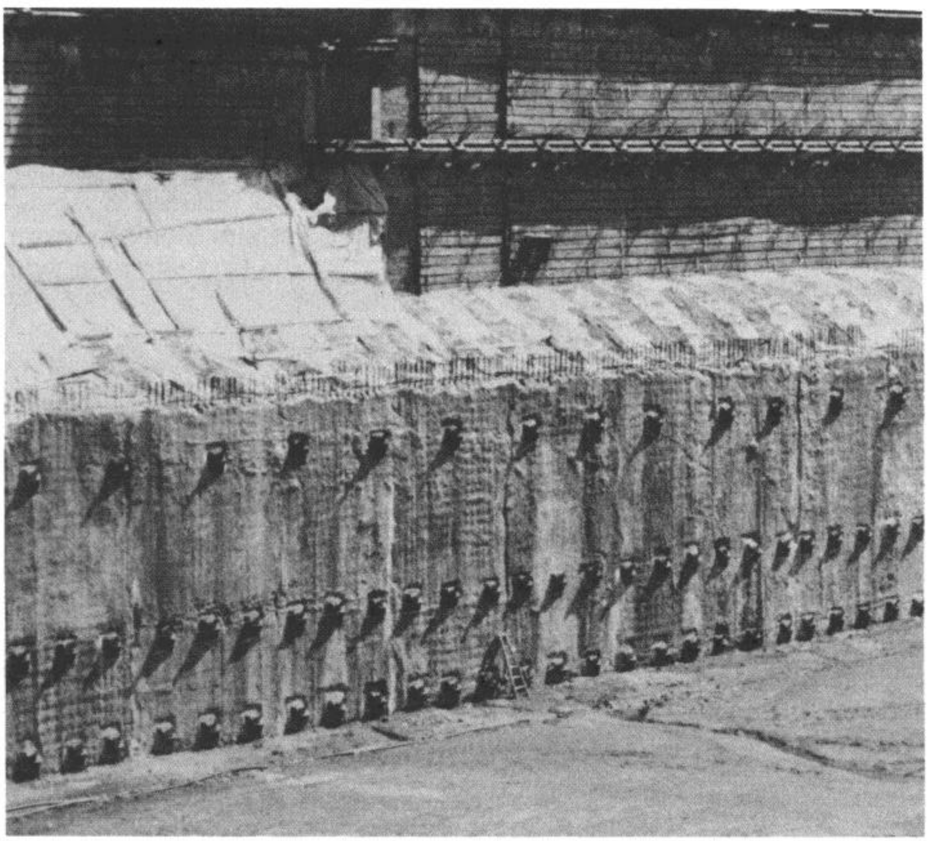


Fig. 1.35 — Two Levels of Excavation Stabilized by Post-Tensioned Tie-Backs. Upper Wall of Soldier Piles and Wooden Lagging Braced by Two Layers of Tie-Backs with Steel Walers. Lower Level of Reinforced Slurry Wall Construction with Multiple Levels of Tie-Backs.

1.10 FOUNDATIONS, PAVEMENTS AND SLABS-ON-GROUND

1.10.1 Mat Foundations

Due to unfavorable soil conditions at a building site, post-tensioned mat foundations are occasionally used as an alternate to more expensive pile foundations. 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. 1.37, the tendon layout required for this purpose can be visualized as an upside down flat plate. The post-tensioning tendons utilized for the mat in Fig. 7.37 were 1/2-in.] diameter, 270k, unbonded strand tendons.

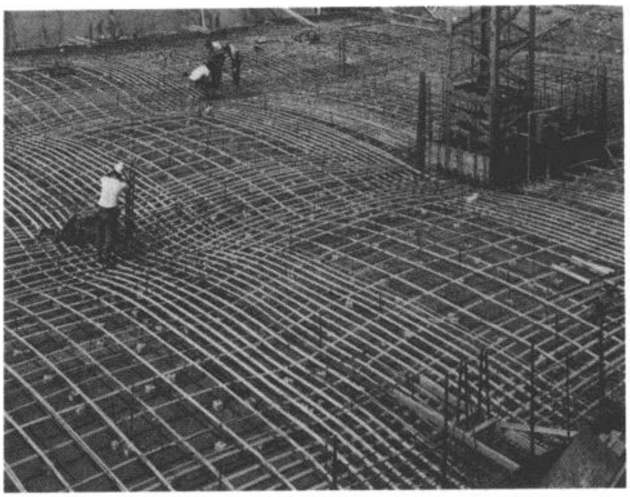


Fig. 1.37 -Tendon Layout for Post-Tensioned Mat Foundation

1.10.2 Beam Strip Foundations

Post-tensioning may also be used in foundations to distribute column loads when the main foundation element is a beam connecting the columns. An illustration of the tendon layout for a post-tensioned beam strip foundation is presented in fig. 7.38. In this case, the tendons consisted of groups of three or four unbonded 1/2-in. diameter, 270k strands.

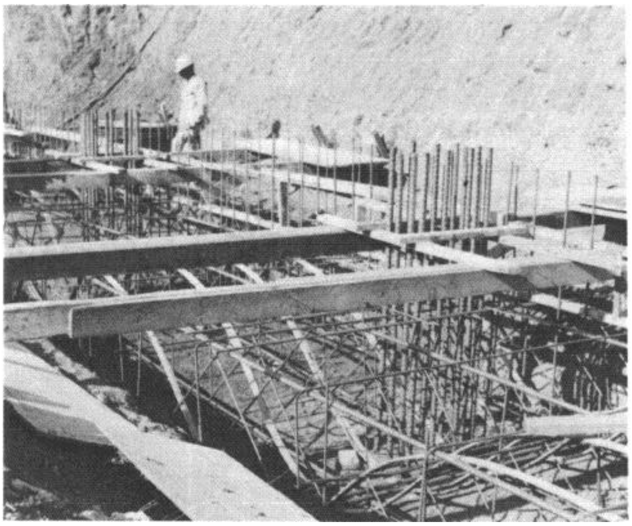


Fig. 1.38 -Tendon Installation for Beam Strip Foundation

1.10.3 Post-Tensioned Slabs on Expansive or Compressible Soils

In areas where expansive and compressible soils occur, there has been widespread use of stiffened (tee beam) post-tensioned slab-on-ground foundations for residential and light commercial construction. The function of the foundation is to provide necessary rigidity to avoid damage to the building or house when the supporting soil expands or contracts at a differential rate due to changes in moisture content. An illustration of a prestressed slab-on-ground installation for a single family residence during concrete placement is shown in Fig. 7.39. Guidance for design and construction of post-tensioned slabs-on-ground used on expansive and compressible soils is presented in a separate PTI publication entitled "Design and Construction of Post-Tensioned Slabs-on-Ground." This publication also includes design examples.

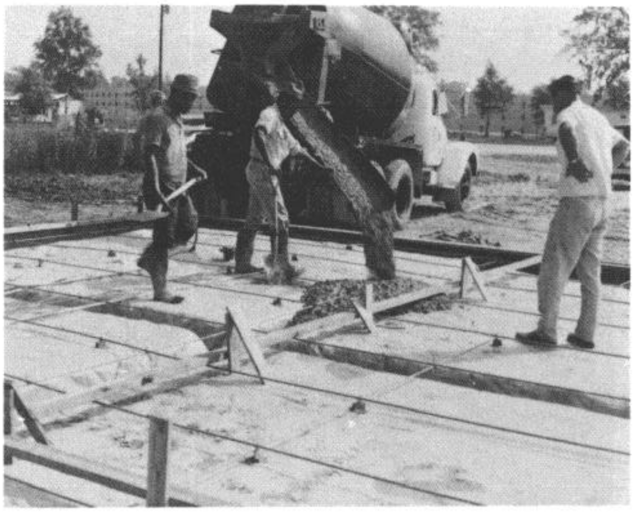


Fig. 1.39 — Tendon installation for Post-Tensioned Residential Slab on Expansive Soil.

Fig. 1.40 — Post-Tensioned Floor Slab, Roundup Centre, Calgary, Alberta, Canada.

1.10.4 Post-Tensioned Commercial and Industrial Floors

Post-tensioned industrial floors such as the 150,000 square foot floor of the Roundup Centre, Calgary, Alberta, Canada shown in *Fig. 1.40* provide initial economy, improved serviceability, and life-cycle cost advantages when compared to other methods of constructing large concrete floors. For example, the 5-1/2 inch thick slab used for the Roundup Centre was built in three 50,000 square foot sections with construction joints at the third points of each section as shown in Fig. 7.47. Only two expansion joints were required for the 150,000 square foot floor along the two interior column lines. Post-tensioning has also been used effectively in construction of "superflat" industrial floors, tennis courts, overlays of deteriorated concrete slabs, driveways, parking lots, and many other similar applications.

The primary reason for the initial 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-tensioning tendons. Additional reduction in industrial floor costs are provided by elimination of most slab joints, reduced construction time, and, in some cases, the elimination of pile supports or drilled shafts. Life-cycle cost advantages of post-tensioned floors are also provided because elimination of joints and cracks reduces both slab and vehicle (fork-lift) maintenance costs. For floors of prefabricated metal buildings, the post-tensioning also serves as a horizontal tie for the horizontal reaction from the building columns. This eliminates the need for conventional reinforcing bars in the slab to dissipate the column reaction into the slab, and at the same time eliminates the possibility of slab cracking associated with such details.

Further discussion of applications and design and construction procedures for commercial and industrial floors is available in a separate PTI publication "Post-Tensioned Commercial and Industrial Floors."

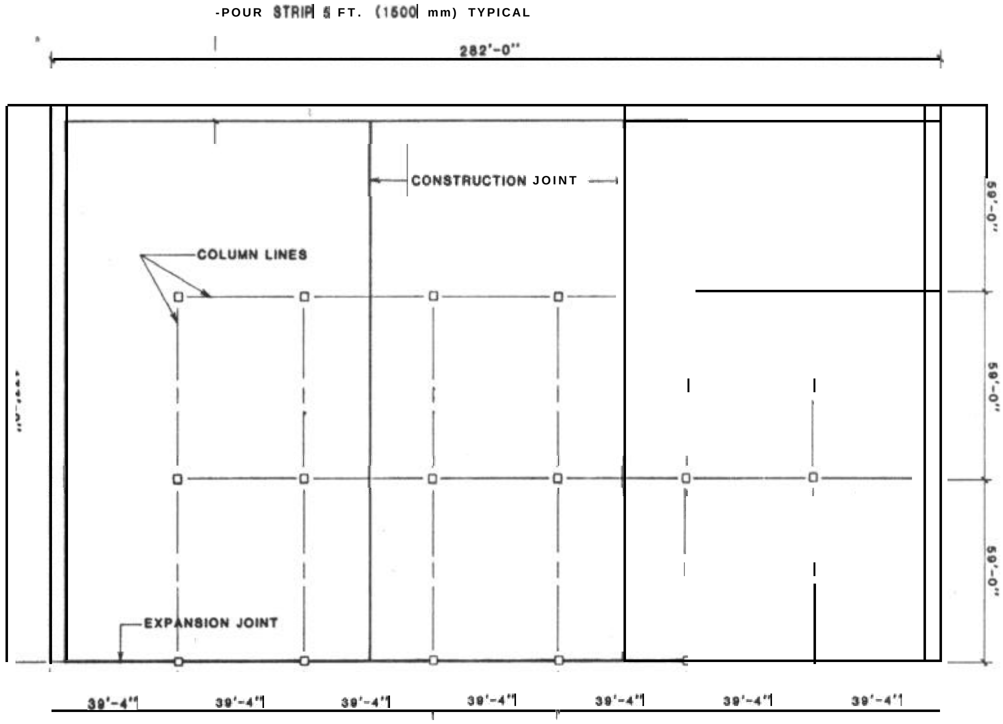


Fig. 1.41 -Construction Joints for 50,000 square feet Section of Roundup Centre Floor.

1.10.5 Highway and Airport Pavements

Four full scale prestressed concrete highway pavement research sections were built between 1971 and 1979 which have a total length of 22 lane miles. These pavements were built in different parts of the United States in various climatic areas and carried a range of traffic. An assessment of performance for these pavements published by the Federal Highway Administration concluded that "Prestressed concrete pavements are competitive on a first cost basis, and provide a viable design alternative to other pavement types. Performance to date would indicate lower maintenance costs will be incurred, and prestressed slab lengths up to 600 feet appear practical." Currently, further research on prestressed concrete pavement overlays is being conducted by the University of Texas at Austin for the Texas Highway Department.

An 800 foot long by 150 foot wide section of a heavily traveled runway at O'Hare International Airport in Chicago was replaced with a post-tensioned runway in 1980. The project was scheduled for a 60-day construction period. It was completed in 56 days. The section of runway, shown in use in *Fig.* 7.42, was heavily instrumented, and it has performed very well. Plans are currently under development for other uses of post-tensioned pavements at O Hare.

Fig. 1.42 — Post-Tensioned Runway, O'Hare International Airport, Chicago, Illinois

1.11 BRIDGES

From the inception of prestressing in Europe, a primary application was the construction of highway bridges. The use of post-tensioning is now a major factor in highway bridge construction in the United States and in various provinces in Canada. 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 highway bridge construction. The advantages of post-tensioning have also been applied to the construction of continuous cast-in-place concrete railway bridges.

1.11.1 Bridges Cast-in-Place on Falsework

The use of post-tensioned concrete box girder bridges cast-in-place on falsework has been shown to be an economical alternative for bridge spans ranging from under 100 ft. to over 300 ft. Further, as illustrated by the Mission Valley Viaduct in San Diego *(Fig. 1-43)* and other applications, this type of construction has also demonstrated economy for falsework heights of at least 150 ft. In addition to the technological and economic advantages related to this type of construction, the aesthetic potential of cast-in-place prestressed concrete bridges is unsurpassed. Further illustrations of post-tensioned bridges of this type are presented in *Figs. 1.44* through *1.46*. A separate PTI publication "Post-Tensioned Box Girder Bridge Manual" provides more detailed discussion of design and construction of all types of cast-in-place post-tensioned bridges.

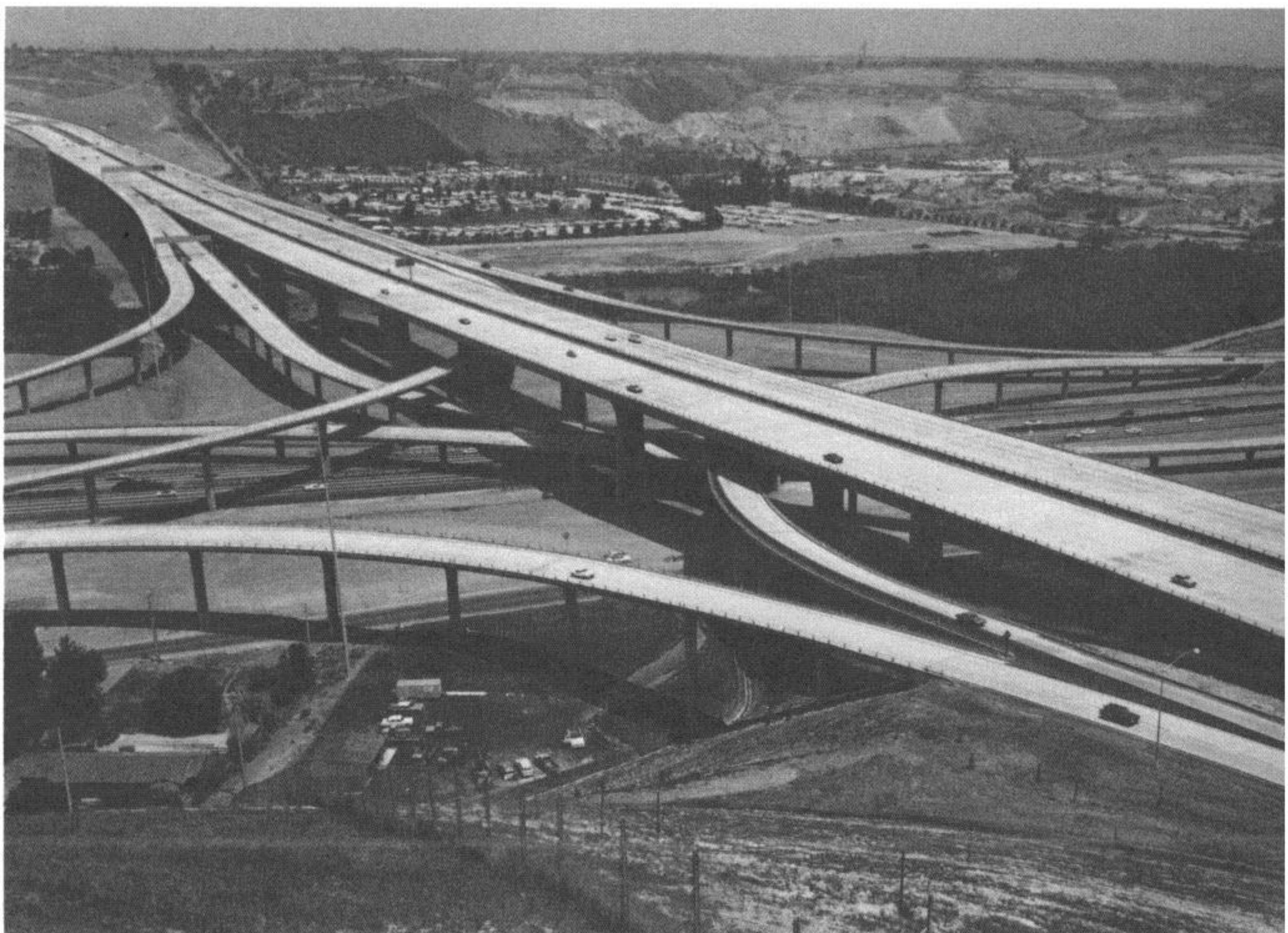


Fig 1 43 Mission Valley Bridge. San Diego, California

Fig. 1.44 — Columbia River South Channel Bridge, I-205 at Portland, Oregon Seventeen spans ranging from 120ft to 200 ft Overall length of 3,114 ft.

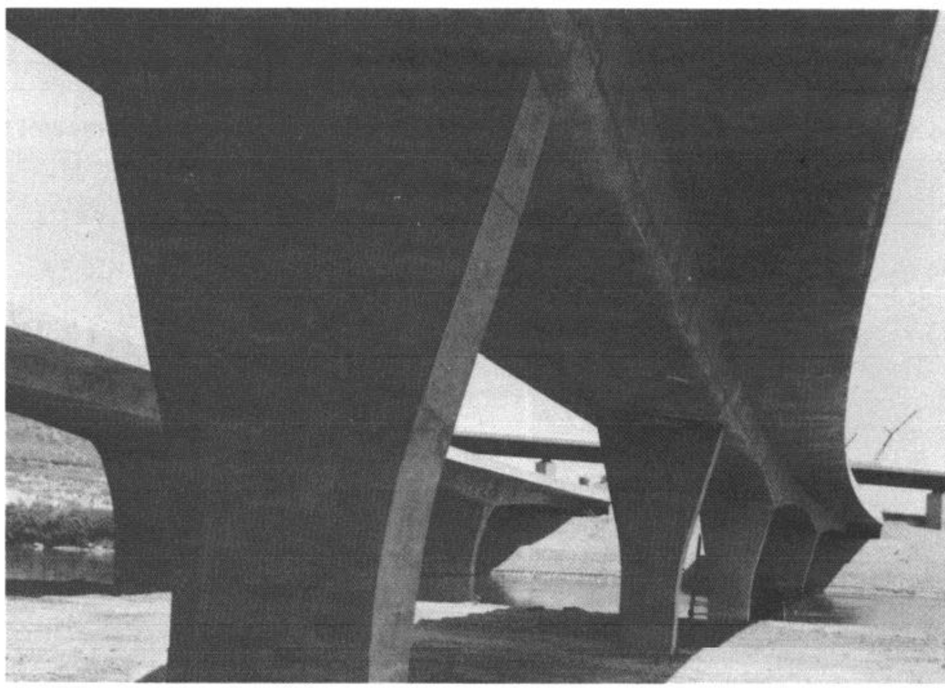


Fig. 1.45 — Yakima River Bridge, SR 182, Benton County, Oregon. Spans range from 140 ft. to 215 ft.

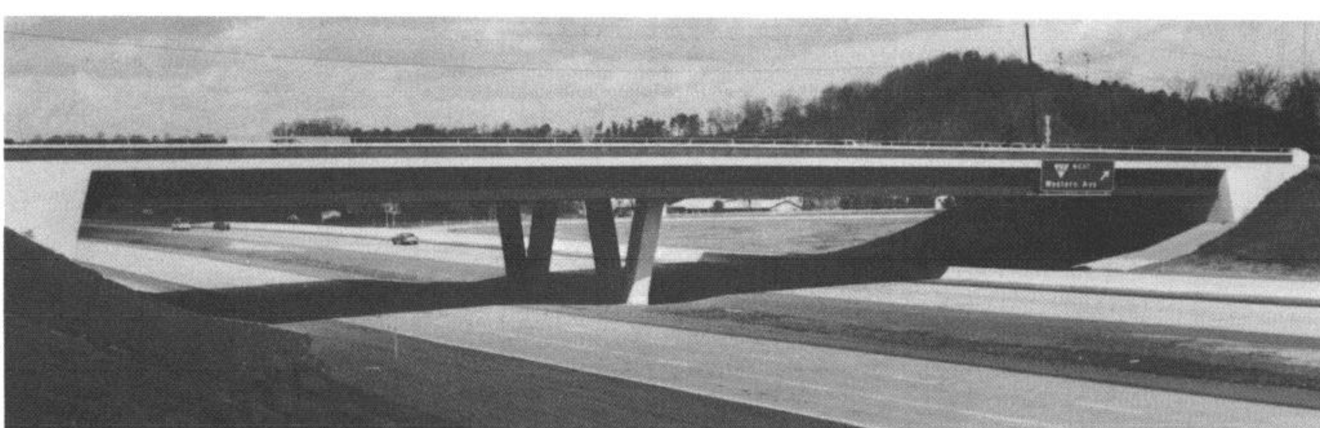


Fig. 1.46 — Interstate Grade Separation Bridge, Tennessee. Spans of 127 and 166.5 ft

1.11.2 Precast Segmental Cantilever Bridges

The use of precast segmental box girder bridges erected in cantilever (without falsework) began in France in 1962. Since that time, this type of construction has become widely used in Europe for spans of 200 to 350 or 400 ft., and has been used throughout the world for many notable bridges. This method of construction combines many of the advantages of cast-in-place post-tensioned bridge construction with the potential for remarkable speed of construction, The latter advantage is further enhanced because the superstructure elements can be cast while the substructure is being built. The "Precast Segmental Box Girder Bridge Manual" published jointly by PTI and PCI contains detailed discussion of the design technology for precast segmental box girder bridges.

One of the first major applications of precast segmental box girder bridge construction in North America was the Bear River Bridge near Digby, Nova Scotia, shown during construction in Fig. 7.47. Although the first of its type in the Province, the Bear River Bridge won a competitive bidding contest against a structural steel alternate. The Bear River Bridge was opened to traffic in December, 1972. The first application of precast segmental box girder bridge construction in the United States was the Corpus Christi Bridge shown in Fig. 7.48. The Corpus Christi Bridge was designed by the Bridge Division of the Texas Highway Department and has spans of 100-ft., 200-ft., 100-ft. Construction of the Corpus Christi Bridge followed extensive testing of a model of the bridge at the University of Texas at Austin which demonstrated reserve strength capacity substantially in excess of specification requirements. The Corpus Christi Bridge was opened to traffic in 1974.

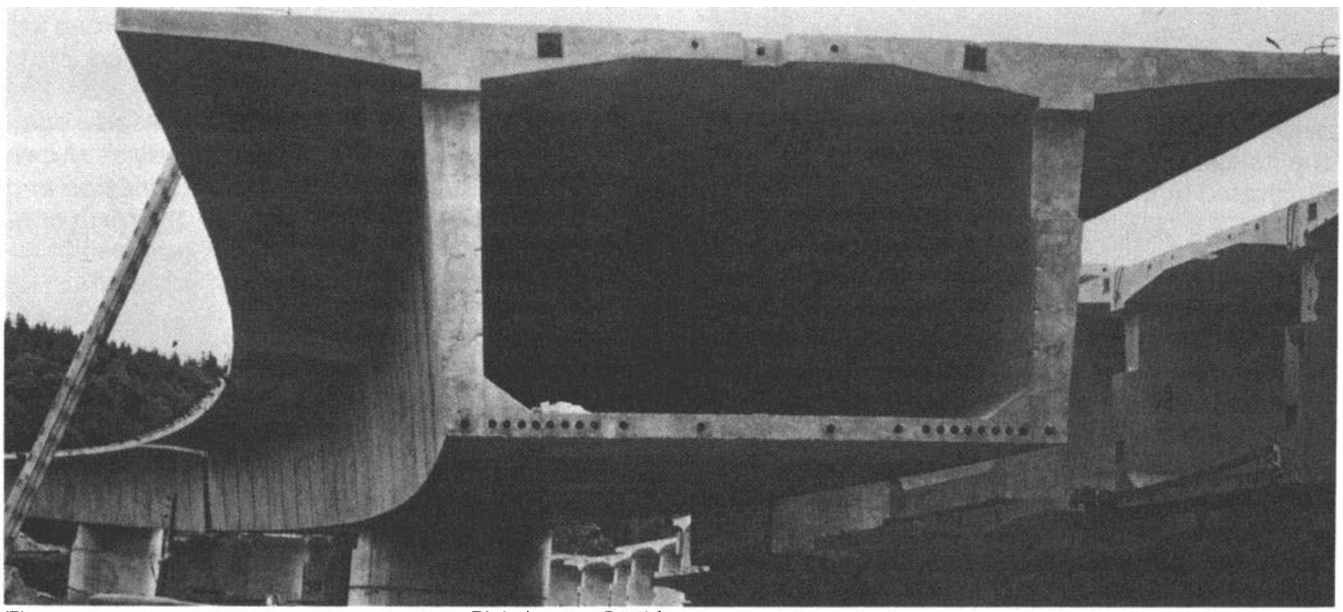


Fig 1 47 Construction View, Bear River Bridge, Digby, Nova Scotia

Fig. 1 48 -John F. Kennedy Memorial Causeway, Corpus Christi, Texas

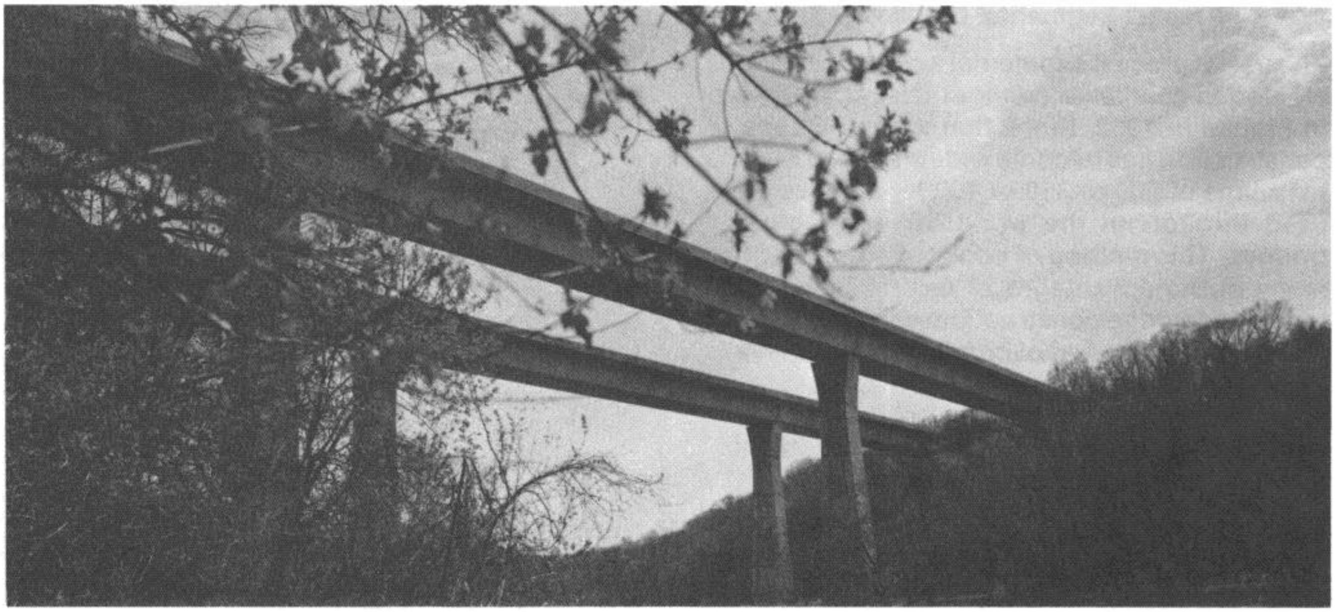


Fig 1 49 The Kishwaukee River Bridge Illinois

Following the success of these first projects, precast segmental construction has become a major factor in the construction of large bridges in North America. A few of the many noteworthy bridges of this type that have been completed in the following years are described below and illustrated in the indicated figures.

- The Kishwaukee River Bridges (Fig. 7.49) near Rockford, Illinois have main spans of 250 ft. and total lengths of 1090 ft. These bridges were the first in the U.S. to be erected with a launching truss, and they were also the first to use bar tendons for the main longitudinal reinforcement.

Fig 1.50 — Seven Mile Bridge, Florida Keys

- The Seven Mile Bridge, Florida Keys (Fig. 1.50), with a total length of 35,863 ft. is the longest continuous prestressed concrete segmental bridge in the world. The 264 spans of 135 ft. length were erected a span at a time with an overhead gantry as shown in Fig. 7.57. Precast segmental design and construction saved 7 million dollars in construction costs, and the project was completed six months ahead of schedule.
- The Wiscasset Bridge, Maine (Fig. 1.52), opened to traffic in 1983 is the first precast segmental box girder bridge in the Northeastern U.S. The Wiscasset Bridge features special engineering and construction features to provide durability under severe winter weather and applications of de-icing salts.
- The Linn Cove Viaduct, Linville, North Carolina (construction view shown in **Fig. 1.53**), illustrates precast segmental construction on curves with radii as small as 250 ft. Due to the "S" shape of the bridge in plan, the superelevation goes from a full 10 percent in one direction to a full 10 percent in the other direction and part way back again in the 1,243 ft. length of the bridge. The segments for the 180 ft. spans are erected in progressive placement with a stiff leg derrick and the use of a temporary erection bent as shown in Fig. 7.53. The use of one-way progressive placement and construction of piers from the top of the bridge deck make the Linn Cove Viaduct the first bridge in the world to be completely erected from the top.

Fig. 1.51 Overhead Gantry, Seven Mile Bridge, Florida, Keys

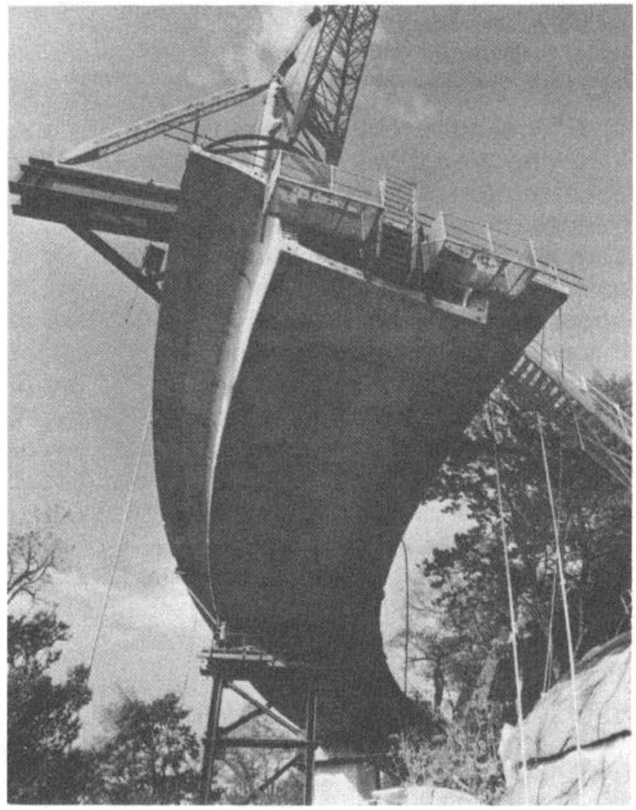


Fig 1 53 — Linn Cove Viaduct, Linville, North Carolina

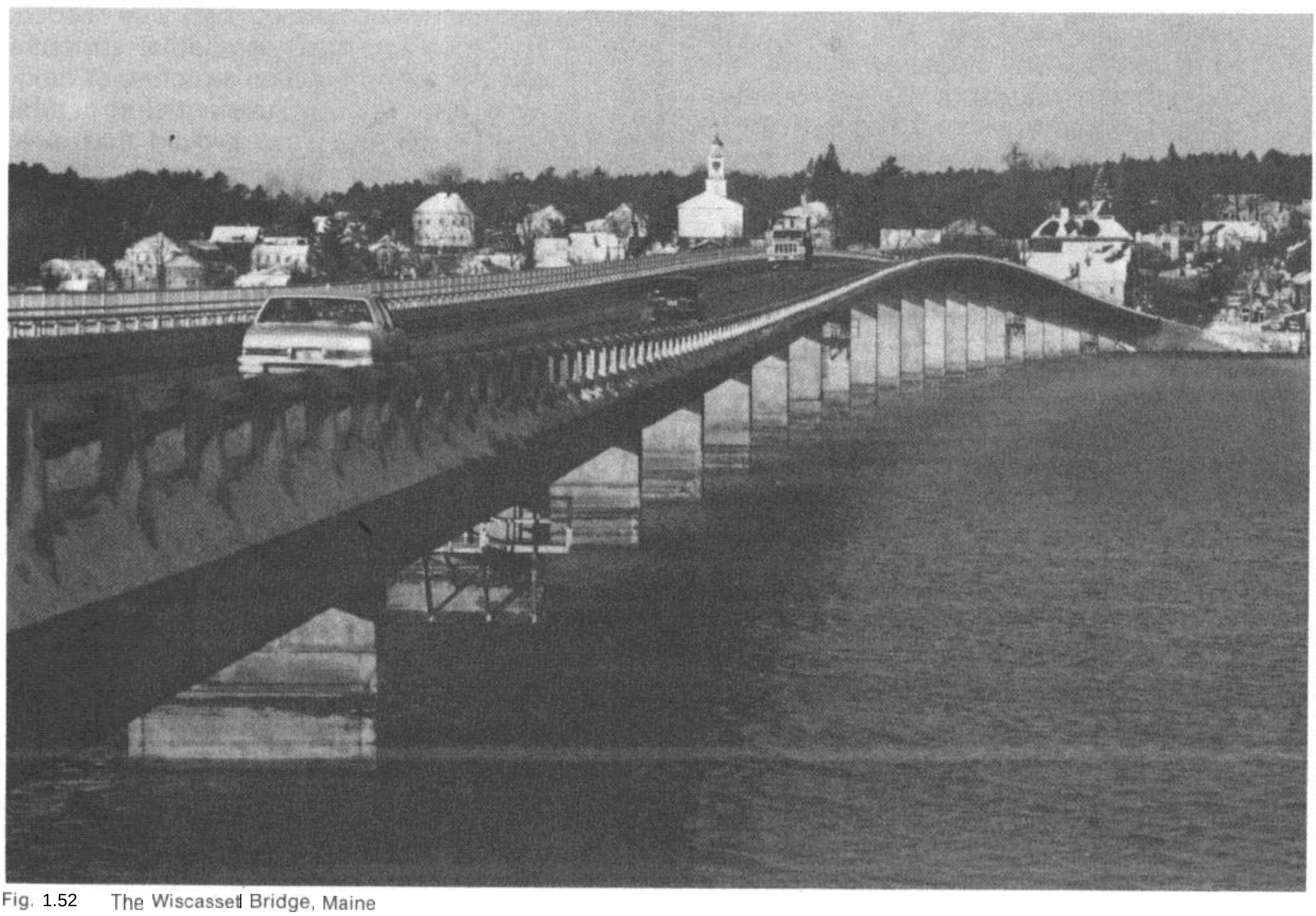


Fig. 1.52 The Wiscasset Bridge, Maine

1.11.3 Cast-in-Place Segmental Cantilever Bridges

The cast-in-place segmental cantilever method of bridge construction developed in Europe following World War II has now been used for construction of many long span concrete bridges throughout the world. This method normally utilizes traveling formwork carriages to cast sections 10 to 15 ft. long at each end of opposing cantilevers. Recently, modifications of this procedure have been found economical for bridges in the 200 ft. to 400 ft. span range where full cantilevers are cast-in-place on falsework and are then stressed segmentally. However, for bridges where the use of falsework is impractical, the cantilever method with cast-in-place segments provides an economical means of concrete bridge construction for spans up to at least 850 ft., the length of the main span of the Gateway Bridge in Brisbane, Australia.

The first North American application of cast-in-place segmental construction was the St. Adele Bridge in Quebec, Canada, completed in 1964. The St. Adele Bridge has main spans of 265 ft. The Knight Street Bridge, Vancouver-Richmond, Canada, with main spans of 360 ft. was completed in 1974. The first United States application was the Pine Valley Creek Bridge near San Diego, California, also completed in 1974. The Pine Valley Creek Bridge shown under construction in Fig. 7.54 has spans of 270, 340, 450, 380 and 270 ft. The piers range in height from 140 to 340 ft. and the roadway is approximately 450 ft. above the water level in Pine Valley Creek. The examples described below are illustrative of the significant accomplishments in cast-in-place segmental cantilever bridge construction since 1974.

- The 750 ft. main span of the Houston Ship Channel Bridge shown in *Fig. 7.55* is the longest in the Americas for a segmental bridge constructed by the cantilever method. The total length of thechannel crossing is 1,500 feet, consisting of the main span of 750 feet and two side spans of 375 feet each. The cross section consists of a two-cell box with a slab overhang of 6 feet. Three webs, 12 to 16 inches wide, are spaced at 23feet. The box is 15 feet deep at midspan with a haunch at the piers 47.5 feet deep. The top slab varies from 10.5 inches to 24.5 inches; the bottom slab varies from 10 inches to 4 feet at the pier.

  The bridge is stressed longitudinally with a mix of 12 and 19 - 0.6 inch diameter strand tendons. All ducts were adequate for 19 strand tendons in the event that additional construction prestress was required. Prestressing bars also were used in the top slab to provide sufficient post-tensioning to advance the travelers without stressing the permanent strand tendons. This eliminated tendon threading and stressing from the critical path.

  The roadway surface of the four-lane bridge was post-tensioned transversely with 4 - 0.6 inch diameter strand tendons. This provided a crack-free deck, thereby enhancing the durability and serviceability of the structure. To permit the webs to carry the high shear forces without cracking, the webs were post-tensioned with two strands 0.6 inches in diameter looped to form a 4 strand tendon.

- Douglas Bridge crossing Gastineau Channel, Juneau and Douglas, Alaska. The Douglas Bridge, Fig. 7.56, consists of 3 spans, 330 ft.-620 ft.-330 ft., with a total length of 1,280 ft.

  Because of marine navigation and channel configuration requirements, a long main span (in excess of 600 feet) was needed. The post-tensioned segmental structure was the only long span structure of moderate cost without substantial structural steel. Experience has proved that steel structures in marine sites require substantial maintenance during their lives. Prestressed concrete, on the other hand, has shown extended maintenance free service. This long, maintenance free life was the deciding factor in selecting post-tensioned concrete at this site.

  The bridge structure is a single box section of variable depth. The side cantilever lengths are 12 feet. The box section, in addition to its structural advantage of high torsional stiffness, provided a location for many utilities. The box section protects the utilities from the sometimes severe weather while providing a permanent work platform for utility-maintenance and repair.

  The structure is post-tensioned longitudinally, the deck is post-tensioned transversely, the webs contain vertical stressing, and looped tendons provide vertical stressing for the piers. Mild steel reinforcing was used to resist transverse bending in the web, and also the bottom flange, which is not transversely stressed.

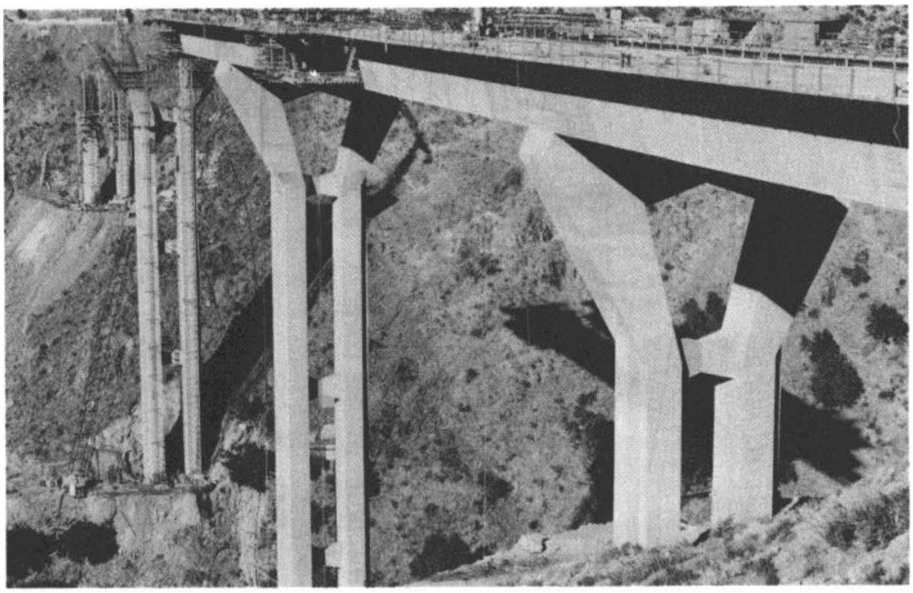


Fig. 1.54 — Construction View of-Pine Valley Creek Bridge near San Diego, California.

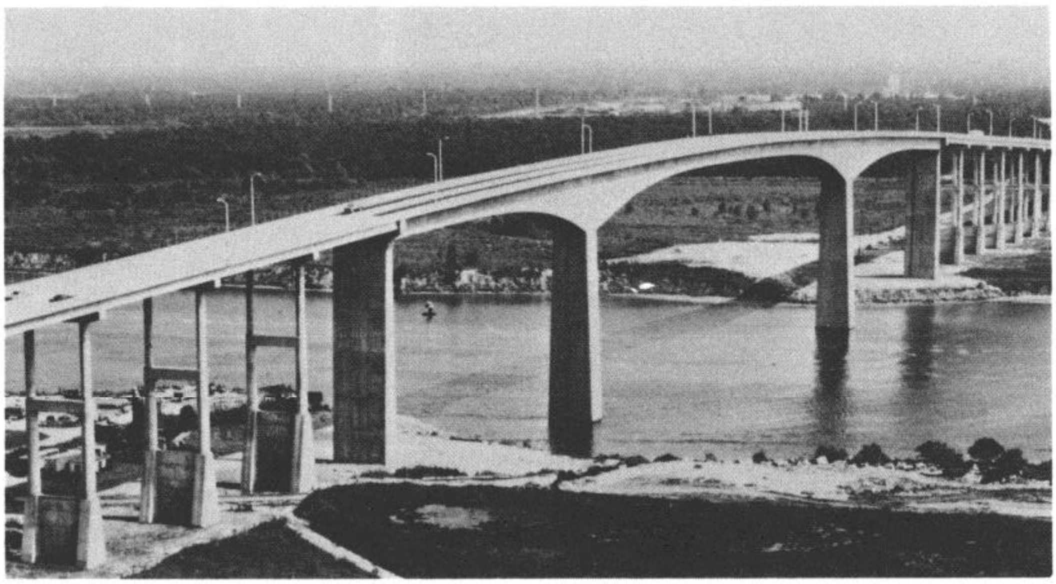


Fig. 1.55 — Houston Ship Channel Bridge, Houston, Texas.

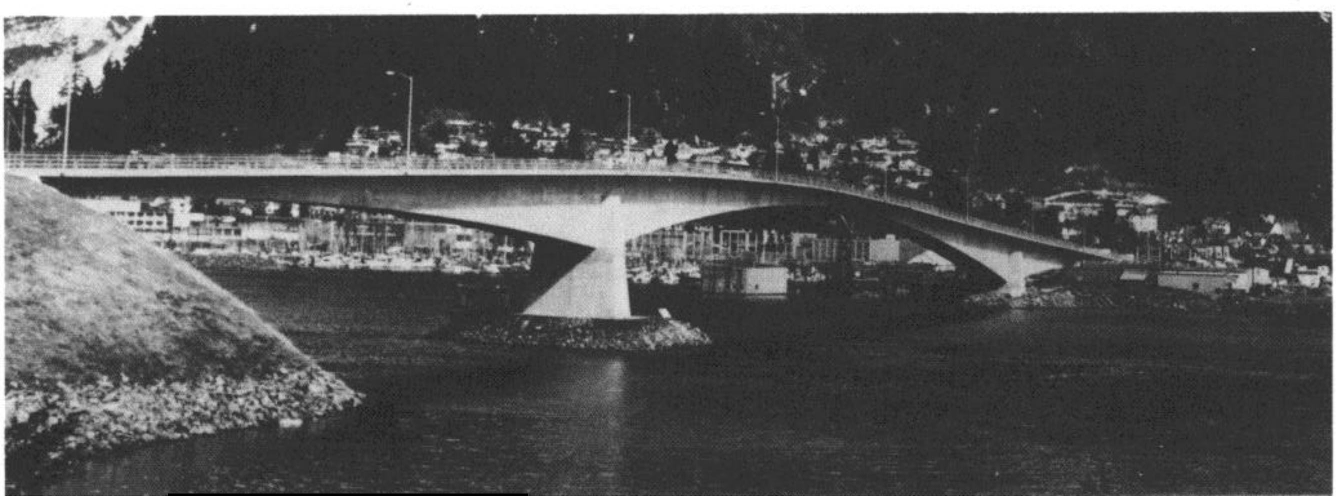


Fig. 1.56 — Douglas Bridge Crossing Gastineau Channel, Juneau and Douglas, Alaska.

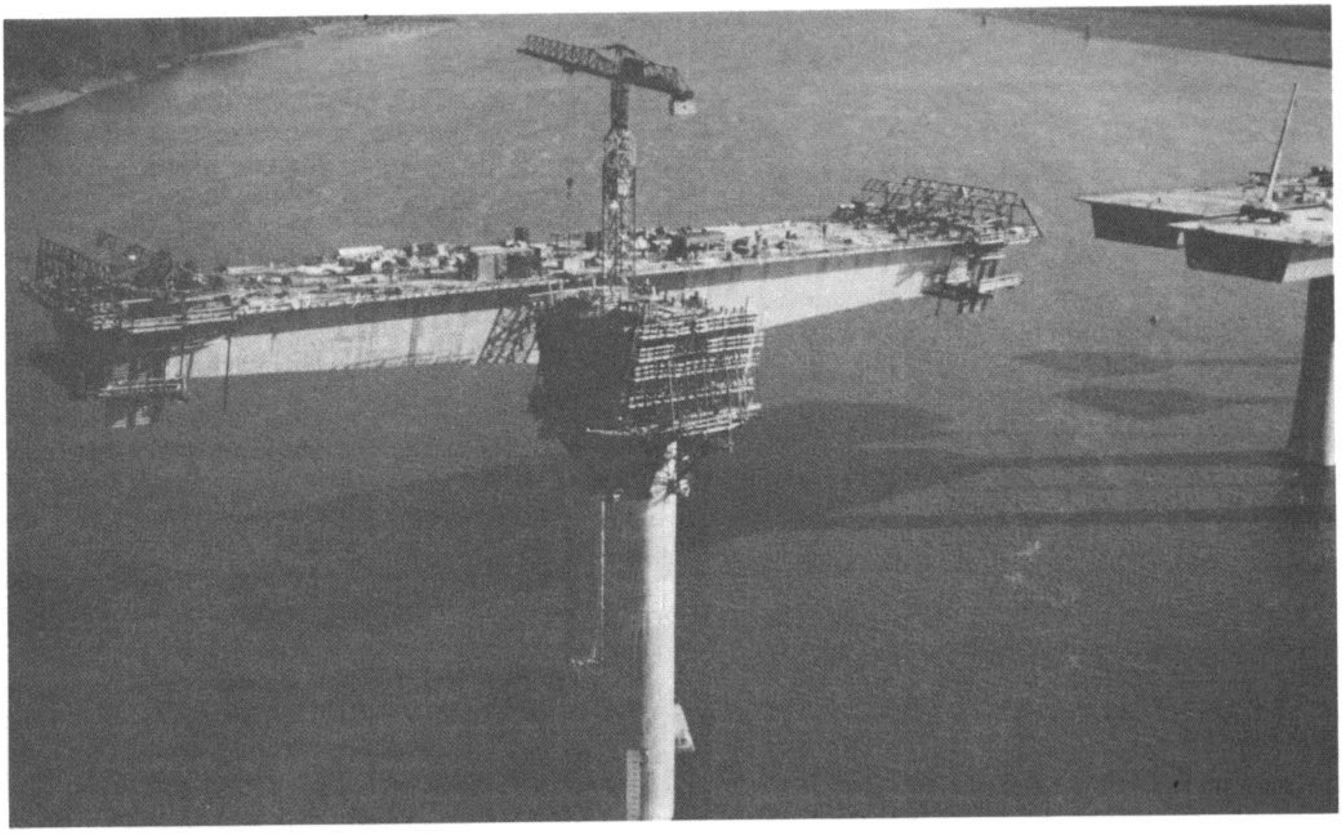


Fig. 1.57 — Construction View, North Channel I-205 Columbia River Bridge, Portland, Oregon.

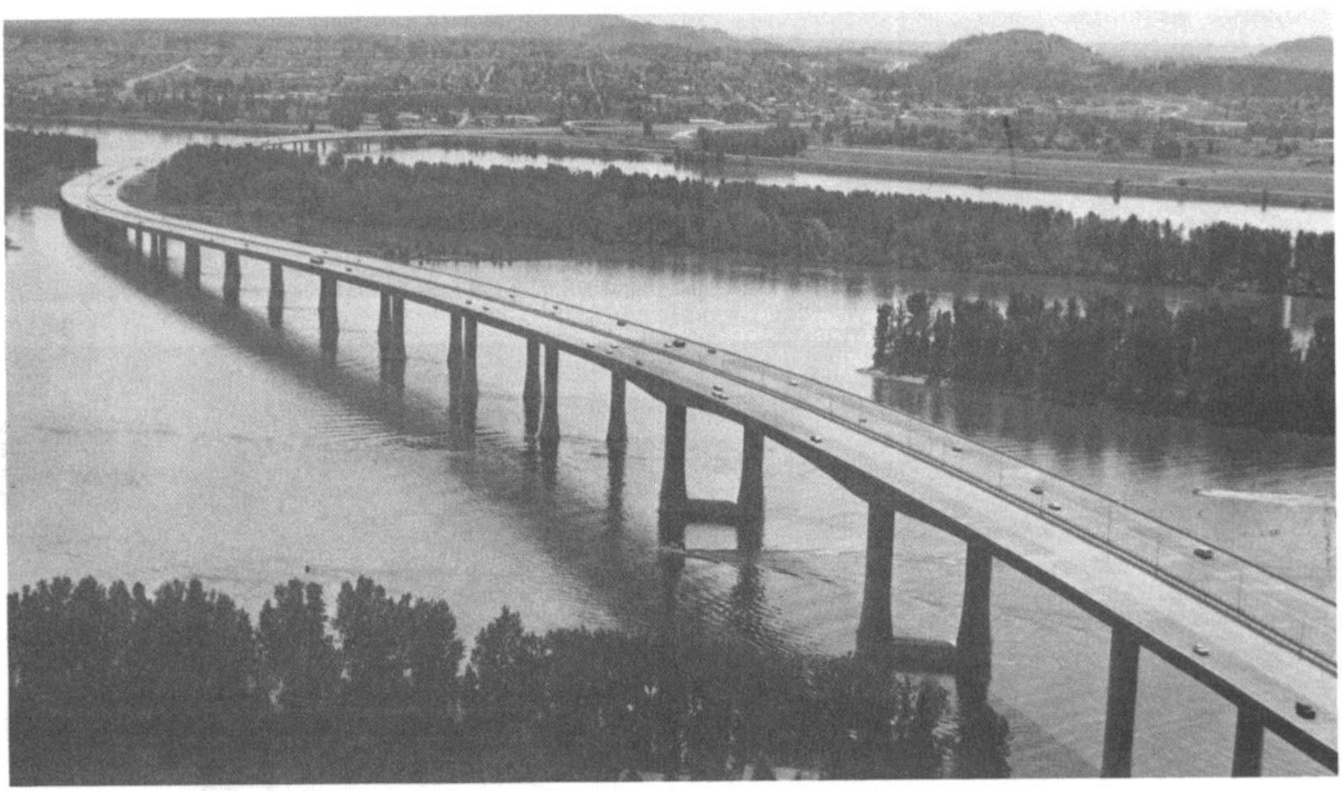


Fig. 1.58 — North Channel I-205 Columbia River Bridge, Portland, Oregon

- North Channel I-205 Columbia River Bridge, Portland, Oregon, Shown under construction in fig. 7.57, and following completion in fig. 7.58, the North Channel I-205 Columbia River Bridge features spans ranging from 240 to 600 ft. for a total length of 5,710 ft. Two separate four-lane roadways, the full shoulders plus a median bikeway result in a total width of 150 ft. out-to-out of the structures. The main span series, which required segments up to 32 feet deep, was built using cast-in-place segmental cantilever construction to eliminate handling very large segments. Segment geometry had to provide for curvature of both the horizontal and vertical alignments. In addition, segment depths tapered from 17 ft. to 12 ft. as span lengths shortened to complement the descending roadway. The 600 ft. main span provides 150 ft. of clearance for river traffic, and then descends to remain below the flight path approaching the airport at the southern end. Segmental cantilever construction eliminated falsework in the river, reducing river flow problems and navigation conflicts
- Genesee River Bridge, Rochester, New York (Fig. 7.59). The Genesee River Bridge utilizes cast-in-place cantilever construction for the 1,332 ft. long river portion, and conventional box girder construction on falsework for a 718 ft. long section over Genesee Valley Park. The river portion features spans ranging from 180 ft. to 430 ft. In the longitudinal direction, draped strand tendons were used in all spans. Thread bar tendons were used for transverse post-tensioning of the deck slab and vertical post-tensioning of the webs.

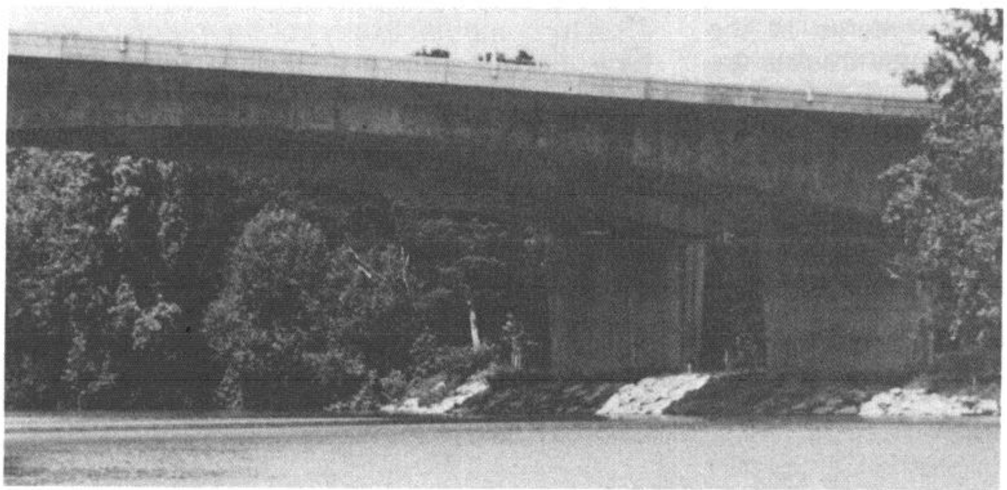


Fig. 1.59 — Main River Spans, Genesee River Bridge, Rochester, New York.

1.11.4 Cable-Stayed Bridges

Cable-stayed bridge technology extends the span range of concrete superstructures to at least 1,200 ft. and possibly to as much as 1,800 ft. The Sunshine Bridge in Tampa (Fig. 1.60), presently under construction, has a main span of 1,200 ft., and will be the longest concrete span in the world when completed. In the last ten years, cable-stayed bridges have emerged as a very economical and esthetically pleasing alternative for bridge spans of 600 to 1,200 ft. A significant number of major bridges are now under design and construction utilizing cable-stayed technology.

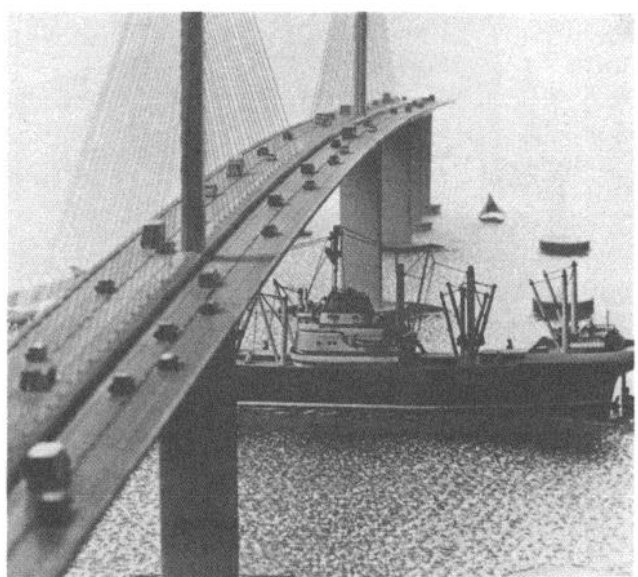


Fig. 1.60 — Model of Sunshine Skyway Bridge, Tampa, Florida

1.11.5 Rapid Transit Bridges

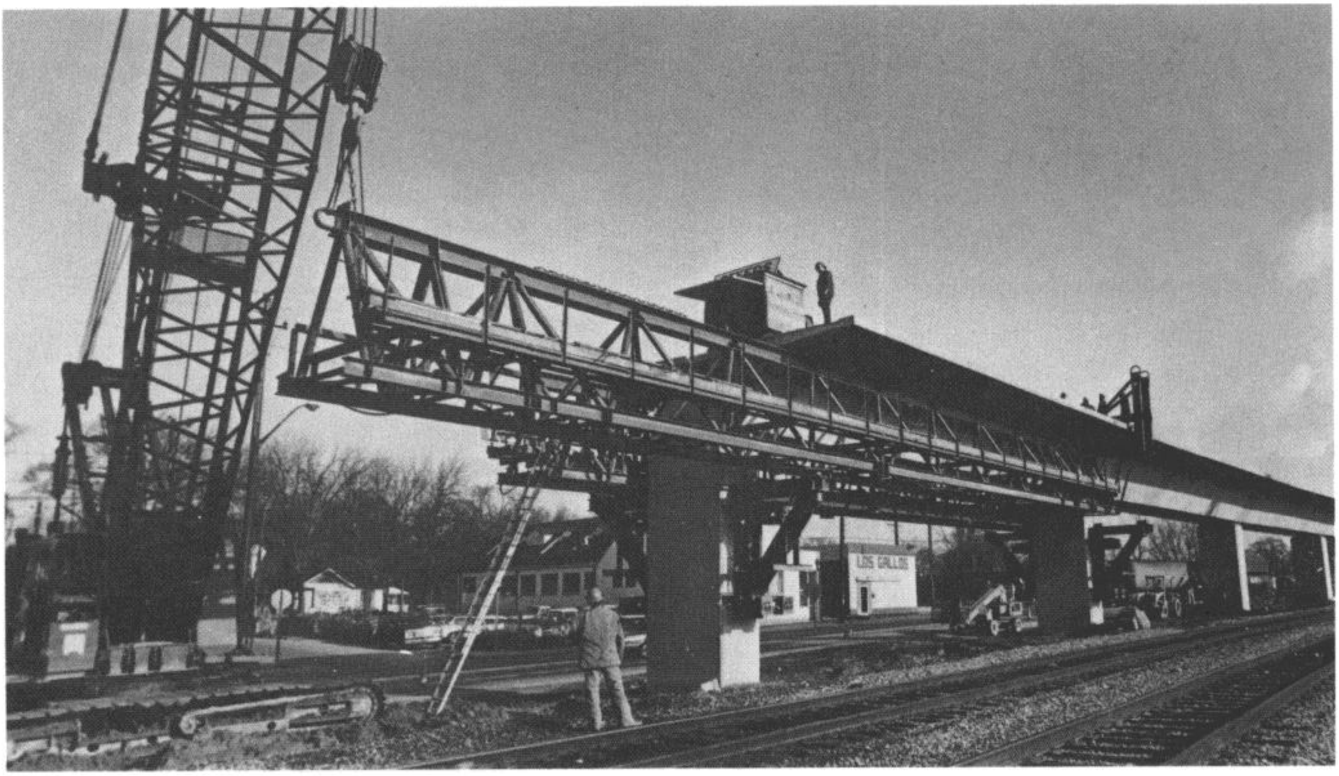
Post-tensioned segmental construction of various types has been applied successfully to major light rail rapid transit and people-mover bridges. Two outstanding examples described below are the Metropolitan Atlanta Rapid Transit Bridges, and the Walt Disney World Monorail in Orlando, Florida.

The first precast segmental concrete railway structures in the United States were recently completed for the Metropolitan Atlanta Rapid Transit Authority (Fig. 7.61). With a combined total length of over 7,000 feet, the structures incorporate significant design and construction innovations which will promote the development of rapid transportation systems in the U.S. A new assembly truss concept developed for this project proved to be a real "breakthrough" for the construction of 70' to 140' spans in highly congested urban areas. Instead of a single truss beneath the superstructure supporting the segments at the bottom slab of the box, the new trusses are triangular and are located on each side of the box section, supporting the box girder under the wings. Through utilization of the triangular trusses for span-by-span erection, it was possible for the contractor to complete one span per day.

The following design and construction techniques are incorporated in the structures; saving money and time while still providing the best bridge possible:

- multiple shear keys in the box webs;
- span-by span erection;
- match-cast segments with no epoxy in the joints;
- transverse pretensioning of the top slab;
- external post-tensioning with tendons within the box girder void, but external to the concrete.

The Walt Disney World Monorail in Orlando, Florida *(Fig.* 1.62) features 7 miles of narrow long-span beams supported on tall, tapering columns. The design, which complements a 6-mile guideway section completed by the same team a decade earlier, makes truly innovative use of post-tensioned prestressed concrete. Beams were precast on site, using a universally adjustable form for curved beams set by computer to any configuration required by the three-dimensional geometry. Curved beams were post-tensioned to balance dead loads immediately after leaving the form while straight beams were pretensioned in a self-stressing form. Careful design of prestress gave zero-camber conditions, both short and long-term. The resulting erection and align-



Fig, 1.61 — Metropolitan Atlanta Rapid Transit Authority Bridges under Construction ment efficiency yielded substantial cost savings. The beams were erected and simply supported on precast columns until the beam-to-column joints were cast. Six spans were then post-tensioned with three tendons to form a 640-foot long continuous structure. The parabolic-haunched soffit of the beams simplified continuous post-tensioning through beamway segments, and allowed the stressing tendons to run straight, yet be effective for both positive and negative moments. Use of continuous post-tensioned segments allowed an increase in span lengths, reduced the number of expansion joints, and eliminated the need for bearings. Future maintenance needs were greatly reduced by the absence of typical connection and bearing hardware.

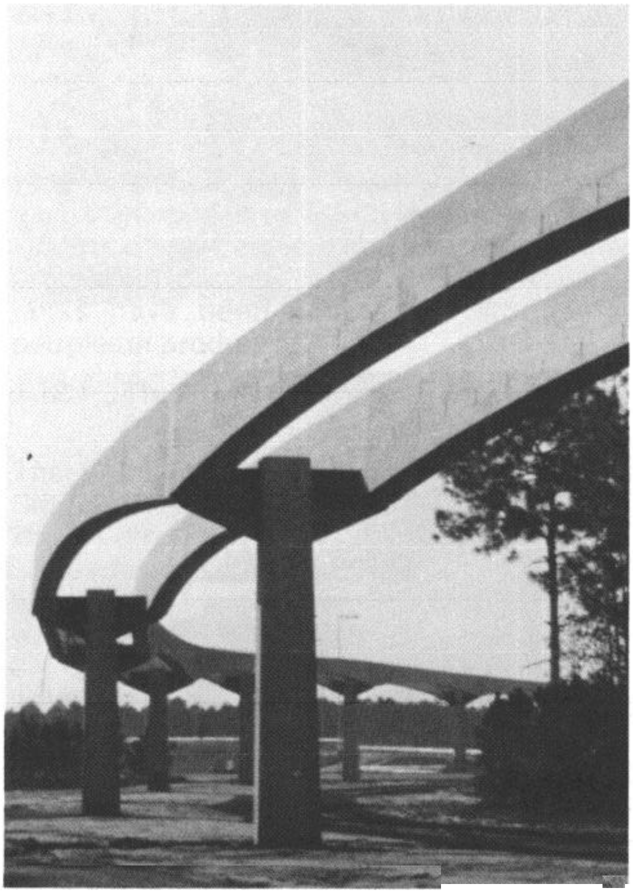


Fig. 1.62 — Curved beam section of Walt Disney World Monorail Orlando, Florida.

1.11.6 Bridge Repair and Replacement

Post-tensioning has been used effectively in bridge repair and replacement projects for all types of steel, concrete and prestressed concrete bridges. Two examples are presented in this section as illustrations of the many possible uses of post-tensioning for this purpose.

The replacement of the original San Lorenzo River Bridge in Santa Cruz County (Fig. 1.63), California is located on a designated scenic route which is also the only road through the area. It spans a waterway which must remain unobstructed due to rain swollen flow and large drift from the surrounding forests. Traffic was to be maintained at all times. To satisfy these controls, a post-tensioned cast-in-place box girder was selected thus providing the reduced structure depth required for clearances, an unobstructed channel and a smooth soffit to accommodate debris. It also simplified partial width construction which satisfied traffic control needs. Portions of the existing piers were incorporated into the final structure. Two separate outside structures of about 14 feet in width were constructed while traffic used the existing structure in the center. Traffic was then switched to these outer structures and the center portion was built using part of the existing piers with new caps. The structure was completed with the closure pours connecting the outer sections. Soon after this structure was completed, it successfully survived a large flood which caused severe damage to many buildings in the immediate area.

External post-tensioning, as shown in fig. 1.64, was used to restore the structural integrity of five cracked cantilever piers for the Route 695 Bridgeover Route 151 in Baltimore, Maryland. A need for remedial action became apparent when bridge inspectors discovered open cracks in the top part of the pier caps indicating overstress through the cantilever section. The five skewed piers supported roadway spans ranging from 71’ to 127’. The pier caps were 63’ long end to end which included equal 15’ cantilever sections. Fabricated structural steel weldment-bearings on each end of the pier cap were used as an anchorage for ten (10) post-tensioning bars (five on each face of the pier cap). The initial post-tensioning force applied per bar varied from 123 kips to 170 kips, depending upon the pier being stressed. In the interest of simplicity and expediency, the post-tensioning mechanism was identical for all five piers; however, the total post-tensioning force on each pier was customized directly for corresponding loads carried by each pier. After the end anchorages were fabricated and the post-tensioning bars were on the site, erection and post-tensioning was completed in two weeks.

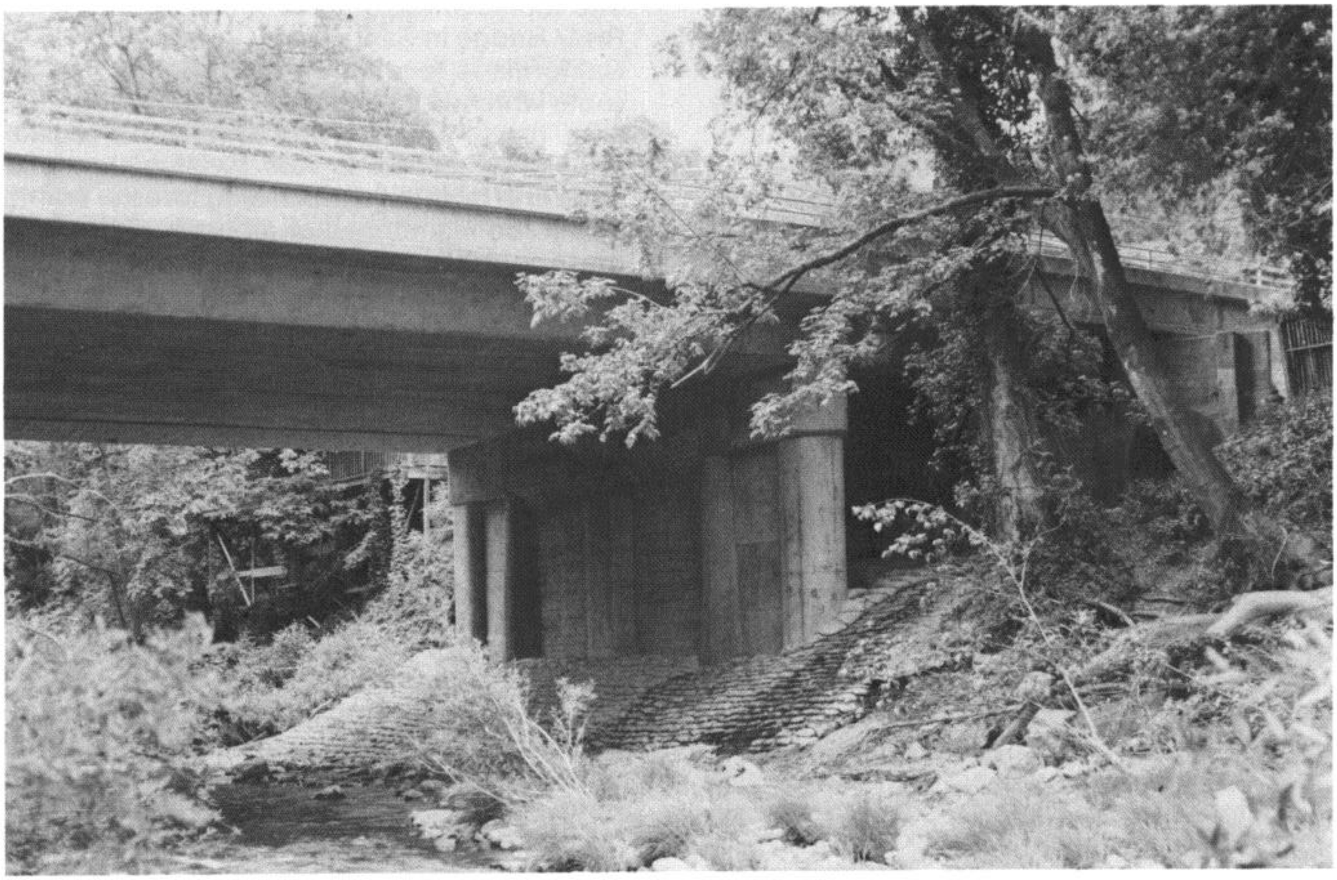


Fig. 1.63 — San Lorenzo River Bridge, Santa Cruz County, California

Fig. 1.64 — Repair and Strengthening of pier caps by use of external post-tensioning. Route 695 Bridge over Route 151. Baltimore, Maryland.

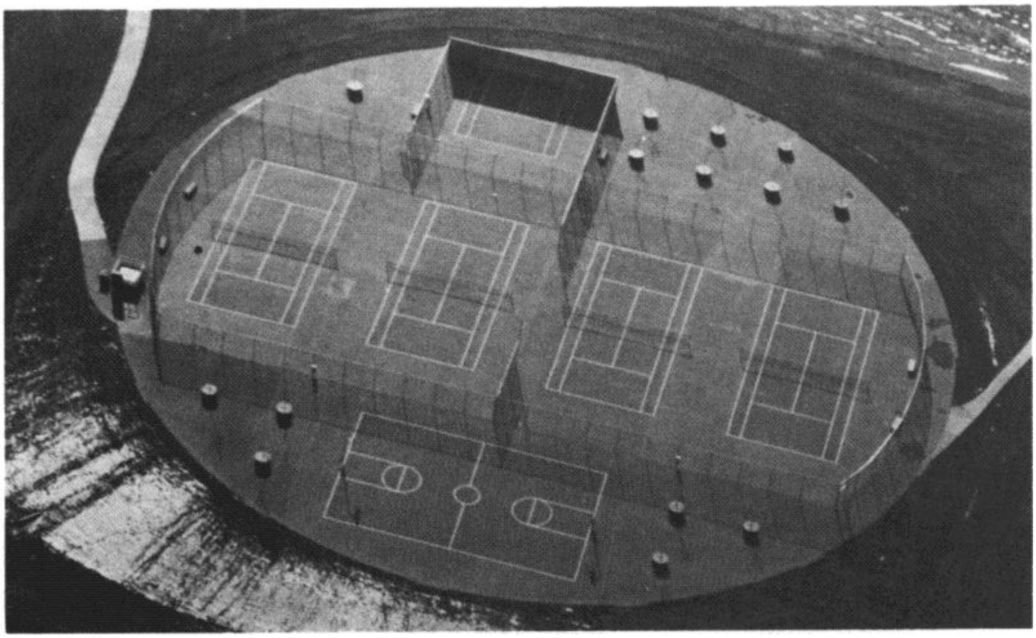


Fig. 1.65 — Treated Water Storage Reservoir, Arvada, Colorado.

1.12 STORAGE TANKS

Post-tensioned prestressed concrete tanks have been extensively used for water storage, storage of coal, cement and a variety of other products. Relatively recently, post-tensioned tanks have been adapted to storage of petroleum products.

1.12.1 Treated Water Storage Reservoir Arvada, Colorado

The 10 million-gallon Arvada Treated Water Storage Reservoir shown in Fig. 7.65 is the largest circular concrete tank in the United States designed and constructed using a rotationally and translationally restrained floor to wall joint. The tank has an inside diameter of 275 ft., and 24 ft. high walls, post-tensioned both vertically and horizontally. The 59,400 sq. ft. roof is a concrete flat slab, post-tensioned in two directions. There were objections at first to building the tank in a residential area. For this reason the tank itself was buried and the site planned as a recreational park.

1.12.2 North-West Reservoir, Regina, Saskatchewan

For the North-West reservoir in Regina, Canada, the challenge of locating a massive industrial structure on a restricted site in a predominately residential area was identified early in the conceptual design stages. The hydraulics of the water-works system and a partially completed freeway interchange bordering the site precluded any flexibility in locating the structure and complicated the problem of building the reservoirwith minimal impact on the residential community. In a sensitive response to this challenge, a round post-tensioned precast concrete structure with architectural precast concrete panels and complementing land forms and planting was selected.

Prestressed, post-tensioned, precast concrete was favored over steel, wire-wound shotcrete and cast-in-place structural systems after consideration of the following:

1. First Cost - preliminary cost estimates predicted a saving of about seven percent over the closest competing system - normally reinforced, cast-in-place concrete.
2. Watertightness- prestressing of wall panels in two directions - vertical pre-tensioning and circumferential post-tensioning-enhanced watertightness.
3. Quality Control - excellent quality control was achieved since fabrication was undertaken under plant conditions and the panels were post-tensioned in the field.
4. Speed of Construction - the project schedule dictated that the reservoir be commissioned prior to the peak summer season, forcing construction operations to continue during typical harsh winter conditions. Plant production of the majority of the structure and post-tensioning in the

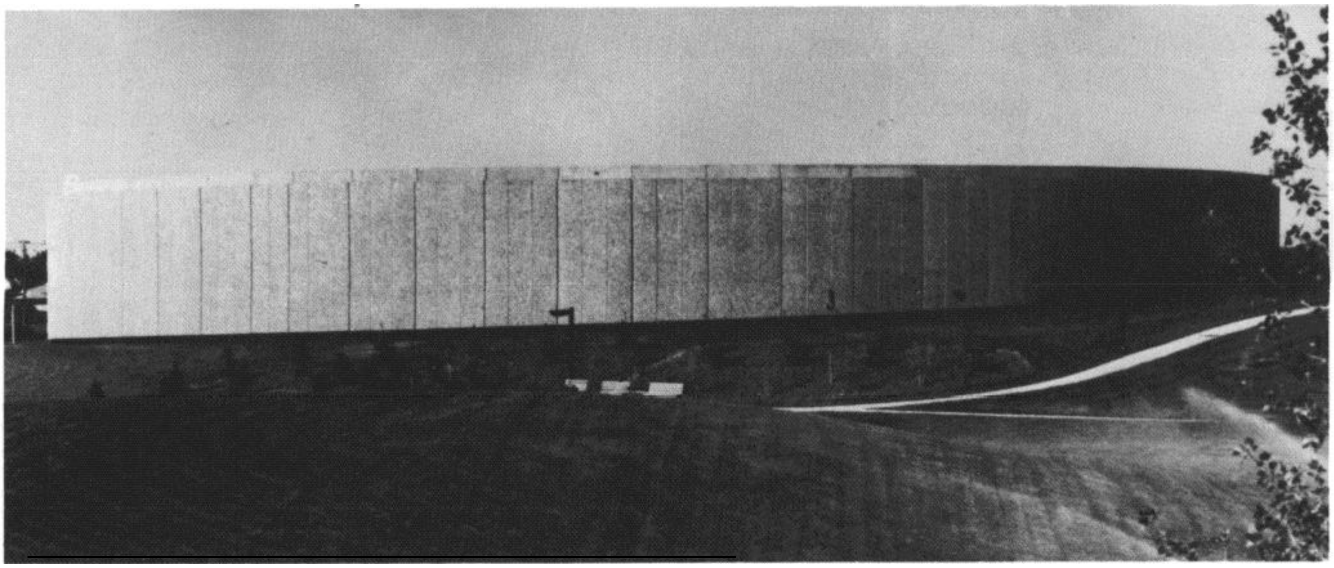


Fig. 1.66 — North-West Reservoir, Regina, Saskatchewan

field was the only apparent way to meet the demands of this schedule.

As indicated by the completed tank shown in Fig. 1.66, the result has been a net gain for the residents of the area, in that a neighborhood park, with an eye-pleasing water storage reservoir providing a backdrop to the landscape and a buffer to the interchange, has been created for year-round recreation.

1.13 NUCLEAR CONTAINMENT VESSELS

The safety of a nuclear power station is the first consideration in every reactor structure design. 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. 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, Colorado, was the first of its type in the United States, Fig. 7.67 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.

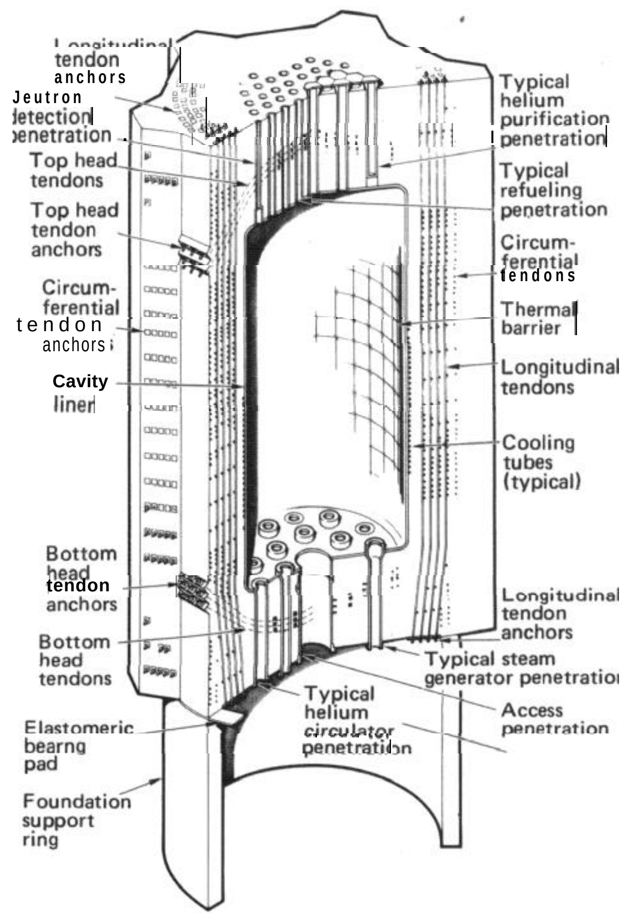


Fig. 1.67 — Details of Fort St. Vrain Reactor Vessel

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. 7.68 shows construction of the post-tensioned containment structure of the Palisades Nuclear Plant near South Haven, Michigan.

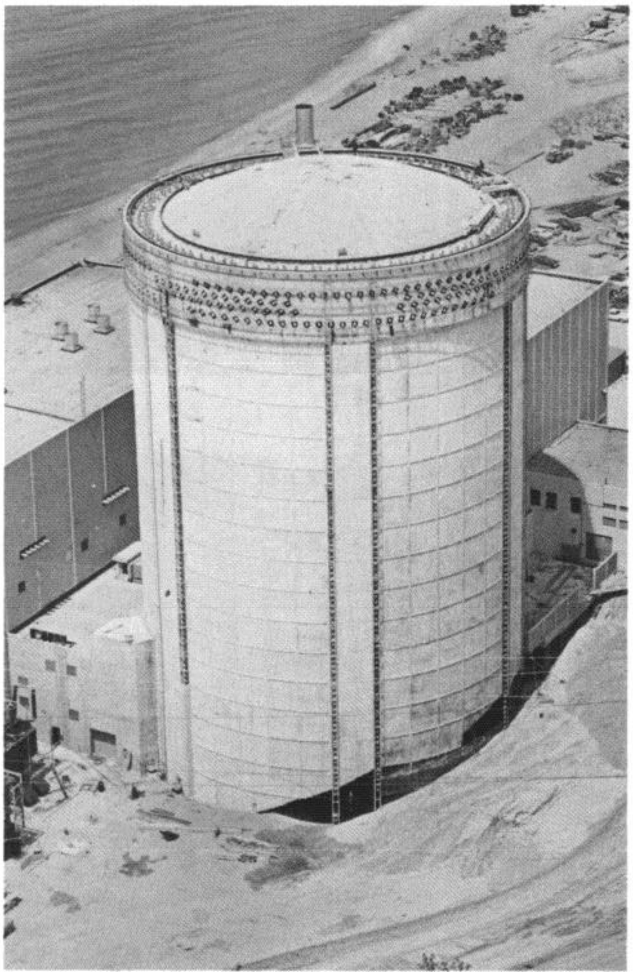


Fig. 1.66 — Construction View of Post-Tensioned Containment Structure of Palisades Nuclear Plant Near South Haven, Michigan

1.14 SPECIAL APPLICATIONS

1.14.1 Stage Post-Tensioning

The City of Chicago recently extended its rapid transit line to O'Hare International Airport to provide a direct link to downtown, the "Loop". The new Chicago Transit Authority station at O'Hare is located beneath an existing six-story 14,000 car 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 (7) transfer girders was needed to support the garage over the top of the station. Each girder, with dimensions of 10 ft. x 10 ft. x 75 ft., supports five stories of the existing garage, plus the ground level parking. In addition, they were designed to accommodate a top-to-bottom construction procedure.

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 *(Fig. 7.69 and 1.70)* 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. 1.71)* eliminating the need for temporary supports. Special temporary post-tensioning was required for this stage since the support of the girder by the garage columns and caissons represented the opposite of the final structural action of the girders.
- After construction of the permanent supports for the girders, before cutting off the

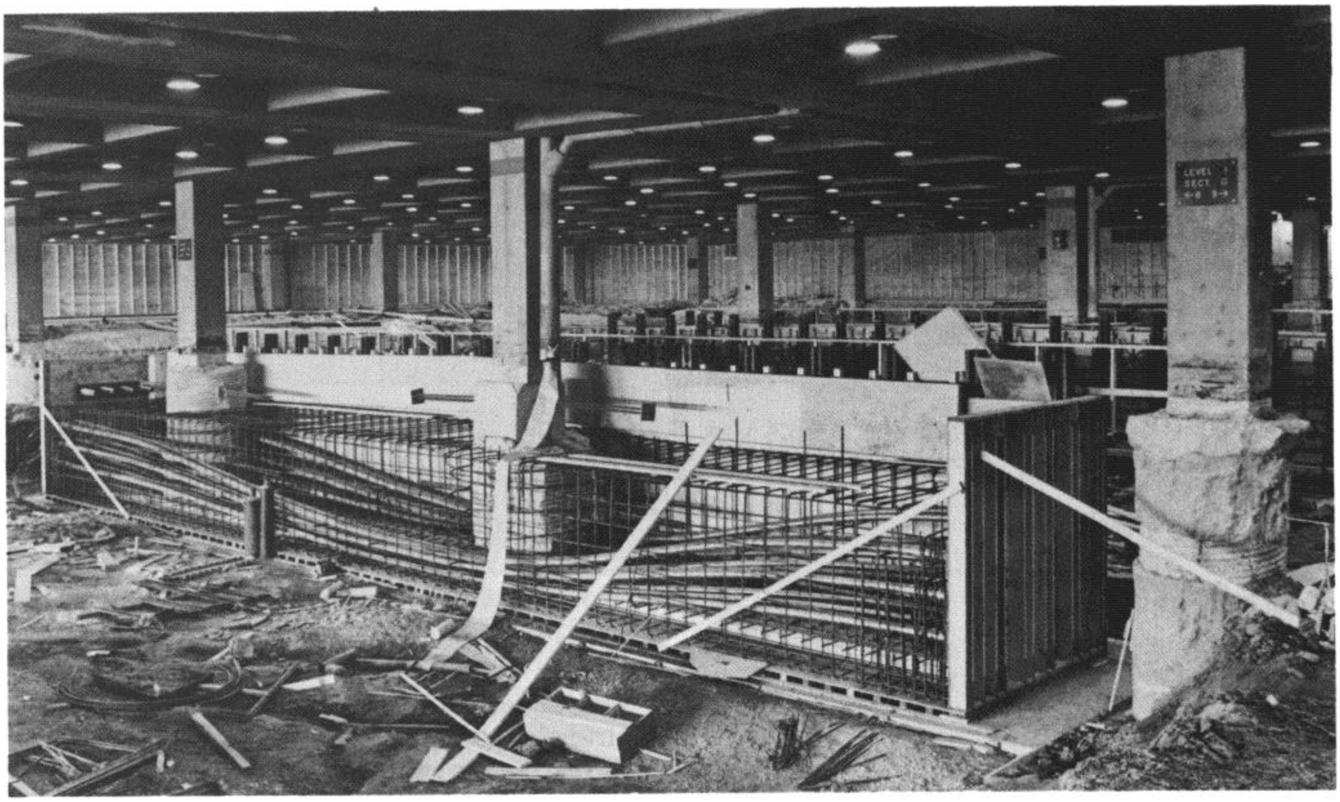


Fig. 1.69 -Transfer Girder Construction. O'Hare International Airport, Chicago.

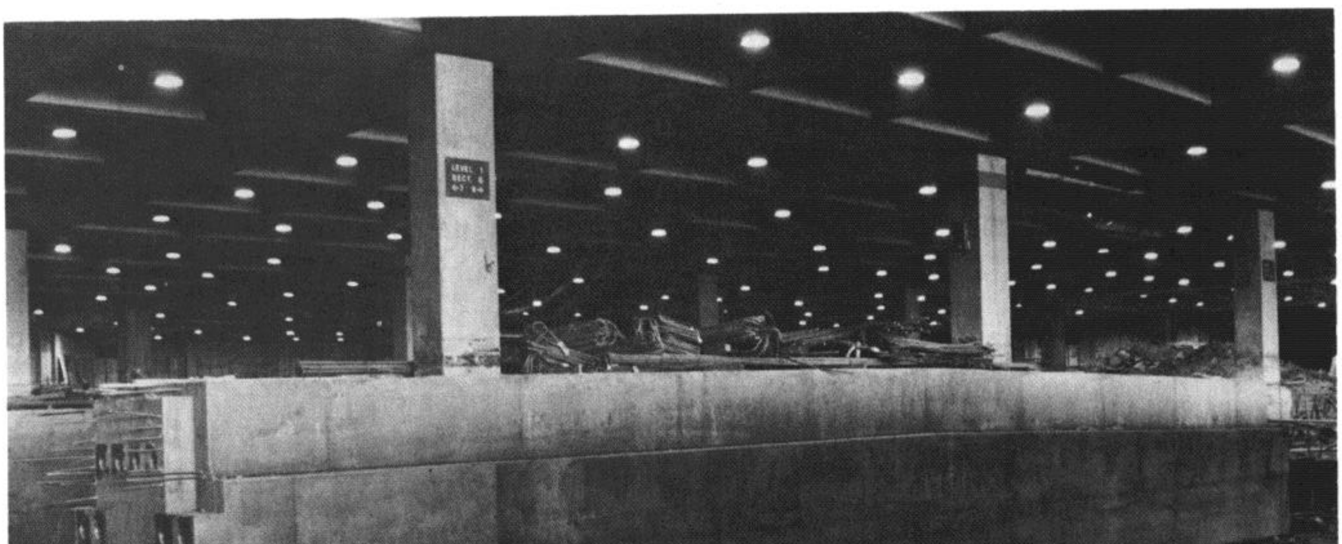


Fig. 1.70 — Completed Transfer Girder, O'Hare international Airport, Chicago.

existing garage caissons, precisely computed final post-tensioning and detensioning was applied to transfer all 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. 7.72 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. 7.73, the completed transfer girders are in place during the final station construction phase.

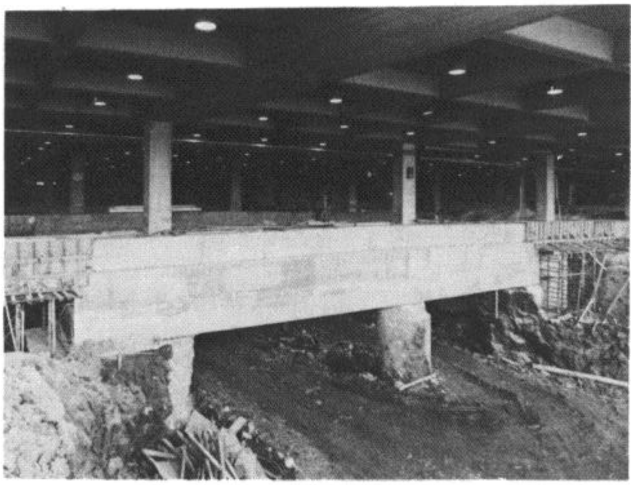


Fig. 1.71 Excavation Under Transfer Girders Supported on Original Caissons.

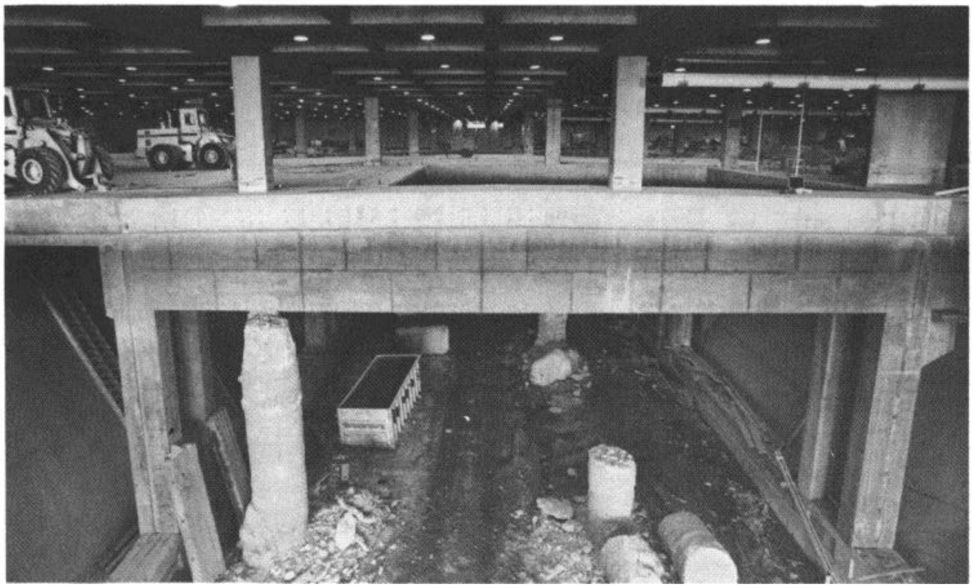


Fig. 1.72 — Cutting Off Original Caissons - Girder Supported on New Columns.

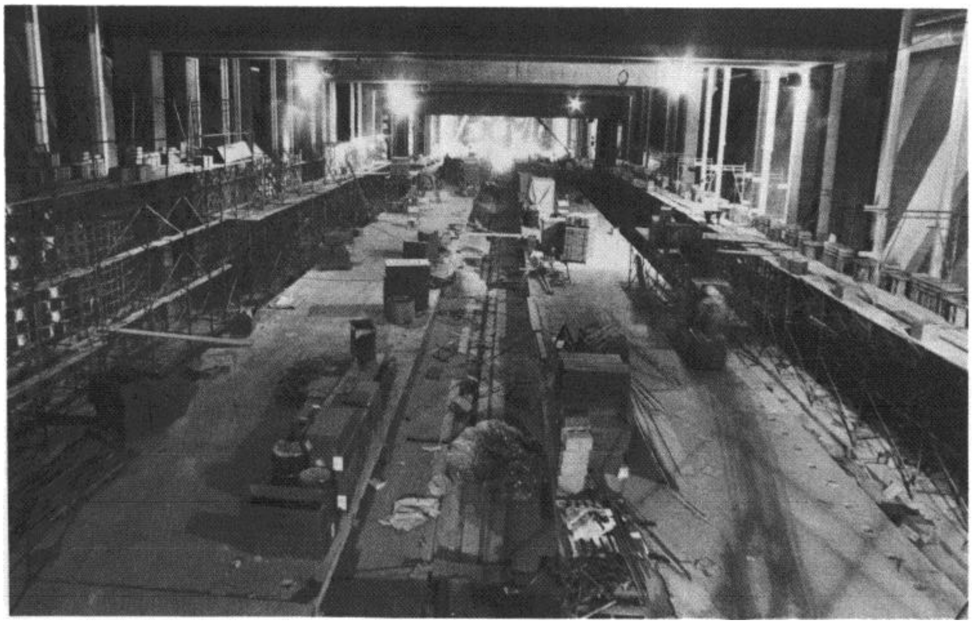


Fig. 1.73 — Final Station Construction Phase.

1.14.2 Floating Structures

Post-tensioning has been used in the construction of many types of floating structures including boats, oil exploration and storage platforms, marinas, barges and other similar applications. A major project of this type was the reconstruction of the west half of the original Hood Canal Floating Bridge which was destroyed in 1979during what a University of Washington Meteorologist called a violent "100 year" storm. The loss of this bridge caused great social and economic impact in the region, and it was necessary to restore the bridge to traffic as soon as possible. Due to the great water depth of the Hood Canal, a floating structure was considered to be the most economical means of spanning across the canal. The design and construction of this floating bridge is unique. It is the world's only floating bridge to be built over tidal water and forming a major link in a public transportation system.

Presently there are no national codes nor standards covering the design and construction of a floating bridge. It was necessary to formulate a set of design criteria for this project to ensure that the structure would:

(1) Perform adequately during normal storm conditions.

(2) Be comfortable to travel on during normal storm conditions.

(3) Safely sustain extreme storm conditions without appreciable damage and without sinking.

(4) Safeguard against progressive collapse; and

(5) Require only minimal maintenance and withstand the detrimental effects of the environment.

A continuous concrete floating pontoon bridge, post-tensioned in threedirections, met the above objectives.

This project consists of 12 post-tensioned concrete pontoons 360 feet long, 60 feet wide and 18 feet high. Each pontoon consists of 40 precast elements and cast-in-place interconnecting pours. Each precast element is post-tensioned vertically. The precast elements together with the cast-in-place pours are post-tensioned longitudinally and transversely to complete a watertight pontoon. The pontoons are then joined together at the job site by a system of longitudinal post-tensioning. **Fig.** 7.74 shows a pontoon being placed in position.

Construction began in January, 1981 and the completed bridge ***(Fig.*** 7.75) was opened to traffic in October, 1982.

Fig. 1.74 Pontoon Placement. Hood Canal Floating Bridge, Washington

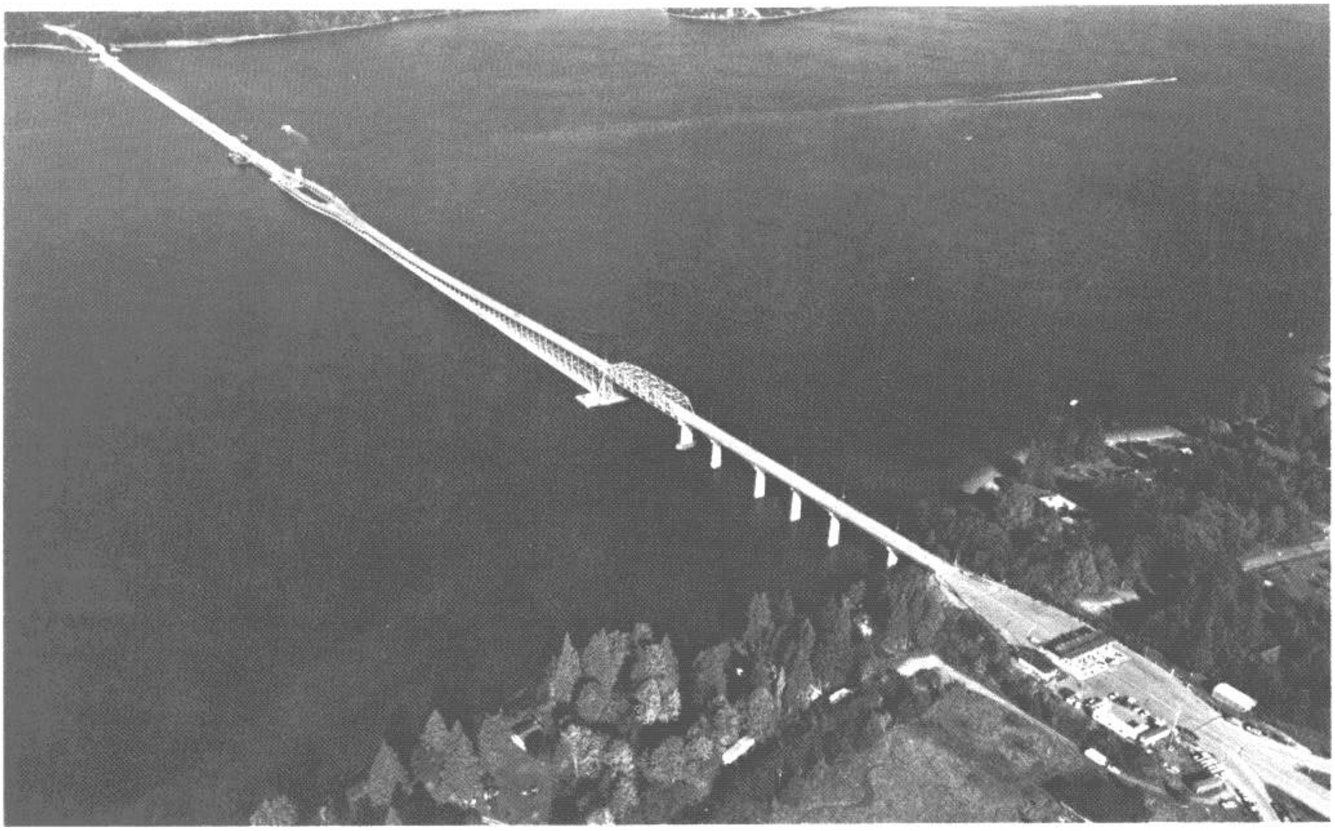


Fig. 1 75 Hood Canal Floating Bridge, Construction of West Section in Progress at Far Shore.

1.14.3 Calgary Olympic Saddledome - Coliseum
Calgary, Alberta, Canada

Representative of many monumental structures utilizing post-tensioning, the Calgary Olympic Saddledome-Coliseum, shown under construction in Fig. 7.76 and following completion in Fig. 7.77, is believed to have the largest span concrete roof in the world. The coliseum is designed to accommodate approximately 20,000 spectators for ice shows, rock concerts or other spectacles, or 17,000 seats for hockey. The building is also suitable for large trade shows, particularly those with large and heavy exhibits. The height of the building above event level ranges from 69 to 133 ft., and the roof span between high points is 444 ft.

Post-tensioning was used in the following elements of the structure:

1. Caisson piles subjected to tensile vertical forces. The advantage of this procedure was to control cracking and subsequent deterioration.
2. Post-tensioning was used in main frames supporting bleachers, floors and ring beam. In view of the fact that the superstructure was virtually entirely prefabricated, post-tensioning was used to provide complete continuity of the structure which, in this instance, is subjected to substantial forces.
3. The roof was post-tensioned in each direction with unbonded tendons. Post-tensioning was introduced to control variable tensile stresses during the life of the roof, thus controlling cracking of the main cable encasement and hence reducing the risk of deterioration of the roof covering resulting from otherwise excessive cracking of the roof structure.
4. Post-tensioning was used vertically in A-frames to control the lateral movements of the roof. In spite of the enormous forces involved, it was possible to use relatively slender prefabricated members and still achieve the structural requirements.
5. The coliseum is believed to incorporate the first post-tensioned ice slab in existence. Post-tensioning was used to control cracking under extensive changes in temperature, and to thereby eliminate possible crack related deterioration of refrigeration piping.

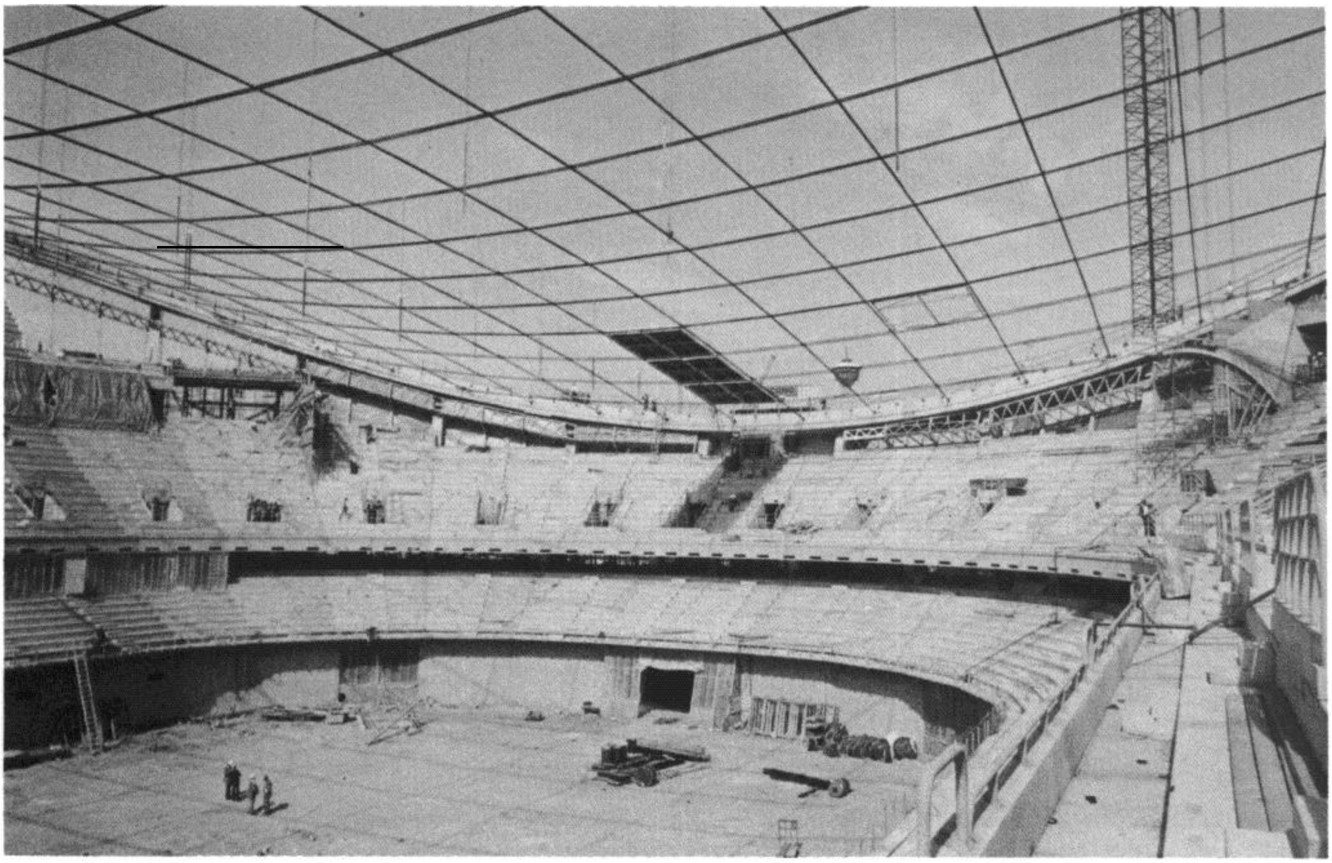


Fig. 1.76 — Construction View, Calgary Olympic Saddledome-Coliseum

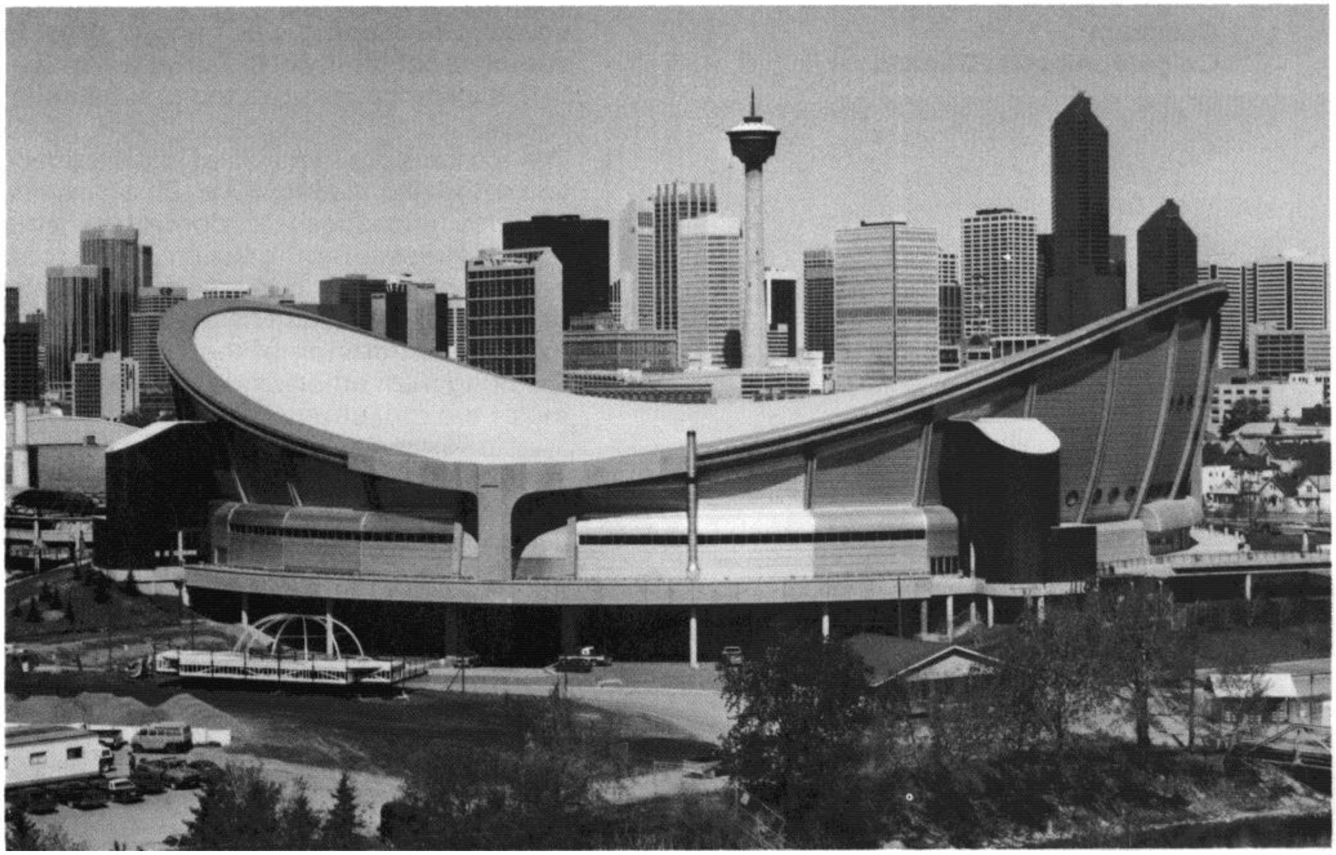


Fig. 1.77 — Completed View Calgary Olympic Saddledome-Coliseum

Chapter 2

Post-Tensioning Systems

2.1 GENERAL

Post-tensioning systems may conveniently be divided into three categories depending on whether the stressing tendon is wire, strand or bar. The discussion in this Section is based on these categories and is intended to provide a general view of the various available systems. Design and detailing data for most of the commercially available systems is presented in Section 2.2.

Post-Tensioned construction is classified as “bonded” or "unbonded" depending on whether the tendon ducts are filled with grout after stressing (bonded), or whether the tendons are greased and plastic covered (unbonded). Use of unbonded construction eliminates the time and cost involved in grouting which becomes an important economic factor in applications such as floor slabs of apartment buildings which usually contain a large number of small tendons. Bonded tendons have structural advantages which are more important for beams and primary structural members. Such members usually utilize a small number of relativelly large tendons, hence the grouting costs are less significant. For unbonded construction, single seven wire strand tendons are normally used. Larger capacity multiple strand, wire or bar tendons are usually grouted. The tendon anchorages shown in this Section are representative of the major variations in types of anchorage for wire, strand, and bar tendons supplied by the various fabricators.

Wherever the term “maximum effective prestressing force” is’ used in this section, the force was calculated as:

0.6 tendon area x minimum guaranteed ultimate tensile strength.

This force is used in this discussion only to provide an indication of the forces available in a single tendon. In some cases, it may not be possible to achieve this force throughout the tendon length in an actual structure due to friction losses and the magnitude of long-term prestress losses.

2.1.1 Wire Systems

Wire post-tensioning systems utilize tendons made up of high tensile wires of 0.250 inch diameter. The minimum guaranteed ultimate strength of the’wires is 240,000 psi. Maximum effective prestressing forces available in a single wire tendon range from two wires at 14.1 kips to 208 wires at 1466 kips. Tendons with more than 60 wires are primarily used in nuclear containment vessels or as rock anchors.

fig. 2.7 shows a button-head anchorage for a large tendon for a rock anchor application. The threaded hole in the center of the anchorhead is used.for a threaded pull-rod attachment to the stressing jack. The 0.375 inch diameter button-heads, which provide anchorage for the individual wires, are formed by a cold-upsetting process. Tendons of this type are usually cut to the exact length and button-headed in the shop. However, equipment is available for field btton-heading for those applications where it is not practicable to prefabricate the tendons to an exact length.

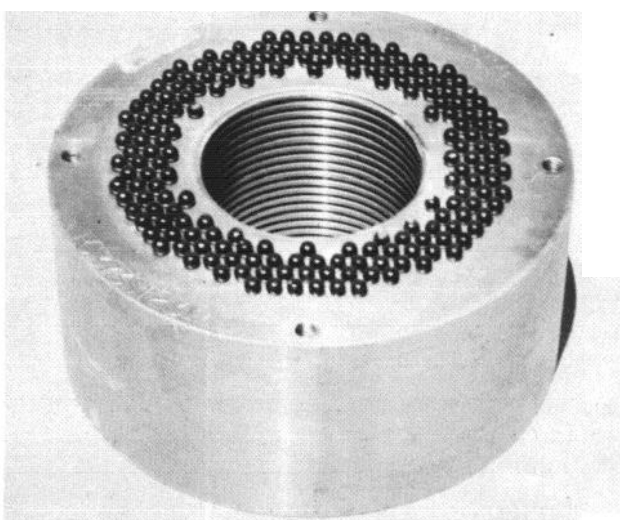


Fig. 2.1 — Button-head anchorage.

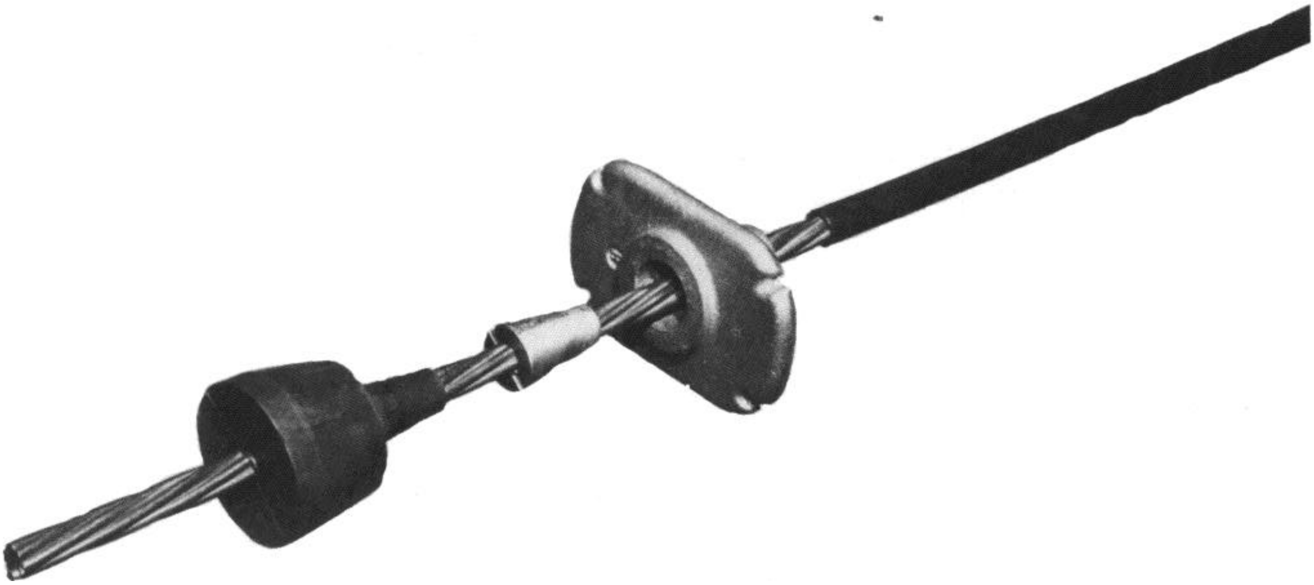


Fig. 2.2 — Conical wedge anchorage for a single-strand unbonded tendon

2.1.2 Strand Systems

Strand systems are based on use of 3/8 in., 7/16 in., 1/2 in., or 0.6 in. diameter 270 K 7-wire strand. Strand systems provide maximum effective post-tensioning forces in a single tendon ranging from a single 3/8 in. diameter 270 K strand at 13.8 kips to 55 - 1/2 in. diameter strands at 1,364 kips.

Anchorages for strand systems utilize the wedge principle. A view of a single strand unbonded tendon anchorage is shown in Fig. 2.2. The anchorage consists of a cast wedge plate in which the tendon is gripped by a two-piece wedge. After the concrete has obtained the necessary strength, the strand is stressed and the conical wedge grippers are inserted around the strand in a conical hole in the bearing plate to provide anchorage. The black rubber element to the left of the wedge in the figure is used to provide a block-out in the concrete to allow for stressing the tendon.

An anchorage for a grouted 12-strand tendon is shown in fig. 2.3 in this case, each strand is anchored by a two-piece conical wedge. The larger strand tendons (containing 31 to 55 1/2 in. diameter 270k strands) presented in various subsections of Section 2.2 are primarily used for heavy construction applications such as long span bridges, nuclear containment vessels, and rock anchors.

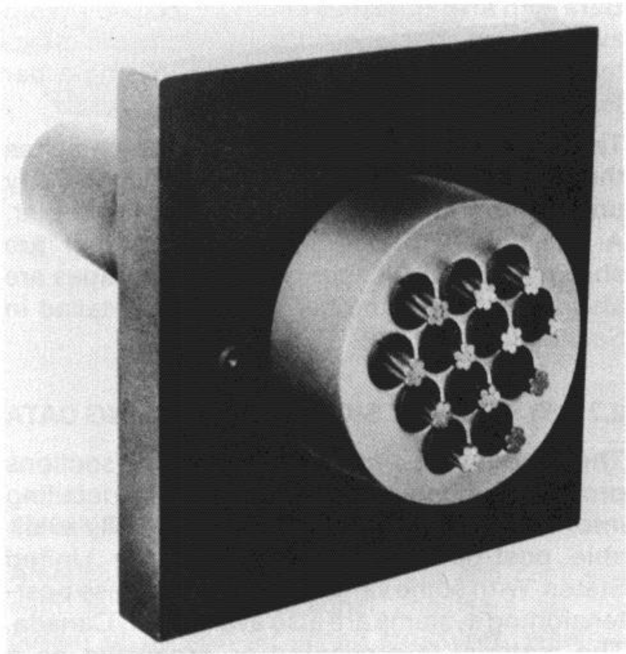


Fig. 2.3 — 12-strand tendon anchorage, stressing end, grouted.

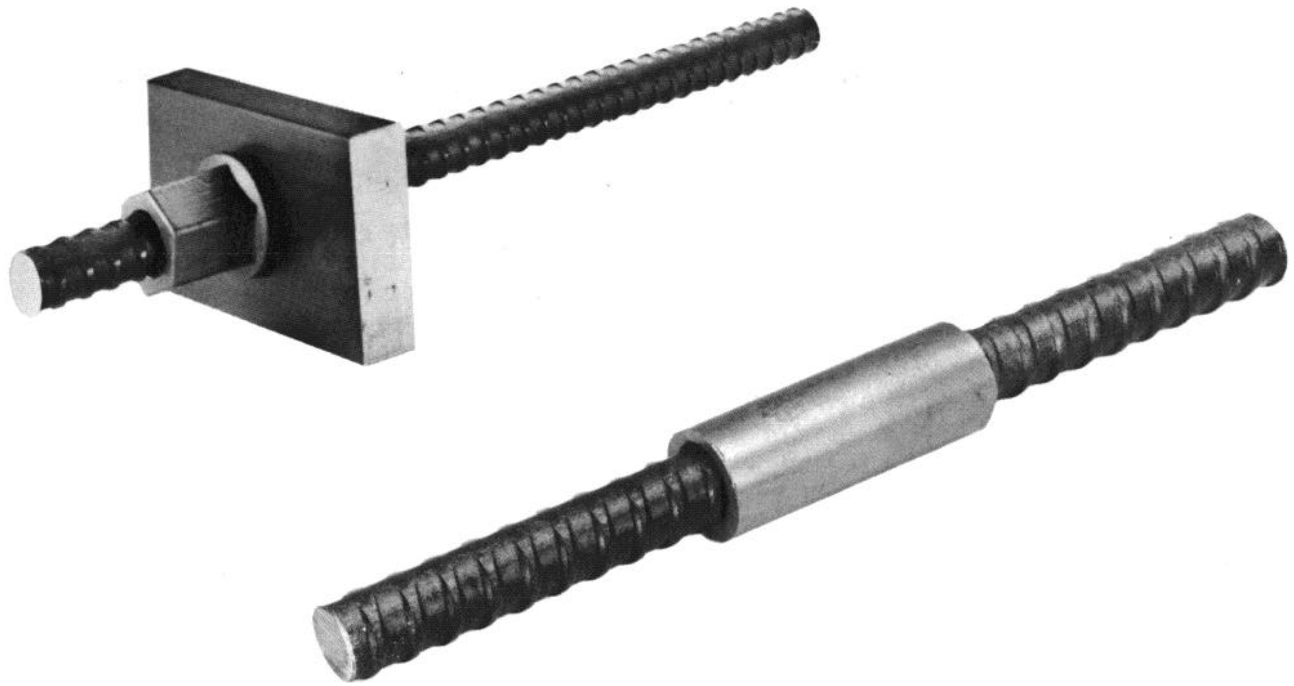


Fig. 2.4 — Threadbar system anchorage and coupler.

2.1.3 Bar System

The bar post-tensioning system available uses bars with an ultimate strength of 150,000 psi, and with diameters ranging from 5/8 in. to 1-3/8 in. Bars with an ultimate strength of 160,000 psi are available on special order. The maximum effective prestressing force available in single bar tendons ranges from 17 to 143 kips.

The bar system has deformations which act as threads and make it possible to cut the bar at any point and screw on either an anchor or a coupler. A bar tendon anchorage and a coupler are shown in *Fig.* 2.4. Other types of achorages are also available for the bar system as detailed in Section 2.2.7.

2.2 SYSTEMS DESIGN AND DETAILING DATA

The material presented in the following sections provides comprehensive design and detailing information for most of the commercially available post-tensioning systems in the United States. With some variations, most of these post-tensioning systems are also available in Canada. The material is presented or organized on a company basis rather than on the basis of system type (wire, strand or bars). Some of the companies supply a variety of systems utilizing different tendon materials.

Within each of the systems described in this section there may be new anchorages under development or existing anchorage details that are not included in the data presented. In the latter case, the anchorages are usually for special purposes, or are local or regional variations of the basic details described below. In case of a project with post-tensioning requirements that do not appear to be satisfied by the systems information shown, the post-tensioning companies may be able to provide specialized or modified post-tensioning materials to meet job requirements.

In addition to the systems for which design and detailing information are presented in this section, there are some other post-tensioning systems in use in the United States and Canada. Companies distributing such systems include Advanced Construction Enterprises, Inc.; Concrete Construction Systems, Inc.; Inryco, Inc.; Post-Tensioned Associates, Inc.; Post-Tensioned Structures, Inc.; Post-Tension Systems, Inc.; Pre-Stress Concrete, Inc.; RPS Cable Corporation; Seneca Construction Systems; Suncoast Postension Corporation; and Western Concrete Structures Company, Inc.

The addresses and telephone numbers of offices of companies whose systems data are presented in this section are listed in Section 2.2.18.

2.2.1 AMERICAN CABLE COMPANY, INC.

American Cable Co., Inc. manufactures and sells a complete line of monostrand hardware for unbonded tendons to the building industry. All hardware has been tested and meets or exceeds all'requirements of the Post-Tensioning Institute Guide Specifications for Post-Tensioning Materials.

Monostrand hardware and equipment available is as follow:

1. Extruded 3/8", 7/16", 1/2" and .6" — wire 270K cable
2. Anchors and wedges
3. Chairs and pocketformers
4. Jack jaws and splice chucks
5. Hydraulic pumps and rams sold and serviced or leased.

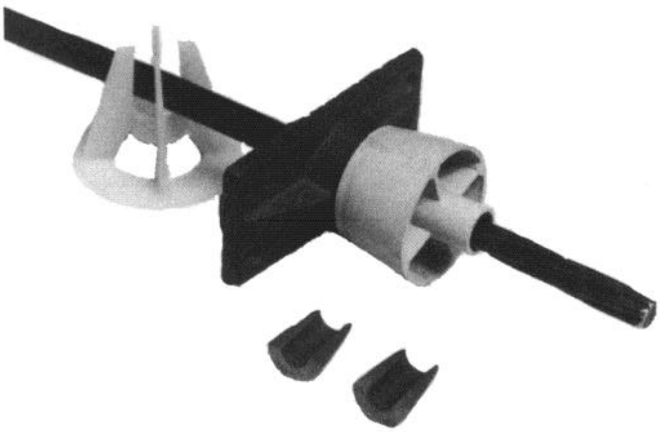


Fig. 1. Assembled post-tensioning system.

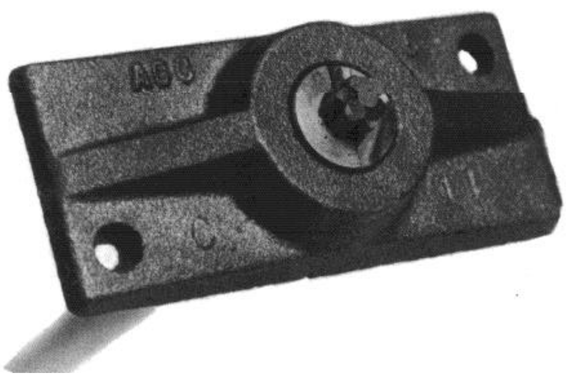


Fig. 2. Monostrand anchor with seated wedges.

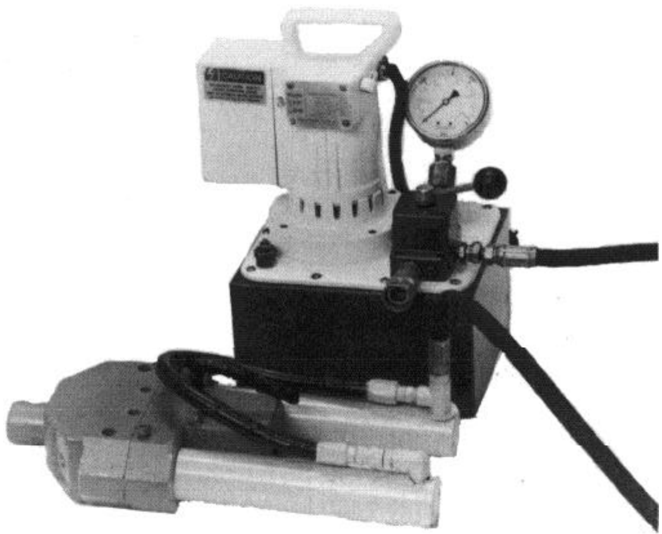


Fig. 3. Hydraulic pump and stressing ram.

INSTALLATION SERVICES

American Cable Co., Inc. has extensive experience in the post-tensioning of concrete slabs on grade and slabs on piling. These post-tensioned foundations have been used for tennis courts, single family homes, apartment houses, metal buildings, shopping centers, office buildings and parking areas. Several of the installations are shown on the following page.

ENGINEERING SERVICES

American Cable Co., Inc. employs and contracts with registered professional engineers for the design of all post-tensioned structures, whether slab on ground or elevated structures. Our engineers will work with your architect and/or contractor to determine the economic and structural advantages of using a post-tensioned structural system on your project.

TYPICAL PROJECTS UTILIZING AMERICAN CABLE COMPANY SYSTEMS

TENNIS COURTS

Most of the concrete tennis courts in Louisiana are now being post-tensioned. Contractors that specialize in tennis court construction have found that post-tensioning minimizes cracking, eliminates expansion joints and decreases water penetration. The post-tensioned slab compares favorably in cost with a reinforced slab and the time of installation is reduced.

Fig. 4. Tennis Courts.

SINGLE FAMILY HOMES

Post-tensioned slabs for single family homes are very common in Texas, Louisiana and parts of Mississippi. In some towns, 95% of all new slabs are post-tensioned. It has been found that where conditions exist that require perimeter and grade beams to be reinforced with #5 and #4 rebars, respectively, and where 6 x 6 x 6 mesh is required, a post-tensioned slab is generally a more economical and structurally superior alternative.

In the New Orleans area where piling is required, a post-tensioned design is generally more economical.

Fig. 5. Single Family Homes

APARTMENTS AND CONDOMINIUMS

The foundation slabs for apartments, condominiums and shopping centers require more conventional reinforcement than single family homes. It has been found that the heavier the conventional design, the more economical the post-tensioning alternative becomes. It is common in the New Orleans area to save as much as $0.75 per square foot with the post-tensioned design for these types of structures.

Fig. 6. New Orleans Apartments.

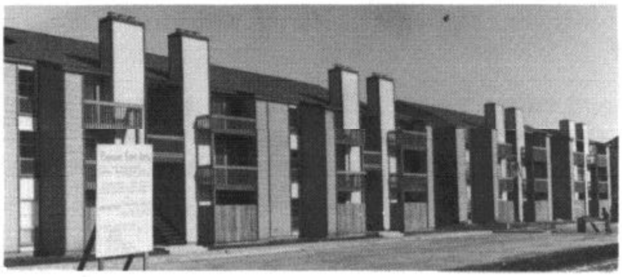


Fig. 7. Condominiums.

Fig. 8. Shopping Centers.

METAL BUILDINGS AND WAREHOUSES

An increasing number of contractors and owners are recognizing the economic and structural advantages of using post-tensioning in warehouse slabs. Typically a 6" thick slab, conventionally reinforced with mesh and #5 bars, can be replaced with a 4 to 4½" post-tensioned slab that is structurally superior for crack control and load carrying capability. The total savings with post-tensioning, considering the steel elimination and concrete reduction, makes the post-tenioned alternate very attractive economically.

2.2.2 AMSYSCO, INC.

AMSYSCO, Inc. is a supplier of post tensioning systems and related services. The modern manufacturing plant and general offices are housed in the new brick and masonary building shown above within a 25 minute drive from downtown Chicago. The AMSYSCO material, services and applications are listed below.

A. To General Contractors:

- 270 ksi, 7 wire strand as per ASTM-A416 cut to length
- shop drawings, stressing calculations and mill test reports
- Stressing jacks and hydraulic pumps with calibration chart
- Support chairs for the tendons
- Technical field service
- Consultations to develop convenient and economical pour sequences
- On time delivery of material to the jobsite

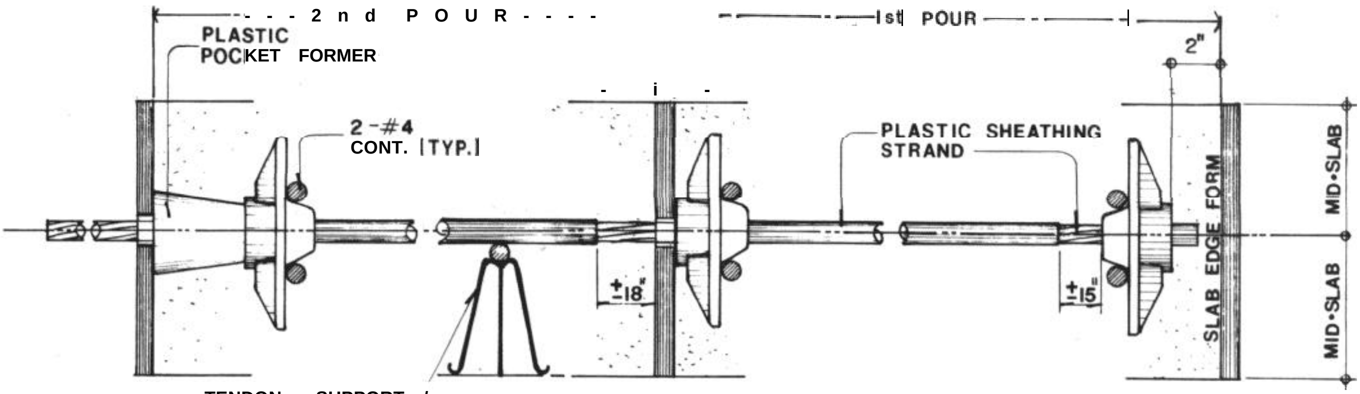
B. To Engineers, Architects and General Contractors

- Guaranteed proposal on DESIGN-BID projects
- Design assistance
- Expertise in developing the most economical concrete frames
- Cost estimates and guide specifications

C. AMSYSCO Applications

- High rise office and apartment structures
- Multi level parking garages
- Water tanks, silos and other storage facilities
- Industrial, residential construction and slab on grade
- Schools, gymnasiums, pavements
- Special applications

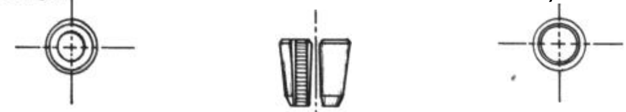
THE AMSYSCO POST -TENSIONING SYSTEM



STRESSING END ANCHOR

INTERMEDIATE ANCHOR

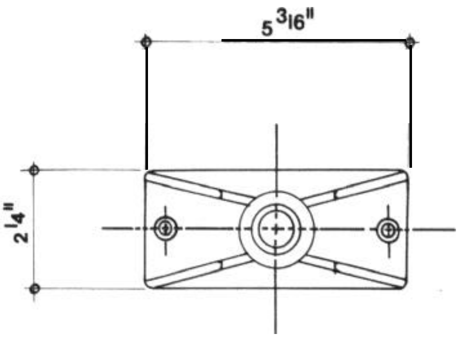
DEAD END ANCHOR,



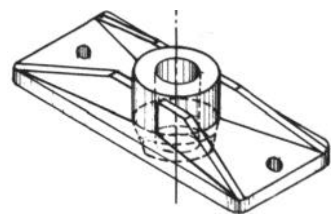
TOP VIEW WEDGE

SIDE' VIEW WEDGE

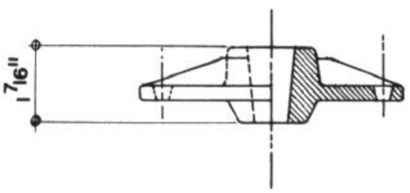
BOTTOM VIEW WEDGE



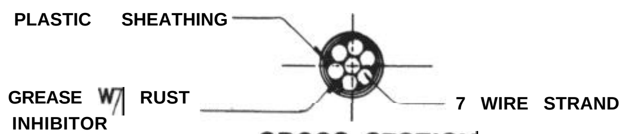
TOP VIEW-ANCHOR



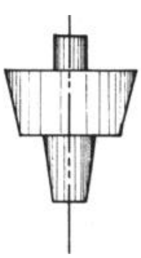
ISOMETRIC VIEW -ANCHOR



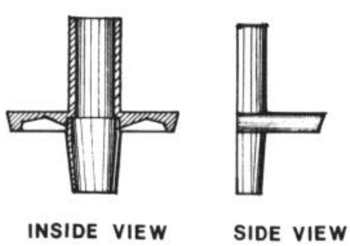
SIDE VIEW -ANCHOR



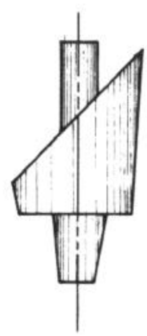
CROSS SECTION TENDON



REGULAR POCKET FORMER



SPLIT POCKET FORMER



45° POCKET FORMER

AMSYSCO Monostrand Systems

The AMSYSCO monostrand system utilizes either 0.5 inch dia. or 0.6 inch dia. 7 wire strand manufactured as per ASTM-A416. The steel is either low steel relaxation or stress relieved strand. The strand is coated with rust inhibiting grease prior to being encased in a durable plastic sheathing. Dead end and intermediate anchors are attached to the tendons in the plant. The tendons are cut to length, coiled, tagged and color coded for easy identification prior to shipping to the jobsite. Strict quality control is used in the plant. The stressing jacks and hydraulic pumps are properly calibrated in our plant. Either spring return or hydraulic power seating type stressing jacks can be supplied.

TECHNICAL DATA

1. **Strand**

Nominal diameter	0.5 inch	0.6 inch
Modulus of elasticity (E)	**28** x 10^6 psi	**28** x 10^6 psi
Ultimate strength	41.3 kips	58.6 kips
Maxm. force at stressing	33 kips	46.9 kips

2. **Anchor**
   Length = 5 inches to 5.1875 inches
   Width = 2.25 inches
   Depth = I.4375 inches
   Cross section area = 11.67 sq. inches
   Minimum strength of concrete at stressing = 2100 psi (Recommend 3000 psi)

3. **Wedges**
   The two piece wedges are made of high strength steel and are properly heat treated.

4. **Grease**
   The grease is compounded from highly refined petroleum oils and long chain metallic hydro-carbons. It is designed to provide a rust protective coating that is both flexible and stable from below 0°F (-17.8°C) to 300°F (148.9°C).

5. **Plastic sheathing**
   The 40 to 42ml. thick plastic sheathing formed over the strand is durable and withstands handling during transportation and construction.

6. **Stressing jack - hydraulic return type**
   The jack (CBS) is of one-piece construction from custom forged aircraft industry quality steel. It weighs only 38 pounds with 3 inch standard nose. The stroke of the jack is 8.5 inches.

7. **Hydraulic pump**
   The hydraulic pump (OTC) is equipped with 10,000 psi pump, universal motor (1 1/8 h.p., 12,000 r.p.m., 115 volt, 60/50 cycle A. C. single phase, draws 27 amps at full load).

AMSYSCO

Recent Projects

Project & Location	Description	Structural Engineer
North Loop transp. Center Chicago	18 story office Building over 13 story parking 1,250,000 sq. ft.	Skidmore, Owings & Merrill Chicago
Sinclair Community College Dayton	3 story parking & pedestrian bridge 600,000 sq. ft.	Richard Shell Assoc. Dayton
Toledo General Hospital Toledo, Ohio	7 story parking garage 550,000 sq. ft.	S.S.O.E. Toledo
Webstrand Office Building Toledo, Ohio	9 story office 3 story parking 500,000 sq. ft.	S.S.O.E. Toledo
St. Louis Center Garage St. Louis	6 story parking 550,000 sq. ft.	S.S.E. Inc. St. Louis
Tampa Rehabilitation Center Tampa, Florida	6 story parking 460,000 sq. ft.	Reynolds, Smith & Hills
Miami International Airport Miami, Florida	6 story parking 600,000 sq. ft.	Professional Engrs. Team Miami
Barnes Hospital St. Louis	3 story underground parking 400,000 sq. ft.	Becker, Becker, Pannell St. Louis
Conrad Hilton Hotel Chicago	7 story parking 250,000 sq. ft.	Christefanos Assoc. Chicago
2 East 8th Apartment Building Chicago	26 story apartment 500,000 sq. ft.	Kolbjorn Saether Chicago
Hill View Apartments Hoffman Estates, II.	7 story twin towers 250,000 sq. ft.	Shah Engin. Chicago
Athletic Facility Stadium Miami, Ohio	Multistrand system in beams	Osborn Co. Cleveland, Ohio
Hermitage Apartment Building Chicago, Illinois	29 floors over parking 400,000 sq. ft.	Beer Gorski & Graff Chicago, Illinois

2.2.3 CCL DIVISION NICHOLSON CONSTRUCTION COMPANY

NCC/CCL MULTISTRAND POST-TENSIONING SYSTEMS

CCL Systems Limited is one of the largest manufacturers of prestressing equipment in the world, marketing a wide range of equipment and systems for both pre- and post-tensioning. The company is represented in over 70 countries and is an international organization with over 25 years experience in prestressed concrete.

Nicholson Construction Company has 30 years of Specialty Foundation Construction experience and is recognized as a pioneer in the design and construction of Prestressed Rock and Soil Anchors in the United States. Nicholson is a full service Design/Construct(Turnkey) Company capable of taking a project from Engineering concept to Project Completion. Services are provided across a broad range of project needs including:

- Ground anchors
- Permanently anchored walls and marine bulkheads
- Excavation support
- Post-tensioning systems
- Mine subsidence prevention and repair
- Dam stabllization
- Piling and drilled shafts
- Pin piles
- Slurry walls and trenches
- Special directional drilling
- Insert Wall Systems(TM) & soil nailing
- Underpinning
- Grouting

Nicholson's advancement in the construction industry is due to creative planning and design, utilization of state-of-the-art technology, development of efficient and cost effective engineering and construction solutions to satisfy project requirements, along with the foresight to develop and utilize new technologies in the changing construction marketplace.

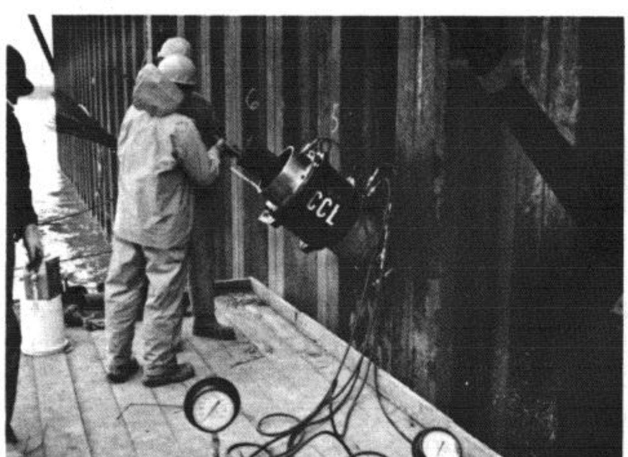
Nicholson has used all types of prestressing steel in the construction of anchors and unique foundation systems for buildings, bridges, dams and retaining walls for more than 15 years. With this expertise and specialized equipment available in house, the decision was made to expand into the prestressed construction field.

NCCKCL provides combined expertise in the areas of construction techniques, equipment and material to the general contractor. A competent staff of engineers is available to assist designers through all stages of construction planning.

Products are continuously being developed based on high quality components produced in the facilities of NCC/CCL to meet our own rigid specifications and inspection, as well as all applicable domestic and international test requirements.

The NCCKCL systems are designed to offer the engineer a complete range of anchorages and prestressing forces to meet the most difficult requirements, while providing economy and trouble-free site operation.

NCCKCL is confident in the integrity of their products and is pleased to introduce to you the NCCKCL Post-Tensioning Systems for the United States.



Stressing 22 strand anchor; City of St Louis

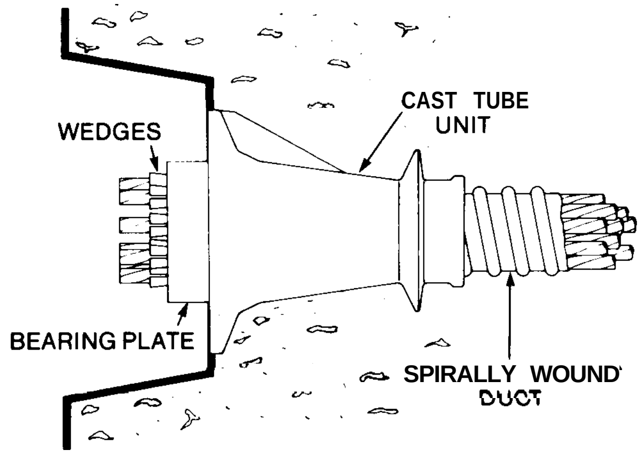
Standard Anchorages

1.0 This anchorage is used at the stressing end of a tendon and can be used as a "passive" anchorage, should it be exposed, to allow the insertion of wedges.

1.1 NCC/CCL Standard Anchorages are sized to accept tendons of 4, 7, 12, 19 and 27 strands of 0.5 inch or 0.6 inch diameter. Designs for other sizes are available, call our engineering department for further details. The complete standard anchorage assembly consists of a cast tube unit and an anchor plate with individual tapered holes in which the strands are anchored by hardened steel wedges, The cast tube unit serves two purposes. First, it acts as a bearing plate, transferring the prestress force from the anchor plate to concrete and second, it acts as a transition piece which gides strands from the anchor plate into the tendon duct.

1.2 Intermediate numbers of strands can be accommodated if desired, by omitting wedges from the anchor plate in a uniform pattern. For special conditions when standard anchorage units cannot be incorporated, consult our engineering department.

1.3 This anchorage should not be used as a buried dead end as the wedges can be dislodged due to normal concreting vibrations. Special anchorages have been designed for this case.



1.4 Provision is made in the tube unit in the form of a tapped hole for grout injection when stressing is completed. This grout entry point should be situated at the top when the tube units are fixed to the formwork.

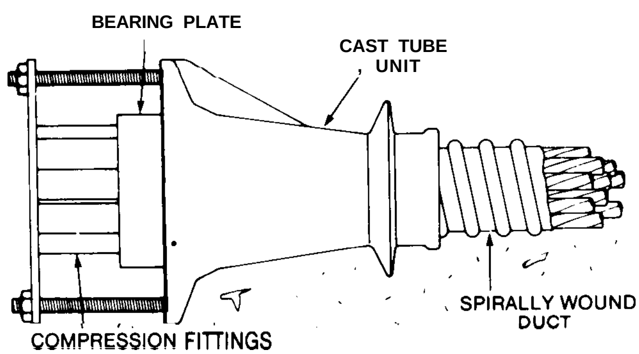
1.5 Four standard metric tapped holes are drilled in the face of the tube unit in order to facilitate fixing to the formwork, location of the jack bearing ring and the fitting of grout caps.

Dead End Anchorages

2.0 NCC/CCL Dead End Anchorages are used where the end of a prestressing cable is buried in concrete or inaccessible during the stressing of the tendon.

2.1 The dead end anchorage can accept the same strand configurations as the standard anchorage and uses the same tube unit to guide the strands. The strand passes through a parallel hole bearing plate and is anchored by means of a compression fitting which is swaged onto the strand ensuring a positive anchor. The fittings are swaged to the strand using an extrusion rig activated by a hydraulic pump. A retaining plate is bolted to studs in the four tapped holes of the tube unit to ensure that the compression fittings bear evenly on the bearing plate.

2.2 The tube unit should be fixed with the grout exit point at the top to prevent air being trapped in the duct during grouting. A vent pipe should be attached to permit proper tendon grouting.



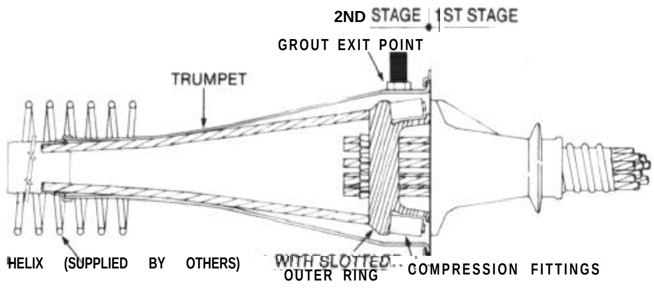
2.3 In cases where prestress is not required up to the end of a tendon and where lateral space permits, some economy may be achieved by employing bonded or semi-bonded dead-end anchorages on certain sizes of tendon. For further information consult NCC/CCL.

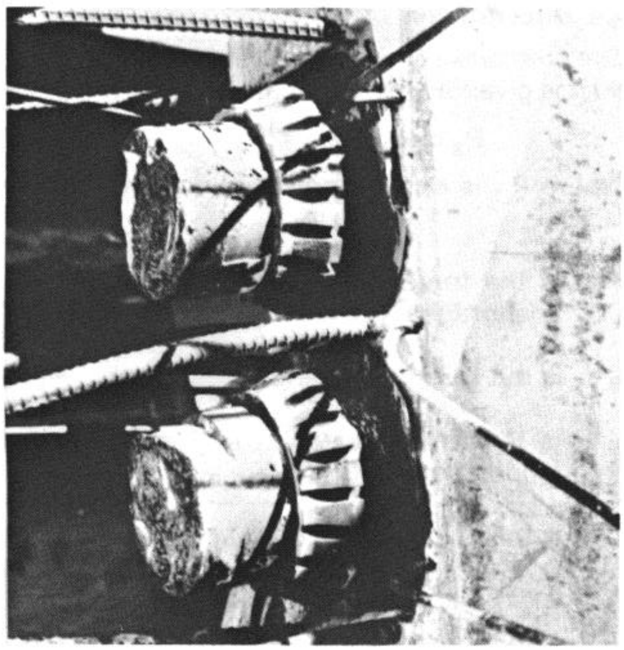
Anchorage Coupling

3.0 The NCC/CCL anchorage coupling facilitates the continuity of a previously tensioned tendon, thusallowing uninterrupted post-tensioning forces across construction joints.

3.1 In continuous bridge deck construction, it may be necessary to extend prestressing cables as construction proceeds. The NCC/CCL anchorage coupling is available in all 12, 19 and 27 strand configurations.

3.2 The first stage of stressing is carried out in the same way as with the Standard Anchorage, except that at the continuous end the anchor plate is replaced by a one-piece coupler which incorporates an outer ring to accommodate the compression fitting swaged to the strands of the second stage cable and taper holes in the same configuration as the standard anchor bearing plate. When first stage stressing and grouting is complete, the compression fittings are swaged to the strand using the extrusion equipment and the strands are fitted into the slots of the outer ring. The strand is deviated through a shaped trumpet which also prevents the ingress of concrete. The trumpet contains a grout exit point which should be placed at the top to prevent any air being trapped during grouting, and the small end of the trumpet should be securely taped to the duct.





First stage of coupling after grouting with grout cap removed.

General Design Information

4.0 The following representative design data have been developed in accordance with ASTM A-416 Standards, British Standard 4447: The Testing of Prestressing Anchorages, F.I.P. Recommendations for Acceptance and Application of Post-Tensioning Systems, and A.C.I. Standard 318.

4.1 Concrete Strength

All NCC/CCL anchorages are designed for a minimum concrete cylinder strength at transfer of 4000 PSI.

Diameter	Guaranteed Ultimate Tensile Strength	Steel Area	Weight	Specified Characteristic breaking load
Inches (mm)	KSI (N/mm^2)	$Inches^2$ (mm^2)	LB/FT (g/m)	KIPS (kN)
0.5 (12.7)	270 (1860)	0.153 (100)	0.525 (785)	41.3 (186)
0.6 (15.2)	270 (1860)	0.215 (139)	0.740 (1100)	58.6 (260)

4.2 Strand Data

The tendon data which is shown on the anchorage data sheets is based on strand conforming to ASTM A-416, latest revision: (All properties are nominal).

Strand to other specifications can usually be accommodated, but reference should be made to NCC/CCL in case different wedge grips should be required.

4.3 Wedge Set

Wedge set in the anchorage during transfer causes loss of prestress. Wedge set in the non-stressing anchorage and in the stressing jack pulling wedges causes an apparent increase in extension but no loss of prestress.

Depending on the care with which wedges are tapped into position,prior to stressing, the average values for all diameters of strand are:

Stressing anchorage—
3/16" (5 mm)

Jack pulling wedges —
3/16" (5 mm)

Non-stressing anchorage —
1/8" to 3/8" (3 to 9 mm)

Check measurements can be made on site if required.

4.4 Duct Friction

The prestressing force at any distance 'x' along a duct is given by the formula:

$$P_x = P_o e^{-(\mu \propto + Kx)}$$

Where P_o is applied prestressing force at the jack

P_x is the force at the distance 'x' from the anchorage

e is the base of Napierian logarithms

μ is the total coefficient of friction of the tendon duct

a is the total angle of deviation, in radians

K is an allowance for lack of alignment or wobble of the duct

Typically, the value of μ varies between 0.20 and 0.25, depending upon thecondition of the tendon and the duct, (surface rust increases friction). Friction may be reduced by applying water-soluble oil which is afterwards removed by flushing the duct with water. K is a factor depending upon thefrequencyof supports; with the duct strongly supported at 3.3'(1 .Om) centers K may be interpolated between .01 per foot (0.0033 per metre) for ducts of 2-7/16" diameter (62 mm) and .006 per foot (.0017 per metre) for ducts of 3-15/16" diameter (100 mm).

Alternatively, lack of alignment may be regarded as additional curvature θ and added to the angle a . Typical values are 0.033 radians per foot (0.010 radians per metre) for ducts of 2-7/16" diameter (62 mm) and 0.016 radians per foot (0.005 radians per metre) for ducts of 3-15/16" diameter (100 mm).

The friction formula then becomes:

$$P_x = P_o e^{-\mu (\propto + \theta x)}$$

4.5 Jack Friction

Pressure gauges used with NCC/CCL jacks are calibrated to read the load actually applied to the tendon and thus no allowance need be made for jack friction. Load cells can be used if required as a secondary check to the applied jack load.

4.6 Grouting

Re-usable grout caps are available to seal the anchorages during grouting. These are bolted to the anchorage castings. For injection, the grout pipe is screwed into the hole provided in the anchorage casting which allows the grout to flow smoothly into the duct. Intermediate vent points should be provided along the duct, particularly at high points. Grout vent saddles are fixed to the duct and a hole punched in the duct. The vent is connected to the surface with a plastic or steel tube. Typically, the grout mix consists of five gallons of water per 94 pound sack of Portland Cement Type I or II. Specific admixtures should be added only as required and directed by the engineer.

4.7 End Block Reinforcement

Steel reinforcement is required around each anchorage to resist the bursting forces which cause tensile stresses in the concrete. The data sheet (Section 5) shows the area of reinforcement required on each axis of the anchorage; it may be provided either in the form of stirrups or a spiral.

Mild steel is preferred although high tensilesteel may be substituted if desired. However, it should be realized that visible surface cracking may occur due to the higher strains at working stresses.

The spacing of the stirrups or the pitch of the spiral must be such that all the steel is placed within a distance of '2a' from the anchorage face (where 'a' is the distance from the center of the anchorage to the nearest edge of concrete or to the center between anchorages, which ever is the lesser).

Where more than one anchorage occurs, the reinforcing cages must be linked together to accommodate the tensile stresses between anchorages; this may be achieved by overlapping rectangular stirrups. In the case of spirals, additional stirrupsare required to encompass all the anchorages.

5.0 Anchorage Details

The information given in Tables 5.1 and 5.2 represent standard NCC/CCL anchorage sizes. Tendons with intermediate numbers of strands are available in the NCC/CCL system by special anchorage adaptation. Designs for special applications can be obtained by contacting our office.

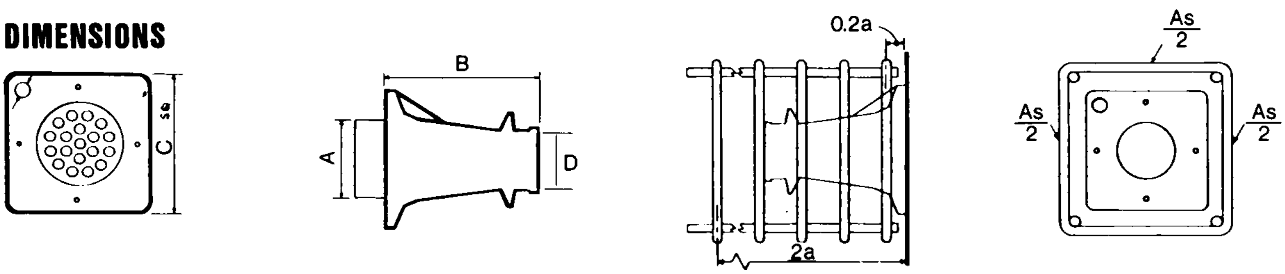
*STRAND DIAMETER	TENDON TYPE	NO. STRANDS	MIN. GUARANTEED ULT STRENGTH fpu As (Kips)	MAX TEMPORARY FORCE 0.8 fpu As (Kips)	TRANSFER FORCE AT 0.7 fpu As (Kips)
0.5"	N/CCL 4.5	4	165.2	132.2	115.6
	7.5		289.1	231.3	202.4
	12.5	1:	495.6	396.5	346.9
	19.5	19	784.7	627.8	549.3
	27.5	27	1115.1	892.1	780.6
	55.5	55	2271.5	1817.2	1590.1
0.6"	N/CCL 4.6	4	234.4	187.5	164.1
	7.6	7	410.2	328.2	287.1
	12.6	12	703.2	562.6	492.2
	19.6	19	1113.4	890.7	779.4
	27.6	27	1582.2	1265.8	1107.5
	37.6	37	2168.2	1734.6	1517.7
	47.6	47	2754.2	2203.4	1927.9

*ASTM A416 Grade 270

Table 5.1

All data is subject to revision as new developments are made.

DIMENSIONS



STRAND DIAMETER	TENDON TYPE	A inches (mm)	B inches (mm)	C inches (mm)	D inches (mm)	As/2 in² (mm²)
0.5"	N/CCL 4.5	3 5/16 (85)	3 1/2 (90)	4 3/4 (120)	1 7/8 (48	0.845 (545)
	7.5	4 1/8 (105)	5 1/4 (133)	6 1/8 (155)	2 1/4 (57	1.479 (954)
	12.5	5 11/16 (145)	6 11/16 (170)	8 1/16 (205)	3 3/16 (81	2.539 (1636)
	19.5	7 1/16 (180)	9 5/8 (245)	9 7/8 (250)	3 3/8 (86	4015 (2590)
	27.5	6 11/16 (220)	10 13/16 (275)	11 13/16 (300)	4 3/16 (106	5.720 (3690)
	*55.5					
0.6"	N/CCL *4.6					
	7.6	4 15/16 (125)	6 5/16 (160)	7 1/2 (190)	2 3/4 (70)	2.108 (1360)
	12.6	7 1/16 (180)	10 (255)	9 7/16 (240)	35116 (85)	3.612 (2330)
	19.6	8 11/16 (220)	10 13/16 (275)	11 13/16 (300)	4 3/16 106)	5.720 (3690)
	27.6*	10 5/8 (270)	18 5/16 (465)	14 (355)	4 5/8 118)	8.127 (5243)
	*37.6					
	*476					

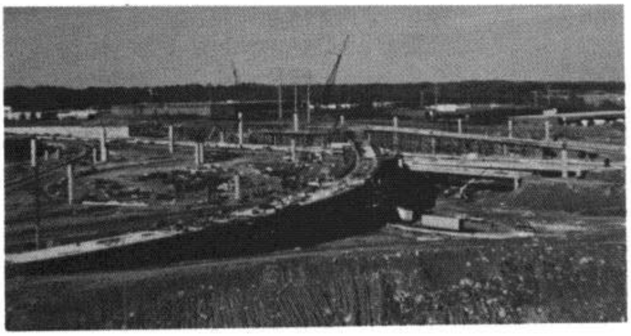
Table 5.2

***Designs for these and other sizes are available. For dimensions and details, contact** our office. All data is subject to revision as new developments are made.

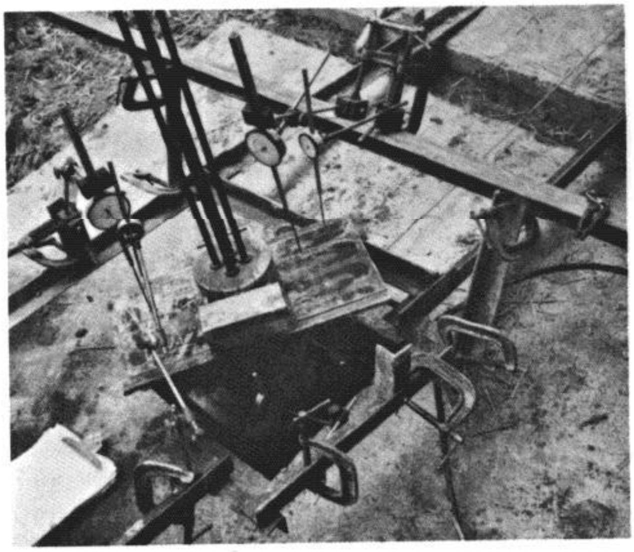
NOTE: Bearing plates can be used in place of cast tube unit if desired.

REPRESENTATIVE NCC/CCL PROJECTS

Barker Dam Stabilization
Battery Park City Financial Center
Mid-Orange Correctional Facility
Wall No. 14, Georgia DOT.
TVA Wilson Dam
I-85/I-285 Interchange, Georgia DOT



I-85/I-285 Interchange, Georgia DOT.



In place creep test of four strand tendon Ohio DOT, Cincinnati

Seaboard Systems RR
Consolidation Coal, Arkwright Mine

Anchor wall Consolidation Coal Arkwright Mine

Wanaque Dam

Stressing test Wanaque Dam, Wanaque, New Jersey

Charleroi/Monessen Bridge
Stage I Light Rail Transit, Pittsburgh

Dookers Hollow Bridge
Jersey Central Power & Light Bulkhead
City of St. Louis Bulkhead
Mt. Lebanon Tunnel
Fort Norfolk Bulkhead
J & L Steel Water Treatment

CCL DIVISION

CCL Division
Nicholson Construction Company

2.2.4 CEC SYSTEMS, INC.

Post-Tensioning Specialists

CEC Systems specializes in post-tensioning. Our exclusive focus is the application of post-tensioning systems in all types of commercial and residential construction.

Since the company was founded in 1977, CEC has handled an average of 65 major projects each year all over the western United States. CEC's post-tensioning experience includes residential slabs on grade, foundation mats, industrial floors, parking structures, rock and soil anchors, tie-backs, and low- and medium-rise commercial buildings-from the simplest to the most complex.

CEC's 'Systems" Approach

The CEC team offers total post-tensioning service, from first concept to final installation. Our extensive design capabilities allow us to start with preliminary specifications and engineer the entire post-tensioning system. Tendons, anchors, and other gear are fabricated in our own yard, and the system is installed at the construction site. CEC Systems can handle every kind of post-tensioning task from beginning to end.

We combine'our engineering capabilities with a thorough understanding of the economics of construction. We estimate costs accurately and quickly, and we are expert at developing ways to employ post-tensioning to avoid cost overruns and increase profitability.

CEC's Services and Capabilities

- Monostrand post-tensioning systems
- Multistrand post-tensioning systems
- Tie-backs and soil anchors
- Stressing equipment
- On-site installation
- Design and engineering of post-tensioning systems
- Cost estimating and redesign to make budget
- Design and fabrication of specialized post-tensioning equipment

Walnut Creek Center No. 1, a CEC project located in Walnut Creek, California. Low-rise office and underground-parking structure using a two-way grid with post-tensioned ductile frame.

Airport Plaza, a CEC project in Concord, California. A medium-rise office building using post-tensioned beams precast and erected on site. Post-tensioning was employed in order to control long-span deflection.

CEC MONOSTRAND SYSTEM

CEC's monostrand post-tensioning system uses a 0.5inch seven-wire tendon with a guaranteed ultimate strength of 41,300 Ibs. Other types and sizes of tendon are available for specific applications.

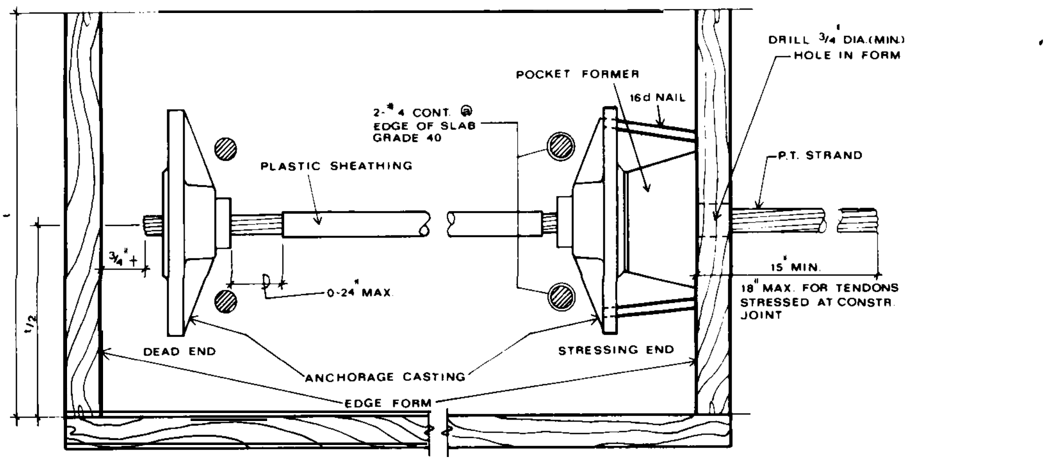
All CEC coated-tendon materials are manufactured in accordance with ASTM A416 and PTI guide specifications for post-tensioning components. First, a viscous rust-inhibiting, low-friction grease is forced around the tendon under pressure, then a tough and durable plastic sheath is extruded onto the tendon. The result is a high-quality unbonded tendon with extremely low friction.

CEC-5 and CEC-B anchors are used for both dead-end and stressing-end anchorages and are approved by the International Conference of Building Officials (ICBO Report No. 3894).

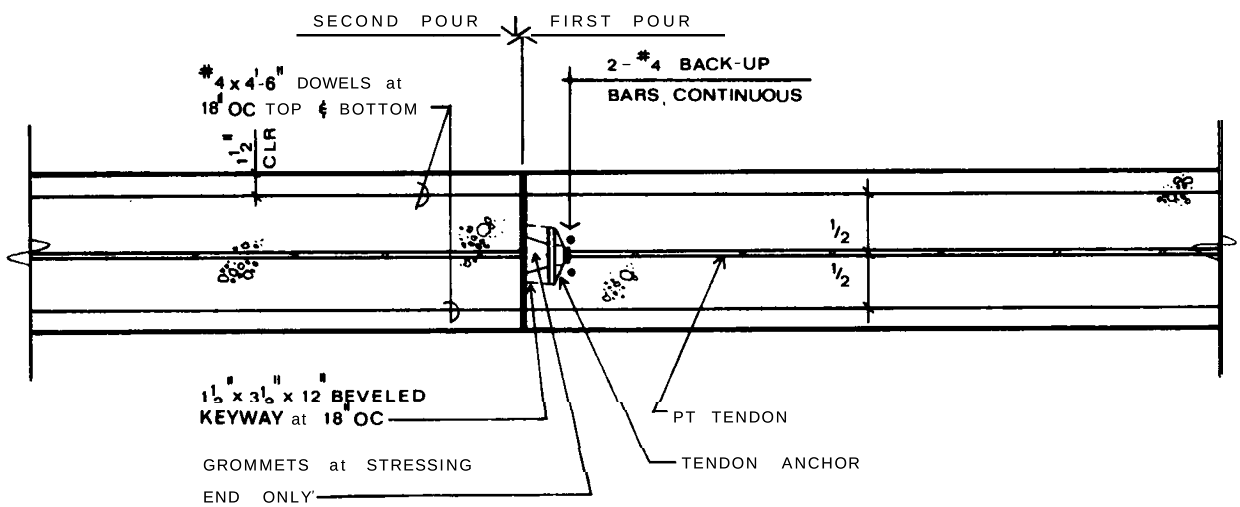
The CEC-5 anchorage consists of a ductile iron casting and a two-piece truncated cone-shaped wedge. The casting has a rectangular shape with the dimensions 2.5 inches by 5 inches by 1.75 inches, yielding a net bearing area of 12.0 square inches. The ductile iron conforms to ASTM A536, Grade 80-65-06. The wedge is made of case-hardened AISI-C12114, Cl215 steel. It is 1 inch in diameter and 1.25 inches long, and the interior is serrated to receive the strand.

The CEC-B anchor is a barrel-type anchor 2 inches in diameter and 1.5 inches long with a cone-shaped hole to receive the wedge. The anchor is malleable iron complying with ASTM A536 Grade 75-50-05 and is used with an ASTM A36 steel bearing plate designed for concrete bearing as required by the code. The same wedge is used with both CEC-B and CEC-5 anchors.

As the schematic shows, the assembled CEC monostrand post-tensioning system consists of preseated dead-end anchor, sheathed tendon, intermediate anchor, stressing-end anchor, gripping wedges, pocket former, and strand clamp. Anchors may be attached to forms by bolts, nails, or wires.

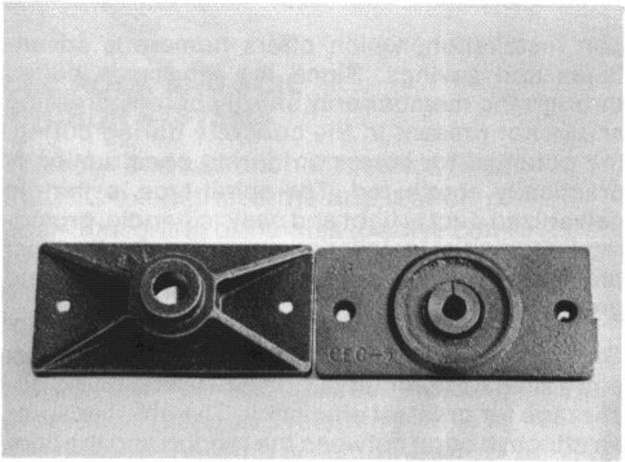


Schematic representation of the CEC monostrand system.

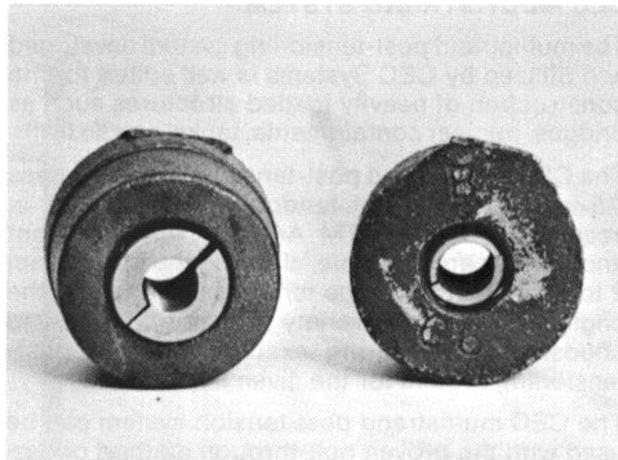


Typical intermediate stressing joint.

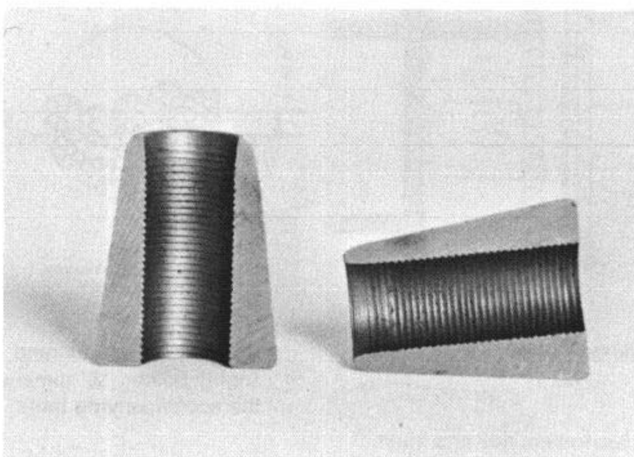
2.2.4 CEC SYSTEMS, INC. — continued



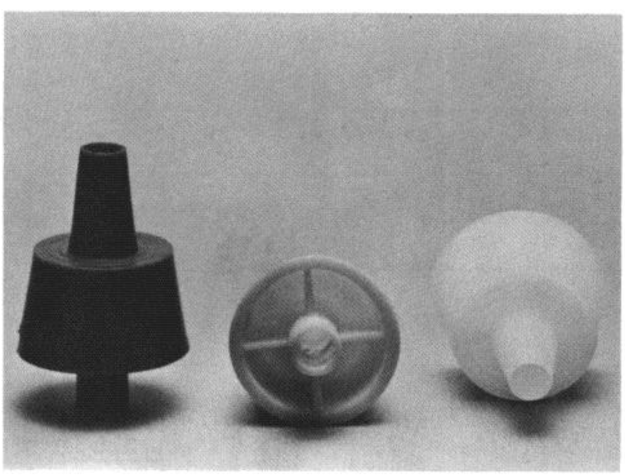
The CEC-5 anchorage, front (right) and back (left) views, with wedges in place.



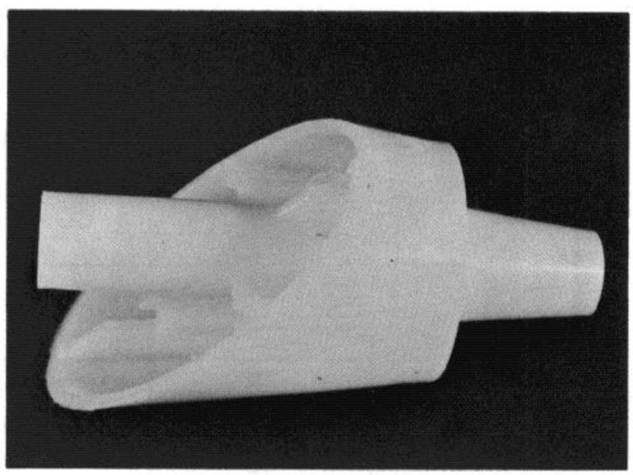
The CEC-B anchorage, front (left) and back (right) views, with wedges in place.



Tendon wedges used for both CEC-5 and CEC-B anchorages.



Standard pocket formers for the CEC monostrand system.



45" pocket former.

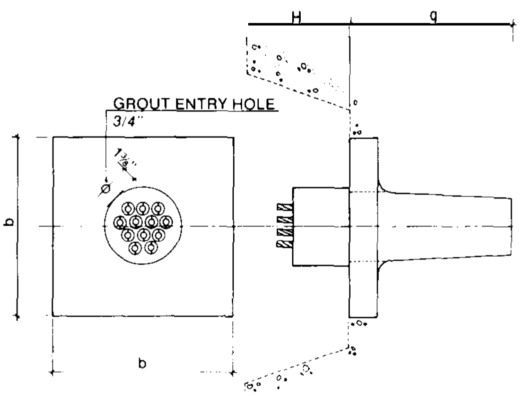
CEC MULTISTRAND SYSTEM

The multistrand post-tensioning system developed and utilized by CEC Systems is well suited for the construction of heavily loaded structures such as bridges, nuclear containments, tanks, and so forth.

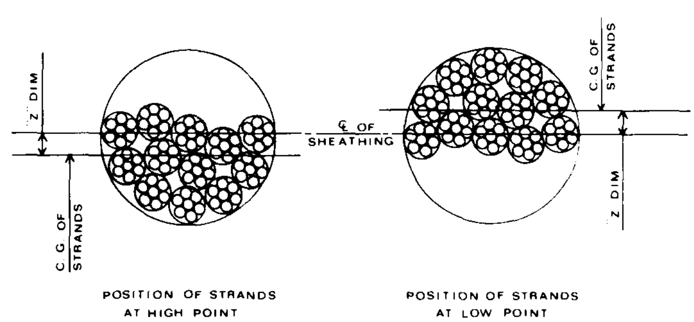
The CEC multistrand post-tensioning system uses 0.5-inch 270K strand tendons manufactured in accordance with ASTM A416. Several different anchorages are available, allowing the use of from 2 to 31 multistrand-type tendons. As a result, the engineer has a wide variety of stressing forces to choose from, ensuring exactly the right post-tensioning system for the given application.

The CEC multistrand post-tension system can be used with the proven pull-through method of tendon installation, which offers numerous advantages and savings. Since the tendon is pulled through the member only shortly before stressing and is not present in the concrete during curing, the potential for corrosion during construction is practically eliminated. The spiral-type, semirigid galvanized duct is light and easy to handle, providing faster initial installation steps. Finally, the duct prevents concrete leakage onto the tendon and greatly reduces friction.

Grouting is performed after stressing via the grout entry hole in the multistrand post-tensioning anchorage for greatest efficiency. The grout ensures an effective bond between the tendon and the concrete and preserves the tendon against corrosion.



Front and side views of the anchorage used in the CEC multistrand system.



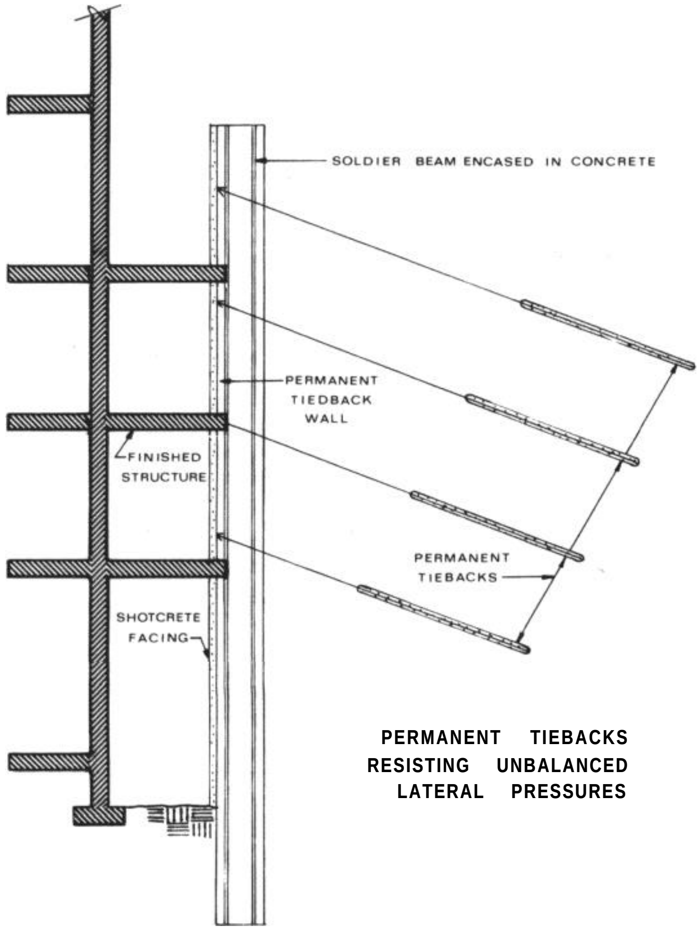
Positions of multistrand tendons at high (left) and low (right) points. "Z" dimension corresponds to the data in the accompanying table.

JACK REFERENCE	NO. OF .5" STRANDS	ANCHORAGE DIMENSIONS				SHEATHING		"Z"
		TRUMPET REF	b	q	H	Ω	Φ	
G-60	3	cc-11	4¾"	4"	5⅛"	1¾"	1¾"	1/2
	4	cc-12	5½"	4"				1/2
G-100	5	cc-13	7"	5½"	5⅛"	2"	2"	5/8
	6							1/2
	7							1/2
G-200	8	cc-14	8¼"	7"	5⅛"	2½"	2"	3/8
	9							3/8
	10	cc-15	9½"	7"	5⅛"	3"	2½"	1/2
	11							3/8
	12							3/8
	13	CC-16	10¼"	9⅞"	5½"	3¼"	3"	5/8
	14							5/8
	15							1/2
	16	CC-21	11½"	11⅞"	5½"	3⅝"	3¼"	5/8
	17							5/8
	18							5/8
	19							5/8

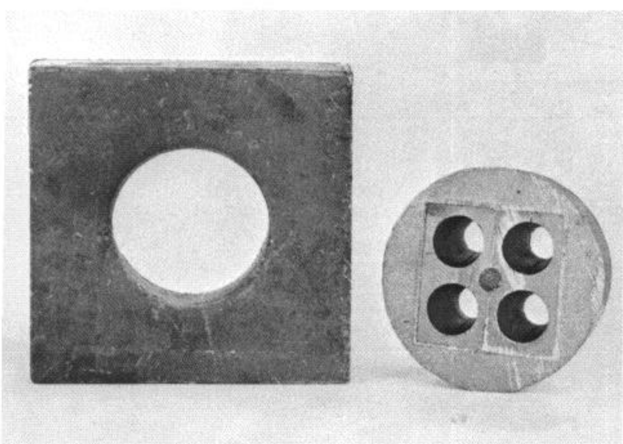
Multistrand system specifications.

CEC ROCK AND SOIL ANCHOR SYSTEMS

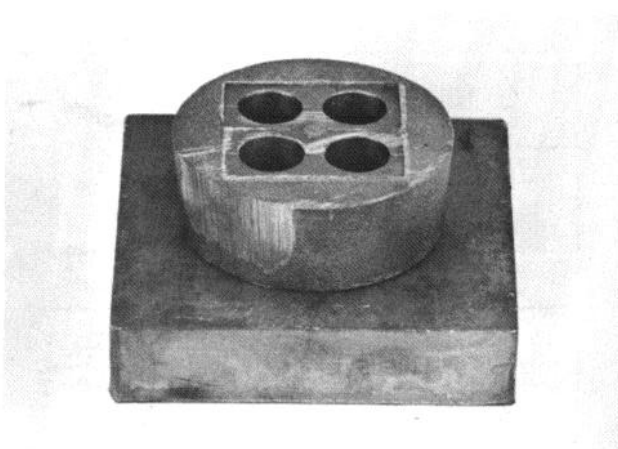
The rock and soil anchor systems used by CEC exemplifies the application of post-tensioning principles to solve construction problems — in this case, those posed by the mechanics of soil and rock. Rock and soil anchors find their most common application in stabilizing slopes and severe grades, holding excavation faces and retaining walls in place, and anchoring or securing structures that require protection against uplifting or overturning.



Rock and soil anchor system installed and stressed.



CEC rock and soil anchor unassembled; the bearing plate is on the left, the wedge block on the right.



CEC rock and soil anchor assembled.

CEC STRESSING EQUIPMENT FOR POST-TENSIONING

Equipment

The CEC system for stressing tendons in multi-strand post-tensioned concrete applications uses the best available equipment. The system comprises the following components:

Hydraulic ram with jack chair and pulling head.
Working capacity: 200 tons with adapter for 100 tons

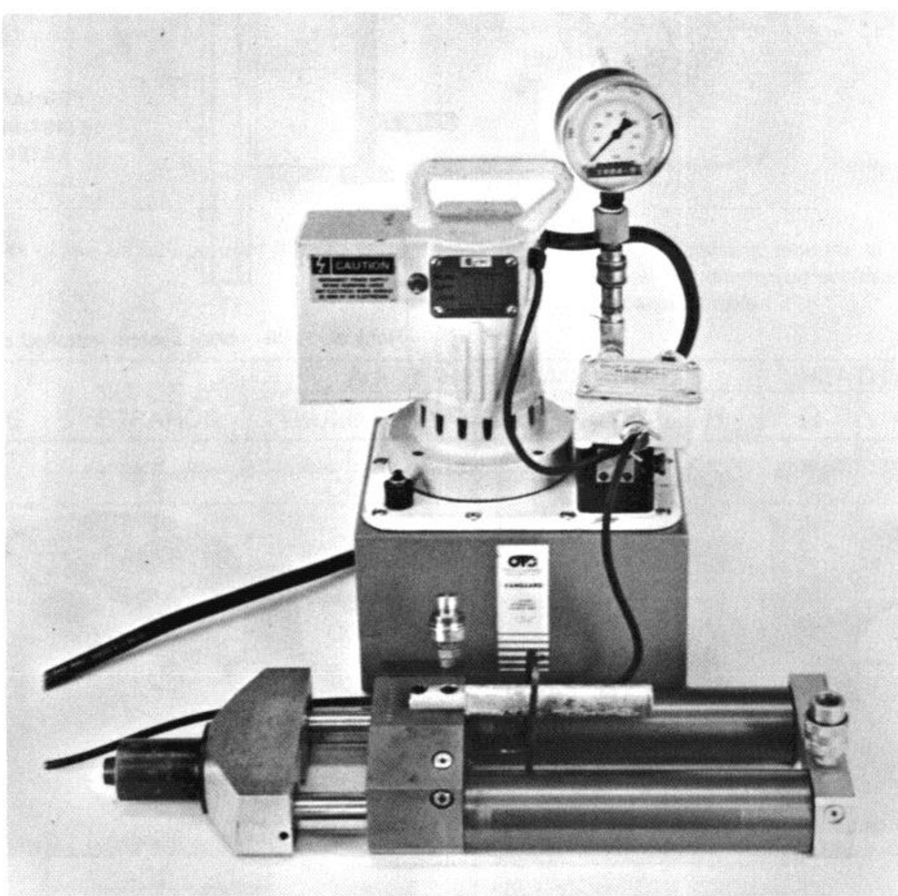
Electric hydraulic pump
Type B1, 5.5 hp, 200 cu in/min at 10,000 psi

High-pressure hoses

Calibrated pressure gauge 10,000 psi

Stressing Procedure

1. Post-tensioning shall not commence until concrete test cylinders cured under job-site conditions indicate that the concrete in the member has attained the strength specified in the drawings.
2. The maximum temporary tensile stress (jacking stress) shall not exceed 80% of the specified minimum ultimate strength of the prestressing steel.
3. Refer to CEC shop drawings for the required jacking force and elongation per tendon.



CEC stressing system.

2.2.5 CONTINENTAL CONCRETE STRUCTURES, INC.

Continental Concrete Structures, Inc. is an advanced technology engineering firm, specialized in post-tensioned prestressed concrete construction.

The Company comprises a fully staffed Engineering Department and a post-tensioning materials manufacturing plant, integrated to provide complete design services, post-tensioning system materials and construction site assistance to Architect/Engineering and the construction industry. The Company maintains all of the necessary fabrication machinery, specialized equipment and manufacturing space to promptly meet the demands of the largest or smallest post-tensioned project.

The Engineering Department is equipped with the latest computerized design facilities, utilizing internally developed state of the art design programs for practically any building or bridge structure. The programs are continually modified and upgraded to reflect the latest code and industry changes.

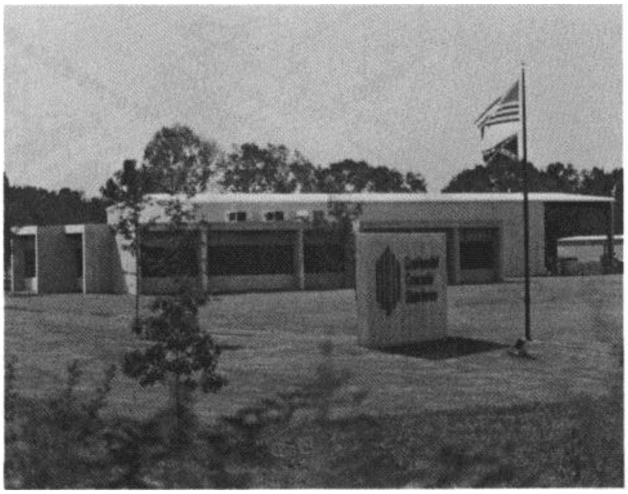
The Company also provides complete field labor services or technical jobsite assistance, to assure proper installation and stressing of the post-tensioning systems.

Post-tensioning systems manufactured by Continental Concrete Structures are shown on the following pages. All systems are based on ½ inch, 7 wire 270 ksi strand conforming to ASTM A-416, low relaxation type. the unbonded mono-strand tendons are coated with a corrosion inhibiting low friction compound and continuously encased in a tough plastic sheathing. For special applications in corrosive environment, the tendonsarecompletelyencapsulated through the use of end caps and connector tubes at the anchorages, providing a water tight enclosure around the prestressing steel.

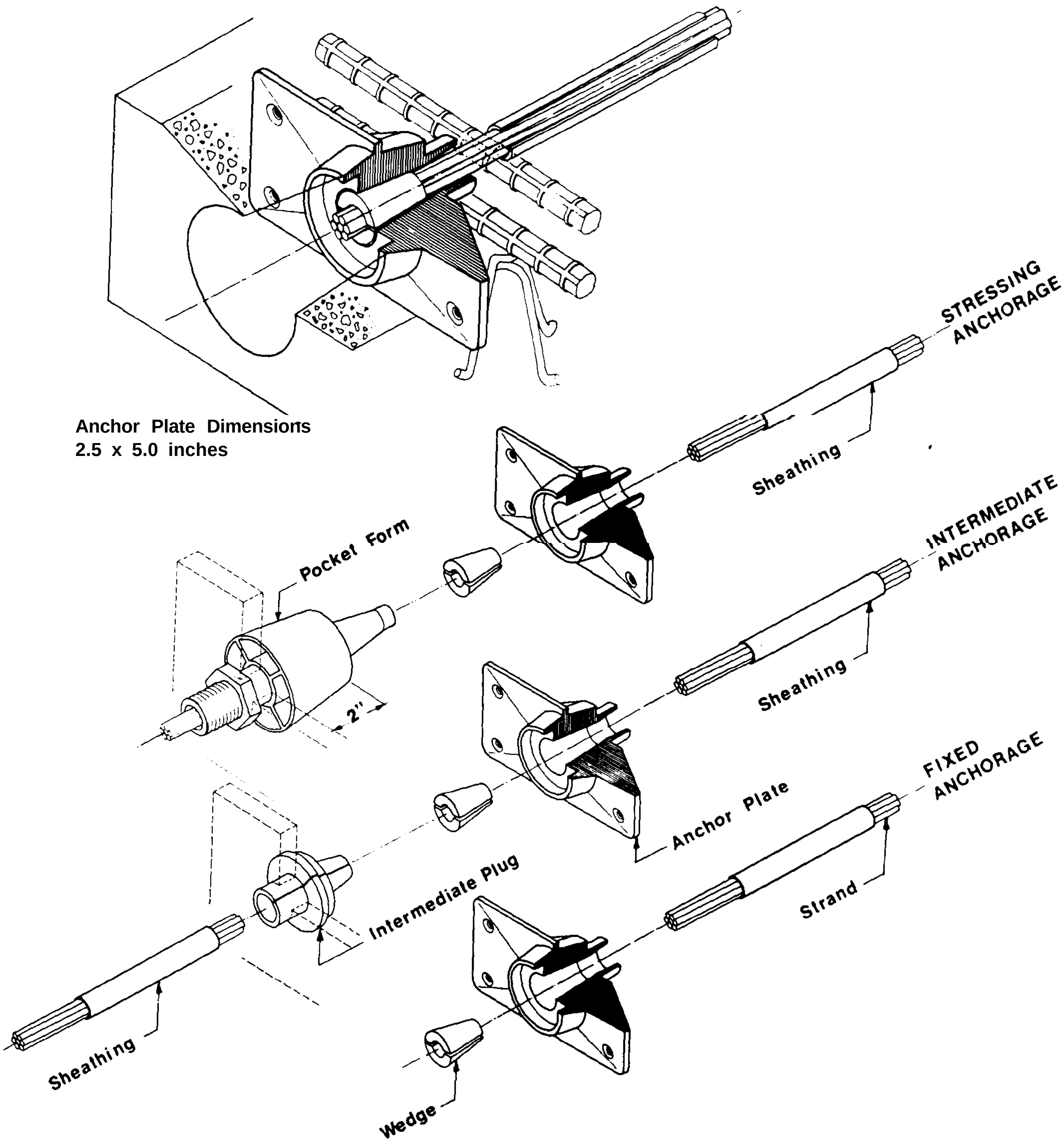
This mono-strand system has proven its effectiveness and versatility in many thousands of square feet of building construction, from high rise buildings, parking garages, foundation slabs to residential grade slabs.

The bonded multistrand systems are designed to meet the larger force requirements normally encountered in bridge construction, transfer girders or beams with long spans or particularly heavy loads. The efficient anchorage design maximizes space utilization in the anchor zone while providing easy to install components at the construction site.

All post-tensioning systems supplied by Continental Concrete Structures comply with the current PTI Specifications, and applicable local building codes.



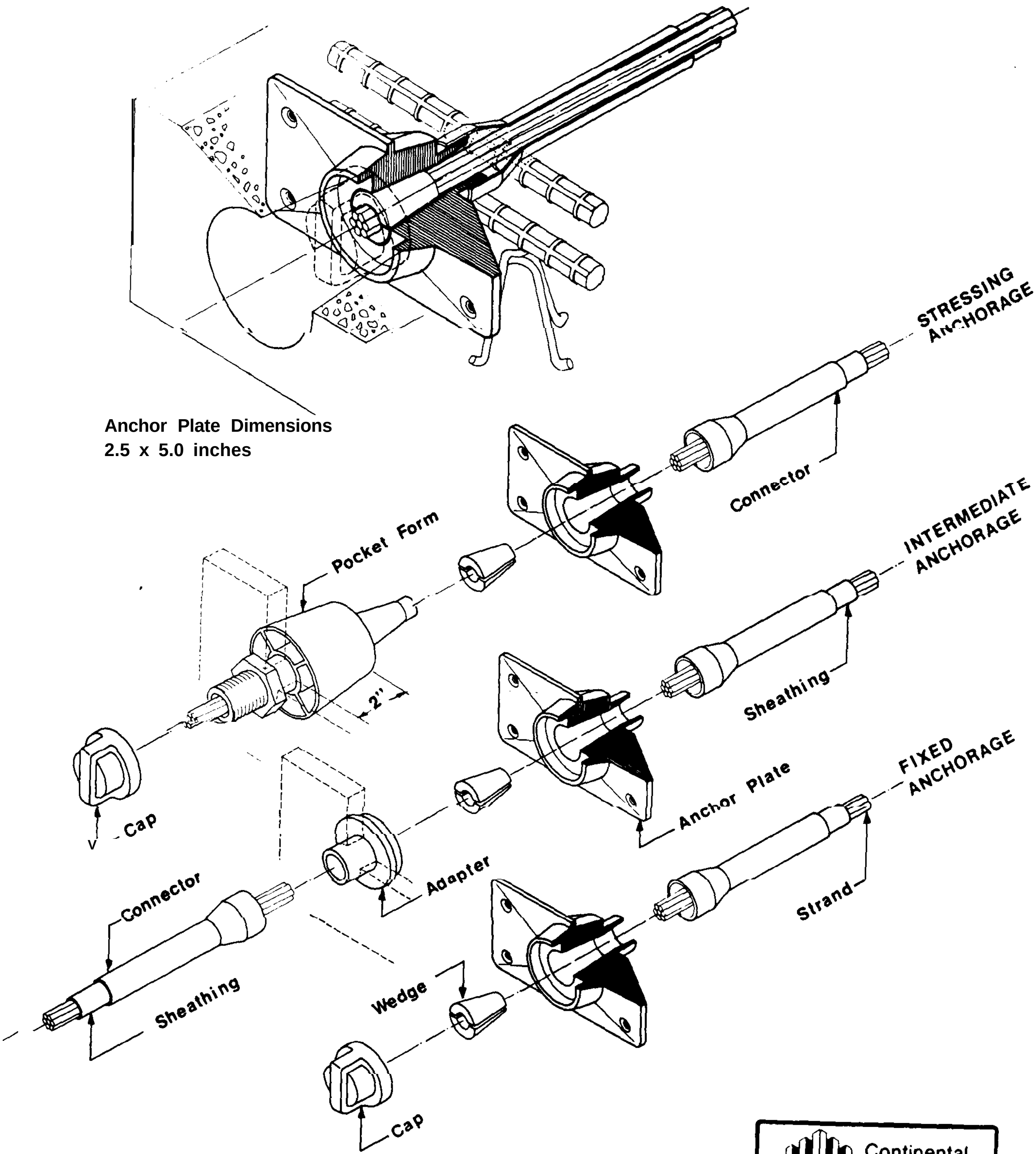
MONO-STRAND TENDON FOR NORMAL ENVIRONMENT



Anchor Plate Dimensions
2.5 x 5.0 inches

Continental Concrete Structures

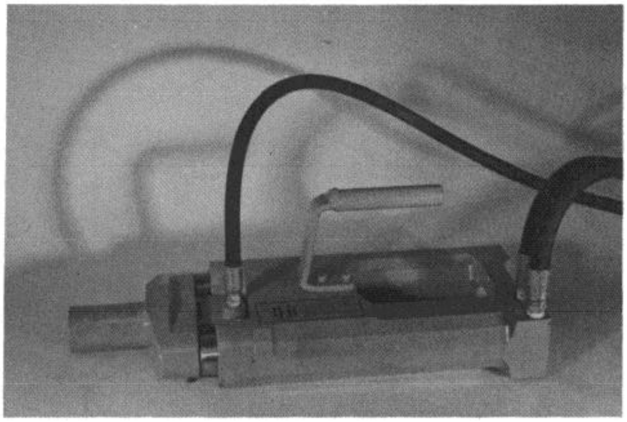
MONO-STRAND TENDON FOR AGGRESSIVE ENVIRONMENT



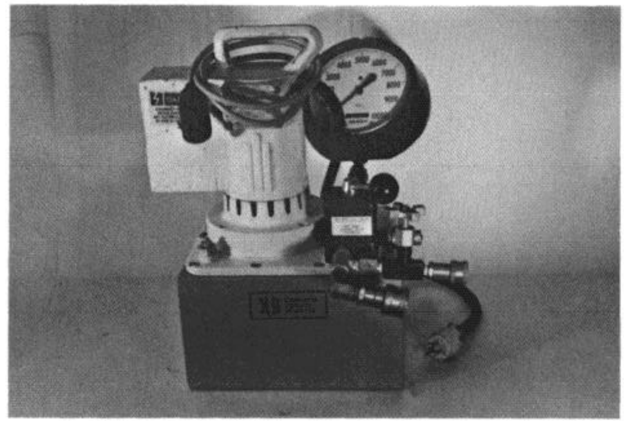
Continental Concrete Structures

MONO-STRAND STRESSING EQUIPMENT

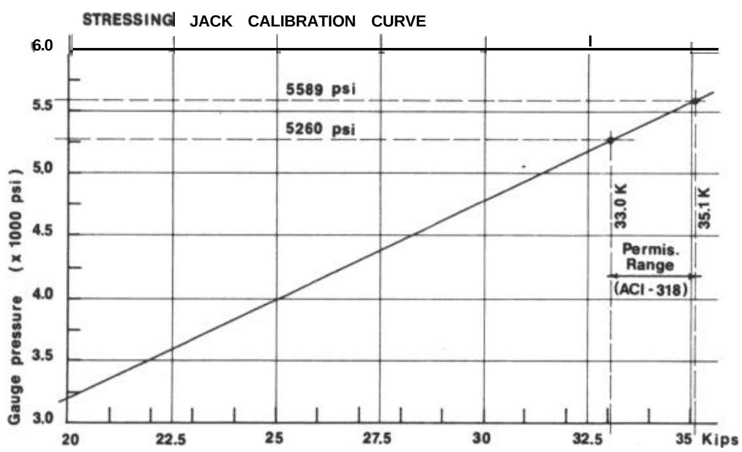
Open Throat Jack



Hydraulic Pump

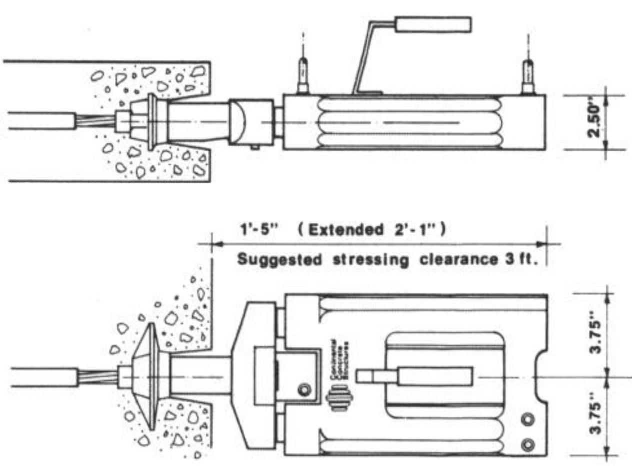


Jack Calibration



Nolo: Final effective tendon force depends on short and long term losses.
Contact this office for an accurate evaluation.

Jack Clearance

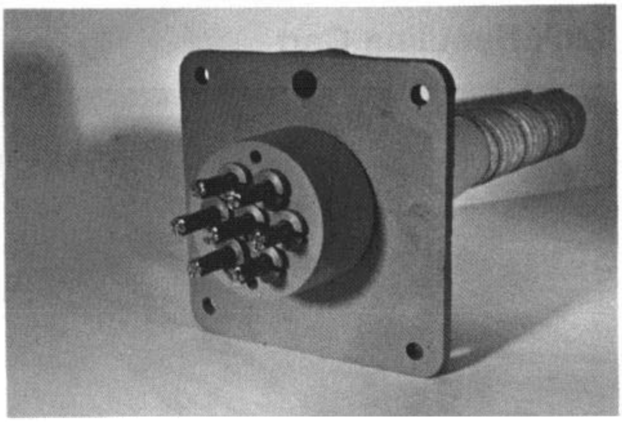


Stressing Jack Data:

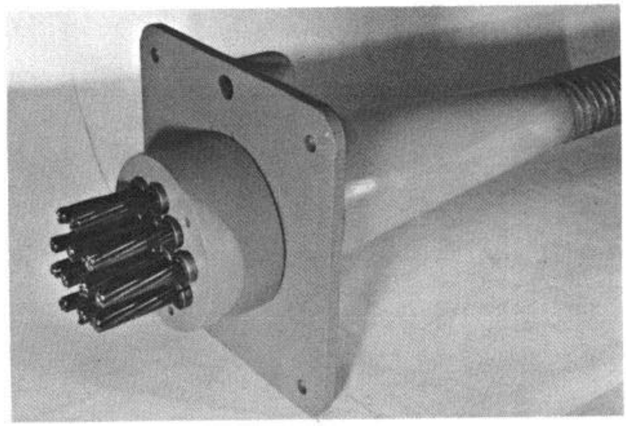
Max. Stroke	8.00 in.
Effective Ram Area	8.28 sq.in.
Operating Pressure	5280 - 5590 psi.
Gauge Face Diameter	8.00 in,
Operating Weight	47.00 lbs.
Force at Stressing	33.04 - 35.10 kips.
Force at Seating	28.91 kips.

Continental Concrete Structures

MULTI-STRAND TENDONS

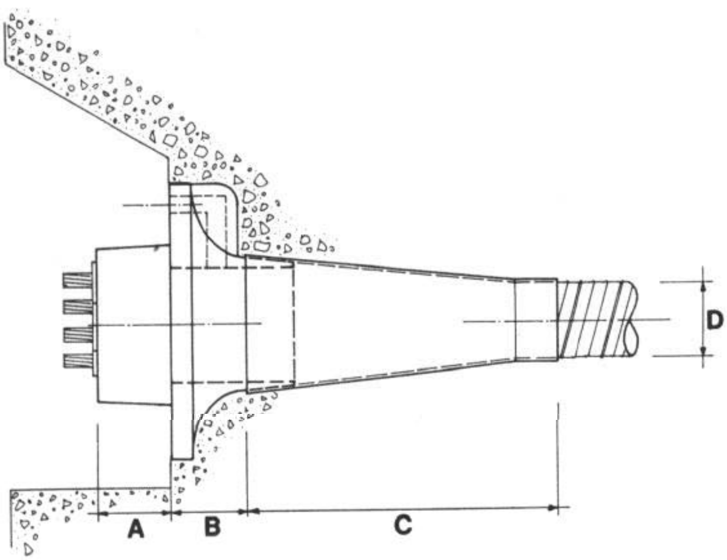


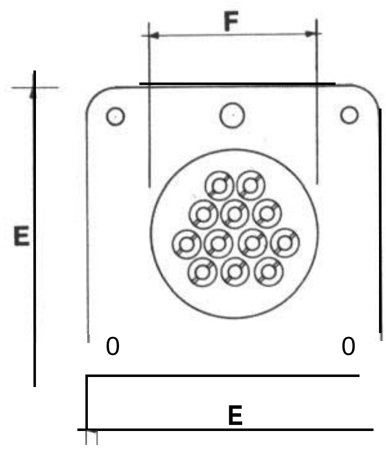
7 - 0.5" Anchorage



12 - 0.5" Anchorage

Anchorage Details





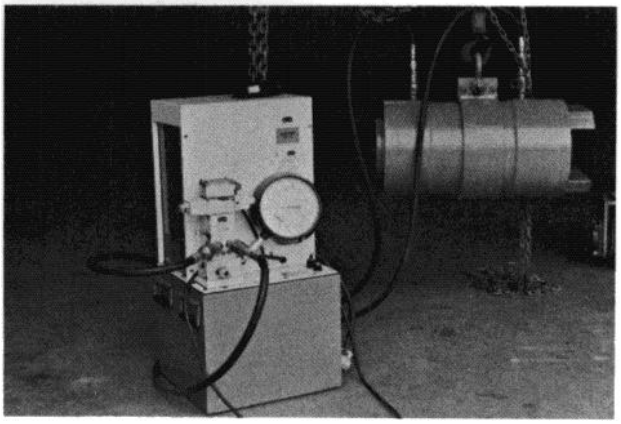
Dimensions:

Tendon Size:	7Ø0.5	12Ø0.5	19Ø0.5	31Ø0.5
Fult. k	289.1	495.6	784.7	1280.3
0.8 Fult. k	231.3	396.5	627.8	1024.2
0.6 Fult. k	173.5	297.4	470.8	768.2
Dimensions :				
A in	2.50	2.75	3.25	4.00
B in	2.25	2.50	3.00	3.50
C in	10.50	11.50	12.50	13.50
D in	2.00	2.75	3.38	4.38
E in	8.50	10.50	13.50	18.00
F in	4.38	5.88	7.25	9.25

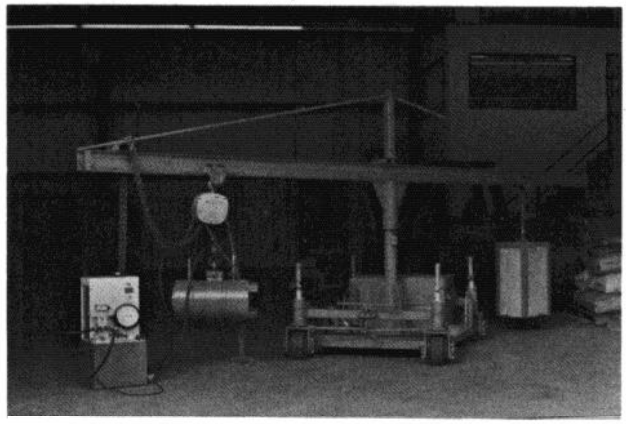
Continental Concrete Structures

MULTI-STRAND STRESSING AND GROUTING EQUIPMENT

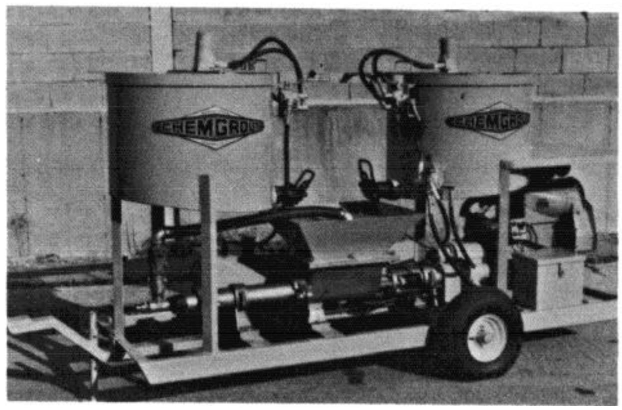
Stressing Jack and Hydraulic Pump



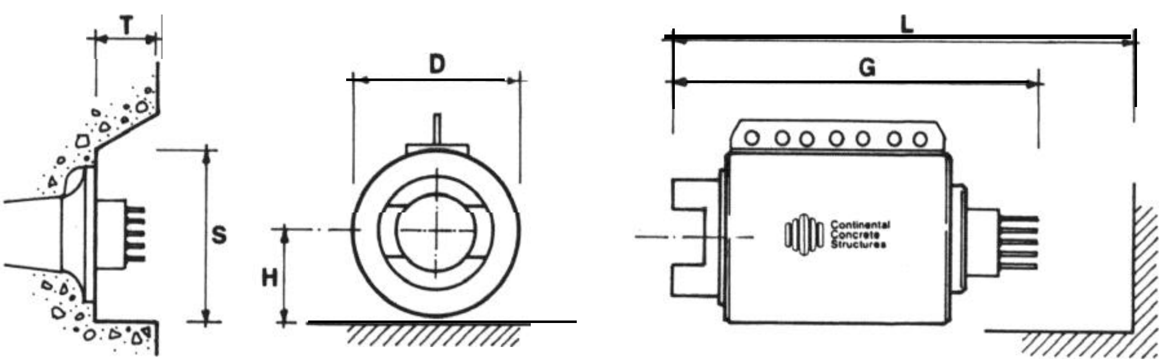
Jack Handling Cart



Grout Mixer and Pump



Jack Clearance



Dimension		D	G	H	L	S	T
7 ø 0.5	IN	12	43	6.5	67	13	5
12 ø 0.5	IN	15	45	7.5	69	15	6
19 ø 0.5	IN	17	51	9.0	82	18	7
31 ø 0.5	IN	22	53	11.5	87	23	7

Continental Concrete Structures

2.2.6 CONTINENTAL STRUCTURES, INC.

INTRODUCTION

Continental Structures is a post-tensioning subcontracting and engineering organization capable of manufacturing all types of post-tensioning systems for virtually every type of post-tensioned concrete structure, including commercial and residential buildings, parking structures, bridges, tanks and nuclear structures. There is within the organization a combined experience total of 80 years from which engineers and contractors can obtain technical assistance.

SERVICES

Continental Structures provides post-tensioning tendons f.o.b jobsite, or furnished and installed, depending on location and available placers. Design calculations and assistance to structural engineers can be provided by an engineering staff experienced in post-tension design. Reinforcing steel can be furnished and installed when it occurs in conjunction with post-tensioning. This work is usually done by a local rebar fabricator. Continental Structures will also coordinate alternate bids or design/construct bids on entire structural frames, in conjunction with local concrete contractors.

PRODUCTS

Mono-Strand System consisting of single unbonded 1/2" diameter, 270 ksi strand tendons, greased and plastic sheathed. This system is particularly suited for office buildings, parking structures, hotels, and other moderately loaded structures.

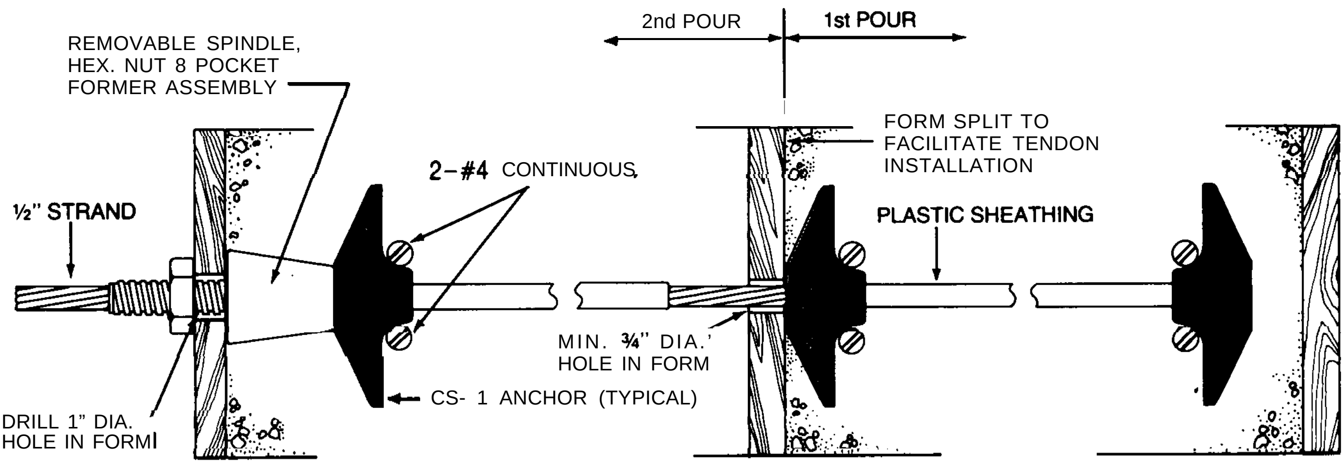
Multi-Strand System consisting of multiple bonded 1/2" diameter 270 ksi strand tendons encased in metal duct and placed by the pull-thru method. This system is particularly suited for bridges, tanks, nuclear structures, transfer girders, and other heavily loaded structures.

RECENT PROJECTS

Project & Location	Description	Structural Engineer	General Contractor
Continental Park Plaza El Segundo, Calif.	7 Story Office & 8 Level Parking	NAM Engineering Redondo Beach, Calif	Morley Const. Co. Los Angeles, Calif.
Speer Center Denver, Colorado	Office Bldg., Apt. Bldg. & Parking	Richard Weingardt Denver, Colorado	Mardian Const. Denver, Colorado
Marriott Hotel & Parking Torrance, Calif.	17 Story Hotel & 3 Level Parking	Meyer Associates Rockville, Maryland	Morley Const. Co. Los Angeles, Calif.
Creekview Towers Denver, Colorado	2-l 3 Story Apartment Bldgs.	Ihienfeldt, Peterson Denver, Colorado	Western Empire Const. Denver, Colorado
One Columbus Plaza Phoenix, Arizona	12 Story Office Building	Read, Jones, Christoffersen	TGK McCarthy Const. Phoenix, Arizona
Century Square Pasadena, Calif.	12 Story Tower 5 Level parking	Arch. & Engr. Collab. Los Angeles, Calif.	Ray Wilson Co. Los Angeles, Calif.
Betawest Office Bldg. Phoenix, Arizona	4 Story Office Building	Opus Corp. Minneapolis, Minn.	Opus Corp. Phoenix, Arizona
California Plaza Los Angeles, Calif.	6 Story Office Building	Martin & Huang Inter.. Los Angeles, Calif.	HCB Contractors Los Angeles, Calif

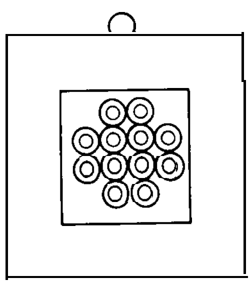
POST TENSIONING SYSTEMS

Mono-System

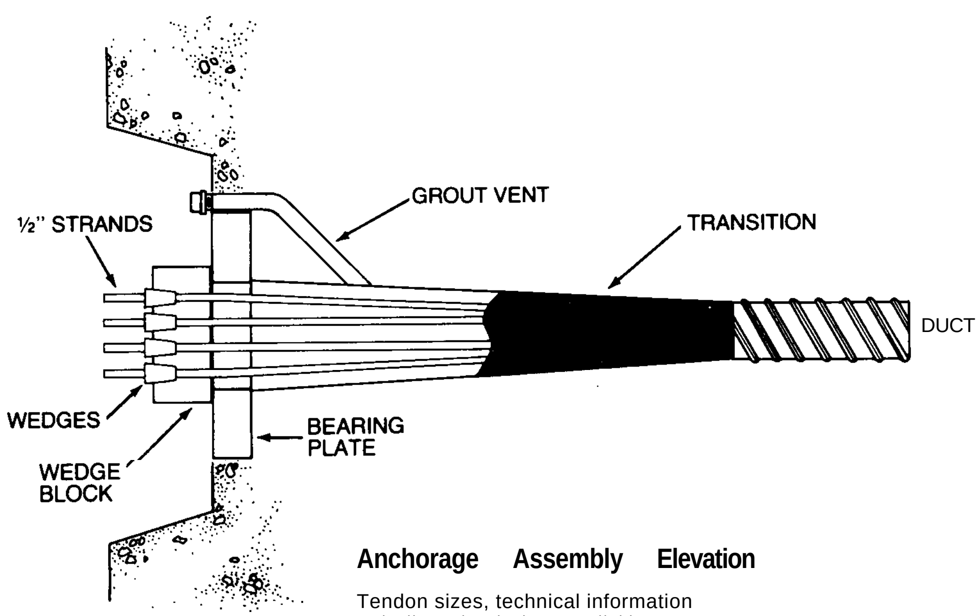


Stressing End **Intermediate Stressing** **Fixed End**

Multi-System



Anchorage End View



Anchorage Assembly Elevation

Tendon sizes, technical information
and dimensional data available upon request.

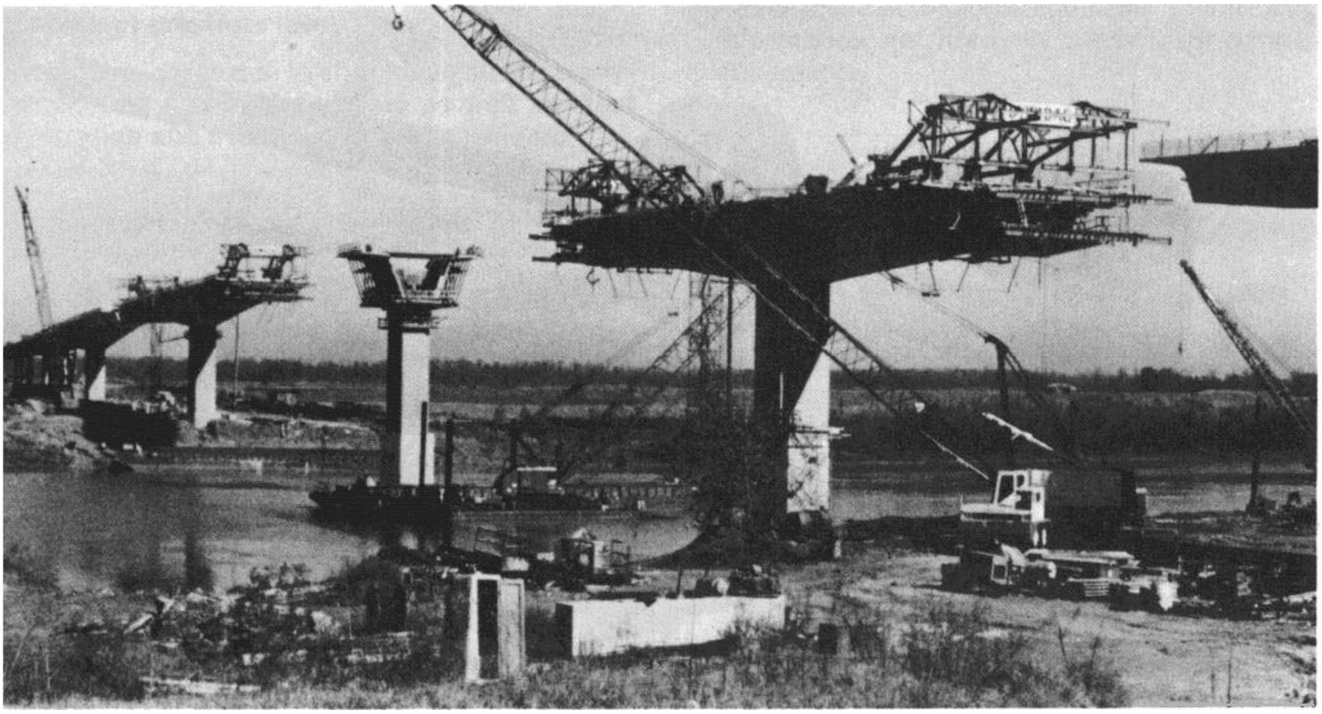
2.2.7 DYWIDAG SYSTEMS INTERNATIONAL

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC. and DYWIDAG SYSTEMS INTERNATIONAL, CANADA, LTD. serve the construction industry in the United States and Canada, Based on the extensive experience and the latest research and development of their century old parent company, Dyckerhoff & Widmann, AG, the firms offer a wide range of services. These include an in-house engineering staff, technical expertise, special construction equipment as well as post-tensioning systems and a reinforcing bar and splice system. Installation and design and build services are also available.

Thefirmsspecialize in high technology engineering oriented construction such as prestressed concrete segmental bridges, including furnishing of special formwork and form supporting equipment. The DSI firms are leaders in the development of rock and soil anchors, uplift anchors and rock bolts for the mining, tunneling, and heavy construction industries.

The DYWIDAG Threadbar System and the DYWIDAG Strand System allow the optimum combination of post-tensioning Systems and the most economic solution to a given post-tensioning problem. The DYWIDAG grade 60 Reinforcing Threadbar offers an efficient and versatile reinforcing bar splice system capable of developing up to the full strength of the bar in tension and/or compression.’

FAA Air Traffic Control Towers at Dallas/ Fort Worth, TX and 15 other USA airports.



Form travelers, threadbar and multistrand post-tensioning systems, Red River Bridge, Boyce, LA.

DYWIDAG THREADBAR POST-TENSIONING SYSTEM

The components of Dywidag Threadbar System are manufactured in the United States exclusively by Dywidag Systems International. Used world-wide since 1965, the threadbar system provides a simple, rugged method of efficiently applying prestress force to a wide variety of structural systems including post-tensioned concrete, rock and soil anchor systems.

Available in ⅝", 1", 1¼", and 1%" nominal diameter, Dywidag Threadbars are hot rolled and proof stressed alloy steel conforming to ASTM A 722.

The Dywidag Threadbar prestressing steel has a continuous rolled-in pattern of threadlike deformations along its entire length. More durable than machined threads, the deformations allow anchorages and couplers to thread onto the threadbar at any point.

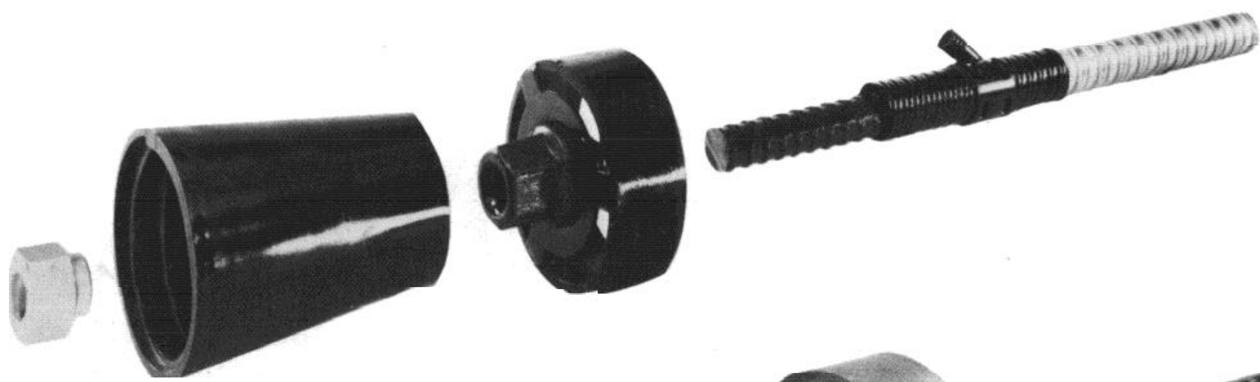
The strength of the Dywidag Threadbar anchorages and couplers exceeds the requirements of ACI 318. Test reports are available for the main components of the system.

Conforming to the requirements of ASTM A 615, the threadbar deformations develop an effective bond with cement or resin grout. The continuous thread simplifies stressing. Lift off readings may be taken at any time, and the prestress force increased or decreased as required.

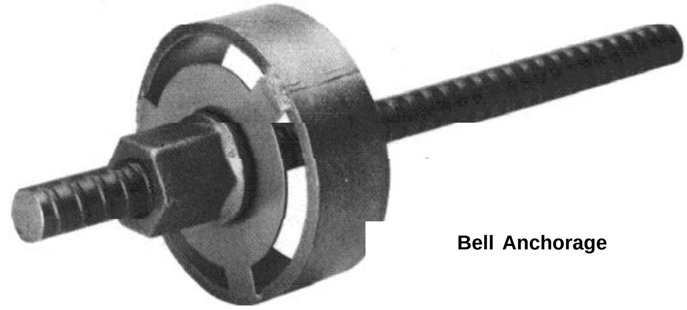
The Dywidag Threadbar System is primarily used for grouted construction. All components of the system are designed to be fully integrated for quick and simple field assembly. Sheathing, sheathing transitions, grout sleeves, and grout tubes all feature thread type connections.

Placing Dywidag tendons is simplified through the use of re-usable plastic pocket formers. Used at each stressing end, the truncated, cone shaped pocket former can extend through or butt up against the form bulkhead.

Available in 60' mill lengths, threabbars may be cut to specified lengths before shipment to the job site. Or where circumstances warrant, the threadbars may be shipped to the job site in mill lengths for field cutting with a portable friction or band saw. Threadbars may be coupled for ease of handling or to extend a previously stressed bar.



Exploded view: Tendon Assembly with Bell Anchorage and Pocket Former



Bell Anchorage

DYWIDAG THREADBAR POST-TENSIONING SYSTEM DETAILS

PRETRESSING STEEL PROPERTIES

Nominal Threadbar Diameter (inches)	Ultimate Stress (f_{pu}-ksi)	Cross Section Area (Aps-inches^2)	Ultimate Strength (f_{pu} A ps)	Prestressing Force — (kips)			Weight (lbs./ft.)	Minimum Elastic Bending Radius (ft.)	Maximum Threadbar Diameter (inches)
				$0.80 f_{pu} A_{ps}$	$0.70 f_{pu} A_{ps}$	$0.60 f_{pu} A_{ps}$			
⅝	157	0.28	43.5	34.8	30.5	26.1	0.98	26	0.693
1	150	0.85	127.5	102.0	89.3	76.5	3.01	52	1.201
1	160	0.85	136.0	108.8	95.2	81.6	3.01	49	1.201
1¼	150	1.25	187.5	150.0	131.3	112.5	4.39	64	1.457
1¼	160	1.25	200.0	160.0	140.0	120.0	4.39	60	1.457
1⅜	150	1.58	237.0	189.6	165.9	142.2	5.56	72	1.630
1⅜	160	1.58	252.8	202.3	177.0	151.7	5.56	67	1.630

STEEL STRESS LEVELS

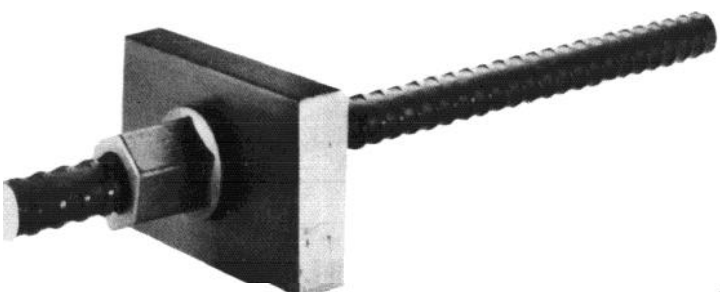
Dywidag Threadbars may be stressed to the allowable limits of ACI 318. The maximum jacking stress (temporary) may not exceed $0.80 f_{pu}$ and the transfer (lockoff) may not exceed $0.70 f_{pu}$ based on a yield strength of $0.85 f_{pu}$.

ACI 318 does not stipulate the magnitude of prestress losses or the maximum final effective (working) prestress level.

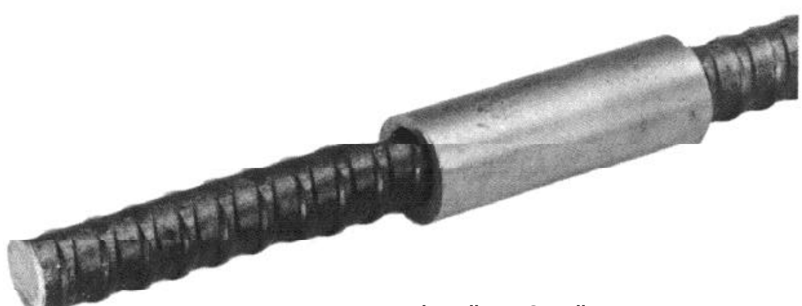
Prestress losses due to shrinkage, elastic shortening and creep of concrete as well as steel relaxation and friction must be considered.

The final effective (working) prestress level depends on the specific application. In the absence of a detailed analysis of the structural system, $0.60 f_{pu}$ may be used as an approximation of the effective (working) prestress level.

Actual loss calculations requirestructural design information not normallv present on contract documents.



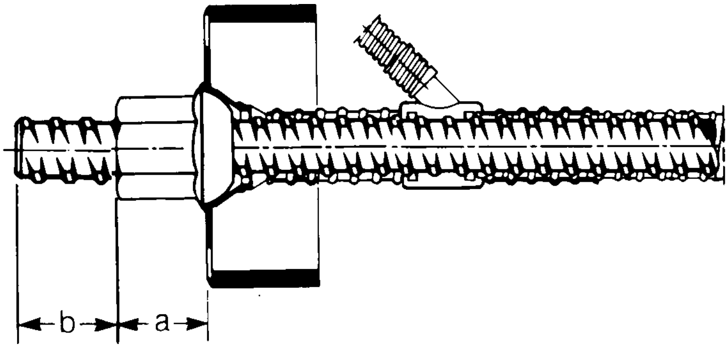
Threadbar Plate Anchorage



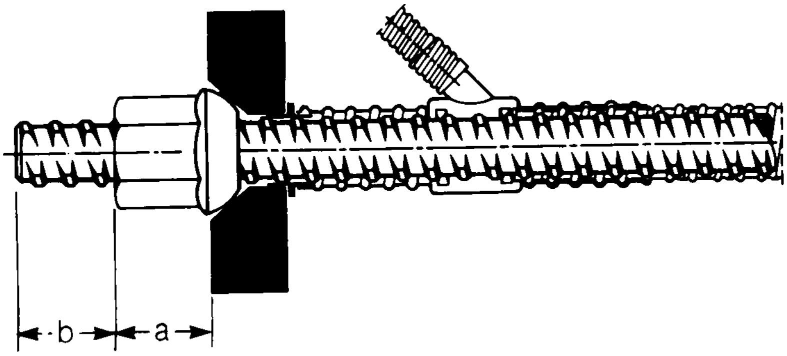
Threadbar Coupling

DYWIDAG THREADBAR POST-TENSIONING SYSTEM DETAILS

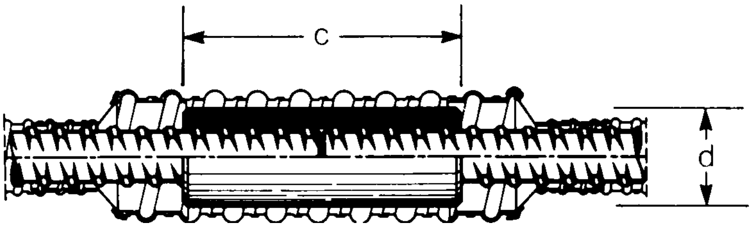
DYWIDAG THREADBAR BELL ANCHORAGE



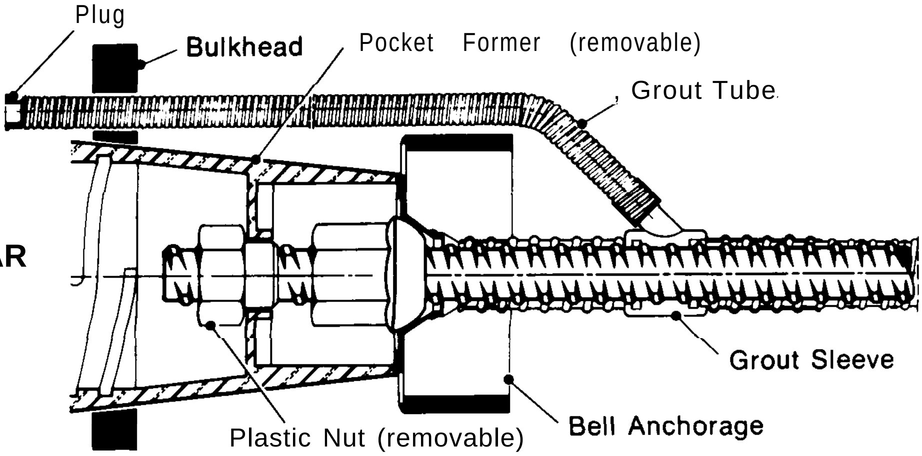
DYWIDAG THREADBAR PLATE ANCHORAGE



DYWIDAG THREADBAR COUPLER



DYWIDAG THREADBAR TENDON ASSEMBLY



STRESSING END

DYWIDAG THREADBAR POST-TENSIONING SYSTEM DETAILS

Anchorage Details

Threadbar Diameter (inches)		⅝	1	1¼	1 %
Bell Anchor Size (inches)		3¼ Ø × 1½	5½ Ø × 2⅝	6¾ Ø × 2⅝	7¾ Ø × 3⅛
Anchor Plate Size* (inches)		2 × 5 × 1 3 × 3 × ¾	4 × 6½ × 1¼ 5 × 5 × 1¼	5 × 8 × 1½ 6 × 7 × 1½	5 × 9½ × 1¾ 7 × 7½ × 1¾
Nut Extension (inches)	a	19/16***	1 %	2½	2¾
Min. Bar Protrusion** (inches)	b	2½	3	3½	4

*Other plate sizes available on special order. **To accommodate stressing.
***Import nut extension: 1".

Coupler Details

Threadbar Diameter (inches)		⅝	1	1¼	1 %
Length (inches)	c	4 %	5½	6¾*	8 %
Diameter (inches)	d	1¼	2	2 %	2 %

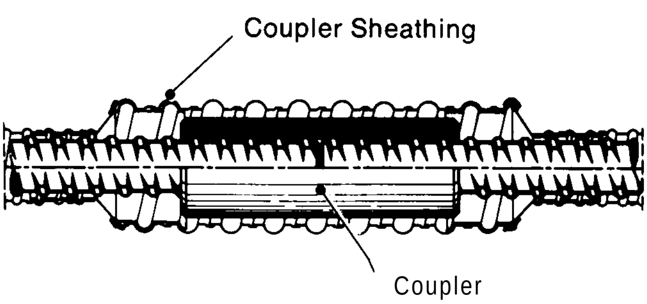
*7½" long coupler available on special order.

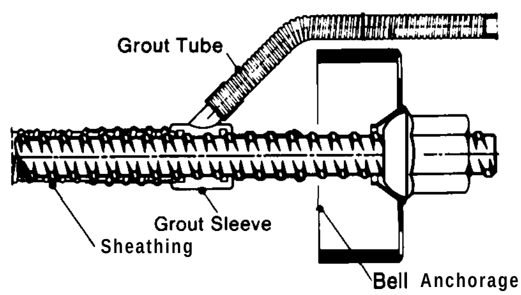
Sheathing Details

Threadbar Diameter (inches)	⅝	1	1¼	1 %
Threadbar Sheathing O.D. (inches)	1	1½	1⅞	2
Threadbar Sheathing I.D. (inches)	¾	1¼	1⅝	1¾
Coupler Sheathing O.D. (inches)	1 %	2¾	3¼	3¾
Coupler Sheathing I.D. (inches)	1⅜	2⅜	2⅞	3⅜

Pocket Former Details

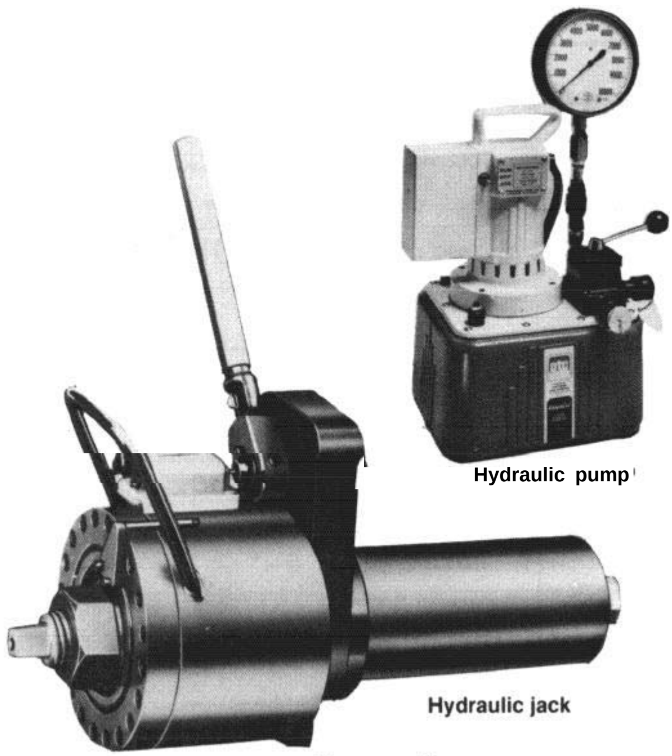
Threadbar Diameter (inches)	⅝	1	1¼	1 %
Length (inches)	4¾	7	8	8 %
Maximum Diameter (inches)	3 %	5 %	6½	6½





FIXED END

DYWIDAG THREADBAR POST-TENSIONING SYSTEM DETAILS



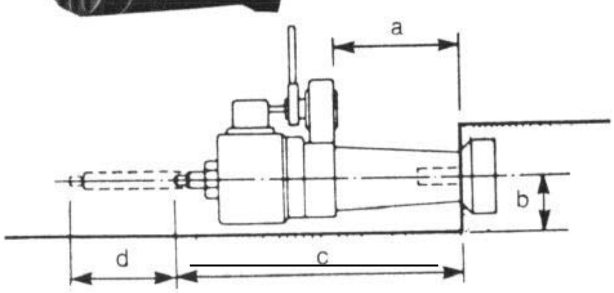
Hydraulic pump

Hydraulic jack

STRESSING

Dywidag Threadbars are stressed using compact, lightweight electric-powered hydraulic jacks. Easily handled by one man, the jack fits over a pull rod designed to thread over the threadbar protruding from the anchor nut. The jack nose contains a socket wrench and ratchet device which allows the nut to be tightened as the threadbar elongates.

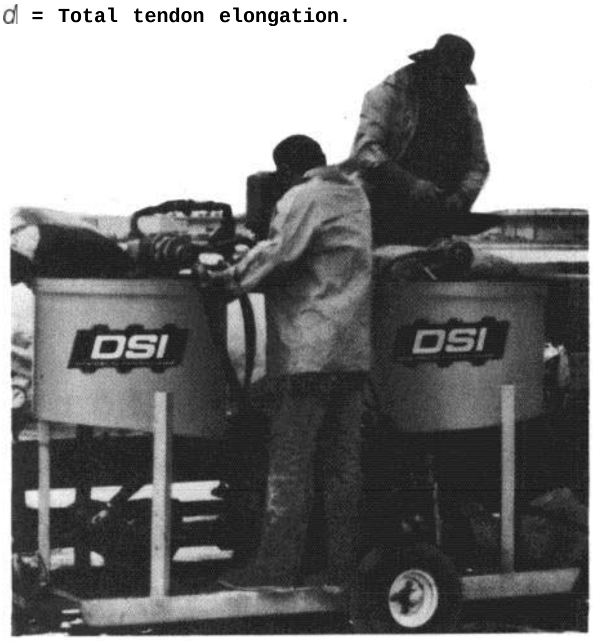
The magnitude of the prestress force applied is monitored by reading the hydraulic gauge pressure and by measuring the threadbar elongation. The elongation can be measured directly by noting the change in threadbar extension. Also, a counter mounted on the jack records the revolutions of the anchor nut which is a direct measure of the threadbar elongation.



d = Total tendon elongation.

STRESSING DATA

Jack Capacity (Kips)	60	150	250
Jack Application	5/8	1, 1¼	1¼ 1%
a (inches)	7%	8½	11
b (inches)	3¼	4	6
c Min. (inches)	24	26	30
Weight (lbs.)	50	80	100



GROUTING

Grouting completes the installation process for post-tensioned concrete construction. The grout is important in protecting the steel from corrosion and contributes significantly to the ultimate strength of the structure.

A portable grout mixer is used to flush out the tendon sheathing to remove debris. Then cement and water grout are pumped into the grout tube at one end of the tendon using a grout tube at the other end as a vent. An admixture is used to control expansion and pumpability.

DYWDAG THREADBAR ROCK & SOIL ANCHORS

The Dywidag Threadbar System is used extensively in rock and soil anchor construction because of its versatility, strength, performance characteristics and off-the-shelf availability of most components.

Dywidag Threadbars may be used individually or in multiples depending upon the magnitude of force requirements or upon drilling considerations.

The anchor plate need not be perpendicular to the Dywidag Threadbar. The curved surface of the anchor nut accommodates up to 5° misalignment of the threadbar with the bearing plate up to 25 degrees can be corrected by using a set of wedge washers with the anchor nut. Threadbars may be shipped to the job site in mill lengths for field cutting with a portable friction or band saw. Threadbars may be coupled for ease of handling or to extend a previously stressed bar.

CORROSION PROTECTION OPTIONS

Unprotected Anchors

Unprotected anchors are recommended for temporary use only. The stressing length is unprotected while the bond length is covered with cement grout. Unprotected anchors may be subject to corrosion, however, the relatively large diameter of the Dywidag Threadbar offers more corrosion resistance than smaller diameter prestressing steel elements.

Simple Protected Anchors

Simple protected anchors may be used for temporary anchors or permanent anchors in non-aggressive rock or soil. A polyethylene sheathing covers the stressing length. For permanent anchors the threadbar is coated with a corrosion inhibitor before the polyethylene sheathing is installed. The bond length is covered with cement grout.

Double Protected Anchor

Double protected anchors are recommended for anchors with a long service life and for an environment where aggressive materials or stray electrical currents are expected.

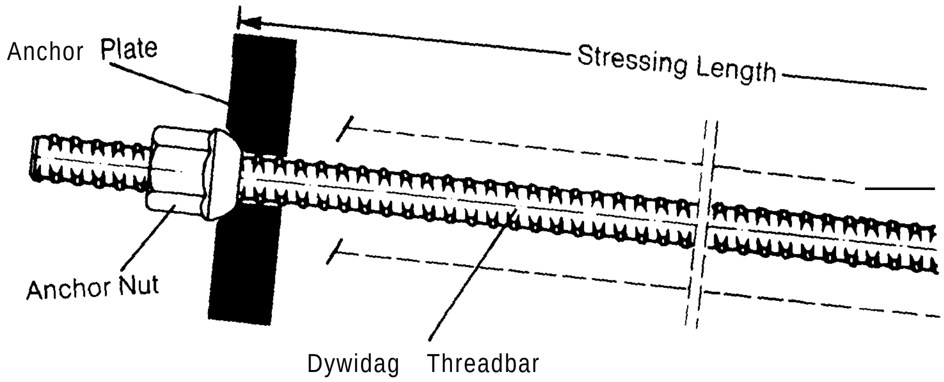
A corrugated PVC sheathing is installed over the bond length and the stressing length of the anchor. The annular space between threadbar and PVC is fulljy grouted before the anchor is installed. To accommodate the elongation during stressing, a short length of threadbar is left free of the corrugated sheathing behind the stressing anchor.

A smooth PVC sheathing is installed over the corrugated sheathing in the stressing length. This accommodates elongation during stressing. The PVC sheathing makes a slip joint connection with a steel tube welded to the anchor plate. The steel tube is filled with a mastic corrosion inhibitor. A plastic cap filled with the corrosion inhibitor protects theanchor nut yet allows future stress adjustment.

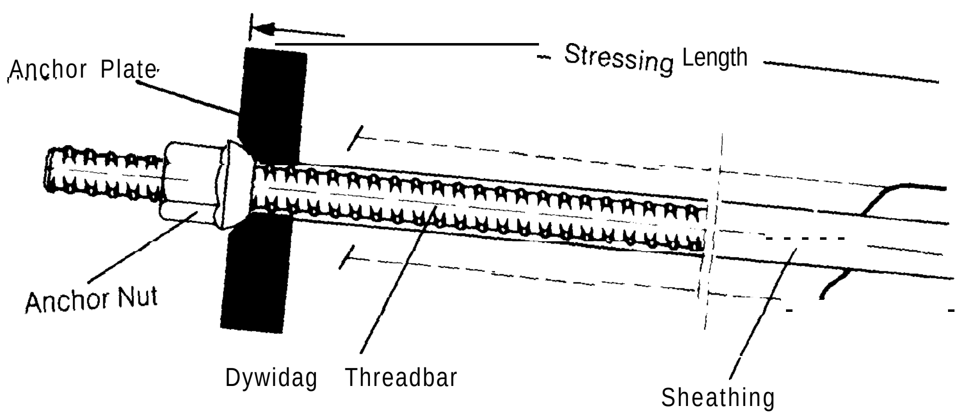
The PVC sheathing prevents intrusion of any corrosive substances. The grout around the threadbar provides a chemical corrosion protection by embedding the bar in a highly alkaline environment. The threadbar deformations minimize the size and control the distribution of any cracks that develop in the stressing length, fully maintaining the protection action of the grout cover.

DYWIDAG THREADBAR ANCHOR DETAILS

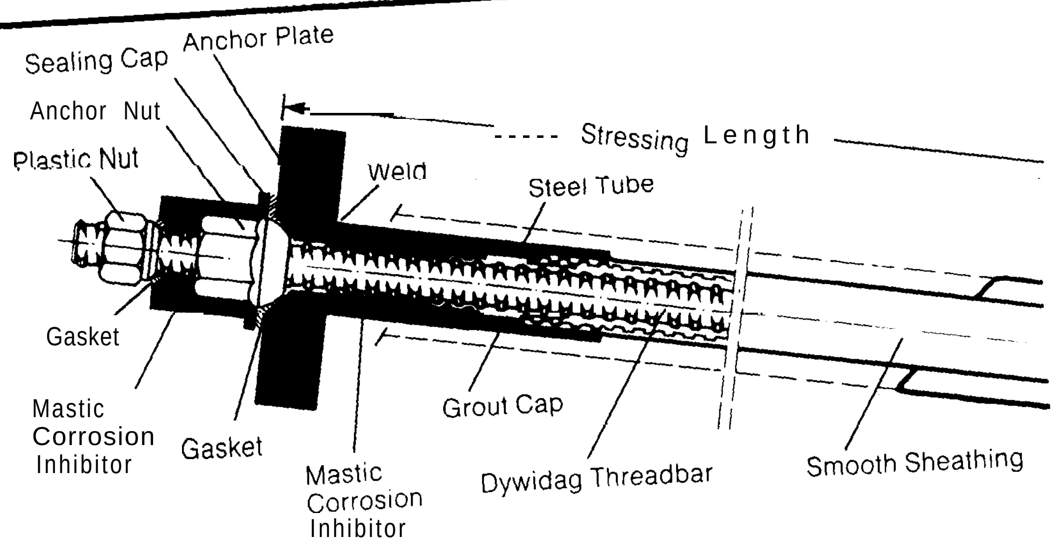
DYWIDAG THREADBAR ANCHOR WITHOUT CORROSION PROTECTION



DYWIDAG THREADBAR ANCHOR WITH SIMPLE CORROSION PROTECTION

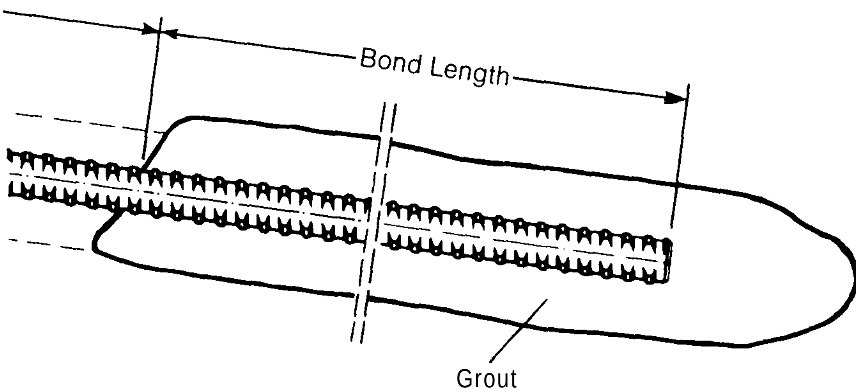


DYWIDAG THREADBAR ANCHOR WITH DOUBLE CORROSION PROTECTION

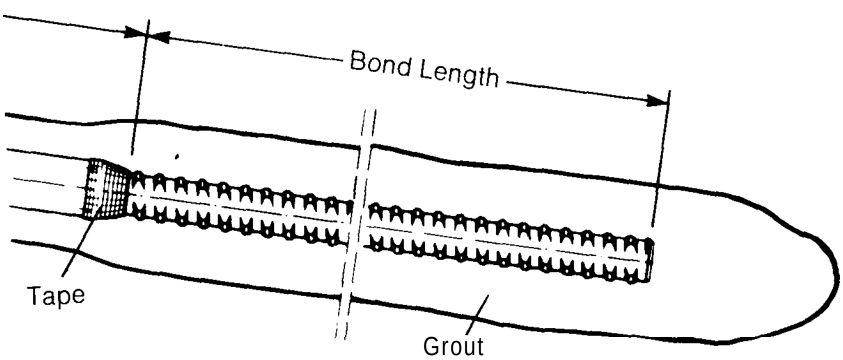


DYWIDAG THREADBAR ANCHOR DETAILS

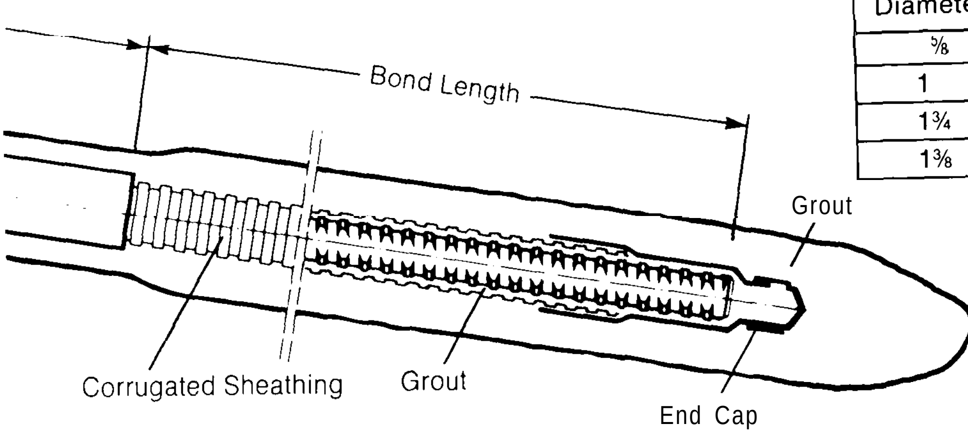
Nominal Threadbar Diameter	Max. Anchor Diameter (in.)	
	Without Coupler	With Coupler
⅝	**0.687**	1.250
1	1.187	**2.000**
1¾	**1.438**	**2.375**
1⅜	**1.625**	**2.625**



Nominal Threadbar Diameter	Max. Anchor Diameter (in.)	
	Without Coupler	With Coupler
=/a	1 .000	1.625
1	1.625	2.125
1%	1.875	**2.500**
1%	**2.000**	**2.750**



Nominal Threadbar Diameter	Max. Anchor Diameter (in.)	
	Without Coupler	With Coupler
⅝	1.125	1.625
1	2.375	2.500
1¾	2.875	3.125
1⅜	2.875	**3.125**



DYWIDAG THREADBAR RESIN ANCHORS

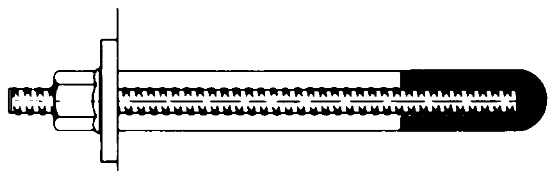
Theadbars, bonded to the rock or concrete by a fast curing polyester resin grout, are used extensively for slope stabilization, tie backs, tie downs, and for roof bolts. The resin grout develops a bond superior to that developed by cement grout, Fast gelling resin allows transfer of load to the rock formation within minutes after installing the anchor.

Resin anchored threadbars are installed in all types of rock or concrete. Track drills, tire mounted drills, jacklegs or stopers may be used to both drill the hole and install the Dywidag bolt.

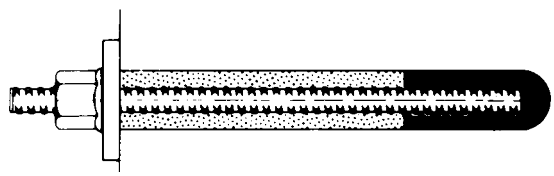
Polyester resin is packaged in cartridge form and is available in various diameters and gel times. The cartridge consists of a heat sealed tube of polyester film containing both the resin and the catalyst. The resin and catalyst are separated by a barrier which prevents chemical interaction. The resin cartridges are placed in the bore hole before the threadbar is inserted. The resin gels after the components are mixed during the installation of the Dywidag Threadbar.

Threadbars with resin point anchorage are used to apply a compressive force across layered rock strata. Threadbars may be installed using fast setting resin as the point anchorage in conjunction with slow setting resin as a corrosion protection for the free stressing length. Threadbar tension is applied after the fast setting resin has cured but before the slow setting resin cures.

Dywidag Resin Anchors are used where expansion shells are inappropriate. The resin anchorage length is easily adjusted to fit the varying rock conditions. Dywidag Resin Anchors may be installed in bore holes located at any angle above or below horizontal.



Threadbar With Resin Point Anchorage



Fully Resin Anchored and Grouted Threadbar

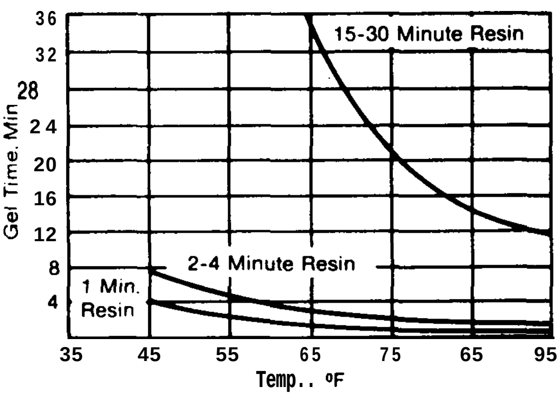
Wedge Washer Assembly

Threadbar and Resin Cartridge

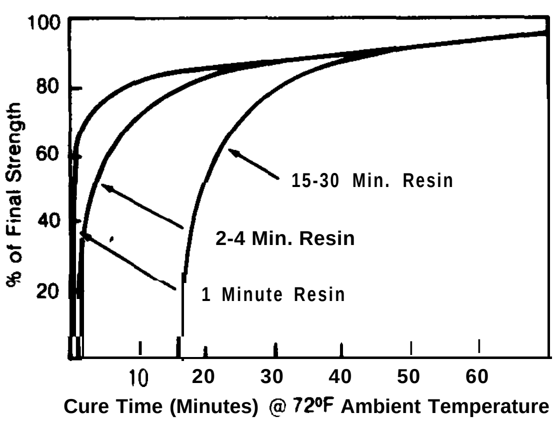
DYWIDAG THREADBAR RESIN ANCHOR DESIGN

Polyester resin grout is unaffected by blasting and provides a corrosion protection equal to cement grout. Resin is unaffected by fresh water, salt water, mild alkalies and mild acids.

Gel Time vs. Temperature



Resin Strength vs. Cure Time



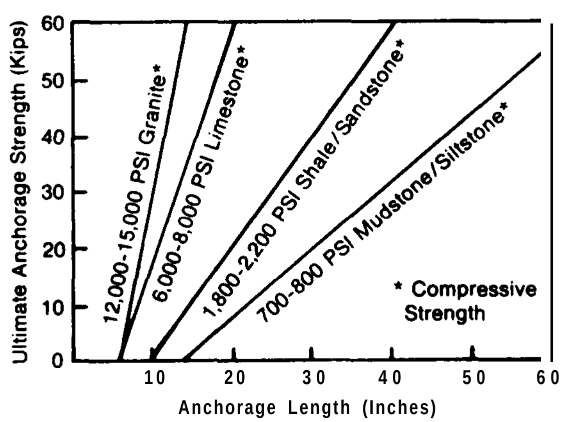
After drilling, each bore hole must be cleaned with either air or water before the resin cartridge is installed. Standing or flowing water does not affect the resin but may cause deterioration of the hole.

The anchorage length varies with the structure of the rock formation. The properties of the polyester resin do not govern the design of the anchorage zone length. All design assumptions should be verified by field tests.

The resin gel time and cure time are temperature sensitive. To insure proper behavior, the ambient temperature of the rock or concrete must be monitored.

The bore hole and cartridge diameter must be compatible with the diameter of the threadbar specified.

Anchorage Strength vs. Anchorage Length



RESIN YIELD CHART*

ASTM A 722 Nominal Threadbar Diameter Inches	Hole Diameter (inches)										
	1%	1%	1%	1%	1%	1%	1%	1%	2	2	2%
	Cartridge Diameter (inches)										
	1%	1¼	1%	1%	1%	1%	1 9/16	1 9/16	1 9/16	1%	1%
1	2.25	1.75	1.25*		1.52	1.16	1.19				
1%				1.53		1.85	1.69	1.30		1.28	
1%							2.27	1.61	1.21	1.52	1.00

*Ratio of resin column length after insertion of Dywidag Threadbar to resin column length before inserting Dywidag Threadbar. Resin yield chart information provided by Celtite, Inc.

Yields are calculated; no waste or allowance for over drilling is included. Site trials should be conducted to determine actual resin requirements.

Example: When using 1” Dywidag Threadbar in 1%” diameter hole with 1%” diameter resin cartridge, resin yield multiplier is 1.25*. A 12” resin column yields 15” of resin after the insertion of the Dywidag Threadbar.

DYWIDAG MULTISTRAND POST-TENSIONING SYSTEM

The Dywidag Multistrand System is available in the United States from Dywidag Systems International.

The Dywidag Multistrand System uses 0.6” diameter 7 wire 270 K strand conforming to ASTM A 416. Tendons are fabricated with either stress relieved or low relaxation strand depending on project specifications.

Standard Dywidag Multistrand anchors are designed for 3, 4, 5, 9, 12, 15, and 19 strands. Individual strands may be omitted from the strand patterns to obtain tendons of a smaller capacity, yet allow use of standard anchorage components, and standard Dywidag stressing equipment.

Tendons with up to 48-0.6” strands or 0.5” strand tendons of various sizes can be supplied for special applications where lead time permits.

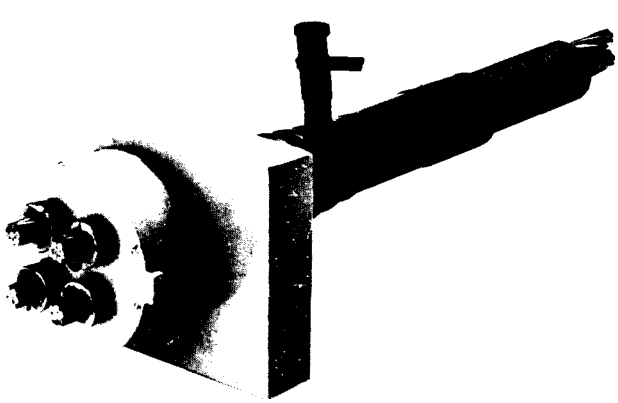
Dywidag Multistrand tendons may be placed in ducts prior to or after placing concrete by pulling or pushing techniques. Prefabricated tendons may be pulled into preplaced ducts using a hydraulic winch. Tendons may also be assembled by repeatedly pushing individual strands into preplaced ducts using a hydraulic pusher.

Dywidag stressing equipment is specifically designed for the standard Dywidag hardware. For efficiency and reliability, the Dywidag jacks are designed to power seat all wedges simultaneously.

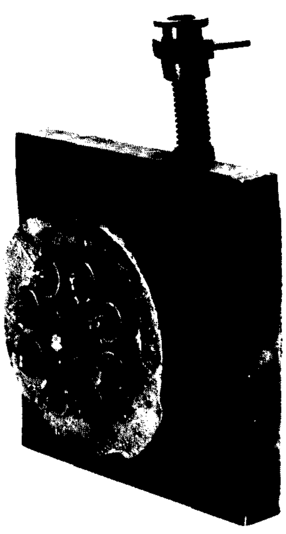
The Dywidag Multistrand anchors exceed the strength requirements of ACI 318 and AASHTO specifications. Test data are available for the main components.

Grouting completes the installation process for Dywidag Multistrand tendons. The grout protects the prestressing steel from corrosion and contributes significantly to the ultimate strength of the structure.

Dywidag Multistrand tendons are also used as high capacity earth or rock anchors. For this application special consideration must be given to the corrosion protection of all components of the earth or rock anchor. Information concerning the Dywidag double corrosion protection system for multistrand tendons is available upon request.



4-0.6” Multistrand Anchorage Type P (plate)



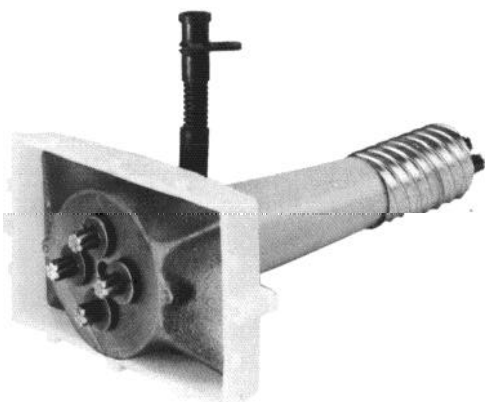
9-0.6” Multistrand Anchorage Type P (plate)

2.2.7 DYWIDAG SYSTEMS INTERNATIONAL — continued

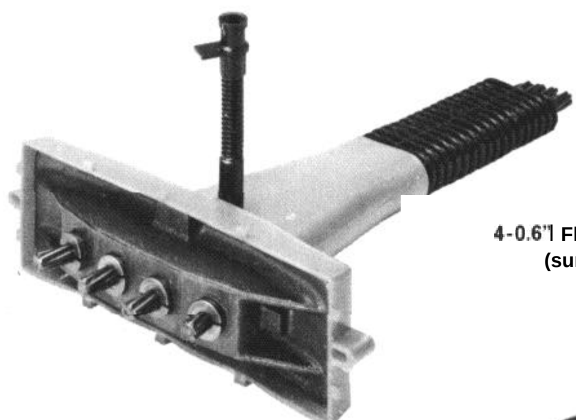
PRESTRESSING TENDON PROPERTIES

Standard Tendon Size	Number of 0.6" Strands	Ultimate Stress (f_{pu} ksi)	Cross Section Area (A_{ps}-inches^2)	Ultimate Strength ($f_{pu}A_{ps}$-kips)	Prestressing Force — kips			Nominal Weight (lbs./ft.)
					0.80 $f_{pu}A_{ps}$	0.70 $f_{pu}A_{ps}$	0.60 $f_{pu}A_{ps}$	
3-0.6	3	270	0.650	175.5	140.4	122.85	105.30	2.22
4-0.6	4	270	0.868	234.4	187.5	164.10	140.60	2.96
5-0.6	5	270	1.085	293.0	234.4	205.10	175.80	3.70
9-0.6	9	270	1.953	527.3	421.8	369.10	316.40	6.66
12-0.6	12	270	2.604	703.1	562.5	492.20	421.90	8.88
15-0.6	15	270	3.255	878.9	703.1	615.20	527.30	11.10
19-0.6	19	270	4.123	1113.2	890.6	779.20	667.90	14.06

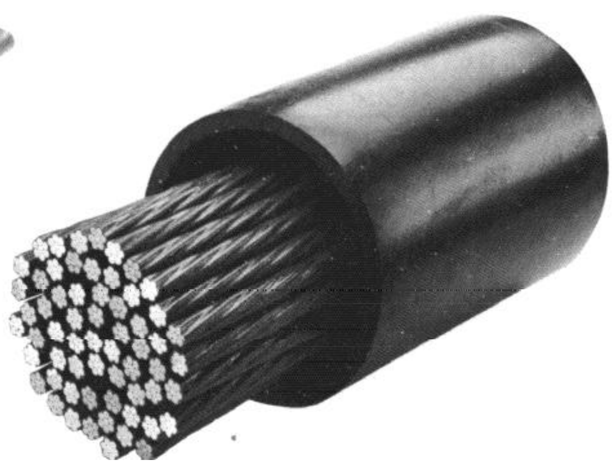
Dywidag Multistrand tendons may be stressed to the allowable limits of ACI 318 and AASHTO specifications. Accordingly, the jacking (temporary) stress may not exceed 0.80 f_{pu} and the transfer (lock off) stress may not exceed 0.70 f_{pu}. The final effective (working) prestress level depends on the specific application, properties of the materials utilized and the characteristics of the structural member as well as the construction procedures.



4-0.6" Multistrand Anchorage (plate type)



4-0.6" Flat Multistrand Anchorage (surface mounting type)



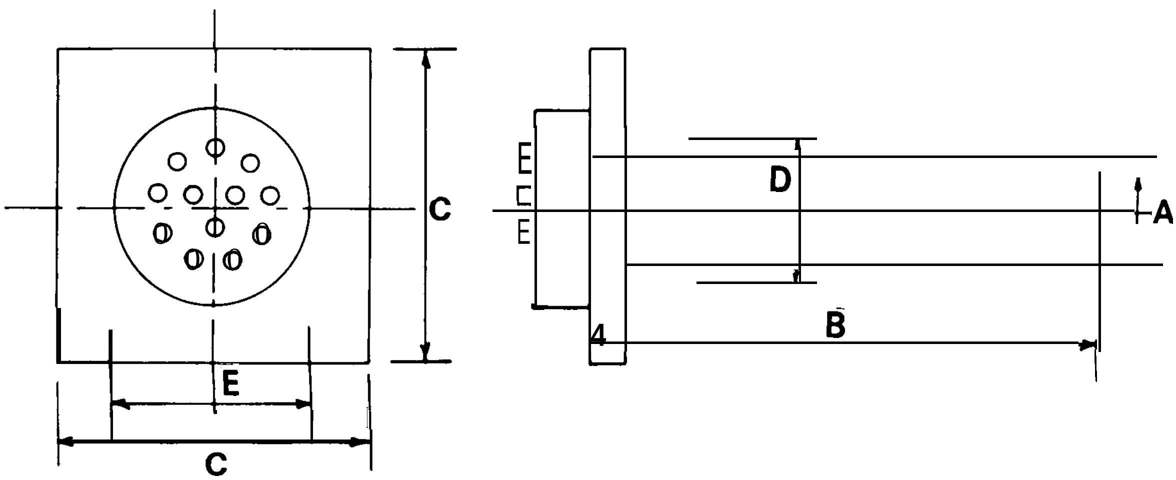
61-0.6" Multistrand Tendon (special cable stay)

DYWIDAG MULTISTRAND POST-TENSIONING SYSTEM

ANCHORAGE DETAILS TYPE P* (PLATE TYPE)

Standard Tendon Size		4-0.6	5-0.6	9-0.6	12-0.6	15-0.6	19-0.6
Number of 0.6" strands		4	5	9	12	15	19
Nominal Duct Diameter (inches)	A	2	2 3/8	2 3/4	3 1/4	3 3/4	4
Bearing Plate Size (inches)	C	7 7/16	3 1/4	11 1/8	12 7/8	14 3/8	16 1/4
Wedge Plate Diameter (inches)	E	5	5 3/8	7 1/8	8	8 7/8	10
Transition Length (inches)	B	3 1/2	9	19	20 1/2	25	30
Maximum Transition Diameter (inches)	D	3 1/2	3 3/4	5 1/4	6 1/4	6 5/8	7 3/4

*Other sizes of bearing or wedge plates are available where lead time permits. Wedge plates may be square or round.



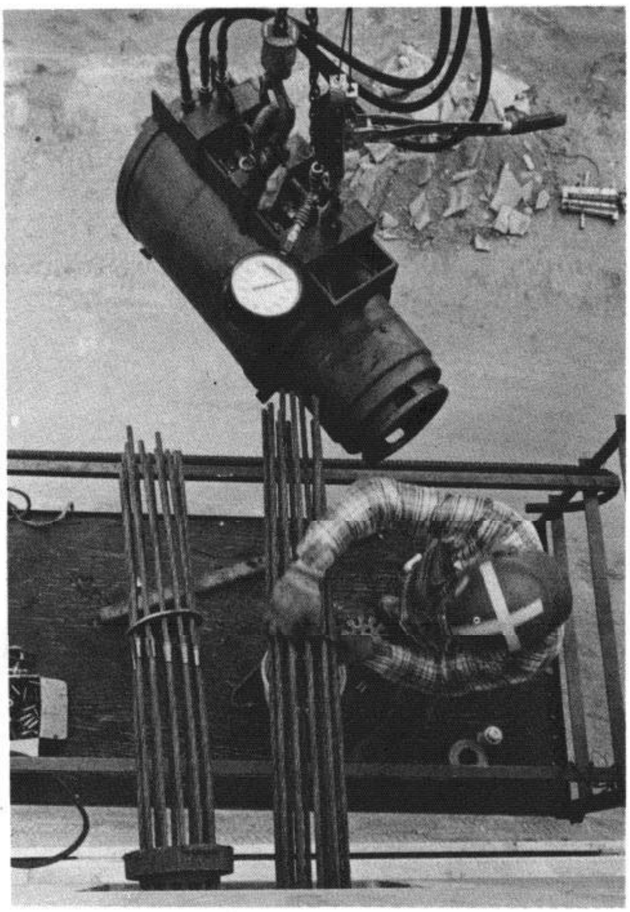
ANCHORAGE DETAILS TYPE S* (SURFACE MOUNTING TYPE)

Tendon Size	3-0.6	4-0.6	4-0.6
No. of 0.6" Strands	3	4	4
Nominal Duct Size (inches)	2 1/4 × 1	3x1	2" Ø
Casting Size (inches)	4x 10	4x 13	5x9
Casting Thickness (inches) (maximum)	2 1/4	2 1/4	2
Trumpet Lennth (inches)	9 1/2	11 1/2	13

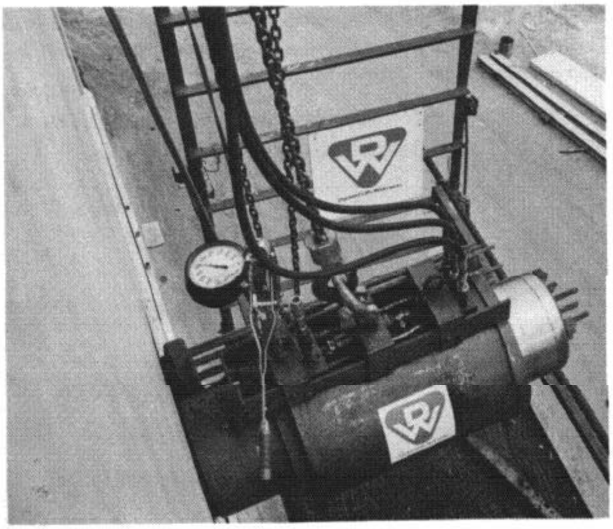
*Anchorage is placed in pre-placed trumpet face after concrete is placed. Pocket former fits surface of trumpet.

DYWIDAG MULTISTRAND INSTALLATION DETAILS

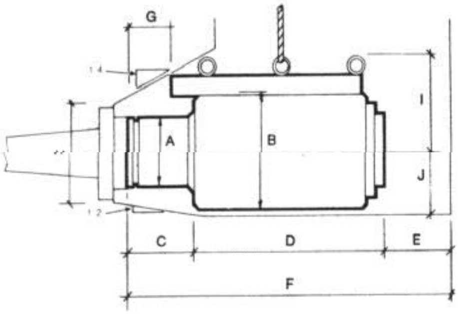
Dywidag spiro duct is manufactured to the requirements of each project. The corrugated duct is available in 2" to 6" diameters in lengths to 40 feet. Duct sections may be easily coupled as needed using corrugated splice sleeves.



Dywidag Multistrand Tendons are typically assembled in pre-placed duct. Strands are inserted into the wedge plates and anchored individually with two part wedges.



All strands comprising a tendon are stressed simultaneously. Strand extensions pass through the hydraulic jack and are secured during stressing by reusable stressing wedges. Anchorage wedges are hydraulically seated securing all strands in wedge plates at a predetermined force and elongation.

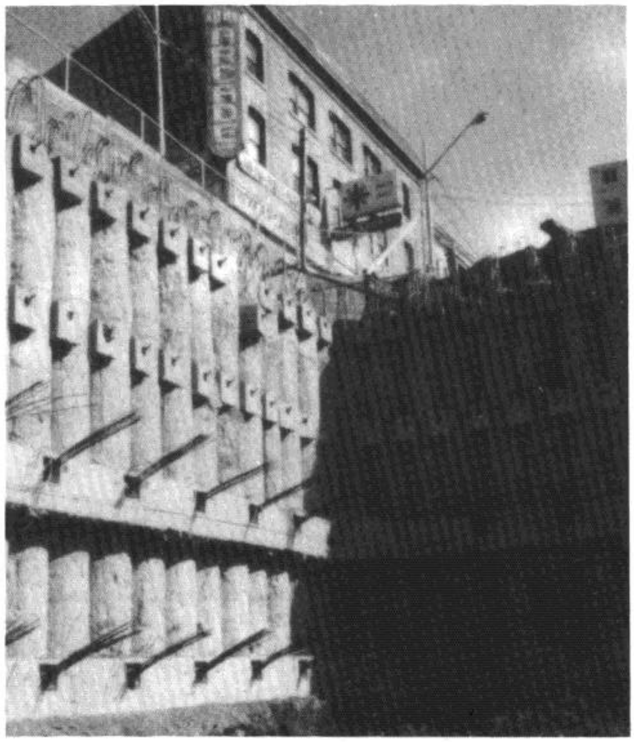


JACK AND CLEARANCE DIMENSIONS (INCHES)

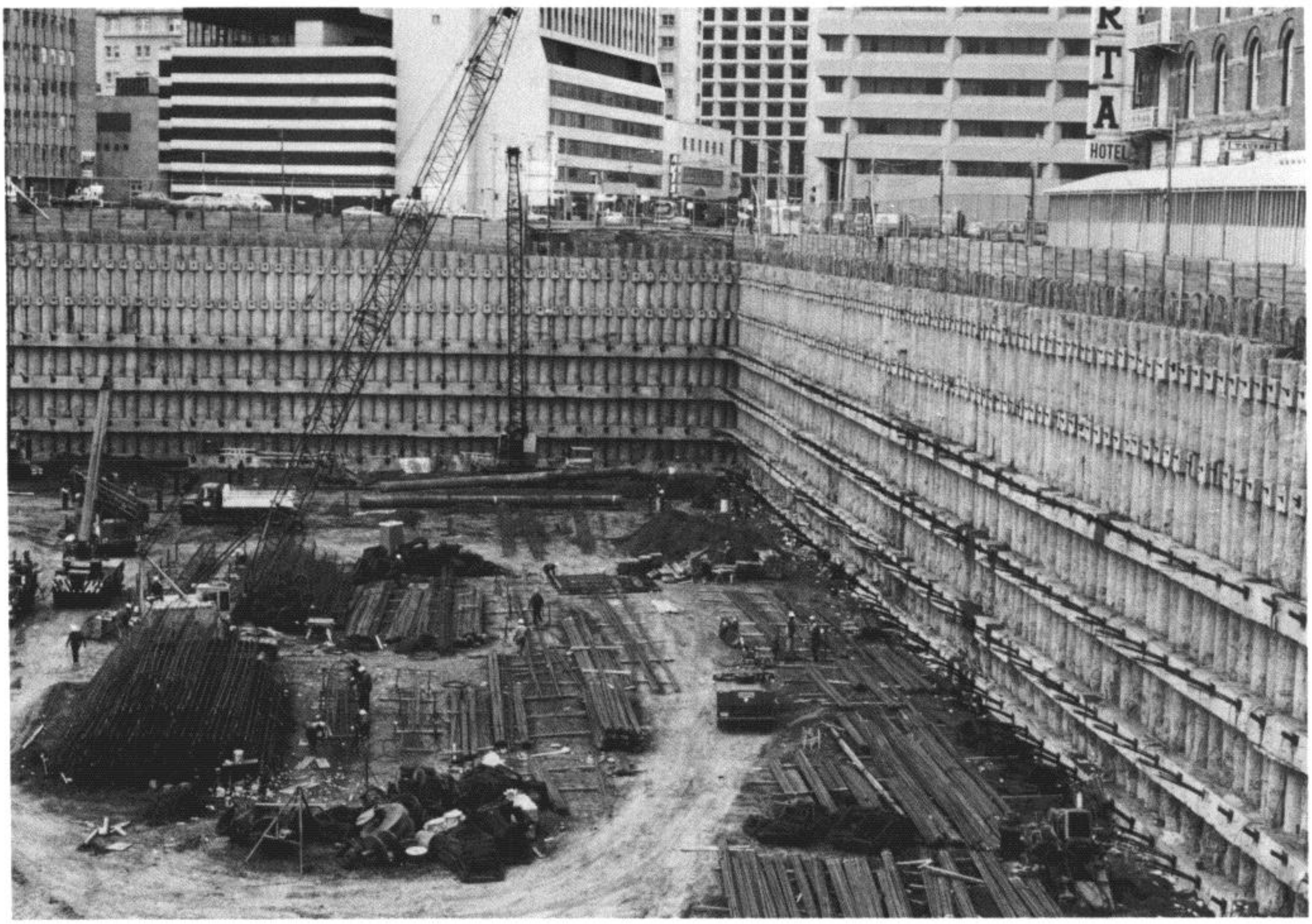
Standard Tendon Size	A	B	C	D	E	F	G	H	I	J
4-0.6	5	9⅜	7	18	30	55	5	8⅜	9 5/16	6½
9-0.6	10	14⅜	9	27	40	76	6	12	11 13/16	9
12-0.6	12	15⅞	14	26	46	86	7	13¾	13 9/16	10
15-0.6	13½	20⅜	14	32	51	97	8	15¼	15 13/16	12
19-0.6	13½	20⅜	13	32	51	96	8	17¼	15 13/16	12

'All available jacks not listed

Four rows of Double Corrosion Protected Dywidag Threadbar Anchors and Double Corrosion Multistrand Anchors were used together for the Excavation at the Convention Center, Edmonton, Alberta, Canada. The 63 foot cut employed a permanent wall constructed of 324 tangent concrete piles. The retention system consisted of 504 1-3/8" single Dywidag Threadbar Anchors and 549 Multistrand Anchors. All anchors utilized the Dywidag Post Grouting System.



Dwyidag Threadbar and Dywidag Multistrand Soil anchors, Convention Center, Edmonton, Alberta, Canada.



DYWIDAG MULTISTRAND ROCK AND SOIL ANCHORS

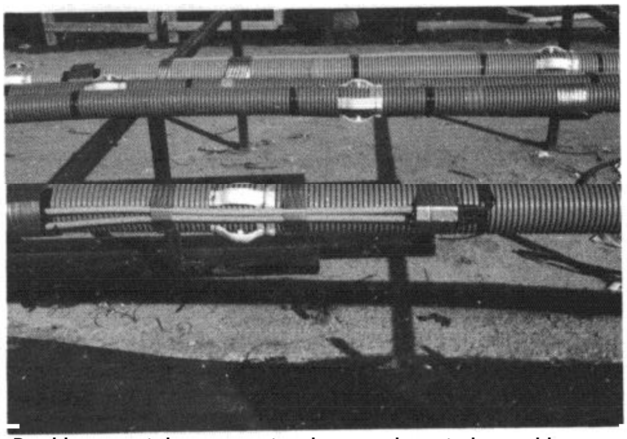
Double corrosion protected Dywidag Multistrand Anchors are recommended for permanent anchor installations or where aggressive soil or rock or stray electrical currents may be encountered.

Corrugated PVC sheathing is installed over the bond length of the anchor. The annular space between the strand and the PVC is filled with grout after installation.

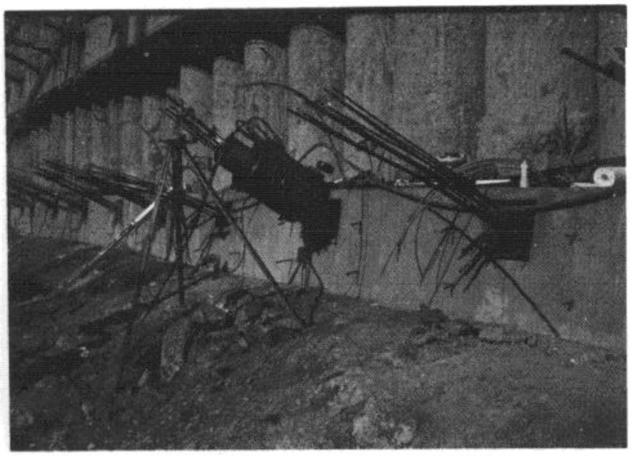
The free stressing length of the strand is inserted in a smooth plastic sheath filled with a mastic corrosion inhibitor. Grout installed in either one or two stages encases all plastic and PVC sheathing.

Dywidag Multistrand Anchors are site fabricated as the bore hole lengths are determined. Grouting of Dywidag Anchors may occur before the anchor head is installed or through the anchor head depending on the installation specifications. A packer may be used to separate the free stressing length from the anchor length where required.

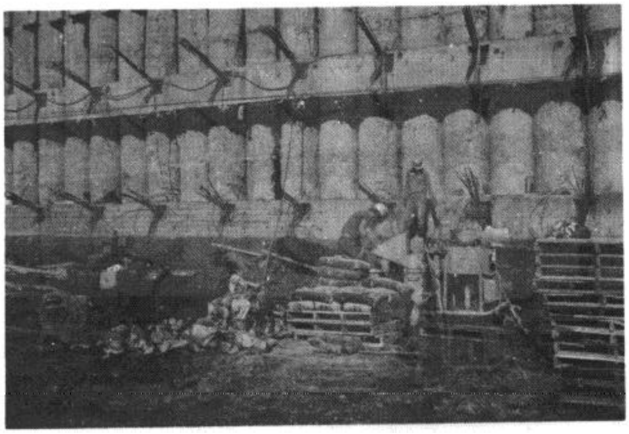
Double corrosion protected Dywidag Multistrand Anchors support tangent pile retention system.



Dywidag grout hose, grout valves, and centering guides are secured to tendon before placement. Post grouting system valves expand and open under pressure to allow regrouting as needsd. System is flushed through vent near anchorage to allow reusage.



Dywidag Multistrand Soil Anchors are stressed with double acting hydraulic lack. Stressing of each tier of anchors preceeds excavation below the level of the anchors to control deformation of the retention system.

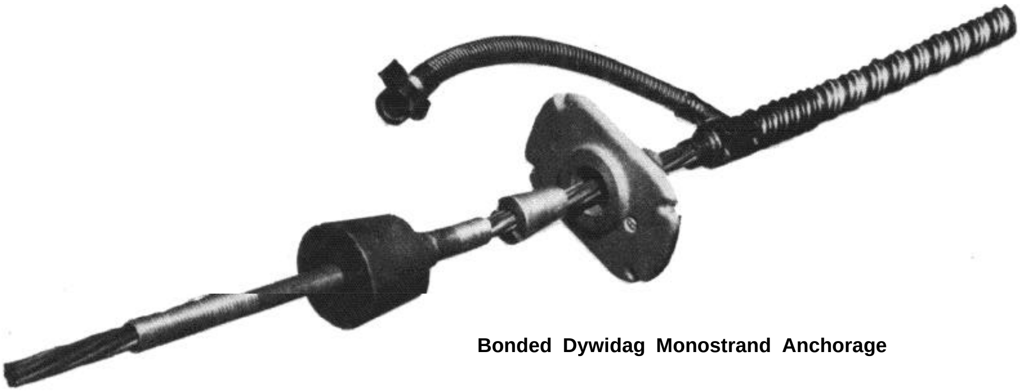


Regrouting of double corrosion protected Multistrand Anchors is accomplished through, the Dywidag Post Grouting System.

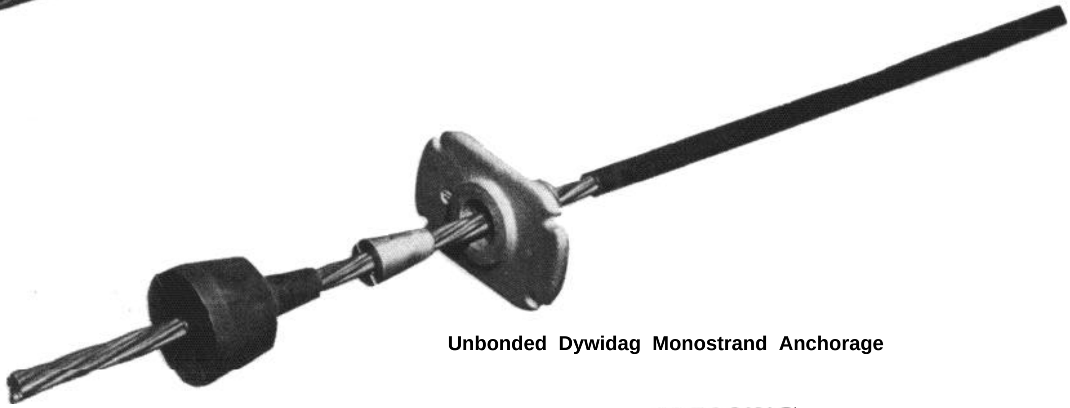
DYWIDAG MONOSTRAND POST-TENSIONING SYSTEM

The DYWIDAG Monostrand System uses 0.6" diameter seven wire 270 K strand. conforming to ASTM A 416. In general, a grouted system with corrugated metal ducts, it is ideally suited for all flat slabs, whether suspended or slabs on ground. The system features easy grout tube connectors. The bearing plate may be either nailed to the form or fastened with a recoverable threaded pipe.

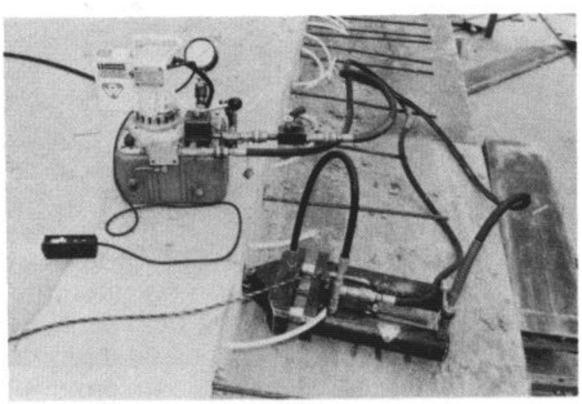
The DYWIDAG Monostrand System can also be used in unbonded applications with greased and smooth P.E. sheathed strand tendons.



Bonded Dywidag Monostrand Anchorage



Unbonded Dywidag Monostrand Anchorage



Dywidag Monostrand Stressing System

PRESTRESSING SYSTEM DATA

Ultimate Stress	f_{pu}	(KSI)	270
Ultimate Strength	$f_{pu}\ A_{ps}$	(KIPS)	58.6
Maximum Overstress Force	$0.8f_{pu}A_{ps}$	(KIPS)	46.9
Maximum Lockoff Force	$0.7f_{pu}A_{ps}$	(KIPS)	41.0
Maximum Effective Force	$0.6f_{pu}A_{ps}$	(KIPS)	35.2
Cross Section Area	A_{ps}	(IN²)	0.217
Nominal Weight		(LBS/FT)	0.74
Bearing Plate Length		(IN)	5¾
Bearing Plate Width		(IN)	3¾
Nominal Duct Diameter		(IN)	1¼
P.E. Sheathing Diameter		(IN)	¾

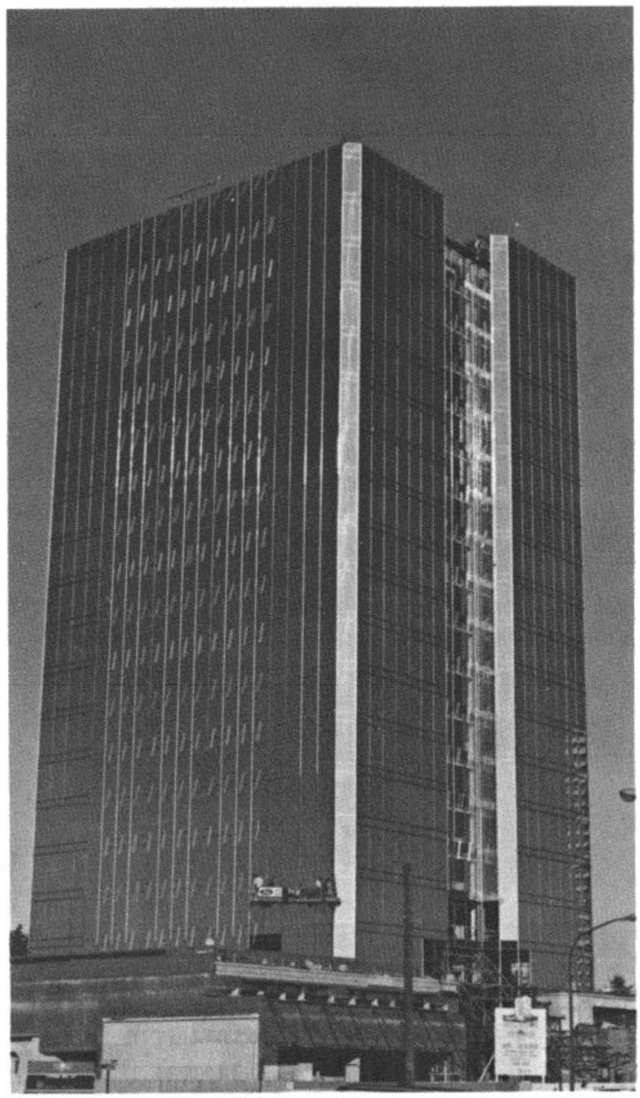
2.2.8 GENSTAR STRUCTURES LIMITED

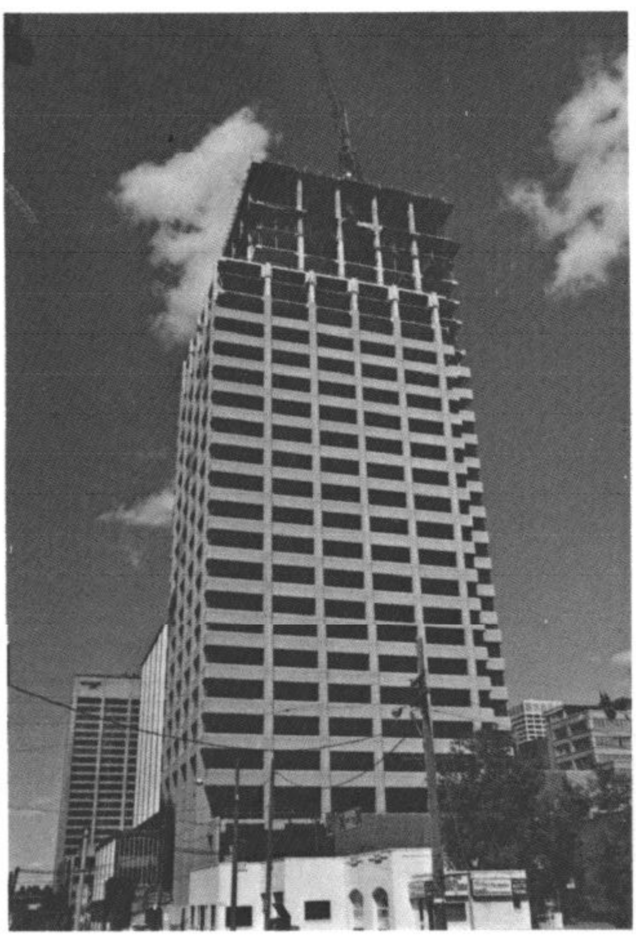
INTRODUCTION

Genstar Structures Limited is a diversified company involved in all aspects of prestressed concrete construction.

Post-Tensioning Services, including both Mono and Multi-Strand systems, are provided through the Post-Tensioning Division.

Through its relationship with Freyssinet International the Post-Tensioning Division provides the knowledge, expertise and technical advantages of one of the world's most respected names in post-tensioned, prestressed concrete.





Services Provided

Genstar Structures Limited, a Company Member of the Post-Tensioning Institute, provides a complete, approved, post-tensioning service, from technical assistance for design engineers through to the finished-in-place post-tensioning system.

MONO-STRAND

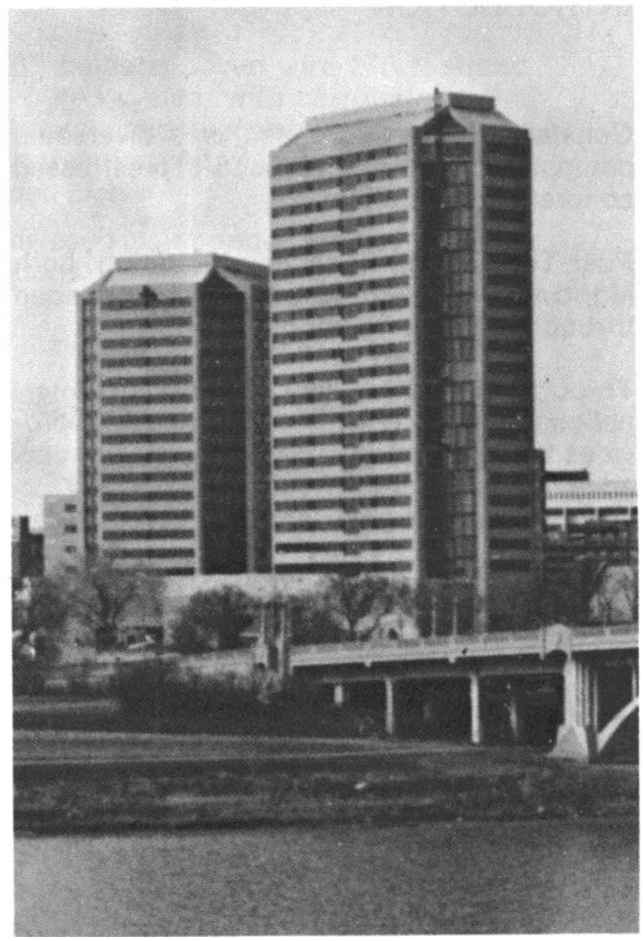
The unbonded system supplied by Genstar Structures Limited is a practical, proven and economical procedure for the introduction of a post-tensioning system into cast-in-place concrete floor slabs.

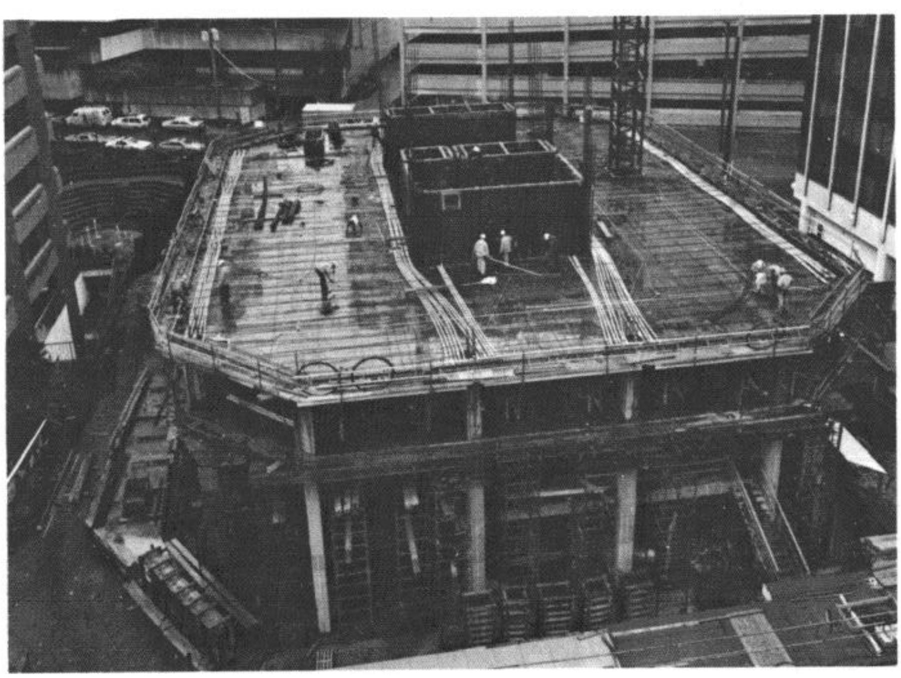
The system incorporates both the Genstar and Titan anchorages and offers versatility to the design engineer by providing a choice of either 13 mm 0 or 15 mm 0 tendons with ultimate capacity of 184.0kn or 261 .0kn respectively.

The assembly consistsof high strength steel strand, coated with rust inhibiting grease and encased with a plastic sheath.

The stressing end incorporates a self-recessing anchorage which is patched after completion of the stressing operation. The patching provides fire and corrosion protection for the anchorage.

All anchorage assemblies conform to the guide specifications for post-tensioning materials as set out by the Post-Tensioning Institute.

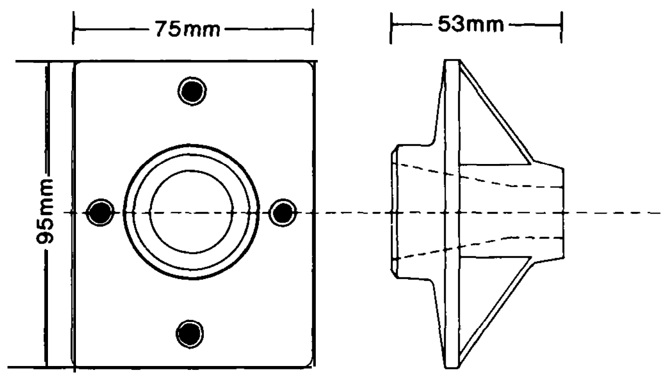




Post-tensioning of concrete slabs offers several distinct advantages: thinner slabs leading to lower structural dead weightandlowerfloortofloor heights; longer spans which provideflexibility in partitioning; moreconvenient parking due tofewercolumns; lower construction costs due to savingsin material, laborand time.

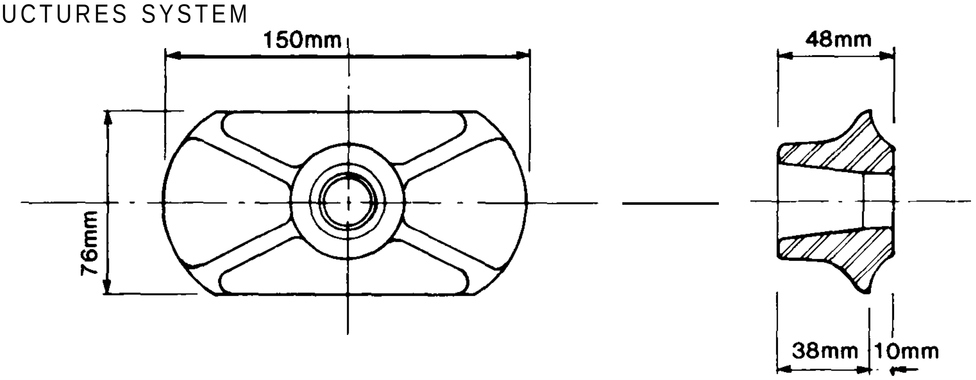
MONO-STRAND ANCHORAGE

TITAN SYSTEM



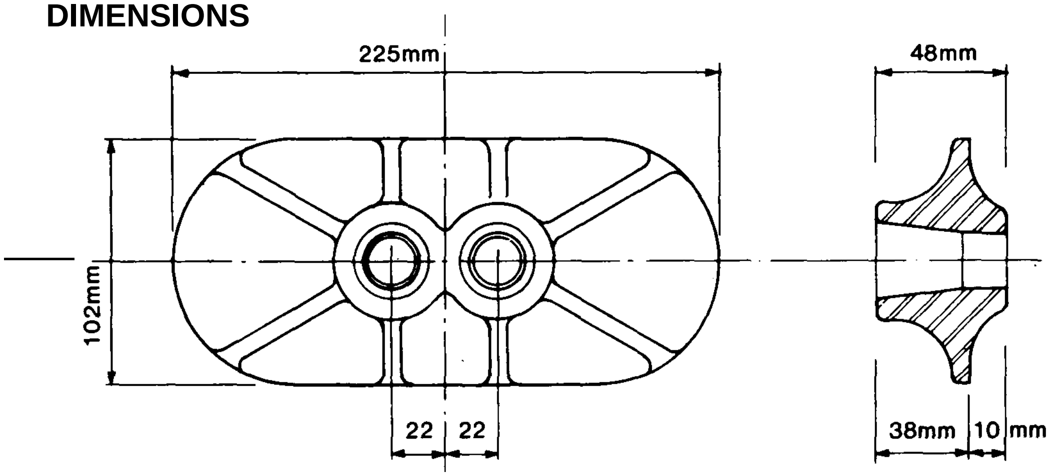
TITAN MI. ANCHORAGE

GENSTAR STRUCTURES SYSTEM



GENSTAR STRUCTURES 1C15 SINGLE ANCHOR

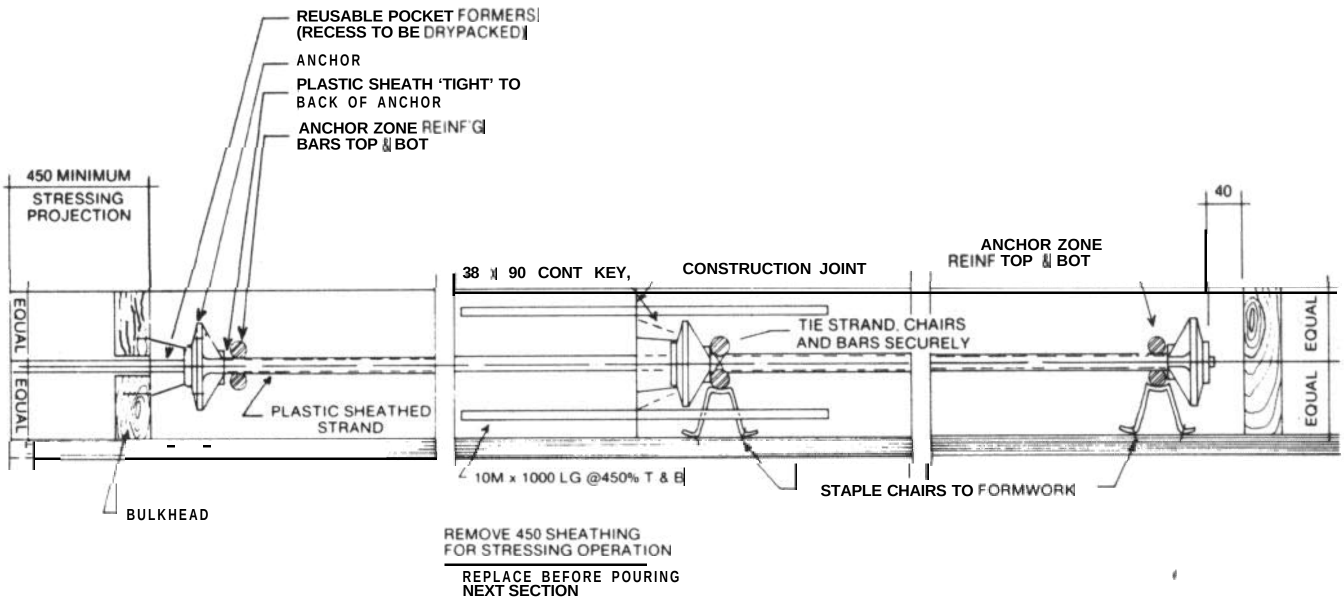
DIMENSIONS



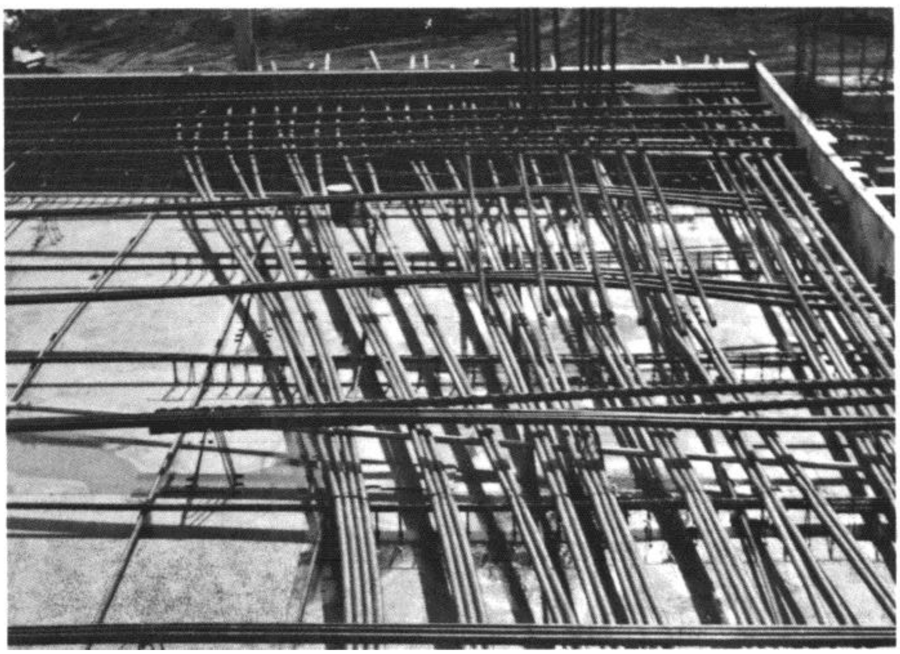
GENSTAR STRUCTURES 2C15 DOUBLE ANCHOR

MONO-STRAND

INSTALLATION DETAILS



STRESSING END - 1 MID POINT ANCHOR - 2 DEAD END - 3



Unbonded tendons in place on a large high-rise office tower project.

The simplicity of the Genstar Structures System has been designed to provide optimum flexibility to the structural engineer and to the general contractor.

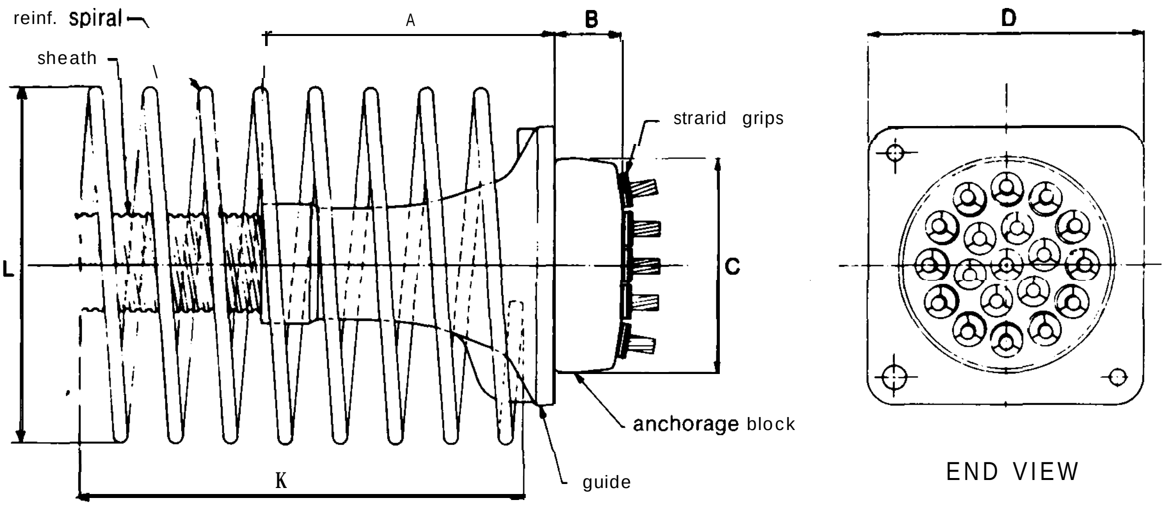
MULTI-STRAND BONDED POST-TENSIONING SYSTEM

Genstar Structures supplies and installs bonded post-tensioning for any application from a single strand tendon up to a 27 strand tendon and in special circumstances up to a 37 strand tendon.

The system employed is the "K-System". This is the most recent system developed by Freyssinet International and is one that has been used very successfully in many different construction applications in various parts of the world.

The method is to install a semi-rigid steel conduit in the structure along with the anchor guides. The tendons are then winched through the conduit. The next step is the pouring of the concrete and after it has obtained the required strength the tendons are stressed using hydraulic jacks. After stressing the conduit is pumped full of a non-shrink grout.

DETAILS OF FREYSSINET ANCHORAGES



STRESSING ANCHOR DETAIL

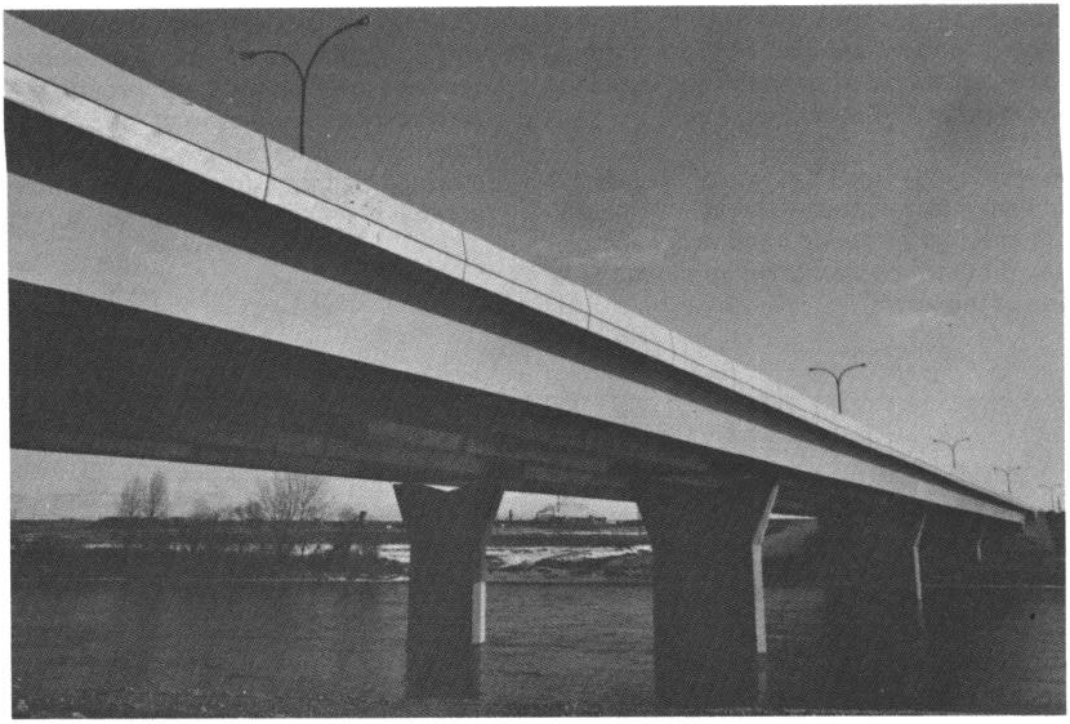
PRINCIPAL DIMENSIONS

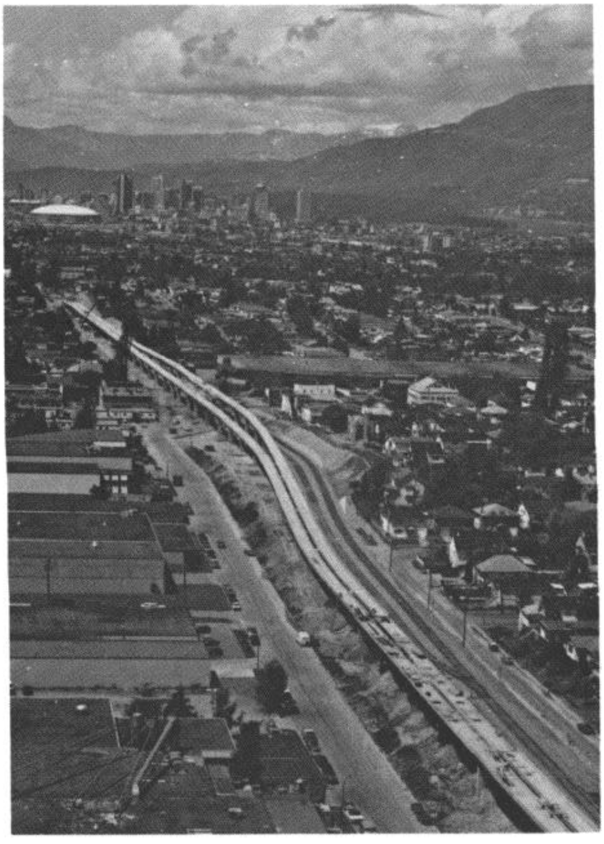
ANCHOR TYPE	NO. OF STRANDS	SHEATH I.D.*	STRESSING ANCHOR DIMENSIONS A MM	B MM	C MM	D MM	REINFORCING SPIRAL DIMENSIONS K MM	L MM	BAR DIA.	NO. TURNS
7K13	5- 7	53	122	51	120	139	260	200	10M	7
12K13	8-12	67	158	57	140	210	266	250	15M	7
19K13	13-19	79	190	60	185	245	266	320	15M	7
4K15	3- 4	53	122	51	120	139	266	200	10M	7
7K15	5- 7	67	158	57	140	210	306	250	10M	7
12K15	8-I 2	79	190	63	162	245	356	280	15M	7
19K15	13-19	102	280	70	216	336	458	388	15M	7

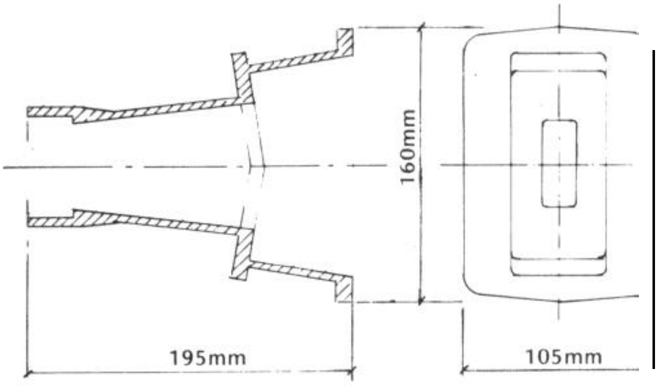
*For outside diameter add 6MM.

LARGER TENDON INFORMATION AVAILABLE ON REQUEST.

MULTI-STRAND







GENSTAR STRUCTURES 2B 13-15 ANCHORAGE

A recent development in bonded post-tensioning is the 2B 13-15 anchorage, specifically designed to be used in conjunction with a flat duct for incorporation into thin structural sections.

The versatility of the anchorage systems supplied by Genstar Structures are easily adaptable to both cast-in-place and precast concrete construction.

2.2.9 LANG TENDONS, INC.

Innovative Manufacturer of Extrusion Coated Strand

Lang Tendons Incorporated is a manfacturer/distributor of extrusion coated strand systems for construction. We concentrate on specialized applications requiring high quality, corrosion protected tendons and specially adapted hardware. Our approach is to work with both engineers and contractors in order to provide products that satisfy the project specifications and the need for productivity.

Lang Tendons Incorporated, formerly known as PIC Incorporated, was founded in 1969 to serve prestressing needs. In 1970 the company's founder invented extrusion coated strand. Over the years the company has developed systems which capitalize on the characteristics of extrusion coated strand, and is now a licensee of the founder's patent.

Specialized POLYSTRAND® Systems

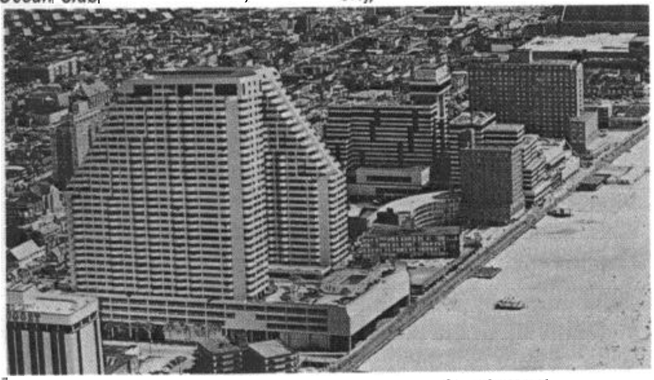
Systems offered by Lang Tendons Incorporated include ground anchors, unbonded post tensioning for cast in place and precast concrete structures, hoops for circular vessels, barrier cables, tension roofs, tendons for concrete slabs or pavements.

Recognized Corrosion Protected Rock and Soil Anchor Systems

Our rock and soil anchor system is specified by engineers to provide required long-term corrosion protection. State highway officials have selected our system for permanent use on Interstate highway projects.

We custom fabricate 0.6" 58K POLYSTRANDS into anchors of whatever length, capacity and level of corrosion protection the ground environment warrants. Our stressing

Ocean Club Condominiums, Atlantic City, NJ



® POLYSTRAND is Lang Tendons' trademark for its extrusion coated 7-wire strand.

Permanent anchors for bulkhead, Port of Panama City, FL

hardware has been engineered and developed for ease and consistency of stressing.

Benefits of Extrusion Coated Strand

All of our strand products are fabricated from extrusion coated 7-wire prestressing strand meeting ASTM A-41 6. Our process covers the strand with a pressure die-formed encasement of corrosion inhibitor and then encapsulates the fully coated strand with a molten polypropylene sheath. As the extruded sheath cools tightly around the encapsulated strand, all voids between the sheath and steel are eliminated and any corrosive elements permanently excluded. With the chemically inert continuous sheath in place, corrosion of the steel cannot take place.

Polypropylene Sheathing

For permanent applications, Lang Tendons Incorporated uses polypropylene for its extruded plastic sheathing because it has excellent resistance to chemicals and abrasion. Furthermore, it has negligible penetration by water and is resistant to environmental stress cracking.

Unbonded Friction Coefficient Guaranteed

POLYSTRAND transmits the post-tensioned force along its length due to the combined effects of the full charge of corrosion inhibitor and the smooth inner surface of the circular, seamless sheath. The coefficient of friction between the strand and the polypropylene sheath is guaranteed to be 0.05 or less. This maximizes force transmission and assures that the design assumptions for the structure are satisfied.

2.2.9 LANG TENDONS, INC.

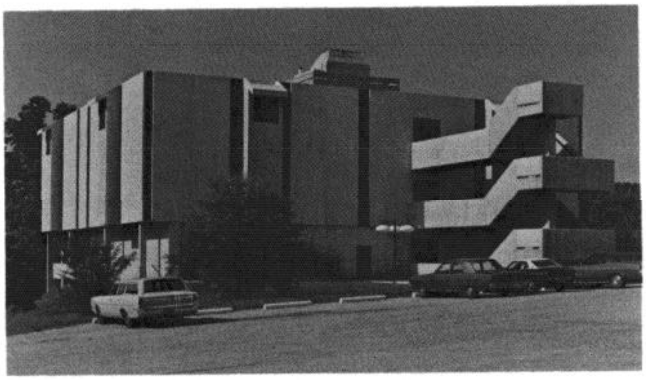
Curved Tendon Alignments

The low coefficient of friction/high force transmission and long term corrosion protection of POLYSTRAND are favorable for use in specialized applications with a high degree of tendon curvature. These include architectural panels, water, sewage and chemical storage tanks, silos, etc.

For such applications our patented LOKCOUPLERS, used to jack one end of a POLYSTRAND against another, allow tendons to be continuous and can be used to eliminate exposed ends.

Architectural Panels

Architectural panels, economically post-tensioned with looped tendon layouts, remain crack free during erection and service and are not subject to spalling or staining due to corrosion. Panels strengthened by post-tensioning have diverse application, such as building cladding or spandrels, tank walls. bridge decks and roadside sound barriers.

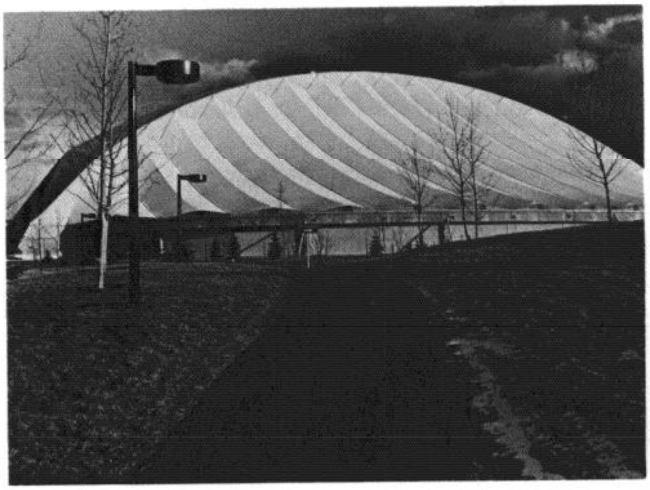


Architectural panels, Community College, Cecil County, MD

Concrete Tanks

Concrete tanks, either cast in place or of precast elements, are made water tight and permanent (even for highly corrosive wastes) by post-tensioning with POLYSTRAND. The PIC developed PRETITE Tank System, using panels post-tensioned together, provides several million gallon containments at very low costs per gallon.

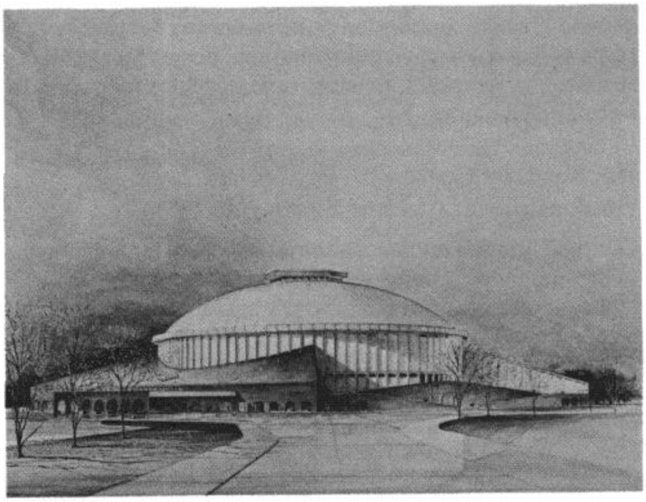
Hoops for sewage tank repair, Bergen County, NJ



Roof tendons, Lindsay Park Centre, Calgary, Canada

Tension Roofs and Cable Stayed Bridges

Due to its corrosion protected performance, low cost, and quick delivery characteristics, POLYSTRAND is an excellent tendon for aerial applications, such as cable domes, fabric tension roofs, and cable stayed bridges. Development of specialized hardware enhances this specialized use of extrusion coated strand.



Cajundome, Lafayette, LA

Crack Free Pavements

Pavements strengthened by post-tensioning with high quality 58K POLYSTRAND provide long life and crack free service. Better performance is achieved in spite of reduced steel, concrete and base material costs. In addition to first cost savings, future maintenance costs are minimized.

Repairs of Cracked Vessels

Cracked wire-wound sewage digesters, rebar reinforced coal silos, etc. have been inexpensively repaired with circumferential hoops of ultraviolet protected POLYSTRAND.

2.2.9 LANG TENDONS, INC.

PDLYSTRAND Permanent Ground Anchors

Extruded POLYSTRAND GROUND Anchors provide surety of corrosion protection to engineers. The following specification is recommended for inclusion in contract documents so that the quality level sought by the designer is achieved.

Specification: Permanent Ground Anchors

3.0 MATERIAL

The material shall be as follows:

3.1 Prestressing steel shall be 7-wire strand, grade 270K, stress relievad or low relaxation steel, ASTM A-416.

3.2 Corrosion Inhibiior shall be as follows:

3.2.1 Drop point 350°F (176.6°C) min. by ASTM D-566,
3.2.2 Flash point 350°F (176.6°C) min. by ASTM D-92,
3.2.3 Water content 0.1% max. by ASTM D-95
3.2.4 Rust test pass 1 ,1 ,1 by ASTM D-95,
3.2.5 Water soluble ions:

Chlorides	10 ppm. max.	by ASTM B-51 2
Nitrates	10 ppm. max.	by ASTM D-992
Sulfides	10 ppm. max.	by APHA 427D (15th ed.)

Corrosion inhibitor shall be Viscosity Oil Co. VISCONORUST 3166 or an approved equal.

3.3 Polypropylene shall be as follows:

3.3.1 Designation grade II 26500D by ASTM D-2146,
3.3.2 Environmental stress crack resistance: No failures at 1,000 hrs. by ASTM D-l 693

Polypropylene shall be Himont USA, Inc. PRO-FAX 6823 or 7823 used singularly or in combination or an approved equal.

4.0 CORROSION PROTECTION

4.1 Corrosion protection shall be determined by whether ground conditions are non-aggressive or aggressive.

Non-aggressive conditions exist when:
1) Ph greater than 4.5,
2) Soil resistivity less than 2,000 ohm-cm,
3) Sulfides present,
4) No recorded deterioration of buried Portland cement concrete structures

Aggressive conditions exist when:
1) Ph less than 4.5,
2) Soil resistivity greater than 2,000 ohm-cm,
3) Sulfides present,
4) Recorded deterioration of buried Portland cement concrete structures

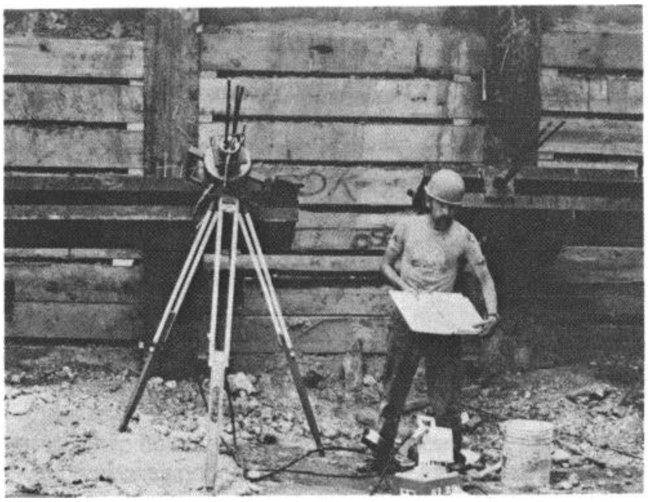
4.2 Corrosion protection for non-aggressive conditions shall be as follows:

4.2.1 Bond length shall be clean, bare prestressing steel and shall have a cover of at least 0.5 inch of Portland cement grout.

4.2.2 Unbonded length shall be fully coated by corrosion inhibitor and then encapsulated by a polypropylene sheathing extruded onto it.

4.2.2.1 Corrosion inhibitor shall fully encapsulate the bare prestressing steel. The film of corrosion inhibitor shall give a circular encasement diameter of 0.005 inch to 0.010 inch greater than the diameter of the bare strand.

4.2.2.2 Polypropylene shall be seamless, melt extruded and shrunk tightly onto the corrosion inhibitor. The sheath shall have a coefficient of friction with the steel of less than 0.05, have a wall thickness of not less than 60 mils and exert a positive pressure on the corrosion inhibitor.



4.2.3 The anchorage shall insulate the tendon from electrical grounding using a method approved by the Engineer.

4.3 Corrosion protection for aggressive conditions shall be as follows:

4.3.1 Bond length shall be clean, bare prestressing steel that is centralized and encapsulated with Portland cement grout inside of a corrugated polypropylene sheath. This sheath shall cover the bonded length plus 2 feet of the unbonded length and shall be covered by at feast 0.5 inch of Portland cement grout.

4.3.2 Unbonded length shall be fully coated by corrosion inhibitor and then encapsulated by a polypropylene sheath extruded onto it.

4.3.2.1 Corrosion inhibitor shall fully encapsulate the bare prestressing steel. The film of corrosion inhibitor shall give a circular encasement diameter of 0.005 inch to 0.010 inch greater than the diameter of the bare strand.

4.3.2.2 Polypropylene shall be seamless, hot melt extruded and shrunk tightly onto the corrosion inhibitor. The sheath shall have a coefficient of friction with the steel of less than 0.05, have a wall thickness of not less than 60 mils and exert a positive pressure on the corrosion inhibitor.

4.3.3 The extruded polypropylene shall have a watertight seal with the base of the anchor head, using a method approved by the Engineer.

5.0 QUALITY CONTROL

5.1 Manufacturer of the permanent ground anchors shall certify that the requirements of this specification have been satisfied.

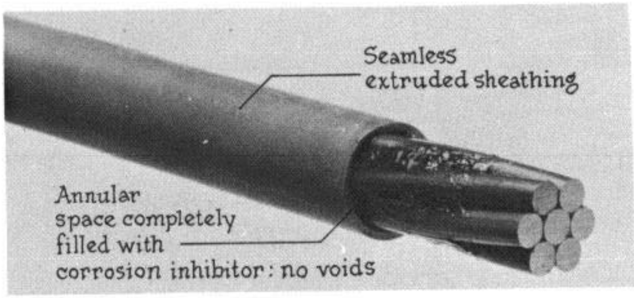
5.2 Field inspection shall occur to satisfy the following:

5.2.1 Full coating with corrosion inhibitor; visually determine smoothness of extruded polypropylene sheath. Reject if the helical lay of the strand wire is seen as relief on the exterior of the polypropylene sheath.

5.2.2 Minimum wall thickness of polypropylene sheath; random maasurement of the outside diameter of the polypropylene sheath. Reject if diameter is less than diameter of bare prestressing steel plus 0.010 inch plus 2 times the specified wall thickness.

5.2.3 Damage to the polypropylene sheath; visually examine polypropylene sheath for presence of corrosion inhibitor, remove corrosion inhibitor and examine for sheath damage. Repair polypropylene sheath in accordance with the recommendations of the manufacturer.

2.2.9 LANG TENDONS; INC.



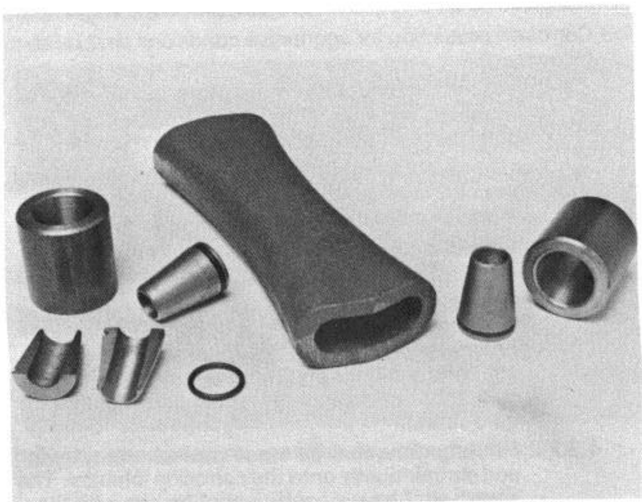
Monostrand and Multi-strand Hardware

Lang Tendons offers a full range of anchorage hardware and stressing equipment for its POLYSTRAND systems. Representative parts are pictured below. Large capacity hardware and jacks for multi-strand tendons and anchors are also available.

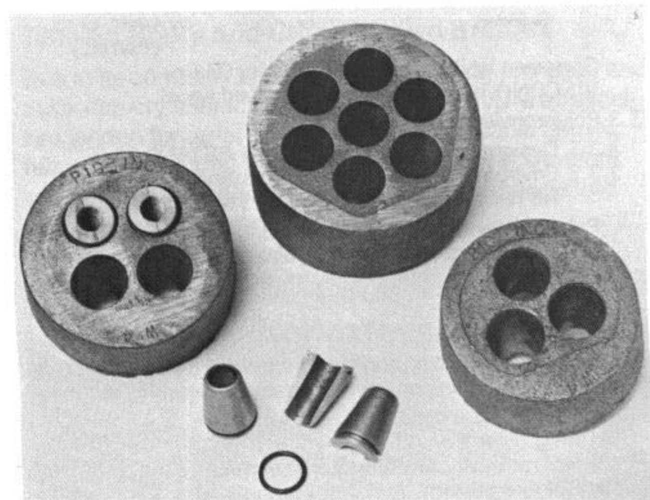
Specification of Strand Strength Properties (ref. ASTM A-416)

The 7-wire grade 270 steer strand shall have strength as follows:

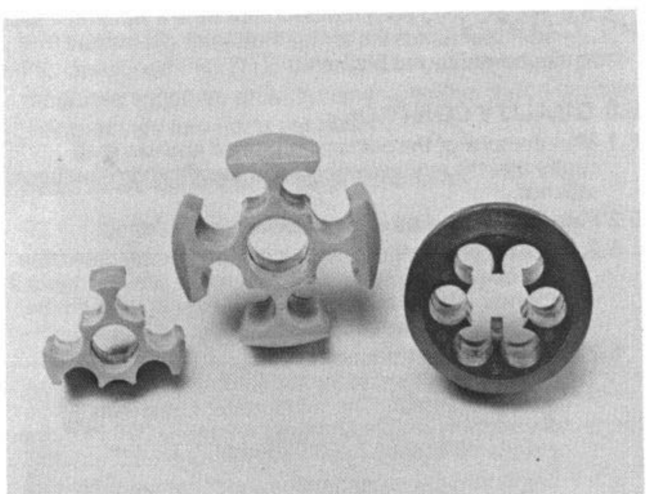
Strand Diameter	0.5 inch	0.6 inch
Guaranteed Ultimate Strength	41.3 kips	56.6 kips
Maximum Temporary Force (80% of Ultimate)	33.0 kips	46.9 kips
Design Force (60% of Ultimate)	24.9 kips	35.2 kips
Cross Sectional Area	0.153 in.^{2}	0.217 in.^{2}
Modulus of Elasticity	28×10^6 psi	28×10^6 psi
Weight of Steel	0.52 lb/ft	0.74 lb/ft



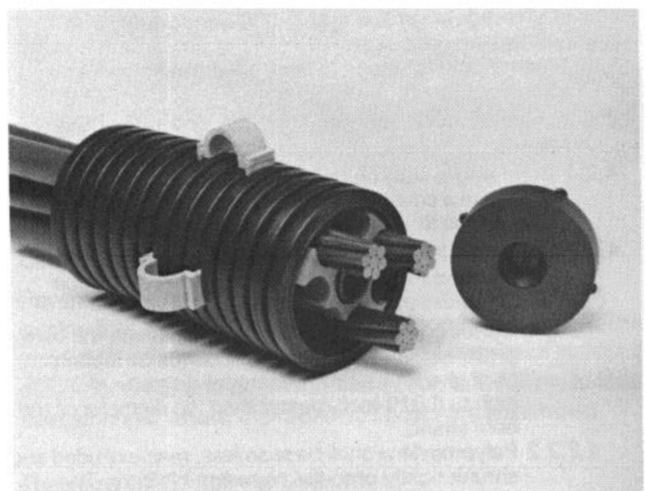
LOKCOUPLER® (patented) and Anchors for end-to-end jacking L to R: PIN 614-Body, PIN 602 Wedge, PIN 401-LOKCOUPLER, etc.



Multi-strand Anchorheads for 58K POLYSTRAND® L to R: P/N 641, 671,631 and P/N 602-wedge



Ground Anchor Spacers L to R: P/N 811, 812, 814



Encapsulation for Corrosion Protection of Grouted Anchors CORFLO Sheath, with P/N 838-Centralizers, and P/N 836-End Cap

2.2.10 # LINDEN POST-TENSIONING

A Ceco Industries Company

LINDEN POST-TENSIONING CORPORATION

Linden Post-Tensioning Corporation is a designer and a fabricator of post-tensioning systems. The company performs preliminary designs and assists engineers of record with final designs for reinforced concrete structures. Linden assists the design/build developer and/or the contractor and is able to bid pre-designed structures competitively. Linden, headquartered in Atlanta, Georgia, is strategically located to service the eastern, southeastern and midwestern regions of the United States.

Linden Post-Tensioning is a subsidiary of Ceco Industries. Through its subsidiaries, Ceco is the nation's leader in concrete construction services and is a manufacturer of building products for the construction industry.

THE MONOSTRAND SYSTEM

Tendons and Anchors

Monostrand tendons use ½" diameter seven-wire strand in single or multiple groups. The strand is stress relieved* and conforms to ASTM A-416. The Monostrand System features are:

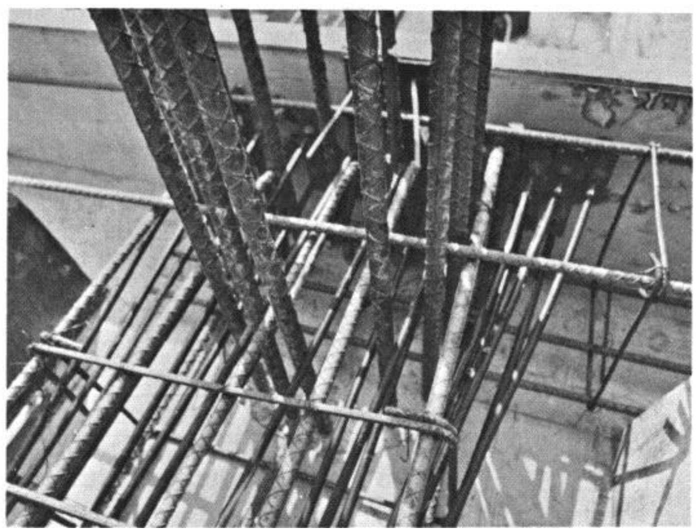
- Rectangular anchor plates that provide flexibility to meet the restrictions of thin slabs or narrow beams.
- Fixed end anchorages that are attached and pre-set at the factory to reduce field labor and prevent strand slippage at the fixed end.
- Tension that can be applied separately to successive sections of the total length by stressing at any intermediate point, and then continuing the same strand, eliminating the need for coupling.
- Reusable pocket formers that come with anchor. The entire assembly attaches to the formwork, and when removed after concreting, leaves a small, clean stressing pocket that is easily patched,
- Stressing equipment that is small and lightweight, easily handled by one man. Operation is semi-automatic and fast.

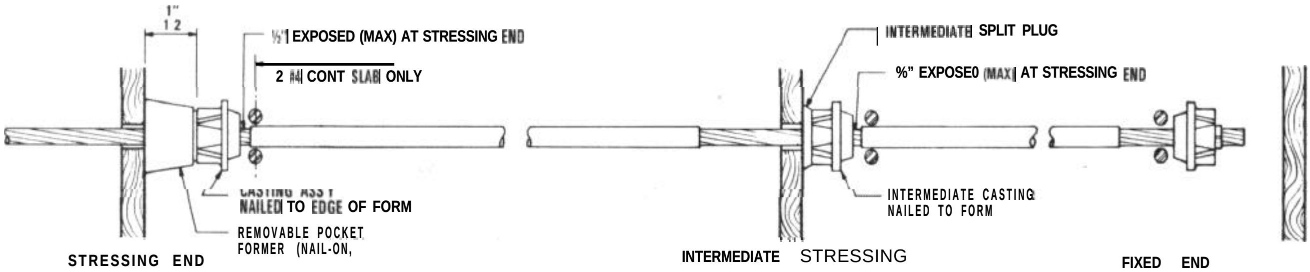
The advantages of the Monostrand Post-Tensioning System place the flexibility of prestressing within the scope of any project. With this flexibility, long spans, cantilevers and heavy loading are no longer a problem.

Design Data

Strand Area	.153 in^2
Maximum Jacking Force 0.80 f_{pu}	33.0 Kips
Maximum Anchoring Force 0.70 f_{pu}	28.9 Kips
Maximum Effective Force 0.60 f_{pu}	24.8 Kips*

*NOTE: Low relaxation strand information available upon request.



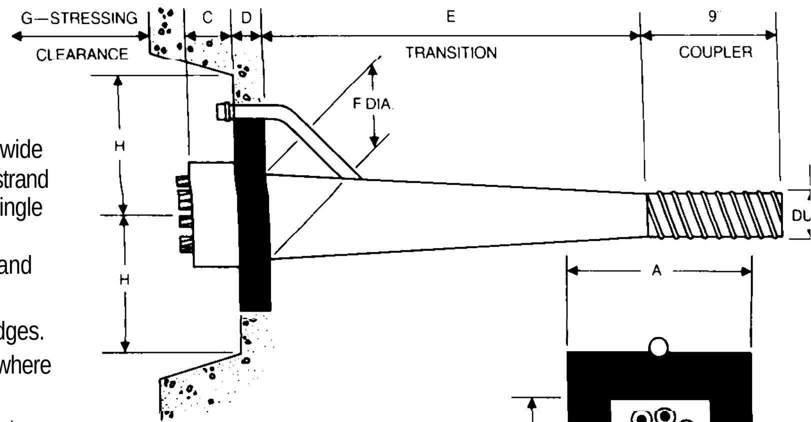


FORMING DETAILS FOR NAIL-ON POCKET FORMER SYSTEM

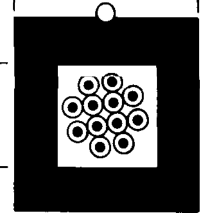
THE **MULTISTRAND** SYSTEM

The Multistrand System provides post-tensioning for a wide range of force requirements from the 297 KIP* twelve strand system to the 694 KIP* twenty-eight strand system in single strand increments. This system is commonly used with grouted tendons. The many applications of the Multistrand System include:

- Cast-in-place and precast, continuous long span bridges.
- Beams or transfer girders in commercial structures where long spans and heavy loadings are present.
- Tension rings used to support domes or arches in all types of construction.
- Tanks or containment structures.



ANCHORAGE ASSEMBLY ELEVATION



ANCHORAGE END VIEW

ANCHORAGE ASSEMBLY		12s	16s	20s	24s	26s
Number of ½" Dia. 270ᵏ Strands		12	16	20	24	28
Strand Area (Based on 0.153 in*)		1.836	2.448	3.060	3.672	4.284
Maximum Jacking Force 0.80 f_{pu}	(Kips)	396	529	661	793	925
Maximum Anchoring Force 0.70 f_{pu}	(Kips)	347	463	578	694	810
*Maximum Effective Force 0.60 f_{pu}	(Kips)	297	396	496	595	694
Rigid Duct Diameter	O.D. (In)	2%	3	3 9/16	3 9/16	3 15/16
Square Bearing Plate	A (In)	10%	13	14¼	15¼	16%
Square Wedge Plate	B (In)	6	8	8	8	8
Wedge Plate Thickness	C (In)	2¾	3	4	4	4
Bearing Plate Thickness	D (In)	1½	1¾	2	2 %	2¾
Transition Length	E (In)	22	35	35	35	31
Maximum Transition Diameter	F (In)	4 %	6¾	6¾	6¾	6¾
Stressing Clearance	G (Ft)	8′-0″	8′-0″	8′-0″	8′-0″	8′-0″
Stressing Clearance	H (In)	12	12	12	12	12
Stressing Equipment		200T	500T	500T	500T	500T

*NOTE: Forces shown are computed based upon stress-relieved wire Forces for low relaxation strand are available upon request

LINDEN POST-TENSIONING CORPORATION
A Ceco Industries Company

2.2.11 NATIONAL POST-TENSIONING SERVICES, INC. & POST-TENSION OF TEXAS, INC.

MANUFACTURING FACILITIES HOUSTON, TEXAS

NATIONAL POST-TENSIONING SERVICES, INC. and its subsidiary, POST-TENSION OF TEXAS, INC. have been instrumental in the introduction of the post-tensioning application for slab-on-grade construction throughout the country. Our continuing effort to improve and refine this application, both internally and through active promotion of the post-tensioning industry, since 1973, have produced an extremely viable and economical post-tensioning system with superior quality and service our trademark.

We are recognized innovators and through our depth of successful experience in slabs-on-grade, commercial foundation and elevated structures we have maintained rapid and steady growth, not only in volume, but in recognition as a leader in the post-tensioning industry.

We maintain production of quality components for our system utilizing our own manufacturing facilities, To further extend and maintain quality on the job site, we directly employ and train our own field personnel for installation and stressing operations.

Through our professional and technically trained representatives we act as liaison between the owner, the contractor, the engineer and the architect in order to provide increased understanding and effective use of the structural, aesthetic, and economic advantages offered by Post-tensioning.

PRODUCTS AND SERVICES

The NATIONAL POST-TENSIONING SERVICES, INC. and POST-TENSION OF TEXAS, INC. system is highly adaptable utilizing 1/2", 7/16", and 3/8" diameter 270K stranded tendons in conjunction with the PTI-5 anchorage system.

Flexibility of our system is illustrated by its use in single family foundations, multi-family foundations, industrial floors, commercial foundations, tennis courts, tilt wall panels, parking areas, pipe storage racks, elevated structures, highway paving, and bridge structures throughout the United States and abroad.

Reliability of our system is proven by the fact that we have furnished materials for use in over 300 million square feet of concrete construction.

2.2.11 NATIONAL POST-TENSIONING SERVICES, INC. & POST-TENSION OF TEXAS, INC. — continued

NATIONAL POST-TENSIONING SERVICES, INC. and POST-TENSION OF TEXAS, INC. offer the following services:

- Feasibility studies
- Preliminary design lay-out including preparation of placement drawings
- Technical design and specifications assistance
- Design through independent registered professional engineers
- Stressing calculations
- Three day delivery
- Trained field personnel for installation of cables
- Trained field personnel for stressing of cables
- Technical assistance at job site
- Guaranteed cost estimates

UNBONDED TENDONS

We produce our tendons under a quality conscious program which incorporates the highest quality materials available, applied in workmanship and quality to satisfy the most rigorous requirements. Our system utilizes high strength 270Kstrand meeting all ASTM A-416 latest revised specifications.

We offer both stress relieved 270K strand and low relaxation 270K strand. The latter, through its heat treating process, offers a greater final force through reduced post-stressing relaxation.

We completely encase the strand with a lithium base grease to prevent corrosion and to reduce friction at time of stressing. The greased strand is coated with an extruded jacket of high density polyethelene which provides a durable protective sheathing to reduce fricftion and prevent bonding of the strand with concrete.

Recent friction tests have shown our product to have characteristics which would allow longer than normally accepted single end stresses. This provides an excellent design alternative to the generally accepted maximum of 100 feet for a single end stress.

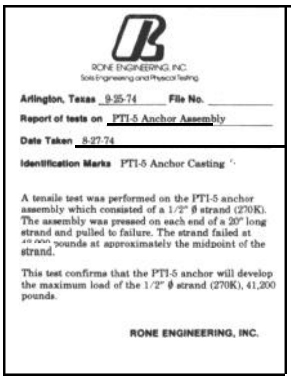
The properties listed below are nominal or minimum ASTM A-416 specifications. Actual production will vary, with actual values normally greater than those listed. Certified mill tests are available for all material furnished by us.

270K STRAND VALUES	1/2"	7/16"	3/8"
ULTIMATE BREAKING STRENGTH	41.3 KIPS	31.0 KIPS	23.0 KIPS
YIELD AT 1% EXTENSION	35.1 KIPS	26.4 KIPS	19.6 KIPS
TEMPORARY FORCE	33.0 KIPS	24.8 KIPS	18.4 KIPS
STRESSING FORCE	28.9 KIPS	21.7 KIPS	16.1 KIPS
DESIGN FORCE	24.8 KIPS	18.6 KIPS	13.8 KIPS
ELONGATION AT STRESSING FORCE	0.00680 in/in	0.00680 in/in	0.00680 in/in
AREA SQUARE INCHES	0.153 sq. in.	0.115 sq. in.	0.085 sq. in.
WEIGHT PER 100 FEET	520 lbs.	390 lbs.	290 lbs.
MODULUS OF ELASTICITY	28,000,000 PSI	28,000,000 PSI	28,000,000 PSI

PTI-5 ANCHORAGE SYSTEM

Cur PTI-5 Anchorage System is the result of our continued effort to produce the most reliable and adaptable anchorage possible. The PTI-5 Anchorage System utilizes a high grade ductile iron cast anchor and 4 degree wedges. The PTI-5 anchor is interchangeable with our 1/2", 7/16" and 3/8" wedges.

The PTI-5 Anchorage System is in strict accordance with specifications of the Post-Tensioning Institute's Post Tensioning Manual; Tentative Recommendations for Concrete Members Pre-stressed With Unbonded Tendons - ACI Journal, February, 1969; ACI Standards 318-71; Federal Housing Administration Minimum Property Standards; and, has exceeded the testing criteria of static, dynamic, and cyclic loading conditions tests as per the following certified results:


RONE ENGINEERING, INC.
Soils Engineering and Physical Testing

Arlington, Texas 9-25-74 File No.

Report of tests on PTI-5 Anchor Assembly

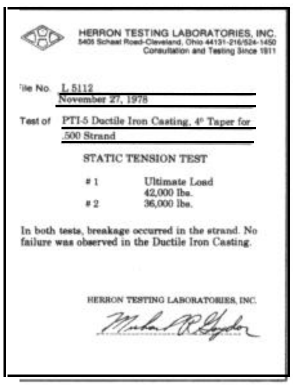
Date Taken 8-27-74

Identification Marks PTI-5 Anchor Casting

A tensile test was performed on the PTI-5 anchor assembly which consisted of a 1/2" ∅ strand (270K). The assembly was pressed on each end of a 20" long strand and pulled to failure. The strand failed at 42,000 pounds at approximately the midpoint of the strand.

This test confirms that the PTI-5 anchor will develop the maximum load of the 1/2" ∅ strand (270K), 41,200 pounds.

RONE ENGINEERING, INC.


HERRON TESTING LABORATORIES, INC.
5405 Schaaf Road-Cleveland, Ohio 44131-216/524-1450
Consultation and Testing Since 1911

File No. L 5112
November 27, 1978

Test of PTI-5 Ductile Iron Casting, 4° Taper for .500 Strand

STATIC TENSION TEST

# 1	Ultimate Load 42,000 lbs.
# 2	36,000 lbs.

In both tests, breakage occurred in the strand. No failure was observed in the Ductile Iron Casting.

HERRON TESTING LABORATORIES, INC.


DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
REGIONAL OFFICE
1100 COMMERCE STREET
DALLAS, TEXAS 75202

REGION VI

July 10, 1975

IN REPLY REFER TO:
6FA

Post-Tension of Texas, Inc.
915 Crosstimbers
Houston, Texas 77022

Gentlemen:

Subject: Post-Tensioned Anchorage System
Post-Tension of Texas, Inc.—Anchor PTI-5

We are attaching a copy of the memorandum of acceptance on the above referenced system.

Sincerely,

for Guy J. Seghers, Jr.
Assistant Regional Administrator for
Housing Production and Mortgage Credit

Attachment


sps Laboratories
A SERVICE OF sps TECHNOLOGIES

Highland Avenue
Jenkintown
Pennsylvania 19046
215-884-7300
TWX: 510-665-1718
Telex: 83-4254

REPORT OF TEST DATA

Date 9/21/78

Part No. PTI-5 Anchor Wedge Casting

Purchase Order No. 3461

TESTS	RESULTS
Dynamic (fatigue) test assembly block for mono strand wedge anchor testing using a 5-8 anchor assembly.	500,000 Cycles— No Failure
Testing performed on a 250,000 lb. Amsler Universal testing machine.	
Test load: 27,300 lbs. max. 24,800 lbs. min.	
Test Speed—750 cycles per minute	

P-I-I-5 ANCHOR AND 4" WEDGE

FACTORY SEATED DEAD END PI-I-5

APPLICATION AND MEASUREMENT OF STRESSING LOADS

NATIONAL POST-TENSIONING SERVICES, INC. and POST-TENSION OF TEXAS, INC. technically train and solely employ all stressing crews. Our crews areequipped to properly apply tension, professionally make field repairs, and acceptably report and resolve any problems encountered.

Our twin stroke stressing ram is of our own design and manufacture, with readily interchangeable components which provide a durable, and reliable unit to complete your stressing requirements.

As per recommendations of the Post-Tensioning Institute Manual, stressing may be monitored by either translating the gauge pressure of the stressing unit into force transmitted at the anchorage, or by calculation of theoretical elongation. We must be notified prior to contract if elongation measurement is required. Elongation may be calculated as follow:

$$\text{ELONGATION (inches)} = \frac{PL}{AE} \text{ with}$$

P= Prestress force in pounds per square inch (normally 70% of ultimate)
L= Length of tendon in inches (normally form to form)
A= Cross-sectional area of steel in square inches
E= Modulus of elasticity of steel in pounds per square inch

Actual physical properties of strand vary from production to production. These must be obtained from Certified Mill Test reports furnished by the producer. Acceptable deviations in computed and actual elongation are in excess of ± 8%.

Stressing may be safely completed when concrete attains 1800 PSI strength.

STRESSING LOAD POUNDS TO HYDRAULIC PRESSURE

Gauge Reading	Ram Area	Strand Load
1000 PSI	4.47 sq. in.	4.47 KIPS
2000 PSI	4.47 sq. in.	8.94 KIPS
3000 PSI	4.47 sq. in.	13.41 KIPS
4000 PSI	4.47 sq. in.	17.88 KIPS
5000 PSI	4.47 sq. in.	22.35 KIPS
6000 PSI	4.47 sq. in.	26.82 KIPS
7000 PSI	4.47 sq. in.	31.29 KIPS
7500 PSI	4.47 sq. in.	33.53 KIPS
8000 PSI	4.47 sq. in.	35.76 KIPS
9000 PSI	4.47 sq. in.	40.23 KIPS
10000 PSI	4.47 sq. in.	44.70 KIPS

TWIN STROKE RAM AND HYDRAULIC PUMP

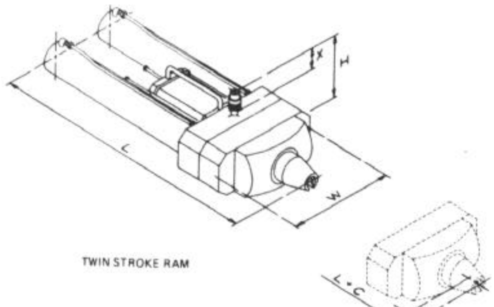


TABLE OF CLEARANCE

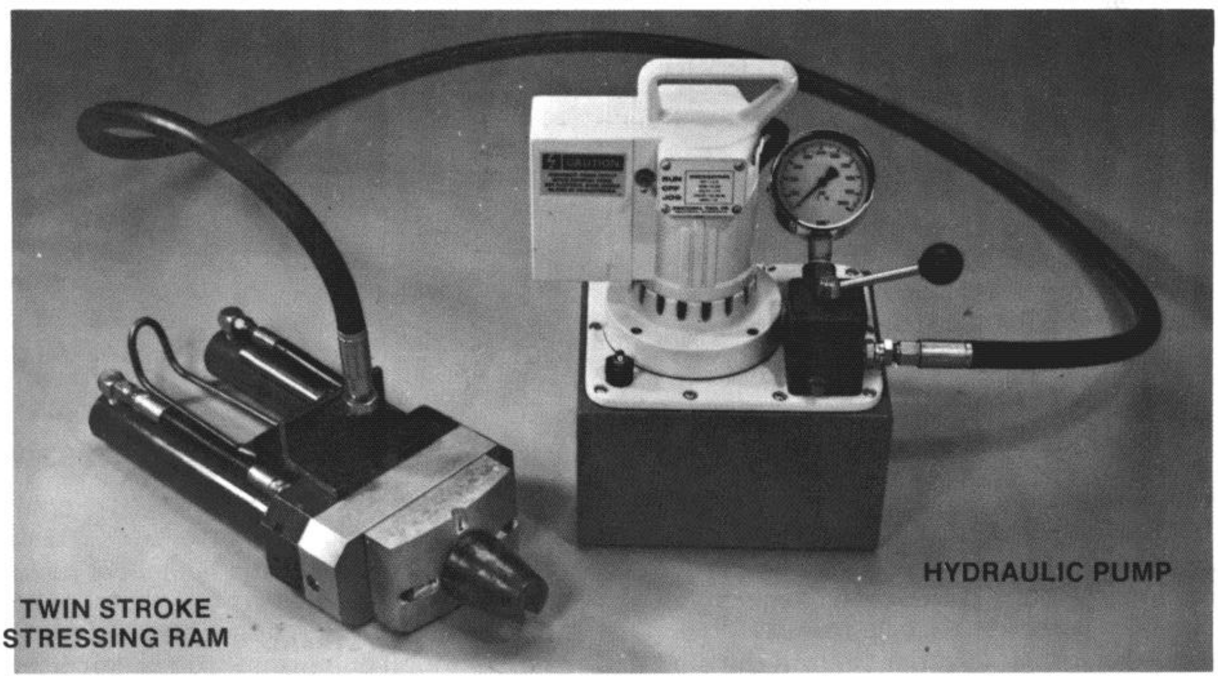
Gross Weight — 56 lbs.

L	Maximum Length	1'-8-3/8"
W	Maximum Width	9"
H	Maximum Height	6-1/4"
C	Minimum Required Clearance (10-1/8" Stroke Plus 1/2" Clearance)	2'-7"
X	Hydraulic Hose Coupling (Height)	2-1/4"
Y	Conical Pocket Former Length (Reference Dimension)	1.6"

STRESSING CHART

Ram Area — *4.47* sq. in.

STRAND SIZE	GAUGE PRESSURE	STRAND LOAD	TYPICAL ELONGATION AT STRESSING LOAD
3/8"	Temporary Load 80% 4116	18.4 KIPS	
	Stressing Load 70% 3602	16.1 KIPS	.00680 in/in
	Design Load 60% 3087	13.8 KIPS	
7/16"	Temporary Load 80% 5548	24.8 KIPS	
	Stressing Load 70% 4854	21.7 KIPS	.00680 in/in
	Design Load 60% 4161	18.6 KIPS	
1/2"	Temporary Load 80% 7382	33.0 KIPS	
	Stressing Load 70% 6465	28.9 KIPS	.00680 in/in
	Design Load 60% 5548	24.8 KIPS	

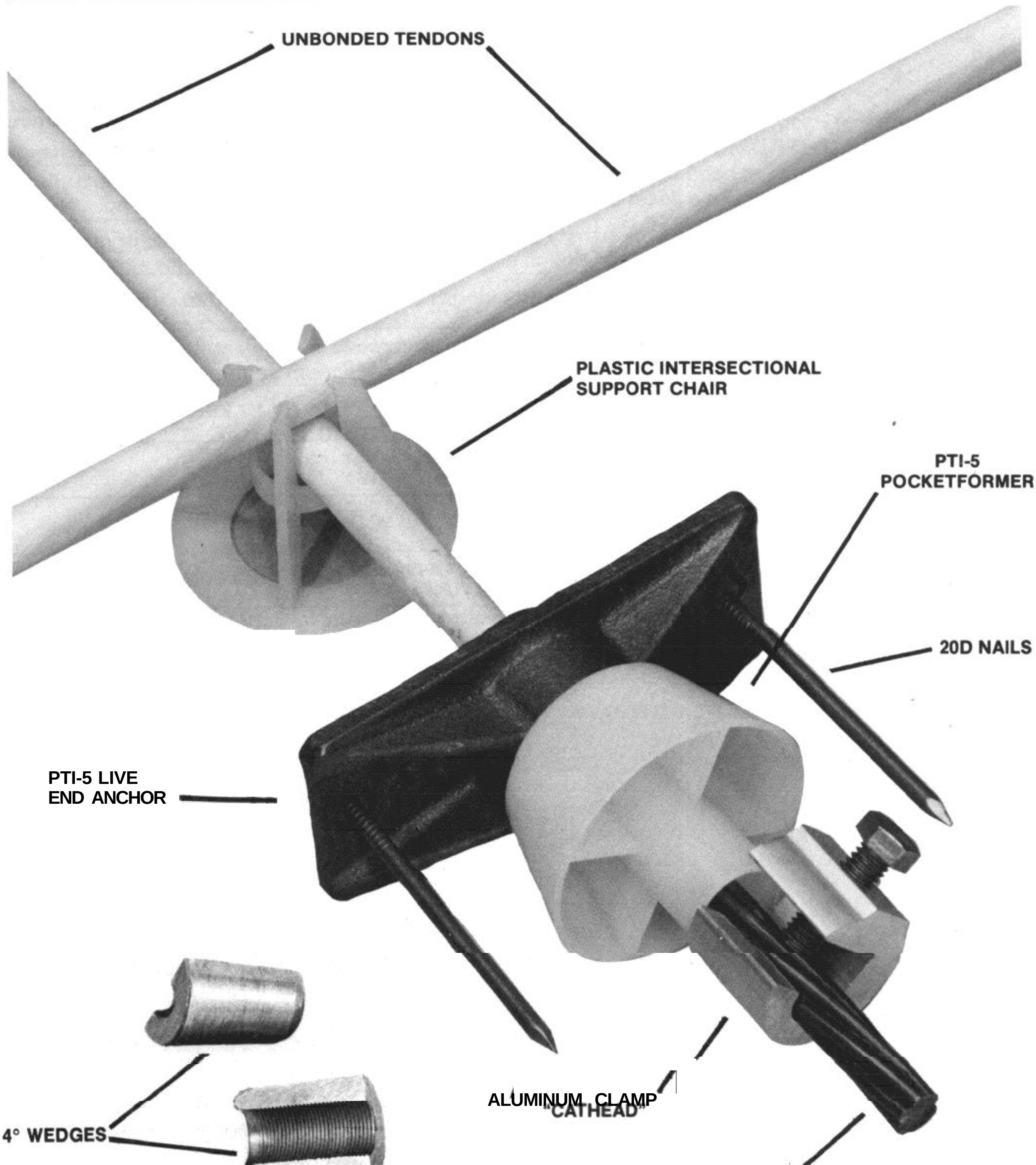


TWIN STROKE STRESSING RAM

HYDRAULIC PUMP

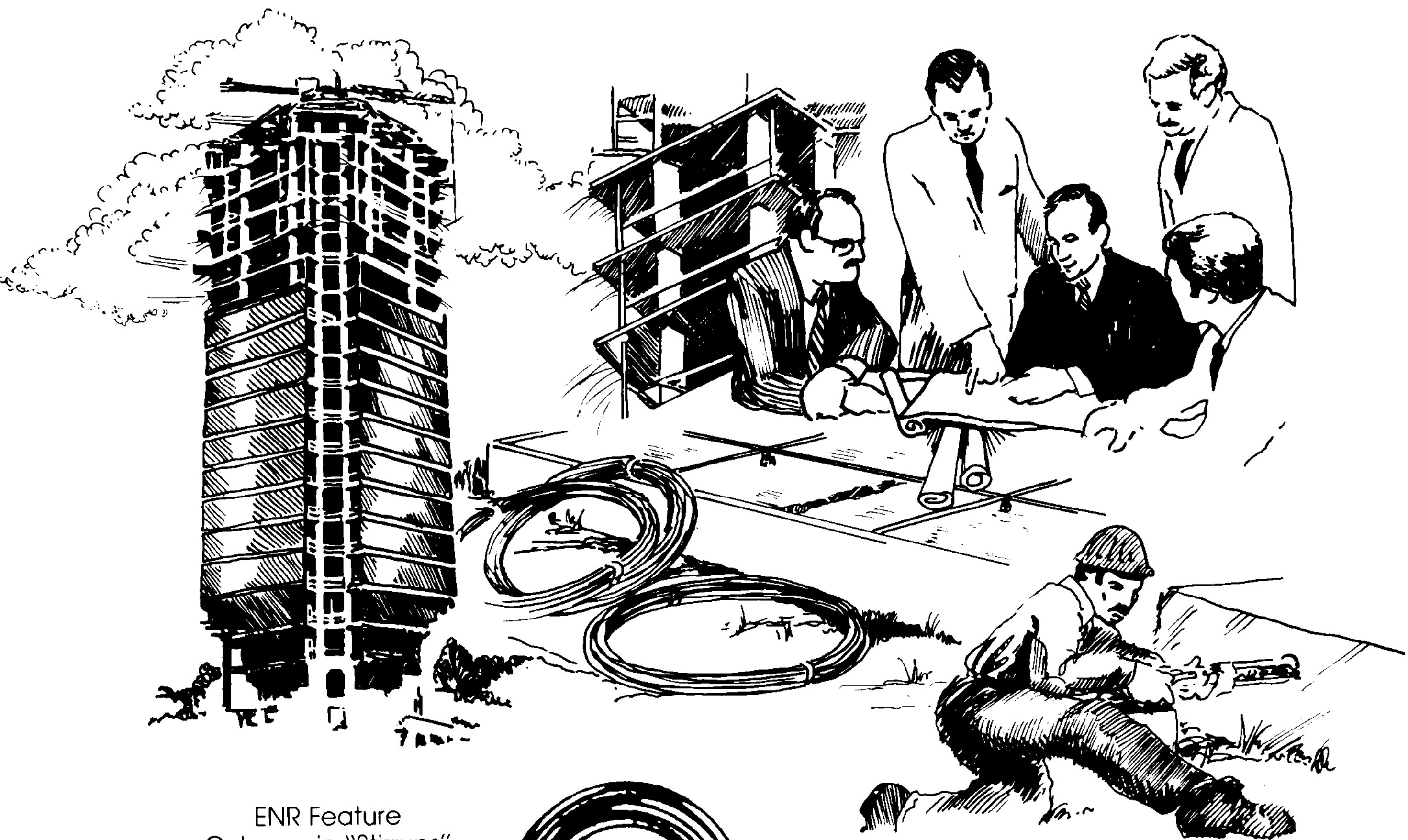
2.2.11 NATIONAL POST-TENSIONING SERVICES, INC. & POST-TENSION OF TEXAS, INC. — continued

COMPONENTS OF NATIONAL POST-TENSIONING SERVICES, INC. AND POST-TENSION OF TEXAS, INC. POST-TENSIONING SYSTEM



2.2.12 PATTRIDGE POST-TENSION, INC.

A SUBSIDIARY OF PPT, INC.



ENR Feature
Columns in "Stirrups"
March 22, 1984 issue
, Pages 22 & 23

PATTRIDGE
POST-TENSION, INC.
A SUBSIDIARY OF PPT, INC.

INTRODUCTION

Pattridge Post-Tension, Inc., is a manufacturer of post-tensioning materials which have been used across the United States on projects of various types. Since its incorporation in 1979, dynamic growth and numerous accomplishments have established Pattridge Post-Tension, Inc., as a responsible member of the post-tensioning industry.

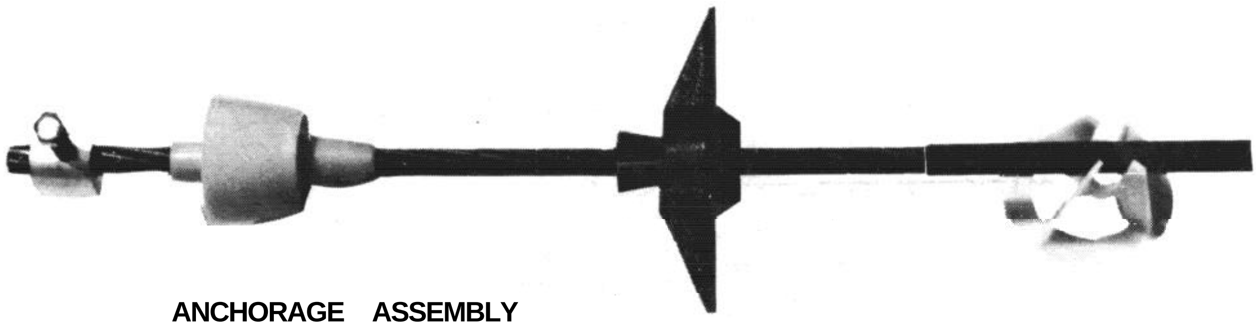
SERVICES

Pattridge Post-Tension, Inc., offers developers, architects, engineers, and contractors assistance with preliminary estimating and design by providing information regarding projected concrete member sizes, reinforcing quantities, and a budget installed post-tensioning figure. We also have the capability to assist engineers in the design of specific projects. Ours is a "teamwork" philosophy, working closely with the design team and the contractor taking care to consider realistic construction practices and limitations as well as the technical aspects of the structural design. Home-based in Shreveport, Louisiana, a problem soil area, our expertise is extended to residential and commercial slabs on grade as well as to parking decks, hotels, condominiums, office buildings and other structures.

Our experience has proven the economies of using a post-tensioned concrete system. The system's ability to reduce concrete depth, allow longer spans economically, provide water tight, crack-resistant slabs, and reduce construction time has brought many projects into budget.

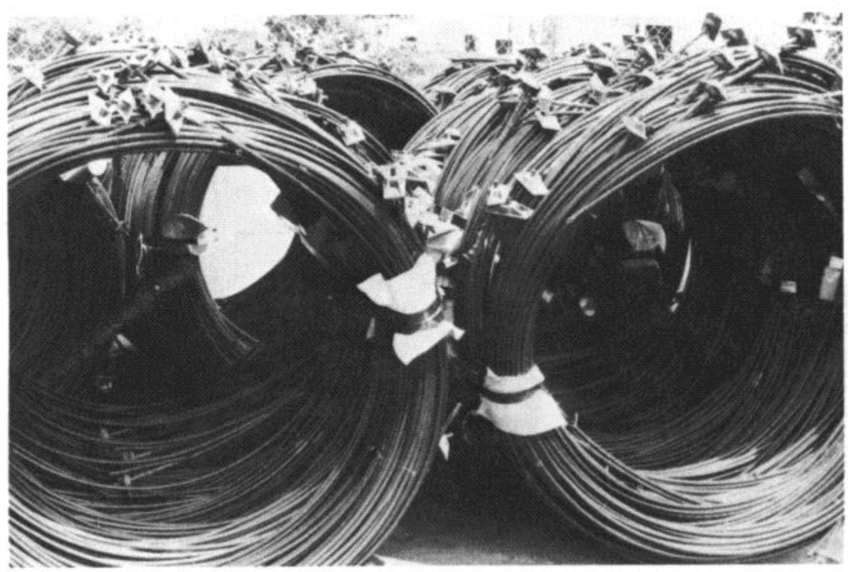
MATERIALS

Pattridge Post-Tension, Inc., is a supplier primarily of the unbonded mono-strand system, which allows for easier erection and greater construction speed. Materials can be provided in bulk, or fabricated for specific projects. The prestressing steel used is 270 ksi seven-wire strand manufactured in accordance with ASTM A416, in 3/8", 1/2", and 0.6" diameters. The strand is coated with a rust-inhibiting lithium-based grease, over which is a durable protective plastic sheath.



ANCHORAGE ASSEMBLY

Our electronic fabrication line dispenses to accurate measurements and the dead ends are color coded as per shop drawings. The computer is used to label each bundle of fabricated cable for ease of jobsite identification.



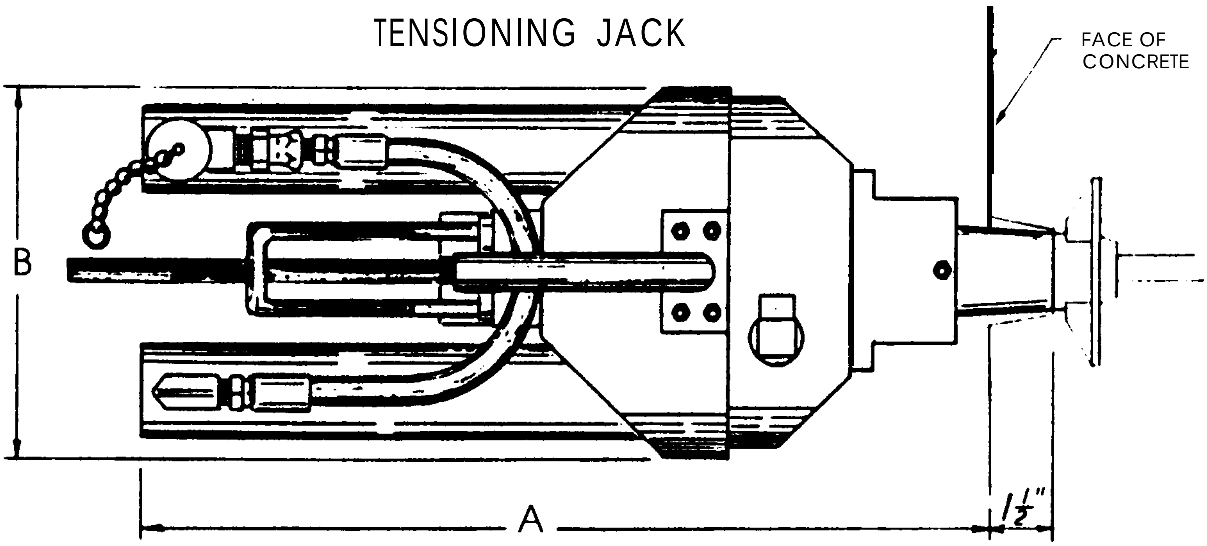
FABRICATION

Individual tendons are fabricated by specific length for each project. Identification is made by color coding to correspond with the strand's placement as shown on the shop drawings, as well as by being individually tagged. Dead or fixed-end anchorages are pressed on and intermediate anchors are installed on the strand in our plant. Each group of tendons is bundled together and labeled according to its location in the project, and then shipped to the jobsite.

INSTALLATION

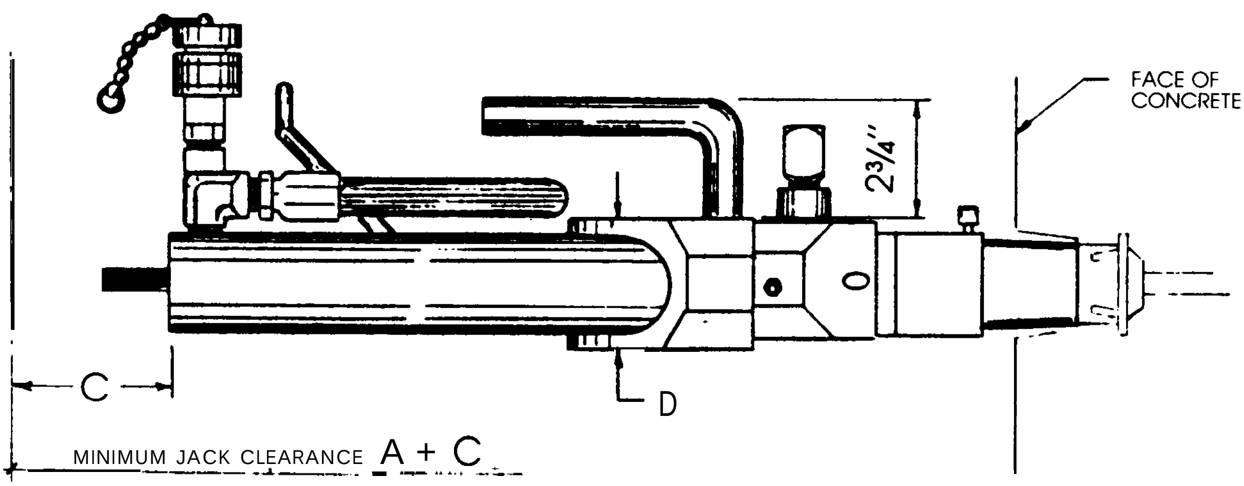
In some geographic areas, Pattridge Post-Tension, Inc., provides a materials and installation package. In other areas we can assist in locating companies experienced in the placement and tensioning of post-tensioning materials. Or a customized package of technical assistance for training and/or inspection can be offered to meet the needs of the contractor.

TENSIONING JACK

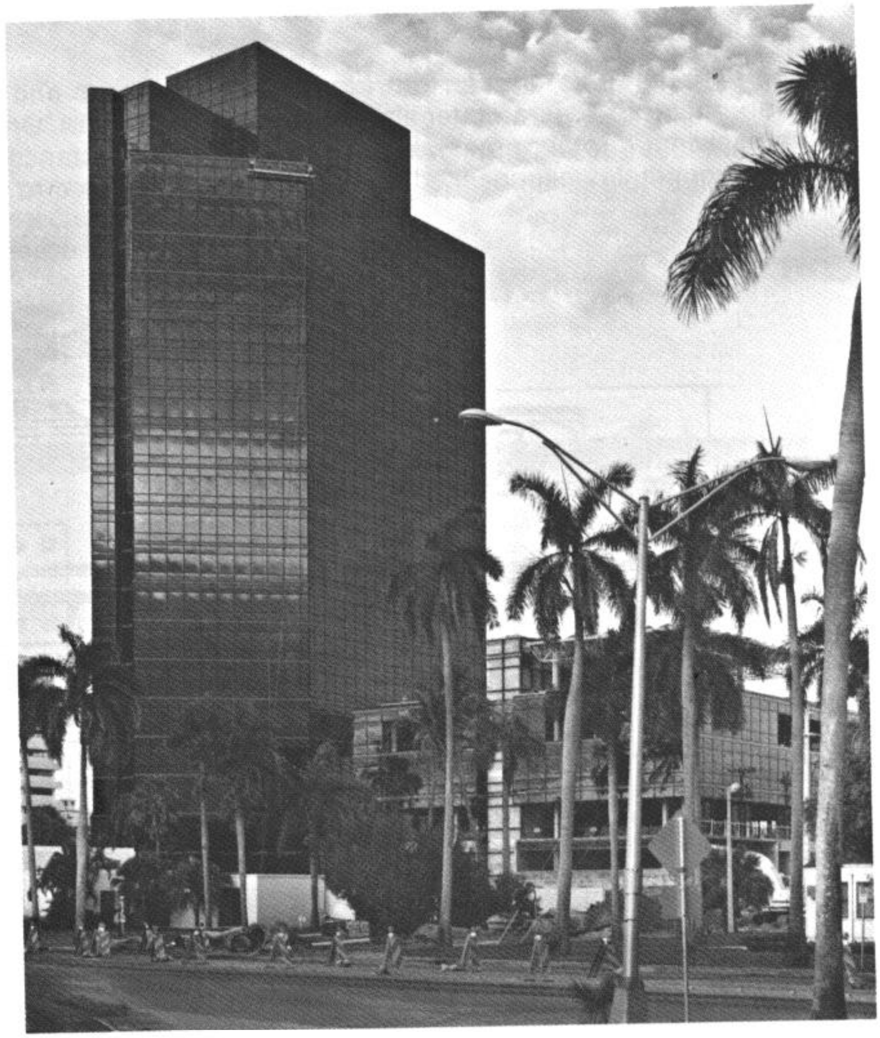


PLAN

STROKE INCHES	A	B	C	D
10	19.5	8.75	10	3



SIDE



Northbridge Centre
West Palm Beach, Florida

21-Story Office Building

Unbonded post-tensioned slabs with bonded transfer wall at lower levels to carry 17 floors extending over the lower structure.

Bankers Life Parking Garage
Des Moines, Iowa

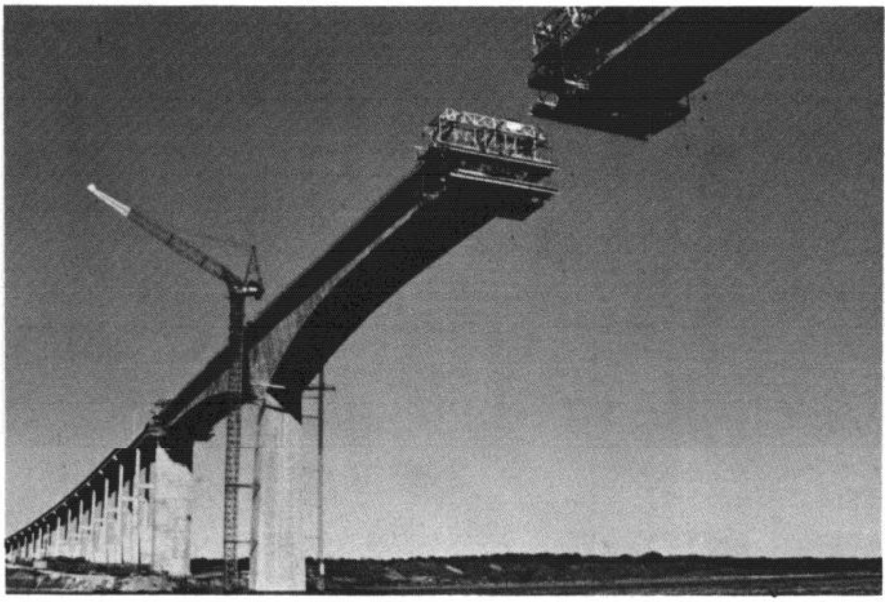
8 Parking Levels

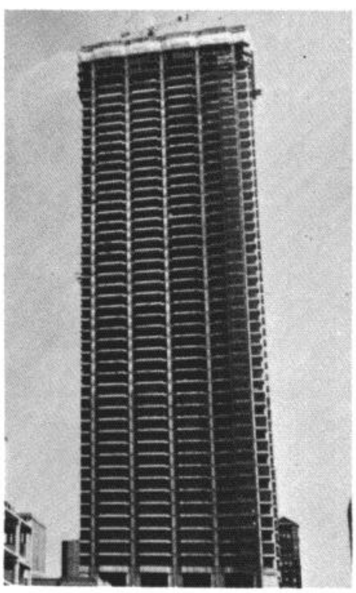
One-way post-tensioned slab and beam. Precautions were made for corrosion resistance.

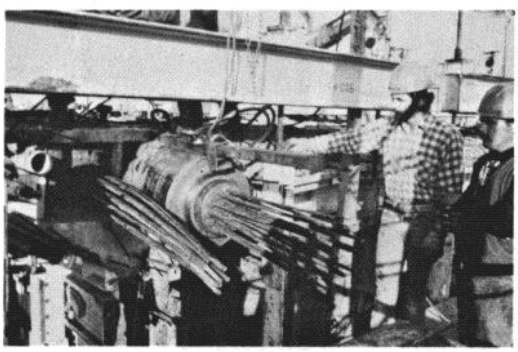
2.2.13 THE PRESCON CORPORATION

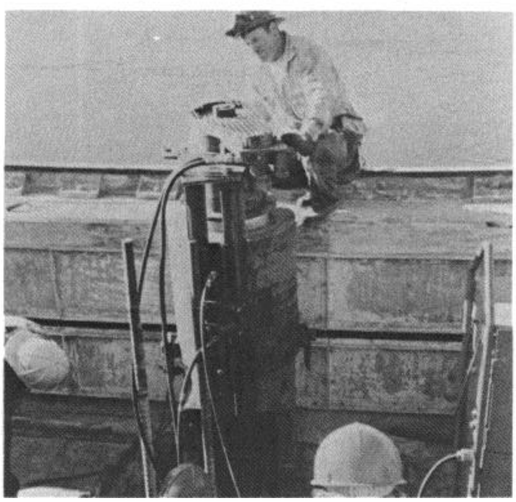
freyssinet

THE PRESCON CORPORATION





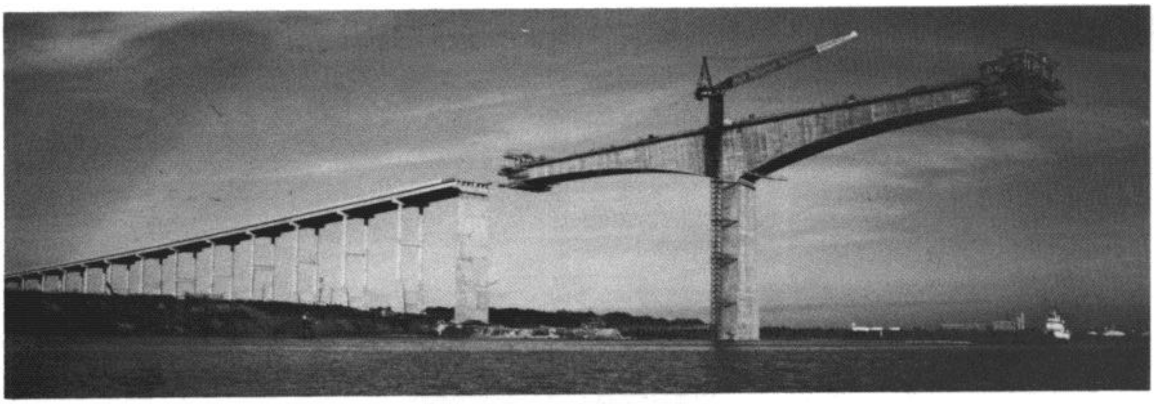




THE PRESCON CORPORATION, a division of the Campenon Bernard Group and member of the Freyssinet International Group, specializes in the application of prestressed technology, furnishing engineering services, specialized construction systems, and post-tensioning systems for major construction projects.

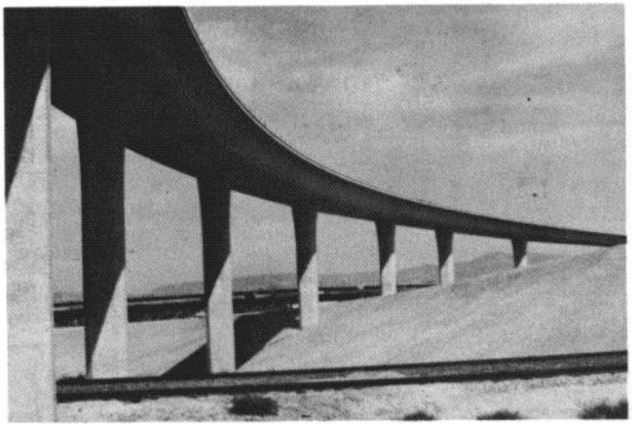
Prescon's experience and service to the construction industry in the United States goes back to 1950. Prescon's internationally recognized parent firm of Campenon Bernard was founded 32 years earlier in 1918 and its association with the late Eugene Freyssinet, inventor of prestressed concrete, in 1933. The group has been in the forefront of progress within the construction industry.

Today, alone or in collaboration with local and international partners, the group is capable of providing a full range of diversified engineering and construction services ranging from technical assistance to full responsibility for the execution of major projects.



1.

2.



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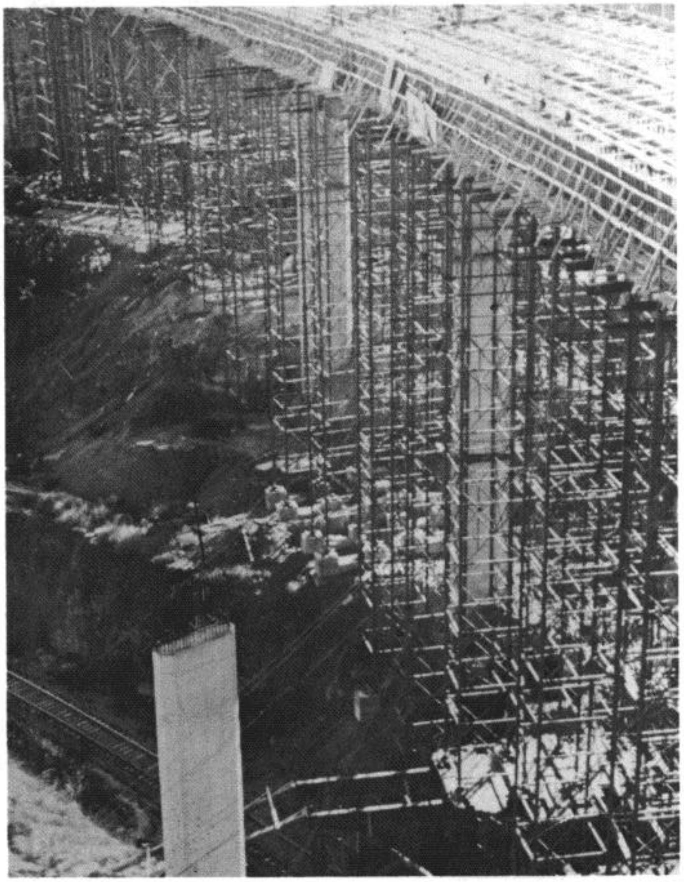

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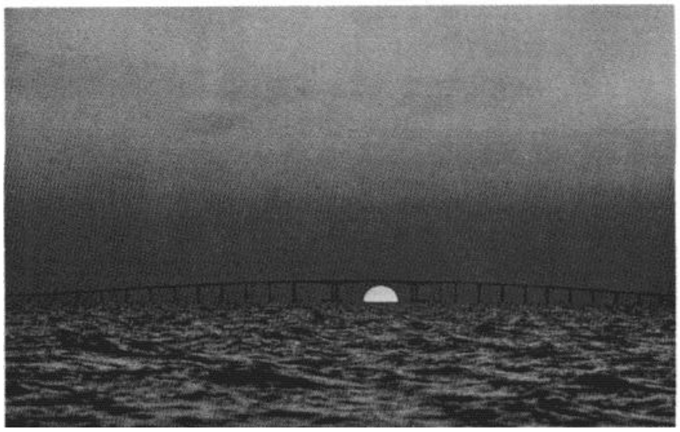
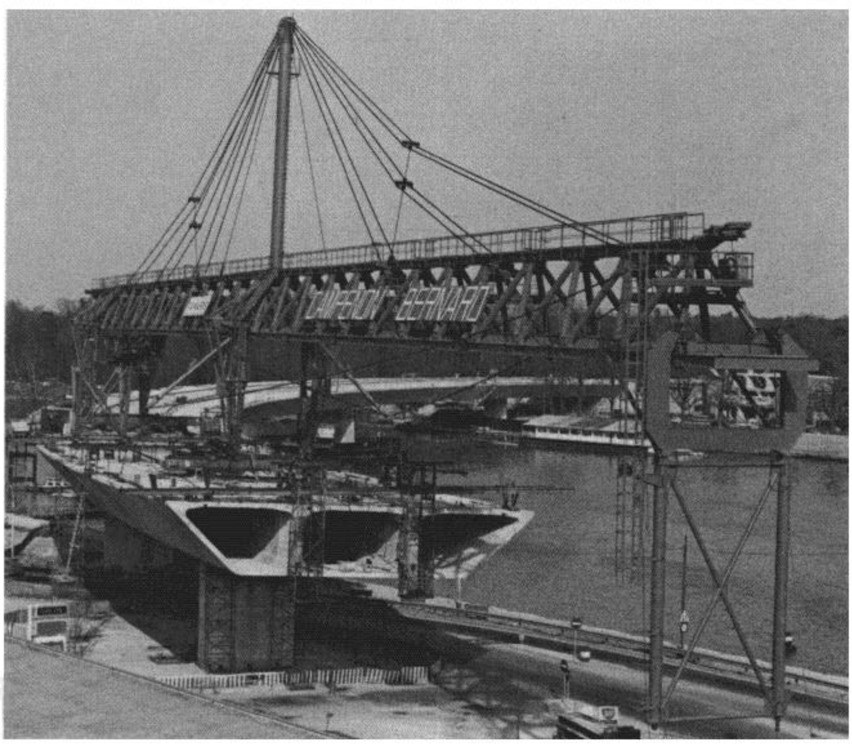


5.

1. Cast-in-place Segmental JESSE H. JONES MEMORIAL BRIDGE HOUSTON, Texas
2, Precast segmental VAIL PASS, Colorado
3 and 4. Cast-in-place
5. DAUPHIN ISLAND BRIDGE, Alabama
6. Precast segmental L32 BRIDGE. Austria
7. SAINT-CLOUD BRIDGE, France

BRIDGES

6. 

7. 

Prescon and Campenon Bernard/Freyssinet Group have been an industry leader in the development of applications and contributions to bridge construction techniques.

Precast segmental concrete bridge construction as developed by the Freyssinet Group has virtually revolutionized design and construction procedures for a growing variety of bridge requirements.

The concept of elimination of high level, traffic obstructing, or terrain restrictive falsework motivated the development of cast-in-place segmental, and incrementally launched (pushed) bridges.

For the bridge construction industry, Prescon can provide the following:

- Post-Tensioning Tendons, Labor and Equipment.
- Design Assistance and Onsite Technical Instruction.
- Specialized Construction Equipment:
  - Flat Jacks
  - Launching Girders
  - Traveler-Form Systems
  - Erection Equipment
- Bridge Repair Services.
- Turn-Key Bridge Packages.

CABLE STAYS

Present dav construction techniques' use Cable Stays for many types of structures, such as aircraft hangar's, telecommunication towers, large chimneys, cooling towers, electrical pylons, and long-span stayed bridges. The utilization of Cable Stays on bridges is experiencing an impressive and fast development.

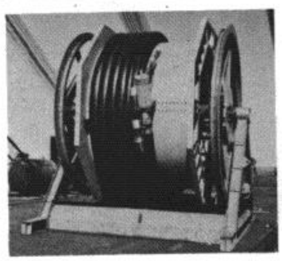
The Prescon/Freyssinet team has developed, tested, fabricated and installed both the "Hi-Am" type wire Cable Stays, and Freyssinet Stays of 0.6 inch (15mm) diameter strand in numerous structures throughout the world.

In typical cable stayed structures the stays are subjected to large load variations so that their static strength as well as their fatigue stength become critical. This inherent nature of stays has been extensively tested on representative models carried out in officially recognized laboratories in various countries.

The Prescon/Freyssinet stays have a wide range of capacities, with a current maximum ultimate force of 5330 kips (23700 kN). The stays can be either shop fabricated or site fabricated, depending upon the type of stay, site accessibility, and stay length.

1 Freyssinet Stay
2. Hi-Am Stay
3. PASCO-KENNEWICK, Washington
4. COATZACOALCOS Bridge, Mexico


1.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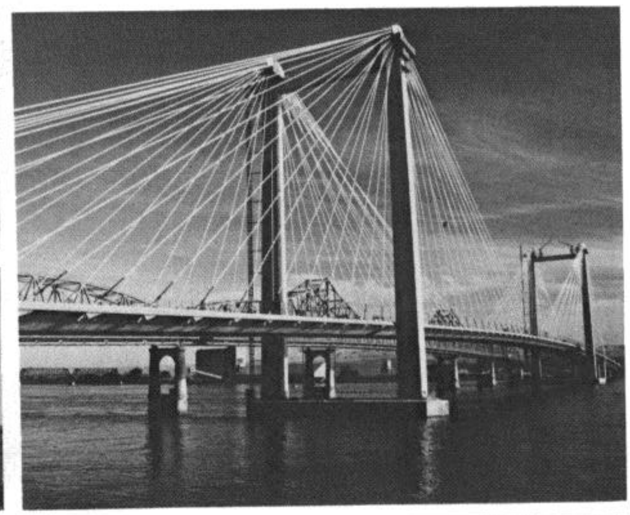
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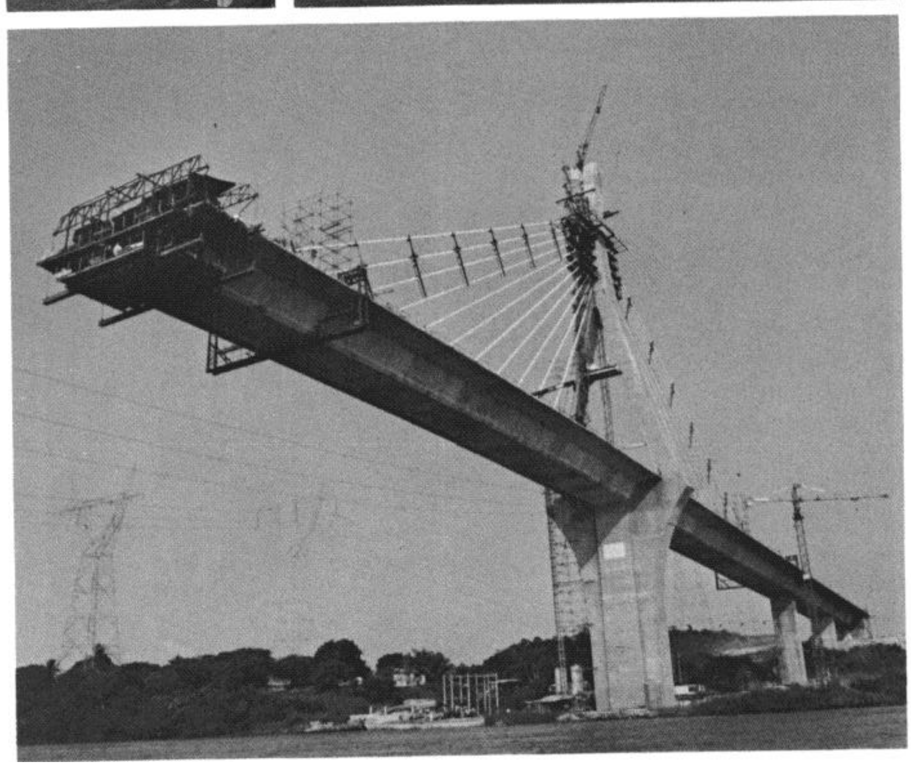
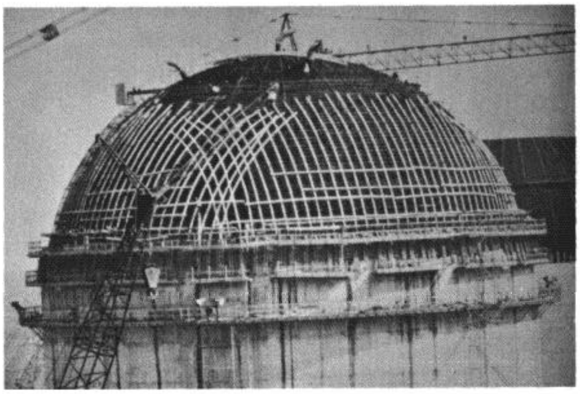

4.

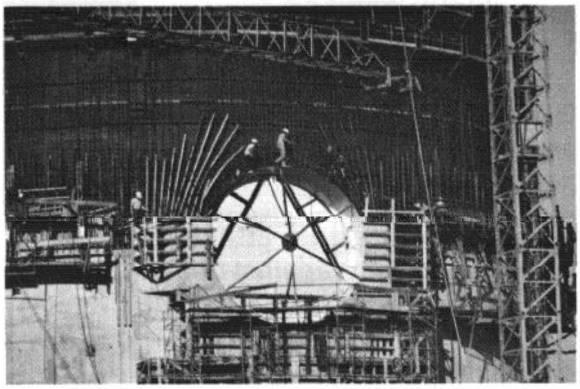
NUCLEAR CONTAINMENTS

The prestressing of nuclear containment structures has presented many unique challenges to the post-tensioning industry.

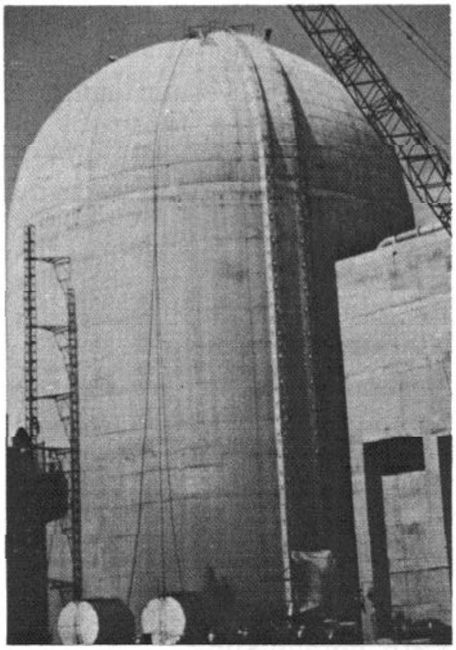
Prescon, a leader in this field of work since the first projects were completed in the United States, has made significant contributions in the areas of:

- Development and Testing
- Installation Equipment
- Special Jacking Systems
- Onsite Fabrication Facilities
- Quality Assurance Programs
- Surveillance Work





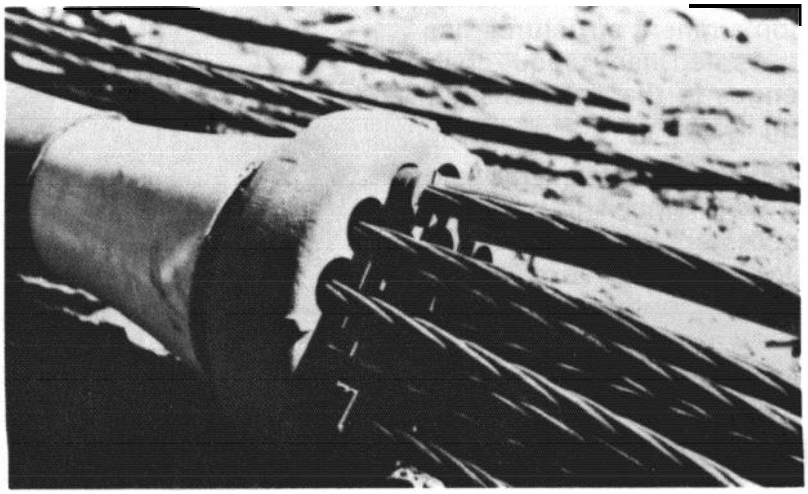
NAME OF PLANT	NET OUTPUT Mw	OWNER	TENDON TYPE
UNITED STATES			
TURKEY POINT III	725	FLORIDA POWER AND LIGHT	PRESCON BUTTON-HEADED WIRE
TURKEY POINT IV	725	FLORIDA POWER AND LIGHT	"
OCONEE I	890	DUKE POWER	"
OCONEE II	890	DUKE POWER	"
OCONEE III	890	DUKE POWER	"
ARKANSAS	850	ARKANSAS POWER & AND LIGHT	"
ARKANSAS II	915	ARKANSAS POWER & AND LIGHT	"
CALVERT CLIFFS I	845	BALTIMORE GAS AND ELECTRIC	"
CALVERT CLIFFS II	845	BALTIMORE GAS AND ELECTRIC	"
MILLSTONE II	830	NORTHEAST NUCLEAR COMPANY	"
TROJAN	1 130	PORTLAND GAS AND ELECTRIC	"
CRYSTAL RIVER	825	FLORIDA POWER CORP	"
SOUTH TEXAS I	1 250	HOUSTON LIGHT AND POWER	"
SOUTH TEXAS II	1 250	HOUSTON LIGHT AND POWER	"
TAIWAN			
MAANSHAN I	950	TAIWAN POWER CORP	37K6
MAANSHAN II	950	TAIWAN POWER CORP	37K6



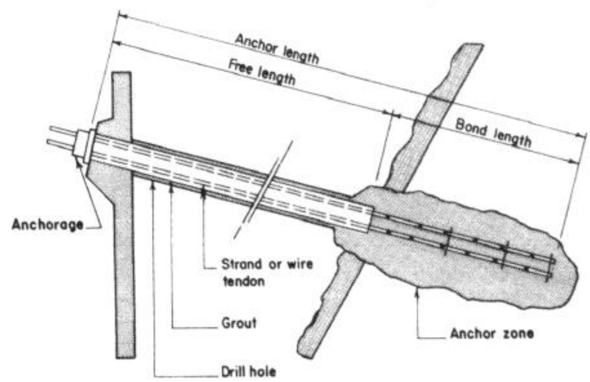
GROUND ANCHORS

GROUND ANCHORS are classified according to the type of ground surrounding the fixed anchorage, i.e. sand, gravel, clay or rock. They can be used to perform the following functions:

- To anchor a structure to the ground and prevent its movement relative to the ground.
- To prestress the ground to improve its strength, stability, or its resistance to applied structural loads.
- To resist earth pressures created during excavation.



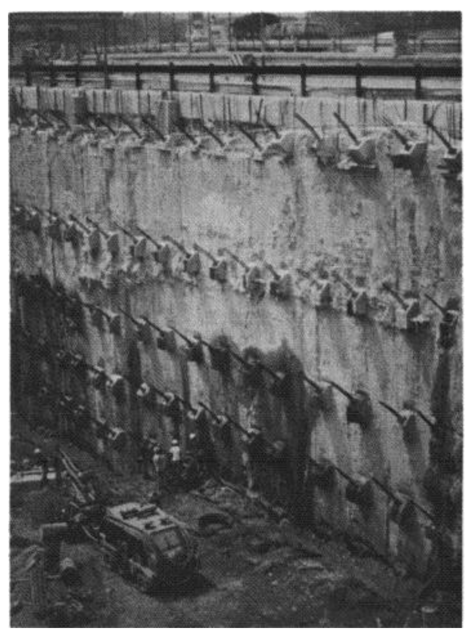
The increasing acceptance of prestressed Ground Anchors has considerably extended the field of application of this structurally exciting technique, i.e. structural braces, protection against floatation forces, highway construction, enlargement of or stabilization of existing dams, marine and dry dock construction, tunnels, pile testing, and transmission tower overturning resistance.



The Prescon/Freyssinet team has remained in the forefront of Ground Anchor development since their inception in the early 1930's. Prescon has a complete range of anchorage systems, both temporary and permanent, with ultimate capacities exceeding 2400 kips (10600 kN).

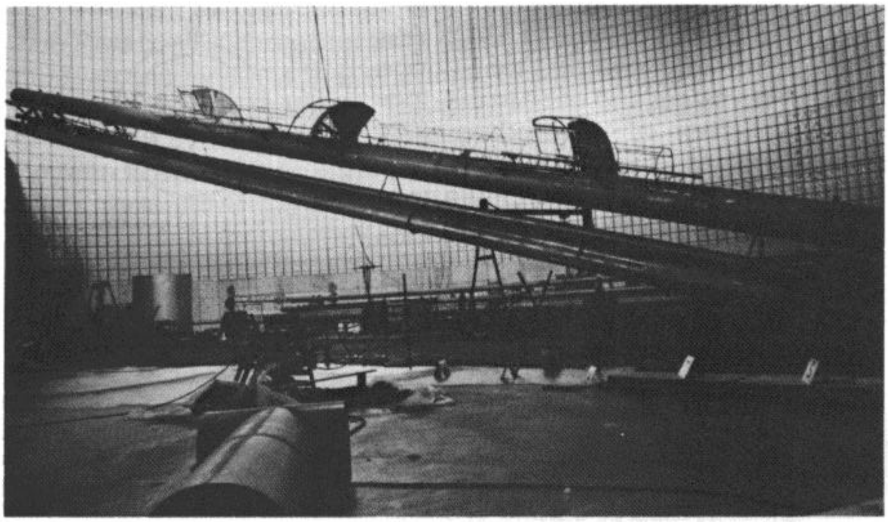
Prescon has the experience, the capability and the resources to supply the following:

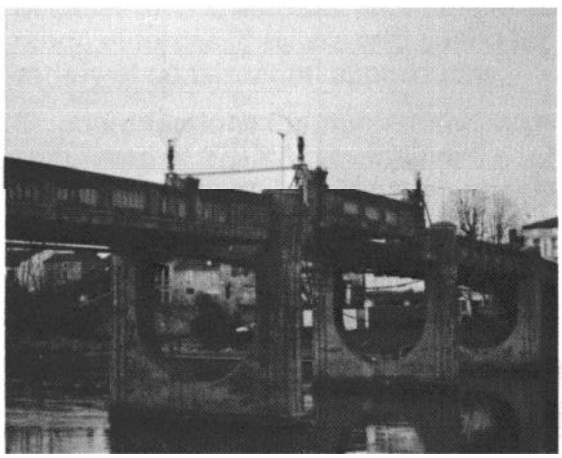
- Post-Tensioning Materials
- Anchorage Fabrication
- Anchorage Installation and Testing
- Design Assistance
- Anchorage Surveillance
- Anchorage Removal



2.2.13 THE PRESCON CORPORATION — continued

CONTROLED LIFTING AND MOVING

1. 

2. 

3. 

In recent years equipment having capacities far outside the range of conventional winches has been required for the purpose of lifting, lowering and horizontal movement. To meet this demand we have developed our hydraulic jacks to make available a selection of compact, precise, but yet enormously powerful and reliable equipment ideally suited to these operations. Individual units ranging in capacity from 15 Ton to 560 Ton are now available and in all cases are designed for fully automated working, requiring no adjustments or repositioning of the equipment for the complete operation once installed. Capacities of movement are limited only by the number of jack units required with relation to the movement geometry.

All problems in this field have of course their own particular difficulties and requirements. Our international team of experts will be happy to advise on both the general scheme and detailed application of the equipment.

- Longitudinal launching of bridges during construction.
- Transverse sliding of bridges to allow construction to take place adjacent to final required position.
- Lifting precast segments for bridge construction.
- Lifting complete bridge sections into position.
- Lifting and lowering of bridge falsework systems.
- Caisson lowering during bridge pier construction.
- Lifting complete operations platform of concrete offshore oil platforms from assembly position at the base of the platform towers.
- Installation of pipework and services in offshore oil platform towers.
- Lifting complete roofs and' roof sections into position after assembly at ground level.
- Erection of chimney units either lifting from above or below.
- Installation of heavy plant inside buildings where heavy crane access is not available.
- Installation of nuclear reactor components.

1. Raising of steel structures at Montoir-de-Bretagne, France
2. Lifting of the cantilever spans (300t) by 1,30 m Libos Bridge, France
3. Lifting of floors for the Maves Radi Tower, France

FREYSSI FLAT JACKS

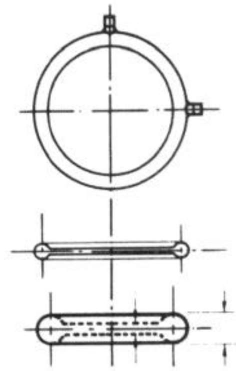
Freyssi Flat Jacks find wide application in the civil. and structural construction industry wherever application or control of large forces are required or where structural or foundation strains have to be induced. Essentially simple and compact, they are as often used in situations for which their use was not foreseen, such as remedial measures or structural additions, as they are used in new constructions in which they form a part of the structural concept. The Freyssi Flat Jacks have been used on a world wide basis, to solve an astonishing variety of structural and civil engineering problems:

- Control of Thrust Forces
- Prestressing Between Abutments
- Adjustment of Support Reactions
- Structural Pre-Loading
- Structural Lifting
- Underpinning
- Measurement of Forces

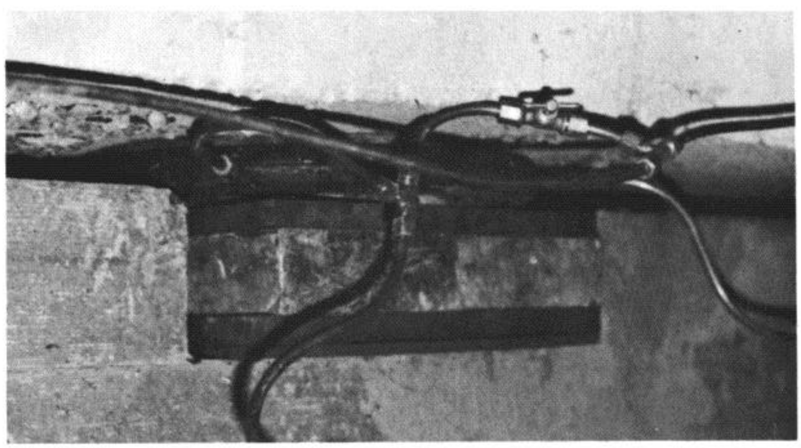
Flat Jacks can be operated with oil for temporary use or where Flat Jacks, after utilization, are to remain permanently in the structure they are filled with a hard-setting grout or epoxy resin.

Our experienced staff is fully qualified to advise on all possible applications of Flat Jacks, to assist in preparing practical preliminary schemes and to supervise site installations and operations.

THE FREYSSI FLAT JACK was originally invented by Eugene Freyssinet and used in the construction of some of his early bridges. In its simplest form, it is a thin pressure capsule capable of exerting, hydraulically, extremely large forces through a restricted movement; the stroke can be extended by "stacking" jacks in series.



Reference number	Outside diameter (Inches)	Force at 2200 PSI Jack closed (Tons)	Force at 2000 PSI Jack fully open (Tons)	Minimun gap for insertion (I) (Inches)
12	4.7	9.8	8.5	1 1/2
15	5.9	18	15	1 1/2
22	8.7	46	40	1 1/2
25	9.8	62	56	1 1/2
27	10.6	74	66	1 1/2
30	11.8	94	87	1 1/2
35	13.8	127	120	1 1/2
42	16.5	197	189	1 1/2
48	18.9	262	250	1 1/2
60	23.6	427	418	1 3/4
75	29.5	690	680	1 3/4
87	34.2	940	930	1 3/4
92	36.2	1075	1065	1 3/4
108	42.5	1490	1480	1 3/4



Use of flat jacks for raising bridge deck

NOTES:

(I) The minimum access gap required to place flat jack and steel thrust plates between the surfaces to be jacked.

(II) All flat jacks are 1 1/4 inches thick and have a maximum stroke of 1 inch.

(III) The normal range of jacks is circular as indicated in the table. However, both oblong and rectangular jacks may be custom built for special applications using ends or corners of standard radii.

(IV) If you have a problem which might possibly be solved by the use of the jacks, or require further information, please contact our offices for assistance.

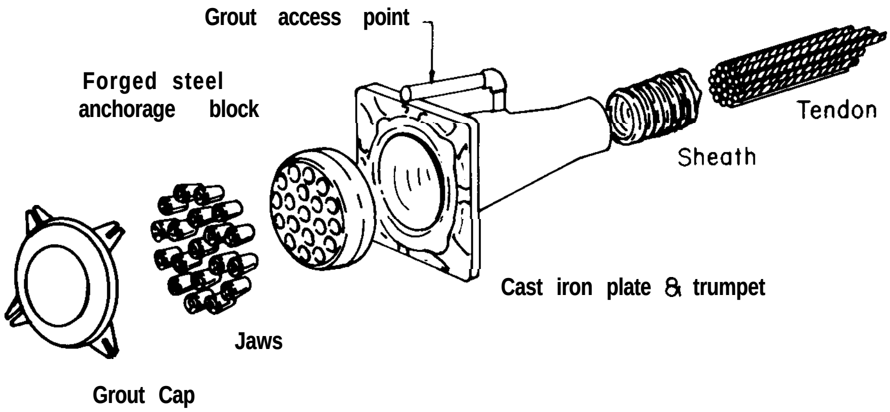
PRESCON POST-TENSIONING SYSTEMS

The Prescon Corporation offers a wide range of multi strand systems that encompass a multitude of construction applications. The design and development of many of these systems were defined by Eugene Freyssinet as early as 1928.
The Freyssinet Organization has become a respected authority on prestressing techniques throughout the world.

A great number of structures using these systems have been in service for as long as 50 years. The strand tendons presently marketed, were first introduced in 1959. They have been used in thousands of structures throughout the world under every climatic condition. They have proven to be extremely reliable, economical and simple to use. Because strand tendons are made to develop a wide range of forces, they are well suited for a great variety of construction applications.

Standard tendons for multi-strand systems are composed of 270 KSI seven-wire 0.6 inch diameter and 0.5 inch diameter strand in either stress-relieved or low-relaxation quality.

For enclosures, semi-rigid galvanized steel conduit is recommended. In addition to extremely low wobble coefficient, the conduit permits the "pull through" method of installation, where the strands are placed just a short time before stressing. However, other types of enclosures maybeusedsuchas flexible sheath, circular or oval and preformed holes. All tendons are suitable for grouting, grease filling or unbonded applications.



The Prescon Corporation offers button-headed wire tendons for use in special applications, such as nuclear containments, rock anchors, and other construction applications. Information on our wire post-tensioning systems is available upon request, contingent upon project requirements,

POST-TENSIONING SYSTEMS DETAIL

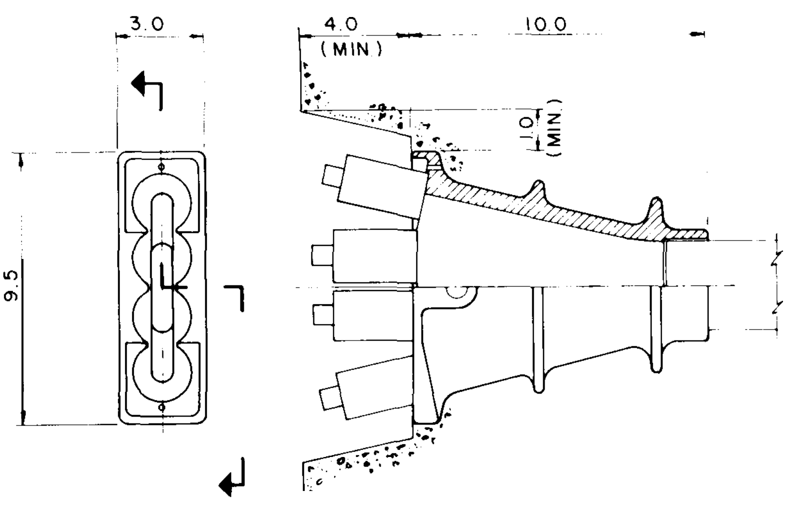
SELECTION OF PRESCON POST-TENSIONING SYSTEM

Anchorage	N° 0.5" Strand	N° 0.6" Strand	Ultimate Force (Kips)
4S5	2-4	—	82 - 165
4S6	—	2-4	117 - 234
4cc65	2-4	—	82 - 165
4cc66	—	2-4	117 - 234
7K5	5-7	—	206 - 289
12K5	8-l 2	—	330 - 496
12K6	—	5-12	292 - 703
19K5	13-19	—	537 - 785
19K6	—	13-19	761 - 1113
27K5	20-27	—	826 - 1116
37K5	28-37	—	1157 - 1529
37K6*	—	20-37	1171 - 2167
55K5*	38-55	—	1570 - 2274

"DETAILS OF SYSTEMS AVAILABLE UPON REQUEST

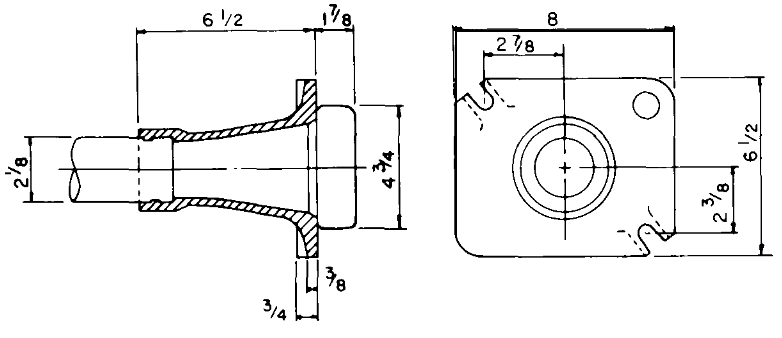
PRESCON '4S6' SYSTEM

The Prescon '4S6' System is designed for use in bridge decks, slabs or other structural systems that require a narrow anchorage, in order to accommodate adequate concrete coverage. The anchorage can be used with 0.6 inch diameter or 0.5 inch diameter strand, and utilizes flat or round (metal or plastic) duct. The system is designed to stress each strand individually.



PRESCON '4CC' SYSTEM

The Prescon '4CC' System is an economical parallel strand anchorage system, where all strands are stressed simultaneously. The anchorage can be used with 0.6 inch diameter or 0.5 inch diameter strand, and utilizes round or flat (metal or plastic) duct.

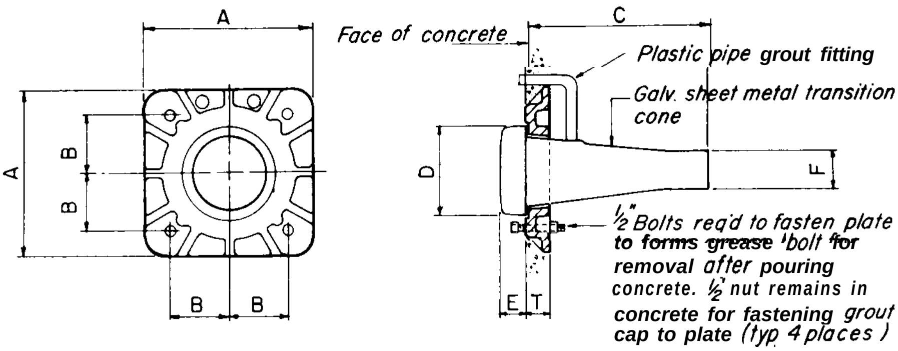


The Prescon 'K' System is the latest development by Freyssinet International. The 'K' Systems provide cable forces ranging from 100 tons to 1100 tons breaking load.

To achieve the maximum static and dynamic security in the tendon, each hole is drilled at its correct angle in relation to the tendon pattern and all unwanted deviations in the strand path are avoided.

PRESCON 'KP' SYSTEM

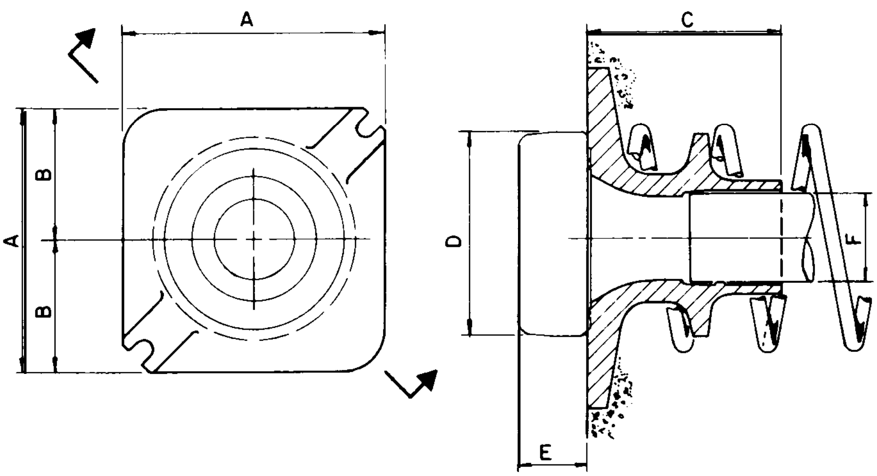
The Prescon 'KP' System is an economical single bearing plate anchorage, designed in accordance with AASHTO specification stipulating bearing stresses at service load to be no greater than 3000 psi.



Type	Str Dia	NO. Str.	A	B	C	D	E	F	T	RAM
12 K 5 P	0.5	12	10.6	3.65	4.25	5.51	2 24	2.75	1.6	K 200
12 K 6 P	0.6	12	12.6	4.152	6.25	6.38	2.48	3.25	1.88	K 350
19 K 5 P	0.5	19	13.50	5.0	9.4	7.28	2.36	3.25	1 88	K 350
19 K 6 P	0.6	19	16 38	5.69	11.2	8.56	2.56	4.00	2.28	K 500
27 K 5 P	0.5	27	16.38	5.69	11.2	8.56	2.56	4.00	2.28	K 500
27 K 6 P	0.6	27	19.37	6.25	12.8	9.92	3.07	4.38	2.88	K 700
37 K 5 P	0.5	37	19.37	6.25	12.8	9.92	3.07	4.38	2.88	K 700

PRESCON 'KD' SYSTEM

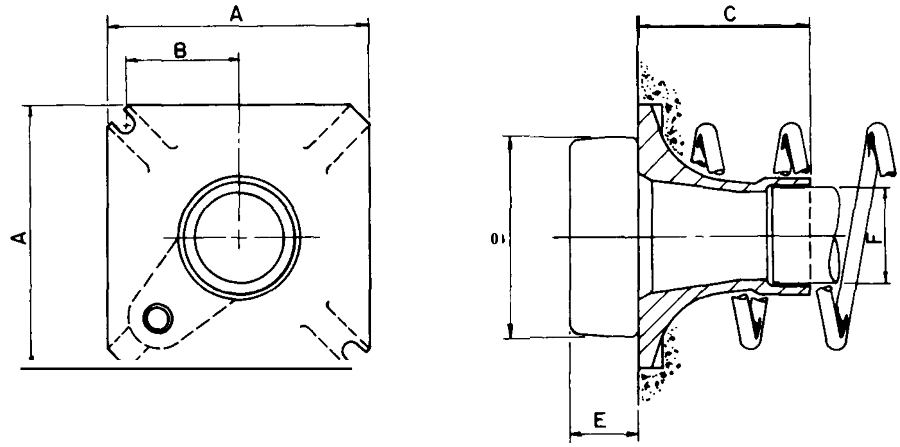
The Prescon 'KD' System is a double bearing plate anchorage designed in accordance with the same AASHTO specification noted in the 'KP' System, but the overall size of the anchorage is reduced in order to facilitate placement in restrictive geometric structural shapes.



Type	Str Dia	No. Str.	A	B	C	D	E	F		RAM
7 K 5	0.5	7	6.9	3.45	5.12	4.72	1.97	2.125		200 T

PRESCON 'KE' SYSTEM

The Prescon 'KE' System is an economical cast ductile iron trumplate designed in accordance with FIP, and ACI standards. It is most commonly used in structures utilizing a high strength of concrete, approximately 5000 psi, at time of stressing.

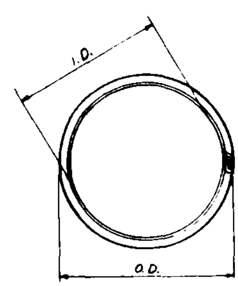


Type	Str Dia	No. Str.	A	B	C	D	E	F		RAM
12 K 5	0.5	12	8.25	3.5	5.25	5.51	2.48	2.75		K 200
12 K 6	0.6	12	9.62	4.12	6.25	6.38	2.48	3.25		K 350

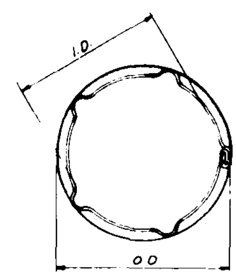
PRESCON DUCT

Two different types of duct are available, smooth and deformed. Smooth duct is so called because it has no deformations between the spirally formed continuous seam. Deformed duct features a series of spiral configurations between the spiral formed continuous seam. Smooth duct sections are joined together with short sections of similarly formed duct in the next larger size. These sleeves are centered on the joint between duct sections and taped to the duct at the sleeve terminals for post-tensioned applications.

Deformed duct sections are also joined together with short sections of similarly formed duct in the next larger size, except that the sleeve is screwed on the abutting ducts and the terminals taped. Again this type joint is applicable to post-tensioning applications.



X SECT.
Smooth Profile



X SECT.
Corrugated Profile

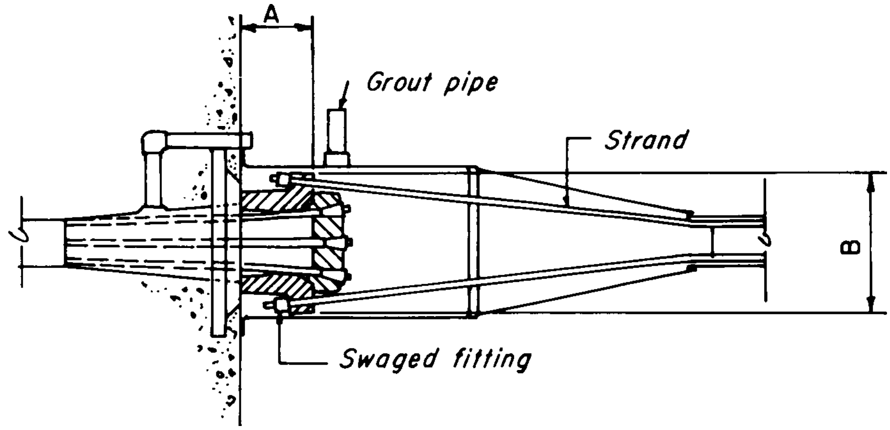
Nominal Size	Gage	I.D.	O.D.	Wt Per Foot Galvanized Steel
1 7/8"	26	1.83" (46.5 mm)	2.02" (51.3 mm)	.54 Lbs.
2 1/8"	26	2.13" (54.0 mm)	2.32" (59.0 mm)	.61 Lbs.
2 3/4"	26	2.70" (68.5 mm)	2.90" (73.7 mm)	.76 Lbs.
3 1/4"	26	3.27" (83.0 mm)	3.46" (88.0 mm)	.86 Lbs.
3114"	24	3.22" (81.8 mm)	3.46" (88.0 mm)	1.10 Lbs.
4"	26	4.04" (102.5 mm)	4.23" (107.5 mm)	1.05 Lbs.
4"	24	3.99" (101.3 mm)	4.23" (107.5 mm)	1.35 Lbs.
4 3/8"	24	4.39" (111.5 mm)	4.63" (117.5 mm)	1.60 Lbs.
4 3/8"	22	4.34" (110.3 mm)	4.63" (117.5 mm)	1.95 Lbs.
4 3/4"	24	4.75" (120.5 mm)	5.00" (127.0 mm)	1.78 Lbs.
4 3/4"	22	4.70" (119.3 mm)	5.00" (127.0 mm)	2.16 Lbs.
5,,	24	4.97" (126.2 mm)	5.22" (132.5 mm)	1.86 Lbs.
5"	22	4.92" (125.0 mm)	5.22" (132.5 mm)	2.26 Lbs.

Total manufacturing capability covers gages 28 through 20 and duct diameters to 30". Detailed information for unlisted sizes available on request.

PRESCON 'KC' SYSTEM

The Prescon 'KC' System is an economical range of couplers, designed for simple assembly on site. The first-stage tendon is stressed and anchored in the normal way using standard equipment and the dead-end of the second tendon is assembled around it, using swaged grips on each strand to afford maximum security.

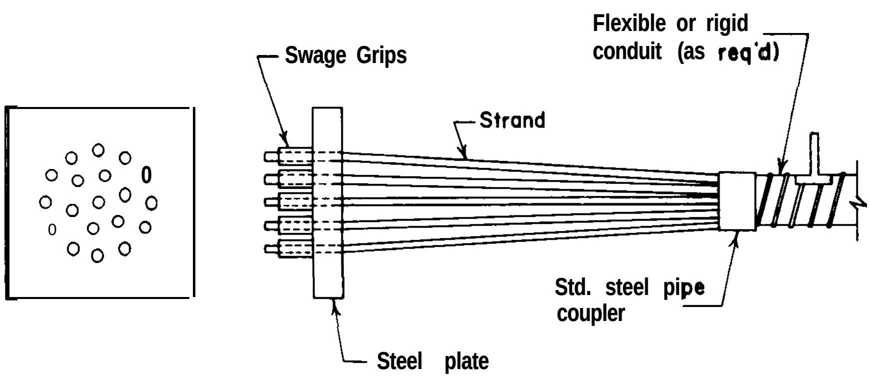
The coupler assembly is enclosed with a conical cover which has a grout access point for second stage grouting.



	12 K 5	12 K 6	19 K 5	27 K 5 19 K 6
A	5.31	5.12	5.51	5.71
B	8.00	8.93	8.93	11.57

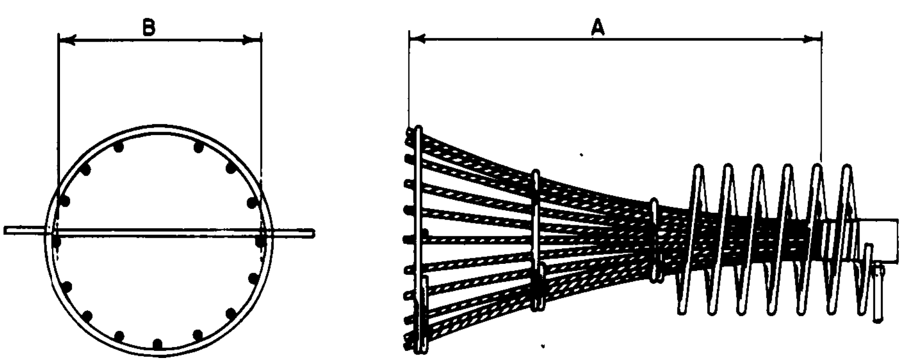
PRESCON 'SW' SYSTEM

The Prescon 'SW' System utilizes a plate and swaged grip, which enables the design force to be trans-mitted to the dead end plates. This anchorage system is used when the design force is required at the end of the structural member, and accessibility to this end is restricted. The standard 'K' System anchorage should be used if accessibility is not a problem. Size and exact design of this anchorage is dependent upon the restrictions of the structural system.



PRESCON 'BS' SYSTEM

The Prescon 'BS' System utilizes the bond length of the strand in order to develop the design force of the tendon. The use of this anchorage should be restricted to the following: when the design force may be located a distance from the end of thestructural member; accessibility to the end of the structural member is restricted. Exact design of this anchorage is dependent upon the restrictions of the structural system.



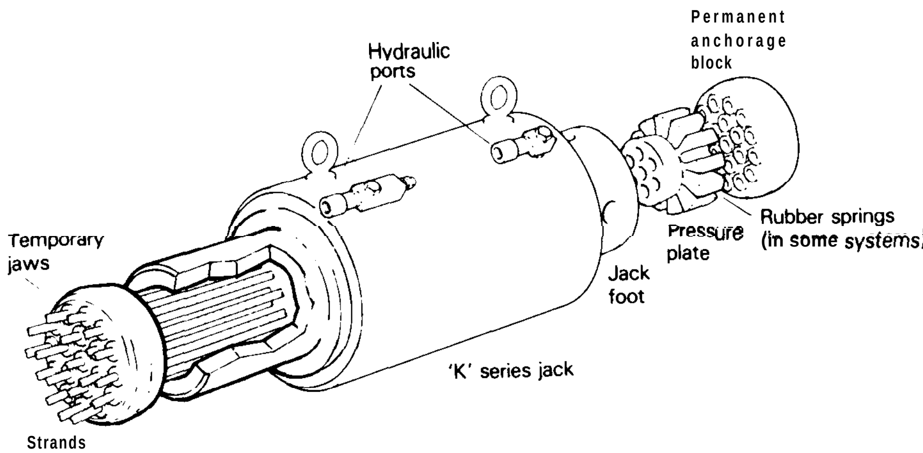
PRESCON TENSIONING EQUIPMENT

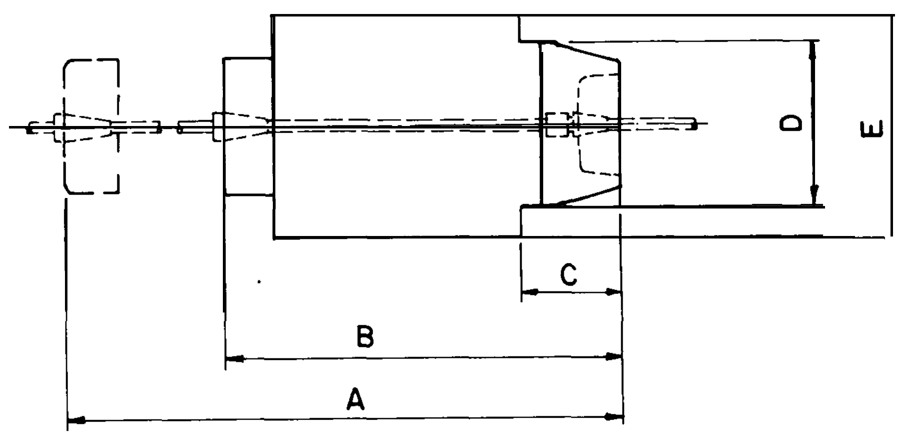
JACKS

The 'K' Range of jacks are center-hole rams of the hydraulic double-acting type with fixed cylinder and moving piston.

The attachment of the strand to the jack is by specially designed wide-angle, multi-use Jaws, which are self-releasing on completion of jacking.

The system of seating the permanent anchorages is by either a retaining plate or mechanical rubber springs which reduce the anchorage seating losses to a minimum.





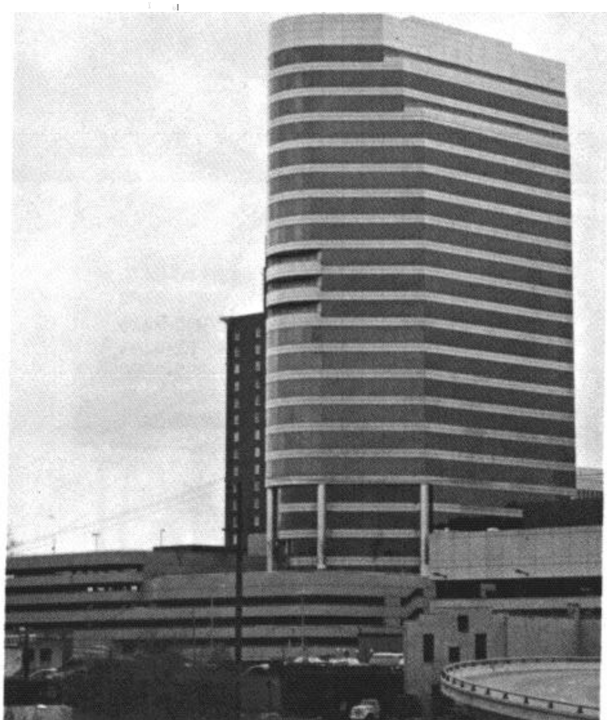
RAM	ANCHOR	A	B	C	D	E
K 100	7 K 5	29	21	3.25	7.75	11.75
K 200	12 K5	34	26	6.75	8.00	14.00
K 350	12K5-19K5	41	31	6.75	10.25	17.50
K 500	19K6-27K5	36	26	2.75	14.50	20.25
K 700	27K6-37K5	44	34	—	—	24.00
K 1000	37 K 6	50	40	10.75	9.50	28.50

2.2.14 STEEL SERVICE COMPANY, INC.

Since 1957, Steel Service Company, Inc., a Subsidiary of Gold Fields American Industries, Inc., has been an important part of the construction industry in the Southeast and Midwest furnishing reinforcing steel. In 1981, we expanded our operations to include post-tensioning materials. With our fabricating facility located at our homeoffice in Knoxville, Tennessee, we are able to easily service our marketing area by utilizing our own trucking fleet. All hardware used by Steel Service has been tested and meets or exceeds all requirements specified by the Post-Tensioning Institute.

Engineering

With Steel Service's engineering capabilities, we specialize in fast-track and design-build projects and supply both reinforcing bars and post-tensioning materials. We take a system approach with our design capabilities. We are able to assist in the design of any size structure so as to utilize both reinforcing bars and post-tensioning materials to their maximum capabilities according to Code Requirements. Thus, our customers can be guaranteed a cost-efficient concrete structure.



Riverview Tower, Knoxville, TN

Experience

In our relatively short existence in the post-tensioning area, we have been involved in a large listing of major projects. Our customers are our reputation and below is a partial listing of projects which we have been involved in.

1. Opryland Convention Center
   Nashville, TN
   Ross Bryan Engrs.
   Hardaway Const. Co.

2. Citizens National Bank Plaza
   Sevierville, TN
   Quickel & Bennett Engrs.
   Blalock Const. Co.

3. Chapple Office Bldg.
   Nashville, TN
   Quickel & Bennett Engrs.
   Sharondale Const. Co.

4. High Point Parking Structure
   High Point, NC
   PY-VAVRA Arch.-Engrs.
   Fowler-Jones Const. Co.

5. Highland Ridge Garage
   Nashville, TN
   Ross Bryan Engrs.
   Rogers Const, Co.

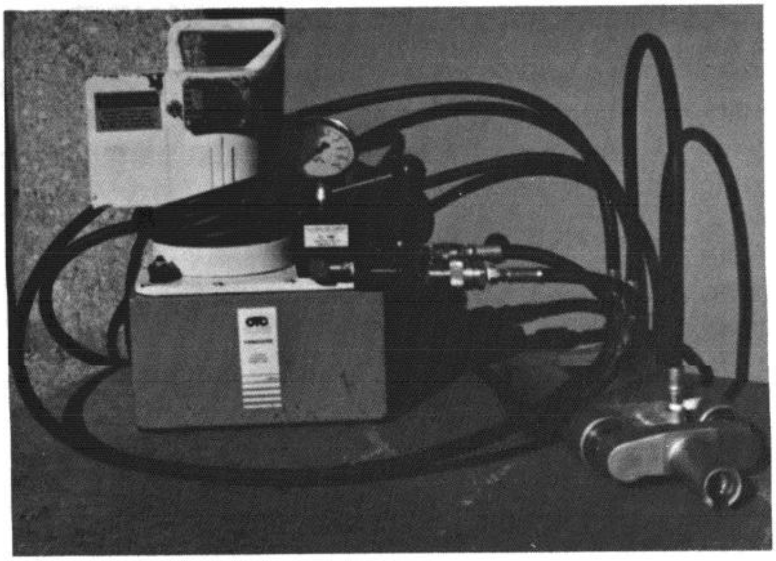
6. Sumner Memorial Hospital
   Gallatin, TN
   Hart, Freeland, Roberts, Inc.
   Ray Bell Const. Co.

7. Mayo Professional Bldg.
   Pikeville, KY
   White, Walker & Reynolds
   Ray Bell Const. Co.

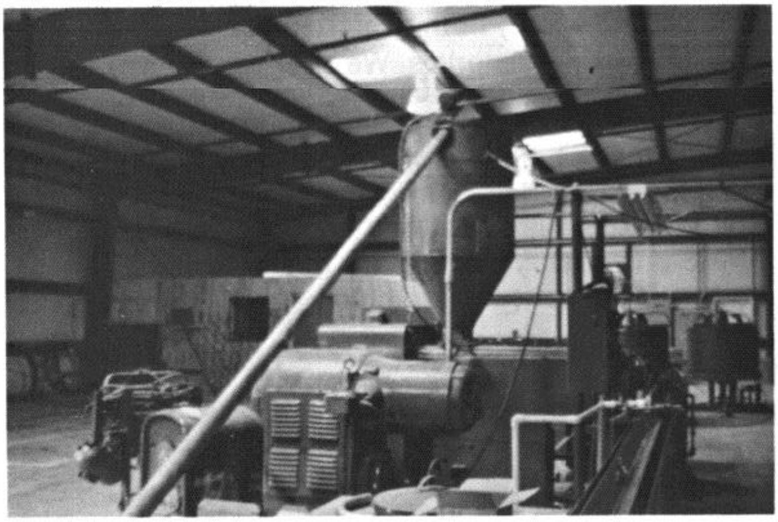
8. Huntington Center Parking Garage
   Columbus, OH
   S.O.M.
   Dugan & Meyers/Gust K. Newberg

9. Riverview Tower
   Knoxville, TN
   Bennett & Pless
   Johnson & Galyon, Inc.

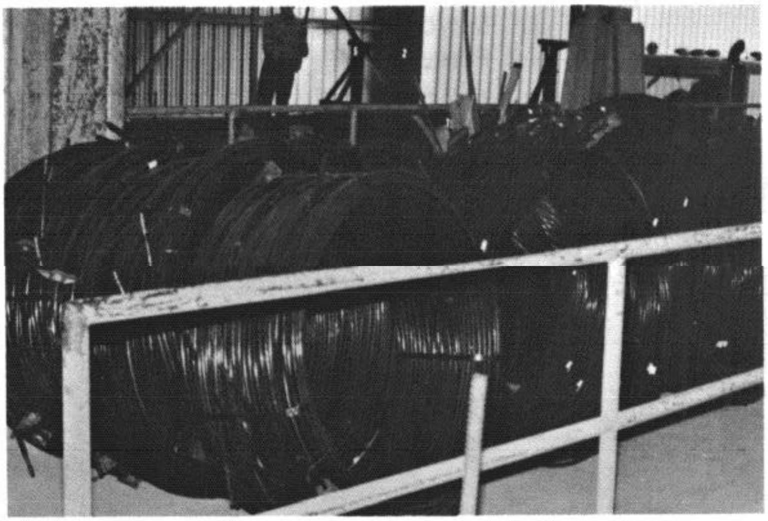
2.2.14 STEEL SERVICE COMPANY, INC. — continued



We utilize modern stressing equipment. Calibrated on a regular basis. All jacks are Open Throat for ease of stressing.



With fabricating facility in Knoxville, TN, we are able to use our existing trucking fleet to service jobs as required and avoid lengthy job site storage.



Offices

Knoxville, TN	615/546-5472
Nashville, Tn	615/832-6442
Louisville, KY	502/245-0256
St. Albans, WV	304/722-4291
Collierville, TN	901/853-8250
Johnson City, TN	615/928-0141
Indianapolis, IN	317/842-0071
Worthington, OH	614/431-1387
St. Louis, MO	314/993-8333

2.2.15 STRESSCON, INC.

INTRODUCTION

The dramatic increase in the use of post-tensioned concrete in recent years requires that engineers and contractors have access to the best information available regarding the design and construction of post-tensioned concrete structures. Stresscon was founded on the concept of providing this information in a timely and accurate manner, a factor that controls company policy and is reflected in on-the-job performance.

ENGINEERING SERVICES

Stresscon is committed to assist engineers and contractors in making decisions regarding the use of post-tensioning. For this reason, we maintain close business relationships with leading specialists in the design and construction of post-tensioned concrete structures. Services include feasibility studies, layout assistance, cost estimates, and final design.

PRODUCTS

Stresscon specializes in the fabrication of 1/2 inch diameter 270 k unbonded strand tendons. The tendons are fabricated with a corrosion inhibiting grease and a durable plastic sheath.

CONSTRUCTION SERVICES

Once the decision to use post-tensioned concrete has been made, we fabricate and ship to the jobsite 1/2" greased and plastic sheathed post-tensioning tendons, cut to length and color coded. All tendons are supplied in accordance with plans and specifications and approved shop drawings. Complete sets of stressing equipment, including on-the-job training whenever necessary, are provided, Also, in the case of a complete design-build concept, we provide all engineering, reinforcing steel, and post-tensioning steel furnished and installed. This work is sub-contracted to local rebar fabricators and placers, assuring close control of all work at the jobsite. Recent projectsare illustrated and listed below and on the following page.

2700 N Central Office Building and Parking Garage, Phoenix, Arizona

PROJECTS

One Columbus Parking Garage, Phoenix, Arizona

Project	— Koll Center North Parking Structure, Irvine, California		
Description	— 6 Level Parking Structure		
Architect	— Glencal, Inc.	Rebar Supplier	— Continental-Hagen Corp.
Engineer	— Teng Li & Associates	P.T. Supplier	— Stresscon, Inc.
Contractor	— Glencal, Inc.	Placer	— Continental-Hagen Corp.

Project	— Bonneville Office Tower, Las Vegas		
Description	— 8 Story Office Building		
Architect	— Winter Delamare Architects	Rebar Supplier	— Century Steel
Engineer	— Ralph Wadsworth	P.T. Supplier	— Stresscon, Inc.
Contractor	— Camco Construction Co.	Placer	— Century Steel

Project	— One Columbus Parking Structure, Phoenix		
Description	— 5 Level Parking Garage		
Architect	— Desmond Parking Association	Rebar Supplier	— Reppel Steel
Engineer	— Desmond Parking Association	P.T. Supplier	— Stresscon, Inc.
Contractor	— McCarthy-Western Co.	Placer	— Sentry/J.D. Steel Co.

Project	— 6th and G Street Garage, Anchorage		
Description	— 8 Story Parking Structure		
Architect	— Kumin Associates	Rebar Supplier	— Alcan Steel
Engineer	— KPFF	P.T. Supplier	— Stresscon, Inc.
Contractor	— Hoffman Construction Co.	Placer	— Alcan Steel

Project	— 2700 North Central, Phoenix		
Description	— 14 Story Office Building and 6 Story Parking Structure		
Architect	— Opus Corporation	Rebar Supplier	— Arizona Rebar
Engineer	— Opus Corporation	P.T. Supplier	— Stresscon, Inc.
Contractor	— Opus Corporation	Placer	— Bear River Steel Co.

Project	— Union Park Center II, Salt Lake City		
Description	— 7 Story Office Building & 2 Story Parking Structure		
Architect	— Niels E. Valentiner	Rebar Supplier	— Masco, Inc.
Engineer	— Reevly Engineering	P.T. Supplier	— Stresscon, Inc.
Contractor	— Jacobsen Morrin Robbins	Placer	— Masco, Inc.

Project	— The Abacus Tower, Phoenix		
Description	— 17 Story Office Building		
Architect	— Allen & Philip Architects	Rebar Supplier	— Continental-Hagen Corp.
Engineer	— Robin E. Parke & Associates	P.T. Supplier	— Stresscon, Inc.
Contractor	— The Weitz Company	Placer	— Continental-Hagen Corp.

Project	— Quincy Park Apartments, Denver		
Description	— 8 Story Apartment Building		
Architect	— Lombardi & Associates	Rebar Supplier	— J. D. Steel Co.
Engineer	— Gerald Schlegel	P.T. Supplier	— Stresscon, Inc.
Contractor	— Commercial Forming Co.	Placer	— J. D. Steel Co.

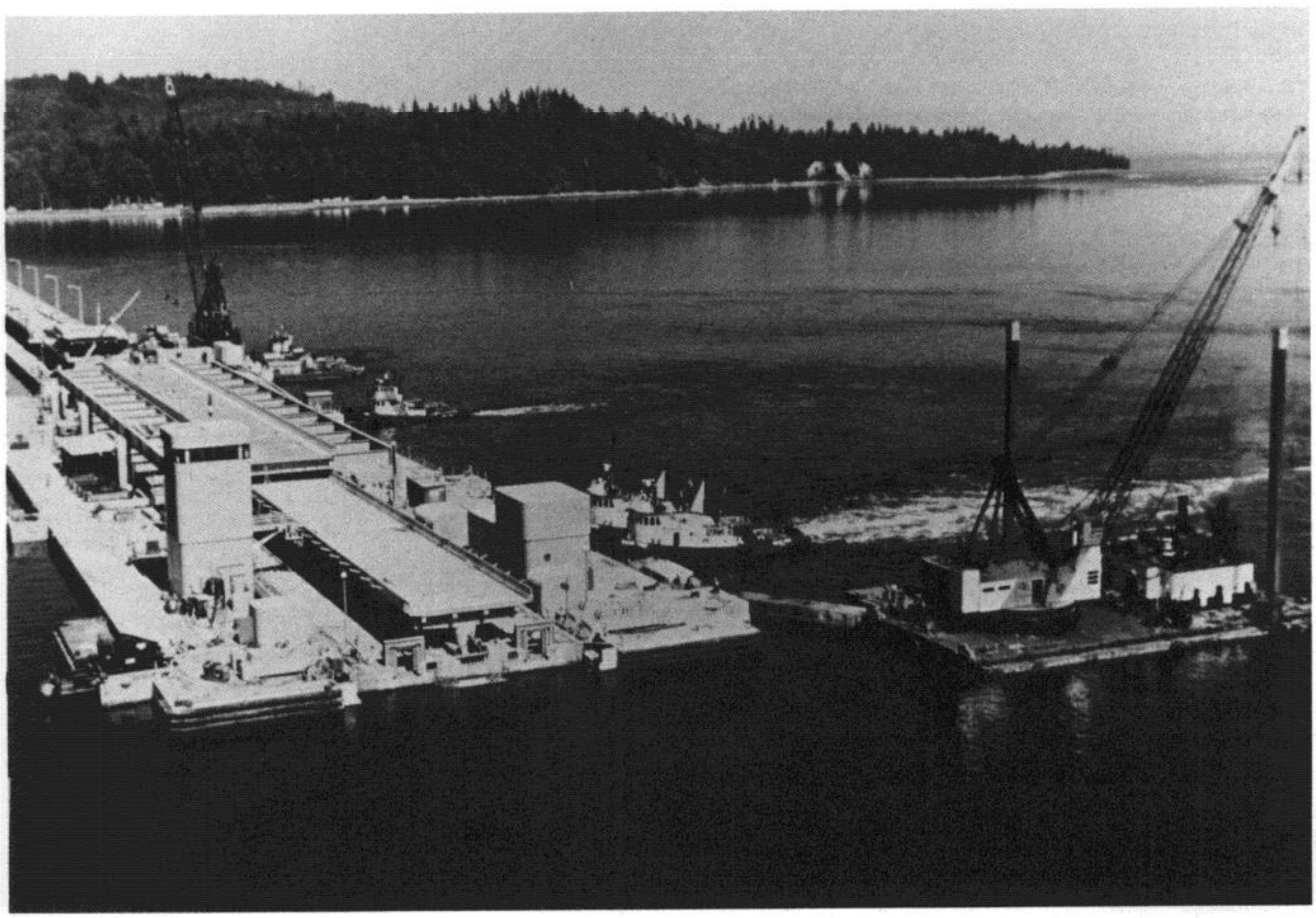
2.2.16 STRESSTEK DIVISION, CONMAR CORPORATION

STRESSTEK POST-TENSIONING

The Stresstek post-tensioning system offers a range of tendon sizes from single strand tendons through 37 strand tendons in both one-half inch and .6 inch diameter 270k strand. Tendon sizes can vary by increments of one strand. Compact section strand can also be used, giving the 37.6 tendon an ultimate strength in excess of 1,250 tons. In addition to these features of flexibility with respect to range of forces, a variety of anchorages and equipment options are available in the Stresstek System which results in great versatility for prestressing applications.

Stresstek tendons are available in both the grouted (bonded) design and greased (unbonded) design in all sizes.

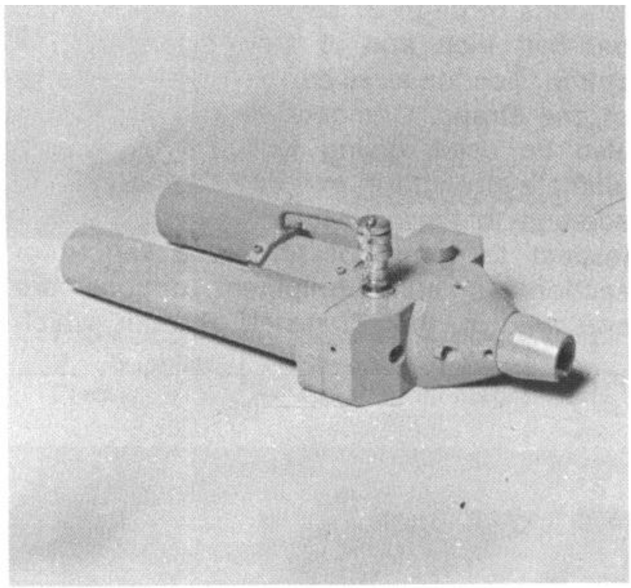
Reliability of Stresstek tendons and equipment has been a consideration leading to its use in many major projects in the United States and abroad. Some of these projects, along with basic data describing the Stresstek System are presented in the following pages.



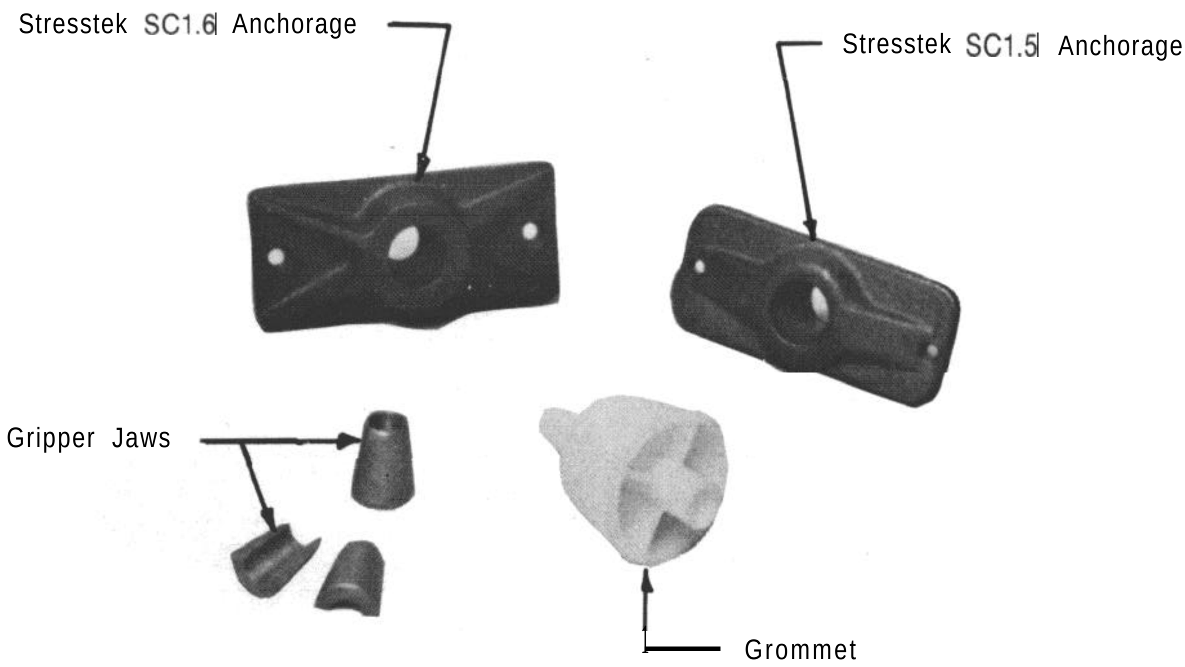
A navigation unit for a new floating bridge across the Hood Canal demonstrates an interesting application of the Stresstek post-tensioning system. Buoyant post-tensioned concrete pontoons as long as 312 feet were constructed in a graving dock, floated out and joined with other elements to form an important link in Washington's highway system. Closed length (shown) is 900 feet. When open to highway traffic the length is 1,200 feet.

STRESSTEK MONOSTRAND SYSTEM

The majority of unbonded installations utilize single-strand tendons. The following illustrations relate specifically to Stresstek monostrand cables. The 270K seven-wire strand conforming to ASTM A-416, because of its reliability and economy, is the most commonly used prestressing steel in the world. This material, protected by a special rust preventative grease, encased in a plastic sheathing, and fitted with specially designed ductile anchorages, comprises the Stresstek unbonded tendon as normally used in building construction.



The standard stressing unit of Stresstek monostrand tendons is a saddle ram which permits gripping the strand at any intermediate point along the tendon.



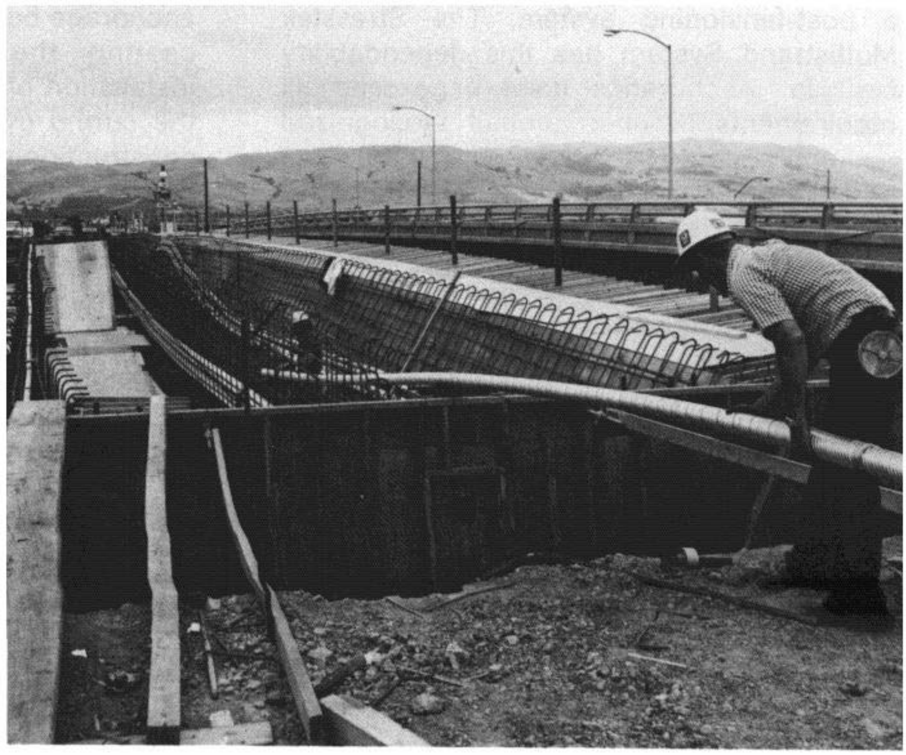
STRESSTEK MULTISTRAND SYSTEM

Dependability is the most important feature of a post-tensioning system. The Stresstek Multistrand System has this dependability built in. Anchorages meet or exceed all requirements of internationally recognized building codes. The system utilizes the most efficient prestressing steel: seven-wire, 270K strand, which is astandard, highly reliable and readily available material.

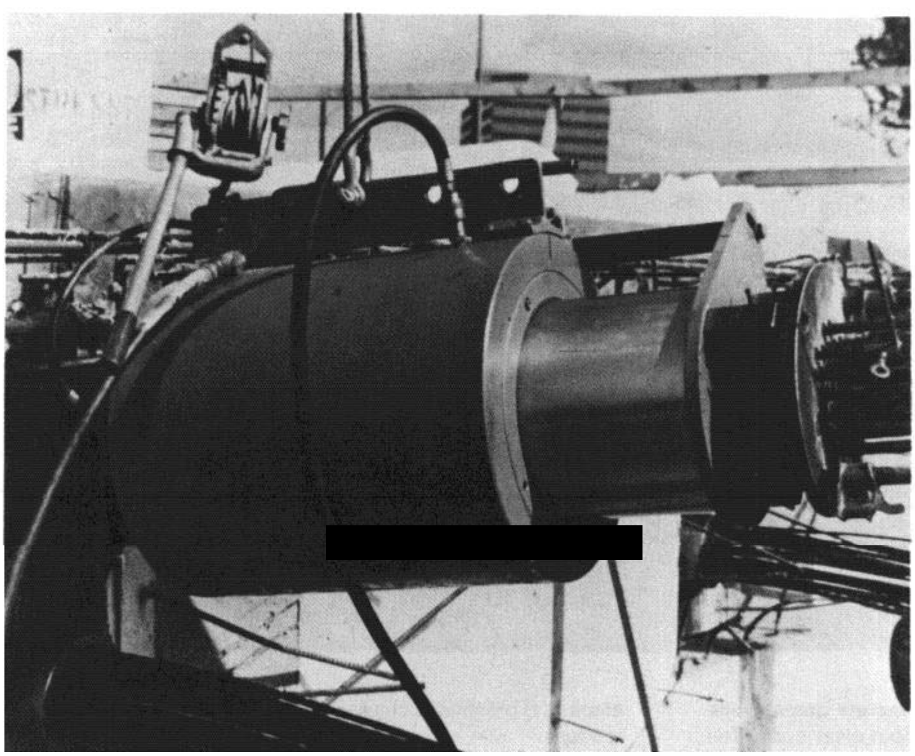
Although prefabricated post-tensioning tendons are available, the most common methods of installing the Stresstek System are by the "pull-through" and "push through" methods. Only the tendon sheathing and the anchorage bearing plates are placed prior to casting the member. Postponing the installation of prestressing steel until during the curing cycle considerably shortens the construction schedule, but more importantly, it minimizes corrosion. The prestressing steel is installed, stressed (when concrete strength is attained) and immediately protected by injecting cement grout or a corrosion inhibiting grease. The length of time the prestressing steel is exposed to environmental corrosion is thus brought to a minimum.

Seattle's Kingdome boasts the largest concrete dome in the world. This unique structure has a 660 foot clear span. The stadium is prestressed by a wide range of Stresstek multistrand tendons.

2.2.16 STRESSTEK DIVISION, CONMAR CORPORATION — continued



Typical duct installation method for Stresstek multistrand tendons is pictured. Duct weighing only 1/2 to 2 pounds per toot is placed prior to concreting. Prestressing steel is later installed by pushing or pulling strand through duct. This method results in excellent quality control and the most cost effective means of tendon placement.



Stresstek equipment is designed with reliability and ease of use in mind. Multistrand tendons are stressed with center-hole jacks fitted with a means of controlling anchor set. A 500 ton unit is shown stressing a S19.6 tendon,

TENDON SELECTION

Structural analysis will establish the necessary final post-tensioning force (Pf) required for the element under design. This force is the minimum nominal force which remains through the life of the structure after the effects of shrinkage, creep, elastic shortening, relaxation, friction, etc. have taken place.

The following tables are convenient for choosing Stresstek multistrand tendons for specified force requirements. The table shows the maximum number of strands and other data for a given anchorage size. Any lesser number of strands than the maximum may be used.

½" DIAMETER 270K STRAND SYSTEM

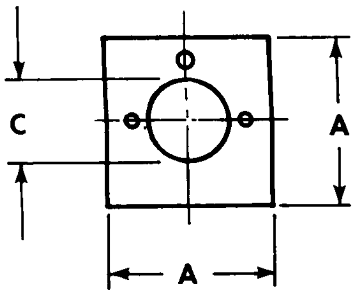
Anchorage Designation	Tendon Size	Steel Area		Weight		Temporary Jacking Force @ 0.80 f's		Maximum Anchor Force @ 0.70 f's		Minimum Duct Size	
	Maximum Number of ½" 270K Strands	Sq. mm.	Sq. in.	Kg/meter	Lb/foot	KN	Kips	KN	Kips	Mm	Inches
S4.5	4	395	0.612	3.126	2.100	589	132	515	116	45	1.75
S7.5	7	691	1.071	5.471	3.675	1,030	231	902	202	57	2.25
S13.5	13	1,283	1.989	10.160	6.825	1,914	430	1,674	376	70	2.75
S19.5	19	1,875	2.907	14.849	9.975	2,797	628	2,447	549	83	3.25
S31.5	31	3,060	4.743	24.227	16.275	4,563	1,024	3,993	896	108	4.25
S37.5	37	3,652	5.661	28.916	19.425	5,446	1,223	4,765	1,070	114	4.50

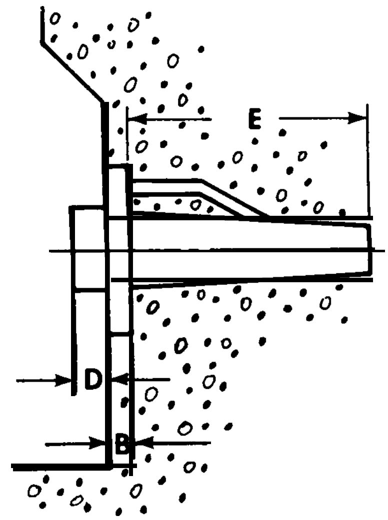
.6" DIAMETER 270K STRAND SYSTEM

Anchorage Designation	Tendon Size	Steel Area		Weight		Temporary Jacking Force @ 0.80 f's		Maximum Anchor Force @ 0.70 f's		Minimum Duct Size	
	Maximum Number of .6" 270K Strands	Sq. mm.	Sq. in.	Kg/meter	Lb/foot	KN	Kips	KN	Kips	Mm	Inches
S4.6	4	560	0.868	4.397	2.954	834	187	731	164	45	1.75
S7.6	7	980	1.519	7.696	5.170	1,463	328	1,280	287	64	2.50
S13.6	13	1,821	2.821	14.290	9.600	2,716	609	2,377	533	83	3.25
S19.6	19	2,661	4.123	20.888	14.032	3,969	890	3,474	779	95	3.75
S31.6	31	4,342	6.727	34.080	22.894	6,480	1,453	5,669	1,271	121	4.75
S37.6	37	5,182	8.029	40.676	27.325	7,734	1,734	6,766	1,517	133	5.25

TENDON DATA

ANCHORAGES WITH FABRICATED BEARING PLATE/TRUMPET





½" Diameter Strand System

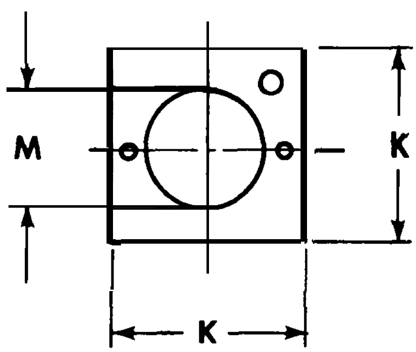
Nomenclature		Anchorage Designation											
		S4.5		S7.5		S13.5		S19.5		S31.5		S37.5	
		Cm	Inches	Cm	Inches	Cm	Inches	Cm	Inches	Cm	Inches	Cm	Inches
Bearing Plate	A	16.5	6.5	21.0	8.25	29.2	11.5	35.6	14.0	44.5	17.5	49.5	19.5
Bearing Plate Thickness	B	1.9	0.75	3.2	1.25	3.8	1.5	5.1	2.0	5.7	2.25	6.4	2.5
Anchorhead Diameter	C	11.0	4.35	11.0	4.35	15.2	6.0	17.3	6.85	22.5	8.87	27.9	10.75
Anchorhead Thickness	D	4.4	1.75	4.4	1.75	6.6	2.6	7.6	3.0	8.6	3.4	10.2	3.68
Trumpet Length	E	20.3	8.0	30.5	12.0	40.6	16.0	50.8	20.0	61.0	24.0	61.0	24.0

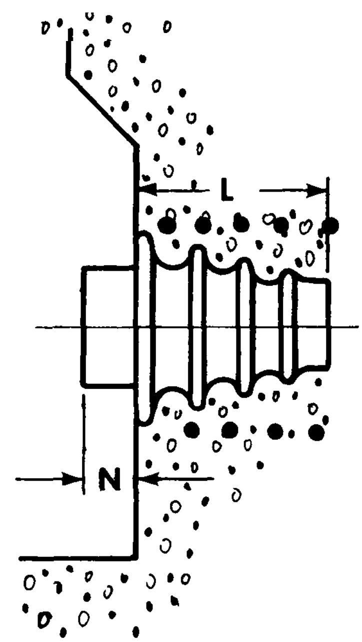
.6" Diameter Strand System

Nomenclature		Anchorage Designation											
		S4.6		S7.6		S13.6		S19.6		S31.6		S37.6	
		Cm	Inches	Cm	Inches	Cm	Inches	Cm	Inches	Cm	Inches	Cm	Inches
Bearing Plate	A	19.1	7.5	24.8	9.75	34.3	13.5	40.6	16.0	53.3	21.0	58.4	23.0
Bearing Plate Thickness	B	2.5	1.0	3.5	1.37	5.1	2.0	6.4	2.5	7.6	3.0	8.9	3.5
Anchorhead Diameter	C	11.0	4.35	13.6	5.35	17.4	6.85	19.9	7.85	25.0	9.85	27.9	11.0
Anchorhead Thickness	D	4.4	1.75	6.4	2.5	7.6	3.0	8.6	3.4	11.3	4.44	12.1	4.75
Trumpet Length	E	30.0	12.0	30.0	12.0	50.8	20.0	58.4	23.0	71.1	28.0	91.4	36.0

TENDON DATA

ANCHORAGES WITH CAST BEARING PLATE/TRUMPET



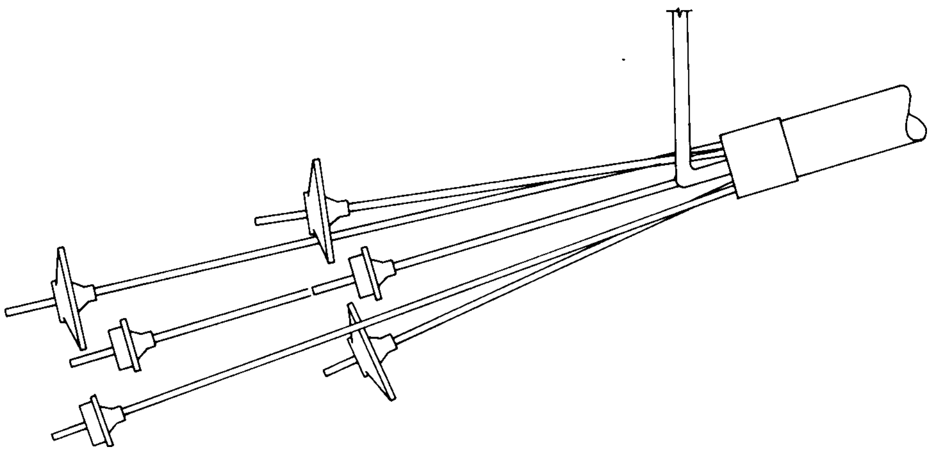


Nomenclature		Anchorage Designation							
		SCB 13.5		SCB 19.5		SCB 13.6		SCB 19.6	
		Cm	Inches	Cm	Inches	Cm	Inches	Cm	Inches
Casting	K	22.9	9.0	27.9	11.0	27.9	11.0	35.6	14.0
Casting Length	L	22.1	8.7	26.7	10.5	26.7	10.5	40.6	16.0
Anchorhead Diameter	M	15.2	6.0	17.3	6.8	17.3	6.8	19.9	7.85
Anchorhead Thickness	N	6.6	2.6	7.6	3.0	7.6	3.0	8.6	3.4

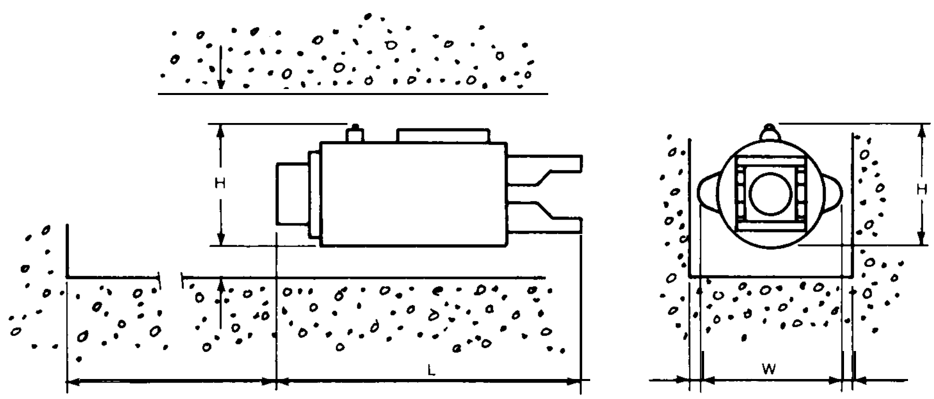
SPLAYED DEAD-END ANCHOR

The splayed dead-end anchorage solves a placing problem in some cases where reinforcement congestion in the anchorage zone makes placing other types of anchorages difficult.

Note that the monostrand castings which anchor each strand individually in the concrete are splayed and staggered to permit adequate space for placing concrete around them and the distribution of anchorage stresses in the concrete over a large area.



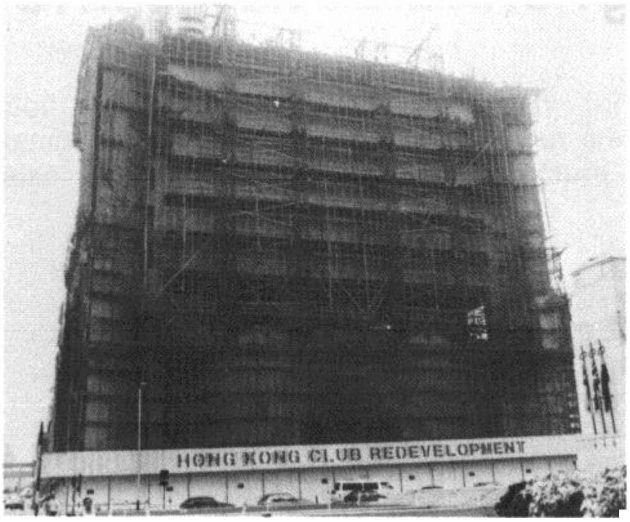
MULTISTRAND JACKING EQUIPMENT



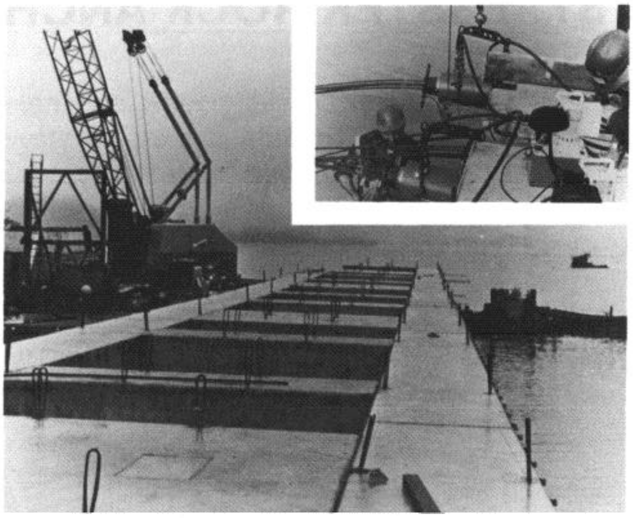
Stresstek Jack Dimensions, Weights and Working Clearances

Capacity, Tons	W, Inches	L, Inches (1)	H, Inches	Center Hole Diameter, Inches	Approximate Weight, Lbs.
100 Ton	14	33	13	3.1	400
200 Ton	17	37	16	4.4	800
400 Ton	22	40	21	5.4	1,500
500 Ton	25	42	26	7.8	3,100
800 Ton	30	43	30	10.5	3,600

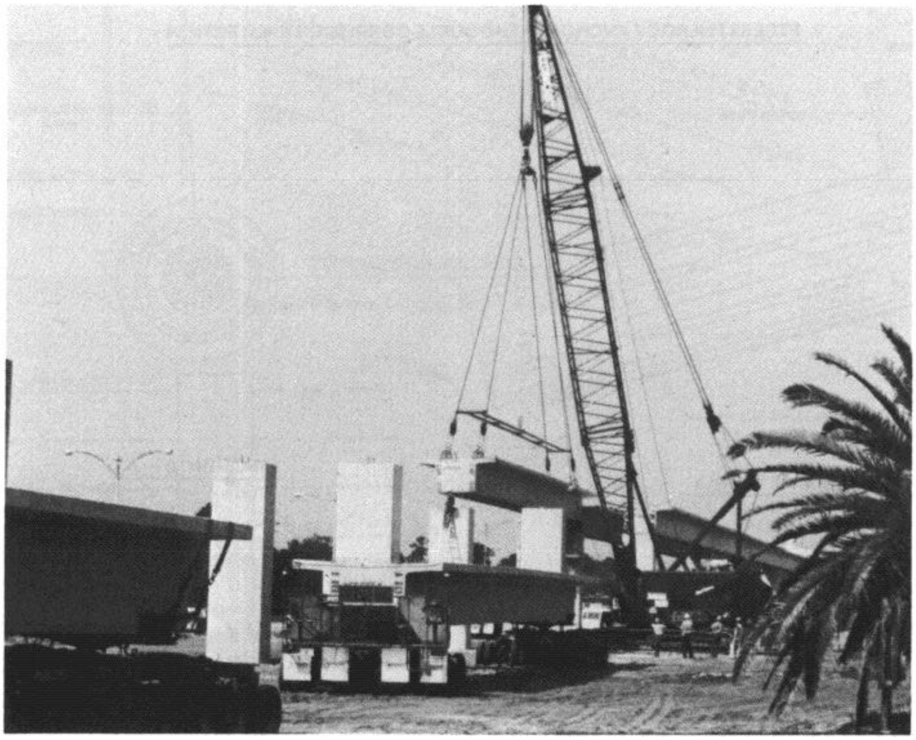
(1) Lengths shown are for rams with 12 inch strokes, except for 100 ton jack which has an 8 inch stroke.



The Hong Kong Club employs Stresstek tendons to provide long clear spans for this landmark in the Colony.

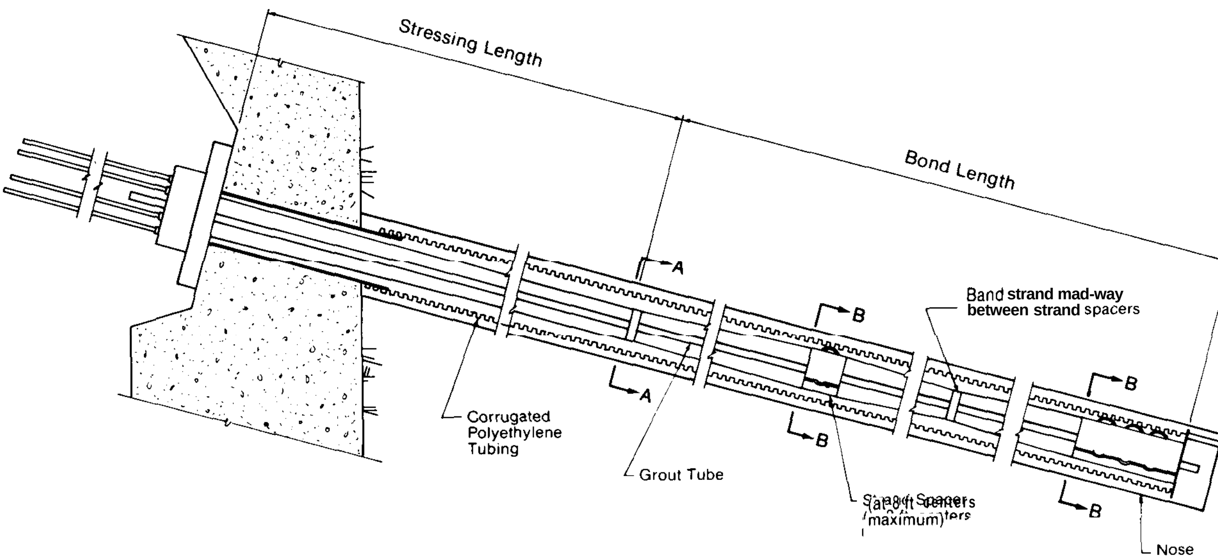


Stresstek prestressing methods contribute to the success of some unusual construction as illustrated here on a floating concrete breakwater at Auk Bay, Alaska. Tendons were threaded through ductsembedded in precast segments, joining 40 foot units to form 240 elements of a breakwater 1,200 feet in length.

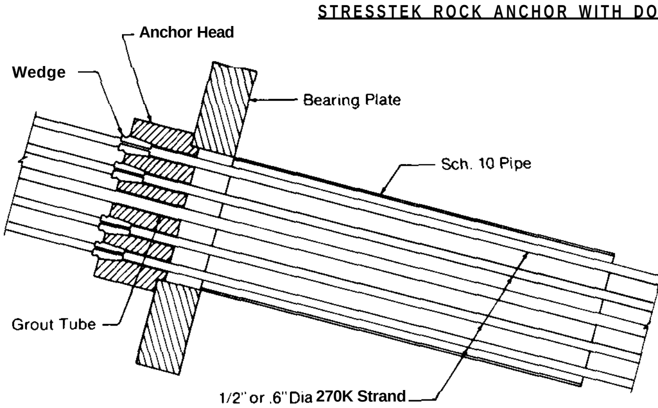


Stresstek System services and reliable equipment helped RTJV (Stevin, Capiletti and LT Contracting) produce 350 precast, post-tensioned girders on schedule for the Dade County Rapid Transit in Miami, Florida.

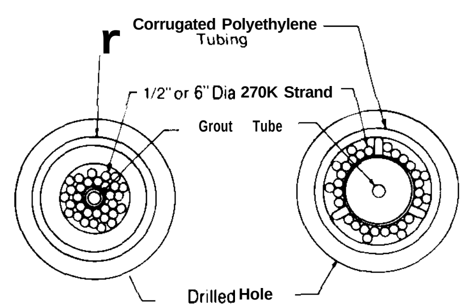
STRESSTEK ROCK ANCHORS



STRESSTEK ROCK ANCHOR WITH DOUBLE CORROSION PROTECTION



ANCHORAGE ASSEMBLY



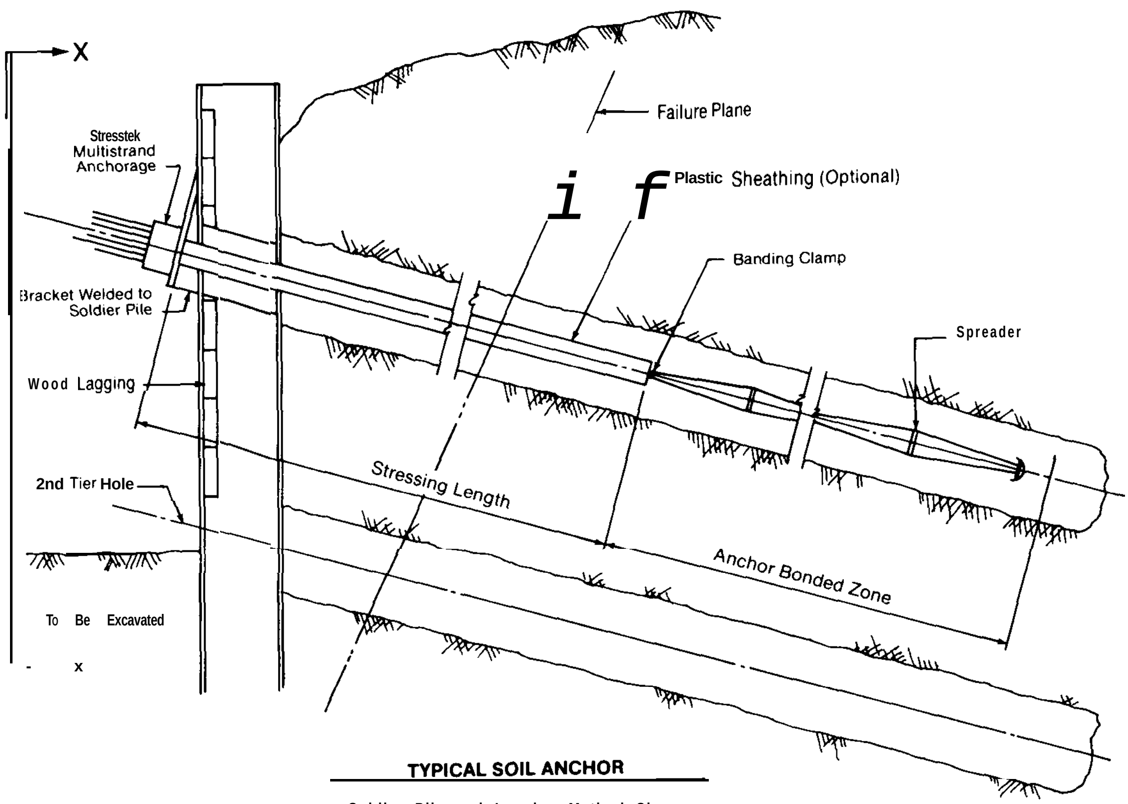
SECTION A - A

SECTION B - B

Design requirements for rock anchors vary considerably, making it necessary to have many optional features available. The above drawing illustrates a Stresstek anchor having a secondary barrier against corrosive elements ("double corrosion protection"). A third barrier can also be provided for Stresstek tendons where extreme conditions exist.

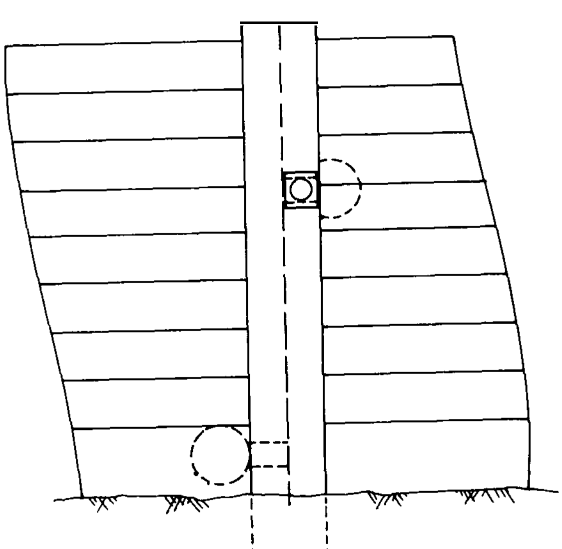
STRESSTEK SOIL ANCHORS

Soil tie-backs are used extensively for temporary construction (illustrated) but are also employed as permanent structure elements. Specialists in the design of tie-back systems are available through area Stresstek representatives.



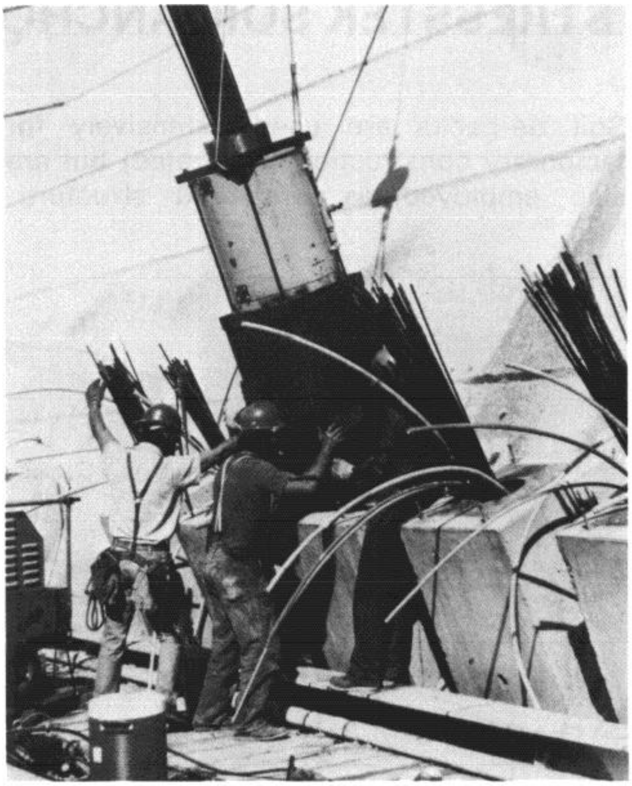
TYPICAL SOIL ANCHOR

Soldier Pile and Lagging Method Shown

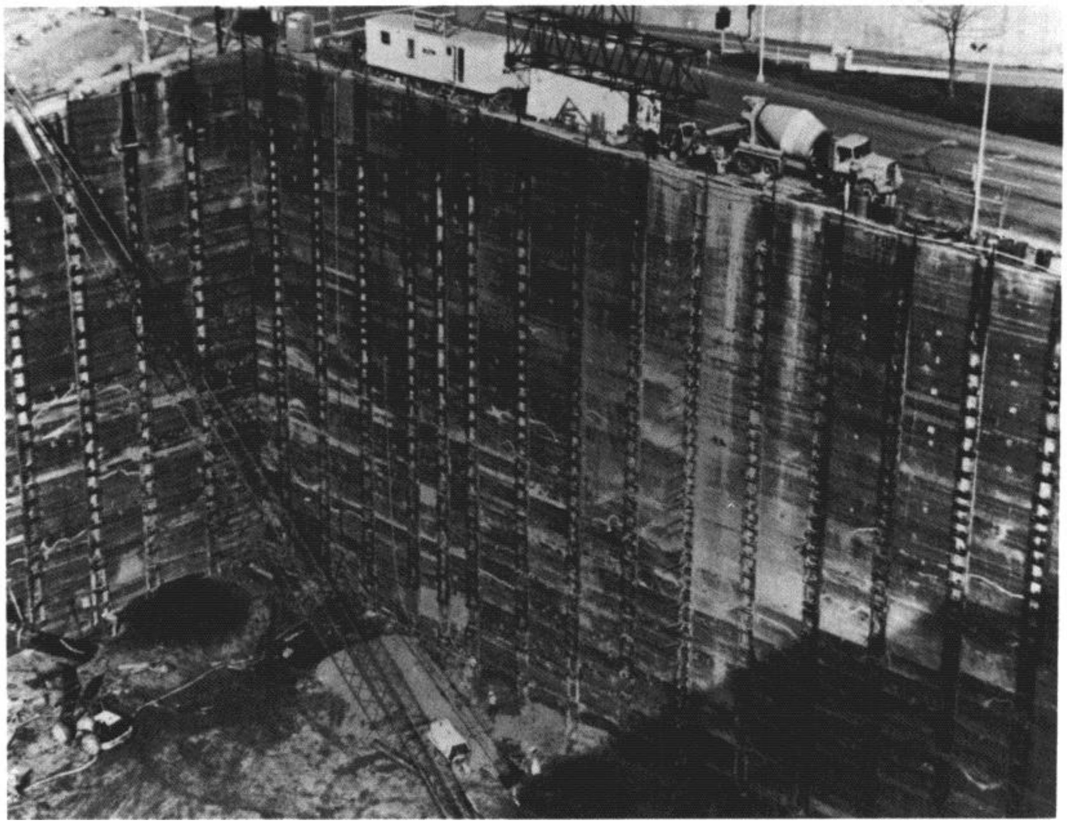


VIEW X - X

Stresstek S37.6 rock anchors helped stabilize a navigation lock wall at John Day Dam on the Columbia River. Tendons were designed to permit installation and removal of load cells under the anchorheads, without detensioning, for later monitoring of prestressing force. All tendons have an ultimate capacity of 2,168 kips each.



Stresstek tendons used as earth anchors tie back one of America's deepest building excavations for the Columbia Center in downtown Seattle.

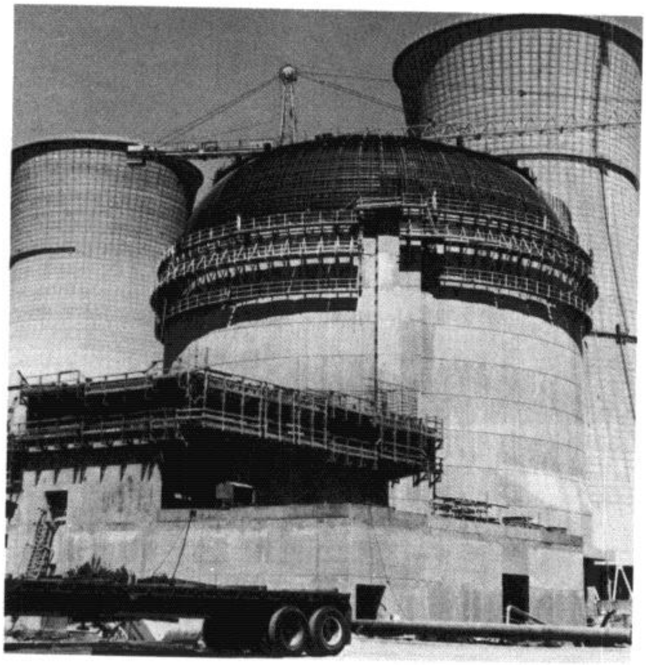


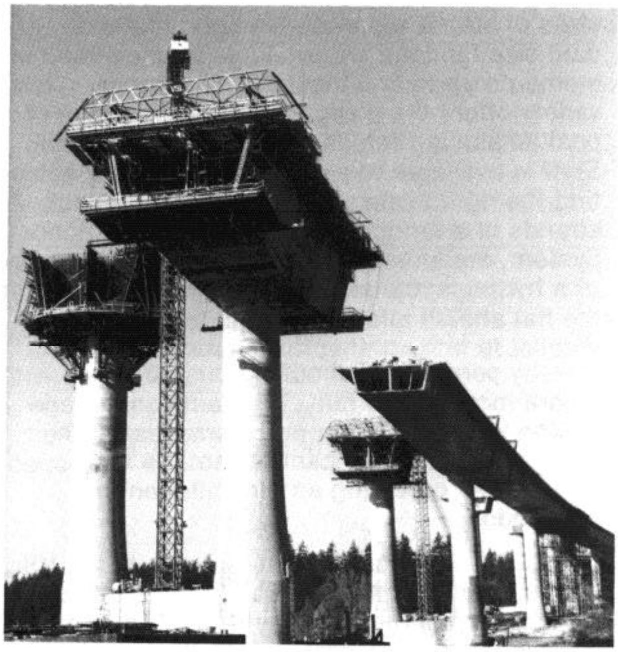
2.2.17 VSL CORPORATION

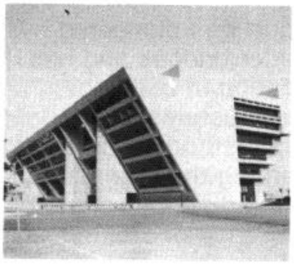
VSL Corporation

Introduction

VSL Corporation is a diversified firm engaged in construction and transportation projects which require special know-how and innovative engineering, manufacturing and construction capabilities. Since its inception in 1966, the company has strived to offer a broad and technically superior line of systems and services for all phases of a project. Today, the company is well recognized as one of North America's leading specialty contractors, with offices and plants located throughout the United States.







VSL Multistrand System

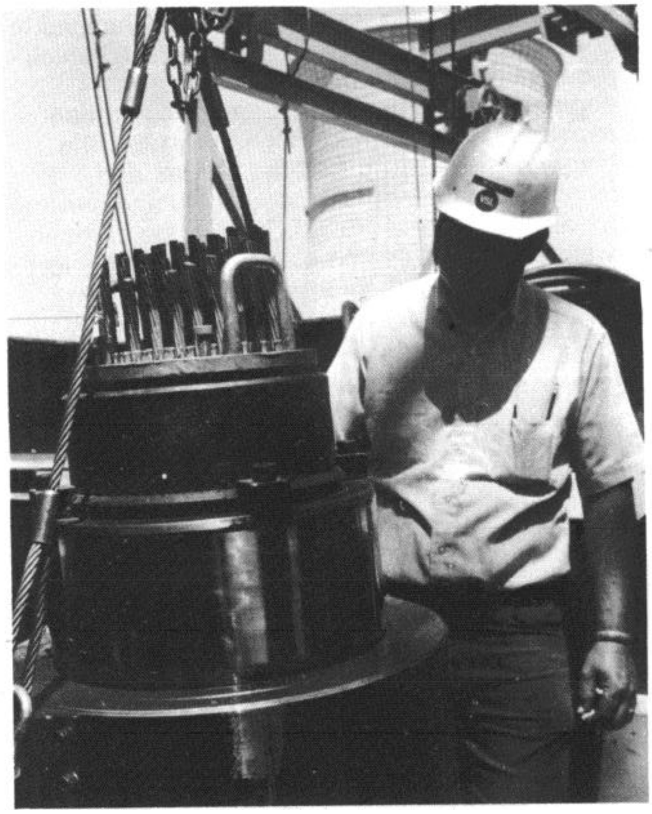
The anchorage components of the VSL Multi-strand System have remained unchanged in principle for over twenty years. With these proven anchorages, VSL has been able to concentrate on applications which in many ways have revolutionized the use of post-tensioning.

The VSL Multistrand System employs 0.5 or 0.6 inch diameter, 270K strand manufactured in accordance with ASTM A416. Other types and sizes of strand are available upon request. Standard size tendons are available in one-strand increments up to and including 55 strands. This variety offers the designer a large selection of post-tensioning forces, and VSL's Engineering Staff is available to provide assistance in selecting the right system for the specific project. All strands of a tendon, except for the SO Anchorage System, are stressed simultaneously by means of a hydraulic center hole jack. In the SO System, the flat sheath allowsthe strands to be held parallel to one another during stressing and thereby permits nonsimultaneous stressing with a light monostrand ram. The same anchorage design is applicable from the smallest to the largest unit - a significant advantage for inspection, stocking, testing and installation of the tendons.

One of the most significant features of the VSL Multistrand System is its ability to be installed using the "pull-through" method. With this method, only the sheath is installed prior to the placement of concrete. The prestressing steel is not placed until after concrete cure, when the structure is ready to be stressed, thus eliminating the possibility of corrosion. The reduction in weight, due to the initial absence of prestressing steel in the sheath, greatly facilitates the placing operation and results in a decrease of elapsed time between concrete pours. Placement of the strand in the sheath occurs during the curing cycle which is float time on the critical path.

The "pull-through" technique was pioneered by VSL Corporation in box girder bridges and has subsequently found application in buildings, nuclear reactors and other structures. Semi-rigid sheath is employed for this method of installation which, in addition to preventing the entrance of concrete mortar, provides the distinct advantage of reduced friction. The absence of prestressing steel prior to placement of concrete and the nature of semi-rigid sheath sharply reduce the wobble coefficient. A reduction in the amount of prestressing steel results. The alternate method of installation uses a shop-fabricated tendon employ ing flexible sheath.

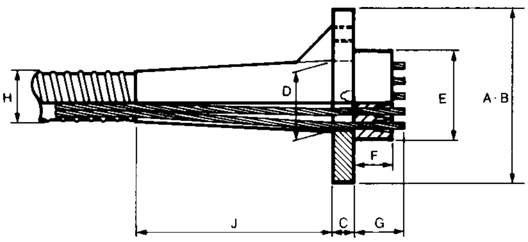
For either type of tendon, cement grout or corrosion inhibiting grease is used for protection.



Anchorage Components

VSL E Stressing Anchorage

This anchorage is composed of an anchor head, wedges, and a bearing plate with trumpet. The bearing plate is positioned in the structure concurrently with the formwork, whereas the anchor head is not placed until the time of stressing the tendons.

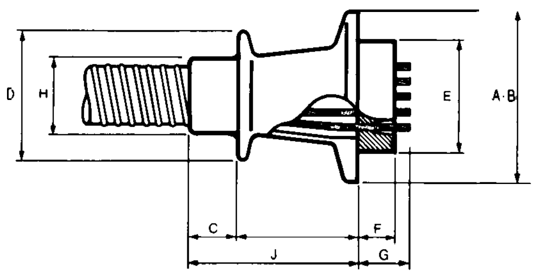


Anchorage Type	VSL 5-3	5-7	5-12	5-19	5-31	5-55	6-12	6-19
Ultimate Capacity (kips)	124	289	496	785	1280	2272	703	1114
A	5.50	8.25	10.75	13.50	17.50	24.00	12.25	15.75
B	5.50	8.25	10.75	13.50	17.50	24.00	12.25	15.75
C	0.75	1.25	1.50	1.75	2.25	3.00	1.75	2.25
DO	1.88	2.88	4.13	5.25	6.75	9.00	4.63	6.00
Eø	3.38	4.50	6.00	7.00	9.00	12.50	7.00	8.75
F	2.40	2.40	2.40	2.95	3.95	6.00	3.15	3.95
G	3.40	3.40	3.40	4.00	5.00	7.00	4.15	5.00
Hø	2.12	2.12	2.81	3.56	4.50	6.00	3.56	4.31
J	4.00	8.00	12.00	23.00	34.00	42.00	18.00	28.00

Dimensions in inches.

VSL EC Stressing Anchorage

This anchorage is distinguished from the VSL E by the way in which it transmits the load to the concrete. This is done through an integral bearing plate incorporating the trumpet and an intermediate flange. This arrangement permits a reduction in the bearing area of the anchorage. It is therefore used where the space available at the end of the element is restricted.

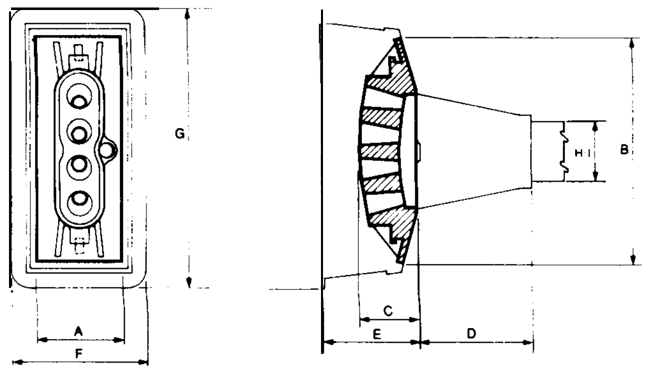


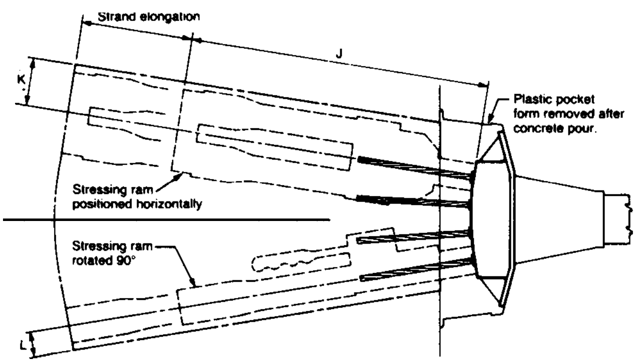
Anchorage Type	VSL 5-3	5-7	5-12	5-19	5-31	6-12	6-19
Ultimate Capacity (kips)	124	289	496	785	1280	703	1114
A	5.32	6.89	8.86	11.00	13.98	10.43	13.00
B	5.32	6.89	8.86	11.00	13.98	10.43	13.00
C	1.96	2.17	2.17	2.17	2.56	2.36	2.56
De	3.94	5.12	6.69	8.27	10.43	7.87	9.84
Eø	3.38	4.50	6.00	7.00	9.00	7.00	8.75
F	2.40	2.40	2.40	2.95	3.95	3.15	3.95
G	3.40	3.40	3.40	4.00	5.00	4.15	5.00
He	1.94	3.15	3.78	4.53	5.67	4.53	4.72
J	4.92	5.32	7.09	10.24	13.39	9.95	11.42

Dimensions in inches.

VSL SO Stressing Anchorage

This anchorage is used for four-strand grouted tendons placed in flat sheathing. The strands are stressed individually by a monostrand ram and locked off in the anchor head which bears on the embedded plastic form.



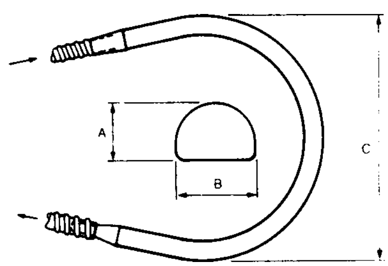


Anchorage Type	VSL SO5-4	SO6-4
A	3.50	3.50
B	11.00	11.00
C	2.87	2.87
D	6.25	6.25
E	5.00	5.00
F	5.62	5.62
G	13.00	13.00
H	3.00	3.00
I	1.00	1.00
J	24.00	24.00
K	4.00	4.00
L	2.50	2.50

Dimensions in inches.

VSL L Fixed-End Anchorage

With this anchorage the profile of the tendon describes an arc of 180°, enabling the tendon to be brought back to the vicinity of its starting point. The anchorage consists of a rigid U-shaped tube of semi-circular cross section. Normal reinforcement should be provided to prevent cracking of the concrete behind the anchorage.

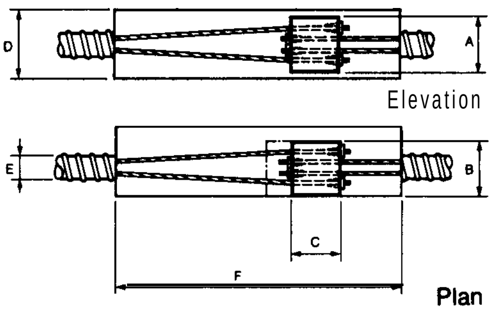


Anchorage Type	VSL 5-3	5-7	5-12	5-19	5-31	6-12	6-19
A	2.50	2.50	2.50	3.50	4.50	3.50	4.50
B	3.00	3.00	4.00	6.25	9.50	6.25	9.50
C	48.00	48.00	48.00	50.00	51.00	50.00	51.00

Dimensions in inches.

VSL Z Center Stressing Anchorage

This anchorage consists of a prismatic block containing holes through which the strands from the two ends of the tendon pass. The strands are locked off by wedges, held in place on the passive side by-retainer plates.

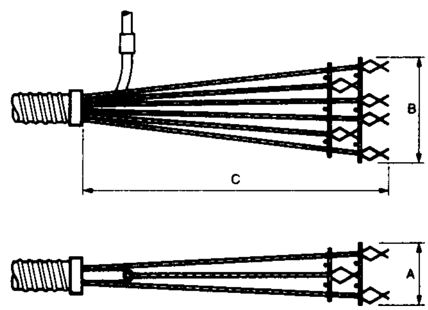


Anchorage Type	VSL 5-4	5-6	5-12	6-12
A	6.30	7.88	11.00	11.85
B	3.50	5.25	5.50	6.30
C	2.75	3.55	5.50	6.30
D	7.00	8.50	12.50	13.75
E	2.12	2.12	2.75	3.50
F	30.00	42.00	60.00	68.00

Dimensions in inches

VSL H Fixed-End Anchorage

Post-tensioning force is transmitted to the concrete by bond on the exposed length of the strands and by the bulb-shaped forms at their ends. The bulbs may be arranged in either a square or a rectangular pattern. A rebar grid at the end of the anchorage keeps the strands in position. Mild reinforcement and a clamping ring prevent bursting of the concrete at the point where the strands start to diverge.



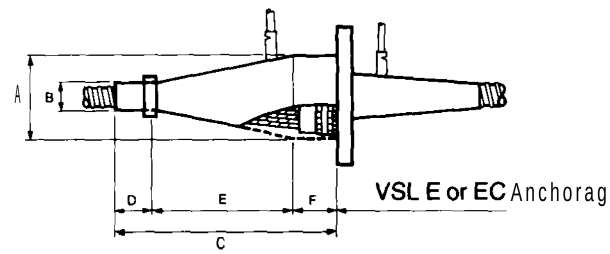
Anchorage Type	VSL 5-4	5-7	5-12	5-19	5-31	6-12	6-19
A	6.00	6.75	12.25	12.25	18.50	15.30	15.50
B	6.75	7.50	10.63	15.50	17.00	13.00	18.50
A*	2.75	2.75	7.50	7.50	8.63	9.00	9.00
B**	12.25	14.50	13.75	18.50	26.50	17.00	22.50
C	36.00	48.00	48.00	48.00	48.00	48.00	52.00

1. Dimensions and arrangements shown are the most common but can be altered where desired.
2. Spirals are generally used in light weight concrete (110 lbs/cu ft) and for tendons of 12 strands and larger, and are usually composed of 5 turns of ½ inch bar.

Dimensions in inches.
*Denotes square pattern.
**Denotes rectangular pattern.

VSL K Coupler

This coupler enables a new tendon to be connected to an already placed and stressed tendon. It is used mainly in bridge construction, where the work advances by successive stages. Each strand is individually coupled to the coupling head by means of a compression fitting.

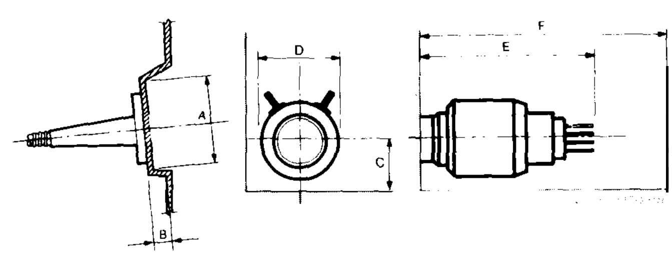


Anchorage Type	VSL 5-3	5-7	5-12	5-19	5-31	6-12	6-19
A	5.13	6.75	7.88	9.50	13.75	9.50	10.63
B	2.50	2.50	3.00	3.75	5.00	4.00	4.75
C	17.00	21.25	22.50	26.75	37.75	27.25	30.50
D	2.75	3.25	3.50	4.50	5.50	4.50	5.00
E	9.00	12.75	13.50	16.75	26.75	17.50	20.00
F	5.25	5.25	5.50	5.50	5.50	5.25	5.50

Dimensions in inches.

Recess Dimensions and Jack Space Requirements

As a rule, the stressing anchorages should not project beyond the finished surface of the concrete due to operating and aesthetic reasons. They must, however, be accessible until the prestressing operations have been completed. Accordingly, recesses are formed for them which are subsequently filled with concrete to provide the structure with a smooth external surface and to protect the anchorages and strands against corrosion.



Jack Type	A	B Minimum	B Normal	C	D	E Minimum	E Normal	F Minimum	F Normal
E5-3	14	5	5	7	13	26	51	40	75
E5-7	16	5	5	9	16	27	52	40	75
E5-12	18	5	5	10	18	27	52	40	75
E5-19 E6-12	22	6	7	10	18	30	55	45	80
E5-31 E6-19	27	6	7	11	20	32	57	45	80
E5-55	36	9	10	18	30	60	70	90	100

Dimensions in inches.

The minimum values for E and F should only be used in special cases as they require short-stroke jacks which may not always be available.

Selecting the Prestressing Unit

The following points have to be considered in selecting the prestressing unit:

Working force
Loss of prestress
Ultimate strength of prestressing steel
Allowable stresses in prestressing steel
Anchorage type
Cable sheath diameter
Minimum spacing of sheaths
Minimum concrete cover

The 270K 7-wire strand for prestressed concrete with an ultimate stress (f'_s) of 270,000 psi is produced and tested in accordance with the requirements of ASTM A 416. Physical properties of the 0.5" dia. 270K uncoated and stress-relieved strand are as follows:

Ultimate strength	41,300 lb.
Yield strength (at 1% extension)	35,100 lb.
Approx. modulus of elasticity	28,000,000 psi'
Min. elongation at rupture	3.5% in 24 inches

*This figure may vary slightly for different manufacturers

0.5" dia. 270 ksi strands

Unit VSL	No. of strands	Steel area sq. in.	Weight lb./ft.	Sheath diameter (inches) Flexible metal I.D.	Rigid thin wall O.D.	Max. temp. force 0.8 f's kips	Initial force 0.7 f's kips
E5-3	2	0.306	1.050	1.25	2.12"	66.1	57.8
	3	0.459	1.575	1.50	2.12"	99.1	86.7
E5-4	4	0.612	2.100	1.63	2.12"	132.2	115.6
E5-7	5	0.765	2.625	1.75	2.12	165.2	144.5
	6	0.918	3.150	1.88	2.12	198.2	173.5
	7	1.071	3.675	2.00	2.12	231.3	202.4
E5-12	8	1.224	4.200	2.00	2.25	264.3	231.3
	9	1.377	4.725	2.13	2.44	297.4	260.2
	10	1.530	5.250	2.25	2.44	330.4	289.1
	11	1.683	5.775	2.38	2.81	363.4	316.0
	12	1.836	6.300	2.50	2.81	396.5	346.9
E5-19	13	1.989	6.825	2.63	3.00	429.5	375.8
	14	2.142	7.350	2.63	3.00	462.6	404.7
	15	2.295	7.875	2.75	3.19	495.6	433.6
	16	2.448	8.400	2.88	3.19	528.6	462.6
	17	2.601	8.925	3.00	3.56	561.7	491.5
	18	2.754	9.450	3.00	3.56	594.7	520.4
	19	2.907	9.975	3.13	3.56	627.8	549.3
E5-22	20	3.060	10.500	3.25	3.75	660.8	578.2
	21	3.213	11.025	3.25	3.75	693.8	607.1
	22	3.366	11.550	3.38	3.75	726.9	636.0
E5-31	23	3.519	12.075	3.50	3.94	759.9	664.9
	24	3.672	12.600	3.50	3.94	793.0	693.8
	25	3.825	13.125	3.63	3.94	826.0	722.7
	26	3.978	13.650	3.63	3.94	859.0	751.7
	27	4.131	14.175	3.75	4.31	892.1	780.6
	28	4.284	14.700	3.88	4.31	925.1	809.5
	29	4.437	15.225	3.88	4.31	958.2	838.4
	30	4.590	15.750	4.00	4.50	991.2	867.3
	31	4.743	16.275	4.00	4.50	1024.2	896.2
E5-55	55	8.415	28.875	5.50	6.00	1817.6	1590.4

Dimensions in inches.
"Smaller diameter sheath available upon request.

Selecting the Prestressing Unit

Physical properties of the 0.6" dia. 270K uncoated and stress-relieved strand are as follows:

Ultimate strength	58,600 lb.
Yield strength (at 1% extension)	41,020 lb.
Approx. modulus of elasticity	28,000,000 psi*
Min. elongation at rupture	3.5% in 24 inches

*This figure may vary slightly for different manufacturers.

0.6" dia. 270 ksi strands

Unit VSL	No. of strands	Steel area sq. in.	Weight lb./ft.	Sheath diameter (inches) Flexible metal I.D.	Sheath diameter (inches) Rigid thin wall O.D.	Max. temp. force 0.8 f's kips	Initial force 0.7 f's kips
E6-12	8	1.736	5.902	2.81	3.00	375.0	328.2
	9	1.953	6.640	2.81	3.00	421.9	369.2
	10	2.170	7.378	3.00	3.56	468.8	410.2
	11	2.387	8.116	3.00	3.56	515.6	451.2
	12	2.604	8.854	3.00	3.56	562.6	492.2
E6-19	13	2.821	9.591	3.25	3.75	609.4	533.3
	14	3.038	10.330	3.25	3.75	656.3	574.3
	15	3.255	11.067	3.25	3.75	703.2	615.3
	16	3.472	11.805	3.50	3.94	750.1	656.3
	17	3.689	12.543	3.50	3.94	797.0	697.3
	18	3.906	13.280	3.63	3.94	843.8	738.4
	19	4.123	14.018	3.75	4.31	890.7	779.4

Dimensions in inches.
* 'Smaller diameter sheath available upon request.

Grouting

The purpose of grouting a cable is to provide an efficient bond between the prestressing steel and the concrete member, as well as to protect the steel from corrosion. To accomplish this, the grout should possess the following properties:

- Good fluidity
- No thickening during injection
- Minimum bleeding and segregation
- No reduction of volume; i.e., no shrinkage
- Frost resistance
- Adequate strength

For detailed grouting recommendations, refer to PTI Manual, "Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete."

To a large extent, the successful grouting of prestressing tendons depends on the equipment. In order to achieve a proper grout, the use of the best mixing and pumping equipment is essential. VSL Grouting Equipment possesses the necessary features to provide a proper grout.

VSL Monostrand System

The VSL Monostrand System has been specifically designed to reduce the field labor requirements of installation and stressing. The system can be used economically for slabs, beams, transverse girders, foundations and a variety of other applications.

This system utilizes the same proven principle of anchoring employed by the VSL Multistrand System. This system uses either 0.5 or 0.6 inch, 7-wire strand with guaranteed ultimate strengths of 41,300 and 58,600 lbs. respectively. Other sizes and types of strand are available upon request.

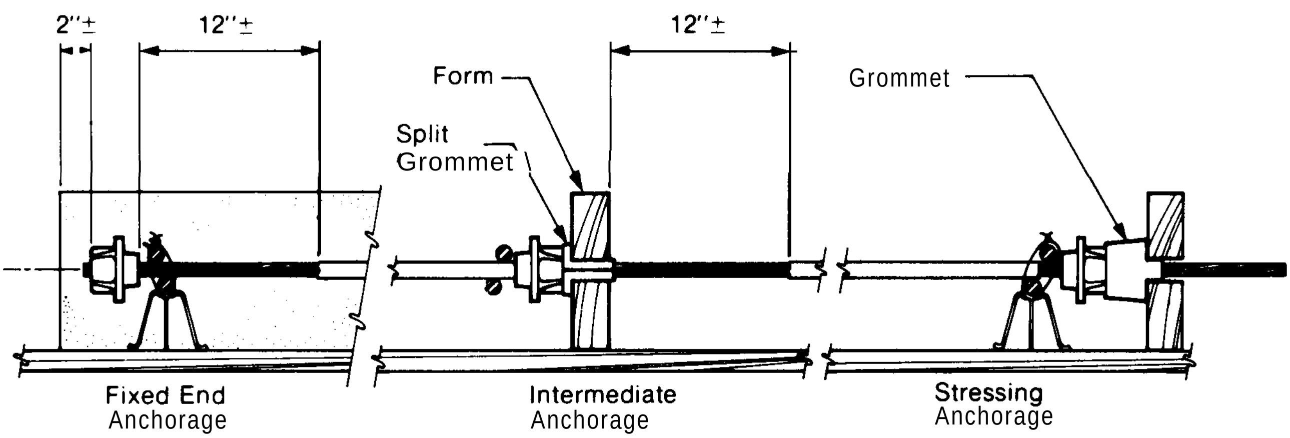
The variety of systems offers the designer a selection of available post-tensioning forces, and VSL's Engineering Staff is available to provide assistance in selecting the right system for the specific project. For the post-tensioning of beams, either the VSL Multistrand System or a corresponding number of VSL Monostrand System tendons may be used. Separate monostrand tendons are placed in flat patterns at the high and low points of the tendon profile, providing maximum eccentricity and therefore the most effective use of the post-tensioning steel.

The following anchorages are used with the VSL Monostrand System:

- Stressing, intermediate and fixed anchorages VSL T and SN
- Early stressing anchorage VSL SNW

The VSL Monostrand System has been exposed extensively to both static and dynamic tests, satisfying all nationally recognized building codes and specifications for post-tensioning material.

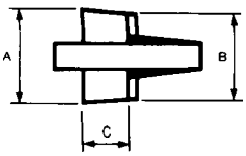
Typical Slab Section



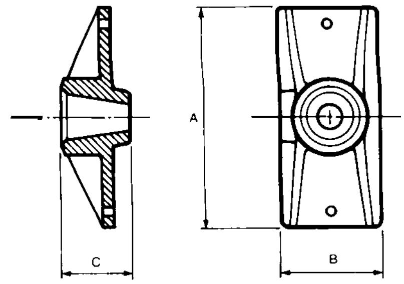
Anchorage Components

VSL SN Stressing Anchorage

The VSL SN Anchorage consists of a casting and a pair of VSL Wedges. Since the stressing anchorage is attached to the form prior to the installation of strand, greater flexibility is achieved as the tendons can be easily threaded through congested, heavily reinforced beams or under electrical ducts in the flat slabs. This anchorage is used for both intermediate and fixed-end applications. The SNW is used in situations where its larger size is required for early stressing or lower than normal concrete strengths.



Grommet-G



Anchorage Casting

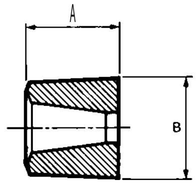
Component	Length or Diameter	Width or Diameter	Depth or Thickness	Bearing Area (in*)	Conc. Strength at Stressing' (lb/in²)
	A	B	C	—	—
S5N	5.00	2.25	1.50	11.25	2050
S5NW1	5.25	2.88	1.50	15.09	1500
S5NW2	4.00	3.50	1.50	14.00	1700
S6N	4.63	3.50	1.63	16.19	2100
S6NW	6.00	3.50	1.63	21.00	1600
G 5	2.25	2.00	1.25/2.25	—	—
G 6	2.50	2.13	1.25/2.25	—	—

Dimensions in inches.

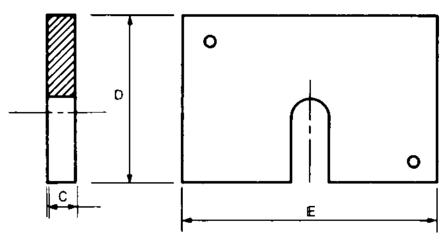
'Values are based on ACI formula $f_b = 0.8 f'_{ci} \sqrt{A_2/A_1 - 0.2} \leq 1.25 f'_{ci}$ with edge distance of 1" for hardrock concrete.

VSL T Stressing Anchorage

The T Anchorage consists of a bearing plate, cast anchor head, and a pair of VSL Wedges. The anchor head can be preplaced on the strand in the plant when used as an intermediate anchorage. When this is the case, a slotted bearing plate is used. The T is also used as a stressing anchorage at the end of a beam or slab when a closure pour is provided, and it is desirable to modify anchorage spacing with a specially designed bearing plate to clear reinforcing or other obstructions.



Anchor Head



Slotted Bearing Plate

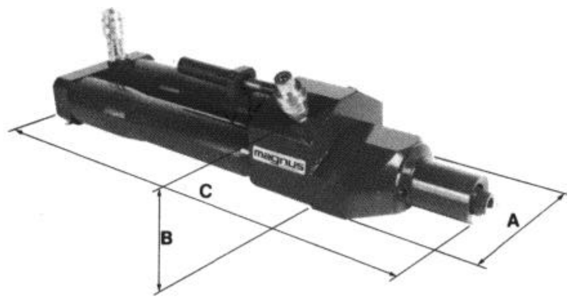
Anchorage	A	B	C	D	E
T5	1.75	1.75	0.50	2.50	4.50
T6	2.12	2.06	0.63	3.50	4.50

Dimensions in inches.

Stressing Equipment

VSL Stressing Rams

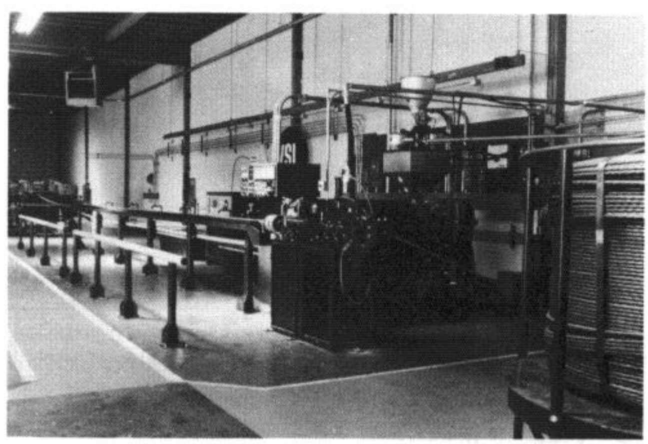
The VSL Twin Rams are small, lightweight, and easily handled by one person. They are designed for use on all anchorages, thereby eliminating the need for two sets of equipment on a job where intermediate or customized beam anchorages are employed. Jacks can be used to stress tendons located as close as 1.5 inches to the side of a form or stressing blockout. With the use of easily interchangeable nose pieces, the rams can reach between column bars spaced as close as 2 inches to stress recessed tendons. The VSL Twin Ram stressing operation requires less than 1 minute to set up and stress a tendon.



Ram Type	A	B	C	Stroke	Approx. Weight
ML53	6.25	4.13	17.00	3.00	30
ML56	6.25	4.13	20.00	6.00	36
ML58	6.25	4.13	22.00	8.00	40
MA54	7.75	4.00	22.50	4.00	50
MA56	7.75	4.00	24.50	6.00	56
MA58	7.75	4.00	26.50	8.00	62
ML63	9.75	3.50	16.00	3.00	35
ML66	9.75	3.50	19.00	6.00	42
ML68	9.75	3.50	21.00	8.50	49

Prestressing Steel

Prestressing steel employed by the VSL Monostrand System is either 0.5 or 0.6 inch 7-wire strand manufactured in accordance with ASTM A416 and has a guaranteed ultimate tensile strength of 270 ksi. Other sizes and types of strand are available upon request. The prestressing steel is coated with a lubricating corrosion inhibitor prior to being extruded in a tough protective plastic sheath.

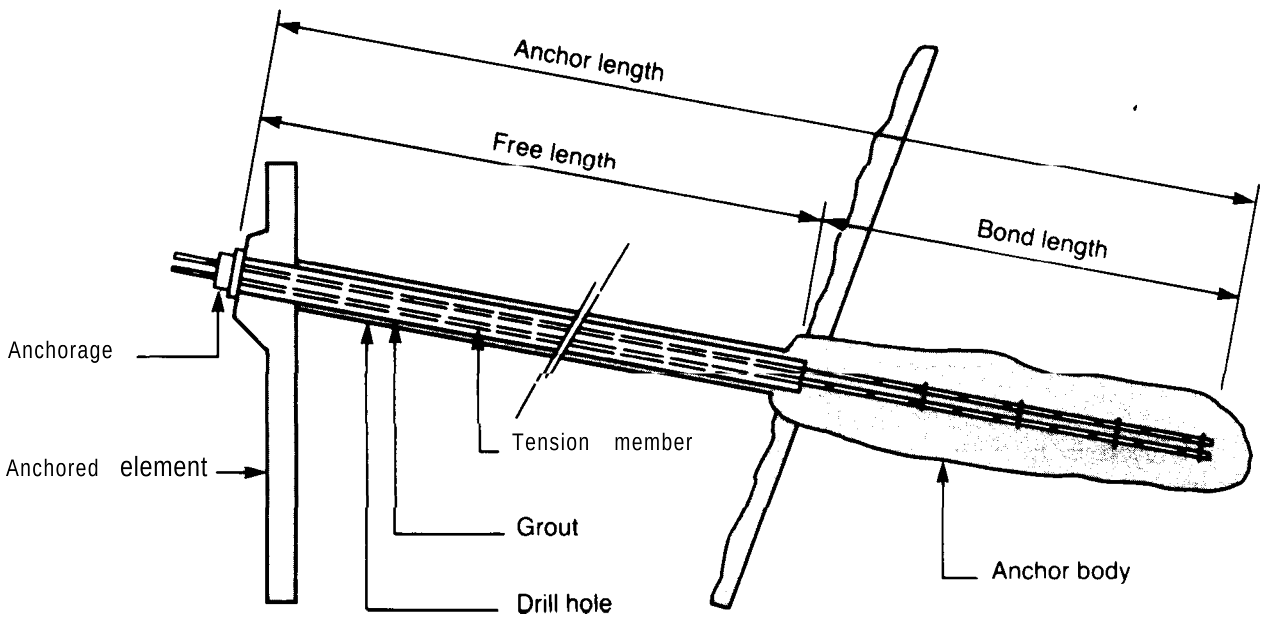


Technical Data

Nominal diameter:	0.5 in.	0.6 in.
Cross sectional area:	0.153 sq. in.	0.217 sq. in.
Modulus of elasticity:	28,000 ksi	28,000 ksi
Ultimate strength:	41.3 kips	58.6 kips
Max. temporary force at jacking:	33.0 kips	46.9 kips
Max. anchoring force:	28.9 kips	41.0 kips

VSL Prestressed Rock and Soil Anchors

Rock and Soil Anchors represent a special application of the technique of prestressing in the field of rock and soil mechanics. The first applications of prestressed anchors began in the thirties and became firmly established in the sixties. VSL Rock and Soil Anchors were used for the first time in 1957. Since then, the VSL organization has acquired worldwide practical experience in all fields of anchor technology and has kept abreast of, and contributed to, the significant progress attained through the use of this technology. The most common applications today are found in the stabilization of slopes, the anchoring of retaining walls and excavation faces, and the anchoring of structures subjected to high uplift, overturning or concentrated forces. Other applications of importance are those which have led to radical changes in methods of construction. The raising and strengthening of dams would have been almost inconceivable, or at least would have required enormous quantities of materials, if prestressed anchors had not been available. The use of rock anchors has fostered the construction of underground caverns with totally new methods of rock removal which no longer require timber or steel roof supports. Today, VSL Anchors are available in a broad range of sizes and types suitable for all service conditions.



Anchor Data

The following points are suggested for consideration when selecting a prestressing unit:

- Working force
- Loss of prestress
- Allowable stresses in prestressing steel
- Drill hole diameter
- Bond length

The 0.5" **7-wire** strand for prestressing application has an ultimate strength (f_s'') of 270,000 psi and is produced and tested in accordance with the requirements of ASTM A-416. Physical properties of 0.5" strand are as follows:

Guaranteed ultimate strength	41,300 lb.
Yield strength(at 1% extension)	35,100 lb.
Approx. modulus of elasticity	**28,000,000** psi
Min. elongation at rupture	3.5% in 24inches

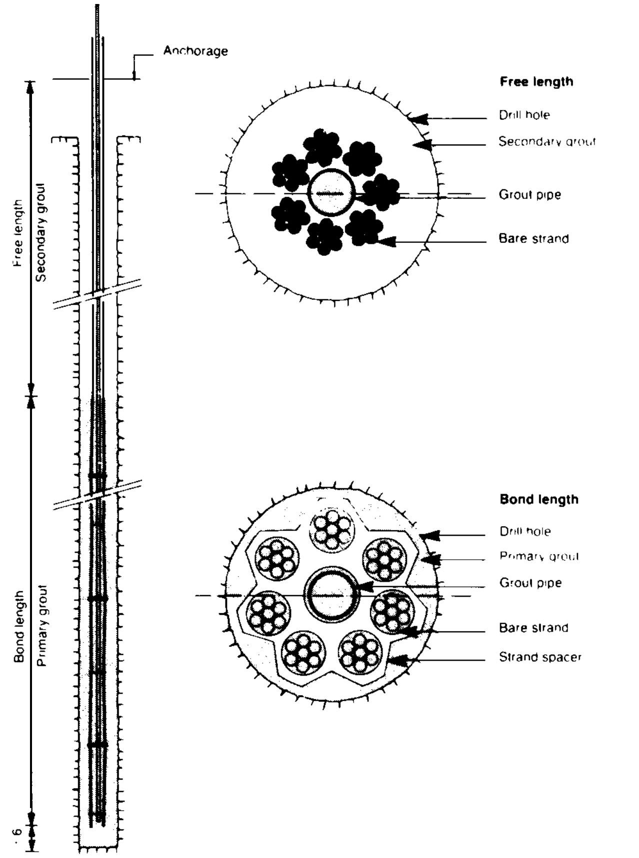
Single Corrosion Protected Rock Anchor

This anchor is used as either a temporary or permanent rock anchor in a non-aggressive environment. The maximum ultimate capacity of standard units is 2,147 kips.

In sound rock, drilling is usually accomplished by percussion methods with an air powered down-the-hole hammer. In less competent rock, rotary drills similar to those used in water well drilling are employed. Core drilling is sometimes used through concrete where large quantities of reinforcing steel are present.

½" dia. 270 ksi strands

Max. No. of Strands	4	7	9	11	13	17	21	27	31	52
Drill Hole Diameter, in.	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**	**5.0**	**5.5**	**6.0**	**6.5**	**9.0**

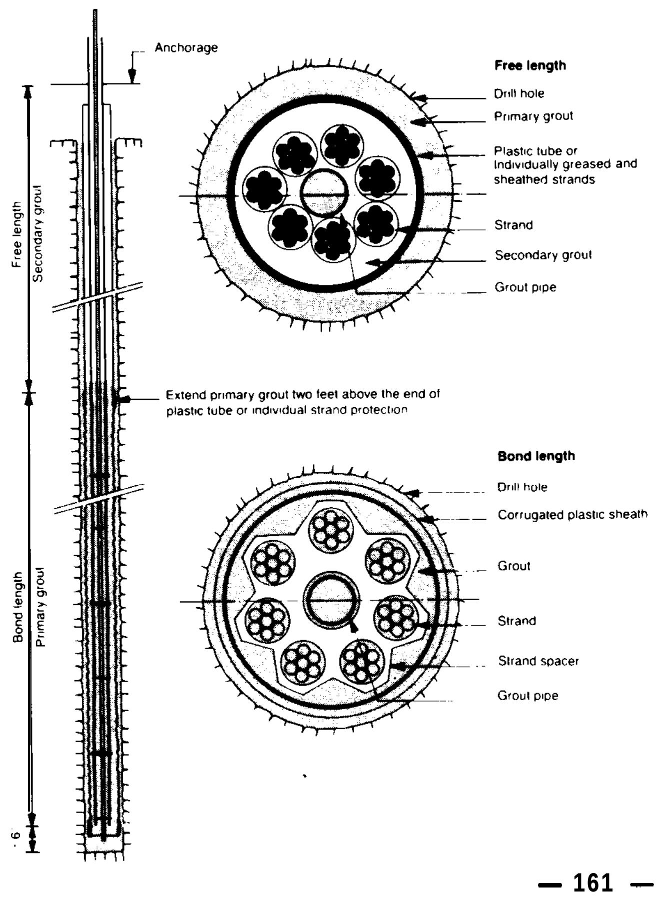


Double Corrdsion Protected Rock Anchor

This anchor is used where the environment is aggressive and protection against corrosion is necessary. The maximum ultimate capacity of the drilling corresponding to those for a single corrosion protected rock anchor are employed. In the case of either the single or the double corrosion protected anchor, it is very important that the anchor is installed in a hole that is watertight.

½" dia. 270 ksi strands

Max. No. of Strands	4	7	9	11	13	17	21	27	31	52
Drill Hole Diameter, in.	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	11.0



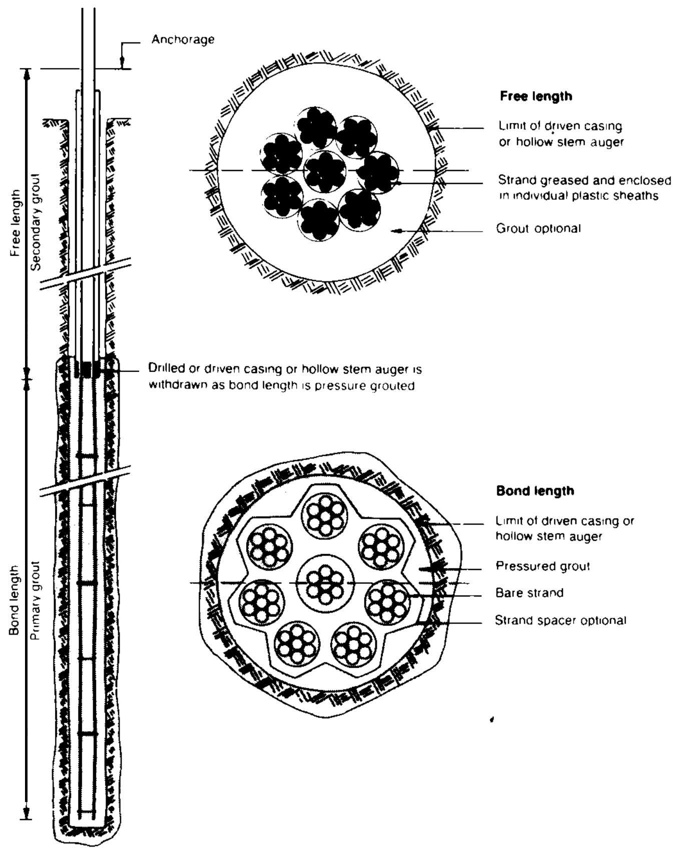
Pressure Bulb Soil Anchor

The bulb soil anchor is normally employed as a temporary anchor where loose sand or gravel make it difficult to maintain an open hole after drilling. The normal maximum ultimate capacity of the anchor is 290 kips. While a bulb anchor is usually considered temporary, it is possible to use it as a permanent anchor by taking protective measures as described for the double corrosion protected rock anchor.

Two methods are used to install a pressure bulb anchor. In the first, a casing is driven to the required depth by an air track. The tension member is placed in the casting and, as the casting is retracted, the anchor is pressure grouted. In the second method, the tension member is placed in the stem of a hollow-stem auger. The auger is withdrawn as the anchor is pressure grouted.

½" dia 270 ksi strands

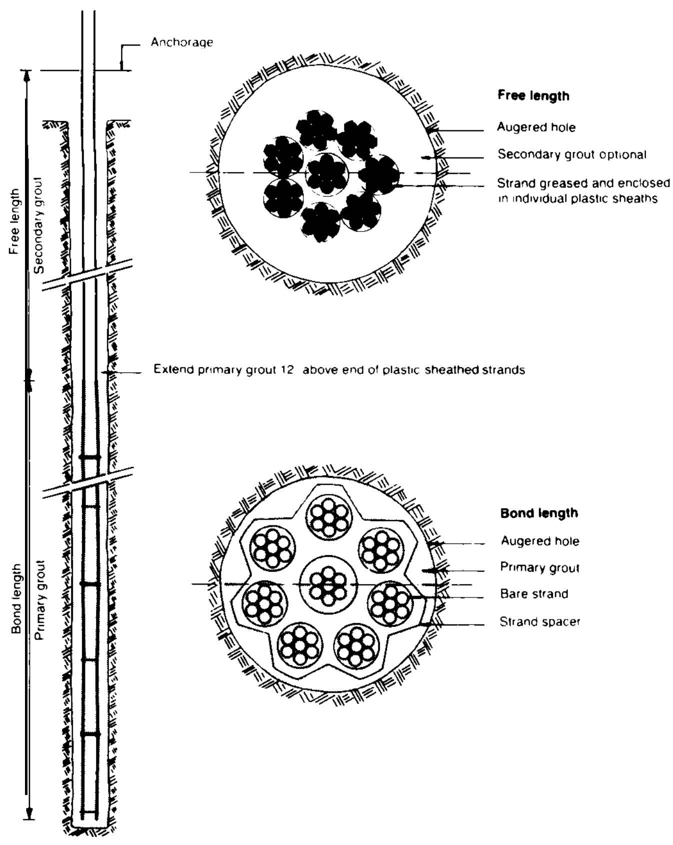
Maximum no. of strands	4	7
Casing or auger center hole diameter, in..	2.5	3.0



Augered Soil Anchor

This is the most widely used soil anchor. It is employed under all conditions where cohesive soil properties allow the hole to remain open after drilling. Normally the anchor is temporary, but if a procedure similar to that utilized for the double corrosion protection of rock anchors is applied, it can be used as a permanent anchor. The normal ultimate capacity of the larger anchor of this type is 413 kips.

Drilling equipment normally used for this anchor is the continuous flight auger, although a Kelly Bar auger can also be used successfully. Diameters vary from 6 to 24 inches.



Anchorage Components

VSL E Stressing Anchorage

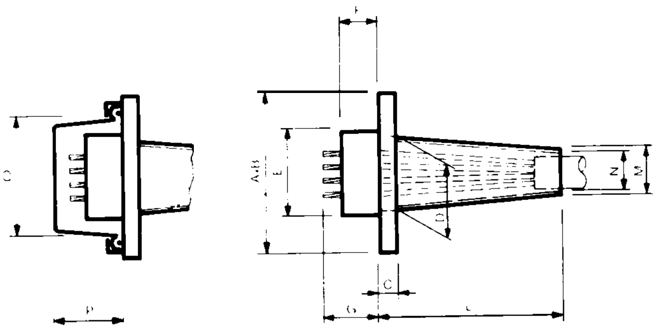
The E Anchorage consists of an anchor head, wedges and bearing plate. If the anchorage must be accessible for surveillance purposes, it may be covered with a removable protective cap.

All strands of the anchor are stressed simultaneously but are locked off individually by wedges in the conical holes of the anchor head. Throughout the range of VSL Anchorages, which incorporate from 1 to 55 strands, the principle of anchoring the strand is the same from the smallest to the largest unit.

The VSL Stressing Anchorages are designed to meet all the special requirements which may be required of an anchor. These requirements must be known in advance to enable the anchor to be properly selected. This is particularly true when the anchor:

- Is a surveillance anchor, or
- Must be restressed at a later time or
- Must be detensioned and restressed.

Corrosion protection of the anchor head is obtained by filling the protective cap with cement mortar or with a corrosion resisting grease.

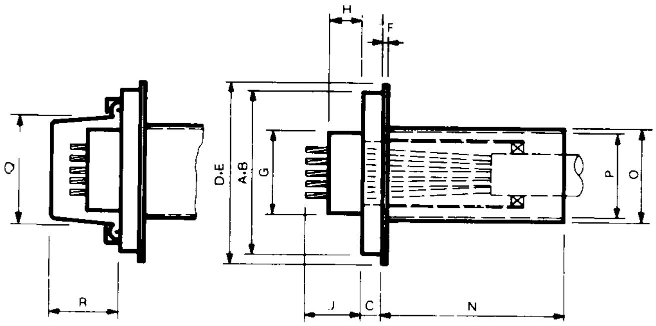


Anchorage Type	VSL ER5-3	ER5-6	ER5-9	ER5-16	ER5-28	ER5-52
A	5.38	7.50	9.38	12.50	16.38	22.25
B	5.38	7.50	9.38	12.50	16.38	22.25
C	0.75	1.25	1.25	1.75	2.00	3.00
D	2.25	2.88	4.13	5.25	6.75	9.00
E	3.38	4.50	6.00	7.00	9.00	12.50
F	2.40	2.40	2.40	2.95	3.95	6.00
G	3.40	3.40	3.40	4.00	5.00	7.00
L	4.00	8.00	12.00	23.00	34.00	42.00
M	2.00	2.50	2.75	3.50	4.50	6.00
N	1.50	2.00	2.25	3.00	4.00	5.50
O	4.38	5.50	7.00	8.00	10.00	14.00
P	5.00	5.00	5.00	6.00	7.00	9.00

Note: The last digit in the anchorage designation indicates the maximum number of 0.5 in. diameter strands that can be accommodated in the anchorage.

Template Anchorage

The VSL Template Anchorage is used where it is desirable to place a concrete element against the rock or soil prior to the drilling operation. The basic anchorage is that of a Type E with the template plate as the only variation. The bearing plate and anchor head are installed on the anchor after its insertion and primary grouting. The corrosion protection of the anchor head and the free lengths can be obtained either by cement grouting or injection with corrosion resisting grease.



Anchorage Type	VSL ER5-3	ER5-6	ER5-9	ER5-16	ER5-28	ER5-52
A	5.38	7.50	9.38	12.50	16.38	22.25
B	5.38	7.50	9.38	12.50	16.38	22.25
C	1.00	1.25	1.50	2.00	2.50	3.50
D	6.38	8.50	10.38	13.50	17.38	23.25
E	6.38	8.50	10.38	13.50	17.38	23.25
F	0.38	0.38	0.38	0.38	0.38	0.38
G	3.38	4.50	6.00	7.00	9.00	12.50
H	2.40	2.40	2.40	2.95	3.95	6.00
J	3.40	3.40	3.40	4.00	5.00	7.00.
N	15.00	15.00	15.00	24.00	36.00	44.00
O	3.38	3.88	4.38	6.38	8.00	10.50
P	3.00	3.50	4.00	6.00	7.50	10.00
Q	4.38	5.50	7.00	8.00	10.00	14.00
R	5.00	5.00	5.00	6.00	7.00	9.00

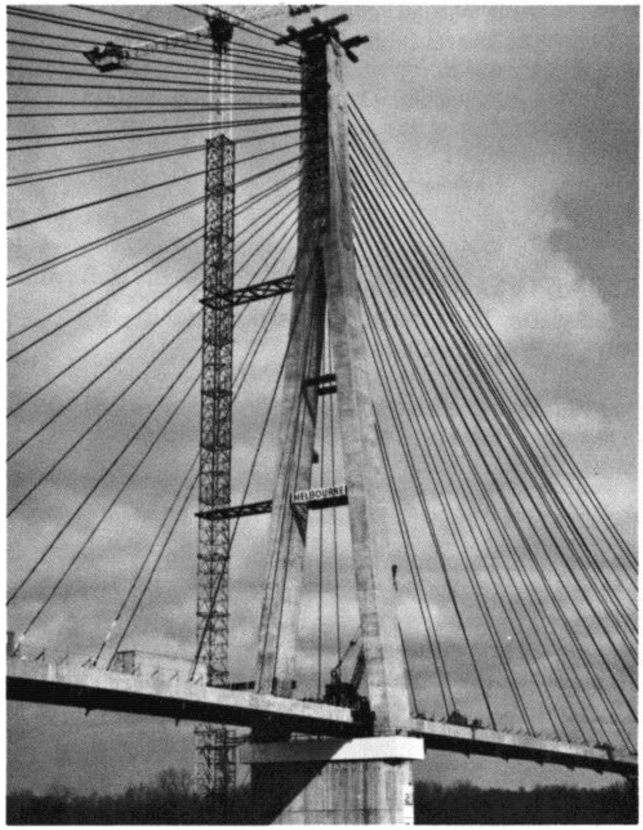
Note: The last digit in the anchorage designation indicates the maximum number of 0.5 in. diameter strands that can be accommodated in the anchorage.

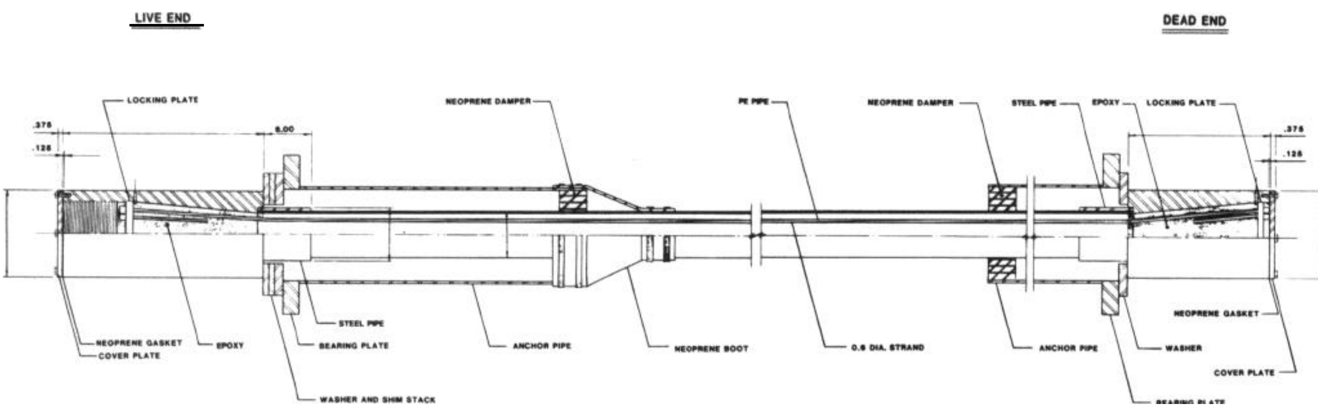
VSL Stay Cable Systems

VSL Stay Cable Systems represent another special adaptation of the well established VSL Multistrand Post-Tensioning System. The systems consist of standard post-tensioning strands used as the tensile members of the stay cables, and specially designed anchorages for stressing and securing the cables.

The VSL strand members have a high breaking strength, which results in reduced steel consumption and lighter cable stays. The strands' high modulus of elasticity provides relatively small elongations.

Another advantage of VSL Stay Cable Systems is their modularity, which enables the required cables to be fabricated from standard units. This allows construction procedures to be standardized and thus simplified.





Typical composition of VSL Stay Cable.

System Components

VSL Strand Bundle

The VSL Strand Bundle consists of cold drawn, **7-wire,** ASTM A-416 post-tensioning steel of 0.6 inch nominal diameter and low relaxation quality. For additional corrosion protection, the strand may be galvanized or greased and individually sheathed in high-density polyethylene.

Tubing

The VSL Stay Cable System 200 Strand Bundle is encased in a tube of either steel or heavy gauge polyethylene. The ratio of internal diameter to wall thickness is approximately **16:1.**

The strand bundle for the System 250 is encased in a polyethelene tube and coiled on a reel.

Anchorages

The engineering principles of the System 200 Anchorages are identical to those of the VSL Multistrand Post-Tensioning System Anchorages. Both the stressing anchorages and dead-end anchorages consist of a bearing plate, an anchor head, wedges, a transition pipe, a tension ring, a connection pipe and an end cap with grouting vents.

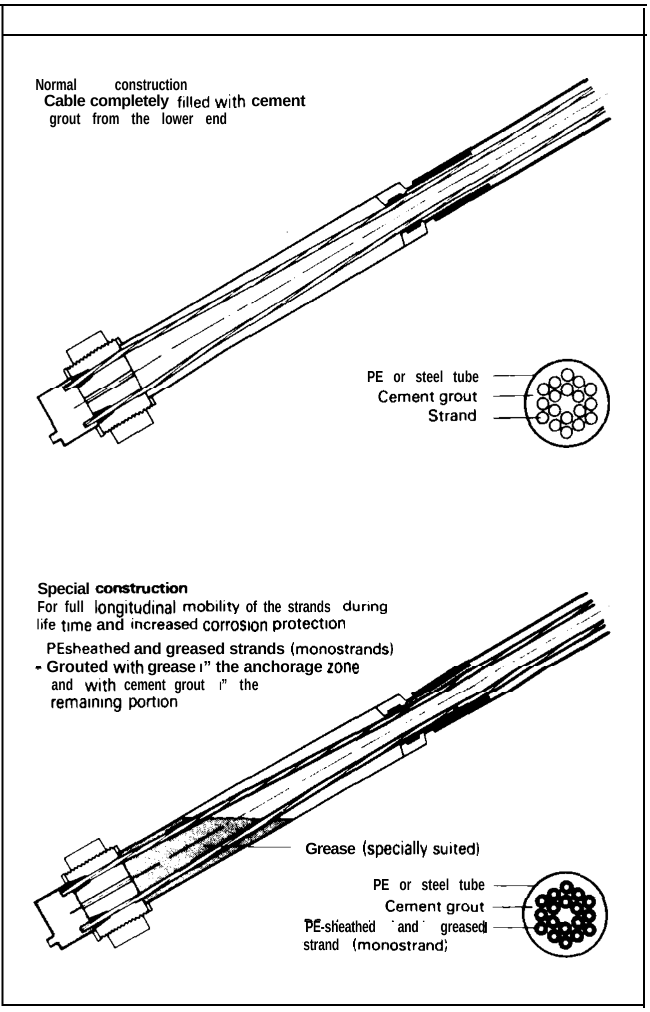
The System 200 stressing anchorage is provided with a threaded head and ring nut which allows tension adjustment or total destressing if required. The transition ring is provided with a neoprene antivibration device which absorbs lateral vibrations caused by wind.

For the VSL Stay Cable System 250, the anchorage is formed by conical steel forgings which create a conical wedge anchor.

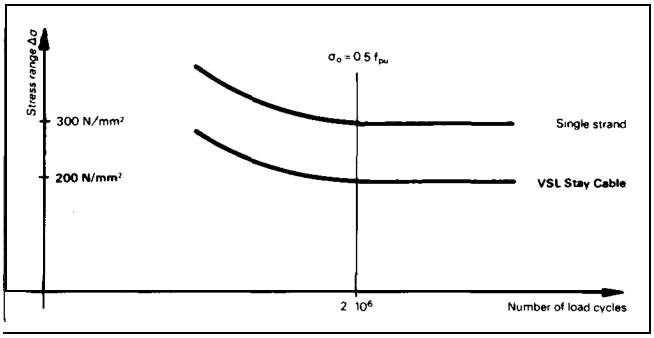
Grouting and Corrosion Protection

The steel or polyethylene tube encasing the strand bundle constitutes the primary corrosion protection of the cable stay. Where full longitudinal mobility of the strands is required, polyethylene sheathed and greased monostrand cables are used.

The grout is normally Portland Cement Concrete and meets the same requirements as that used for injecting multistrand post-tensioning tendons. The grout completely fills the space between the strand bundle and the pipe, and the alkaline properties of the grout provide active corrosion protection.



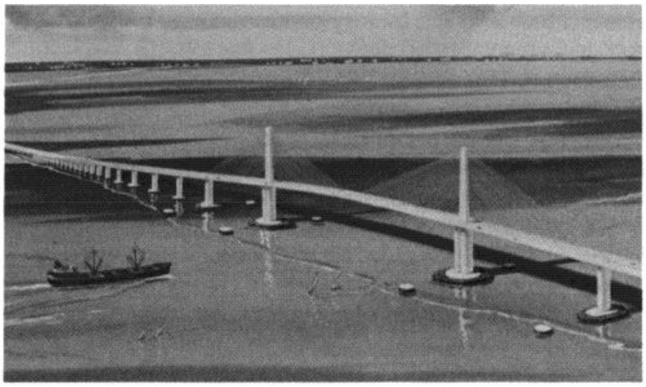
Types of grouting in the anchorage zone



Wohler curves

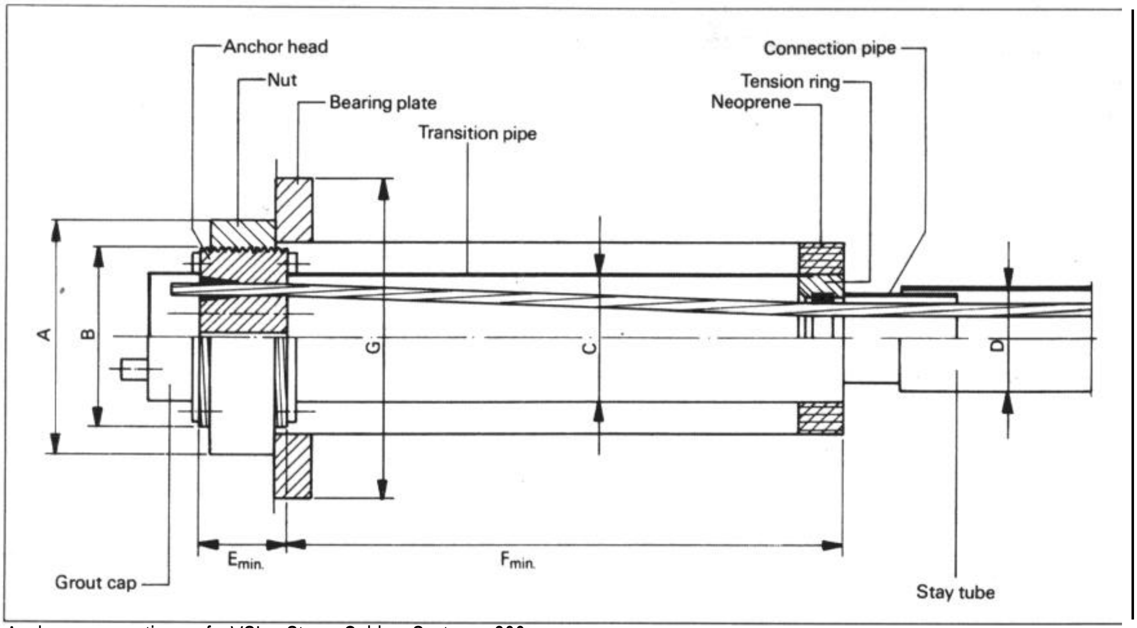
VSL STAY CABLE SYSTEM 200

VSL Stay Cable Systems represent another special application of the VSL Multistrand Post-Tensioning System. The System 200 can be either factory fabricated or field fabricated, and is available in modular standard units.



The system takes advantage of the high breaking strength of ASTM A-416 low relaxation steel strand. The stay cables are sheathed in either heavy gauge polyethylene pipe or steel pipe, and are protected from corrosion by cement grout injection performed after completion of the bridge. Other corrosion protection systems are also available. One such system consists of individual strands which are greased and covered with extruded polyethylene, then bundled and sheathed in polyethylene pipe. Portland Cement grout is then injected into the pipe upon completion of the bridge.

VSL also offers a broad range of anchorage arrangements to provide for cable replacement.



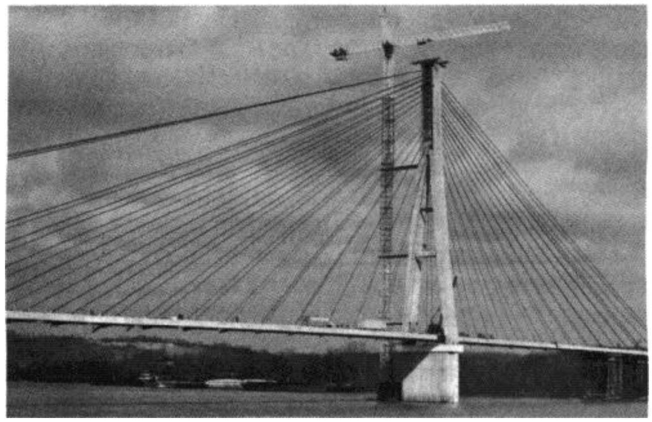
Anchorage section of VSL Stay Cable System 200

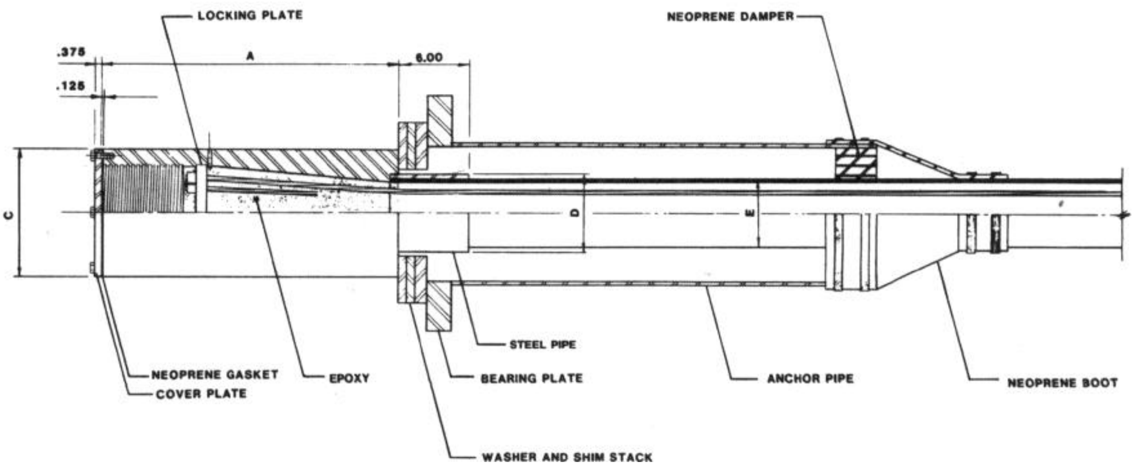
Unit	A	B	C	D	E	F	G	Breaking Load Kips
6-7	6.25	6.30	4.92	3.54	3.54	15.74	9.45	410
6-12	9.45	7.46	6.30	4.33	3.94	27.56	12.60	703
6-19	11.41	9.06	7.67	4.92	4.33	43.31	15.75	1,113
6-31	13'39	11.02	9.84	6.30	5.51	53.15	16.90	1.617
6-37	14.57	11.61	11.02	7.09	6.30	62.99	20.67	2,166
6-61	16.11	14.96	13.96	7.67	7.09	90.55	25.98	3,574
6-91	21.65	17.72	17.72	9.84	6.66	110.23	31.69	47,466

Dimensions in inches.

VSL STAY CABLE SYSTEM 250

The System 250 is a factory produced stay cable which is coiled on a reel and delivered to the jobsite ready for installation. The anchor sockets are filled with a mixture of epoxy, steal balls, and zinc powder. The resulting anchorage has superior fatigue properties as well as a compact anchorage socket. The system also takes advantage of the high breaking strength of ASTM A-416 low relaxation strand and is sheathed in heavy gauge polyethylene pipe. Final corrosion protection is provided by cement grout injection. Cable replacement capabilities are built into the system.





Anchorage Section of VSL Stay Cable System 250

Unit	A	B	C	D	E	Breaking Load Kips
8-19	17.00	12.63	7.88	4.50	3.25	1,113
6-31	20.62	15.66	9.68	5.56	4.06	1,817
6-43	24.25	17.50	10.88	6.63	5.25	2,520
6-55	27.00	19.88	12.38	6.63	5.25	3,223
6-73	32.63	24.00	15.00	7.75	6.12	4,278
6-91	37.50	27.00	16.88	8.63	6.88	5,333
6-103	38.38	28.12	17.56	8.63	7.12	6,036

Dimensions in inches.

2.2.18 ADDRESSES OF POST-TENSIONING MEMBERS OF THE POST-TENSIONING INSTITUTE

AMERICAN CABLE COMPANY, INC.
Route 7, Box 7446
Slidell, LA 70461
(504) 641-6196 '
(504) 529-5891 (New Orleans)

AMSYSCO, INC.
740 Racquet Club Drive
Addison, Illinois 60101
(312) 628-6969

CCL DIVISION
NICHOLSON CONSTRUCTION COMPANY
P.O. Box 98
Bridgeville, PA 15017
(412) 221-4500
Telex 812324 TAS PGH

CEC SYSTEMS, INC.
360 Ferry Street
Martinez, CA 94553
(415) 229-2700

CONTINENTAL CONCRETE STRUCTURES, INC.
P.O. Box 734
1400 Union Hill Road, S.W.
Alpharetta, GA 30201
(404) 475-l 700

CONTINENTAL STRUCTURES, INC.
3200 Fujita Street
Torrance, CA 90505
(213) 530-4951

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
Corporate Office
107 Beaver Brook Road
Lincoln Park, NJ 07035-0488
(201) 628-8700

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
Eastern Division
107 Beaver Brook Road
Lincoln Park, NJ 07035-0488
(201) 628-8700

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
1740 East Joppa Road
Baltimore, MD 21234-3682
(301) 882-6111

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
4 Castlewood Drive
Greenville, SC 29615
(803) 268-4452

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
1240 Johnson Ferry Place
Suite C-l 0
Marietta, GA 30067
(404) 977-5417

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
Central Division
301 Marmon Drive
Lemont, IL 60439-9006
(312) 739-l 100

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
1384 Grandview Avenue
Columbus, OH 43212-2805
(614) 486-8793

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
1113 Pueblo Drive
Richardson, TX 75080-2914
(214) 690-6411

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
Western Division
11526 Sorrento Valley Road
San Diego, CA 92121-1314
(619) 755-6787

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
7225 S.W. 86th Avenue
Portland, OR 97223-7298
(503) 245-8917

DYWIDAG SYSTEMS INTERNATIONAL, USA, INC.
14261 E. 4th Avenue
Suite 261
Aurora, CO 8001 l-8732
(303) 363-8578

DYWIDAG SYSTEMS INTERNATIONAL, CANADA LTD.
Eastern Division
65 Bowes Road, Unit 8
Concord, Ontario, Canada L4K 1H5
(416) 669-4952

DYWIDAG SYSTEMS INTERNATIONAL, CANADA LTD.
100 Alexis Nihon Boulevard
St. Laurent, Quebec, Canada H4M 2N7
(514) 744-6264

DYWIDAG SYSTEMS INTERNATIONAL, CANADA LTD.
Western Division
3701 19th Street, N.E.
Calgary, Alberta, Canada T2E 6S8
(403) 276-7768

DYWIDAG SYSTEMS INTERNATIONAL, CANADA LTD.
10751 Shellbridge Way
Suite 140
Richmond, B.C. V6X 2W8, Canada
(604) 270-l 855

GENSTAR STRUCTURES LIMITED
Post-Tensioning Division
P.O. Box 1650
Calgary, Alberta, Canada T2P 2L7
(403) 272-3191

Regional Office
Vancouver, British Columbia
(604) 324-2535

LANG TENDONS INCORPORATED
Newark Road
Toughkenamon, PA 19374
(215) 268-2221

LINDEN POST-TENSIONING CORPORATION
P.O. Box 1032
Tucker, GA 30085
(404) 491-3790

PATTRIDGE POST TENSION, INC.
1606 N. Market Street
Shreveport, LA 71107
(318) 227-9248

POST-TENSION OF TEXAS, INC.
NATIONAL POST-TENSIONING SERVICES, INC.
520 Thornton
Houston, Texas 77018
(713) 692-5958

Area off ices:

POST-TENSION OF TEXAS, INC.
6526 Lake June Road
Dallas, Texas 75217
(214) 398-8465

NATIONAL POST-TENSIONING SERVICES, INC.
4896 Center Park Blvd.
San Antonio, Texas 78218
(512) 657-5933

POST-TENSION OF TEXAS, INC.
520 Thornton
Houston, Texas 77018
(713) 692-5958

NATIONAL POST-TENSIONING SERVICES, INC.
6901 N. Lamar Blvd.
Austin, Texas 78752
(512) 452-7030

NATIONAL POST-TENSIONING SERVICES, INC.
6405 B. North 50th St.
Tampa, Florida 33610
(813) 623-l 081

2.2.18 Continued

THE PRESCON CORPORATION
Corporate Office & Plant
1338 North W.W. White Road
San Antonio, TX 78219
P.O. Box 20800
San Antonio, TX 78220-0800
(5 12) 662-8500
Telex: 767550

THE PRESCON CORPORATION
Northwest Office
P.O. Box 1842
Vancouver, WA 98668
2700 Northeast Andresen Road
Vancouver, WA 98661
(206) 694-7468

FREYSSINET COMPANY, INC.
P.O. Box 216
181 Main Street
Tuckahoe, NY 10707
(914) 337-0800
Telex: 131-466

FREYSSINET INTERNATIONAL STUP
66, Route de la Reine
92100 Boulogne Billancourt
France '
(1) 604 91 40
Telex: 842-260727

PTE STRAND CO., INC.
1728 N.W. 82nd Avenue
Miami, Florida 33126
(305) 593-5069

STEEL SERVICE COMPANY
1919 Tennessee Avenue
Knoxville, Tennessee 37901
(615) 546-5472

STRESSCON, INC.
2101 West Jackson Street
Phoenix, AZ 85009
P.O. Box 6387
Phoenix, AZ 85005
(602) 254-9818

STRESSCON, INC.
(Service Center)
7741 Alabama, Unit 4
Canoga Park, California 91304
(818) 704-0570

STRESSTEK DIVISION
CONMAR CORPORATION
4775 Hannover Place
Fremont, CA 94538-6311
(415) 651-3764

STRESSTEK
801 North Cass Avenue, Suite 300
Westmont, IL 60559
(3 12) 850-9333

STRESSTEK
1140 Longpoint
Dallas, TX 75247
(214) 688-5955

STRESSTEK
13206 Park Lane
Fort Washington, MD 20744
(301) 292-0392

STRESSTEK
310 First Avenue South
Suite 200
Seattle, WA 98104
(206) 343-7878

TECH-CON SYSTEMS, INC.
80 Allen Road
Slidell, LA 70461
(504) 641-l 225

VSL CORPORATION
Corporate Office
101 Albright Way
Los Gatos, CA 95030
(408) 866-5000
Telex: 821 059

Regional Offices

VSL CORPORATION
Atlanta
5555 Oakbrook Parkway
Suite 530
Norcross, GA 30093
(404) 446-3000
Telex: 700 527

VSL CORPORATION
Austin
937 Reinli, Suite 18
Austin, TX 78751
(512) 450-1065

VSL CORPORATION
Campbell
1077 Dell Avenue
Campbell, CA 95008
(408) 866-5000
Telex: 5101004324

VSL CORPORATION
Dallas
1414 Post & Paddock
Grand Prairie, TX 75050
(214) 647-0200
Telex: 730 566

VSL CORPORATION
Denver
7006 South Alton Way, Building B
Englewood, CO 80112
(303) 779-l 155

VSL CORPORATION
Honolulu
410 Mokauea Street
Honolulu, HI 96819
(808) 841-0161

VSL CORPORATION
Houston
7102 Belgold Street
Houston, TX 77066
(713) 444-9045

VSL CORPORATION
Miami
7223 N.W. 46th Street
Miami, FL 33166
(305) 592-5075
Telex: 803 574

VSL CORPORATION
Minneapolis
11925 12th Ave. South
Burnsville, MN 55337
(612) 894-6350
Telex: 290 457

VSL CORPORATION
Santa Ana
3000 W. MacArthur Blvd.
Suite 275
Santa, Ana, CA 92704
(714) 662-4088

VSL CORPORATION
Seattle
4208 198th Street, S.W.
Lynnwood, WA 98036
(206) 771-3088

VSL CORPORATION
Springfield
8006 Haute Court
Springfield, VA 22150
(703) 451-4300
Telex: 899 487

2.3 ASSOCIATE AND AFFILIATE MEMBERS OF THE POST-TENSIONING INSTITUTE

This section presents addresses of Associate and Affiliate Members of the Post Tensioning Institute along with descriptions of products and services provided by these companies. More detailed information on the products and services provided by most of these companies is presented in Sections 2.3.3 through 2.3.14.

2.3.1 Addresses of Associate Members

THE CRISPIN COMPANY
1301 Texas Avenue
Houston, TX 77002
(713) 224-8000
PRODUCTS: International Marketing, steel products. See Section 2.3.3.

FLORIDA WIRE AND CABLE COMPANY
P.O. Box 6835
Jacksonville, FL 32236
(904) 781-9224
See Section 2.3.4

SHINKO WIRE COMPANY, LTD.
Head Office and Plant
2, Doi-cho 7-chome
Amagasaki, 660, Japan
(06) 41 l-1051
PRODUCTS: Low relaxation grade and normal relaxation grade pre-stressing steel wire and strand. Extra high strength prestressing wire and strand. Galvanized wire and strand. Compact strand (Seven wire drawn strand). Indented wire and strand. See Section 2.3.9

SHINKO WIRE COMPANY, LTD.
Sales Office
Ishizuka-Yaesu Building
l-5-20 Yaesu, Chuoku
Tokyo, 103 Japan
(03) 272-4671

SHINKO WIRE AMERICA, INC.
P.O. Box 218808
Houston, TX 77218
(713) 937-7178

SIDERIUS, INC.
5909 West Loop South
Bellaire, TX 77401
(713) 667-2222
Telex # 791177
PRODUCTS: Uncoated seven-wire stress-relieved and stabilized (low-relaxation) strand to ASTM-A-416-74. See Section 2.3.10.

SIDERIUS, INC.
1185 Avenue of the Americas
New York, NY 10036
(212) 489-7470
Telex # 7162307

DERIVER
80058 Torre Annunziata (Naples)
Via Terrangneta No. 72, ITALY
(081) 8612522
Telex # (843) 710142

SUMITOMO/SWPC/SW Sales
P.O. Box 2555
Dublin, CA 94568
(415) 828-3862
PRODUCTS: Stress-relieved and low-relaxation strand. See Section 2.3.11

SUMIDEN WIRE SALES (SW Sales)
P.O. Box 3048
Costa Mesa, CA 92628
(714) 759-8216

SUMIDEN WIRE PRODUCTS CORP. (SWPC)
1412 El Pinal Drive
Stockton, CA 94205
P.O. Box 8719
Stockton, CA 95208
(209) 466-8924 .

2.3.2 Addresses of Affiliate Members

THE GREAT SOUTHWEST
MARKETING CO., INC.
2645 Perth
Dallas, TX 75220
(214) 351-4625
PRODUCTS: Equipment - fabrication lines, extrusion lines, wrap lines, dead end seaters, rams 5 styles, pumps, etc. Components — U5 2 pc. wedges, 3 pc. wedges, U5 anchors, U5 pocketformers, U5 donuts, U5 splicechucks, chairs, corrosion inhibitors, uncoated, 7 wire stress relieved and low relaxation strand (ASTM A416-74) (ICBO 4164). See Section 2.3.5.

GRIP-TECH INTERNATIONAL
249 W. 131st Street
Los Angeles, CA 90061
(213) 538-l 792
PRODUCTS: Strand wedges; gray ductile, malleable iron castings.

MEADOW STEEL PRODUCTS, IND.
5110 Santa Fe Road
Tampa, FL 33619
(813) 248-1944
PRODUCTS: M-5, M-5F, M-5L anchors. Wedges, 3/8 in. to .6 in., 7 degree systems. Chairs, pocketformers and other items for monostrand post-tensioning system.

MOBIL OIL CORPORATION
3225 Gallows Road
Fairfax, VA 22037
(703) 849-3585
PRODUCTS: Corrosion inhibiting grease for unbonded post-tensioning tendons. See Section 2.3.6.

MOBIL OIL CORPORATION
Western Commercial Division
612 South Flower Street
Los Angeles, CA 90017
(213) 683-6221

MOBIL OIL CORPORATION
Midwest Commercial Division
5700 Broadmoor
Suite 512
Shawnee Mission, KS 66202
(913) 384-l 770

MOBIL OIL CORPORATION
Southwest Commercial Division
5151 Belt Line Road
Suite 600
Dallas, TX 76240
(214) 851-5237

MOBIL OIL CORPORATION
Lakes Commercial Division
30300 Telegraph Road
Suite 305
Birmingham, Ml 48010
(313) 540-0200

MOBIL OIL CORPORATION
Northern Region Office
Woodfield Executive Plaza
600 Woodfield Drive
Schaumburg, IL 60196
(312) 885-6000

MOBIL OIL CORPORATION
Indiana/Ohio Commercial Div.
24500 Center Ridge Road
Suite 425
Westlake, OH 44145
(216) 892-8880

MOBIL OIL CORPORATION
So. Atlantic Commercial Div.
340 Interstate North
Bldg. A-4, Suite 425
Atlanta, GA 30339
(404) 9553649

MOBIL OIL CORPORATION
Mid-Atlantic Commercial Div.
10 Executive Mall
530 E. Swedesford Road
Valley Forge, PA 19482
(2 15) 293-4000

MITSUI & COMPANY (U.S.A.), Inc.
One California Street, Suite 3000
San Francisco, CA 94111
(415) 765-l 195
PRODUCTS: Prestressing wire and strand.

NORTHERN DUCTILE CASTINGS, INC.
555 West 25th Street
P.O. Box 98
Hibbing, MN 55746
(218) 263-8871
PRODUCTS: Manufacturers of gray, ductile, and malleable iron castings.

CHAMP CORPORATION
22 N. 159 Pepper Road
Barrington, IL 60010
(312) 381-7564
PRODUCTS/SERVICE: Sales representative for gray, ductile, malleable iron and steel foundries.

PIONEER FOUNDRY
7419 Avenue "0"
Houston TX 77001
(713) 923-4950
PRODUCTS: Manufacturers of gray, ductile, and malleable iron castings.

PENNSYLVANIA MALLEABLE & DUCTILE CASTINGS, INC.
101 Champ Boulevard
Landisville, PA 17538
(717) 898-8240
PRODUCTS: Manufacturers of gray, ductile, and malleable iron castings.

PRECISION SCREW PRODUCTS COMPANY, INC.
1718 W. Main Street
P.O. Box 531139
Grand Prairie, TX 75053-l 139
(214) 264-5755
PRODUCTS: Strand wedges for post-tensioning and pre-tensioning. Two piece and three piece one time or reuseable wedges with or without "O"-Rings, 3/8, 1/2, 6/10 rough or smooth. Special engineering per specifications. Manufacturer of all types of Bridge Wedges, certified for use by Federal and State Authorities. Special wedges for Rock or Soil anchors. Specialists in Multistrand and Monostrand wedges for low relaxation strand, four piece wedges if required. Related machined products from steel and plastic. Exports handled by our staff serving all international markets. Supplying the concrete and construction industry since 1946. See Section 2.3.7.

SHELL OIL COMPANY
100 Executive Drive
West Orange, NJ 07052
(201) 325-5450
PRODUCTS: Shell PT grease for unbonded tendons. See Section 2.3.8.

SHELL OIL COMPANY
Chicago
1415 W. 22nd Street
Oak Brook, IL 60521
(312) 8875706

SHELL OIL COMPANY
Cleveland
7121 Pearl Road
Middleburg Heights, OH 44130
(2 16) 842-4000

SHELL OIL COMPANY
East Coast
100 Executive Drive
P.O. Box 1037
West Orange, N.J. 07052
(201) 325-5450

SHELL OIL COMPANY
Houston
P.O. Box 1422
Houston, TX 77251-1422
(713) 439-1000

SHELL OIL COMPANY
West Coast
P.O. Box 4848
Anaheim, CA 92803
(714) 991-9200

SUPREME PRODUCTS DIVISION
THE MEASUREGRAPH COMPANY
3650 Windsor Place
St. Louis, MO 63113
(314) 533-7800
PRODUCTS: Complete line of post-tensioning wedges - 2 piece and 3 piece, with and without retaining circlips ranging in size from .120 in. to .720 in.

Strand chucks and splice chucks.

Hydraulic jack wedges, stressing jaw assemblies and miscellaneous parts.

Anchorage systems for epoxy coated strand, including ductile iron castings.

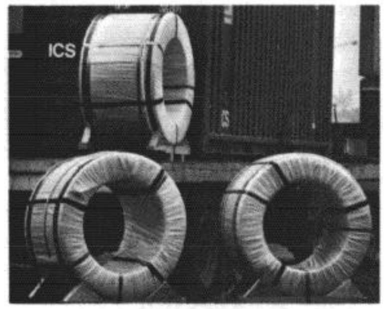
Release solutions, cleaning brushes and related accessories. See Section 2.3.12.

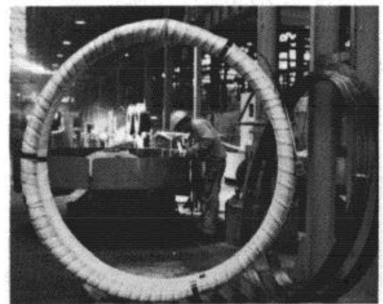
VELZY ENGINEERING & MACHINE, INC.
12501 Gladstone Avenue, Unit A4
Sylmar, CA 91342
(818) 365-5697
PRODUCTS: Manufacturer of Post-Tensioning Jacks and Accessories. See Section 2.3.13.

VISCOSITY OIL COMPANY
3200 S. Western Avenue
Chicago, IL 60608
(312) 847-0224
TWX #91 O-221 -0245
PRODUCTS: Corrosion preventatives, industrial lubricants and metal working coolants. See Section 2.3.14.

2.3.3 THE CRISPIN COMPANY

The Crispin Company represents Acindar-Industria Argentina De Aceros S. A. in marketing of strand and wire to the post-tensioning industry in the United States. We supply seven wire thermo-mechanical stress relieved low relaxation strand complying with ASTM A 416-80 in Grades 250 or 270. Grade 250 strand isavailable in 5/16, 3/8, 1/2 and 0.6 inch diameters, and Grade 270 strand is available in 3/8, 1/2 or 0.6 inch diameters. The thermo-mechanical stress relieving process is done to eliminate residual stresses introduced during drawing and stranding. Our strand is self straightening and it is particularly suitable to be used at elevated temperatures, due to the low relaxation properties. Thermo-mechanical stress relieved low relaxation wire for prestressed concrete is manufactured to meet ASTM A 421-77. Technical specifications for our strand and wire products are presented in the tables below.





Acindar stand (top) and wire (bottom) packed for shipping.

STRAND

STANDARD* SPECIFICATION	Nominal Diameter		Nominal Cross Sectional Area of Steel		Minimum Breaking Load		Nominal Weight		Elongation under load over 610 mm (24 in)	Relaxation loss after 1000 h. at 68°F (20°C)	
	mm	in	mm²	sq.in	kN	lb	kg/1000 m	lb/1000 ft		70% min. Break. Load	80% min. Break.Load
ASTM A-416/ 1980 Grade 250	7,9	5/16	37,43	0.058	64,5	14500	294	197	min. 3.5%	max. 2.5	max. 35
	9,5	3/8	51,61	0.080	89,0	20000	405	272			
	12,7	1/2	93,00	0.144	160,1	36000	730	490			
	15,2	0.6	139,40	0.216	240,2	54000	1.094	737			
ASTM A-416/ 1980 Grade 270	9,5	3/8	54,84	0.085	102,3	23000	432	290			
	12,7	1/2	98,71	0.153	183,7	41300	775	520			
	15,2	0 6	140,00	0.217	260,7	58600	1.102	740			

WIRE

GRADE	NOMINAL DIAMETER		NOMINAL STEEL AREA		NOMINAL WEIGHT		MINIMUM TENSILE STRENGTH		MINIMUM STRESS AT 1 PER CENT EXTENSION		TOTAL ELONGATION UNDER LOAD	Relaxation loss after 1000 h. at 68°F (20°C)	
	mm	in	mm²	sq. in	kg/1000 m	lb/1000 ft	M Pa	psi	M Pa	psi	%	70% Min. T. strength	80% Min. T. strength
Type WA	4,88	0.192	18,70	0.0290	146,62	98.4	1725	250000	1553	225000	min. 4.0	max. 2.5	max. 3.5
	4,98	0.196	19,48	0.0302	154,07	103.4	1725	250000	1553	225000			
	6,35	0.250	31,67	0.0491	248,53	166.8	1655	240000	1490	216000			
	7,01	0.276	38,57	0.0598	303,07	203.4	1620	235000	1459	211500			
Type BA	4,98	0.196	19,48	0.0302	154,07	103.4	1655	240000	1490	216000			
	6,31	0.250	31,67	0.0491	248,53	166.8	1655	240000	1490	216000			
	7,01	0.276	38,57	0.0598	303,07	203.4	1620	235000	1459	211500			

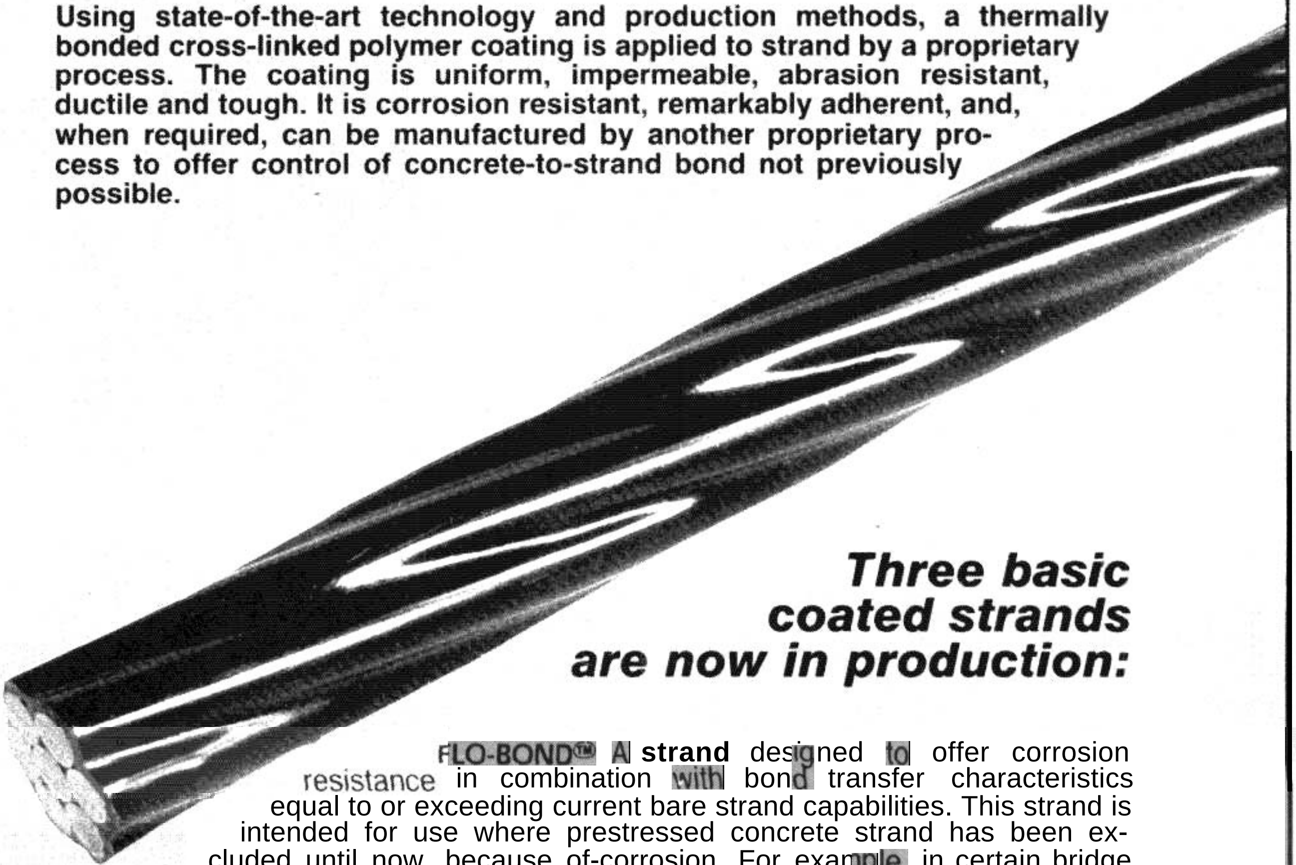
To learn more about ACINDAR and for further information concerning the availability of our products in the United States, please contact THE CRISPIN COMPANY.

ACINDAR
INDUSTRIA ARGENTINA DE ACEROS S.A.

INTRODUCING:

FLO-TECH T.M. *Systems*

Using state-of-the-art technology and production methods, a thermally bonded cross-linked polymer coating is applied to strand by a proprietary process. The coating is uniform, impermeable, abrasion resistant, ductile and tough. It is corrosion resistant, remarkably adherent, and, when required, can be manufactured by another proprietary process to offer control of concrete-to-strand bond not previously possible.



Three basic coated strands are now in production:

FLO-BOND™ A strand designed to offer corrosion resistance in combination with bond transfer characteristics equal to or exceeding current bare strand capabilities. This strand is intended for use where prestressed concrete strand has been excluded until now, because of corrosion. For example, in certain bridge deck panels and in structural members exposed to marine or salt environments.

FLO-BOND SPECIAL™ A specially coated strand designed to provide controlled bond-transfer for such applications as hollow-core slab construction and railroad ties. Its special coating and bond characteristics make it preferable where the sole purpose of application is to improve bond transfer. As such, it offers only modest improvement in corrosion resistance.

FLO-GARD™ A strand designed exclusively for corrosion resistant applications intended to replace alternate methods of corrosion resistance primarily in post-tensioning applications where bonding characteristics are secondary.

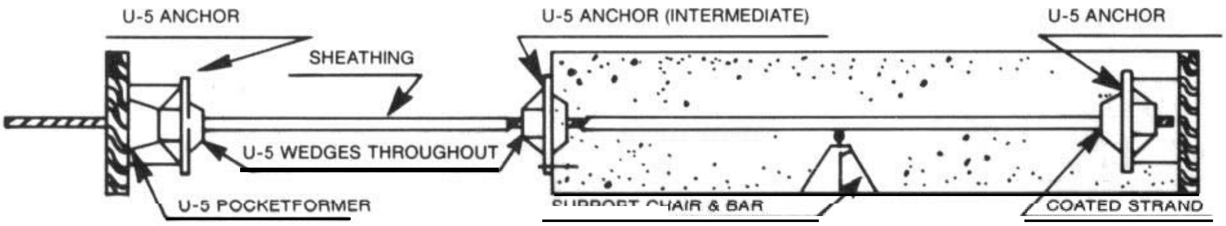
Florida Wire and Cable Company

FOR MORE INFORMATION CALL OR WRITE US AT OUR JACKSONVILLE HEADQUARTERS ADDRESS AND TELEPHONE NUMBER LISTED IN SECTION 2.3.1

2.3.5 THE GREAT SOUTHWEST MARKETING CO., INC.

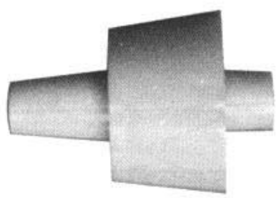
GREAT SOUTHWEST MARKETING AND MANUFACTURING IS A WHOLESALE MANUFACTURER OF POST-TENSION STRESSING EQUIPMENT AND COMPONENTS AND A MARKETING AGENT FOR UNCOATED, 270K 7-WIRE STRAND.

U-5 ANCHORAGE SYSTEM

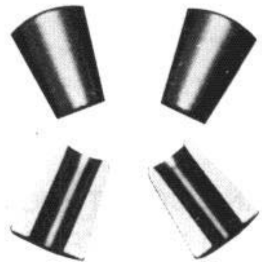


ICBO No.4164

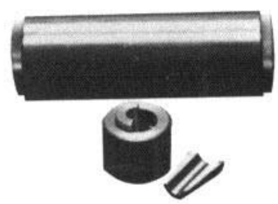
u-5
ANCHOR



u-5
POCKETFORMER

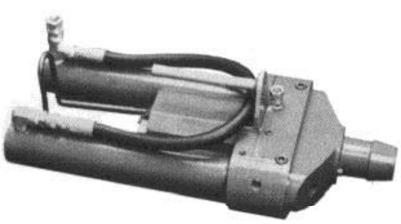


u-5
WEDGE



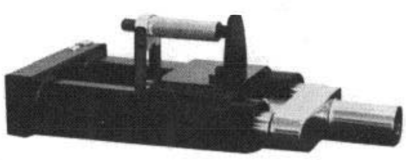
u-5
SPLICE CHUCK
STEEL DONUT

HYDRAULIC STRESSING JACKS



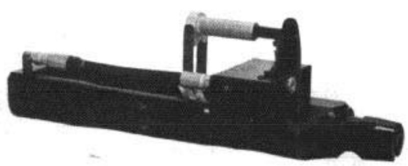
GS-5P

- POWER WEDGE SETTER
- SPRING RETURN CYLINDERS
- STRESSES 7/16", 3/8", 1/2" STRAND
- WEIGHS 54 LBS
- SEQUENCE BLOCK RECOMMENDED
- AVAILABLE FOR .6 STRAND



GS-5A

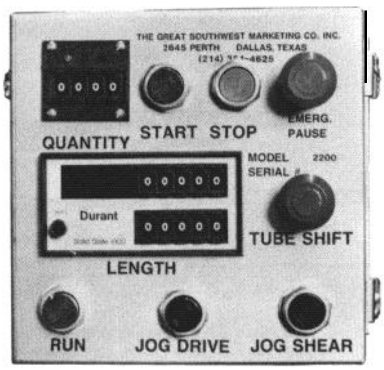
- POWER WEDGE SETTER
- POWER RETURN CYLINDERS
- STRESSES 7/16", 3/8", 1/2" STRAND
- WEIGHS 38 LBS
- SEQUENCE BLOCK RECOMMENDED
- ONE PIECE CONSTRUCTION



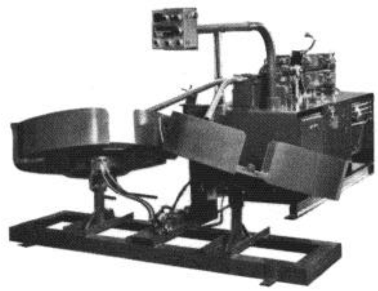
GS-5L

- POWER WEDGE SETTER
- POWER RETURN CYLINDERS
- STRESSES 7/16", 3/8", 1/2" STRAND
- WEIGHS 38 LBS
- "BUILT IN" SEQUENCE BLOCK
- AVAILABLE FOR .6 STRAND

FABRICATING EQUIPMENT AND CONTRACT MACHINING



CONTROL PANEL FOR FABRICATION LINE



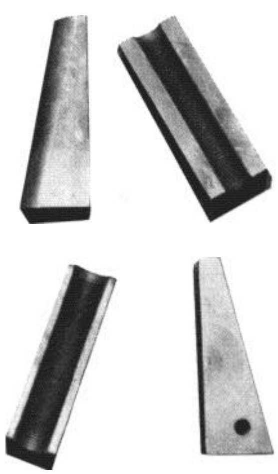
FABRICATION LINE

PAY-OFF FOR FABRICATION LINE

USED BY MORE POST TENSIONERS THAN ANY OTHER

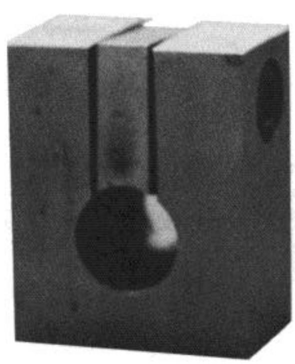
- DESIGN AND CONSTRUCT COMPLETE EXTRUSION LINES
- DESIGN AND CONSTRUCT COMPLETE WRAP LINES

HYDRAULIC JACK COMPONENTS

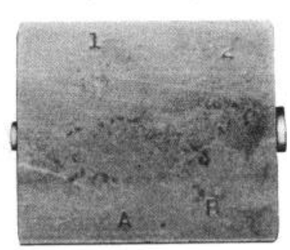


RAM GRIPPERS

- GEMI
- ATLAS
- CUSTOM GRIPPERS

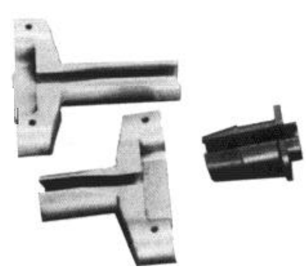


SPLIT - DONUT



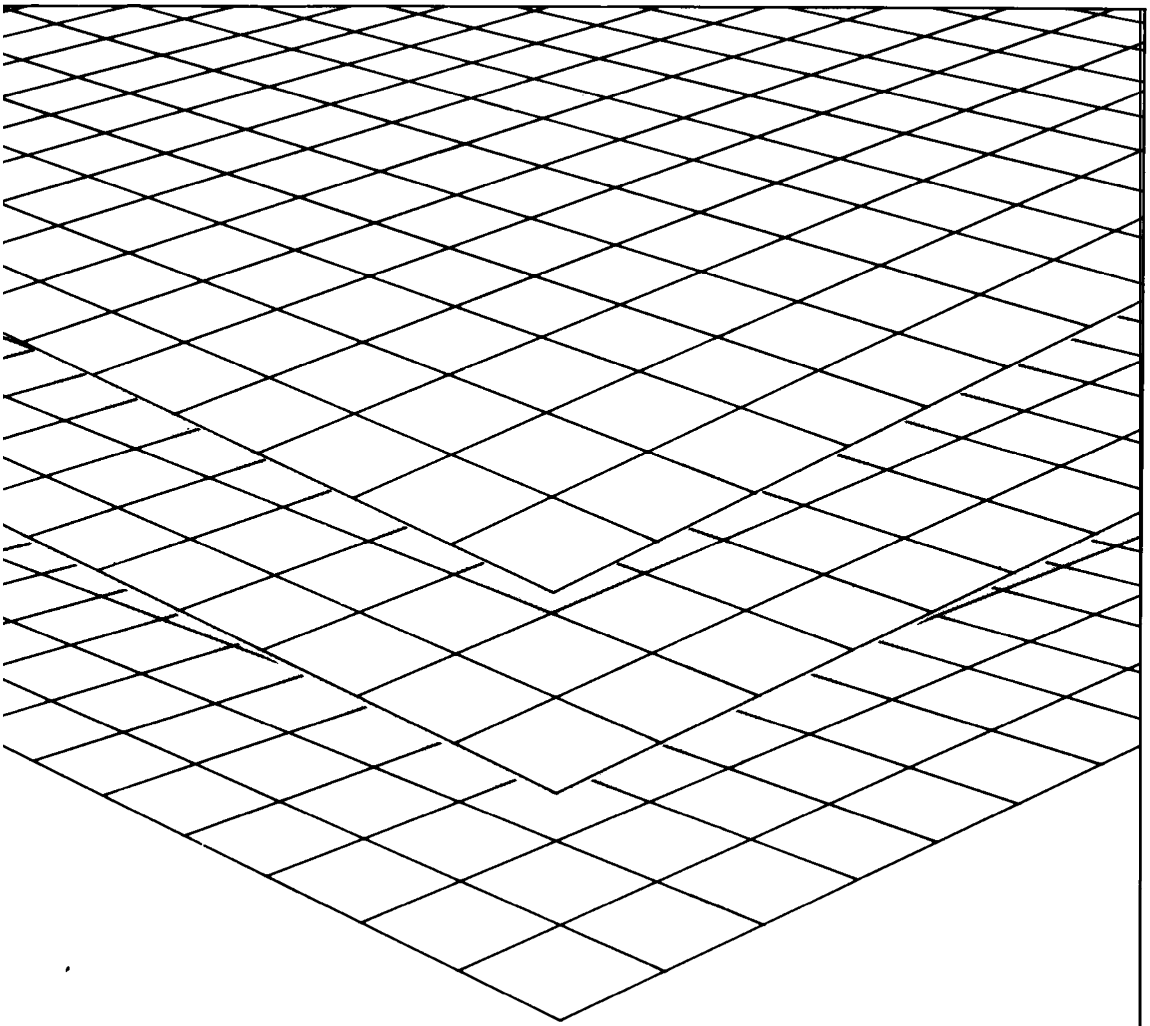
SEQUENCE BLOCK

- AUTOMATIC SEATING VALVE
- USE WITH GS-5P AND GS-5A



ASSORTED NOSE PIECES

2.3.6 MOBIL OIL CORPORATION

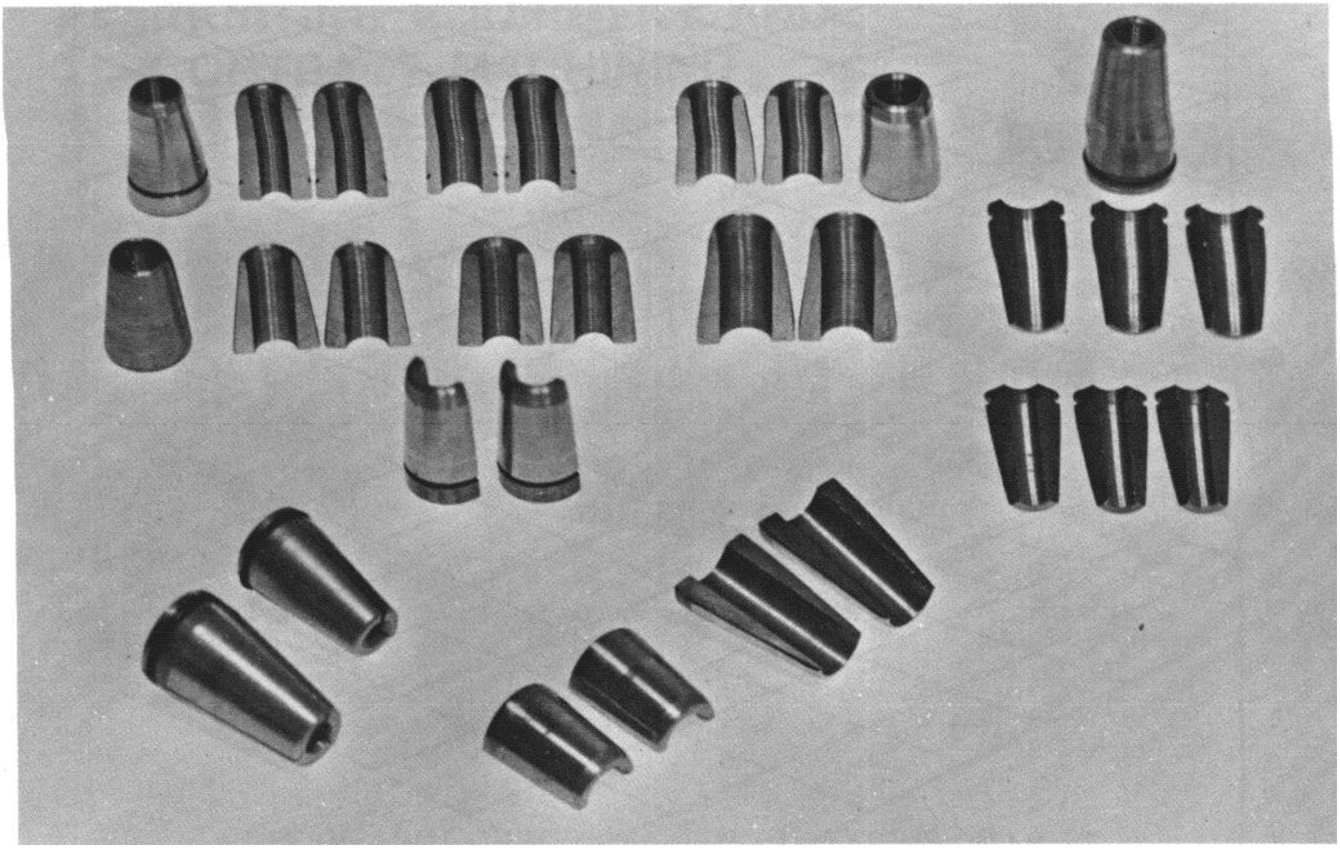


We're a leader in lubricants and rust preventatives.

- Specially formulated products for post tensioned concrete construction
- A full range of high quality petroleum and synthetic corrosion preventives and lubricants
- World wide service and distribution

Mobil"

2.3.7 PRECISION SCREW PRODUCTS COMPANY, INC.



PRODUCTS FROM PRECISION . . . . . SINCE 1946

Presision Screw Products Co., supplies the following products and services to the prestressed concrete industry:

PRODUCTS: Strand wedges, anchors, pocketformers, and intersectional chairs for post-tensioning and pre-tensioning. Two piece and three piece one time or reuseable wedges with or without “0”-Rings, 3/8, 1/2, 6/10 rough or smooth. Special engineering per specifications. Manufacturer of all types of Bridge Wedges, certified for use by Federal and State Authorities. Special wedges for Rock or Soil anchors. Specialists in Multistrand and Monostrand wedges for low relaxation strand, four piece wedges if required. Related machined products from steel and plastic. Exports handled by our staff serving all international markets. Supplying the concrete and construction industry since 1946.

SHELL PT GREASE

Shell PT Grease is the latest in the family of grease products supplied for the post-tensioning industry. PT Grease is newly formulated and achieves the objective of providing superior corrosion protection to unbonded tendon pre-stressing steel through the exclusion of moisture. This is effectively accomplished by a physical shield in the form of a stable grease structure and additives which selectively coat metal surfaces, exclude air and water and provide long-term corrosion protection.

Shell PT Grease incorporates the following characteristics which contribute to its successful performance:

- Sheathing Compatibility — excellent compatibility is exhibited by PT Grease which enables the sheathing to continue its protective cover.
- Effective Moisture Barrier — additives which exhibit a preferential affinity for metal and are capable of displacing air and water are utilized for long-term corrosion protection.
- Selected Base Oils-the base oils used in the manufacture of PT Grease are compatible with pre-stressing steel and inhibit rusting of the steel even in the presence of 2% added water in the base oil.
- Corrosion Protection — PT Grease substantially outperformed a leading competitive product in the severe ASTM B-117 5% Salt Water Spray Test, reflecting its superior ability to provide corrosion protection to steel.
- Freedom from Impurities — contaminants such as chlorides, sulfides and nitrates which could create harmful side effects with pre-stressing steels are virtually absent from PT Grease.

Test	Shell PT Grease Method	Typical Properties
Worked Pen. @ 25°C 60 strokes	ASTM D-217	325
Dropping Pt., °C	ASTM D-2265	189
5% Salt Water Spray Test, 720 Hours	ASTM B-l 17	7, 7
Water Soluble Ions		
Chlorides	Ion Chromatography	1 ppm
Sulfides	Colorimetric	<l ppm
Nitrates	Ion Chromatography	< 1 ppm
Sheathing Compatibility, MDPE		
30 days @ ±100° F		
Hardness Change		-1%
Volume Change		+6.26%
Cracking		None
Rust Test, Steel		
30 days @ 100°F		
Cable in dry oil		No rust
Cable in oil saturated with 2% water		No rust

For availability of Shell PT Grease you can contact one of the Shell Lubricant Sales Offices listed in Section 2.3.2

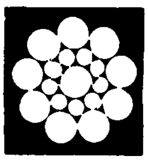
2.3.9 SHINKO WIRE COMPANY, LTD.

Shinko Wire Company Ltd. is capable of manufacturing all kinds of prestressing steels, and has been supplying for over 25 years the highest quality prestressing wire and strand to the U.S. prestressed concrete industry. In addition to the products conforming to ASTM A-421 and A-416 including low relaxation grade, Shinko Wire can supply the following special prestressing wires and strands:

- Extra high strength wire (Normal and Low Relaxation Grade)
- Large diameter nineteen wire strand (Low Relaxation Grade)
- Galvanized wire and strand
- Indented wire and strand

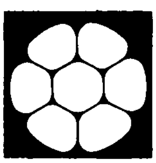
Now, we can also supply extra high strength strand, 300k 1/2 inch, upon request.

● Large Diameter Nineteen Wire Strand



Nominal Diameter in	Nominal Steel Area sq in	**Nominal Weight lb/1000 ft**	Minimum Breaking Load lb	Minimum Load at 1% Exten. lb	**Minimum Elongation %**
0.7	0.323	1,110	87,100	78,500	3.5
0.76	0.378	1,300	101,400	91,300	3.5
0.8	0.420	1,440	111,300	100,300	3.5
0.86	0.485	1.670	128,700	116,000	3.5

● Stabilized Compact Strand (Seven Wire Drawn Strand)



Nominal Diameter in	**Nominal Steel Area sq in**	Nominal Weight lb/1 000 ft	Minimum Breaking Load lb	Minimum Load at 1% Exten. lb	Minimum Elongation %
0.5	0.174	600	47,000	40,870	4.0
0.6	0.256	870	67,440	58,450	4.0
0.7	0.350	1.196	85,430	74,180	4.0

● Galvanized Wire and Strand

Product	Nominal Diameter in	Nominal Steel Area sq in	Nominal Weight with Coating lb/1 000 ft	Minimum Breaking Load lb	Minimum Load et 1% Exten. lb	Minimum Elongation %
Wire	0.196	0.0302	105	6,640	5,440	4.0
	0.236	0.0439	150	9,660	7,900	4.0
	0.276	0.0598	205	13,200	10,800	4.0
7 wire Strand	0.375	0.085	303	21,400	18,200	4.5
	0.438	0.115	407	28,700	24,400	4.5
	0.500	0.153	541	38,200	32,500	4.5
	0.600	0.217	769	54,200	46,100	4.5
19 wire Strand	0.700	0.323	1,110	74,300	63,200	4.5

***Minimum Weight of Zinc Coating: 0.85 oz/ft²**

2.3.10 SIDERIUS, INC.

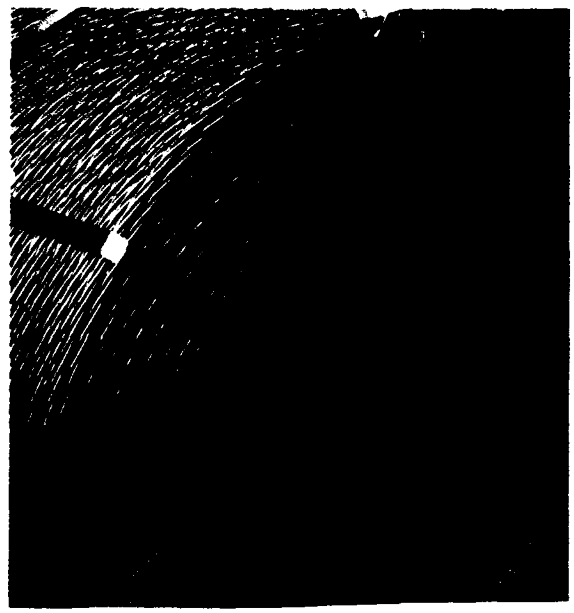
Deriver prestressing wires and strands.

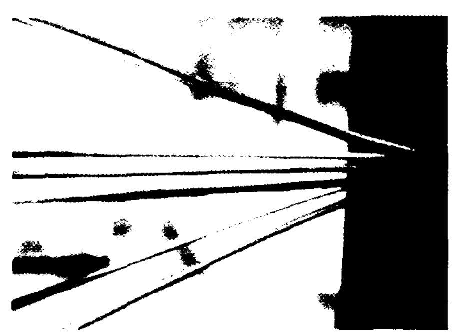
Physical and Mechanical Properties

Uncoated seven-wire stress-relieved and stabilized strand for prestressed concrete

(acc. to ASTM A 416-80 gr. 270)

PROPERTIES	UNIT	PRODUCTS		
Nominal diameter	in mm	**⅜** **9,53**	**½** 12.70	**0,6** 15.24
Nominal steel area	in* mm*	0,065 **54,84**	0,153 **98,71**	0,217 **140,00**
Variations in diameter	**in** **mm**	+ 0,026 and - 0,006 for all sizes + **0,66** and - **0,15** for all sizes		
Dii. betw. center and outer wires diameter	in mm	0,002 **0,0508**	0,003 **0,0762**	0,004 **0,1016**
Stranding pitch	—	12 to 16 times the nom. diameter		
Nominal weight	**lb/1000 ft** **Kg/1000 m**	290 432	520 775	740 1102
Breaking Strength(l)	lbf **kN**	23.000 **102,3**	41.300 **183,7**	56.600 **260,7**
Load at 1% Extension	lbf **kN**	20.700 **92,1**	37.200 **165,3**	52.700 **234,7**
Elongation under load	%	≥ **3,5** for all sizes		
Relaxation after 1000 h at 66°F **(20°C) with:**				
1) init. load: 70% m.b.str.	%	≤ 2.5 for all sizes		
2) init. load: 60% **m.b.str.**	%	≤ **3,5** for all sizes		
(1)**Minimum** actual ultimate strength of **each** wire	**N/mmq**	1665	1661	1662





Standards: Deriver prestressing steels conform to all international and domestic standards including: ASTM **A416-80;** ASTM **A421-79;** ASTM **A648-79;** BS 58%: 1980; DIN 4227; and S.I.A. 162 or any other specifications the customer desires.

Distributed by

Siderius, Inc.

U.S. Subsidiary of Finsider International

Finsider Group

2.3.11 SUMITOMO ELECTRIC INDUSTRIES, LTD.

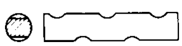
Sumitomo Electric Industries, Ltd., Sumiden Wire Products Corporation and Sumiden Wire Sales...

the Perfect Combination.

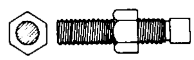
Through Sumitomo Electric Industries, Ltd. and Sumiden Wire Products Corporation, American and Japanese engineers have combined their skills and technologies to produce superior tensioning materials for prestressed concrete.

Sumitomo Electric Industries, Ltd. and Sumiden Wire Products Corporation provide the most complete line of dependable tensioning materials available.

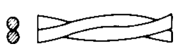
Tensioning Materials



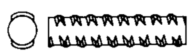
Indented Wire



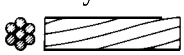
Round Bar



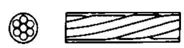
Two-Ply Wire



Threaded Bar



Seven-Wire Strand



Unbonded Seven-Wire Strand

Nineteen-Wire Strand

Unbonded Nineteen-Wire Strand

Tensioning Equipment and Accessories

Tensioning Jack and Pump

Pushing Machine

Heading Machine

All types of Strand, Wire and Bar Anchorage

Sumitomo Electric Industries, Ltd.
Special Steel Wire Division

Sumiden Wire Products Corporation
Sumiden Wire Sales

2.3.12 SUPREME PRODUCTS DIVISION

Supreme Products has been producing and supplying the very finest of Anchorage Hardware to the Prestressed Concrete Industry for nearly 30 years. Our products are used in over 50 countries throughout the world.

Supreme's efforts to remain an industry leader resulted in the first grips to efficiently anchor 270 K.S.I. Strand and, more recently, Low Relaxation Strand. Supreme was first, and to our knowledge is still the only manufacturer to proof test 100% of the anchorage bodies which are produced in our plants.

Currently Supreme is extending the range of our newly developed line of anchorage hardware for epoxy coated strand, which has been successfully used on new projects and repair of existing prestressed structures.

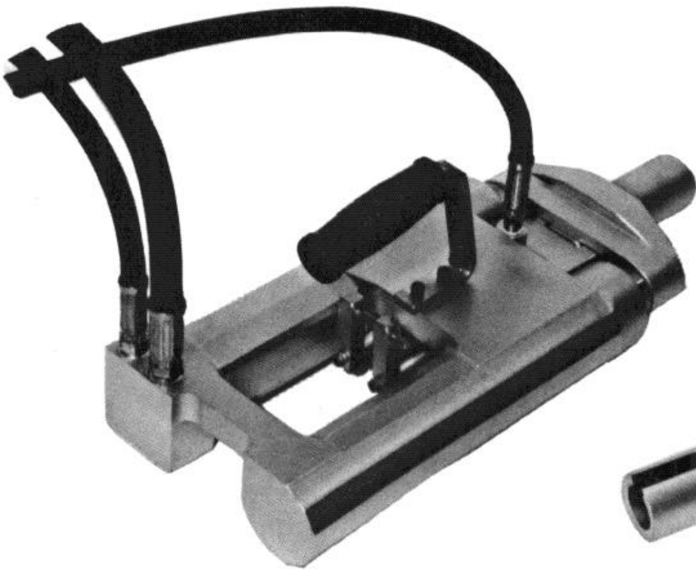
Supreme offers the following products and services:

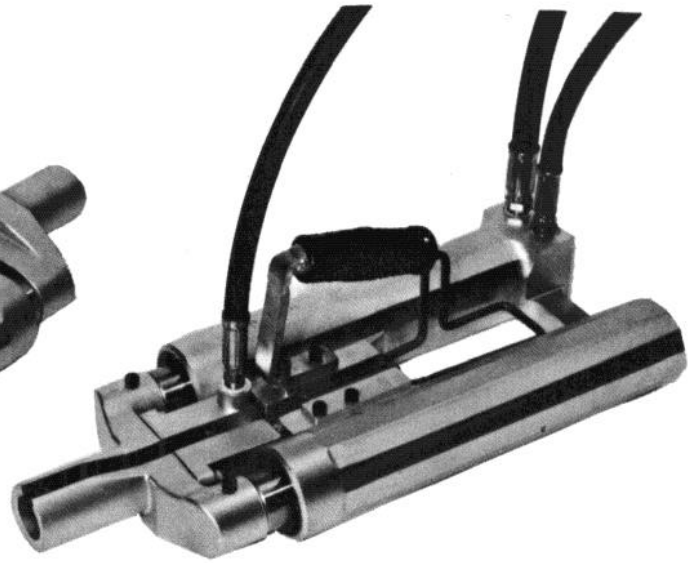
POST-TENSIONING WEDGES	Sizes range fron .120 In. (3 MM) to .720 In. (18.3 MM). Both 2 piece and 3 piece, with or without retaining circlips. Use range includes S.O.G., building structures, rock anchors, bridges and nuclear containment structures.
REUSABLE PRODUCTS	Complete size range includes strand and wire chucks and their parts. 3 piece stressing jaws fit several post-tensioning systems.
SPLICE CHUCKS	One-Time-Use and Reusable Chucks to splice two strands together. Thousands are used yearly to quickly and efficiently correct field and shop errors or make deliberate connections.
STRESSING JACK GRIPPERS	Custom made to fit and size strand — very long term usage built in through correct heat treatments. 1/2 inch and .6 inch usually in stock.
CAST ANCHORS	Newly created anchors designed to develop strength potential in latest higher strength low-relaxation strands — including epoxy coated.
FORENSIC — TROUBLE SHOOTING	Consultation on new product needs, stressing problems, proper manufacturing, heat treating and inspection procedures. 70 years manufacturing and 25 years prestressing project related experience.

2.3.13 VELZY ENGINEERING & MACHINE, INC.

Manufacturer of Monostrand Post Tensioning Jacking Systems and Labor Saving Accessories.

Wrap & Seal Post Tensioning Cable Processing Lines built to order.





STRESSING JACKS
3/8" & 1/2" 270K Strand

Ram Area -- 6.26 sq. in.

Operating Pressure at 33K -- 5254 PSI

Hydraulic Return - Pressure Seating

Length of Stroke	Weight
6"	33 lbs.
8-1/2"	38 lbs.
12"	45 lbs.

Spring Return - Spring Seating

Length of Stroke	Weight
3"	25 lbs.
9"	38 lbs.

Hydraulic Seating Attachments and Spring Seating Attachments ranging from 7/8" to 12" standard sizes.

Jack Gripper Sizes -- 3/8" & 1/2" for 270K Strand.

2.3.14 VISCOSITY OIL COMPANY

Viscosity Oil Company has been a supplier of the International Post-Tensioning Industry with unbonded coatings for the past eighteen years.

Through research and development we have continually strived to provide the industry with products using the latest technology available.

We take pride in assisting Architect Engineers, Post-Tensioning Contractors and End Users in meeting their corrosion protection requirements.
Our customers include licensees of major International Post-Tensioning Systems: Bureau BBR, Freyssinet, and Losinger/VSL, in both commercial and nuclear work.

The following Visconorust Corrosion Preventive products are used by the Post-Tensioning Industry in unbonded systems.

NUCLEAR

Extended Protection Coating — Visconorust 2090P-4 Casing Filler
Temporary Coatings — Visconorust 1601 Amber; Visconorust 1702 Amber

COMMERCIAL

Plastic Coated Mono-Strand — Visconorust 3166 (standard product)
Visconorust 3166H (meets PTI specifications)
Water Removable Coatings — Visconorust 8415E
Extended Life Coating — Visconorust 4663 (severe duty service)
Rock Anchors — Visconorust 2889
Temporary Parts Storage — Visconorust 1050

OTHER PETROLEUM PRODUCTS

Complete Line — Lubricants
Metal Working Fluids
Greases
Cleaners
Speciality Products for Unique Application

Complete specifications and technical information available upon request.

VISCOR
LUBRICANTS SINCE 1892

Chapter 3

Specifications

3.1 GUIDE SPECIFICATIONS FOR POST-TENSIONING MATERIALS

3.1.1 GENERAL

This specification was developed as a guide to engineers designing and specifying post-tensioned concrete structures and also as a basis for establishing the testing phase of a certification program for post-tensioning materials.

The system and text presented herein requires that each user make the changes, additions or deletions necessary to adapt these specifications to his specific job condition and specification format.

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3.1.2 Scope

(1) This specification provides the minimum requirements for post-tensioning materials which are a part of this prestressed concrete structure.

The intent of the specification is to cover post-tensioning systems applicable to all common prestressed concrete structures. There are, however, certain special structures which, either because of their service requirements or structural behavior, might impose requirements on the post-tensioning system which exceed the minimum requirements of these standard specifications. It is recommended that, in such cases, a special set of specifications be written.

(2) The provisions of this specification shall also govern for non-structural applications.

Non-structural applications might include topping slabs, waterproofing slabs on fill, crack control, and deflection control. For non-flexural or membrane type structures primarily under tensile forces, the provisions, where appropriate, are intended to apply.

3.1.3 Definitions

(1) **Tendon** — The complete assembly consisting of anchorage and prestressing steel with sheathing when required. The tendon imparts prestressing forces to the concrete.

(2) **Bonded Tendons** — Tendons which are bonded to the concrete through grouting or other approved means, and therefore are not free to move relative to the concrete.

(3) **Unbonded Tendons** — Tendons in which the prestressing steel is permanently free to move relative to the concrete to which they are applying their prestressing forces.

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(4) **Anchorage** — The means by which the prestressing force is permanently transmitted from the prestressing steel to the concrete.

(5) **Prestressing Steel** — That element of a post-tensioning tendon which is elongated and anchored to provide the necessary permanent prestressing force.

(6) **Coating** — Material used to protect against corrosion and/or lubricate the prestressing steel.

(7) **Sheathing** — Enclosure around the prestressing steel to avoid temporary or permanent bond between the prestressing steel and the surrounding concrete.

Different requirements are imposed upon sheathings for bonded and unbonded tendons. In unbonded tendons, the sheathing does not transmit any bond stresses from the prestressing steel to the concrete and therefore has to assure the freedom of movement of the prestressing steel and form an adequate cover over the coated tendon. In bonded tendons, bond stresses will be transmitted through the sheathing, and it must be of such material and/or configuration to effective/y allow this stress transfer.

The void in the concrete in which the tendon is to be located can also be pre-formed (e.g. by inflatable and removable tubes) and the tendon pulled through subsequently. With pre-formed voids no additional shea thing will be required.

(8) **Coupling** — The means by which the prestressing force may be transmitted from one partial-length prestressing tendon to another.

3.1.4 Prestressing Steel

(1) **Wire** — Wire used in post-tensioning tendons shall conform to ASTM A421, "Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete" (see Section 3.4.1).

(2) **Strand** — Strand used in post-tensioning tendons shall conform to ASTM A416, "Specifications for Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete" (see Section 3.4.2).

(3) **Bar** — Bars used in post-tensioning tendons shall conform to ASTM A722, "Specifications for Uncoated High-Strength Bar for Prestressing Concrete" (see Section 3.4.3).

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(4) Wire, or strand, not specifically itemized in ASTM A421 or A416, may be used provided they conform to the minimum requirements of thesespecificationsand have no properties which make them less satisfactory than those listed in ASTM A421 or A416.

(5) Bars not specifically itemized in ASTM A722, including bars with a minimum ultimate tensile strength greater than 150,000 psi (1035 MPa), may be used provided they conform to all other requirements of this specification (including, in particular, those of Sections 4, 5, and 6 therein) and have no properties which make them less satisfactory than those listed in ASTM A722.

3.1.5 Bonded Tendons

(1) **Anchorages** -The anchorages shall develop at least 95 percent of the actual ultimate strength of the prestressing steel, tested in an unbonded state without exceeding anticipated set. The actual strength of the prestressing steel shall not be less than specified by the applicable ASTM Standard, and shall be determined by tests of representative samples of the tendon material in conformance with ASTM Standards. The anchorage shall be so arranged that the prestressing force of the tendon may be verified prior to removal of the stressing equipment.

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Provisions should be made for new steels which would include new sizes, or improved characteristics of relaxation, bond, or mechanical properties. However, use of wire, strand, or bar prestressing steels not covered by ASTM Specifications should be permitted only when the supplier provides conclusive test data substantiating that all characteristics of the material are comparable to the properties of steels conforming to the applicable ASTM Specification. In particular, the stress corrosion characteristics of steels produced by quench and temper heat treatment should be carefully evaluated.

Relaxation properties of new steel should be determined by adequate testing (minimum 7,000 hours).

The ASTM A722 covers bars with minimum ultimate tensile strength of 150,000 psi. Bars with 160,000 psi ultimate tensile strength meeting all the requirements of ASTM A722 have been used extensively in the United States

This specification is intended primarily for post-tensioning tendons used in flexural members, where the location of the critical section of the member under service and ultimate load is some distance away from the anchorage itself. An ultimate anchorage capacity of 95 percent of the actual ultimate strength of the prestressing steel is generally greater than the design strength of post-tensioning tendons. The bond length between the anchorage and the critical section will provide additional capacity at the critical section. The combination of bond and anchorage, tested in the bonded state should develop 100 percent of the minimum specified ultimate strength of the prestressing steel.

If, in the unbonded state, the anchorage develops at least 95 percent of the actual ultimate strength of the prestressing steel, and at least 2 percent elongation when measured in a minimum gage length of 10 ft., it need not be tested in the bonded state.

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(2) **Sheathing-Sheathing** material for bonded tendons shall be strong enough to retain its shape and resist unrepairable damage during construction. It shall prevent the entrance of cement paste from the concrete. Material left in place shall not cause harmful electrolytic action nor deteriorate. The inside diameter shall be at least 1/4 in. larger than the nominal diameter of single wire, bar or strand tendons, or in the case of multiple wire, bar or strand tendons, the inside cross-sectional area of the sheathing shall be at least two times the net area of the prestressing steel. Sheathing shall be capable of transmitting forces from grout to the surrounding concrete.

Sheathing shall have grout openings at each end and at all high points except where tendon curvature is small and the tendon is relatively level, such as in continuous slabs.

Drain holes shall be provided at low points if the tendon is to be placed, stressed and grouted in freezing climate.

(3) **Grout —** (see Section 3.3, "Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete.")

(4) **Couplings —** Couplings of bonded tendons shall be used only at locations specifically indicated and/or approved by the Engineer. Couplings shall not be used at points of sharp tendon curvature. All couplings shall develop at least 95 percent of the actual ultimate strength of the prestressing steel without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplings and/or coupling components shall be enclosed in housings long enough to permit the necessary movements, and fittings shall be provided to allow complete grouting of all the coupling components.

(5) **Multiple Tendons —** Tendons composed of multiple strands, wires of bars in a common sheath should be tensioned simultaneously unless the effects of interferences between the elements are considered.

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Some State Highway Department specifications require high point grout vents only for structures over 400 ft. in length.

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(6) Ultimate Strength — Ultimate strength for bonded tendons shall not be taken greater than the ultimate capacity of the anchorages or couplings when they are located at critical sections under ultimate load. Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall be sufficient to develop the specified ultimate strength of the prestressing steel.

3.1.6 Unbonded Tendons

(1) **Anchorages —** The anchorages of un bonded tendons shall develop at least 95 percent of actual ultimate strength of the prestressing steel without exceeding anticipated set. The actual strength of the prestressing steel shall not be less than specified by the applicale ASTM Standard, and shall be determined by tests of representative samples of the tendon material in conformance with ASTM Standards. The total elongation under ultimate load of the tendon shall not be less than 2 percent measured in a minimum gauge length of 10 ft. Additional requirements for anchorages of unbonded tendons are presented in Sections 3.2.3(1), 3.2.3(2), 3.2.3(3), and 3.2.3(5).

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Bond lengths required to develop that portion of the guaranteed ultimate tensile strength of the prestressing steel not developed by the anchorage or coupler may be determined by the results of bond tests on un tensioned pres tressing s teel.

In developing these specifications, consideration was given to both previously published specifications and currently available test data on the performance of unbonded members. Of particular importance are the specifications for sta tic strength and ductility set for anchorages and couplings in Sections 3.1.6(1) and 3.1.6(4). The following considerations led to these minimum requirements:

Static Strength — The upper bound of the design strength of unbonded tendons in'the ACI 318 Building Code is the minimum specified yield strength of the tendon material, which is 85 percent of the minimum guaranteed ultimate tensile strength for stress relieved strand, and 90 percent of the minimum guaranteed ultimate tensile strength for low relaxation strand. In nearly all cases, the design tendon strength will be substan tially less than the yield strength. Accordingly, the requirement that anchorages for unbonded tendons develop 95 percent of the actual strength of the tendon material (which is normally a few percent greater than 95 percent of the minimum guaranteed ultimate strength) provides a substantial safety margin between the strength of the tendon at the anchorage and the design strength of the tendon.

Static Ductility — Along with a strength requirement, it is important that specifications for unbonded tendons include a ductility requirement. This is usually expressed as a minimum percent elongation in a ten foot gauge length under total load. This requirement ensures that the anchorage used does not damage the prestressing steel and lead to a failure at an elongation below specified. The tendon should elongate appreciably to avoid the possibility of a brittle type failure. Test data indicate that maximum strain which can be expected in an unbonded tendon in a concrete flexural member is in the order of 1 percent. Because of the sensitivity of the strain in these *high-stress regions, and to provide a comfortable margin of safey, 2 percent is specified as the required total elongation under ultimate load. A tendon satisfying this requirement will possess ductility capacity greater than the member which contains it.*

(2) Coating — See Section 3.2.5 for corrosion preventive coating requirements for unbonded tendons.

(3) Sheathing — See Section 3.2.4 for requirements for sheathing for unbonded tendons.

(4) Couplings — Couplings of unbonded tendons shall be used only at locations specifically indicated and/or approved by the Engineer. Couplings shall not be used at points of sharp tendon curvature. All couplings shall develop at least 95 percent of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The couplings of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplings and/or components shall be enclosed in housings long enough to permit the necessary movements. All the coupling components shall be completely protected with a coating material prior to final encasement in concrete. The coating material used on couplings shall conform to the requirements of Section 3.25.

(5) Multiple Tendons — Tendons composed of multiple strands, wires, or bars in a common sheath should be tensioned simultaneously unless the interferences between the elements are considered.

(6) Ultimate Strength — Ultimate strength for unbonded tendons shall not be taken greater than the ultimate capacity of the anchorages or couplings.

3.1.7 Bearing Stresses

The average bearing stresses on the concrete created by the anchorage plates shall not exceed the values allowed by the following equations:

Experience has shown these formulas to yield satisfactory values as a guide in the size selection of bearing plates. Actual bearing stress distribution and its effect on the anchorage zone must be taken into account in the design of the structural member. This design falls outside the scope of this specification dealing only with the post- tensioning materials.

At service load —

$f_{cp} = 0.6\ f'_c \sqrt{A'_b/A_b}$

but not greater than 1.25 f'_c

At transfer load —

$f_{cp} = 0.8\ f'_{ci} \sqrt{(A'_b/A_b) - 0.2}$

but not greater than 1.25 f'_{ci}

where

f_{cp} = permissible compressive concrete stress

f'_c = compressive strength of concrete

f'_{ci} = compressive strength of concrete at time of initial prestress

A'_b = maximum area of the portion of the concrete anchorage surface that is geometrically similar to and concentric with the area of the anchorage

A_b = bearing area of the anchorage

As used in the above equation f_{cp} is the average bearing stress, P/A, in the concrete computed by dividing the force P of the prestressing steel by the net projected area, A_b, between the concrete and the bearing plate or other structural element of the anchorage which has the function of transferring the force to the concrete.

Special reinforcement, required for the performance of the anchorage, shall be indicated by the tendon supplier.

3.1.8 Static and Dynamic Test Requirements

(1) **Static Tests** — The test assembly shall consist of standard production quality components and the tendons shall be at least 10 ft. long. The test assembly shall be tested in a manner to allow accurate determination of the yield strength, ultimate strength and percent elongation of the complete tendon to insure compliance with this specification. The specimen used for the static test need not be one that has been subjected to dynamic loading.

The specifier may not wish to require static and dynamic testing for his project since these tests are obviously expensive. In lieu of test, data from prior tests could be submitted (Section 3.1.9 *(2), or the provisions of Sections 3.1.9 (3) may be satisfactory).*

The static test is essentially a tensile *test of* an *assembled tendon. The test specimen should be assembled using standard production quality components, which are sampled at random.*

It is also desirable that the static test represent as closely as possible actual conditions under *which a tendon has to perform in a structure. Thus, the test should include a bearing plate embedded in concrete or, in systems using other means to transmit the prestressing force to the concrete, duplicate the actual working conditions of the anchorage in its concrete environment.*

It is necessary that the test allows direct and accurate determination of all results expected from the testing. The failure mode at specimen tendon capacity should be the same as that for which the test is designed, unless a series of limiting tests are conducted for the different modes of failure a tendon assembly can undergo. Interpolation between test values is acceptable but extrapolation beyond them is not.

(2) **Dynamic Tests —** Dynamic tests shall be performed on representative tendon specimens. In the first test, the tendon shall withstand, without failure, 500,000 cycles from 60 percent to 66 percent of its minimum specified ultimate strength. In the second test, the tendon shall withstand, without failure, 50 cycles from 40 percent to 80 percent of its minimum specified ultimate strength. The period of each cycle involves the change from the lower stress level to the upper stress level and back to the lower. The specimen used for the second dynamic test need not be the same used for the first dynamic test. Systems utilizing multiple strands, wires, or bars may be tested utilizing a test tendon. The test tendon shall duplicate the behavior of the full size tendon and generally shall not have less than 10 percent of the capacity of the full size tendon. Dynamic tests are not required on bonded tendons, unless the anchorage is located or used in such manner that repeated load applications can be expected on the anchorage.

Dynamic tests are conducted to investigate or prove the fatigue behavior of a tendon assembly. Since only unbonded tendons experience changes of stress'levels over their entire length, they are the only tendons for which dynamic tests are required. In bonded tendons, only local changes in stress can occur, and it is required only to know the fatigue properties of the prestressing steel in question. It is recommended that anchorages be located in zones where stress changes either do not exist, or are small, for example, at or near the neutral axis in flexural members.

The test as outlined is not intended to reproduce an actual case of fatigue load expected of any specific structural element. Nevertheless, a standard test, indicating the capability of the tendon to resist cyclic loading had to be established. The load limits of 60 to 66percentof ultimate tensile strength are considered representative for prestressed concrete flexural members. The 500,000 cycles was established as a minimum value to represent the effect of fatigue.

With the advent of very large post-tensioning tendons, it was recognized that the availability of testing machines capable of performing a true fatigue test of full capacity tendons was very uncertain. The main feature in a dynamic test is considered to be the possible effect of the anchorage in reducing the fatigue strength of the prestressing steel itself. The design of the different anchorage components is a separate mechanical problem, which can be approached in an analytical *manner. For this reason, a reduced capacity fatigue test is considered acceptable. The minimum 10 percent capacity requirement was included as a practical limit for testing large capacity tendons.*

3.1.9 Specification Compliance Requirements

(1) **Conformance Testing-The** adequacy of a tendon system shall be confirmed by satisfactory static and dynamic conformance tests in accordance with the minimum requirements outlined in Section 3.1.8.

(2) Compliance — Data shall be submitted to show compliance with the provisions of Section 3.1.8.

(3) Prestressing Steel — Certified mill test results and typical stress-strain curves shall be submitted when requested. The typical stress-strain curve shall be obtained by approved standard practices. For materials not covered by Sections 3.1.4 (1), 3.1.4 (2), or 3.1.4 (3), the guaranteed tensile strength, yield strength, elongation, composition and other pertinent data shall be submitted. In this case, samples from each heat (or lot in case of strands), properly marked, shall be provided for verification of prestressing steel quality.

3.2 SPECIFICATION FOR UNBONDED SINGLE STRAND TENDONS

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3.2.1 General Considerations

3.2.1.1 Scope

This specification provides detailed minimum requirements for single strand unbonded post-tensioning tendons for incorporation in prestressed concrete structures. Specifications are presented for tendons in normal (non-corrosive) environments, and for tendons in aggressive (corrosive) environments. The provisions of this specification shall also govern for nonstructural applications. The system and text presented herein require that each user make the changes, additions or deletions necessary to adapt these specifications to the specific job conditions and specification format.

This specification is in addition to and is intended to be used in conjunction with the "Guide Specification for Post-Tensioning Materials" presented in Section 3.1. These specifications shall supercede or govern in any case of conflicting provisions with the "Guide Specification for Post-Tensioning Materials."

Unbonded single strand tendons fabricated by methods other than these specifically itemized in these specifications may be used when the

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The intent of this specification is to provide detailed specifications for all common uses of unbonded post-tensioning tendons. There are, however, certain special structures or applications which either because of their service requirements or structural behavior might impose requirements on the post-tensioning system which exceed the minium requirements of these standard specifications. It is recommended that, in such cases, a special set of specifications be written.

S truc tures exposed to corrosive environments include all structures subjected to direct or indirect applications of de-icer chemicals, seawater, brackish water, or spray from these sources, and possibly some structures in the immediate vicinity of seacoasts exposed to salt air. Nearly all enclosed buildings (office buildings, apartment buildings, warrehouses, manufacturing facilities) are considered to be normal (non-corrosive) environments. Exposed structures (such as parking structures) in areas with very little or no snow would also be considered suitable applications for the specification for tendons in normal (non-corrosive) environments.

Non-structural applications might include topping slabs, waterproofing slabs on fill, and post-tensioning used only for control of cracking or deflection. For non-flexural or membrane type structures primarily under tensile forces, the provisions, where appropriate, are intended to apply.

The "Guide Specification for Post- Tensioning Materials" includes strength requirements for anchorages and static and dynamic test requirements for unbonded tendons which are not reproduced in this specification. These requirements are considered applicable to unbonded tendons in accordance with the provisions of this section.

supplier provides conclusive test data substantiating that all characteristics of the unbonded tendons, in particular, the corrosion resistive characteristics, are comparable to the characteristics of tendons fabricated in accordance with these specifications.

3.2.1.2 Definitions

(1) **Tendon-The** complete assembly consisting of anchorage and prestressing steel with sheathing when required. The tendon imparts prestressing forces to theconcrete.

(2) Bonded Tendons — Tendons which are bonded to the concrete through grouting or other approved means, and therefore are not free to move relative to the concrete.

(3) Unbonded Tendons — Tendons in which the prestressing steel is permanently free to move relative to the concrete to which they are applying their prestressing forces.

(4) Anchorage — The means by which the prestressing force is permanently transmitted from the prestressing steel to the concrete.

(5) Prestressing Steel — That element of a post-tensioning tendon which is elongated and anchored to provide the necessary permanent prestressing force.

(6) Coating — Material used to protect against corrosion and/or lubricate the prestressing steel.

(7) Sheathing — Enclosure around the prestressing steel to avoid temporary or permanent bond between the prestressing steel and the surrounding concrete.

Different requirements are imposed upon sheathings for bonded and unbonded tendons. In unbonded tendons, the sheathing does not transmit any bond stresses from the prestressing steel to the concrete and therefore has to assure the freedom of movement of the prestressing steel and form an adequate cover over the coated tendon. In bonded tendons, bond stresses will be transmitted through the sheathing, and it must be of such material and/or configuration to effective/y allow this stress transfer.

The void in the concrete in which the tendon is to be located can also bepre-formed (e.g. by inflatable and removable tubes) and the tendon pulled through subsequently. With pre-formed voids no additional sheathing will be required.

(8) Coupling -The means by which the prestressing force may be transmitted from one partial-length prestressing tendon to another.

3.2.2 Prestressing Steel

(1) Prestressing steel used in single strand unbonded post-tensioning tendons shall conform to ASTM A-416 Grade 250 or 270k, Regular Stress Relieved or Low Relaxation Type.

(2) Strand not specifically itemized in ASTM A-416 may be used provided it conforms to the minimum requirements of this specification and has no properties which make it less satisfactory than those listed in ASTM A-416.

Provision should be made for new steels which would include new sizes, improved characteristics of relaxation, or improved mechanical properties. However, use of prestressing steels not covered by AS TM Specifications should be permitted only when the supplier provides conclusive test data substan tia ting that all characteristics of the material are comparable to the properties of steels conforming to the ASTM Specifications. In particular, the stress corrosion characteristics of steels produced by quench and temper heat treatments should be carefully evaluated. Relaxation properties of new steels should be based on a minimum test period of 1000 hours.

(3) Certified Mill Test Reports shall be furnished upon request for each coil or pack of strand, containing as a minimum the following test information:

- Heat number and identification.
- Standard chemical analysis for heat of steel.
- Ultimate tensile strength.
- Yield strength at 1% extension under load.
- Elongation at failure.
- Modulus of elasticity.
- Diameter and net area of the strand.
- Type of material (stress-relieved or low relaxation).

(4) Relaxation losses for low relaxation type material shall be based on relaxation tests of representative samples for a period of 1000 hours, when tested at 70°F and stressed initially to not less than 70 percent of the minimum guaranteed breaking strength of the strand.

It is not practical to run 7000 hour relaxation tests on each pack of strand. For qualitative identification of low relaxation strand, a short term relaxation test of 30 minutes to ten hours will suffice. A maximum relaxation percentage of 1.2 percent has been suggested for identification of low relaxàtion steel in the 30 minute

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The tests shall be in accordance with ASTM A-416, and ASTM E-328.

test. However, a 30minute test willnotprovide an accurate indication of the ultimate relaxation value. Very precise testing procedures are required with mechanical (not hydraulic) equipment in a room with rigid temperature control to accurately evaluate steel relaxation losses.

(5) Low relaxation strand shall be provided with a mill applied continuous permanent physical marking to permit field identification.

Low relaxation strand is identified by producers in packs by tags, pack markings and other means, as well as by mill certificates. Identification of low relaxation material in individual tendons at the jobsite requires the additional marking required by this specification provision. Such marking is considered necessary to avoid inadvertent reduction of structural capacity through use of stress-relieved strand in place of low relaxation strand. To provide manufacturer's time to develop a process to perform this marking, this specification should not be imposed until January 1, 7986.

(6) The material shall be packaged at the source in a manner which prevents physical damage to the strand during transportation and protects the material from deleterious corrosion during transit and storage.

For corrosion protection of strand packs, they are usually wrapped in paper impregnated with vapor phase inhibitor powder.

3.2.3 Anchorages and Couplings

(1) Tendon anchorages and couplings shall be designed to develop the static and dynamic strength requirements of Section 3.1.6 (1) and Sections3.1.8 (1) and (2) of the PTI "Guide Specifications for Post-Tensioning Materials."

Castings shall be nonporous and free of sand, blow holes, voids and other defects.

Experience with anchor castings indicates that satisfactory performance is achieved with a finish of the anchorage wedge seating area not exceeding a microfinish of 125 for stressing end anchorages, and not exceeding a microfinish of 250 for fixed anchorages.

(2) The average compressive concrete bearing stress of anchorages shall not exceed the limits set forth in Section 3.1.7.

Oversized anchorages may be used to allow for early stressing. However, the increase in time dependent prestress losses due to concrete creep and shrinkage must be taken into consideration.

(3) For wedge type anchorages, the wedge grippers shall be designed to preclude premature failure of the prestressing steel due to notch or pinching effects under the static and/or dynamic test load conditions stipulated under Section 3.2.3 (1), for both

For fixed anchorages, the finish of the outer surface of the wedge that bears against the wedge seat may have a microfinish of up to 250. This microfinish of 250 for fixed anchorage wedges and anchor (wedge) seating areas in anchor castings is intended to safeguard stress relieved and low relaxation prestressing steel materials.

against loosening of wedges during shipment of tendons. The additional roughness can be accommodated for fixed anchorages since the anchorage is attached under plant conditions in accordance with Section 3.2.6.4 (2).

(4) Couplings shall be used only at locations specifically indicated on the contract documents or as approved.

Couplings shall be coated with the same corrosion preventative coating used on the strand and shall be enclosed in sleeves which permit necessary movements during stressing.

(5) Anchorages intended for use in corrosive environments shall include design features permitting a watertight connection of the sheathing to the anchorage, and watertight closing of the wedge cavity, for stressing and non-stressing (fixed) anchorages. Intermediate stressing anchorages shall be designed to permit complete watertight encapsulation of the prestressing steel.

Sufficient corrosion protection of the anchor casting is normally provided by the alkalinity of the bonded concrete encasement. Past experience and research indicate that anchor castings are much less senstive than the strand from the standpoint of the need for corrosion protection, and that additional corrosion protection of the anchor casting is not necessary. For any application where additional corrosion protection of anchor castings is considered essential, it may be obtained by various means.

3.2.4 Sheathing

(1) The tendon sheathing for unbonded single strand tendons shall be made of a material with the following properties:

- Sufficient strength to withstand unrepairable damage during fabrication, transport, installation, concrete placement and tensioning.
- Watertightness over the entire sheathing length.
- Chemical stability, withoutembrittlement or softening over the anticipated exposure temperature range and the service life of the structure.
- Non reactive with concrete, steel and the tendon corrosion preventive coating.

The sheathing may be produced by either an extrusion process, heat sealing process, or any other process which assures a watertight enclosure over the tendon.

(2) Minimum thickness of the sheathing used in normal (non-corrosive) environments shall not be less than 0.025 inches for medium or high density polyethylene or polypropylene. Sheathing thickness for tendons used in corrosive environments shall not be less than 0.040 inches for medium or high density polyethylene or polypropylene.

The increased sheathing thickness specified for corrosive environments is intended to provide increased resistance to damage during construction.

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(3) The sheathing shall have an inside diameter at least 0.010 inches greater than the maximum diameter of the strand.	*Is is preferable that the sheathing provide a smooth circular outside surface. The sheathing should not visibly reveal the lay of the strand.*
(4) For applications in corrosive environments, the sheathing shall be connected to all stressing, intermediate and fixed anchorages in a watertight fashion, thus providing a complete encapsulation of the prestressing steel.	*A watertight connection may be achieved by either using special connector pieces, or by spirally wrapping polyethylene adhesive tape in a double layer over the coated strand, connecting the sheathing to the anchor.*

3.2.5 Corrosion Preventive Coating

(1) The corrosion preventive coating material shall have the following properties:

- Provide corrosion protection to the prestressing steel.
- Provide lubrication between the strand and the sheathing.
- Resist flow from the sheathing within the anticipated temperature range of exposure.
- Provide a continuous non-brittle film at the lowest anticipated temperature of exposure.
- Chemically stable and non-reactive with the prestressing steel, the sheathing material, and the concrete.

(2) The film shall be an organic coating with appropriate polar, moisture displacing and corrosion preventive additives.

(3) Minimum weight of coating material on the prestressing strand shall be not less than 2.5 pounds of coating material per 100 feet of 0.5 inch diameter strand, and 3.0 pounds of coating material per 100 feet of 0.6 inch diameter strand. The amount of coating material used shall be sufficient to ensure essentially complete filling the annular space between the strand and the sheathing. The coating shall extend over the entire tendon length.	*The corrosion test in Table 3.2.1 is based on a coating thickness of 0.005 inches. The quantities of coating material specified provide a minimum coating over the crests of the strand of approximate/y 0.005 inches.*

(4) Test results in accordance with Table 3.2.1 shall be provided for the corrosion preventive coating material.

TABLE 3.2.1

PERFORMANCE SPECIFICATION FOR CORROSION PREVENTIVE COATING

TEST	TEST METHOD	ACCEPTANCE CRITERIA
1. Dropping Point °F ("C)	ASTM D-566 or ASTM D-2265	Minimum 300 (148.9)
2. Oil Separation @ 160°F (71.1°C) % by weight	FTMS 791 B Method 321.2	Maximum 0.5
3. Water, % Maximum	ASTM D-95	0.1
4. Flash Point, °F (°C) (Refers to oil component)	ASTM D-92	Minimum 300 (148.9)
5. Corrosion Test 5% Salt Fog @ 100° F (37.8°C) 5 mils, minimum hours (Q Panel Type S)	ASTM B-l 17	For normal environments: Rust grade 7 or better after 720 hours of exposure according to ASTM D-610. For corrosive environments: Rust grade 7 or better after 1000 hours of exposure according to ASTM D-610. (1)
6. Water Soluble Ions (2)		
a. Chlorides, ppm maximum	ASTM D-512	10
b. Nitrates, ppm maximum	ASTM D-992	10
c. Sulfides, ppm maximum	APHA 427D (15th Ed.)	10
7. Soak Test 5% Salt Fog at 100°F (37.8°C) 5 mils coating, Q panels, type S. Immerse panels 50% in a 5% salt solution and expose to salt fog	ASTM B-l 17 (Modified)	No emulsification of the coating after 720 hours of exposure.
8. Campatibility with Sheathing		
a. Hardness and volume change of polymer after exposure to grease, 40 days @150° F.	ASTM D-4289	Permissiblechange in hardness 15% Permissible change in volume 10%
b. Tensile strength change of polymer after exposure to grease, 40 days @ 150" F.	ASTM D-638	Permissible change in tensile strength 30%

(1) Extension of exposure time to 1000 hours for greases used in corrosive environments requires use of more or better corrosion inhibiting additives.

(2) **Procedure:** The inside (bottom and sides) of a 1 L Pyrex breaker, approximate O.D. 105 mm, height 145 mm, is thoroughly coated with 100 ± 10 g of corrosion preventive coating material. The coated breaker is filled with approximately 900 cc of distilled water and heated in an oven at a controlled temperature of 100° F ± 2" F for 4 hours. The water extraction is tested by the noted test procedures for the appropriate water soluble ions. Results are reported as ppm in the extracted water.

COMMENTARY TO TABLE 3.2.1

The tests for corrosion prevention coatings presented in Table 3.2.1 are considered to be base line 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 case other and more comprehensive tests may be necessary to ascertain their adequacy.

TEST #1 and #2:

Limiting the Dropping Point to 300°F minimum is intended to ensure product stability under elevated temperatures, possible during tendon fabrication and installation. Together with Test #2, the bleeding of the lighter components from the coating is minimized.

TEST #3:

Water content is limited to exclude the presence of free water in the coating material.

TEST #4:

This test refers to the oil component in the coating material. Too low a flash point indicates higher content of volatile derivatives, which affect the long term stability and change of consistency of the coating material.

TEST #5:

This test provides a method to determine the effectiveness of the corrosion preventive properties of the coating. The method is a standard test used for corrosion preventive coatings such as paints, etc. The acceptance criteria of grade 7 or better (according to ASTM D-610) after 720 hours of exposure requires that only 0.3 percent of the area exposed can have indications of corrosion. (See *Fig. 3.2.1* — Examples of Area percentages from ASTM D-610). The test is conducted on 3 x 6 inch steel panels with a coating thickness of 0.005 inches. When determining the percent of area corroded, only the area inside 1/4 inch from the edges of the panel is evaluated.

TEST #6:

Water soluble ions known to cause corrosion are limited by this requirement.

TEST #7:

The Soak Test is designed to determine the ability of the coating to provide corrosion protection after having been exposed to standing water for a period of time. Certain coatings will absorb water to the extent that they will emulsify and break down the barrier against moisture reaching the steel. This test will guard against inadvertent use of such coatings.

TEST #8:

Certain pertroleum derivatives react with polyethylene or polypropylene, changing its physical properties to the point where they are no longer usable as sheathing materials. This test is required to preclude the use of coatings with such derivatives.

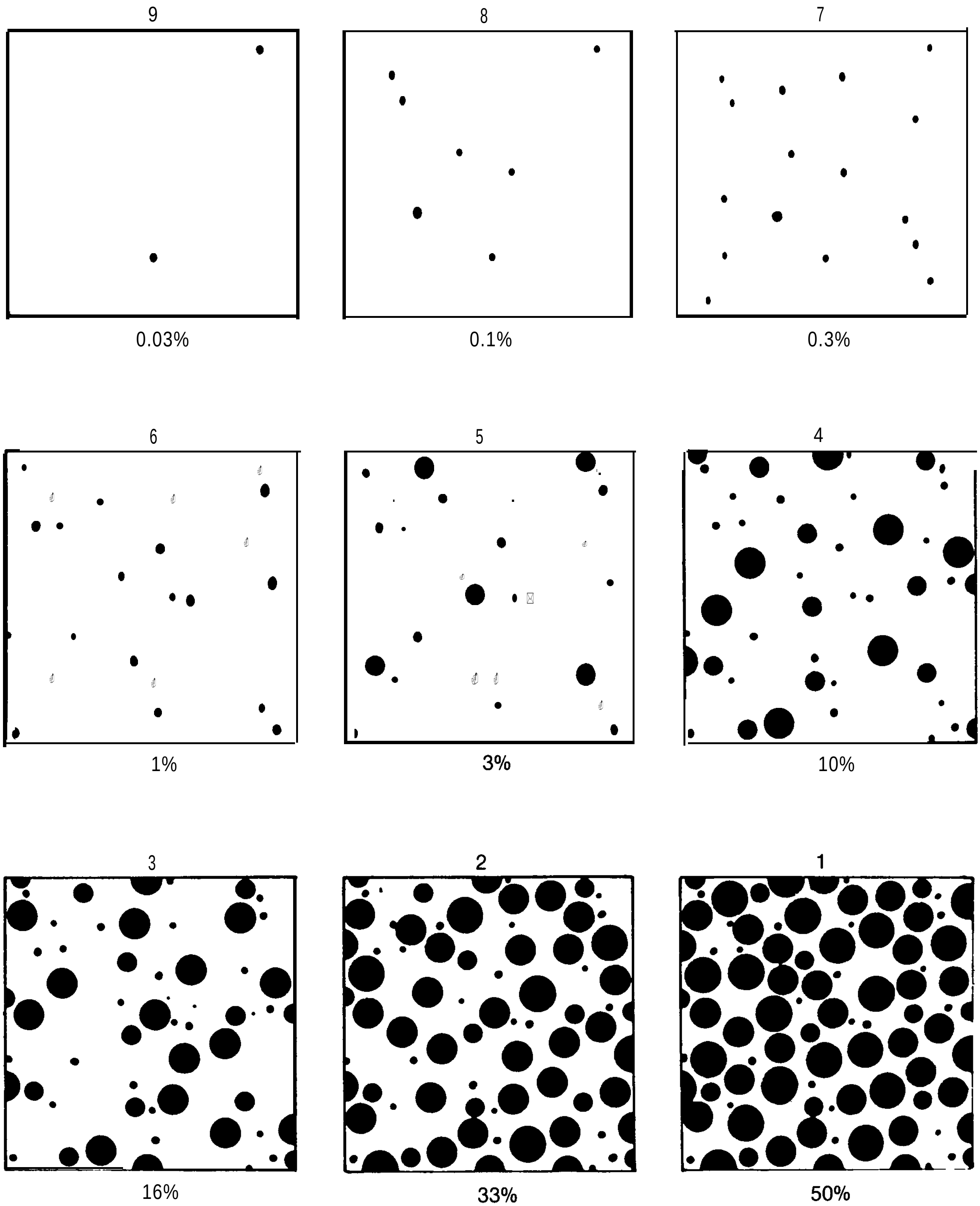

Fig. 3.2.1 — Examples of Area Percentages. .

(Reprinted, with permission, from ASTM Standard D610)

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3.2.6 Installation Requirements

3.2.6.1 General

(1) Prestressing tendons shall be firmly supported at intervals not exceeding 4 feet to prevent displacement during concrete placement. Placing tolerances shall be in accordance with theapplicable Construction Specifications.

Tendons should be attached to supporting chairs or reinforcement in such a way that the sheathing is not damaged.

Vertical deviations in tendon location should be kept to 1/4 in. for slab thickness dimensions less than 8 in., 3/8 in. in concrete with dimensions over 8 in. but not over 2 ft., and 1/2 in. inconcrete with dimensions over 2 ft. These tolerances should be considered in establishing tendon cover dimensions, particularly in applications exposed to deicer chemicals or salt water environments where use of additional cover is recommended.

Horizontal plane deviations which may be necessary to avoid openings, ducts, chases, inserts, etc., should have a radius of curvature of not less than 21 ft. Slab behavior is relatively insensitive to horizontal location of tendons.

(2) The tendons shall not be exposed to excessive temperatures, welding sparks or electric ground currents.

Excessive temperatures are defined as temperatures which deleteriously affect the prestressing steel, anchorages, protective coating, or sheathing material.

3.2.6.2 Stressing Anchorages

(1) Stressing anchorages shall be installed perpendicular to the tendon axis. Curvature in the tendon profile shall preferably not be closer than three feet from the stressing anchorage.

When tendon curvature starts closer than 3 feet from a stressing location, special attention must be given to ram and wedge centering during the stressing operation. With sharp curvatures at the anchorages, local friction will develop adversely affecting the tendon efficiency and elongation.

(2) Stressing anchorages shall be attached to the bulkhead forms by either bolts, nails, or threaded pocket former fittings. The connections shall be sufficiently rigid to avoid accidental loosening due to construction traffic or during concrete placement. Minimum concrete cover for the anchorage shall not be less than the minimum cover to the reinforcement at other locations in the structure.

(3) Pocket formers used to provide a void form at stressing and intermediate stressing anchorages shall positively preclude intrusion of concrete or cement paste into the wedge cavity during concrete placement. The depth of the pocket former from the edge of the concrete to the face of the anchorage shall not be less than 1-1/2 inches for normal environments nor 2 inches for corrosive environments.

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3.2.6.3 Intermediate Anchorages

(1) Intermediate anchorages may be installed either embedded in concrete or bearing against the hardened concrete at the construction joint. In the latter case, the anchorage shall have a flat bearing side and the concrete bearing area shall be smooth and without ridges.

(2) When placing intermediate anchorages against already hardened concrete, special attention must be paid to the perpendicularity between the bulkhead form and tendon during tendon placement. This type of anchorage is not recommended for use in corrosive environments.

(3) Minimum cover requirements of section 3.2.6.2 (2) apply to intermediate anchorages.

3.2.6.4 Fixed Anchorages

(1) Fixed end anchorages shall be installed on the tendon at the suppliers plant prior to shipment to the job site.

(2) For wedge type anchorages, the fixed end wedges shall be seated, with a load of not more than 80% of the minimum ultimate tensile strength of the tendon for stress relieved strand or for low relaxation strand. The seating load shall be sufficient to ensure adequate capacity of non-stressing anchorages.

Due to variations in equipment and materials, a tolerance of plus or minus 3 percent is recommended on the load for seating fixed end wedges.

(3) Fixed end anchorages shall be placed in the formwork at the locations shown on the placing drawings, and securely fastened to the reinforcing steel. Minimum cover requirements of Section 3.2.6.2 (2) apply to fixed end anchorages.

(4) Fixed end anchorages intended for use in corrosive environments shall be closed or capped at the wedge cavity side with a watertight cover. This cover shall preferably be shop installed, after filling the void around the wedge grips with corrosion preventive coating material comparable to that used as a corrosion preventive coating over the length of the tendon (see Table 3.2.1).

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3.2.6.5 Sheathing Inspection

After installing the tendons in the forms and prior to concrete casting, the sheathing shall be inspected for possible damage.

In corrosive environments, damaged areas shall be repaired by restoring the corrosion preventive coating in the damaged area, and repairing the sheathing. Repairs of sheathing shall be watertight, and must be approved by the engineer of record.

Tape used to repair sheathing shall be adhesive moisture proof tape, spirally wrapped around the tendon to provide at least two layers of tape.

3.2.7 Tendon Stressing

(1) Hydraulic stressing rams used to stress unbonded single strand tendons shall be equipped with stressing grippers which will not notch the strand more severely than normal anchoring wedges.

(2) Stressing rams and gauges shall individually be identified and calibrated against known standards at intervals not exceeding six months. Calibration certificates for each jack used shall be available upon request.

(3) Elongation measurements shall be made at each stressing location to verify that the tendon force has been properly achieved. Measured elongations shall agree with calculated elongations within ±5%. Discrepancies exceeding ±5% shall be resolved with the designer/engineer of record.

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For tendons used in non-corrosive environments small damaged areas in the tendon sheathing may be permitted without repair.

It is preferable to calibrate rams and gauges together as a unit. However, gauges may be calibrated against a master gauge of known accuracy, provided the rams are calibrated, against the same master gauge.

Correlation of calculated and measured elongations within a ±5% tolerance requires that the elongation calculations be based on the correct modulus of elasticity and area of steel of the tendon or tendons under consideration. Further, the friction and wobble coefficients used are average valuesand may very slightly from project to project. Variations in calculated and measured elongation values in excess of 5% should be evaluated from the standpoint of the number of tendons involved and thestructural significance of the variation. Excess elongation resulting from a friction coefficient smaller than that assumed in calculations is usually not a structural problem.

(4) Stressing records shall be filled out during the tensioning operation, with the following data recorded as a minimum:
- Tendon mark or identification.
- Required elongation.
- Gauge pressure to achieve required elongation.
- Actual elongation achieved.
- Actual gauge pressure.
- Date of Stressing operation.
- Signature of the stressing operator or inspector.
- Serial or identification number of jacking equipment.

Stressing records shall be turned over to the owner or their representative for verification and safekeeping.

3.2.8 Tendon Finishing

(1) Trimming of excess tendon length. As soon as possible after tendon tensioning and satisfactory check of elongation, the excess tendon length shall be cut. The tendon length protruding beyond the wedges after cutting shall be between 0.75 and 1.25 inches.

The tendon may be cut by means of either oxyacetylene cutting, abrasive wheel or hydraulic shears. In case of oxyacetylene cutting of the tendon, care shall be taken to avoid directing the flame toward the wedges.

(2) Stressing pockets shall be filled with non shrink mortar as soon as practical after tendon stressing and cutting. Under no circumstances shall the grout or mortar used for pocket filling contain chlorides or other chemicals known to be deleterious to the prestressing steel.

For tendons used in corrosive environments, the exposed strand and wedge areas shall be coated with tendon coating material comparable to that used over the length of the tendon and a watertight cap shall be applied over the coated area. Prior to installing the pocket mortar, the inside concrete surfaces of the pocket shall be coated or sprayed with a resin bonding agent.

It is recommended that stressing pockets be filled within 15 days after removal of the stressing tails. Earlier filling of stressing pockets is desirable when practical.

3.3 RECOMMENDED PRACTICE FOR GROUTING of POST-TENSIONED PRESTRESSED CONCRETE

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3.3.1 General

3.3.1.1. Scope and purpose.

(1) These recommendations cover the grouting of post-tensioning tendons of prestressed concrete members.

(2) The purpose of grouting is to provide permanent protection to the post-tensioning steel and to develop bond between the prestressing steel and the surroundng concrete.

These procedures also apply to grouting of rock and soil anchors. However, since the hardened grout in rock and soil anchor applications sustains the fullpost-tensioning force, more detailed consideration must be given to grout strength and injection procedures.

3.3.1.2 Definition of Terms.

All terms and symbols shall be as defined below and in "Guide Specifications for Post-Tensioning Materials", Section 3.1.3.

(1) **Admixture** — Any material added to the grout other than portland cement and water.

(2) Duct — The hole or void provided in the concrete for the post-tensioning tendon.

(3) **Grout-A** mixture of cement and water with or without admixtures.

Although sand has not been used in grouting practices in the United States, it may have advantages in tendons with large void areas. Fly ash and pozzolans are occasionally used as filler material in the United States.

(4) Grout Opening or Vent-An inlet, outlet, or vent in the duct for grout, water or air.

(5) Post-Tensioning-The method of prestressing concrete in which the tendon is stressed after the concrete has reached a specified strength.

(6) Post-Tensioning Tendon — The complete assembly consisting of anchorage and prestressing steel with sheathing when required. The tendon imparts prestressing forces to the concrete.

(7) Prestressing Steel-That element of a post-tensioning tendon which is elongated and anchored to provide the necessary permanent prestressing force.

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3.3.2 Materials	
(1) **Portland Cement —** Portland cement should conform to one of the following: Specifications for portland cement -ASTM C150, Type I, II or III.	*Normally, Type III cement is only used for cold weather grouting. Trial mixes are necessary to determine an appropriate mix design using Type III cement.*
Cement used for grouting should be fresh and should not contain any lumps or other indication of hydration or "pack set".	
(2) Water-The water used in the grout should be potable, clean and free of injurious quantities of substances known to be harmful to portland cement, or prestressing steel.	*Known harmful substances are chlorides, florides, sulphites and nitrates.*
(3) Admixtures — Admixtures, if used, should impart the properties of low water content, good flowability, minimum bleed, and expansion if desired. Its formulation should contain no chemicals in quantities that may have harmful effect on the prestressing steel or cement. Admixtures containing chlorides (as CL in excess of 0.5% by weight of admixture, assuming 1 pound of admixture per sack of cement), florides, sulphites and nitrates should not be used.	*Admixtures common/y used to provide expansion of the grout may also reduce the water requirement, or improve flowability at a given water content, and retard set. Such admixtures are often used. However, research on basically horizontal tendons in semi-rigid ducts indicates that satisfactory grout quality may be achieved without admixtures.*
Aluminum powder of the proper fineness and quantity or other approved gas evolving material which is well dispersed through the other admixture may be used to obtain 5% to 10% unrestrained expansion of the grout.	*Current international standards suggest that bleeding may be measured in a metal or glass cylinder with an internal diameter of approximately 4 inches, with a height of grout of approximately 4 inches. However, recent research in the United States indicates that more representative test results may be achieved using a grout specimen of approximately 20 inch height and 1 1/4 inch diameter. During the test, the container should be covered to prevent evaporation. It is suggested that the following approximate limits on bleeding be used to evaluate the acceptability of the grout: 2% of the volume 3 hours after mixing; a maximum of 4%. In addition, the separated water should be absorbed after 24 hours.*
All admixtures should be used in accordance with the instructions of the manufacturer.	
3.3.3 Ducts	
(1) **Forming —** (a) **Formed Ducts —** Ducts formed by sheath left in place should be of a type that would not permit the entrance of cement paste. They should transfer bond stresses as required and should retain shape under the weight of the concrete. Metallic sheaths should be of a ferrous metal, and they may be galvanized.	*Materials commonly used for formed ducts are 22 to 28 guage galvanized or bright spirally wound or longitudinally seamed steel strip with flexible or semi-rigid seams.*
(b) Cored Ducts — Cored ducts should be formed with no constrictions which would tend to block the passage of grout. All coring material should be removed.	
(2) Grout Openings or Vents — All ducts should have grout openings at both ends.	

For draped cables all high points should have a grout vent except where cable curvature is small, such as in continuous slabs. Grout vents or drain holes should be provided at low points if the tendon is to be placed, stressed and grouted in a freezing climate. All grout openings or vents should include provisions for preventing grout leakage.

Research indicates that high point grout vents may be eliminated for bridge tendons up to 400 ft. long in semi-rigid conduit.
Material used for grout vents or drain holes may be either plastic or ferrous metal.

Grout vent details for rock and soil anchors require special considerations particular to the application.

(3) **Duct Size** — For tendons made up of a plurality of wires, bars, or strands, duct area should be at least twice the net area of the prestressing steel.

For tendons made up of a single wire, bar or strand, the duct diameter should be at least 1/4 inch larger than the nominal diameter of the wire, bar or strand.

(4) **Placement of Ducts** — After placing of ducts, reinforcement and forming is complete, an inspection should be made to locate possible duct damage. Ducts should be securely fastened at close enough intervals to avoid displacement during concreting.

All holes or openings in the duct must be repaired prior to concrete placing.

Grout openings and vents must be securely anchored to the duct and to either the forms or to reinforcing steel to prevent displacement during concrete placing operations.

There are two methods of placing tendons. First, preassembled tendons may be placed as a unit prior to placing concrete. Second, bearing pla tes and duct sheathing may be installed prior to placing the concrete, and then after concreting, theprestressing steel and anchorages are installed. Ties for pre-placed tendons must be adequate to support the tendon weight. When only the duct is placed prior to concreting, ties must resist bouyancy forces.

3.3.4 Equipment

(1) The grouting equipment should include a mixer capable of continuous mechanical mixing which will produce a grout free of lumps and undispersed cement. The equipment should be able to pump the mixed grout in a manner which will comply with all provisions of this recommended practice.

(2) Accessory equipment which will provide for accurate solid and liquid measures should be provided to batch all materials.

(3) The pump should be a positive displacement type and be able to produce an outlet pressure of at least 150 psig. The pump should have seals adequate to prevent introduction of oil, air or other foreign substance into the grout, and to prevent loss of grout or water.

(4) A pressure gauge having a full scale reading of no greater than 300 psi should be placed

It is suggested that standby water flushing equipment should be available where difficult grouting conditions exist. This equipment should be in addition to the grouting equipment. The standby water flushing equipment should utilize a different power source than the grouting equipment, have sufficient capacity to flush out any partially grouted enclosures if necessary due to blockage or breakdown of grouting equipment, and should be capable of developing a pressure of at least 300 psig.

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at some point in the grout line between the pump outlet and the duct inlet.	
(5) The grouting equipment should contain a screen having clear openings of 0.125 inch maximum size to screen the grout prior to its introduction into the grout pump. If a grout with a thixotropic additive is used, a screen opening of 3/16 inch is satisfactory. This screen should be easily accessible for inspection and cleaning.	*A thixotropic grout undergoes marked changes in fluidity depending on whether the grout is in motion or quiescent. This property is produced by additives.*
(6) The grouting equipment should utilize gravity feed to the pump inlet from a hopper attached to and directly over it. The hopper must be kept at least partially full of grout at all times during the pumping operation to prevent air from being drawn into the post-tensioning duct.	
(7) Under normal conditions, the grouting equipment should be capable of continuously grouting the largest tendon on the project in no more than 20 minutes.	
3.3.5 Mixing of the Grout	
(1) Water should be added to the mixer first, followed by portland cement and admixture, or as required by the admixture manufacturer.	
(2) Mixing'should be of such duration as to obtain a uniform thoroughly blended grout, without excessive temperature increase or loss of expansive properties of the admixture. The grout should be continuously agitated until it is pumped.	*Equipment currently in use normally requires 1-1/2 to 3minutes tosatisfactorilymix thegrout.*
(3) Water should not be added to increase grout flowability which has been decreased by delayed use of the grout.	
(4) Proportions of materials should be based on tests made on the grout before grouting is begun, or may be selected based on prior documented experience with similar materials and equipment and under comparable field conditions (weather, temperature, etc.). The water content shall be the minimum necessary for proper placement, and when Type I or Type II cement is used should not exceed a water-cement ratio of 0.45 (approximately 5 gallons of water per sack of cement). The water content required for Type III cement should e established for a particular brand based on tests.	*Hardened grout made in accordance with this specification at a temperature of 65 degrees F. and a relative humidity of approximately 70% will produce 28 day compressive strengths of about 4000 psi when cured under confined conditions.*

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The pumpability of the grout may be determined by the Engineer in accordance with the U.S. Corps of Engineers Methods CRD-C79 (See Section 3.3.8). When this method is used, the efflux time of the grout sample immediately after the mixing should not be less than 11 seconds. The flow cone test does not apply to grout which incorporates a thixotropic additive.

3.3.6 Grouting

3.3.6.1 Preparation of the duct.

(1) Flushing of metal ducts should be optional with the post-tensioning contractor.

Historically, flushing has been used to clear the duct of foreign materials, and to wet the duct and tendon surfaces to improve the groutability. When tendons are flushed, the water may be removed by oil-free air, or it may be displaced by the grout. In recent years, grouting experiences with semi-rigid conduit and prestressing steel placed after concreting indicate that f/using is not necessary and may be undesirable for large tendons since it is difficult to remove the water from the duct.

(2) Ducts with concrete walls (cored ducts) should be flushed to ensure that the concrete is thoroughly wetted.

(3) Water used for flushing ducts may contain slack lime (calcium hydroxide) or quicklime (calcium oxide) in the amount of 0.1 pounds per gallon.

3.3.6.2 Injection of the grout.

(1) All grout and high point vent openings should be open when grouting starts. Grout should be allowed to flow from the first vent after the inlet pipe until any residual flushing water or entrapped air has been removed, at which time the vent should be capped or otherwise closed. Remaining vents should be closed in squence in the same manner.

(2) The pumping pressure at the tendon inlet should not exceed 250 psig.

When pumping grout, pressures in excess of 250 psi result in separation of water and cement, which may cause a blockage. Excessive pressures could also result in cracking or damage to the structural element. It is therefore advisable to keep grout pumping pressures under this level. This can be done by visually monitoring the pressure gauge, or by equipment which includes automatic or manual bypasses.

(3) If the actual grouting pressure exceeds the maximum recommended pumping pressure, grout may be injected at any vent which has been, or is ready to be, capped as long as a one-way flow of grout is maintained. If this procedure is used, then the vent which is to be used for injection should be fitted with a positive shut-off.

(4) When one-way flow of grout cannot be maintained as outlined in *Sections 3.3.6.2 (1) and3.3.6.2* (3) above, the grout should be immediately flushed out of the duct with water.

(5) Grout should be pumped through the duct and continuously wasted at the outlet pipe until no visible slugs of water or air are ejected and the efflux time of the ejected grout should not be less than the injected grout. To insure that the tendon remains filled with grout, the outlet and/or inlet should be closed. Plugs, caps or valves thus required should not be removed or opened until the grout has set.

Current research indicates that use of standpipes at high points of grouted tendons is a satisfactory substitute for a positive means of shut-off. Standpipes permit free expansion which tends to push out any bleed water that may occur at high points.

Vertical or nearly vertical tendons made up of strands, which tend to act as filters for the grout, require special consideration. Because of exaggerated bleed, special grouting techniques should be used to ensure complete filling of the top portion of the tendon. This may be achieved by two stages of grouting, free expansion of grout pushing the bleed water out at the high points, or admixtures which increase the water retentivity so that bleed is con trolled.

3.3.7 Temperature Considerations

(1) In temperatures below 32 degrees Fahrenheit, ducts should be kept free of water to avoid damage due to freezing.

This is normally accomplished with low point drains.

(2) Concrete temperature The temperature of the concrete should be 35° F. or higher from the time of grouting until job cured 2 inch cubes of grout reach a minimum compressive strength of 800 psi.

At 35 degrees F, grout may be expected to reach 800 psi cube strength in about 5 days.

(3) Grout temperature — Grout should not be above 90° F. during mixing or pumping. If necessary, the mixing water should be cooled.

Difficulties in pumping grout may occur when the grout temperature in the mixer exceeds 90 degrees F.

3.3.8 METHOD OF TEST FOR FLOW OF GROUT MIXTURES (Flow-Cone Method)

CRD-C 79-58 *(Issued Sept. 1, 1958)*

Scope

1. This method of test covers the procedure to be used both in the laboratory and in the field for determining the flow of grout mixtures by measuring the time of efflux of a specified volume of grout from a standardized flow cone.

Apparatus

2. (a) **Flow Cone. —** The flow cone shall conform to the dimensions and other requirements indicated in *Fig. 3.3.8.7.*

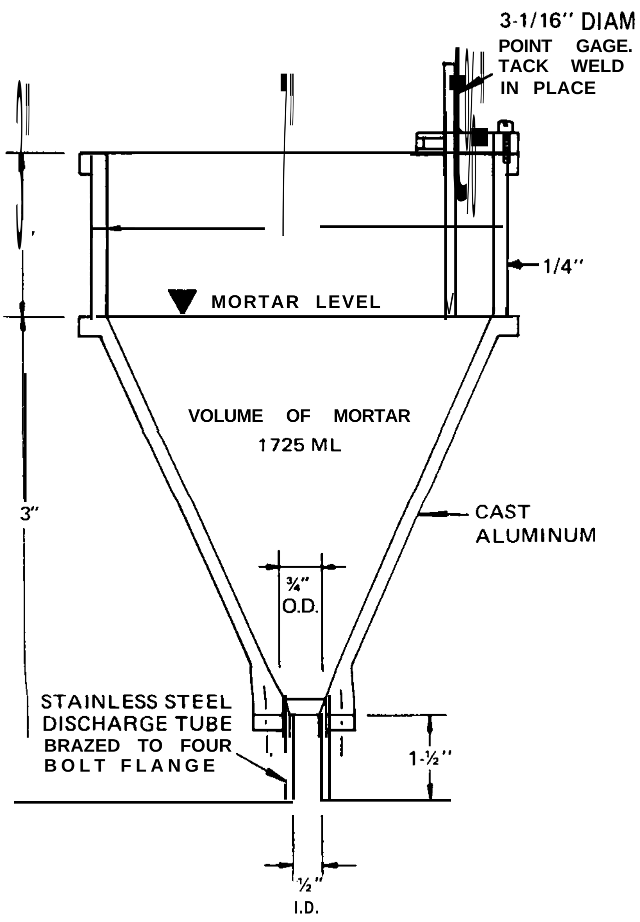

Fig. 3.3.8.1 — Cross section of flow cone

(b) **Stop Watch. —** A stop watch having a least reading of not more than 0.2 sec.

Calibration of Apparatus

3. The flow cone shall be firmly mounted in such a manner that the top will be level and the cone free from vibration. The discharge tube shall be closed by placing the finger over the lower end. A quantity of water equal to 1725± 1 ml shall be introduced into the cone. The point gage shall be adjusted to indicate the level of the water surface.

Sample

4. The test sample shall consist of 1725± 1 ml of grout.

Procedure

5. Moisten the inside surface of the flow cone (See note below). Place the finger over the outlet of the discharge tube. Introduce grout into the cone until the grout surface rises into contact with the point gage. Start the stop watch and remove the finger simultaneously. Stop the stop watch at the first break in the continuous flow of grout from the discharge tube. The time indicated by the stop watch is the time of efflux of the grout. At least two tests shall be made for any grout mixtures.

Note: A recommended procedure for insuring that the interior of the cone is properly wetted is to fill the cone with water and, one minute before beginning to add the grout sample. allow the water to drain from the cone.

Report

6. The report shall include:
   - (a) Average time of efflux to the nearest 0.2 sec,
   - (b) Temperature of the sample at the time of test,
   - (c) Ambient temperature at the time of test,
   - (d) Composition of the sample, and
   - (e) Information on the physical characteristics of the sample.

American Association State Highway and Transportation Officials Standard
AASHTO No M 204

Standard Specification for UNCOATED STRESS-RELIEVED WIRE FOR PRESTRESSED CONCRETE[1]

This standard is issued under the Axed designation A 421; the number immediately following the designation indicates the year of original adoption or. in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval.

This specification has been approved for use by agencies of the Department of Defense for listing in the DoD Index of Specifications and Standards.

1. Scope

1. I This specification covers two types of uncoated stress-relieved round high-carbon steel wire commonly used in prestressed linear concrete construction, as follows:

1. I. I *Type BA* wire is used for applications in which cold-end deformation is used for anchoring purposes (Button Anchorage), and

1.1.2 *Type WA* wire is used for application in which the ends are anchored by wedges, and no cold-end deformation of the wire is involved (Wedge Anchorage).

1.2 Supplement I describes low relaxation wire and relaxation testing for that product.

1.3 The values stated in inch-pound units are to be regarded as the standard.

2. Applicable Documents

2.1 ASTM Standards:

A 370 Methods and Definitions for Mechanical Testing of Steel Products (including Supplement IV)[2]

E 30 Chemical Analysis of Steel, Cast Iron, Open-Hearth Iron, and Wrought Iron"

E 380 Metric Practice"

2.2 Military Standards:

MIL-STD- 129 Marking for Shipment and Storage[5]

MIL-STD- 163 Steel Mill Products, Preparation for Shipment and Storage[5]

2.3 Federal Standard

Fed. Std. No. 123 Marking for Shipments (Civil Agencies)[5]

3. Ordering Information

3.1 Orders for stress-relieved wire under this specification shall include the following information:

3.1.1 Quantity (lb),

3. I .2 Diameter,

3.1.3 Type of anchorage (BA or WA),

3.1.4 Packaging,

3.1.5 ASTM designation and date of issue, and

3.1.6 Special requirements, if any.

NOTE I-A typical ordering description is as follows: 40 000 lb, 0.250-in. diameter wire, Type BA in approximately 1000-lb 60-in. diameter coils to ASTM A 42 I dated ——.

4. Manufacture

4.1 Process-The steel shall be made by the basic-oxygen, open-hearth, or electric-furnace process.

4.2 Internal Soundness-A sufficient discard shall be made to ensure freedom from injurious piping and undue segregation.

4.3 Wire-The wire shall be cold-drawn to size and suitably stress relieved after cold drawing by a continuous heat treatment to produce the prescribed mechanical properties.

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[1] This specification is under the jurisdiction of ASTM Committee A- I on Steel. Stainless Steel and Related Alloys. and is the direct responsibility of Subcommittee A01.05 on Steel Reinforcement.

Current edition approved Aug. I. 1980. Published December 1980. Originally published as A 42 | - 58 T. Last Previous edition A 421 - 77.

[2] *Annual Book of ASTM Standards.* Parts I, 2.3.4. 5. and 10.

[3] *Annual Book o/ASTM Standards.* Part 12.

[4] *Annual Book of ASTM Standards.* Part 4 I

[5] Available *from* Naval Publications and Forms Center. 5801 Tabor Ave., Philadelphia. Pa. 19120.

5. Chemical Requirements

5.1 Variations in manufacturing processes and equipment among wire manufacturers necessitate the individual selection of an appropriate chemical composition at the discretion of the manufacturer.

5.2 Phosphorus and sulfur values shall not exceed the following:

Phosphorus	0.040%
Sulfur	0.050%

5.3 An analysis may be made by the purchaser from finished wire representing each heat of steel. Samples for analysis shall be obtained by milling the wire in such a manner as to obtain a sample representative of the entire cross section. Prior to milling, the surface shall be cleaned to remove all foreign matter. All such individual determinations shall not vary from the limits shown in 5.2 by more than 0.008 %.

5.4 For referee purposes Methods E 30 shall be applied.

6. Physical Requirements

6.1 *Tensile Strength*—The tensile strength of Type BA wire and Type WA wire shall conform to the requirements prescribed in Table 1.

6.2 *Yield Strength:*

6.2.1 The minimum yield strength for all wire, measured by the 1.0 % extension under load method, shall not be less than 85 % of the specified minimum breaking strength.

6.2.2 The extension under load shall be measured by an extensometer calibrated with the smallest division not larger than 0.0001 in./in. of gage length.

6.2.3 The initial load corresponding to the initial stress prescribed in Table 2 shall be applied to the specimen, at which time the extensometer is attached and adjusted to a reading of 0.001 in./in. of gage length. The load shall then be increased until the extensometer indicates an extension of 1 %. The load for this extension shall be recorded. The stress corresponding to this load shall meet the requirements for stress at 1 % extension prescribed in Table 2.

6.3 *Elongation*—The total elongation under load of all wire shall not be less than 4.0 % when measured in a gage length of 10 in. or 250 mm. The elongation shall be determined by an extensometer which is placed on the test specimen after a load corresponding to the initial stress prescribed in Table 2 is applied. If the fracture takes place outside of the gage length, the elongation value obtained may not be representative of the material. If the elongation so measured meets the minimum requirements specified, no further testing is indicated; but if the elongation is less than the minimum requirements, the test shall be discarded and a retest made.

7. Diameter and Permissible Variations

7.1 Wire meeting the requirements of this specification is normally ordered in the diameters shown in Table 1.

7.2 The diameter of the wire shall not vary from the nominal diameter specified by more than ±0.002 in. (0.05 mm).

7.3 The wire shall not be out-of-round by more than 0.002 in. (0.05 mm).

8. Workmanship and Finish

8.1 *Cast*—A wire sample having a chord length of 60 in. (1524 mm) shall have an offset at the center of the chord of not more than 3 in. (76 mm). This is equivalent to a chord of an arc of a circle not less than 25 ft (7.6 m) in diameter.

8.2 *Type BA Wire*—Type BA wire shall be of suitable quality to permit cold forming of buttons for anchorage. Splitting shall not be considered a cause for rejection if the button anchorage is capable of developing the minimum required tensile strength of the wire.

8.3 The wire shall be free of kinks.

8.4 The wire shall be furnished in firmly tied coils having a minimum inside diameter of 48 in. (1219 mm). Each coil shall be of one continuous length.

8.5 There shall be no welds or joints in the finished wire. Any welds or joints made during manufacture to promote continuity of operations shall be removed.

8.6 The wire shall not be oiled or greased. Slight rusting, provided it is not sufficient to cause pits visible to the naked eye, shall not be cause for rejection.

8.7 Temper colors which may result from the stress-relieving operation are considered normal as regards the finished appearance of the wire.

9. Sampling

9.1 Unless otherwise agreed upon between the manufacturer and the purchaser, one test specimen shall be taken from each 10 coils or less in a lot (Note 2) and tested to determine compliance with 6. I, 6.2, 6.3, 7, and 8.1.

NOTE 2-The term "lot" means all the coils of wire of the same nominal wire size contained in an individual shipping release or shipping order.

10. Packaging and Marking

10.1 The size of the wire, ASTM specification number, heat number, and name or mark of the manufacturer shall be marked on a tag securely attached to each bundle of wire.

10.2 For Government Procurement Only-When specified in the contract or order, and for direct procurement by or direct shipment to the government, material shall be preserved, packaged, and packed in accordance with the requirements of MIL-STD- 163. The applicable levels shall be as specified in the contract. Marking for shipment of such material shall be in accordance with Fed. Std. No. 123 for civil agencies and MIL-STD-129 for military agencies.

11. Inspection

I 1. I The purchaser shall state at the time of order whether outside inspection is required or waived. If outside inspection is required, the manufacturer shall afford the inspector representing the purchaser all reasonable facilities to satisfy him that the material is being furnished in accordance with this specification. All tests and inspections shall be made at the place of manufacture prior to shipment, unless otherwise agreed upon at the time of purchase, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

12. Certification

12.1 If outside inspection is waived, a manufacturer's certification that the material has been tested in accordance with and meets the requirements of this specification shall be the basis of acceptance of the material.

13. Rejection

13.1 Unless otherwise specified, any rejection based on tests made in accordance with this specification shall be reported to the manufacturer within a reasonable length of time.

SUPPLEMENT

I. LOW-RELAXATION WIRE

S1. Scope

S 1.1 This supplement delineates only those details which are peculiar to low-relaxation wire, and to the method of relaxation testing related to single wire tendons having properties generally as described in Specification A 421.

S2. Applicable Documents

S2. I ASTM Standard

E 328 Recommended Practice for Stress-Relaxation Tests for Materials and Structures[6]

S3. Test Method

S3.1 Low-relaxation strand shall be tested as prescribed in Recommended Practice E 328.

S4. Relaxation Properties

S4.1 Low-relaxation wire shall meet the physical requirements of this specification, with the added requirement that relaxation after 1000 h under the conditions of Section S5 shall not be more than 2.5 % when initially loaded to 70 % of specific minimum tensile strength or not more than 3½ % when loaded to 80% of specified minimum tensile strength of the wire.

S5. Yield Strength

S5.1 Yield strength of low-relaxation wire as described in 6.2 shall not be less than 90 % of the specified minimum tensile strength of the wire.

S6. Conditions of Relaxation Test

S6. I If required. relaxation evidence shall be provided from the manufacturer's records of tests on similarly dimensioned wire of the same grade.

[6] *Annual Book of ASTM Standards.* Parts 10 and 41

S6.2 The temperature of the test specimen shall be maintained at 68 ± 3.5°F (20 ± 2°C).

S6.3 The test specimen shall not be subjected to any loading prior to the relaxation test.

S6.4 The initial load shall be applied uniformly over a period of not less than 3 min and not more than 5 min, and the gage length shall be maintained constant; load relaxation readings shall commence 1 mm after application of the total load.

S6.5 Over-stressing of the test sample during the loading operation shall not be permitted.

S6.6 The duration of the test shall be 1000 h or a shorter computed period extrapolated to 1000 h which can be shown by records to provide similar relaxation values.

S6.7 The test gage length shall be at least 60 times the nominal diameter.

TABLE 1 Tensile Strength Requirements

Nominal Diameter, in. (mm)	Tensile Strength. min. psi (MPa)	
	Type BA	Type WA
0.192 (4.88)	[a]	250 000 (1725)
0.1% (4.98)	240 000 (1655)	250 000 (1725)
0.250 (6.35)	240 000 (1655)	240 000 (1655)
0.276 (7.01)	235 000 (1620)	235 000 (1620)

[a] This size is not commonly furnished in Type BA wire.

TABLE 2 Yield Strength Requirements

Nominal Diameter. in. (mm)	Initial Stress. psi (MPa)	Minimum Stress at I 4 Extension. psi (MPa)	
		Type BA	Type WA
0.192 (4.88)	29 000 (200)	[a]	212 500 (1465)
0.1% (4.98)	29 000 (200)	204 000 (1407)	212 500 (1465)
0.250 (6.35)	29 000 (200)	204 000 (1407)	204 000 (1407)
0.276 (7.01)	29 000 (200)	199 750 (1377)	199 750 (1377)

[a] This size is not commonly furnished in Type BA wire.

Standard Specification for UNCOATED SEVEN-WIRE STRESS-RELIEVED STEEL STRAND FOR PRESTRESSED CONCRETE[1]

This standard is issued under the fixed designation A 416; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates tbe year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This specification has been approved for use by agencies of the Department of Defense and for listing on the DoD Index of Specifications and Standards.

ε1 NOTE—Sections 11 through 14 were renumbered editorially in July 1984.

1. Scope

1.1 This specification covers two grades of seven-wire, uncoated, stress-relieved steel strand for use in pretensioned and post-tensioped prestressed concrete construction. Grade 250 and Grade 270 have minimum ultimate strengths of strengths of 250 000 psi (1725 MPa) and 270 000 psi (1860 MPa), respectively, based on the nominal area of the strand.

1.2 Supplement I describes low-relaxation strand and relaxation testing for that product. Low relaxation strand shall not be furnished unless ordered, or by arrangement between purchaser and supplier.

1.3 The values stated in inch-pound units are to be regarded as the standard.

2. Applicable Documents

2.1 ASTM Standards:

A 370 Methods and Definitions for Mechanical Testing of Steel Products[2]

E 328 Recommended Practice for Stress-Relaxation Tests for Materials and Structures[3]

2.2 Military Standards:

MIL-STD- 129 Marking for Shipment and Storage[4]

MIL-STD- 163 Steel Mill Products Preparation for Shipment and Storage[4]

23 Federal Standard:

Fed. Std. No. 123 Marking for Shipments (Civil Agencies)[4]

3. Description of Term Specific to this Standard

3.1 strand—all strand shall be of the seven-wire type having a center wire enclosed tightly by six helically placed outer wires with a uniform pitch of not less than 12 and not more than 16 times the nominal diameter of the strand.

4. Ordering Information

4.1 Orders for seven-wire stress relieved strand under this specification shall include the following information:

4.1.1 Quantity (feet),

4.1.2 Diameter of strand,

4.1.3 Grade of strand,

4.1.4 Packaging,

4.1.5 ASTM designation and year of issue, and

4.1.6 Special requirements, if any.

NOTE 1—A typical ordering description is as follows: 84 000 A, ½ in., Grade 270 strand, in 12 000-ft spool-less packs to ASTM A 416 - XX.

5. Materials and Manufacture

5.1 *Base Metal*—The base metal shall be carbon steel of such quality that when drawn to suitable round wire sizes and fabricated into strand sizes, and stress relieved after stranding, it

[1] This specification is under the jurisdiction of ASTM Committee A-1 on Steel, Stainless Steel and Related Alloys, and is the direct responsibility of Subcommittee A01.05 on Steel Reinforcement.

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[2] *Annual Book of ASTM Standards.* Vol. 01.04.

[3] *Annual Book of ASTM Standards.* Vol. 03.01.

[4] Available from Naval Publications and Forms Center, 5801 Tabor Ave., Philadelphia, PA 19120.

shall have the properties and characteristics prescribed in this specification.

5.2 Wire-The wire from which the strand is to be fabricated shall have a common dry-drawn finish.

5.3 Stress Relieving-After stranding, all strand shall be subjected to a stress-relieving continuous heat treatment to produce the prescribed mechanical properties. Temper colors which may result from the stress-relieving operation are considered normal for the finished appearance of this strand.

6. Physical Requirements

6.1 Breaking Strength-The breaking strength of the finished strand shall conform to the requirements prescribed in Table 1, and shall be determined as prescribed in Supplement VII of Methods and Definitions A 370.

6.2 Yield Strength-The minimum yield strength, as prescribed in Table 2, as measured by the 1 % extension under load method, shall not be less than 85 % of the specified minimum breaking strength.

6.2.1 The extension under load shall be measured by an extensometer calibrated with the smallest division not larger than 0.000 1 in./in. of gage length.

6.2.2 The initial load indicated in Table 2 shall be applied to the specimen, at which time the extensometer is attached and adjusted to a reading of 0.001 in./in. of gage length. The load shall then be increased until the extensometer indicates an extension of 1 %. The load for this extension shall be recorded and shall meet the requirements prescribed in Table 2.

6.3 Elongation:

6.3.1 The total elongation, under load, of the strand shall be not less than 3.5 % and shall be measured in a gage length of not less than 24 in. or 610 mm. The elongation shall be determined by an extensometer which is placed on the test specimen after an initial load has been applied. The initial load is equivalent to 10 % of the required minimum breaking strength as prescribed in Table 2. Following an extension of 1.0 %, the extensometer may be removed and loading continued to ultimate failure. The elongation value is then determined by the movement between the jaws gripping the material on the new base length of jaw-to-jaw distance to which will be added the value of 1.0 % determined by the extensometer.

6.3.2 Specimens that break outside of the extensometer or in the jaws and yet meet the minimum specified values, are considered as meeting the elongation requirements of this specification.

6.3.3 If the minimum elongation requirement is met prior to initial rupture, it is not necessary to determine the final elongation value.

6.4 If any sample breaking within the grips or the jaws of the testing machine results in values below the specified limits for breaking strength, yield strength, or elongation, the results shall be considered invalid and retesting shall be required.

7. Dimensions and Permissible Variations

7.1 The size of the finished strand shall be expressed as the nominal diameter of the strand in fractions or decimal fractions of an inch.

7.2 The diameter of the center wire of any strand must be larger than the diameter of any outer wire in accordance with Table 3.

7.3 Permissible Variations in Diameter:

7.3.1 All Grade 250 strand shall conform to a size tolerance of ±0.016 in. (±0.4 1 mm) from the nominal diameter measured across the crowns of the wires.

7.3.2 All Grade 270 strand shall conform to a size tolerance of +0.026 in. -0.006 in. (+0.66 -0.15 mm) from the nominal diameter measured across the crowns of the wire.

7.3.3 Variation in cross-sectional area and in unit stress resulting therefrom shall not be cause for rejection provided the diameter differences of the individual wires and the diameters of the strand are within the tolerances specified.

7.4 Specially dimensioned stress-relieved strand with nominal diameters up to 0.750 in. (19.05 mm) may be employed, providing that the breaking strength is defined, and the yield strength, as defined in 6.2, is not less than 85 % of the specified minimum breaking strength. All other requirements shall apply.

8. Workmanship

8.1 Joints:

8.1.1 There shall be no strand joints or strand splices in any length of the completed strand unless specifically permitted by the purchaser.

8.1.2 During the process of manufacture of the individual wires for stranding, welding is permitted only prior to or at the size of the last heat treatment (patenting).

8.1.3 During fabrication of the strand, butt-welded joints may be made in the individual wires, provided there is not more than one such joint in any 150-ft (45-m) section of the completed strand.

NOTE 2—When specifically ordered as "Weldless-Grade", a product free of welds shall be furnished. When this grade is specified, no welds or joints are permitted except as detailed in 8.1.2.

8.2 The finished strand shall be uniform in diameter and shall be free of imperfections not consistent with good commercial stranding practice.

8.3 When the strand is cut without seizings, the wire shall not fly out of position. If a wire, or wires, flies out of position and can be replaced by hand, the strand will be considered satisfactory.

8.4 The strand shall not be oiled or greased. Slight rusting, provided it is not sufficient to cause pits visible to the naked eye, shall not be cause for rejection.

9. Sampling

9.1 One specimen for test shall be taken from each 20-ton (18-Mg) production lot of finished strand. Test specimens shall be cut from the outside end of reels or either end of coils or reelless packs. Any specimen found to contain a wire joint should be discarded and a new specimen obtained.

10. Test Methods

10.1 The test specimens as selected in 9.1 shall be tested by the method prescribed in Supplement VII of Methods and Definitions A 370.

11. Inspection

11.1 The purchaser shall state at the time of order whether inspection by the purchaser at the plant is required or waived. If purchaser inspection is required, the manufacturer shall afford the inspector representing the purchaser all reasonable facilities to satisfy him that the material is being furnished in accordance with this specification. All tests and inspections shall be made at the place of manufacture prior to shipment, unless otherwise agreed upon at the time of purchase, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

12. Rejection

12.1 In case there is a reasonable doubt in the first trial as to the ability of the strand to meet any requirement of this specification, two additional tests shall be made on samples of strand from the same coil or reel, and if failure occurs in either of these tests, the strand shall be rejected.

13. Certification

13.1 If outside inspection is waived, a manufacturer's certification that the material has been tested in accordance with, and meets the requirements of, this specification shall be the basis of acceptance of the material.

13.2 The manufacturer shall, when requested in the order, furnish a representative load-elongation curve for each size and grade of strand shipped.

14. Packaging and Marking

14.1 The strand shall be furnished on reels or in compact coils having a minimum core diameter of 24 in. (610 mm), unless otherwise specified by the purchaser. Lengths on reels, in coils, or in reelless packs shall be as agreed upon at the time of purchase. The strand shall be well protected against mechanical injury in shipping as agreed upon at the time of purchase. Each reel, coil, or reelless pack shall have a strong tag securely fastened to it showing the length, size, grade, ASTM designation A 416, and the name or mark of the manufacturer.

14.2 Low-relaxation strand produced meeting the requirements of Supplement I must be specially identified.

14.3 *For Government Procurement Only*—**When specified in the contract or order, and for** direct procurement by or direct shipment to **the** U.S. government, material shall be preserved, packaged, and packed in accordance with the requirements of MIL-STD- 163. The applicable levels shall be as specified in the contract. Marking for shipment of such material shall be accordance with Fed. Std. No. 123 for civil agencies and MIL-STD- 129 for military agencies.

TABLE 1 Breaking Strength Requirements

Nominal Diameter of strand, in.	Nominal Diameter of strand, mm	Breaking Strength of Strand, lbf (kN)	Nominal Steel Area of Strand, in? (mm²)	Nominal Weight of Strands, lb/1000 ft (kg/1000 m)·
		Grade 250		
¼ (0.250)	6.35	9 ooo (40.0)	0.036 (23.22)	122 (182)
5/16 (0.3 13)	7.94	14 500 (64.5)	0.058 (37.42)	197 (294)
⅜ (0.375)	9.53	20 000 (89.0)	0.080 (51.61)	272 (405)
7/16 (0.438)	11.11	27 000 (120.1)	0. I08 (69.68)	367 (548)
½ (0.500)	12.70	36 ooo (160.1)	0.144 (92.90)	490 (730)
(0.600)	15.24	54 ooo (240.2)	0.216 (139.35)	737 (1094)
		Grade 270		
⅜ (0.375)	9.53	23 000 (102.3)	0.085 (54.84)	290 (432)
7/16 (0.438)	11.11	31 000 (137.9)	0.1 15 (74.19)	390 (582)
½ (0.500)	12.70	41 300 (183.7)	0.153 (98.71)	520 (775)
(0.600)	15.24	58 600 (260.7)	0.217 (140.00)	740 (I 102)

TABLE 2 Yield Strength Requirements

Nominal Diameter of Strand, in.	Nominal Diameter of Strand, mm	Initial Load, lbf (kN)	Minimum Load at I % Extension, lbf (kN)
		Grade 250	
¼ (0.250)	6.35	900 (4.0)	7 650 (34.0)
5/16 (0.3 13)	7.94	I 450 (6.5)	12 300 (54.7)
⅜ (0.375)	9.53	2 000 (8.9)	17 000 (75.6)
7/16 (0.438)	11.11	2 700 (12.0)	23 000 (102.3)
½ (0.500)	12.70	3 600 (16.0)	30 600 (136.2)
(0.600)	15.24	5 400 (24.0)	45 900 (204.2)
		Grade 270	
⅜ (0.375)	9.53	2 300 (10.2)	19 550 (87.0)
7/16 (0.438)	11.11	3 100 (13.8)	26 350 (117.2)
½ (0.500)	12.70	4 130 (18.4)	35 100 (156.1)
(0.600)	15.24	5 860 (26.1)	49 800 (221.5)

TABLE 3 Diameter Relation Between Center and Outer Wires

Nominal Diameter of Strands, in.	Nominal Diameter of Strands, mm	Minimum Difference Between Center Wii Diameter and Diameter of any Outer Wire, in.	Minimum Difference Between Center Wii Diameter and Diameter of any Outer Wire, mm
		Grade 250	
¼ (0.250)	6.35	0.001	0.0254
5/16 (0.3 13)	7.94	0.0015	0.043 I
⅜ (0.375)	9.53	0.002	0.0508
7/16 (0.438)	11.11	0.0025	0.0685
½ (0.500)	12.70	0.003	0.0762
(0.600)	15.24	0.004	0.1016
		Grade 270	
⅜ (0.375)	9.53	0.002	0.0508
7/16 (0.438)	11.11	0.0025	0.0685
½ (0.500)	12.70	0.003	0.0762
(0.600)	15.24	0.004	0.1016

SUPPLEMENT

1. LOW-RELAXATION STRAND

S1. Scope

S I. I This supplement delineates only those details that are peculiar to low-relaxation strand, and to the methods of relaxation testing related to seven-wire strand having properties generally as described in Specification A 416.

S2. Appliible Document

S2.1 ASTM Standard:

E 328 Recommended Practice for Stress-Relaxation Tests for Materials and Structures[3]

S3. Test Method

S3.1 Low-relaxation strand shall be tested as prescribed in Recommended Practice E 328.

S4. Relaxation Properties

S4.1 Low-relaxation strand shall meet the physical requirements of this specification, with the added requirement that the relaxation loss after 1000 h under the conditions of S5 shall be not more than 2.5 % when initially loaded to 70 % of specified minimum breaking strength, or not more than 3.5 % when loaded to 80 % of specified minimum breaking strength of the strand.

S5. Yield Strength

S5.1 Yield strength of low-relaxation strand, as described in 6.2, shall not be less than 90 % of the specified minimum breaking strength of the strand.

S6. Conditions of Relaxation Test

S6.1 If required, relaxation evidence shall be provided from the manufacturer's records of tests on similarly dimensioned strand of the same grade.

S6.2 The temperature of the test piece shall be maintained at 68 $\pm$ 3.5°F (20 $\pm$ 2°C).

S6.3 The test piece shall not be subjected to loading prior to the relaxation test.

S6.4 The initial load shall be applied uniformly over a period of not less than 3 min and not more than 5 min, and the gage length shall be maintained constant; load relaxation readings shall commence I min after application of the total load.

S6.5 Over-stressing of the test sample during the loading operation shall not be permitted.

S6.6 The duration of the test shall be 1000 h or a shorter computed period extrapolated to 1000 h which can be shown by records to provide similar relaxation values.

S6.7 The test gage length shall be at least 60 times the nominal diameter. If this gage length exceeds the capacity of the extensometer or testing machine, then a minimum gage length of 40 times the nominal diameter may be substituted.

Standard Specification for UNCOATED HIGH-STRENGTH STEEL BAR FOR PRESTRESSING CONCRETE[1]

This standard is issued under the fixed designation A 722; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (t) indicates an editorial change since the last revision or reapproval.

1. scope

1.1 This specification covers uncoated high-strength steel bars intended for use in prestressed concrete construction. Bars are of a single minimum ultimate tensile strength level of 150 000 psi (1035 MPa).

1.2 Two types of bars are provided: Type I bar has a plain surface and Type II bar has surface deformations. The standard sizes and dimensions of Type I and II bars shall be those listed in Tables 1 and 2 respectively.

1.3 Supplementary requirements of an op tional nature are provided. They shall apply only when specified by the purchaser.

NOTE I-The values stated in inch-pound units are to be regarded as the standard.

2. Applicable Documents

2.1 ASTM Standards:

A 370 Methods and Definitions for Mechanical Testing of Steel Products*

A 700 Practices for Packaging, Marking, and Loading Methods for Steel Products for Domestic Shipment'

E 30 Methods for Chemical Analysis of Steel, Cast Iron, Open-Hearth Iron, and Wrought Iron[3]

3. Ordering Information

3.1 Orders for material under this specification shall include the following information:

3.1.1 Quantity,

3.1.2 Name of material (uncoated high-strength bars for prestressing concrete),

3.1.3 ASTM designation and date of issue,

3.1.4 Size and length,

3.1.5 Type,

3.1.6 Special inspection requirements, if desired (see Section 12),

3.1.7 Special preparation for delivery, if desired (see Section 1 I), and

3.1.8 Supplementary requiremenfs, if desired.

NOTE 2-A typical ordering description is as follows: 50 uncoated high-strength bars for prestressing concrete to ASTM A 722 dated ——; 3/4-in. diameter, 40 ft, 6 in. long, Type II; packed in accordance with A 700; meeting supplementary bending properties.

4. Manufacture

4.1 The bars shall be rolled from properly identified heats of mold cast or strand cast steel.

4.2 The bars shall be subjected to cold stressing to not less than 80 % of the minimum ultimate strength, and then shall be stress relieved, to produce the prescribed mechanical properties.

5. Chemical Requirements

5.1 An analysis of each cast or heat of steel shall be made by the manufacturer from test samples taken during the pouring

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[1] This specification is under the jurisdiction of ASTM Committee A-l on Steel, Stainless Steel and Related Alloys, and is the direct responsibility of Subcommittee AOI .05 on Steel Reinforcement.

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[2] *Annual Book of ASTM Standards.* Vols 0 I .0 I-0 I .05.

[3] *Annual Book of ASTM Standards.* Vol 03.05.

of each cast or heat. The percentages of carbon, manganese, phosphorus, sulfur, and all alloying elements shall be determined.

5.1. I Choice and use of chemical composition and alloying elements, to produce the mechanical properties of the finished bar prescribed in 6.2, shall be made by the manufacturer, subject to the limitations in 5.1.2.

5.1.2 On cast or heat analysis, phosphorus and sulfur shall not exceed the following:

Phosphorus	0.040 %
sulfur	0.050 %

5.2 A product analysis may be made by the purchaser from the finished bar representing each cast or heat of steel. The phosphorus and sulfur contents thus determined shall not exceed the limits specified in 5.1.2 by 0.008 %.

5.3 Method E 30 shall be used for referee purposes.

6. Mechanical Requirements

6.1 All testing for mechanical properties shall be performed in accordance with the requirements of Methods and Definitions A 370.

6.2 Tensile Properties:

6.2.1 Finished bars shall have a minimum ultimate tensile strength of 150 000 psi (1035 MPa).

6.2.2 The minimum yield strength of Type I and Type II bars shall be 85 % and 80 %, respectively, of the minimum ultimate tensile strength of the bars. The yield strength shall be determined by either of the methods described in Methods and Definitions A 370; however, in the extension under load method, the total strain shall be 0.7 %, and in the offset method the offset shall be 0.2 %.

6.2.3 The minimum elongation after rupture, in a gage length equal to 20 bar diameters, shall be 4.0%.

6.3 Test Specimens-Tension tests shall be made using full-size bar test specimens. Machined reduced section test specimens are not permitted. All unit stress determinations shall be based on the nominal area shown in Table 1 or the effective area shown in Table 2.

6.4 *Number of Tests-Two* tensile specimens shall be tested from each bar size rolled from each heat of steel. Whenever one bar size rolled from any one heat exceeds 100 tons (90 Mg), three tension specimens shall be tested. The specimen shall be randomly selected following the final processing operation.

6.5 Retests:

6.5.1 If any tensile property of any tension test specimen is less than that specified, and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

6.5.2 If the results of an original tension test fail to meet specified requirements, two additional tests shall be made on samples of bar from the same heat and bar size, and if failure occurs in either of these tests, the bar size from that heat shall be rejected.

6.5.3 If any test specimen fails because of mechanical reasons such as failure of testing equipment, it may be discarded and another specimen taken.

6.5.4 If any test specimen develops flaws, it may be discarded and another specimen of the same size bar from the same heat substituted.

7. Requirements for Deformations

7.1 Material furnished as Type II bar shall have deformations spaced uniformly along the length of the bar. The deformations on opposite sides of the bar shall be similar in size and shape. The average spacing or distance between deformations on both sides of the bar shall not exceed seven tenths of the nominal diameter of the bar.

7.2 The minimum height and minimum projected area of the deformations shall conform to the requirements shown in Table 3.

8. Measurements of Deformations

8.1 The average spacing of deformations shall be determined by dividing a measured length of the bar specimen by the number of individual deformations and fractional parts of deformations on any one side of the bar specimen. A measured length of the bar specimen shall be considered the distance from a point on a deformation to a corresponding point on any other deformation on the same side of the bar.

8.2 The average height of deformations shall be determined from measurements made on not less than two typical deformations. Determinations shall be based on three measurements per deformation: one at the center of the overall length, and the other two at the quarter points of the overall length.

8.3 To indicate adequately the conformity to the dimensional requirements, measurements shall be taken at random from one bar from each 30 tons (27 Mg) of each lot or fraction thereof.

8.4 Insufficient height, insufficient projected area, or excessive spacing of deformations shall not constitute cause for rejection unless it has been clearly established by determinations on each lot that typical deformation height or spacing does not conform to the minimum requirements prescribed in Section 7. No rejection may be made on the basis of measurements if fewer than ten adjacent deformations on each side of the bar are measured.

NOTE 3-The term "lot" means all bars of the same nominal weight per linear foot contained in an individual shipping release or shipping order.

9. Permissible Variation in Size or Weight

9.1 For Type I bars, the permissible variation from the nominal diameter specified in Table 1 shall not exceed +0.030, -0.010 in. (+0.76, −0.25 mm).

9.2 For Type II bars, the permissible variation from the nominal weight specified in Table 2 shall not exceed +3 %, -2 %.

10. Finish

10.1 The bars shall be free of defects injurious to the mechanical properties and shall have a workmanlike finish.

11. Delivery

11.1 Unless otherwise specified in the contract or purchase order, bars shall be packed for delivery in accordance with the finished-bar manufacturer's standard commercial practice.

11.2 When specified in the contract or purchase order, bars shall be packed in accordance with Practices A 700.

11.3 *Marking:*

11.3.1 Unless otherwise specified in the contract or purchase order, bars shall be sorted by size and each bundle or lift shall be properly tagged with metal tags showing heat number, size, specification number (ASTM A 722) and the name of the finished-bar manufacturer in order to assure proper identification. The tags shall display the following statement: "High-Strength Prestressing Bars." In addition, both ends of each bar shall be painted yellow.

11.32 When specified in the contract or purchase order, bars shall be marked in accordance with Practices A 700.

12. Inspection

12.1 The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works that concern the manufacture of the material ordered. The manufacturer shall afford the inspector all reasonable facilities to satisfy him that the material is being furnished in accordance with this specification. All tests (except product analysis) and inspection, shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

12.2 If specified in the purchase order, the purchaser may reserve the right to perform any of the inspection set forth in the specification where such inspections are deemed necessary to assure that the material furnished conforms to prescribed requirements.

12.3 If outside inspection is waived, the finished-bar manufacturer's certification that the material has been tested in accordance with, and meets the requirements of, this specification, shall be the basis of acceptance of the material.

13. Rejection

13.1 Unless otherwise specified, any rejection based on tests made in accordance with 5.2 shall be reported to the manufacturer within 5 working days from the receipt of samples by the purchaser.

13.2 Material that shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

14. Rehearing

14.1 Samples tested in accordance with 5.2 that represent rejected material shall be preserved for two weeks from the date rejection is reported to the manufacturer. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

SUPPLEMENTARY REQUIREMENTS

The following supplementary requirements shall apply only when specified in the purchase order or contract. Supplementary requirements shall in no way negate any requirements of the specification itself.

S 1. Rending Properties

S1.l The bend test specimen shall stand being bent, at ambient temperature but in no case less than 60°F (16°C), around a pin without cracking on the outside of the bent portion. The requirements for degree of bending and sizes of pins are prescribed in Table 4.

S1.2 The bend test shall be made on full-size specimens of sufficient length to ensure free bending and with apparatus that provides the following:

S 1.2.1 Continuous and uniform application of force throughout the duration of the bending operation.

S1.2.2 Unrestricted movement of the specimen at points of contact with the apparatus and bending around a pin free to rotate or bending about a central pin on a simple span with end supports free to rotate.

Sl.2.3 Close wrapping of the specimen around the pin during the bending operation.

S1.3 Other methods of bending testing may be used, but failures due to such methods shall not constitute a basis for rejection.

S2. Mechanical Coupling

S2.1 For those bars having deformations arranged in a manner to permit coupling of the bars with a screw-on type of coupler, it shall be the responsibility of the finished-bar manufacturer to demonstrate that a bar cut at any point along its length may be coupled to any other length of bar and that a coupled joint supports the ultimate breaking stength of the coupled bars. The coupler type shall be provided or designed by the finished-bar manufacturer.

S3. Reduction of Area

S3.1 The minimum reduction of area from the effective area shall be 20 %.

S4. Chemical Requirements

S4.1 The chemical composition determined as specified in 5.1 shall be reported to the purchaser or his representative.

TABLE 1 Dimensions for Type I (Plain) Bar

Nominal Diameter		Nominal Weight		Nominal Area[a]	
in.	mm	lb/ft	kg/m	in.²	mm²
3/4	19	1.50	2.23	0.44	284
7/8	22	2.04	3.04	0.60	387
1	25	2.67	3.97	0.78	503
1 1/8	29	3.38	5.03	0.99	639
1 1/4	32	4.17	6.21	1.23	794
1 3/8	35	5.05	7.52	1.48	955

[a] The nominal area is determined from the nominal diameter in inches. Values have been converted from U.S. customary to metric units.

TABLE 2 Dimensions for Type II (Deformed) Bar

Nominal Diameter[a]		Nominal Weight		Effective Area[b]	
in.	mm	lb/ft	kg/m	in.²	mm²
5/8	15	0.98	1.46	0.28	181
3/4	20	1.49	2.22	0.42	271
1	26	3.01	4.48	0.85	548
1 1/4	32	4.39	6.54	1.25	806
1 3/8	36	5.56	8.28	1.58	1019

[a] Nominal diameters are for identification only. Values have been converted from metric to U.S. customary units.

[b] The effective area is determined from the bar weight less 3.5 % for the ineffective weight of the deformations.

TABLE 3 Deformation Dimensions for Type II Bar

Nominal Diameter		Deformation Dimensions					
		Maximum Average Spacing		Minimum Average Height		Minimum Projected Area[a]	
in.	mm	in.	mm	in.	mm	in.²/in.	mm²/mm
5/8	15	0.437	11.10	0.028	0.71	0.094	2.41
3/4	20	0.525	13.34	0.038	0.96	0.130	3.40
1	26	0.700	17.78	0.050	1.27	0.168	4.39
1 1/4	32	0.887	22.52	0.064	1.62	0.212	5.40
1 3/8	36	0.987	25.07	0.071	1.80	0.233	6.08

[a] Calculated from equation, MPA = $0.75\pi d\ h/s$
where:
d = nominal diameter,
h = minimum average height, and
s = maximum average spacing.

TABLE 4 Supplementary Bend Test Requirements

Nominal Bar Diameters		Diameter of Pin for 90deg Bend[a]
in.	mm	
5/8	15	$d = 6t$
3/4	20	$d = 6t$
1	26	$d = 6t$
1 1/4	32	$d = 8t$
1 3/8	36	$d = 8t$

[a] d = diameter of pin around which specimen is bent.
t = nominal diameter of bar.

Chapter 4

Recommendations for Prestressed Rock and Soil Anchors

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4.1 SCOPE

These recommendations have been prepared to provide guidance in the application of permanent and temporary prestressed rock and soil anchors utilizing high strength prestressing steel. They represent the present state of the art and outline what are considered the most practical procedures for installation of prestressed rock and soil anchors.

4.2 DEFINITIONS

Anchor: A system used to transfer tensile loads to soil or rock. It includes all prestressing steel, anchorages, grout, coatings, sheathings and couplers if used.

Permanent Anchor: Any prestressed rock or soil anchor for permanent use. Generally more than an 18-month service life.

A permanent anchor has to fulfill its function for an extended period of time, sometimes throughout the service life of the structure, and thus requires special design and supervision.

Temporary Anchor: Any prestressed rock or soil anchor for temporary use. Generally less than an 18-month service life.

Temporary anchors installed in corrosive environments may require special corrosion protection.

Downward Sloped Anchor: Any prestressed anchor which is placed at a slope greater than 5 degrees [0.087 rad] above the horizontal.

Upward Sloped Anchor: Any prestressed anchor which is placed at a slope greater than 5 degrees [0.087 rad] above the horizontal.

Horizontal Anchor: Any prestressed anchor which is placed at a slope between ±5 degrees [0.087 rad] with the horizontal.

Tendon: The complete anchor assembly (excluding grout) consisting of stressing anchorage and prestressing steel with sheathing and coating when required.

Stressing Anchorage: The means by which the prestressing force is permanently transmitted from the anchor tendon to the supported structure.

Coupling: The means by which the prestressing force may be transmitted from one partial-length of prestressing tendon to another.

Sheathing: Enclosure around the prestressing steel to avoid temporary or permanent bond between the prestressing steel and the surrounding grout and/or to provide corrosion protection.

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Coating: Material used to protect against corrosion and/or lubricate the prestressing steel.

Anchor Grout: (Also known as primary grout.) Portland Cement grout that is injected into the anchor hole to provide anchorage at the bond length of the tendon. Resins are also used in place of Portland Cement grout.

Secondary Grout: Material that is injected into the anchor hole to cover the stressing length of the prestressed anchor, providing corrosion protection for the high strength steel. This material may be grout orothersuitable materials.

Consolidation Grout: Portland Cement grout that is injected into the hole prior to inserting the tendon to waterproof or otherwise improve the rock surrounding the hole.

Resin Cartridge: Tubecontaining resin with filler material and a separated catalyst (hardener).

Gel Time: Time between the start of mixing the resin with the catalyst and the liquid phase changing into a rubbery state.

Cure Time: Time from after the resin has gelled to the state when the resin has hardened sufficiently to carry the prestress load (about 80% to 90% of the final strength).

Cohesive Soils: Soils that exhibit plasticity. Generally defined as composed of material more than half of which is smaller than the No. 200 size sieve.

Non-Cohesive Soils: Granular material that is generally nonplastic, composed of material more than half of which is larger than the No. 200 size sieve.

Minimum Guaranteed Ultimate Tensile Strength (GUTS): is the minimum guaranteed breaking load of the tendon as defined in the pertinent ASTM Specification for the tendon material.

Design Load: Anticipated final maximum effective load in the anchor after allowance for time dependent losses or gains.

Proof Load: Temporary prestressing load in an anchor at a force level greater than its design load for testing purposes.

Transfer (Lock-Off) Load: Prestressing force in an anchor after proof loading immediately after the force has been transferred from the jack to the stressing anchorage.

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In order to better define a soil as cohesive or non-cohesive, it is necessary to know the percentage of fines and also to know the Atterberg limits of soilscontaining more than 12 percent fines.

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Alignment Load: A nominal load maintained on a tested anchor when theanchor is unloaded. This load is left in the anchor to keep the testing equipment positioned.

Performance Test: Incremental test loading and unloading of a prestressed anchor recording the movement of the tendon at each increment.

Proof Test: Incremental loading of a prestressed anchor recording the movement of the tendon at each increment.

Creep Test: A test to determine the movement of the tendon at constant load during a certain period of time.

Initial Lift-Off Reading: A check made to determine that the actual transfer load is within 5% of the desired transfer load. This check is made immediately after transferring the load to the stressing anchorage.

Lift-Off Test (Creep Check): Checking the force in the prestressed anchor at any specified time with the use of a hydraulic jack.

Creep Movement: The total movement that occurs during the creep test of an anchor.

Residual Movement: The non-elastic (non-recoverable) movement of an anchor measured during a performance test.

Elastic Movement: The recoverable movement of an anchor measured during a performance test.

4.3 ROCK ANCHORS

4.3.1 Description

A prestressed rock anchor is a high strength steel tendon, fitted with a stressing anchorage at one end and a means permitting force transfer to grout and rock on the other end. The rock anchor tendon is inserted into a prepared hole of suitable length and diameter, fixed to the rock, and stressed to a specified force. The basic components of a prestressed rock anchor are the following (Fig. 4.1):

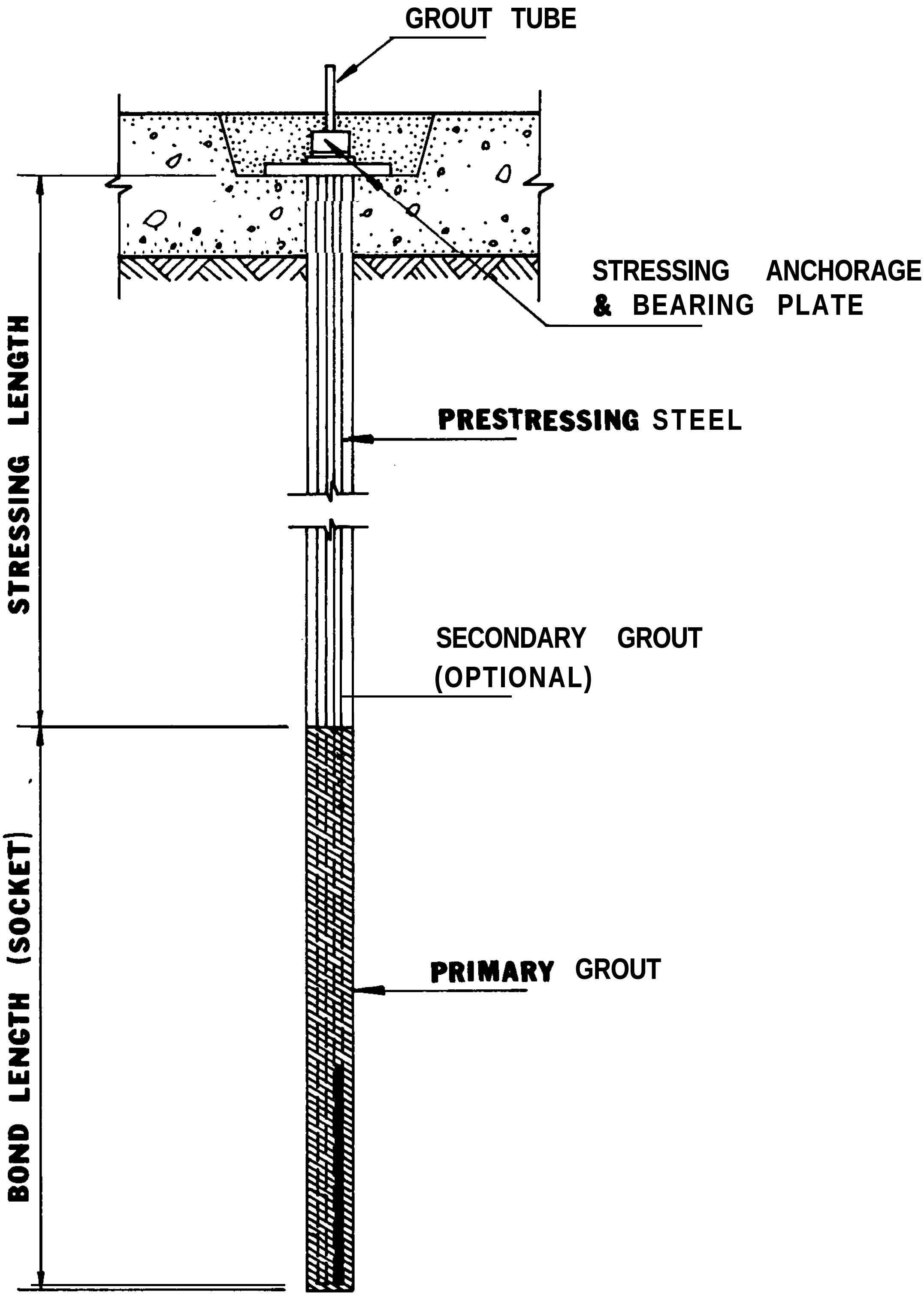

Figure 4.1 — Rock Anchor

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1. Prestressing steel may be single or multiple wires, strands or bars. (Refer to PTI Guide Specifications for Post-Tensioning Materials.) The total length of the rock anchor is composed of two parts:

   a) Bond length (socket), is the portion of the anchor that transmits the force to the surrounding rock.

   b) Free length (stressing length) is the portion of the anchor which is free to elongate elastically during stressing.

2. A stressing anchorage is a device which permits the stressing and anchoring of the prestressing steel under load.

3. A fixed anchorage is at the opposite end of the tendon than the stressing anchorage and is a mechanism which permits the transfer of the induced force to the surrounding grout or rock.

Deformed bars and strand tendons do not normally have fixed anchorages since the anchor load is transferred to the grout by bond.

4. Grout and vent pipes and miscellaneous appurtenances are required for injecting the anchor grout or corrosion protective filler.

Not every rock anchor installation technique requires the use of grout pipes. Grout can be pumped through the drill casing or rods, thereby completely filling the drilled hole.

4.3.2 Design

Rock anchors can be easily installed in downward positions. Horizontal and upward sloping anchors requirespecialized grouting techniques.

4.3.2.1 Site Investigation

Prior to design, a geologic study should be performed by a competent foundation specialist. This study should include an evaluation of site geology and an interpretation of rock core borings. Core drilling to explore the rock quality is an absolute necessity.

Core drilling should be done both in the area where the anchors are to be installed and within the limits of the structure. The rock cores should be available for the anchor contractor to examine.

Any easements required for the installation of anchors should be obtained prior to commencement of the work.

4.3.2.2 System Design

The anchor structure system must be analyzed in order to insure that the anchored structure will function as intended. This analysis must consider system factor of safety, anchor spacing, minimum free length, stability of the anchored structure, overburden pressures, groundwataer conditions, underground obstructions and the geometry of the structure or site.

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4.3.2.3 Factors of Safety

The design load, P, for the anchor equals the maximum anticipated load applied to theanchor times the factor of safety used in the design of the anchored structure. The value of the factor of safety depends upon the type of application, the degree of uncertainty in the design of the structure, and the risk.

The engineer should not compound various factors of safety when designing an anchored structure. The uncertainty and risk associated with the work should only be considered in determining the design load for the anchor. If the engineer applies a separate factor of safety to the applied loads, the design load for the anchor, and the anchor test load, the actual factor of safety for the anchor will have been compounded resulting in an overly conservative design.

4.3.2.4 Anchor Tendon Design

The tendon size is determined such that the design load for the anchor does not exceed 60 percent of the guaranteed ultimate tensile strength (GUTS) of the tendon. The lock off load, which shall be determined by the design engineer, may be larger or smaller than the design load. The recommendations for corrosion protection given in Section 4.6 “Corrosion Protection” shall be considered.

The load in an anchor tendon may either increase or decrease with time depending on the behavior of the structure.

4.3.2.5 Free Stressing Length

The free stressing length should not be less than 15 feet [4.572m].

The minimum stressing length recommended is to prevent significant reductions in transfer load due to stressing anchorage losses or movement.

4.3.2.6 Bond Length

The bond length can be estimated by the following equation:

$$Lb = \frac{P}{\pi \cdot d \cdot \tau_w}$$

Where:

- Lb = bond length
- P = design load for the anchor
- π = 3.14
- d = diameter of the drill hole
- τ_w = working bond stress in the interface between rock and grout

The bond length normally is not less than 10 feet. For normal applications the bond between the tendon and anchor grout is not critical.

Pull-out tests may be used to determine the ultimate insitu bond stress between the rock and the anchor grout. Pull-out tests usually require that the tendon capacity be increased or the bond length reduced in order to fail the anchor. Pull-out test should not be requred if the anchors are tested as described in Section 4.3.7.

The working bond stress used to determine the bond length is normally 25 to 50 percent of the ultimate bond stress.

The ultimate bond stress depends on the:

1. Shear strength of the rock
2. Discontinuities in the rock mass

When selecting the working bond stress the engineer should consider the critical nature of the anchor application, variations in the rock properties, and the installation procedures.

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3. Method of drilling and cleaning the drill hole
4. Drillhole diameter
5. Strength of the grout
6. Grouting procedure

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The drillhole diameter is a function of tendon size and degree of corrosion protection required as well as drilling methods and equipment.

Typical Ultimate Bond Stresses for Rock Anchors

Rock Type	Ultimate Bond Stress Between Sound Rock and Anchor Grout (PSI) [MPa]
Granite & Basalt	250 - 450 [1.72 - 3.10]
Dolomitic Limestone	200 - 300 [1.38 - 2.07]
Soft Limestone	150-200 [1.03 - 1.52]
Slates & Hard Shales	120 - 200 [0.83 - 1.38]
Soft Shales	30 - 120 [0.21 - 0.83]
Sandstone	120-250 [0.83 - 1.72]
Concrete	200 - 400 [1.38 - 2.76]

The typical values shown in this table are not intended for final design. Working bond stresses should be established on a project-by-project basis after review of geologic data and rock cores.

Each rock anchor should be tested in order to verify the load carrying capacity of the anchor and to preload the tendon (see Section 4.3.7 “Anchor Testing and Stressing”).

4.3.3 Fabrication

4.3.3.1 Materials

Anchor material shall be in accordance with PTI Guide Specification for Post-Tensioning Materials.

Anchor material shall consist of either single or multiple elements of the following:

a) Wires conforming to ASTM Designation A421, “Uncoated Stress-Relieved Wire for Prestressed Concrete.”

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b) Strand conforming to ASTM Designation A416, "Uncoated Seven-Wire Stress Relieved Strand for Prestressed Concrete" or ASTM Designation A779 "Uncoated Seven Wire Compacted Stress Relieved Strand for Prestressed Concrete".

c) Steel bars conforming to ASTM Designation A-722, "Uncoated High-Strength Steel Bars for Prestressed Concrete."

Mill test reports for each heat or lot of prestressing material used to fabricate tendons shall be submitted if required by the engineer.

Stressing anchorages shall be capable of developing 95 percent of the actual ultimate tensile strength of the anchor material when tested in an unbonded state.

Couplers for tendon sections shall be capable of developing 95 percent of the actual ultimate tensile strength of the tendon.

Plastic sheathing shall be strong enough to resist damage during handling and installation. It shall be watertight, chemicallystableand nonreactive with concrete, steel and corrosion inhibitor.

Centralizers'shall be capable of positioning the tendon in the drill hole such that a specified minimum grout cover is achieved around the tendon.

Spacers shall be used in multiple element tendons to separate the individual wires, strands or bars of the tendon, so that each element is adequately bonded to the anchor grout.

Corrosion Inhibitor shall be an organic compound with corrosion inhibiting, moisture displacing, and self-healing properties. It shall be chemically and physically stable and nonreactive with concrete, steel, or sheathing.

Anchor grout shall be made using Type I, II, or III Portland Cement conforming to ASTM C-150 specifications.

Chemical additives which can control bleed or retard set may be used with the anchor grout. Additives, if used, shall be mixed in accordance with the manufacturer's recommendations. Expansive admixtures are not recommended for use in anchors except in some applications for secondary grouting and inside a sheathing.

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The use of low relaxation strand is recommended because of its lower long-term losses.

The specifications should define whether the anchorage needs to be of a restressable or destressable type.

Centralizers and spacers are made from steel, plastic or any material non-detrimental to the high strength prestressing steel. Wood spacers should not be used.

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4.3.3.2 Fabrication of Anchors

Tendons shall be either shop fabricated or field fabricated in accordance with approved details, using personnel trained and qualified in this type of work.

Tendons shall be free of dirt, detrimental rust, or any other deleterious substance. The bond length of strands or wires shall be free of grease.

A light coating of rust on the tendon material is normal and will not affect its function. Heavy corrosion or pitting should be cause for rejection of the tendon.

Tendons shall be handled and protected prior to installation in such a manner as to avoid corrosion and physical damage thereto.

Damage like abrasion, kinks, welds and weld splatters, cuts, nicks, will impair the proper performance of the tendon and should be cause for rejection.

Tendons may be either sheathed or unsheathed.

If an unsheathed anchor is used, the stressing length must not be grouted until after the anchor has been tested and the transfer load locked off in the anchor.

If grout is placed around an unsheathed tendon in the stressing length, the tendon will not elongate properly and load will be carried in the stresssing length of the anchor.

For sheathed tendons the sheathing may consist of tubessurrounding individual tendon elements (bar, wire or strand) or a single tube surrounding the elements altogether.

The sheathing material can be. either steel, plastic or any other material nondetrimental to the high strength prestressing steel. Tape may be used to prevent grout from entering under the sheath on individually sheathed elements.

4.3.4 Drilling

Holes for anchors should be drilled where specified and within the tolerances specified by the engineer. The diameter of the drill bit should not be less than 1/8 inch [0.32 cm] smaller than the specified diameter.

Core drilling, rotary drilling, or percussion driling may be employed to drill the anchor hole. Core drilling is generally slow and expensive.

The hole diameter is a function of the grout to rock bond stresses, the diameter of the tendon, the corrosion protection, and the drilling equipment.

Drilling tolerances are controlled by the size of the drill steel or rods, weight of the drill rig, the method of drilling, and the nature of the overburden and rock. Drill holes normally can be started within an angle tolerance of 1 to 3 degrees [0.017 - 0.052 rad] from their planned orientation. A deviation of 1 to 2 inches in 10 feet [25mm to 51mm in 3.05 m] can be maintained with normal drilling methods.

4.3.5 Watertightness

The holes for some or all permanent rock anchors may be tested for watertightness, if specified by the engineer. When specified, the entire hole in rock shall be tested for watertightness by filling it with water and subjecting it to a

Holes are water-pressure tested and consolidation grouted in order to limit grout loss, and to insure proper anchoring of the tendon and corrosion protection. Consolidation grout usually has a water/cement ratio of between 0.45 and

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pressure of 5psi [0.034 MPa] in excess of the hydrostatic head, as measured at the top of the hole. If the free length portion of the hole is in fractured rock or soil, a packer or casing must be used to allow the bond length to be water-pressure tested. If the leakage rate from the hole, over a period of ten minutes, exceeds 0.001 gallons per inch diameter per foot of depth [0.49 ml/mm/m] per minute, the hole should be consolidation grouted, redrilled and retested. Should the second watertightness test fail, the entire process should be repeated.

Temporary rock anchor holes need not be water-pressure tested, since the anchor capacity is verified during testing and corrosion generally presents no problem during the life of the anchor.

Holes adjacent to a hole being tested for watertightness shall be observed during the tests so that inter-hole connections may be detected and sealed.

If artesian or flowing water is encountered in the drill hole, pressure will have to be maintained on the consolidation grout until the grout has initially set.

4.3.6 Insertion and Anchor Grouting

The tendons shall be placed in accordance with approved details or the recommendations of the tendon manufacturer or specialist anchor contractor.

Anchor tendons should not be subjected to unintentional sharp bends. Care must be taken to prevent bending of tendons installed through soil which is likely to settle.

Centralizers should be used with permanent anchors to insure that the tendon does not contact the wall of the drill hole.

If multi-element tendons are used without a fixed anchorage at the lower end, provisions should be made for adequate spacing of the tendon elements to achieve proper grout coverage.

Care shall be taken not to damage the corrosion protective sheathing and/or centralizers during installation of the anchor tendon.

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0.55. It is normal practice to redrill a consolidation grouted hole after the grout has had 24 hours to set up.

Payment for consolidation grouting, redrilling and testing should be based on unit prices since these quantities are unpredictable. Typical payment units should be: water tests (each); cement (94 lb. [42.6kg] bags); redrilling (lin. ft.) [m].

Some anchor grouting techniques allow the grout to be placed under high pressure. If the anchor grout pressure is 25psi [0.17 MPa] or higher than the hydrostatic head at the top of the drill hole, water-pressure tests or consolidation grouting may not be necessary since the hole is sealed during grouting of the anchor.

Grout tubes may be checked with water or compressed air to insure that they are clear.

Centralizersare normally provided at a maximum of 10 ft. [3.05m] center to center throughout the bond length.

Any damage of the corrosion protection shall be repaired following the tendon manufacturer's directions.

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The stressing anchorage shall be installed properly aligned with the anchor tendon axis.

Anchors shall not be used for grounding electric equipment.

Grouting operationsshall generally be in accordance with PTI "Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete."

The grout shall always be injected at the lowest point of the anchor. Grout may be placed prior to inserting the tendon. Grout tubes, casing or drill rods can be used to place the grout.

When sheathed tendons are used, normally the bond length and the free length are grouted simultaneously.

When unsheathed permanent tendons are used, primary and secondary grouting of the drill hole is required. First the primary grout is placed. A means for determining the level of the primary grout must be provided. After the anchor force has been locked-off, the secondary grout shall be injected and the free length grouted.

Any void at the top of the free length shall be filled with grout if the anchor is to be used for a permanent application.

After grouting, the tendon shall remain undisturbed until the grout has cured.

The following data concerning the grouting operation shall be recorded and kept on file by the owner:

- Type of mixer
- Water/cement ratio
- Types of additives (if any)
- Grout pressure
- Type of cement
- Strength Test Samples (if any)
- Volume of first and second stage grout.

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Water separation or bleed in the grout will create a layer of water at the top of any grout stage. Collodial or shear mixers (high energy) and/or a bleed control admixture will help reduce grout bleed.

Sheathed tendons which are grouted over their entire length prior to testing and lock-off may transfer some force to the grout above the bond length.

If sheathed tendonsare used in conjunction with staged grouting, it is common practice to extend the primary grout two feet beyond top of the bond length.

The process of checking the level of the primary grout is expensive. Water flushing to the top of the primary grout is a satisfactory method of determining the level of the primary grout.

Sheathed tendons which are grouted over their entire length at once provide a better and more economical anchor than anchors where the grout is injected in two stages. When stage grouting is performed, a construction joint is created at the top of the primary grout. This joint is often surrounded by poor quality grout and forms a zone where corrosion can attack the tendon.

Strength tests on grout cubes are not normally necessary since the grout is tested as the anchors are tested. The engineer may require grout strength tests if an admixture is used or if the conditions warrant.

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4.3.7 Anchor Testing and Stressing

Each rock anchor shall be tested. The maximum test load shall not exceed 80 percent of the guaranteed ultimate tensile strength (GUTS) of the tendon. The test load shall be simultaneously applied to the entire anchor tendon.

The first three anchors, and a percentage (to be selected by the engineer) of the remaining anchors, shall be performance tested. The remaining anchors shall be proof tested.

4.3.7.1 Performance Tests

The performance test shall be made by incrementally loading and unloading the anchor in accordance,with the following schedule. At each increment, the movement of the tendon shall be recorded to the nearest .001 inches [0.0025 cm] with respect to an independent fixed reference point if possible. The anchor load shall be measured with a pressure guage calibrated with the jack and accurate enough to read 100 psi (690 KPa) changes in pressure. The pump shall be capable of applying each load increment in less than 60 seconds.

A calibrated mastergage shall be kept on the site to periodically check the test gauge.

Movement measurements shall be taken at the Alignment Load and at each load increment.

The increments of load shall be:
(P = Design load for the anchor;)
(AL = Alignment load)

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Anchor testing is normally carried out seven days after grouting for Type I or Type II cements and three days after grouting for Type III cements. At these times, grout with a water/cement ratio of 0.45 will have a compressive strength of about 3500 psi [24.13 MPa].

The percentage of additional anchors performance tested is generally in the range of 1 to 2 percent. The number of performance tests run may be increased, especially when the anchors are being used for permanent applications, when creep susceptability is suspected, or when varying ground conditions are encountered, but normally will not exceed 5 percent of the total number of anchors.

The performance test is used to determine whether the anchor has sufficient load carrying capacity, that the free length has actually been established, and the residual movement (permanent set) of the anchor.

The load is not decreased to zero after each increment of load. A nominal alignment load of between 2 and 10 percent of the design load is maintained in order to keep the testing equipment aligned. The magnitude of this alignment load depends on the type of tendon and the length of the tendon.

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Performance Test

Load	Total Movement	Residual Movement
AL		
.25 P	S_{t1}	
AL		S_{r1}
.25 P		
.50 P	S_{t2}	
AL		S_{r2}
.25 P		
.50 P		
.75 P	S_{t3}	
AL		S_{r3}
.25 P		
.50 P		
.75 P		
1.00 P	S_{t4}	
AL		S_{r4}
25 P		
.50 P		
1.00 P		
1.20 P	S_{t5}	
AL		S_{r5}
.25 P		
.50 P		
1.00 P		
1.20 P		
1.33 P	S_{t6} Testload	
	S_{tn} Readings during load hold	

Adjust to lock-off load

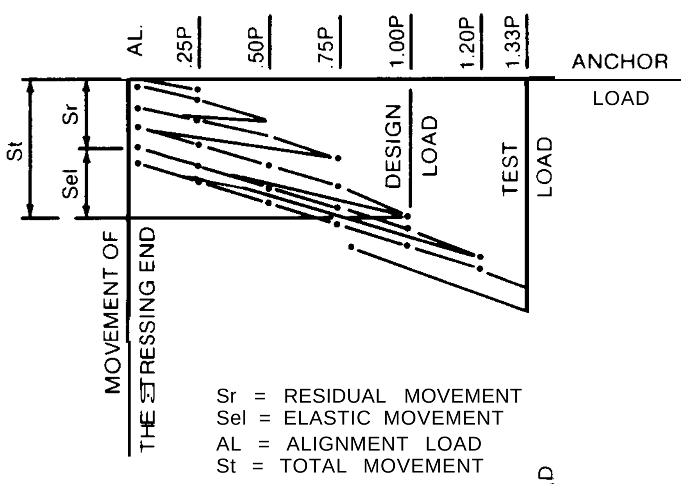
The anchor tendon may be completely unloaded prior to lock-off, if circumstances warrant. Final stressing then does not require further movement readings.

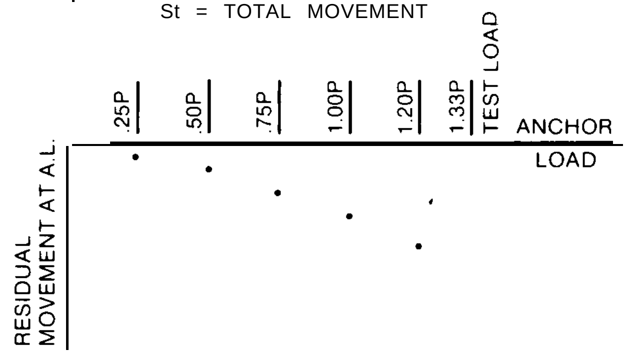
The load shall be held at each increment just long enough to obtain the movement reading. Except for the reading of the residual movement at AL, no movement readings need to be taken during unloading of the anchor.

The test load shall be held or 10 minutes. Total movements with respect to a fixed reference point shall be recorded at 1 minute, 2, 3, 4, 5, 6 and 10 minutes. If the total movement between 1 minute and 10 minutes exceeds 0.04 in. (lmm), the test load shall be held for an additional 50 minutes. Total movements shall be recorded at 15 minutes, 20, 25, 30, 45 and 60 minutes. The Load Hold timeshall start when the pump begins to load the anchor from the 1.20P load to the test load.

COMMENTARY

Performance Test Results





A plot of a performance test is shown above. The upper graph shows the total anchor movement as a function of load. The lower graph shows the residual movement of the anchor as a function of load. The residual movement (permanent set) of the anchor is the non-elastic or unrecoverable movement of the anchor which is measured when the load is released after each loading increment.

The hydraulic jacks used to test anchors may require repumping while holding the test load. The repumping will compensate for small movements or minor hydraulic oil seepage. Care must be exercised not to exceed the test load. Also any changes in the test load due to temperature variations during the load hold period must be corrected.

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The maximum load for performance testing may be increased to 1.50P for special project conditions.

The magnitude of the lock-off load should not exceed 70 percent of the guaranteed ultimate strength of the tendon.

In case it is absolutely impractical or impossible to establish fixed reference points, 3 to 5 anchors shall be performance tested against best possible fixed reference points while increasing the test load to 1.50P. The rest of the anchors shall then be stressed to 1.50P without taking movement readings. In creep susceptible ground, the overload on anchors without fixed reference points shall be increased to 2.0P.

4.3.7.2 Proof Test

The proof test shall be performed by incrementally loading the anchor in accordance with the following schedule. At each increment, the movement of'the tendon shall be recorded to the nearest .001 inches [.0025 cm] with respect to an independent fixed reference point if possible. The anchor load shall be measured with a pressure gage calibrated with the jack and accurate enough to read 100 psi (69 KPa) changes in pressure. The pump shall be capable of applying each load increment in less than 60 seconds. The increments of load shall be:

(P = Design load for the anchor)
(AL = Alignment load)

Proof Test

AL
.25 P
.50 P
.75 P
1.00 P
1.20 P

1.33 P Test Load (10 minute hold)
Adjust to Lock-off load

The anchor tendon may be completely unloaded prior to lock-off, if circumstances warrant. Final

COMMENTARY

An increase in the maximum performance test load above 1.33P, may result in increased tendon steel area and hole diameter and, therefore increased cost.

The maximum load for the performance test verifies the load holding performance of the anchor. The design load includes the engineer's factor of safety; the maximum test load is not a part of the factor of safety. When all production anchors are load tested, maximum test loads of 1.33 x design load, give satisfactory results.

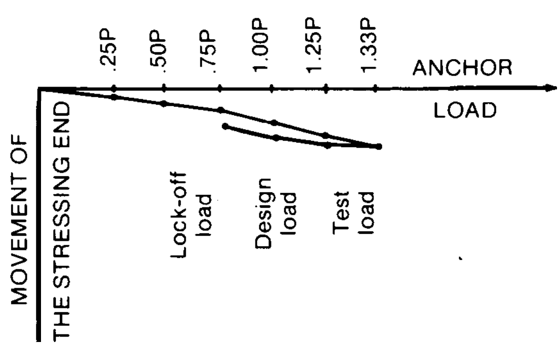
If possible, the regular performance or proof tests should be performed on each anchor.

An increase in tendon capacity may be required in case of a 2.0P test load.

The proof test is a fast, economical test which, when used in conjunction with performance tests, can verify anchor capacity and preload the tendon.

A typical plot of a proof test is shown below. The plot shows anchor movement as a function of load.

Proof Test Results



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stressing then does not require further movement readings.

The load shall be held at each increment just long enough to obtain the movement reading, but not more than 1 minute.

The test load shall be held for 10 minutes. Total movements with respect to a fixed reference point shall be recorded at 1 minute, 2, 3, 4, 5, 6 and 10 minutes. If the movement between 1 minute and 10 minutes exceeds 0.04 in. (1 mm), the test load shall be maintained for 50 more minutes. Additional anchor movements shall be recorded at 15 minutes, 20, 25, 30, 45 and 60 minutes. The period shall start when the pump begins to load the anchor from 1.20P to the test load.

The proof test results should be compared to the performance test results. Any significant variation from the performance test results warrants making a performance test on the next anchor.

4.3.7.3 **Creep Test**

Anchors installed in certain geologic formations may exhibit plastic or cohesive behavior. When performance or proof tests indicate that the rock is sensitive to creep, the rock anchor shall be creep tested as set forth by Section 4.5.5.3 of these Recommendations.

4.3.7.4 Further Checks

Initial Lift-Off Readings:
After transferring the load to the stressing anchorage and prior to removing the jack, a lift-off reading shall be made. The load determined from the lift-off reading shall be within 5 percent of the specified lock-off load. If the load is not within 5 percent of the lock-off load, the end anchorage shall be reset and another lift-off reading shall be made.

Lift-Off Test:
Lift-off tests may be specified by the engineer. The stressing anchorages shall be capable of lift-off in order to check the anchor load. Allowances shall be made for time dependent losses when comparing the lift-off with the previous lock-off load indicated by the initial lift-off reading.

COMMENTARY

The hydraulic jacks used to test anchors may require repumping while holding the test load. The repumping will compensate for small movements or monor hydraulic oil seepage. Care must be exercised not to exceed the test load.

Creep testing of anchors in all but the most decomposed rock formations seldom yields any useful information.

The lift-off tests, if required, are usually done on a random basis and only on permanent rock anchors. The engineer should select those anchors for which lift-off tests are to be made. Lift-off tests can be made a minimum of 24 hours after the transfer load has been locked-off in the anchor. Lift-off tests are normally performed

RECOMMENDATIONS / **COMMENTARY**

within 7 days of when the load was locked-off in the anchor.

For most rock anchor applications, the primary time dependent loss of load is a result of steel relaxation which can be as much as 3 percent of the transfer load in 7 days, depending on the type of steel and stress level. Estimated values of time dependent loss of load can be obtained from the tendon supplier.

4.3.7.5 Cutting of Tendon Protrusions

After an anchor has been accepted by the engineer the portion of the anchor tendon protruding beyond the stressing anchorage may be cut, if not otherwise required. Cutting has to be done according to the tendon manufacturer's recommendations and as approved by the engineer. Care shall be taken not to damage the tendon anchorage.

In general, cutting is done with abrasive blades or a quick acetylene torch cut. When future lift-off tests are planned, an adequate length of tendon must be left protruding over the anchorage to permit jacking.

4.3.8 Acceptance Criteria

The engineer shall investigate the anchor test results and determine whether the anchor is acceptable. An anchor shall be acceptable if:

1. The total elastic movement obtained from a performance test should exceed 80 percent of the theoretical elastic elongation of the stressing length and be less than the theoretical elastic elongation of the stressing length plus 50 percent of the bond length.
2. The total movement obtained from a proof test, measured between 50 percent of the design load and the test load should exceed 80 percent of the theoretical elastic elongation of the free stressing length for this respective load range.
3. The creep rate does not exceed 0.080 inches (2.0 mm)/log cycle during the final log cycle of the performance test, proof test and/or creep test regardless of tendon length and load.
4. The initial lift-off reading shows an anchor load within 5 percent of the specified lock-off load.
5. The lift-off test shows an anchor load within ten percent of the specified transfer load.

If the first anchors installed fail during testing, it will be necessary to modify the design or construction procedures. These modifications may include reducing theanchordesign load by increasing the number of anchors, increasing the bond length, changing the anchor type, or modifying the installation techniques. The engineer and the anchor contractor should work closely together in order to determine the most suitable modifications.

The total elastic movement obtained from a performance test may exceed the recommended upper limit if the upper portion of the bond length is in weak rock or soil.

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COMMENTARY

The engineer shall determine whether an anchor which fails to meet the above minimum acceptance criteria may be incorporated in the work

All test records should be kept on file by the owner.

4.3.9 Long Time Monitoring

The long term performance of permanent anchors can be monitored by using load cells and extensometers. When properly used, these instuments can offer the engineer assurance that the anchors are performing as intended during their service life.

Load monitoring devices, or load cells, have become more reliable in recent years and are being used more frequently for long-term monitoring of anchor loads. It is desirable that a percentage of the anchors installed in special circumstances be equipped with some form of load monitoring device.

Monitored anchors must remain unbonded in the free stressing length. Specifications may require the load in the anchor tendon to be adjustable at a later time.

4.3.10 Records

On completion of the work, the contractor or the owner shall file the final anchor locations with the local government agencies, having jurisdiction, because:

1. Future excavations may unintentionally sever the anchor tendons.
2. Removal of soil may reduce the anchor resistance.
3. Future filling may cause settlements and subsequent anchor bending.

4.4 RESIN ANCHORS

4.4.1 Description

A prestressed resin anchor is a high strength steel tendon, fitted with a stressing anchorage. Deformations on the tendon permit force transfer to the resin grout and rock. The resin is normally packaged in cartridges which are inserted into a prepared hole of suitable length and diameter. Then the anchor tendon is continuously rotated by a power or hand tool and driven into the hole, thus tearing up the cartridges and mixing the contents. After the resin has hardened, the anchor tendon is stressed to a specified force. The basic components of a prestressed resin anchor are the following:

Some techniques call for the use of a liquid pumped resin.

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1. Prestressing steel which in general will be a single high strength steel bar conforming to ASTM Specification A722. Deformations on the bar or a device attached to the botton end of the bar are required to guarantee thorough mixing of the resin and a reliable bond between tendon and resin.
2. Stressing anchoragewhich permits stressing and anchoring the anchor tendon under load. The anchorage shall develop 95 percent of the actual ultimate strength of the tendon.
3. Polyester Resin Cartridge

   The polyester resin cartridge shall contain unsaturated polyester resin and a separated catalyst. The resin shall be a high strength polyester resin, evenly filled with nonreactive, inorganic aggregate of suitable size. The catalyst shall be filled with a nonreactive inorganic filler. Reaction of the catalyst with the resin prior to installation and shredding of the cartridge shall be prevented.

The polyester resin togetherwith a filler material is usually contained in a sausage-shaped tube of polyester films or glass. The catalyst is enclosed in a separate container inside the cartridge, or separated from the resin by a physical and/or chemical barrier.
Liquid resins, where the two components are mixed just before pumping it into the drill hole, are also used.

4.4.2 Resin Properties

Gel Time: Is controlled by the anount and kind of catalyst contained in the cartridge and can range between less than 1 minute to 30 minutes and more. Resin can be classified as:

fast setting:	1 minute or less
medium setting:	5 to 10 minutes
slow setting:	15 to 30 minutes or more

These numbers are based on a 72° [22.2°C] temperature. Warmer temperature will speed up gel time and cooler conditions will lower them. The gel time also depends on the rotational mixing speed.

Cure Time: Also depends on the amount and kind of catalyst as well as temperature, and normally ranges between 5 and 20 minutes or more.

The Specifier as well as the Contractor should consult with the resin manufacturer about actual expected gel and cure times under the specific job conditions for each type of cartridge. A 95° F [35°C] temperature can cut gel times in half and a 45°F [7.2°C] temperature may quadruple standard gel times (72° F) [22.2° C]. For extreme conditions such as temperatures below 45°F [7.2°C], the resin manufacturer should be consulted for special cartridges made to fit the requirements.

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Physical Properties: Resins vary with the manufacturer. Typical properties of the cured resin are:

Compressive strength:	12000 to 18000 psi [82.7 to 124.1 MPa]
Tensile strength:	3000 to 6000 psi [20.7 to 41.4 MPa]
Shear strength:	2500 to 5000 psi [17.2 to 34.4 MPa]

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For detailed information, the resin manufacturer should be consulted. Normally, the physical properties of the resin do not influence the resin anchor design, as the compressive and shear strength of the rock will be the governing factors.

Viscosity: Resin cartridges are available in different viscosities. Viscosity of the resin also depends highly on temperature. Viscosity is reduced by increased temperatures.

Special job conditions may require a resin with non-standard viscosity. For easier installation of long fully encapsulated anchors or for low temperature conditions, the use of a low viscosity resin may be of advantage. To prevent flowout of resin from overhead anchors, a high viscosity resin may be used.

Shelf Life: The shelf life of the resin is stated by the manufacturer with each shipment. Expired resin cartridges shall not be used unless proper performance has been established. The cartridges should be stored in a dry, cool ventilated placeawayfrom direct sunlight. Extreme temperatures and overstacking should be avoided.

Depending on the manufacturer and type of cartridge, the shelf life normally ranges from 6 months to 1 year.

4.4.3 Design Criteria

Resin anchors can be installed in upward, downward, or horizontal position in all types of rock. Resin can provide a fast and reliable anchorage even in weak rock. Standing fresh or salt water in the drill hole does not effect the resin or the curing process but may cause deterioration of the hole.

Selection of the resin cartridge diameter shall be based on the manufacturer's suggested relation between drill hole and bar diameter. For best results, the difference between the hole diameter and bar should be kept to a minimum.

The volume of resin in the cartridges shall be such that the resin will flow towards the drill hole opening during the installation of the bartendon.

The amount of bond length depends primarily on thecompressivestrength of the rock. Additional factors affecting the bond length are: the conditions of the drill hole wall, diameter relationship between bar and drill hole, resin type, bar deformations and proper anchor installation (that is, resin mixing procedure). Charts by the resin manufacturers for the resin anchorage strength versus bond length help in establishing the required anchorage length. However, all

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design assumptions shall be verified by field tests.

Prestressed resin anchors may either have a resin point anchorage with resin only in the bond length or may be fully encapsulated. In the latter case, slow setting resin cartridges are placed after the fast setting ones. The anchor tendon must be prestressed after the fast setting resin has cured but before the slow setting resin can cure. This timing is critical to insure load transfer to the bond length only. Fully encapsulated resin anchors are used when the anchor tendon shall be bonded also in the free stressing length and/or for corrosion protection in the free stressing length.

Corrosion Protection: During installation the anchor tendon is coated with the polyester resin which provides a good protection against corrosion. For permanent anchors the protection by the resin alone may not be sufficient. Further details on corrosion protection are given in Section 4.6 "Corrosion Protection."

4.4.4 Installation

4.4.4.1 Drilling

Holes shall be drilled to the specified depth, angle and diameter using rotary or rotary percussiveequipment. Drill holesshall be cleaned of cuttings and debris by air or water. The diameter of the hole shall be checked with a gauge for compliance with the specifications.

4.4.4.2 Placing of Cartridges

After cleaning the hole, the resin cartridges with the gel time and viscosity specified shall be inserted into the hole taking care not to rupture the skin before the tendon is installed. The specified number of fast setting cartridges shall be installed first and then, if applicable, the specified number of slow setting capsules. For almost horizontal or upward sloped holes a charging stick shall be used to push the cartridges into position. A plastic cap, or equivalent, shall be used on upwards-sloped drill holes to keep the cartridges in place. Cartridges which are older than the stated shelf life shall not be used.

The safety precautions as prescribed by the manufacturer, shall be observed.

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For resin anchors with a fully encapsulated length exceeding 20 ft. [6.1 m], installation may become difficult. In such case the resin manufacturer should be contacted.

This applies for anchors with full encapsulation.

In most cases, checking of the diameter on the first three holes for any rock formation on a project will be sufficient.

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4.4.4.3 Installation of the Anchor Tendon

The anchor tendor shall be inserted until it contacts the first cartridge. At this point, the tendon shall be rotated and advanced down the hole at a penetration rate of approximately 2 to 5 inches [5.1cm to 12.7cm] per second. When the tendon reaches its final position, spinning shall be continued for 15 to 30 seconds or a minimum of 60 revolutions to insure complete mixing of the resin.

Care shall be taken to stop mixing before the gel time of the fast setting resin has expired. Excess resin which flows from the hole shall be cleaned up promptly if required.

The anchor plate shall then be placed such that uniform bearing is provided perpendicular to the tendon axis. A grout pad underneath the plate may be required for bearing on uneven rock faces. Deviations from the right angle between plate and tendon of up to 20° (0.35 rad] are permitted if the deviations are compensated for by suitable wedge washers between plate and anchor nut. At the stressing end, the tendon shall be free of resin, dirt or grout to permit engagement of the anchor nut or other gripping device.

4.4.4.4 Stressing

Stressing shall not start before the resin in the bond length has acquired the specified design strength and shall be finished before any slow setting resin used in thefreestressing length has solidified. Stressing and testing shall be accomplished with hydraulic rams following the applicable portion of the guide lines given in Section 4.3.7 “Anchor Testing and Stressing.” Adequate support shall be provided for the stressing ram to avoid tilting and introducing bending into the anchor tendon. Test load and lock-off load shall be according to the project specifications.

4.5. SOIL ANCHORS

4.5.1. Description

A prestressed soil anchor is a high strength steel tendon, fitted with a stressing anchor at one end and an anchor device permitting force transfer to the soil on the other end. These anchors, which are used in clay, silt, sand or gravel, are inserted in a prepared hole that is drilled or driven into the ground.

COMMENTARY

The equipment used for drilling is also normally used for spinning the anchor tendons through the resin cartridges.

Spinning at less than 60 RPM may cause incomplete mixing of the resin. Difficulties in maintaining minimum RPM may be expected when exceeding a 20 ft. [6.1m] encapsulation length.

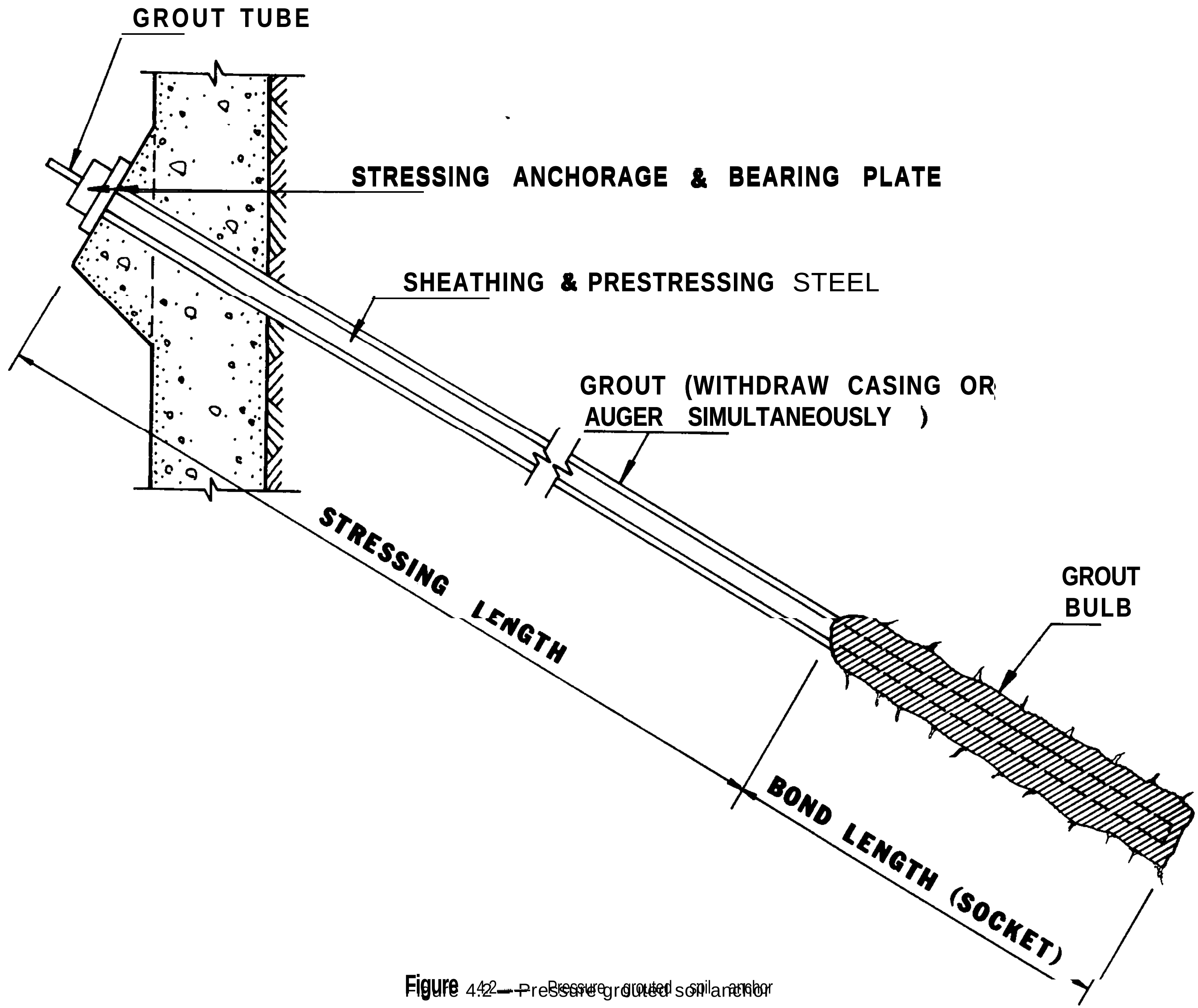

Figure 4.2 — Pressure grouted soil anchor

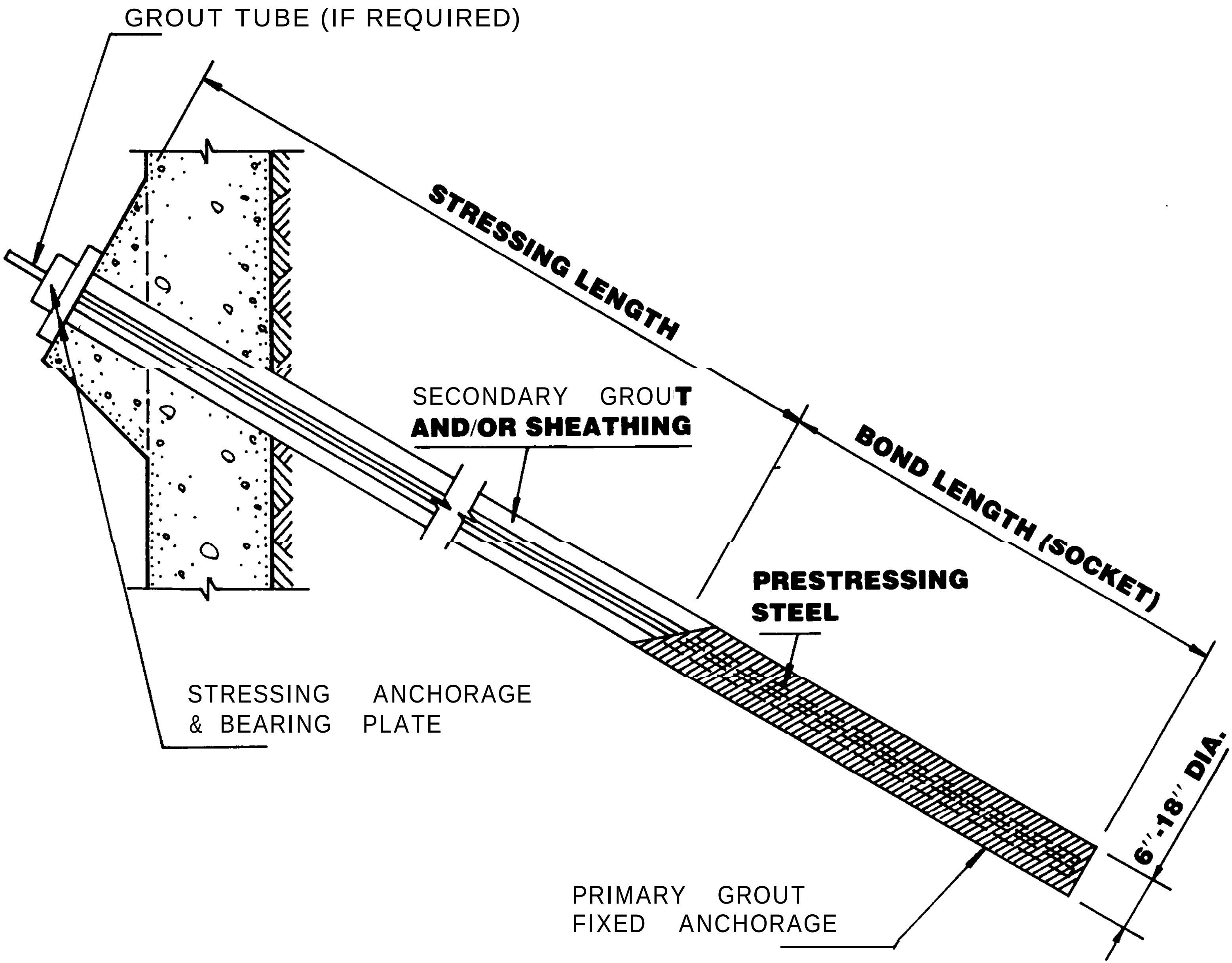

Figure 4.3 — Augered soil anchor

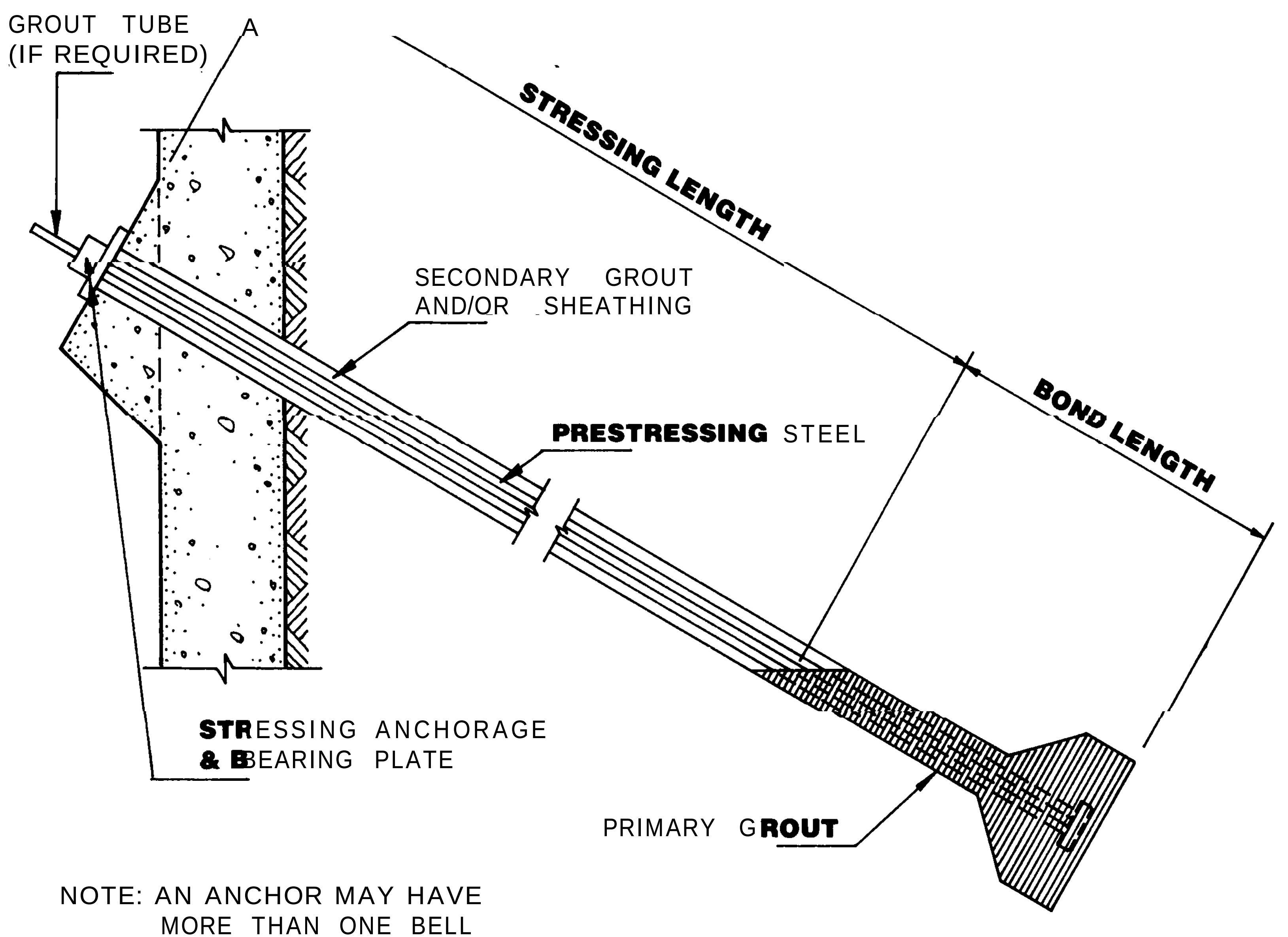

Figure 4.4 — Belled soil anchor

RECOMMENDATIONS

COMMENTARY

There are two basic types of soil anchors — those that rely on friction between the drilled borehole walls and the anchor grout and those that rely on an enlaraged pressure grouted bulb or an underreamed bulb to provide resistance to pull-out. A soil anchor may also utilize a combination of friction and an enlaraged bulb.

Pressure bulb anchors *(Fig.* 4.2) are constructed by driving or drilling a flush jointed casing into the ground. The tendon is then placed and pressure grouting is performed as the casing is withdrawn to form an enlarged grouted bulb in the bond length portion of the anchor.

Friction type anchors *(fig.* 4.3) consist of a uniform sided, straight shaft which is drilled into the ground, the tendon placed into the hole and grout or concrete gravity placed without pressure to anchor the tendon.

Soil anchors may be classified in accordance with the table below:

	Friction	Pressure Bulb	Underream	Combination of Friction & Pressure Bulb	Post Groutina
Drilled Casing					
Cohesive	C	NA	C	C	C
Non-Cohesive	C	C	NA	C	P
Driven Casing					
Cohesive	C	NA	NA	NA	C
Non-Cohesive	C	C	NA	C	P
Hollow Stem Auger					
Cohesive	C	NA	NA	NA	NA
Non-Cohesive	C	P*	NA	P*	NA
Conventional Auger					
Cohesive	C	NA	C	C	C
Non-Cohesive	P**	P***	NA	C**	C
Open Hole Rotary					
Cohesive	C	NA	C	C	C
Non-Cohesive	P**	P***	NA	C**	C

C — *Common*
P — *Possible*
NA — *Not Applicable (Based on the current sate-of-the-art).*
* — *Pressure grouting can be carried out to a limited extant. High pressures may force grout up the auger flighting.*
** — *Conventional auger and open hole rotary drilling in non-cohesive soil can be carried out only with the use of drilling fluids such as bentonite, that will help to hold the hole open.*
*** — *Pressure grouting can be carried out only by grou ting a post-grouting tube in the hole with the tendon and subsequent post grouting of the anchor.*

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Belled soil anchors *(Fig.* 4.4) are constructed by use of augers with an attachment that permits belling or enlarging the bottom of the hole. More than one bell may be used in cohesive soils. Augured holes may vary from 6 to 24 inches [152 to 610mm] in diameter, and lengths may be as much as 100 ft. [30.48m].

Post grouting methods are a perforated grout line and/or a series of valves in the bond length through which grout can be repeatedly injected under very high pressure after the primary grout has set. Local grout bulbs are formed which increase the carrying capacity of the soil anchor.

Subsequent to placement of anchor grout and after the necessary curing period, the soil anchor is proof loaded and then anchored at a specified force. The basic components of a prestressed soil anchor are identical to those of a prestressed rock anchor (see Section 4.3.1 "Description").

4.52 Design Considerations

452.1 Site Investigation

Prior to design, a soils investigation should be performed by a competent foundation specialist. This study should include an evaluation of site geology and an interpretation of the types of soils encountered. As a minimum, standard penetration tests and sampling should be performed at five-foot [1.5m] intervals within each hole. The samples should be preserved and made available for inspection to the anchor designer and/or contractor. Some gradation tests should be performed, especially on fine grained, non-cohesive soils. In cohesive soils, moisture content and Atterberg limits should be determined.

Soil sampling should be done both in the area where the anchors are to be installed and within the limits of the structuare. The spacing of the soil boringsshould be in the range of 50to 100 ft. [15.2 to 30.5 m] depending on the uniformity of the soil conditions. Borings should be carried to at least the expected depth of the soil anchors.

4.5.2.2 System Design

The anchor structure system must be analyzed in order to ensure that the anchored structure will function as intended. The overall stability of the earth mass as well as the assumed failure plane must be analyzed so that the bond length of the anchor is started at least five feet [1.5m] beyond the failure plane. The type of foundation, and nearness and susceptibility of movement of adjacent buildings must be taken into consideration.

Appropriate safety factors and soil pressure loadings must be determined by the engineer or

In most instances, it may be possible to install either a large number of low capacity anchors (30-50 tons design load) [267 to 445 kN] or a smaller number of high capacity (70-150 tons design load) [623 to 1334 kN] soil anchors. The specialty contractor installing the anchors will usually be in the best position to determine the economics of the anchor system chosen. However, the final choice will have to take into account the design economics of the overall structure.

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by the staff of a competent specialty contractor. The vertical component of an inclined anchor must be considered as well as deflection of the structure between anchor points.

The interaction of groups of anchors must be considered when the spacing is close.

4.5.2.3 Factors of Safety

The design load, P, for the anchor equals the maximum anticipated load applied to theanchor times the factor of safety used in the design of the anchored structure. The value of the factor of safety depends upon the type of application, the degree of uncertainty in the design of the structure, and the risk.

4.5.2.4 Anchor Tendon Design

The tendon size is determined such that the design load for the anchor does not exceed 60 percent of the guaranteed ultimate tensile strength (GUTS) of the tendon.

The recommendations for corrosion protection given in Section 4.6 "Corrosion Protection" shall be considered.

4.5.2.5 Free Stressing Length

Minimum stressing lengths of 15 to 25 ft. [4.6 to 7.6 m] are recommended for all soil anchors.

4.5.2.6 Bond Length

The design of the bond length is dependent on the following variables:

1. Method of drilling
2. Soil Properties
   a) Permeability
   b) Density
   c) Angle of Internal Friction ϕ
   d) Shear Strength
   e) Degree of Consolidation
3. Overburden Pressure
4. Hole diameter or designed variations in hole diameter (i.e. underreaming)

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Research to date has shown that at a spacing of six (6) times the grout bulb diameter, there should be no interaction.

The engineer should not compound various factors of safety when designing an anchored structure. The uncertainty and risk associatred with the work should only be considered in determining the design load for the anchor. If the engineer applies a separate factor of safety to the applied loads, the design load for the anchor and the anchor test load, the actual factor of safety for the anchor will have been compounded resulting in an over conservative design.

The minimum stressing lengths recommended are necessary to prevent significant reductions in transfer load due to stressing anchorage losses or movements.

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5. Grout Pressure and Grout Take
6. Number of grouting cycles (postgrouting)

Minimum bond lengths of 15 ft. [4.6 m] are recommended for all types of soil. In general, cohesive soils will require longer bond lengths than cohesionless soils.

The existing theoretical and empirical methods for predicting anchor capacity should only be used for preliminary design estimate purposes. Final anchor capacity shall be verified by field testing each anchor.

A Cohesive Soils

1. Straight Shaft: The bond length can be estimated by the following equation:

$$Lb = \frac{P}{\pi \cdot d \bullet \tau_w}$$

Where:

Lb = bond length
P = anchor design load
d = diameter of the drill hole
τ_w = working shaft friction between the soil and the grout

Soil anchors in soft to medium cohesive soils (standard penetration resistance less than 15 blows per foot) [49 blows per m] are not recommended without extensive testing including long term creep tests. Minimum overburden depths of 15 feet [4.6m] are recommended.

2. Underreamed Shafts: The design of the bond length of underreamed anchors depends on a combination of shaft friction in the non-underreamed straight shaft portion of the anchor and a bearing capacity of the enlarged underreamed portion. The shaft portion may be treated as above. The working bearing capacity of the underreamed portion of single belled anchors may be determined by applying a bearing factor of 7 to 9 to the undrained shear strength of the soil times the net bearing area of

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Normally, the bond length for soil anchors is in the range of 20 to 30 ft. (6 - 9m).

Field testing is the only reliable method of assuring that the design load can be safely carried by the anchor.

The value of τ_w is sometimes expressed as a function (typically 0.3 to 0.5) of the undrained shear strength of the soil. Alternatively and more commonly, an empirical value of 700 to 1400 psf [0.035 to 0.069 MPa] is used as a working stress for shaft friction (τ_w) in medium stiff to very stiff cohesive soils.

Post grouting can increase thecarrying capacity by 20 to 50 percent or more.

Tested values for soil anchors constructed in the U.S. have varied considerably. The following table is a summary of some of the maximum loads achieved.

Soil	Avg. Overburden Depth (ft.) [m]	Bond Length (ft.) [m]	Max. Test Load (tons) [kN]
HOLLOW STEM AUGER 10" ϕ [25.4cm] shaft Soft Silty Clay	17 [5.2]	42 [12.8]	53 [472]
Silty Clay	20 [6.1]	35 [10.7]	55 [489]
UNDERREAMED Residual Mica Schist (18" [45.7cm] ϕ shaft single 40" [101.6cm] ϕ bell)	20 [6.1]	15 [3.8]	75 [667]
HARD CLAY TILL (5⅝" [14.0cm] ϕ shaft multi-belled - 4 bells - 12" [30.5cm] ϕ	60 [18.3]	40 [10.2]	220 [1957]

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the belled diameter. The net bearing area of the bell is the maximum bell cross sectional area minus the shaft cross sectional area. For multi-underreamed anchors, the straight shaft friction, the bearing area resistrance and the friction resistance of the larger shaft formed by the bells are included when calculating anchor capacity.

B Cohesionless Soils

1. Pressure Grouted: Small diameter pressure grouted anchors in cohesionless soil develop far in excess of the load that would be expected from applying conventional soil mechanics theory to this design. Pressure grouting can be achieved either through the casing or by postgrouting techniques. Overburden pressure, angle of internal friction, density and grain size of the soil particles are very significant factors in development of load carrying capacity. The hole diameter, grout pressure, grout take and method of drilling may also affect load carrying capacity but usually to a lesser extent. The resistance of this type anchor to pullout develops from a combination of end bearing and shaft friction of the grout bulb.

As a rough guide, the following may be used for calculating the working load for small diameter (3" to 6") [76-152mm] pressure grouted (60 to 350 psi) [0.41 to 2.41 MPa] anchors installed in cohesionless soils with an average overburden pressure of 20 ft. [6.1m] or more.

Soil	Ultimate Load
Clean Sand/Gravel	10 - 20 kips/ft [146 to 292 kN/m]
Clean Med.-Coarse Sands	7-l 0 kips/ft. [102 to 146 kN/m]
Silty Sands	5-10 kips/ft. [73 to 146 kN/m]

2. Non-Pressure Grouted or Low Pressure (60 psi [0.41 MPa] or less): This type anchor may be treated in the same manner as a straight shaft anchor drilled in cohesive soils except that the allowable working shaft friction will commonly be much higher. A value for τ_w of 10-20 psi [0.069 to 0.138 MPa] (1400-2800 psf) may generally be used when the average overburden pressure is 20 ft [6.1m] or more.

C Post Grouted Anchors

This type of anchor is becoming more common and is generally used in cohesive or mixed type soils to increase the load carrying capacity of the anchor. Some installations have been

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Soil	Avg. Overburden Depth (ft.) [m]	Bond Length (ft.) [m]	Max. Test Load (tons) [kN]
ROTARY DRILLING WITH CASING			
Water flush, Pressure Grouted (5-6" [12.7 to 15.2cm] ϕ shaft			
Medium Dense Silty Sand	25 [8.2]	30 [9.8]	124 1103 l
Medium Dense Silty Sand	40 [13.11	30 [9.8]	100 890]
Loose to Medium Dense Silty Sand	35 [11.51	30 [9.8]	80 712]
Dense Sand and Gravel	60 [19.71	40 13.1]	180 16011
Dense Glacial Till	60 [19.71	40 13.1]	180 16011
Dense Glacial Till	60 [19.71	40 13.1]	240 2135]
DRIVEN CASING PRESSURE GROUTED (3" [7.6cm] ϕ shaft)			
Medium Dense Sand	20 [6.6]	20 [6.6]	94 [836]

Soil	Avg. Overburden Depth (ft.) [m]	Bond Length (ft.) [m]	Max. Test Load (tons) [kN]
POST GROUTED			
Stiff Silty Clay	25 [8.2]	20 [6.6]	105 [934]

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carried out in cohesionless soils with varying results. The load carrying capacity of straight shafted anchors in mixed and cohesive soils can generally be increased by post grouting by 20 to 50 percent or more.

4.5.3 Fabrication

The recommendations given in Section 4.3.3 "Rock Anchors - Fabrication" also apply for Soil Anchors.

Soil Anchors should always be sheathed in the free stressing length to prevent contact of the anchor tendon with the drill hole wall.

4.5.4 Drilling and Installation

4.5.4.1 General

The engineer shall identify special site conditions which may influence choice and method of equipment.

Drilling may be by any appropriate method and equipment the specialist foundation contractor chooses for the particular soil or anchor method. However, the overburden should be cased when there is danger of subsidence around existing structures. Alignment tolerances of 2 to 3 degrees [0.035 to 0.052 rad] for angle of inclination are recommended. Any significant change in method of drilling will require repeating of performance and acceptance testing.

The provisions given in Section 4.3.6 "Insertion and Anchor Grouting" for rock anchors also apply for Soil Anchors.

4.5.4.2 Drilled or Driven Casing

In general, drilled or driven casing of 3" to 6" [76 - 152mm] diameter or hollow stem augers are used in non-cohesive soils. Casing is usually driven by an airpowered "Air Track" or Wagon Drill with a drifter. Eight or ten feet [2.44 or 3.05m] lengths of threaded or welded casing are added as the hole is advanced. The end of the casing string is capped with a loose fitting point that keeps the casing clear of cuttings. Drilled casing is rotated into the ground using rotary hydraulic rigs. An inside drill bit and rod using water or air flush is used to remove cuttings from the casing.

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After the hole is drilled, a string of flush jointed casing clean of debris is in place for the full depth of the hole. The casing is first tremied full of neat cement grout and the tendon is then inserted through the grout to the bottom of the hole. Alternatively, the tendon may be placed into the open casing with a grout tremie tube attached and grouted full by means of this tremie tube. After grout filling is complete, the casing is capped and additional grout is pumped into the casing. This additional grout is forced out the bottom of the casing into the surrounding soil. When the grout pressure rises to the required level, withdrawal of the casing is begun. The entire bond length of the anchor is pressure grouted while the casing is withdrawn. After the casing is pulled up to the bottom of the stressing length, pressure grouting is discontinued. The casing may be kept topped-off and the stressing length gravity grouted for corrosion protection.

COMMENTARY

The quantities of grout used and the size of the pressure bulb vary depending on drilling method and permeability of the soil. In general, a drilled open end casing will “take more grout and result in a larger pressure bulb than a driven closed end casing. (Typical grout takes 1/2 to 1 bag per foot vs. 1/4 to 1/2 bag per foot. Typical diameter 8” to 18” vs. 4” to 8”) [Typical grout takes 1.64 to 3.28 bags per m vs. 0.82 to 1.64 bags per m. Typical diameter 203mm to 457mm vs. 102mm to 203mm]. However, load carrying characteristics of the two types of anchor have proven to be very similar.

Typical grouting pressures vary from 60 to 350 psi [0.41 to 2.41 MPa]. Large amounts of grout in horizontally stratified soil may cause heaving problems. In these conditions grout pressures may have to be held to 2 psi/ft. [0.045 MPa/m] of overburden pressure or the quantity of the pumped grout must be limited.

Only sheathed anchor tendons may be grouted in the free stressing length prior to’ stressing.

454.3 Hollow-Stem Augers

Continuous flight, hollow-stem augers are usually 8” to 15” [20.3 to 38.1 cm] in diameter with a 2%” to 4” [6.4 to 10.2cm] hollow center. In most instances, the tendon is inserted into the hollow-stem auger flighting prior to drilling of the hole. A detachable bit is then secured over the bottom hole of the auger flighting and drilling commences. The auger is rotated and advanced into the ground to the required depth, the bit is removed, usually by reverse rotation of the auger, and grouting commences. Because of the large volume of grout required, a 2½:1 sand cement grout with fluidifiers or water reducing agents is used. The grout travels through the drill head and down the hollow-center auger and is forced out at the bottom. Simultaneously, the auger is withdrawn, either with slight rotation or no rotation. It is important to insure that the bottom of the auger is kept immersed in the grout during withdrawal so that there are no discontinuities in the shaft. The stressing length may be grouted or a soil backfill can be placed into the hole by reverse rotation of the auger.

Anchors placed by hollow-stem augers on a flat angle (less than 30° [0.52 rad]) in cohesionless soils, or under the water table, are not recommended.

Anchors placed by the hollow stem auger method do not require centralizers if the hole is maintained full of a stiff mix grout (8-10 in. slump/200-250mm slump) during extraction of the auger.

A limited amount of pressure grouting can be performed with hollow-stem auger drilling; however, excessive pressure may cause the grout to run up the auger flighting. The resultant hole diameter will be approximately the same size as the outside diameter of the auger (8” to 15”) [203 - 381 mm] except in very permeable soil where it will be slightly larger.

Care must be taken that the grouted cylinder in the free stressing length of the anchor does not transfer load to the structure or a true load test cannot be carried out. This may be achieved by terminating the grouting operation at least one foot (1’) [0.305m] from the structure.

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4.5.4.4 Augered or Open-Hole Rotary Drilling

Open hole rotary and auger drilling, sometimes with belling at the bottom of the anchor, are employed in cohesive soils only. Hole sizes may range from 6 to 24 inches [152 to 610mm] or more in diameter. The diameter of the bell is generally about twice the diameter of the hole.

Auger Drilled — Friction type Anchors (Fig. 4.3) are installed using either continuous flight augers or short augers on a Kelly Bar type of machine. These anchors differ from those drilled in cohesionless soil only in placement. The auger is withdrawn before grouting, and pressure grouting is not used.

Belled Type Anchors (Fig. 4.4) are drilled either by a Kelly Bar type machine using augers and a standard caisson belling bucket or the drilled casing method which employs a small air or mechanically activated underreamer. The cuttings are removed by air or water flushing.

After the hole is drilled to final depth, the tendon is placed in the hole and grout or concrete is placed in the hole by gravity. If the hole is full of drill fluid or water, grout is tremied from the bottom of the hole until the bond length is completely grouted. The stressing length is either grouted or, for temporary anchors only, backfilled with soil.

Centralizers should be used to keep the tendon in the approximate center of the hole.

4.5.4.5 Post Grouted Anchor

This type anchor may be installed in the hole by any of the above described methods, but a sealed, perforated grout tube must be placed in the hole simultaneously with the tendon. After the initial grout has set (24 to 48 hours), additional grout is introduced through the perforated grout tube. The high pressures of this post grouting (300-900 psi) [2.07 to 6.20 MPa] crack the initial grout column and allow the grout to penetrate or compress the soil forming enlarged bulbs. The initial grout in the upper shaft of the anchor forms a seal that keeps the new grout confined and under pressure. If the post grouting tube is flushed after the operation the post grouting process can be repeated several times, if necessary, until the desired anchor capacity is achieved.

The perforated grout tube is sealed by rubber sleeves or one-way valves.

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COMMENTARY

4.5.5 Anchor Testing and Stressing

Each soil anchor shall be tested as set forth by Section 4.3.7 "Anchor Testing and Stressing" of these Recommendations.

When bond lengths are in fine-grained soils, the first two anchors or more, if specified by the engineer, shall be creep tested.

Creep tests should be performed on projects where the anchor is made in soils with a plasticity index greater than 20. Anchors installed in these types of soils have demonstrated creep sensitivity.

4.5.5.1 Performance Test

The performance test shall be as set forth in Section 4.3.7.1 of these Recommendations.

The number of performance tests may be increased, especially when soil anchors are being used for permanent applications or when varying soil conditions are encountered.

4.5.5.2 Proof Test

The proof test shall be as set forth in Section 4.3.7.2 of these Recommendations.

4.5.5.3 Creep Test

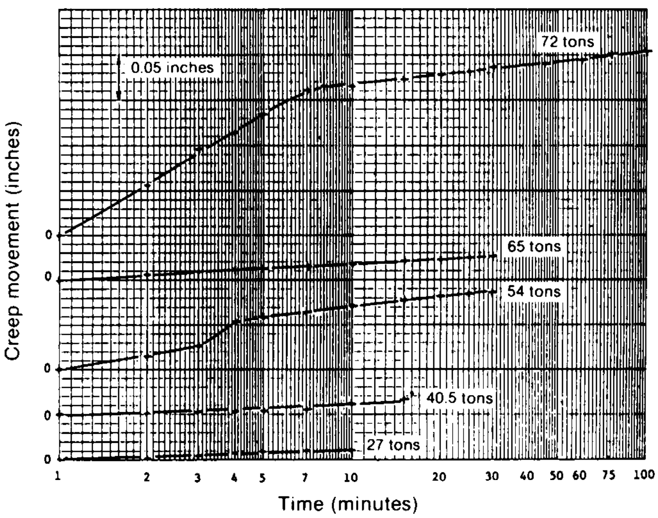
The creep test shall be made by incrementally loading and unloading the tendon in accordance with the schedule given in Section 4.3.7.1 of these Recommendations. At each total movement reading, the load shall be held constant in accordance with the following schedule. A load cell shall be used to monitor the constant load.

CREEP TEST

Load	Observation Period (min) For Tempory Anchors	Observation Period (min) For Permanent Anchors
AL		
.25 P		10
.50 P	10	30
.75 P	15	30
1.00 P	30	45
1.20 P	30	60
1.33 P	100	300

The times for reading the total movement during an observation period shall be 1 minute, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45, 60, 75, 90, 100, 120, 150, 180, 210, 240, 270, 300 minutes.

Typical Creep Movement Plot



Note: 1 in. = 25.4 mm, 1 ton = 8.9 kN

Creep test performed in a single-underreamed tieback installed in a hard-to-very-hard clay.

The family of creep curves are plotted for each increment of load creep as a function of the log (base 10) of time in minutes. The creep rate is the slope of the creep curve over the final time decade of the observation period.

The hydraulic jacks used to test anchors may require repumping during the creep test. The repumping will compensate for small movements or minor hydraulic oil seepage. Care must be exercised not to exceed the test load.

RECOMMENDATIONS

The observation period shall begin when the pump starts to load the anchor from the next lower increment.

The creep rates shown by the family of curves in the Creep Movement Plot should be reviewed and judged by the engineeras reasonable and as showing no indication that future unacceptable movement or creep failure is probable.

If the creep rate exceeds 0.08 inches/logarithmic cycle, the observation period may be extended in an attempt to determine if the creep rate will diminish to 0.08 inches/logarithmic cycle of time.

4.5.5.4 Further Checks

Further checks shall be as set forth in Section 4.3.7.3 of these Recommendations.

4.5.6 Acceptance Criteria

The engineer shall investigate the anchor test results and determine whether the anchor is acceptable. An anchor shall be acceptable if:

1. The total elastic movement obtained from a performance test exceeds 80 percent of the theoretical elastic elongation of the stressing length.
2. The total movement obtained from a proof test, measured from 50 percent of the design load to the test load exceeds 80 percent of the theoretical elastic elongation of the free stressing length of this load increment.
3. The creep rate does not exceed 0.080 inches (2.0mm)/logarithmic cycle of time during the final log cycle of the performance test, proof test and/or creep test, regardless of tendon length and load.
4. The initial lift-off reading shows an anchor load within 5 percent of the specified lock-off load.
5. The lift-off test shows an anchor load within ten percent of the specified transfer load.

The engineer shall determine whether an anchor which fails to meet the above minimum acceptance criteria may be incorporated in the work.

4.5.7 Long-Time Monitoring

The provisions given in Section 4.3.9 "Long-Term Monitoring" for rock anchors also apply for soil anchors.

COMMENTARY

If the first anchors installed fail during testing, it will be necessary to modify the design or construction procedures. These modifications may include reducing the anchor design load by increasing the number of anchors, increasing the bond length, changing the anchor type, or modifying the installation techniques. The engineer and the anchor contractor should work closely together in order to determine the most suitable modifications.

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4.5.8 Records

The provisions given in Section 4.3.10 “Records” for rock anchors also apply for soil anchors.

4.5.9 Destressing of Anchors

Anchors to be destressed at a later time shall be designed and installed such, that destressing can be performed in a safe and easy manner. An anchor tendon may be destressed by either the use of hydraulic jacks or by carefully torch cutting the tendon elements. The destressing operation shall follow an approved procedure and sequence.

Commentary: In some applications, anchors are used as a temporary support of a structure only and for certain reasons may need to be destressed after installation of the permanent support system. The design of a structure where the anchors will be destressed shall consider changes in stresses resulting from destressing, watertightness of the structure, access to the anchorage for destressing and structural movements caused by destressing. Removal of an anchor tendon requires special provisions and it is normally not done. If the tendon must be removed, only the free stressing length of the tendon shall be removed.

4.6 CORROSION PROTECTION

4.6.1 Introduction

Prestressed rock and soil anchors shall be protected against corrosion to suit the application on which they are employed. The type and extent of corrosion protection shall be based on whether the anchor is classified as temporary or permanent (see definitions), the nature of the anchor's environment which would cause corrosion of the prestressing steel and/or anchorage components, and the consequences which would result from a tendon failure.

Adequate testing must be performed to measure the aggressiveness of the anchor environment, especially if field observations indicate corrosion on existing structures or the existence of stray DC currents. The most common and simplest tests are electrical resistivity, pH, sulfides, and sulfates. Critical values if an aggressive anchor environment exists are as follows:

Test	Critical Values
Resistivity	below 2000 ohm-centimeters
pH	below 5.0
Silfides and Chlorides	any presence indicated
Sulfates	above 1000 ppm

Permanent anchors placed in environments where anyone of these tests indicate critical values, must be encapsulated over their full length.

Commentary: Temporary anchors installed in such corrosive environment may require special corrosion protection as well.

The presence of other corrosive elements, such as nitrates, rhodanides, etc., may also be critical for the anchor tendon and require its full encapsulation.

RECOMMENDATIONS

Corrosion protection begins with the storage, fabrication, and handling of the tendon components prior to insertion in the bore hole. Proper care is required to avoid prolonged exposure to the elements, and to avoid mechanical or physical damage which would reduce or impair the future ability of the components to resist any adverse conditions encountered during their service life.

The time the tendon is left ungrouted in the hole and the conditions which will exist within the hole until the tendon is grouted must also be evaluated to determine if corrosion preventive measures are necessary during this phase.

4.6.2 Materials

Prestressed rock and soil anchors shall be protected against corrosion using materials and procedures suitable for the intended service life. The following materials may be used independently or in various combinations to suit the application:

a) Portland Cement grout

b) Plastic pipe or tubing

c) Steel pipe or tubing

d) Greases specially compounded for post-tensioning

e) Epoxies: Polyesters

Galvanizing of any anchor tendon component or cathodic protection is not recommended.

The properties of any corrosion protection material shall not be detrimental to the prestressing steel and shall prevent the intrusion of corrosive environments. Coating materials shall also have the following properties:

a) Stable against deoiling

b) Free from cracks and not brittle or fluid over the entire anticipated range of temperatures;

c) Chemically stable for the life of the tendon;

d) Nonreactive with thesurrounding materials such as concrete, tendons, or sheathing;

e) Corrosion-inhibiting; high-wetting properties;

f) Impervious to moisture.

COMMENTARY

A light coating of rust on the anchor tendon is normal and will not affect the ability of the anchor to perform its function. Heavy corrosion or pitting should be cause for rejection of the anchor tendon.

Thick wall steel pipe may be used for corrosion protection, but only in combination with other protective measurements.

Both methods are not considered to provide a reliable long term corrosion protection.

In the absence of specific temperature requirements, the usual range is 0° F [-20° C] to 160” F [70° C].

RECOMMENDATIONS .

When acidic water can enter the bore hole during the period subsequent to the drilling and flushing operation and prior to tendon insertion and grouting, chemical additives shall be introduced for neutralizing purposes. A minimum pH of 9 is generally considered acceptable when the prestressing steel is in contact with this water. During prolonged periods, the pH shall be monitored at regular intervals and additional neutralization added as required. Concentrated sodium hydroxide and calcium hydroxide have proven effective for this purpose.

4.6.3 Protection Systems

4.6.3.1 Bond Length

Grouting shall be done from the lower end of the drill hole to assure complete encapsulation in the bond zone. Grout cover for the tendon shall be 0.5” [1.3cm] minimum. Centralizers shall be used in sloping holes to assure grout cover.

Holes for epoxy-type anchors shall be no larger than specified by the manufacturer of the resin to assure proper epoxy mixing and coverage.

Plastic smooth and corrugated sheathing used for corrosion protection shall have a minimum wall thickness of 0.040” [1 .Omm].

Epoxy used for coating tendons for corrosion protection shall be applied according to ASTM Specification A772. The epoxy shall be capable of transferring load to the surrounding grout medium. The coating thickness shall be according to the manufacturer’s recommendations. Any damage to the epoxy coating shall be repaired prior to the anchor installation.

4.6.3.2 Bond and Stressing Length Interface

Care shall be taken to assure that the bond grout covers the sheath of the free stressing length for a length of at least 2 feet [0.61m].

The smooth sheath of the free stressing length shall be sealed against the corrugated sheath of the bond length.

COMMENTARY

The bond length is necessarily grouted or epoxied in order to transfer the anchor load. These materials also provide the first level of corrosion protection and, therefore, require proper installation procedures.

Drill holes which require consolidation grouting and redrilling can normally be considered more corrosion resistant because of the extra steps taken to prevent water intrusion.

Additional corrosion protection can be obtained by using a corrugated sheath (plastic or steel) with the anchor placed inside. Grouting inside the sheath may take place before or after exterior grouting. The sequence is usually left to the discretion of the constructor.

The top region of the primary grout is generally a weak zone. After the primary grouting operation is complete, bleed water, impurities, gas bubbles from an expansive agent, etc., tend to collect at the top.

RECOMMENDATIONS

Expansive additives should not be used.

4.6.3.3 Stressing Length

Corrosion protected tendons may be fabricated with either sheathed or unsheathed free stressing length.

Whether or not grout is used outside the sheath in the free stressing length is determined by the preference for added corrosion protection versus concern for load transfer in this region.

Unbonded, sheathed tendons shall be specified when requirements such as extra corrosion protection, simplified or continuous grouting, surveillance of anchors, and anchors which permit subsequent changing of the force level are important or desirable.

Permanent anchors (see definitions) shall have as a minimum a sheath (corrugated or smooth) and either grout or grease between the sheath and the prestressing steel. The grease film shall be a minimum of 0.010 inch [0.25 mm] in thickness. An anchor without these minimums shall be classified as a temporary anchor.

An anchor with no surrounding medium between the prestressing steel and the walls of the hole along the free stressing length should have limited use for temporary anchors only and where aggressive soil environments do not exist. Spacers will not be able to prevent contact of the anchor tendon with the drill hole wall.

For permanent soil anchors, special attention shall be given to that portion of the anchor that is in the naturally aerated upper 10 ft. [3.05m] of soil. This portion of the anchor may be subjected to a fairly large volume of percolating water carrying dissolved oxygen and/or chlorides and sulphates from the atmosphere. In the case of permanent anchors, this zone should have corrosion protection over and above the grout and normal greasing and sheathing. An outer encasement of plastic or steel pipe is recommended.

An exception to the permanent anchor criteria is a dam anchor where the stressing length may be bonded directly to the concrete dam. In such cases, care should be taken to minimize the problems characteristic of the interface zone between the primary and secondary grout.

Thick wall steel pipe may be used for corrosion protection, but only in combination with other protective measures.

A good practice is to extend the corrugated sheathing continuous over the full length of the tendon. When a bond breaker is desired, a smooth wall plastic sheath shall be placed over the corrugated sheath.

Grease shall be compounded to provide corrosion inhibiting and lubricating properties. The allowable content of deleterious substances shall not exceed the limits set forth as follows:

Compound/Test Method/ Max. Quantity - ppm
Chlorides/ASTM-D-512/10
Nitrates/ASTM-D-992/10
Sulfides/APHA - "Test Methods, Sulfides in Water"/10

Epoxy coating may be used as an additional corrosion protection medium, but shall not replace any of the grout or grease requirements for permanent anchors.

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4.6.3.4 Area Underneath the Anchorage

The area immediately behind the stressing anchorage shall be adequately protected from climatic elements, aggressive water, pollutants, or other corrosion inducing agents.

It is important to properly detail this under-anchorage region. Weak grout from bleeding impurities will allow aggressiveelements to attack the anchor if proper protection is not provided.

Pipes or trumpets may be welded to the bearing plate, a seal provided between the anchor and the sheath, and the entire void should be pumped full of corrosion inhibiting grease, mastic or grout.

Casing used during the drilling operations may also be used. A slip joint detail between the casing and the sheath may be incorporated to prevent outflow of the grease or mastic.

In some instances, for example in sheet pile walls, movement of the structure should be expected and proper details should be provided to eliminate corrosion in the under-anchorage area.

Temporary anchors usually do not require special means of corrosion protection. However, some thought is required as to the aggressiveness of the elements and the longevity of the temporary anchor before under-anchorage corrosion protection schemes are eliminated.

4.6.3.5 Stressing Anchorage

Depending on the environment and the time between installation and final protection, the exposed stressing end of the tendon may need to be protected temporarily.

Upon completion of all work on the tendon the anchorage shall be encased in concrete or fitted with a watertight steel or plastic housing filled with grout, grease, bitumen, or other comparable permanent protection. The final protection has to be applied on uncorroded components.

4.7. SPECIFICATIONS FOR ANCHOR WORK

The specifications shall assure that all work will be performed by contractors experienced in this type of work.

The installation of anchors requires specialized equipment, techniques, and careful workmanship. Not every detail of the anchor work can be specified; therefore, a contractor thoroughly experienced with anchor work should be selected to perform the work.

The engineer may specify anchor work in the following three ways:

1. **Open Specification:**

   This leaves the scope and design of the installation up to theanchorcontractor. This method is the most common for securing bids on temporaryanchorwork. The responsibility for design and performance is clearly placed on the contractor. This method allows the

Open specifications are not recommended for permanent anchor application.

contractor to select the most economical anchor system and keeps change orders to a minimum.

2. **Performance Specifications:**

Varying amounts of the design are performed by the anchor contractor and the engineer. The engineer may establish the scope of the work, loadings, elevation of the anchors, minimum free length, whether corrosion protection is required, anchor testing procedures, and instrumentation and monitoring requirements. The method of constructing the anchor, the tendon type, method of corrosion protection, and the bond length are not specified. This method allows the anchor contractor to provide an economical design while satisfying the design requirements. When performance specifications are used, the responsibilities for the work are shared between the engineer and the contractor. Permanent anchor work may be specified in this manner.

3. **Closed Specifications:**

The closed specification results when the engineer designs the complete installation in detail. Closed specifications usually do not insure a better job, instead, they allow contractors not familiar with anchor work to bid. Usually the cost of work obtained by closed specifications is very high and the potential for change orders is great. The engineer is also responsible for the design and performance as long as the contractor has installed the anchors in accordance with the specifications and accepted anchor practices.

Certain materials and drilling methods may still be left open to the choice of the contractor.

In closed specifications the method of payment should be carefully considered.

Bid items for the following may be included:

1. Mobilization and demobilization
2. Number of anchors installed
3. Consolidation grouting (cu. ft.) [m^3]
4. Redrilling (lin. ft.) [m]
5. Performance Test
6. Creep Test
7. Load monitoring devices

An estimate of the anticipated amount of consolidation grouting and redrilling should be included in the specifications and evaluated in the bid quantities.

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Chapter 5

Analysis and Design of Post-Tensioned Structures

5.1 GENERAL

This chapter applies primarily to post-tensioned flexural members used in buildings. However, the procedures described may be adapted to other applicationssuch as bridges, water storage tanks, foundations and dams.

It is usually preferable in post-tensioned design to determine the required prestressing force and hence the number, size, and profile of the tendons for behavior at service loads. The ultimate capacity of the section in flexure and shear must then be checked at thecritical points.

Determination of both the strength and serviceability characteristics of indeterminate post-tensioned structures depends on elastic analysis procedures. The serviceability of a structure is assessed primarily in terms of stresses and elastic and long-term deflections.

5.2 ANALYSIS

5.2.1 Primary and Secondary Moments Due to Post-Tensioning

In simple span post-tensioned beams, as in pretensioned beams, the moments induced by the post-tensioning are directly proportional to the eccentricity of the tendons with respect to the neutral axis of the beam. In continuous, or indeterminate post-tensioned structures, the moments due to post-tensioning are usually not directly proportional to the tendon eccentricity. The difference occurs because the deformations imposed by post-tensioning are resisted by the continuous member at the points where it is continuous with other members in thestructure. This restraint to post-tensioning deformations modifies the reactions and hence affects the elastic moments and shears resulting from the post-tensioning. The moments resulting from the restraints to the prestressing deformations are usually called "secondary moments." This term is used because the moments are induced by the primary post-tensioning moments, Pe, and not because the secondary moment is negligible, nor necessarily smaller than the primary moment.

Primary and secondary moments, as well as the total moments due to post-tensioning are illustrated for a two-span continuous beam in *Fig.* 5.7. This beam has a post-tensioning force, P, acting at a constant eccentricity. Hence the primary moment in the beam is Pe as shown in *Fig.* 5.7 *(b)*. This primary moment will cause a theoretical upward deflection at the center support of $Pel^2/2EI$. The reactions necessary to retain the beam at its original position are shown in *Fig.* 5.7 *(c)*. It is important to note that the secondary moments are functions of the reactions, and that for this reason the secondary moments always vary linearly between the supports. Also, note that, for this case, the secondary moment at the interior support is 150 percent of the primary moment and of opposite sign. The total moment due to post-tensioning may be expressed as the super-position of the primary and secondary moments as shown in *Fig.* 5.7 *(e)*.

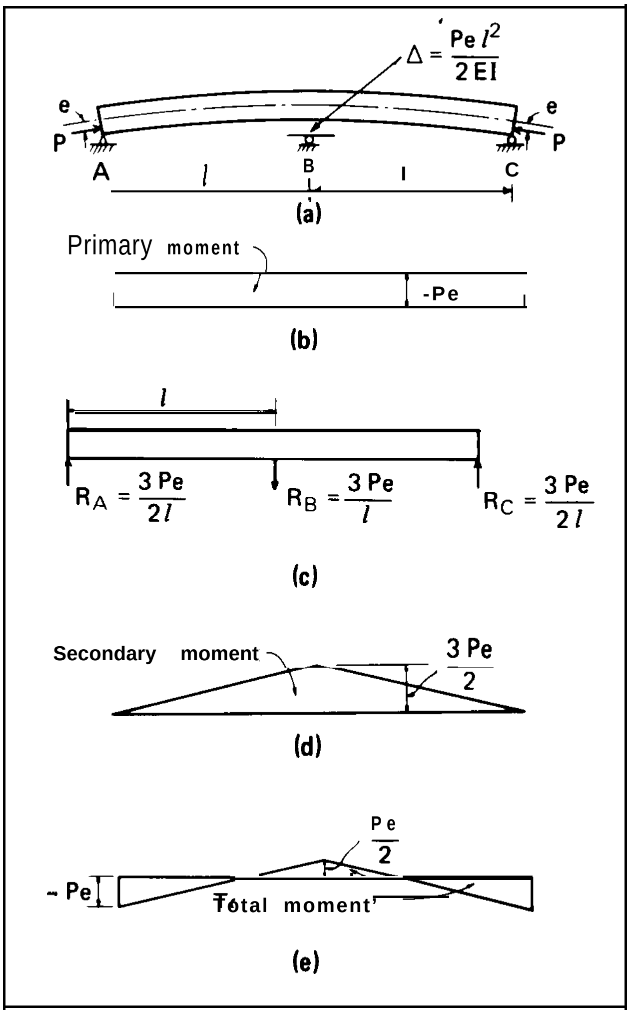


Fig. 5.1 — Primary and secondary moments

Secondary post-tensioning moments play a significant role in the design of continuous post-tensioned structures because they significantly modify calculated stresses and strengths in both positive and negative moment areas. In most continuous structures, the secondary moments have the effect of increasing the magnitude of the positive post-tensioning moments at interior supports, and of reducing the magnitude of the negative post-tensioning moments between supports.

Section 18.10.3 of ACI 318-83 states:

"Moments to be used to compute required strength shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to factored loads including redistribution as permitted in Section 18.10.4."

Application of this code provision usually reduces very substantially the amount of nonprestressed bonded reinforcement required over supports of continuous post-tensioned structures and occasionally requires the use of additional nonprestressed bonded reinforcement in positive moment areas. Calculation of design moments using the secondary post-tensioning moments as prescribed in *Section 18.10.3 of ACI* 378-83 is illustrated in the continuous tee beam design example, *Section* 5.52, and the flat plate design example, *Section 5.5.4.* The most significant effects occur in the tee beam example where the secondary moments increase the design positive moments by about 29 percent, and decrease the magnitude of the design negative moments by about 37.5 percent at exterior columns, and by about 23.9 percent at the interior column.

Since the secondary post-tensioning moments are a result of modified reactions, the design shears are also modified because of the secondary post-tensioning moments. This effect is also illustrated in design examples in *Sections 5.5.2, 5.5.3 and 5.5.4.*

5.2.2 Area-Moment Method of Analysis

The classical area-moment method of determining slopes and deflections in beams may be applied to determination of secondary moments induced by post-tensioning in statically indeterminate structures. The theorems of the area-moment method are outlined in strength of materials and structural engineering text books and will not be repeated here. A variation of the area-moment method, the conjugate beam analogy, is used in the solution of *Example 5.5.2.* This discussion will be confined to determination of the moment effects induced by the post-tensioning tendon. Moments and shears due to all other loadings may be determined by the same procedure or by any other method of elastic structural analysis.

The three-span beam in *Fig.* 5.2 will be used to illustrate application of the area-moment procedure to calculation of secondary moments in indeterminate post-tensioned structures. The beam is post-tensioned by a tendon with a profile made up of a series of parabolic curves. To analyze the structure, the spans are considered to be cut at interior supports and each span is considered as a simple beam on which moments, Pe, are imposed due to the post-tensioning tendon. As illustrated in *Fig. 5.3 (a),* the slope of the end span at the first interior support due to post-tensioning may then be calculated as:

$$\Sigma \frac{Pe\,\Delta l}{EI}\left(\frac{x}{l_1}\right)$$

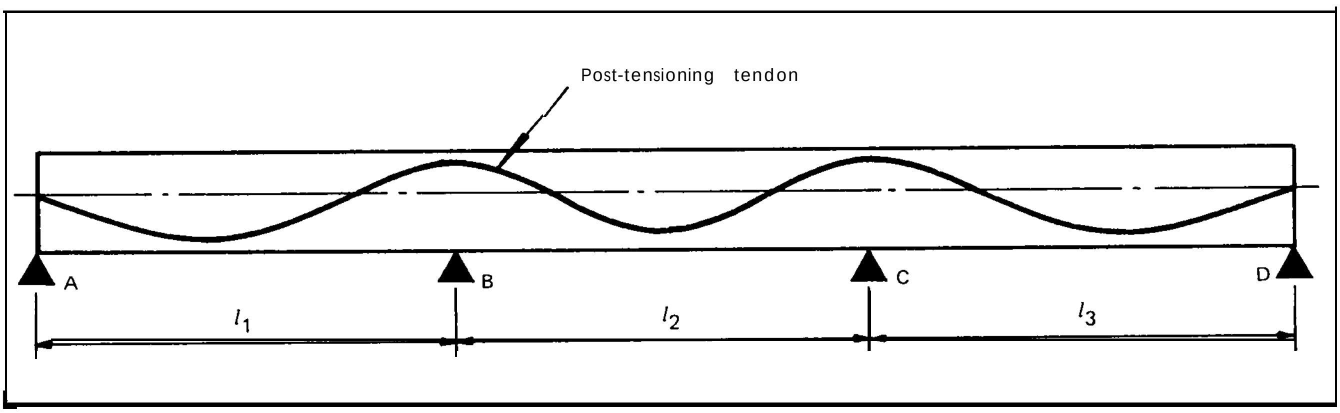


Fig. 5.2 — Continuous post-tensioned beam

The next step in the analysis is to apply a moment on the end span at B (Ms_{BA}, the fixed-end secondary moment at B) which will rotate the beam back to zero slope. For a two span symmetrical structure this moment would be the secondary moment due to post-tensioning. However, for structures of three or more spans or unsymmetrical structures, it is necessary to distribute the fixed-end secondary moments calculated for each span to obtain the final secondary moments. Application of Ms_{BA} at B will result in a triangular moment diagram over the end span as shown in *Fig.* 5.3 *(b)* and the rotation at the interior support due to this moment diagram will be:

$$\frac{Ms_{BA}\, l_1}{3\,EI}$$

Equating rotations due to post-tensioning to the above makes it possible to solve for Ms_{BA} in terms of the moments due to post-tensioning:

$$\frac{Ms_{BA}\, l_1}{3\,EI} = \Sigma \frac{Pe\Delta l_1}{EI}\left(\frac{x}{l_1}\right)$$

$$Ms_{BA} = \frac{3}{l_1^2}\,(\Sigma Pe\Delta l_1\, x)$$

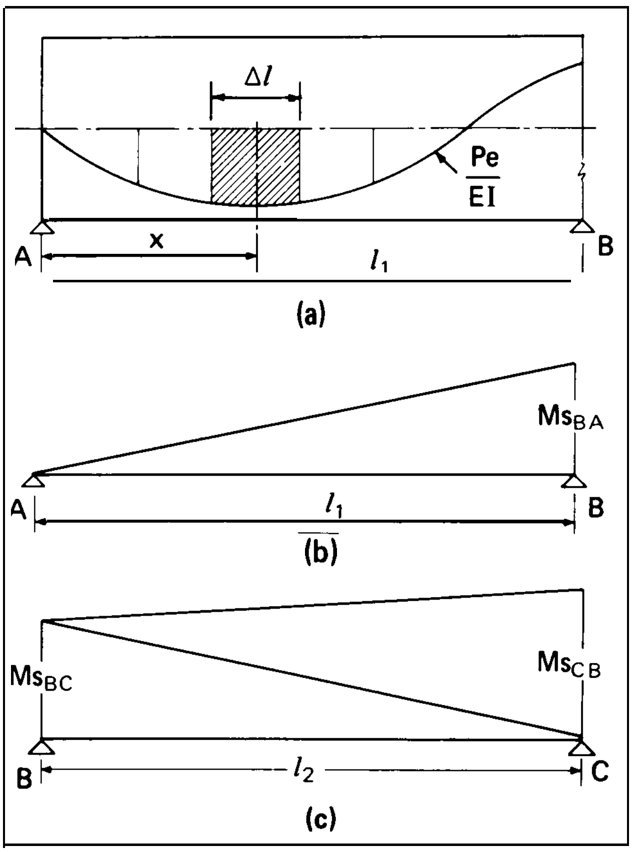


Fig. 5.3 — Area-moment method of analysis

In a similar manner, the rotation due to post-tensioning moments are calculated for the interior span and the other end span. For the interior span, moments are imposed at each end of the span to rotate the beam back to zero slope as shown in *Fig.* 5.3 (c). It is then necessary to solve two simultaneous equations to determine values of Ms_{BC} and Ms_{CB}. After obtaining Ms_{BC}, Ms_{CB}, and Ms_{CD} (the latter in a manner identical to Ms_{BA}), the moment distribution technique may be used to calculate the final secondary moments at the two interior supports.

After determining the final secondary moments at supports the secondary moment effect at any interior span point is readily obtained by linear interpolation between the secondary moments at supports. For stress calculations, final moments due to post-tensioning at any point may then be calculated as the algebraic sum of the primary moments, Pe, and the secondary moments, Ms. For strength calculations the secondary moments may be considered as modifications of the factored dead load and live load moments as described in Section 5.2.7. Note that in the area-moment procedure described above, the secondary moments, Ms, are calculated in terms of the primary post-tensioning moment, Pe.

5.2.3 Equivalent Load Method of Analysis

The effect of a prestressing force on a member can be determined by considering the prestressing force to be replaced by equivalent external loads. In the simple span beam shown in *Fig.* 5.4 (a) containing two sets of tendons, horizontal forces P_1 and P_2 are exerted at the ends of the member along with vertical forces equal to P_2 tan *a.* The vertical forces may be neglected in the design of the beam because they occur directly over a support. In addition to these loads, the curved tendon (assumed parabolic) exerts a continuous upward force on the beam along its entire length. If friction between tendon and concrete is neglected, the pressure exerted on the concrete is normal to the plane of contact and tension in the tendon is constant. The normal pressure exerted by the tendon is equal to the tension in the cable divided by the radius of curvature as shown in *Fig. 5.4 (b)* (for circular tendon geometry).

Due to the shallow nature of most post-tensioned beams, the horizontal component of the tension in the tendon may be assumed equal to the tension. With this assumption, the horizontal component of the tendon force may be assumed constant which allows the tendon to be held in equilibrium with the uniform vertical load as shown in *Fig. 5.4 (c).*

The magnitude of the vertical load exerted by the member to hold the tendon in equilibrium may be derived in various ways. Taking one-half of the tendon as a free body, the forces are as shown in *Fig.* 5.4 *(d).* Summing moments about support A and solving for the load w_P provides:

$$w_p = \frac{8P_2e'}{l^2}$$

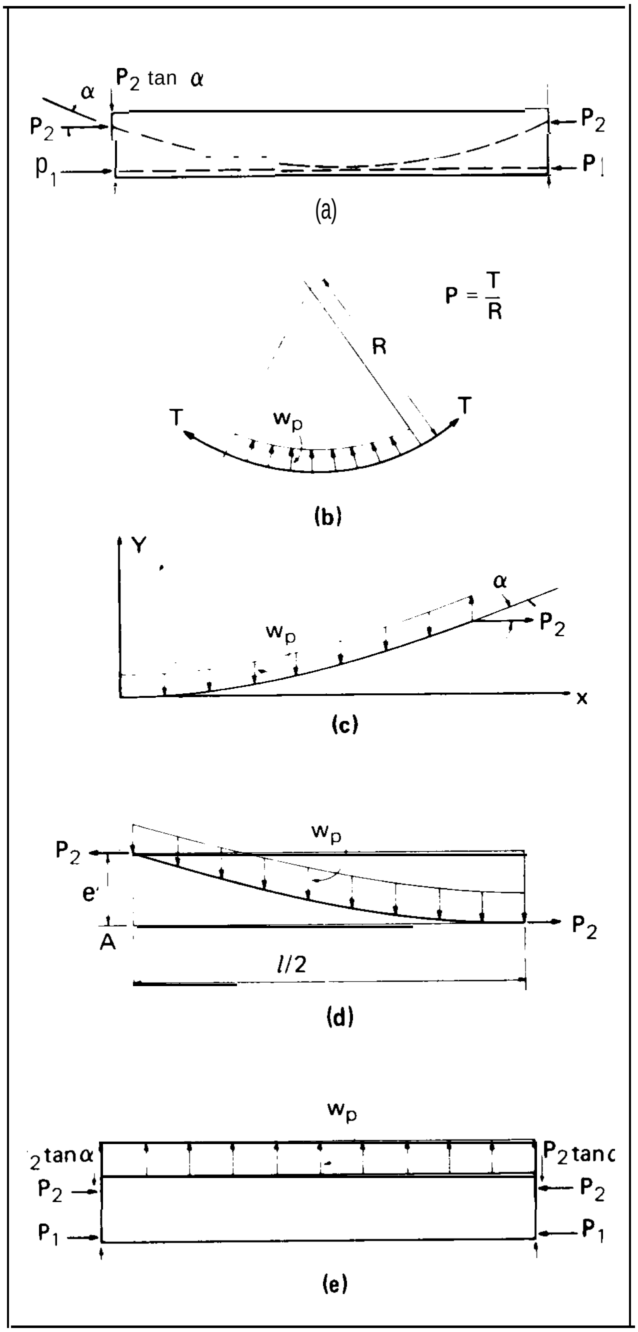


Fig. 5.4 — Equivalent load analysis

The load exerted by the tendon on the member is then equal but opposite this load, or:

$$w_p = \frac{-8P_2e'}{l^2}$$

Note that the sign convention considers w_P to be positive when it acts downward.

Summing up the loads exerted by the tendons on the simple span beam of *Fig.* 5.4 *(a)* provides the loads shown in *Fig.* 5.4 *(e).* Caculating equivalent loads is probably not justified for simple span beams where the moments induced by the tendons are directly proportional to the tendon eccentricity. However, for continuous beams, the use of equivalent loads permits analysis for the total moment effects of the post-tensioning (combined primary post-tensioning moments, Pe, and secondary post-tensioning moments are obtained by the equivalent load analysis) by considering a single additional loading case. To obtain the magnitude of secondary post-tensioning moments at any point, it is only necessary to subtract the primary post-tensioning moment, Pe, from the post-tensioning moment obtained from theequivalent loads.

Using the equivalent load procedure, the total post-tensioning moments at supports may be obtained by calculating fixed end moments and using moment distribution or other methods of indeterminate structural analysis. After the post-tensioning moments have been determined at the supports, post-tensioning moments between supports may be obtained by superimposing the simple beam moment diagram due to the post-tensioning loads on the base line provided by connecting the total post-tensioning moments at supports. However, it may be simpler to obtain interior post-tensioning moments using the tendon profile. Using this approach, the interior moments may be expressed as:

$$M = Pe + M_{AB} + (M_{BA} - M_{AB})\frac{x}{l}$$

$$- P\left[e_A + (e_B - e_A)\frac{x}{l}\right]$$

where M_{AB} and M_{BA} are the total post-tensioning moments at two adjacent supports, P is the tendon force, and e_A, e_B, and e are the eccentricities of the tendon at A, B, and x, respectively. Eccentricity is taken as negative when it falls below the c.g.c. and positive above the c.g.c. For the tendon profile in *Fig.* 5.5, the moment at x is the algebraic sum of the end moments at the

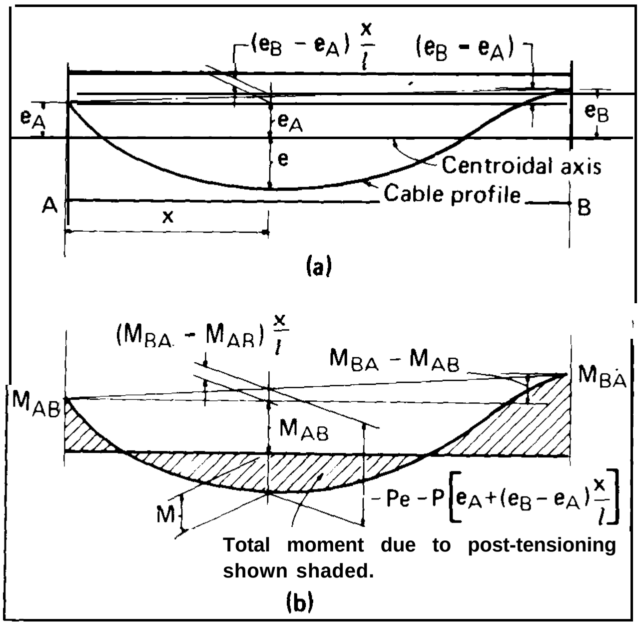


Fig. 5.5 — Interior post-tensioning moments

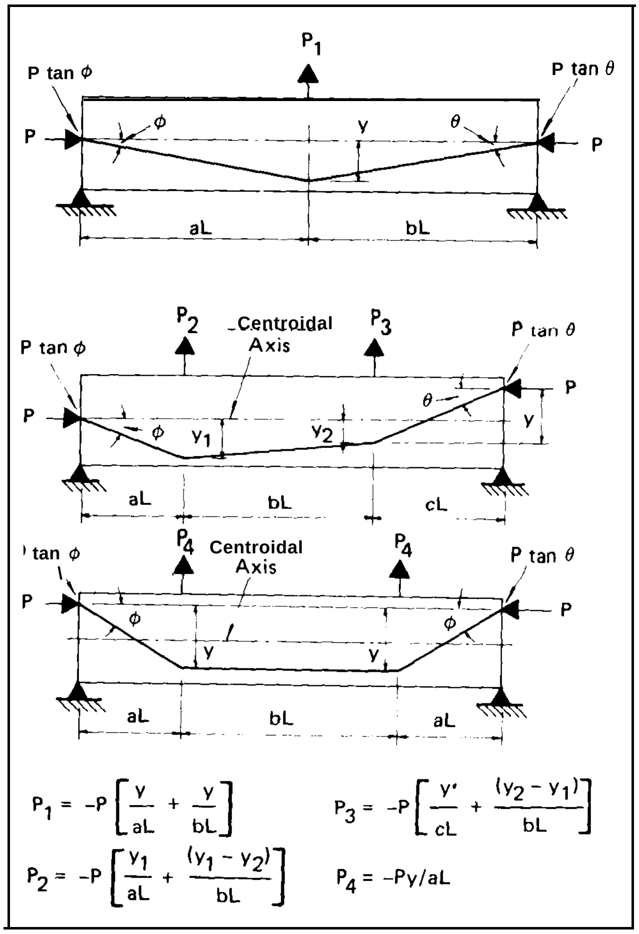


Fig. 5.6 — Straight line tendon

point and the moment of the tendon force P times the distance from the tendon to the straight line connecting the ends of the tendon over the supports.

The equivalent load analysis procedure is further described in *Appendix A.7,* which also presents tables of post-tensioning moment coefficients based on computer analyses using the equivalent load concept. Coefficients for interior support post-tensioning moments are provided for a variety of tendon profiles for two, three, four, and five span structures.

Occasionally, the use of straight line profiles may be appropriate for the post-tensioning tendon. This often happens, for example, in the repair or strengthening of existing structures by addition of external post-tensioning tendons. The effect of straight line tendon profiles is to exert a concentrated force on the structure or structural element at the point or points where the tendon geometry changes direction. The magnitude of the force exerted can be obtained directly from the geometry of the tendon and the force in the tendon as illustrated in *Fig. 5.6.* While the use of straight line tendon profiles is relatively infrequent, it is important for engineers to remember that sharp angular changes in the tendon profile exert significant concentrated forces on the structure. When such angular changes occur, or are detailed in a horizontal direction, the tendon exerts lateral forces on the structure and these forces normally have to be accommodated by use of nonprestressed ties or stirrups sufficient to transfer the concentrated force to the body of the structure. Numerous construction problems have occured because designers and/or contractors have overlooked the necessity of providing for concentrated forces due to angular changes in the tendon geometry. As indicated in *Chapter* 6, it is normally preferable to accomplish the changes in tendon trajectory necessary to avoid openings, inserts, etc., by curving the tendon with a radius of curvature on the order of 21 ft.

5.2.4 Load-Balancing Method of Analysis

The basic concept of load balancing also represents the influence of the tendons by equivalent loads. The concept is illustrated in *Fig.* 5.7 *(a)* where the tendon is selected to directly counteract the imposed loading at the indicated eccentricity, e'. Since the moment induced by the tendon and the load offset each other, the net stress in the beam will be the axial compressive stress from the post-tensioning, P/A. If it is desired to design the beam for zero stress at the bottom fiber at center span (or any other value of stress less than the modulus of

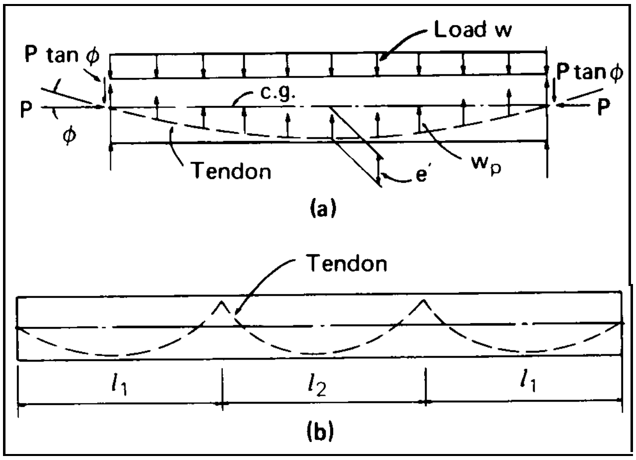


Fig. 5.7 — Load balancing

rupture), it is only necessary to reduce the amount of post-tensioning provided. The net stress on the section may be calculated from P/A + $M_n c/I$ where M_n is the net (unbalanced) bending moment on the section. For continuous designs, the tendon geometry would be assumed as shown in *Fig. 5.7 (b).*

The above example serves to illustrate the salient features of load-balancing. These are that the prestressing force is selected to balance or counteract some portion of the load, and that under the balanced loading conditions the structure will theoretically have no deflection and will be subjected only to an axial compressive stress, P/A, from the tendon. The net moment in the structure at any point is that resulting from the load not "balanced" by the post-tensioning. This is an extremely powerful concept for visualizing the effect of the post-tensioning on the structure, and it greatly simplifies design calculations. Secondary post-tensioning moments can be readily obtained by subtracting the primary moment, Pe, from the moments caused by the balanced load at any point. Application of the load-balancing design procedure is illustrated by the flat plate design example presented in *Section 5.5.4.*

The load-balancing concept was introduced by Professor T.Y. Lin in a paper "Load-Balancing Method for Design and Analysis of Prestressed Concrete Structures" in the June 1963 Journal of the American Concrete Institute, Proceedings Vol. 60, No. 6. The concept can be applied to rigid frames, two dimensional structures, and to shells and folded plates. Those learning to use this method will want to review the above referenced article and the additional references listed in the article. For the convenience of the reader, discussion from Professor Lin's ACI Journal paper concerning the determination of the amount of load to be balanced and the accuracy of the method is presented below:

"Using this concept of load balancing, an important question arises: what should be the loading to be balanced by the prestress? The answer to this question may not be simple. As a starting point, it is often assumed that the dead load of the structure or element be completely balanced by the effective prestress. This would mean that a slight amount of camber may exist under the initial prestress. In the course of time, when all the losses of prestress have taken place, the structure or element would come back to a level position.

Although it seems logical to balance all the dead load, such balancing may require too much prestress. Since a certain amount of deflection is always permitted for a nonprestressed structure under a dead load, it is reasonable to also permit a limited amount of deflection if it would not become objectionable. However, there is greater tendency in prestressed structures to increase their deflections as a result of creep and shrinkage. Hence the deflections should be limited to a smaller value at the beginning.

When the live load to be carried by the structure is high compared to its dead load, it may be necessary to balance some of the live load as well as the dead load. One interesting approach is to balance the dead load plus one half the live load (DL + 1/2LL). If this is done, the structure will besubjected to no bending when one-half of the live load is acting. Then, it is only necessary to design for one-half live loading up when no live load exists, and for one-half load acting down when full live load is on the structure. This idea of balancing dead load plus one-half live load, while theoretically interesting, could result in excessive camber if the live load consists essentially of transient load. Furthermore, it may not result in an economical design.

When attempting to evaluate the amount of live load to be balanced by prestressing, it is necessary to consider the real live load and not the specified design live load. If the specified design live load is higher than the actual live load, only a small amount of the live load or even no live load at all should be balanced. On the other hand, if the actual live load could be much higher than the design live load, especially if the live loading would be sustained, it would be desirable to balance a greater portion of the live load. Theengineer should exercise his judgment when choosing the proper amount of loading to be balanced by prestressing. This should be done while keeping in mind the satisfaction of other requirements such as elastic stress limitations, crack control, and ultimate strength.

The load balancing method can be achieved with considerable accuracy because both the gravity load and the prestressing force can often be predicted with precision. However, variations may be encountered so that the actual loading and the actual prestress may not be as expected. For a relatively stiff member, errors in estimating the weight and the prestress will usually be negligible. For a slender member, even slight variations may result in considerable errors in the estimation of load balancing, and either camber or deflection may result.

As is well known, the modulus of elasticity of concrete and the creep characteristics cannot be predetermined with accuracy. Fortunately, neither the modulus nor theflexural creep would enter into the picture if the sustained load is exactly balanced by the prestressing component. In other words, since there is no transverse load on the member, there will be no bending regardless of the value of the modulus or the creep coefficient.

Owing to frictional resistance in post-tensioning, the prestress and hence its transverse component may vary along the length of a tendon. For the usual case of low frictional loss, an average prestress can be used for computation without introducing serious errors. Should the loss be excessive, its effect must be carefully computed.

Depending on the accuracy desired in the control of camber and deflection, the amount of loading to be balanced must be chosen. If the limits of error can be estimated and if the significance of deflection or camber control can be assessed, it will not be difficult to design the member so as to possess the desired behavior."

At another point in the above referenced article, Professor Lin noted the approximation involved in the assumption of a sharp bend in the tendon geometry over the supports and indicated that it would sometimes be necessary to consider this approximation by making an appropriate investigation. The effects of the downward tendon curvature over supports are described in *Section* 5.2.3 and in *Appendix A. 1,* and may be evaluated by the equivalent load design procedure. However, for practical tendon geometries, it has been shown that the effect of reverse tendon curvature at supports on post-tensioning moments is on the order of 5 to 10 percent. Therefore, since the calculated moments only effect the calculated stresses and not the ultimate capacity, the load-balancing method is sufficiently accurate in most cases without consideration of the reverse tendon curvature at supports. This is particularly true for applications such as post-tensioned flat plates, where the generally used analysis procedures (which have been demonstrated to provide satisfactory structural behavior) involve significant approximations relative to the magnitude of peak moments in the immediate vicinity of columns.

The load-balancing procedure is by far the most convenient design approach for most indeterminate post-tensioned structures, and is recommended as the basic design approach for most applications.

52.5 Ultimate Strength Analysis

The required design capacity in moment and shear is determined by multiplying elastically determined service load moments and shears by the load factors specified in *Section 9.2 of AC/* 378-83. The moments and shears so obtained may be modified for redistribution of moments due to inelastic behavior in accordance with the provisions of *Section 18.10.4 of ACI 378-83.* Further, the dead and live load factored moments (design moments) are to be modified by the secondary moments due to reactions induced by post-tensioning (with a load factor of 1.0) asspecified in *Section 18.10.3 of ACI 318-83* and as described above in *Section 5.2.1.*

5.3 DESIGN

5.3.1 Preliminary Sizing of Members

There are no set span-depth limits for post-tensioned members, but the values in *Table 5.7* are provided as a guide to the preliminary sizing of members. Since the entire gross transformed section is effective in resisting service load deflections in a post-tensioned structure with design stresses below the modulus of rupture, the necessary structural depth is reduced by approximately 1/3 to provide stiffness comparable to a cracked flexural element with non-prestressed reinforcement. The resulting savings in concrete is the most significant factor in the relative economy of post-tensioned construction. Reduced structural depth also provides economies in many other design circumstances, a number of which are illustrated in the applications of post-tensioning described in *Chapter 1.*

Table 5.1 — Typical span-depth ratios

	Continuous spans		Simple spans	
	Roof	Floor	Roof	Floor
One-way solid slabs	50	45	45	40
Two-way solid slabs (supported on columns only)	45-48	40-45		
Two-way waffle slabs (36" pans)	40	35	35	30
Beams	35	30	30	26
One-way joists	42	38	38	35

The above ratios may be increased if calculations verify that deflection, camber, and vibration frequency and amplitude are not objectionable.

5.3.2 Type and Placement of Tendons

A significant aspect of the design of post-tensioned structures relates to providing sufficient space for the post-tensioning tendons and coordinating the location of the tendons with the nonprestressed reinforcement, electrical and telephone conduit, and other embedded items. The available tendon sizes and types are described in *Chapter 2,* which also gives dimensions of anchorage hardware and the ferrous metal conduit used for grouted tendons. Capacities of the individual post-tensioning steel elements currently used are presented in the *Appendix.*

For design purposes, tendon profiles are normally considered to be parabolically curved. Straight line tendon profiles are occasionally used as described in *Section 5.2.3.* In most cases, the most economical location for the tendon in continuous post-tensioned structures is as near the bottom fiber as possible at the points of maximum positive moment, and as near the top fiber as possible over interior supports. As can be inferred from the load-balancing concept, the maximum capacity is obtained from a given tendon by installing it with the maximum total drape. An exception to this occurs in end spans where no additional tendon effectiveness or capacity is obtained by anchoring the tendons above the neutral axis at exterior supports. In this case, the cranked-in moment counteracts the additional balanced load. Actually, this is a fortunate circumstance because in many applications (such as flat plates) it is necessary to locate the anchorage hardware near the neutral axis in order to obtain the necessary edge distance.

See *Section 6.2.2* for further discussion of tendon installation procedures.

5.3.3 Prestress Losses

Since 1954, a value of 25,000 psi has been used to represent the total loss of prestress, except for the effects of friction and anchor set, for normal weight concrete members post-tensioned with stress relieved steel. The Commentary to the *ACI 318-83* Building Code Section 18.6 notes: "Lump sum values of prestress losses for both pretensioned and post-tensioned members which were indicated in previous editions of the commentary are considered obsolete." These lump sum loss values were originally proposed as average values satisfactory for general design use. This was done with the understanding that actual loss values may vary above and below the lump sum values with little, if any, significant effect on the performance of the structure. In this regard, a further quote from the Commentary to *ACI 318-83 Section* 18.6 is pertinent:

"Actual losses, greater or smaller than the computed values, have little effect on the design strength of the member, but affect service load behavior (deflections, camber, cracking load) and connections. At service loads, overestimation of prestress losses can be almost as detrimental as underestimation, since the former can result in excessive camber and horizontal movement."

The basic responsibility for calculating or specifying prestress losses for a structure rests with the engineering or architectural firm or agency responsible for the design. In some cases, concern about the accuracy of the lump sum loss values, or lack of better information, has led to a reluctance on the part of the designing agency to specify the prestress losses. However, a great deal of information has been published to aid the designer in calculating losses. In particular, the July-August 1975 Journal of the Prestressed Concrete Institute contains "Recommendations for Estimating Prestress Losses" by the PCI Committee on Prestress Losses, and the June, 1979 issue of Concrete International includes a paper "Estimating Prestress Losses" which was prepared as part of the work of, and sponsored by, ACI-

ASCE Committee 423, Prestressed Concrete. The Specifications for Bridges published by the American Association of State Highway and Transportation Officials also contains specifications for prestress losses. These sources are recommended for use by engineers in cases where more precise calculation of prestress losses is considered necessary.

In circumstances where a post-tensioning materials fabricator must supply a bid on a job on which the engineer has not specified the prestress losses, the quantity of tendons, and therefore the bid, is a function of the value used for prestress losses. If possible in this case, it is recommended that prestress loss values be determined by the post-tensioning materials fabricator in accordance with the procedures in the references noted above. However, to accomplish this, it is necessary for all post-tensioning materials fabricators submitting bids to have complete design information on the job. In many cases, this design information is not generally or readily available. For this reason, the post-tensioning industry suggests use of the lump sum loss values in *Table* 5.2 for uniformity in determination of tendon requirements on jobs where prestress losses (from all sources except friction) have not been specified by the engineer. The values presented in *Table* 5.2 are average values for general applications. They were developed through an extensive computer study utilizing the best available information, including the latest research on steel relaxation for various post-tensioning tendon materials.*

Table 5.2 — Approximate prestress loss values'

Post-tensioning tendon material	Prestress loss — psi	
	Slabs	Beams and Joists
Stress relieved 270k strand and stress relieved 240k wire	30,000	35,000
Bar	20.000	25.000
Low relaxation 270k strand	15.000	20.000

' Note: This table of approximate prestress losses was developed to provide a common Post-Tensioning Industry basis for determining tendon requirements on projectsin which the magnitudeof prestress losses is not specified by the designer. These loss values are based on use of normal weight concrete and on average values of concrete strength, prestress level, and exposure conditions. Actual values of losses may vary significantly above or below the Table values in cases where the concrete is stressed at low strengths, where the concrete is highly prestressed. or in very dry or very wet exposure conditions. The Table values do not include losses due to friction.

Particular attention is invited to the footnote to Table 5.2 which suggests applications where variations above and below the loss values in the tables might be expected.

5.3.4 Design for Service Loads

In the design of post-tensioned members, the response of the total frame must be considered for the instantaneous prestressing loads applied, as well as for the time-dependent volume changes and the applied service loads.

ACI 318-83 allows service load flexural tensile stresses up to 12 $\sqrt{f'_c}$ for one-way systems if deflections based on transformed cracked section and bilinear moment-deflection relationships are considered. In view of the approximations involved in the generally used analysis procedures for two way flat plates, the service load tensile stress limitation for post-tensioned flat plates is limited to 6 $\sqrt{f'_c}$ unless more rigorous analysis procedures are used.

The serviceability behavior and strength of one-way slabs with different tensile stresses at service load level is indicated by *Fig.* 5.8. Dimensions and other details of slabs A and B are shown in *Fig.* 5.9. Slabs A and B are the prototype designs for model slabs tested at the University of Texas at Austin.** Slab C was designed for a mximum tensile stress of 12 $\sqrt{f'_c}$ and Slab D was designed for zero tensile stress

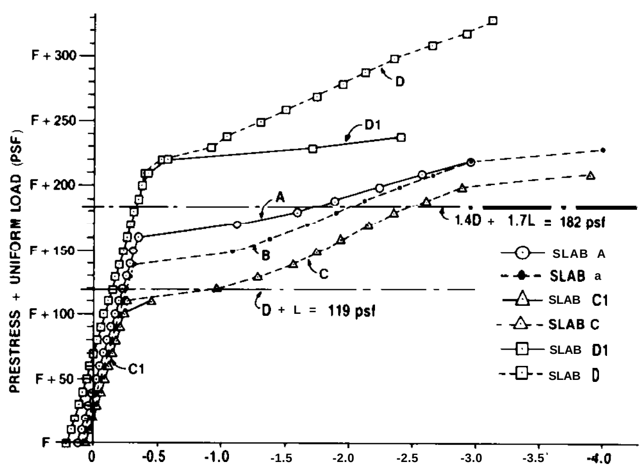


Fig. 5.8 — Vertical displacement at 0.4 span from exterior support (in.), and capacity of one-way slabs.

* For a complete description of the basis for the computer studies made, see "A Method for Predicting Prestress Losses in a Prestressed Concrete Structure" by Glodowski, R. Jr., and Lorenzetti, J.J., PCI Journal, Volume, 17. Number 2, March/April 1972.

** Burns, N. H.. Charney. F.A.. and Vines, W. Ft., "Tests of One-Way Slabs with Unbonded Tendons" PCI Journal, Vol. 23, No. 5, September-October, 1978, pp. 66-83.

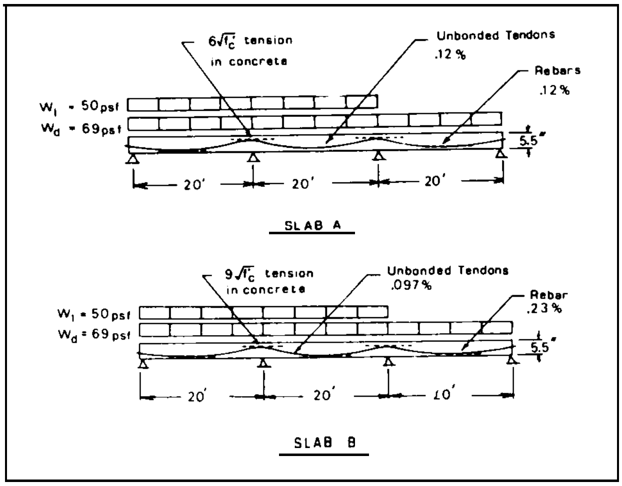


Fig. 5.9 — Prototype design parameters for Slabs A and B

under service loads. Slabs Cl and D1 reflect the same designs as Slabs C and D but without bonded reinforcement. Slabs A, B, C and D all meet the factored load strength requirement of ACI 318-83. However, Slab C cracked slightly below the service load level and deflected nearly 1 inch in the next load increment. The service load deflection of L/250 exceeds the Code limit of L/360. This suggests that the ACI Code limit of 12 $\sqrt{f'_c}$ on the hypothetical tensile stress encompasses the serviceability range of one-way slabs for the span-to-depth ratios normally used.

The design for Slab D indicates the wastefulness of full prestressing for this application. The tendons alone provide 140 percent of the strength required, and the tendons plus minimum code reinforcing bars provide 180 percent of the strength requirement. Since the prestress balances 126 percent of the dead load, some camber growth might be anticipated.

The cost of reinforcement for Slab D would be significantly greater than for the other three designs and the serviceability behavior may be less satisfactory, particularly from the standpoint of possible cracking due to increased restraint to elastic and creep shortening by supporting columnsand walls. In this regard, the shortening of Slab D due to prestress would be more than 200 percent greater than Slab B.

Note in *Fig. 5.8* that Slab A and Slab B are uncracked at load levels significantly above service load (D + L), and that the deflections at service load level are less than one quarter of an inch. This suggests that slabs with normal live loads, and normal span/depth ratios, designed with tensile stresses of 6 $\sqrt{f'_c}$ to 9 $\sqrt{f'_c}$ at service load level will automatically meet ACI Code limits on deflection and will be uncracked due to flexural stresses. The first visible crack in the model test of Slab B (with a design service load tensile stress of 9 $\sqrt{f'_c}$) occurred at a load level of 1.0 D + 1.6 L.

Limitations on deflections of prestressed concrete members are presented in Section 9.5.4 *of ACI 318-83. Section* 9.5.4.1 *of AC/ 318-83* stipulates that the moment of inertia of the gross concrete section may be utilized for uncracked sections in deflection calculations. Deflection calculations are illustrated in the design examples presented in *Section* 5.5. Deflections are a factor of particular importance in design of long span post-tensioned roof structures exposed to the weather. In this case, it is essential to ensure that ponding will not occur which could lead to failure of the roof. Because of the potential danger of ponding, long span roof members are an application where precise calculations should be made of prestress losses and the long term deflection to be expected.

5.3.5 Flexural Strength

Flexural strength of post-tensioned members may be calculated in accordance with the provisions of *ACI 318-83*, particularly *Chapters 9, 10* and 78.

Flexural strength calculations for bridges should be in accordance with *Section 9* of the "Standard Specifications for Highway; Bridges" published by the American Association of State Highway Officials.

The stress developed in a post-tensioning tendon at high load levels is affected by the bond between the tendons and the surrounding concrete. A fully bonded tendon develops steel stresses that are proportional to the applied moment at a section. Assuming a constant section, the critical section for flexure will occur at the point of maximum moment. At this section, the steel stresses under factored loads will usually be beyond the yield stress. In contrast, the application of a load to a member with unbonded tendons does not necessarily producea maximum stress in the steel at the point of maximum moment. Neglecting friction, the ability of the steel to slide in the duct allows the increase in strain to be distributed over the entire length of the tendon. This averaging of load induced strainsgenerally results in a lower value of f_{ps} for unbonded than for bonded tendons. This difference in the development of steel strains in bonded and unbonded tendons led to the development of the different formulas for f_{ps}

which are presented in Sections 78.7.2 (a) and 78.7.2 *(b) of AC/ 318-83.* It is important to recongnize that thedifferent responseof bonded and unbonded tendons is fully recognized by the respective formulas for stress at design loads of *AC/* 378-83. While different amounts of post-tensioning steel are required for the same flexural strength for bonded and unbonded members, both types of tendons satisfy the flexural strength requirements of *ACI 378-83.*

Section 78.9 *of ACI* 378-83 represents minimum requirements for bonded reinforcement for structures using unbonded tendons. The purpose of this reinforcement is to provide control and distribution of cracking at high load levels. This ensures that the structure will respond as a flexural element rather than as a shallow tied arch. Control of crack widths also contributes to shear capacity because of a related reduction of crack depth resulting in a greater depth of the concrete section in compression to resist shear stresses.

In most cases, the most economical design for flexural strength will utilize the maximum permissible tensile stresses for prestressed concrete, although the tensile stress level may be limited by serviceability conditions such as deflection and a desire for watertight construction. Use of higher tensile stresses will reduce the amount of post-tensioning reinforcement required and at the same time, for unbonded construction, will utilize the flexural capacity provided by nonprestressed reinforcement. It has been demonstrated by many tests that the yield strength of nonprestressed reinforcement will bedeveloped at the time the unbonded post-tensioning tendons develop their maximum stress as specified by equations *(18-4)* and *(78-5) of ACI 378-83.*

The location of the resultant steel force for post-tensioned members with nonprestressed reinforcement should be calculated by taking moments of the *forces* about the compression face of the cross-section. Use of forces to determine the center of gravity of the steel force is required because the tendons and nonprestressed reinforcement are operating at different unit streses. The effective depth then becomes:

$$d = \frac{A_{s1}\ f_{ps} d_1 + A_{s2}\ f_y d_2}{A_{s1}\ f_{ps} + A_{s2}\ f_y}$$

Where d = effective depth of all tension reinforcement

A_{s1} = area of tendons

d_1 = effective depth of tendons from compressive face

A_{s2} = area of nonprestressed reinforcement

d_2 = effective depth of nontensioned reinforcement from compression face

The distance from the compression face to the centroid of the compressive force, d_3, can be determined by using the equivalent rectangular stress distribution, 0.85 f'_c, and equating ultimate steel and concrete forces. For rectangular sections, or flanged sections in which the neutral axis lies within the flange, the distance to the centerofthecompressiveforcefrom the compression face may be expressed as:

$$d_3 = \frac{A_{s1}\ f_{ps} + A_{s2}\ f_y}{1.7\ f'_c b}$$

For this case, the ultimate moment capacity may be expressed as:

$$M_u = \phi(A_{s1}\ f_{ps} + A_{s2}\ f_y)\ (d\text{-}ds)$$

For flanged sections in which the netural axis falls outside of the flange, the following formula is suggested for computation of the ultimate moment capacity:

Total steel force provided:

$$T = A_{s1}\ f_{ps} + A_{s2}\ f_y$$

Force required to develop the overhanging flange:

T_1 = 0.85 f'_c (b-b')t

b = effective flange width

b' = width of web

t = thickness of the flange

Force required to develop the web:

$$T_2 = T - T_1$$

The ultimate moment capacity may be expressed as:

$$M = \phi\ T_2 d\ (1 - \frac{0.59T_2}{b'df'_c}) + T_1\ (d\text{-}t/2)$$

5.3.6 Shear Strength

Shear strength calculations for buildings should be made in accordance with ACI 318-83 with particular reference to Chapters 9 and *11.* Shear strength calculations for bridges should be made in accordance with the provisions of *Section* 9 of the "Standard Specifications for Highway Bridges" published by the American Association of State Highway and Transportation Officials.

Shear in the concrete in both statically determinant and continuous post-tensioned members is significantly affected by the shear carried by the post-tensioning tendons. This effect is shown schematically for an interior span of a continuous post-tensioned member in *Fig. 5.70.* In statically determinant members without reverse tendon curvature at the supports, the shear carried by the tendon is transferred back into the reaction by concentrated vertical forces at the anchorages.

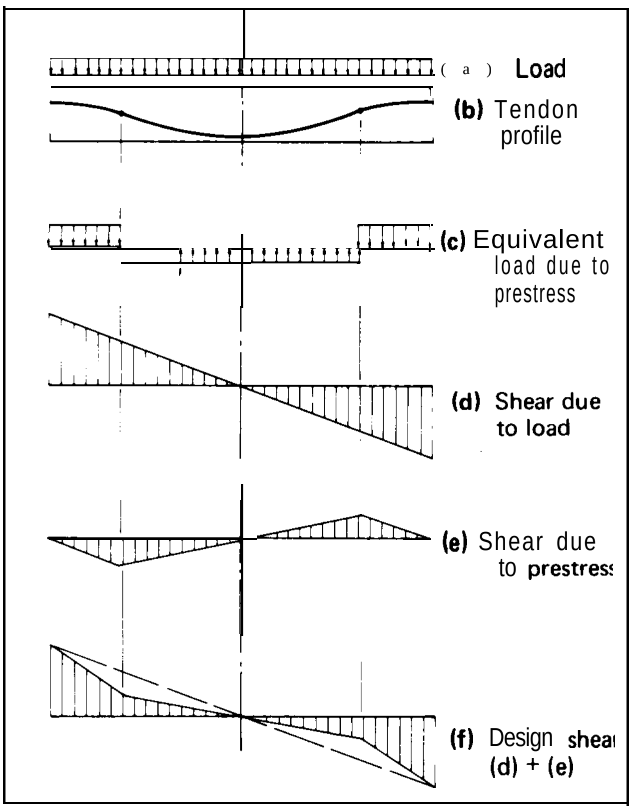


Fig. 5.10 — The effect of shear carried by the post-tensioning tendon on design shear.

Shear design in ACI 318-83 is based on equation (1 l-l):

$$V_u < \phi V_n$$

where V_u is the factored shear force at the section considered, and V_n is the nominal shear strength computed by equation (1 l-2):

$$V_n = V_c + V_s$$

where V_c is nominal shear strength provided by the concrete, and V_s is nominal shear strength provided by shear reinforcement.

For shear reinforcement perpendicular to the axis of the member, the value of V_s is calculated by equation (11-17) of ACI 318-83:

$$V_s = \frac{A_v f_y d}{s}$$

Where A, is the area of shear reinforcement within a distance "s". For inclined stirrups, equation (11-18) of ACI 318-83 may be used to calculate shear strength of the stirrups.

Determination for the value of V_c is based on equations (11-11) and (11-13) of ACI 318-83for "flexure-shear" cracking and "web-shear" cracking, respectively. A discussion of these two equations and the different types of shear cracking is presented in the Commentary to ACI 318-83. As an alternative to use of equations *(11-1*7) and *(1* 7-73) for determination of the shear stress carried by the concrete, equation *(11-10)* was introduced in the 1971 ACI Building Code. Equation *(11-10),* expressed in terms of shear stress, is shown in *Fig.* 5.77 with test data of prestressed beams failing in shear and containing no web reinforcement. The data includes members with a wide range of prestress and concrete strength, including several tests where there was no prestress. The solid points represent test results for members with an effective prestress equal to or greater than 0.4 f'_s.Only two beams with an effective prestress of more than 0.4 f'_s failed at a shear stress less than that predicted by equation *(11-10).*

Equation *(11-10)* is simplified and generally conservative approximation of equation *(11-1*7). The use of equation *(11-10)* is limited to members having prestress equal to 40 percent of the tensile strength of all the flexural reinforcement. When using this equation V_c need not be taken as less than $2\sqrt{f'_c}\, b_w d$ and shall not be greater than $5\sqrt{f'_c}\, b_w d$. The lower limit of $2\sqrt{f'_c}$ is greater than the lower limit of 1.7 $f'_c\, b_w d$ for equation *(11-1*7) which has no restriction on the amount of prestress in the reinforcement. The upper limit of $5\sqrt{f'_c}\, b_w d$ serves as a restriction on V_{cw}.

The basic purpose of shear design is to arrive at members and structures that under severe

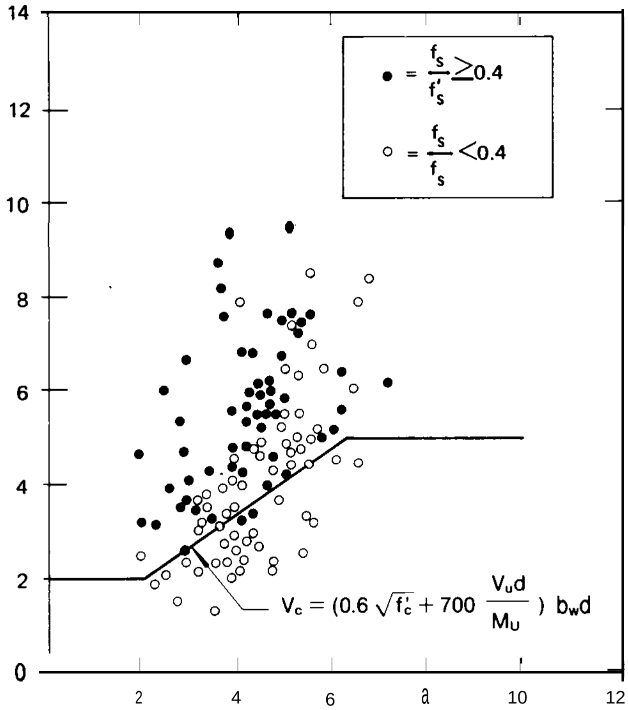


Fig. 5.11 -Alternate equation for computing V_c plotted against test results for beams without web reinforcement."

'Figure taken from a paper "Shear and Torsion in Prestressed Concrete" by Dr. Eivind Hognestad, Portland Cement Association, presenting during PCI Prestressed Concrete Design Seminar, Chicago, June 1971.

overloads will develop their ultimate flexural strength without evidence of serious shear distress. Such members possess considerable toughness and ductility, and they can accommodate many unforeseen circumstances safely. Because of the undersirability of a non-ductile shearfailure, ACI 318-83 requires minimum shear reinforcement for many applications in accordance with equations *(11-14) or (11-15). Section* 77.5.5.7 *of AC/* 378-83 states exceptions to the minimum shear reinforcement requirements which includes slabs, footings, joists, shallow beams, and circumstances where the design shear force, V_u, is less than one-half of the shear force that can be carried by the concrete V_c. However, a minimum amount of stirrup reinforcement is necessary in all post-tensioned joists, waffle slabs, and T-beams to provide a means of supporting tendons in the design tendon profile. When tendons are not adequately supported by stirrups, local deviations of the tendons from the smooth parabolic curvature assumed in design may result during placement of the concrete. When the tendons in such cases are stressed, the deviations from the intended curvature tend to straighten out and this process may impose large tensile stresses in webs of post-tensioned beams, joists, or waffle slabs. Severe cracking has been observed in several instances where no stirrups were provided. Unintended curvature of the tendons may be avoided by securely tying tendons to stirrups which are rigidly held in place by other elements of the reinforcing cage. For bundles of two to four tendons, ties to a minimum of No. 3 stirrups at 2 ft. 6 in. centers is suggested, and for bundles of five or more tendons, ties to a minimum of No. 4 stirrups at 3 ft. 6 in. centers is recommended. This amount and spacing of stirrups is recommended even when the magnitude of the shear stress is such that no stirrups are required under the provisions of *Section 7 7.5.5.7 of AC/ 378-83.* In most cases, closer stirrups spacings will be required to satisfy the shear reinforcement requirements of *ACI 378-83.*

5.3.7 Flexure and Shear in One-Way Slabs

5.3.7.1 Flexure in One-Way Slabs

Post-tensioning tendons exert loads and moments on slabs, opposing gravity loads, moments, and their flexural stresses. Thus, the post-tensioned member must resist only a portion of the dead and live loads in flexure, with the remaining portion counter-acted or balanced by the action of tendons. Through it's anchors, the tendon also exerts an axial compressive stress which reduces the flexural tensile stress resulting from the load not balanced by the tendons. The amount of tendons is selected by satisfying flexural tensile stress limits prescribed by ACI 318-83. The maximum net tensile stress limit by ACI 318-83 is 12 $\sqrt{f'_c}$, but as this exceeds the modulusof rupture, (conventionally assumed as 7.5 $\sqrt{f'_c}$), special considerations of deflection are necessary when tensile stresses exceed 6 $\sqrt{f'_c}$.

As indicated by *Fig.* 5.8, the behavior in flexure of one-way slabs at loads less than the cracking load is linear and elastic, and deflections can be predicted using gross section properties. Where one-way slabs are designed for service load tensile stresses greater than them modulus of rupture (between 7.5 $\sqrt{f'_c}$ and 12 $\sqrt{f'_c}$), behavior remains nearly linear, but at a stiffness less than predicted by gross section properties. The moment gradient in negative moment regions is steep, and cracking is confined to a very narrow zone, extending from the support face only a few slab thicknesses at most. Thus, the amount of bonded steel crossing the cracking zone can have little effect on the width of the cracking zone or on crack spacing. it is only necessary to provide the code specified amount of mild steel to improve flexural behavior of this narrow zone, as tension stresses drop off sharply away from the support face. Mid-span regions exhibit more gradual moment gradients than do support regions. Thus, a small amount of bonded mild steel can be more effective in distribution and control of cracks in positive moment areas. By contrast, simple span beams post-tensioned with unbonded tendons and containing no bonded reinforcement behave after cracking as tied arches. With increasing load and rotation, the single initial crack increases in width and progresses toward the compression face until finally an abrupt compression failure occurs. This effect is eliminated in both slabs and beams by providing the amount of conventional bonded reinforcing steel specified by Section 78.9 of *ACI 318-83.*

Cracking due to high tensile stresses reduces stiffness, and prediction of deflections can no longer be based on elastic gross section properties. The bi-linear approach to deflection prediction specified by ACI 318-83. Section 78.4.2(c) does not adequately describe the behavior of continuous members. The bi-linear approach assumes that the total deflection can be predicted as the sum of the deflection up to cracking calculated by elastic means, and an additional deflection based on the cracked section properties. This approach is approximately correct for simply supported single span members, since the properties of a single section dominate the effective stiffness, and a single load pattern dictates deflection behavior. however, for continuous members two or more sections and load patterns affect the behavior. For most continuous structures, the maximum moments at a given section must be found by evaluating the effects of three load patterns: loads on all spans, loads on adjacent spans, and loads on alternate spans. Since these load patterns can occur in any order and at any time during the life of the structure, it is apparent that the slab must be considered to be cracked at support and mid-span sections where tensile stresses exceed 7.5 $\sqrt{f'_c}$. This produces an effective stiffness at these sections less than that based on gross spection properties. Tests, however, show that the effective stiffness is greater than that based on these very limited portions of the member. In addition, the calculation of cracked section properties is not straightforward. While the force in the unbonded tendons is not affected by local strains, the forces in bonded reinforcing are completely dependent on local strains. The behavior of a single given cross section can be modeled by considering it to be a column under an axial load equal to the effective prestress and subjected to moment. From equilibrium considerations, the crack depth and steel strain can be found by trial and error, and the transformed section properties calculated. Sections away from peak moments remain uncracked and elastic, so deflection behavior cannot be determined from the properties of a single section, and the method of Section 18.4.2(c) of ACI 318-83 underestimates the stiffness of continuous members.

Results of tests give a better picture of the effective stiffness. Fig. 5.12 compares behavior at different load stages for a three-span slab designed for service tensile stresses of 9 $\sqrt{f'_c}$, but loaded to stress levels of 12 $\sqrt{f'_c}$. Degradation of stiffness was maximized by the increased cracking due to service overloading. Had the slab been designed for the higher loads required to produce tensile stresses of 12 $\sqrt{f'_c}$, it would have contained additional bonded reinforcement sufficient to provide the corresponding higher required ultimate capacity. It therefore would have exhibited slightly higher stiffness due to the effects of higher transformed section properties. The straight line is a proposed approximation for effective stiffness of continuous rectangular members containing the code specified amount of bonded reinforcement. As can be seen, Fig. 5.12 is conservative for the range of tensile stresses between 6 $\sqrt{f'_c}$ and 10 $\sqrt{f'_c}$, but somewhat overestimates the

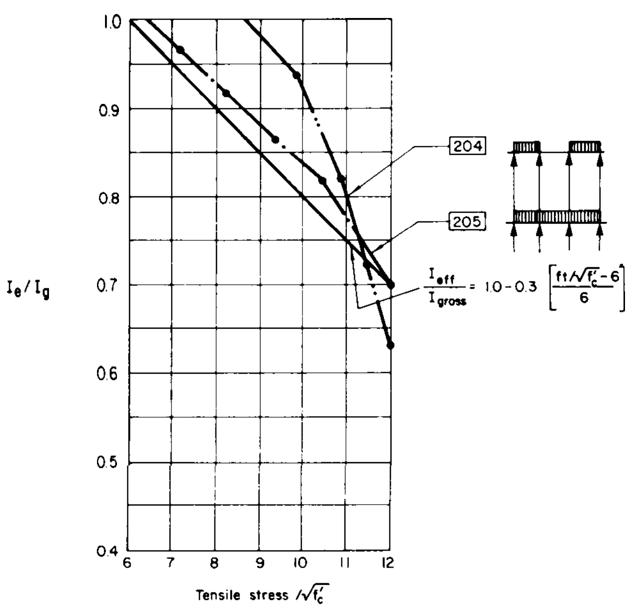


Fig. 5.12 — Effective I for Post-Tensioned One-Way Slabs Designed to Tensile Stress Above 6 $\sqrt{f'_c}$

effective stiffness as $12\sqrt{f'_c}$. However, at $12\sqrt{f'_c}$ loads are higher than the design service load for the test specimen, and it is therefore reasonable to expect a loss of stifffness at that particular point for the test slab.

5.3.7.2 Shear in One-Way Slabs

Typically, one-way slabs are subjected to low shear stresses since high concentrated loading is uncommon. Compared to tee beams, the available shear area is large, and typical slab design shear stresses are well below the lower bound shear capacity of $2\sqrt{f'_c}$ specified by ACI 318-83. Where high shears are encountered, such as from high concentrated loading near supports, equation (1 l-10) of ACI 318-83 may be used to more closely reflect available shear capacity. For conditions of high shear due to permanent concentrated loads, it is recommended that tendons be harped under the load to maximize the effective depth, d, and the shear carried by the tendon.

5.3.8 Flexure and Shear in Flat Plates

5.3.8.1 Flexure in Flat Plates

It is helpful to the understanding of post-tensioned flat plates to forget the arbitrary column strip, middle strip and moment percentage tables which have long been familinr to the designer of reinforced concrete flat plates. Instead, behavior can best be understood by examinining the mechanics of the action of the tendons.

The load balancing approach is an even more powerful tool for examining the behavior of two-way systems than it is for one-way members. By the balanced load approach, attention is focused on the loads exerted on the slab by the tendons, transverse to the plane of the slab. As for one-way slabs, this typically means a uniform load exerted upward along the major portion of the central length of a tendon span, and statically equivalent downward load exerted over a short length between the points of reverse curvature. In order to apply an essentially uniform upward load over the entire slab panel and to satisfy some rationality of statics, these downward loads should be reacted by another structural element. The additional element could be a beam or wall as in the case of one-way slabs, or by columns in a two-way system. However, a look at a plan view of a flat plate reveals that columns provide an upward reaction for only a very small area. Thus, to maintain statical rationality, we must provide, perpendicular to the above ("primary") tendons, another set of tendons ("secondary") to provide an upward load under the downward load. Remembering that the downward load of the "primary" tendon system occurs over a relatively narrow width under the reverse curvatures and that the only available exterior reaction, the column, is also relatively narrow, it becomes obvious that the "secondary" tendon system should be in narrow strips or bands passing over the columns. There are two ways of accomplishing this two-part tendon system to obtain the nearly uniform upward load we desire for ease of analysis. Tendons of the "primary" system could be spaced uniformly in each of two directions and reacted by a "secondary" system along column grid lines in each direction. This method would fit within preconceived ideas of two-way slabs, as the "secondary" tendons are then analogous to two-way beams, but it would not recognize the full potential of the balanced load approach, nor would it be practicable for structures whose columns are not aligned on rectangular grids, The second method does recognize the full potential and, by illustration, aids in the understanding of the contribution of the tendons.

The power of the second method becomes very clear if we examine a slab which has columns of irregular layout. For an example, with reference to Fig. 5.13, let us assume that the "odd" numbered grid lines are offset one-half bay from the "even" numbered grid lines, but columns on a given "letter" grid are aligned. If we reverted to the "column strip" approach illustrated in Fig. 5.14 as for conventionally reinforced slabs, we would find that each span which started in a "column strip" would end in a "middle strip" and tracing of load paths, a rational analysis, and proportioning reinforcement would become difficult if not impossible. So let us return to the basic idea of balancing loads with tendons. The "primary" system of tendons (parallel to "letter" grid lines) can be accomplished with little regard for column location. It is only necessary to place the high points of the tendon profile (where reverse curvature and downward load occur) at the intersection of the tendons with the "number" grid lines. This system is then reacted with the "secondary" tendons placed on the "number" grid lines as shown in Fig. 5.15. By this procedure, the portion of the gravity load balanced by the tendons is carried directly to columns, without any flexural action of the slab. Since this balanced load is typically a large portion of the permanent load on the slab, errors in analysis which are due to incorrect assumptions of load path are a

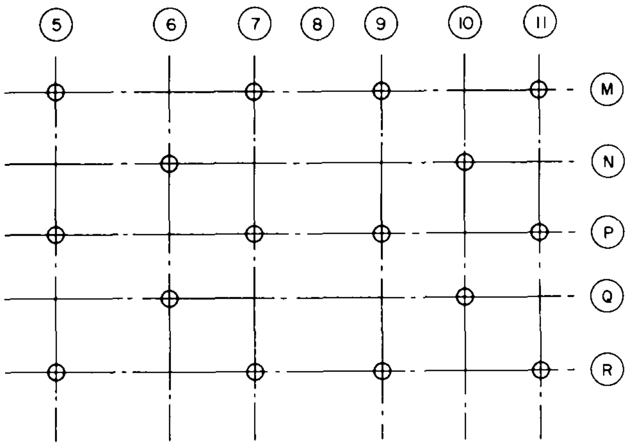


Fig. 5.13 Irregular Flat Plate Column Layout.

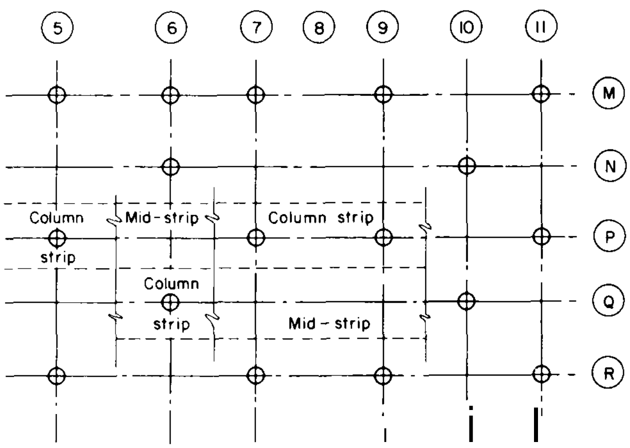


Fig. 5.14 — Column Strips and Middle Strips with Irregular Column Layout

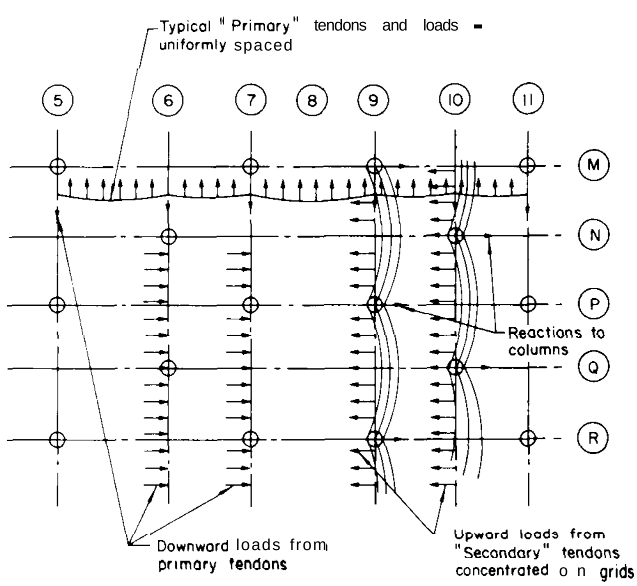


Fig. 5.15 — Load Balancing with Bonded Tendons for Irregular Column Layout

function of relatively small loads, and thus are small. The possible consequences of such errors can be investigated by examining the behavior of the slab under overloads.

Tests and applications have demonstrated that a post-tensioned flat plate behaves as a flat plate regardless of tendon placement. The effects of the tendons are, of course, critical to the behavior as they exert loads on the plate as well as provide reinforcement. Since the tendons exert transverse loads on the plate, these loads may be considered like any other dead or live loads. since the tendon effect is opposite to the effect of gravity loads, the net loads causing bending are relatively small. An additional effect of the tendons is the axial precompression which counteracts flexural tensile stresses. Therefore, at service dead load, the net downward loads which cause bending in the plate are normally very low and the slab is essentially under uniform axial compression.

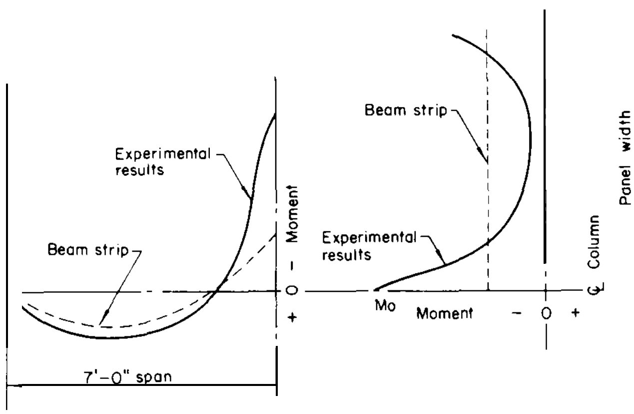


Fig. 5.16 — Distribution of Moments at Midspan in Flat Plate

Examination of the distribution of moments in Fig. 5.16 reveals that negative moments are sharply peaked in the immediate vicinity of the column and that the moment at the column face is several times the moment midway between columns. Since the largest moments are at the column faces, first cracking occurs there at rather low net loads. As the loading is further increased, these cracks extend out from the column faces and positive moment cracks form at mid-span. Finally, full yield line patterns are formed, signaling flexural failure.

Shear behavior will be more fully discussed later, but must be briefly discussed here as it is intimately linked to flexure. First cracking occurs at the column faces at an early load stage. For square panels and equal pre-compression in both directions, the first crack forms at the faces perpendicular to the reinforcing which has the smaller effective depth, regardless of the distribution of tendons. These cracks are quite small and have no significant effect on the stiffness of the slab. However, as cracking extends toward mid-panel with increasing load, the initial cracks at the column face increase in width and depth. This zone is of utmost importance to the shear strength of the slab, and for this reason crack width and depth should be minimized. While compression due to prestress delays the formation of cracks, it is less efficient in controlling crack widths and spacings than nonprestressed bonded reinforcement, placed in the top of slabs immediately adjacent to and above the column. The code specified amount of bonded nonprestressed reinforcement controls and distributes cracks, and allows full mobilization of the potential shear and flexural strength in the column zone. Bonded reinforcement uniformly distributed in the top of slabs within lines one and one-half times the slab thickness either side of the column toughens the punching zone and is effective for maximizing the shear capacity. Bars placed outside that immediate column zone are of no value in controlling cracks in the shear zone. Thus, all mild steel required for support sections should be placed as close to the column vicinity as construction practice will allow. Although bonded tendons provide some control of cracking due to their strain compatibility, few slabs are constructed with bonded tendons, and very limited testing has been conducted on such slabs. Bonded reinforcement placed near mid-panel does not become effective in crack control nor contribute to the flexural capacity until the high load levels where yield lines start to develop. Even then, negative moment strains are much lower near mid-panel than in the column region, so that the full yield force of such bars may never be realized.

In mid-span regions, positive moments are almost uniformly distrubuted across the panel width as shown in Fig. 5.17. First cracking, therefore, occurs much later at mid-spans than at supports. However, since such positive moment cracking is not confined to a small zone there is a noticeable but not marked loss in stiffness caused by that cracking. The behavior of a member which is reinforced only by unbonded tendons has been described as similar to that of a shallow tied arch. This behavior has been observed in one-way statically determinate members, but it appears to

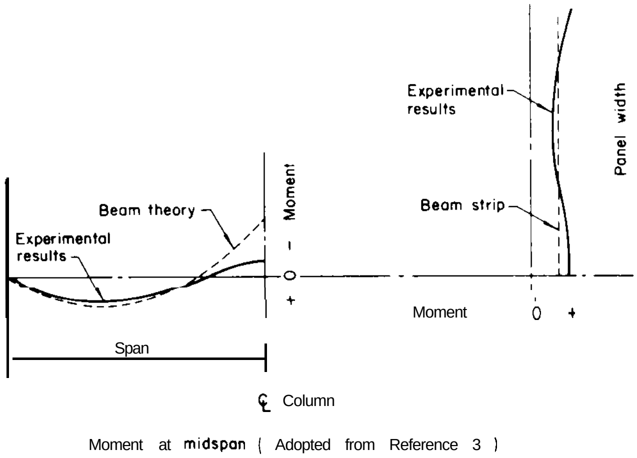


Fig. 5.17 — Distribution of Moments at Midspan in Flat Plate

be lesssignificant for redundant, lightly prestressed structures such as continuous one-way slabs or two-way plates, slabs and waffles. It has been shown in all two-way tests that bonded, mild reinforcing is required in positive moment regions only where ultimate flexural strength requirements exceed the flexural capacity provided by tendons alone, and that no minimum crack control bars are necessary for satisfactory behavior under service or ultimate loads. However, ACI 318-83 requires positive moment bonded reinforcement proportioned in accordance with equation (18-7) where the computed tensile stress in the concrete at service load exceeds $2\sqrt{f'_c}$.

5.3.8.2 Shear in Flat Plates

The ACI 318-83 Building Code was the first to contain specific provisions for punching shear in prestressed flat plates. Likewise, until recent years there was little research on moment transfer in prestressed slabs. The ACI Committee 426 review of code provisions for shear for reinforced concrete slabs has pointed out the need for careful consideration of the effects of transfer of moment as well as transfer of shear between a slab and a column. Thus, while typical interior slab/column connections may not be required by analysis to transfer unbalanced vertical load moments or lateral load moments, moment transfer may actuallyoccurdue to pattern live loading, temperature movements or deformations under lateral loading caused by flexibility of the primary lateral resistance system. Further, for conventional reinforced flat plates, and some prestressed plates as well, punching failure at a column can cause load and moment to be transferred to adjacent columns, and a horizontal progressive shear failure can then theoretically be propagated if the design shear capacity is based on vertical load only. With proper consideration of moment transfer and detailing, this possibility is avoided. The following steps are recommended for shear design of prestressed flat plates:

1. Use the equivalent frame procedure of ACI 318-83 Section 13.7 (excluding Sections 13.7.7.4 and 13.7.7.5) to obtain moments for frame members including unbalanced moments at joints. Use of "prismatic" column stiffness rather than equivalent frame stiffness, while conservative for moment transfer at exterior columns, will lead to underestimation of moment transfer at interior columns, and will also lead to inadequate proportioning of positive moment flexural capacities at exterior spans. Moment capacity provided at an exterior column based on moments obtained using prismatic column stiffness cannot be mobilized due to torsional flexibility of slab sections away from the column, and other sections of the spans will then be required to resist more moment than anticipated. An assumption of zero column stiffness will usually give conservative moment values in the slab, but is also unsatisfactory because there are no unbalanced moments which must be recognized for shear calculations. The equivalent column stiffness of Section 13.7.4 of ACI 318-83 will give good approximation of frame member moments and unbalanced moments at joints, and should always be used.
2. Detail tendons and reinforcing bars for maximum design strength and reserve strength. To maximize design strength and reserve strength, control cracking and toughen the punching zone, bonded steel reinforcement should be uniformly distributed within lines one and one-half times the slab thickness either side of the column. To maximize reserve post-failure strength, a bundle of tendonsshould pass through the column within the column reinforcing cage in at least one direction, if not both. Since the unbonded tendon is not sensitive to local strains, this bundle will provide post-punching failure vertical load capacity in direct tension. Thus it can allow large vertical deflections without rupture of the tendon, and prevent collapse of the slab.

An accepted method for evaluating total shear stress is that reflected in provisions of ACI 318-83. The critical section for shear is assumed to be at d/2 from the support face. The general expression for shear stress is:

$$\phi v_u = \frac{V_u}{b_o d} + \frac{\alpha M_t c_3}{J_c}$$

Where: V_u = factored shear force at section

v_u = shear stress at design (factored) loads

b_o = perimeter of shear section at d/2 from face of column as defined by ACI 317-83, Chapter 11

d = distance from centroid of tendontocompressionfaceindirection of moment transfer, but need not be less than 0.8 h, where h is member thickness

ϕ = capacity reduction factor for shear, 0.85.

a = fraction of moment transferred by shear

$$a = 1 - \frac{1}{1 + 2/3\left(\dfrac{c_1 + d}{c_2 + d}\right)^{1/2}}$$

c_1 = support dimension in the direction of moment transfer

c_2 = support dimension perpendicular to c_1

c_3 = distance from centroid of critical shear section to extreme fiber in direction of moment transfer

M_t = moment to be transferred to column

J_c = polar moment of inertia of critical section

For the simplest case, a square interior column, the shear stress is calculated as shown in Fig. 5.18 with additional notation and constantsas follows:

$$c_1 = c_2\ ;\ c_3 = \frac{(c_1 + d)}{2}$$

$$b_o d = 4d\ (c_1 + d)$$

$$J_c = \frac{d\ (c_1 + d)^3}{6} + \frac{(c_1 + d)\ d^3}{6} + \frac{d\ (c_2 + d)\ (c_1 + d)^2}{2}$$

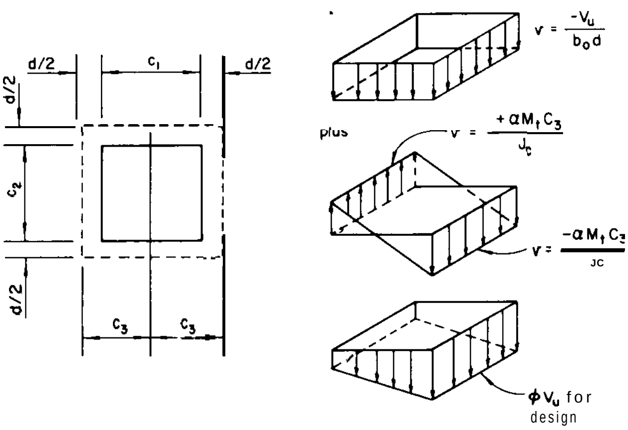


Fig. 5.18 — Moment-Shear Interaction — Relationships for Interior Columns

Shear-Stress Equations:

$$v_c = \frac{V}{A_c} + \frac{\alpha(M_t - Vg)\,c_{AB}}{J_c};\ \text{(Eq. 1a)};$$

$$v_c = \frac{V}{A_c} - \frac{\alpha(M_t - Vg)\,c_{CD}}{J_c};\ \text{(Eq. 1b)}$$

$$\alpha = 1 - \frac{1}{1 + 2/3\,(c_t/c_m)^{1/2}}\ \text{(Eq. 2)}$$

Critical-Section Properties:

$$A_c = d\,(c_m + 2c_t);\ c_{AB} = \frac{c_t^2 d}{A_c}$$

$$c_{CD} = c_t - c_{AB};\ g = c_{CD} - \frac{c_1}{2}$$

$$J_c = \frac{dc_t^3}{6} + \frac{c_t d^3}{6} + c_m d c_{AB}^2 + 2c_t d\,(c_t/2 - c_{AB})^2$$

For exterior columns, part of the unbalanced moment is transferred by the eccentricity of the centroid of the shear section, g, with respect to the column centroid. Thus, M_t, the moment to be transferred by shear and torsion, is the unbalanced moment minus $V_u g$, the moment transferred by shear eccentricity. The calculation procedure and equations for exterior and corner columns are presented in Figs. 5.19 and 5.20, respectively.

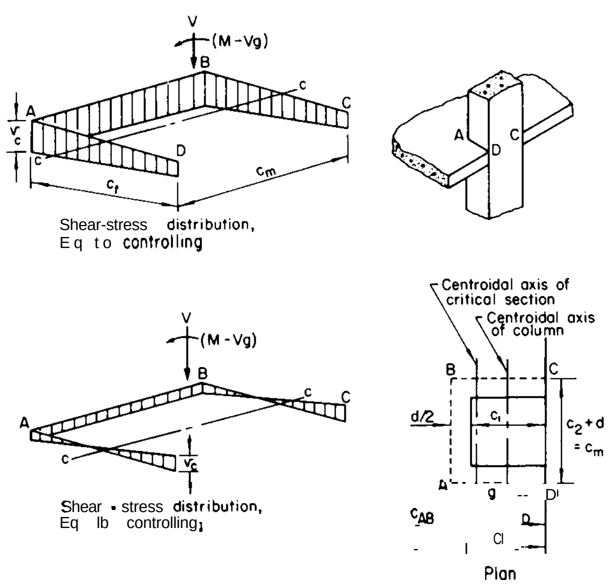


Fig. 5.19 — Moment-Shear Interaction-Relationships for Edge Column Connections

Review of available vertical load punching tests indicate that equation (11-13) of ACI 318-83 conservatively predicts shear strength at interior columns of prestressed two way slabs. Fig. 5.21 shows the results of single-column slab specimen punching shear tests, and results of multi-panel slabs tested in shear. Equation (11-13) (expressed in terms of shear rather than stress) is:

$$V_{cw} = b_o d\,(3.5\sqrt{f'_c} + 0.3\,F/A) + V_p \text{ Where:}$$

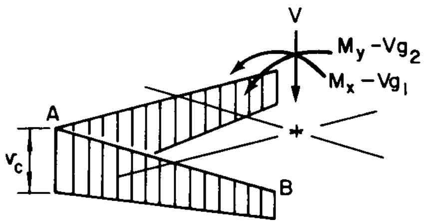
F/A = average precompression in the direction of moment transfer. Effective limit 500 psi, minimum 125 psi.

V_p = shear carried through critical section by tendons. For thin slabs, this 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.

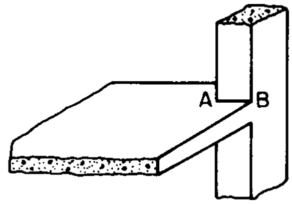
Section 11.1.2.2 of ACI 318-83 provides for use of equation (11-13) for design of interior columns of post-tensioned flat plates (where no portion of the column cross section is closer to a discontinuous edge than four times the slab thickness). For exterior columns, the allowable shear stress in ACI 318-83 is governed by equation (11-36)

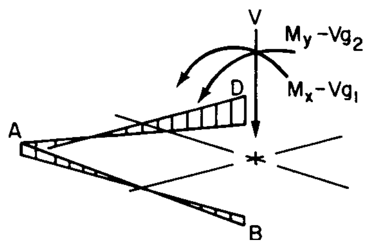
$$V_c = (2 + 4/\beta_c)\sqrt{f'_c}\,b_o d$$

but not greater than $4\sqrt{f'_c}\,b_o d$. β_c is the ratio of

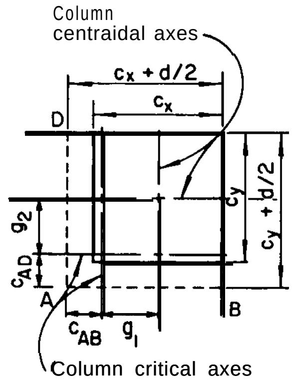


Shear -stress distribution,
Eq. 3a controlling





Shear-stress distribution,
Eq. 3b or 3c controlling



Plan

Shear-Stress Equations

$$v_A = \frac{V}{A_c} + \frac{\alpha_x (M_x - Vg_1)\, c_{AB}}{J_{c_x}} + \frac{\alpha_y (M_y - Vg_2)\, c_{AD}}{J_{c_y}} \qquad \text{(Eq. 3a)}$$

$$v_D = \frac{V}{A_c} + \frac{\alpha_x (M_x - Vg_1)\, c_{AB}}{J_{c_x}} - \frac{\alpha_y (M_y - Vg_2)(c_y + d/2 - c_{AD})}{J_{c_y}} \qquad \text{(Eq. 3b)}$$

$$v_B = \frac{V}{A_c} - \frac{\alpha_x (M_x - Vg_1)(c_x + d/2 - c_{AB})}{J_{c_x}} + \frac{\alpha_y (M_y - Vg_2)\, c_{AD}}{J_{c_y}} \qquad \text{(Eq. 3c)}$$

$$\alpha_x = 1 - \frac{1}{1 + \frac{2}{3}\left(\frac{c_x + d/2}{c_y + d/2}\right)^{1/2}} \text{ (Eq. 4a)}; \qquad \alpha_y = 1 - \frac{1}{1 + \frac{2}{3}\left(\frac{c_y + d/2}{c_x + d/2}\right)^{1/2}} \text{ (Eq. 4b)}$$

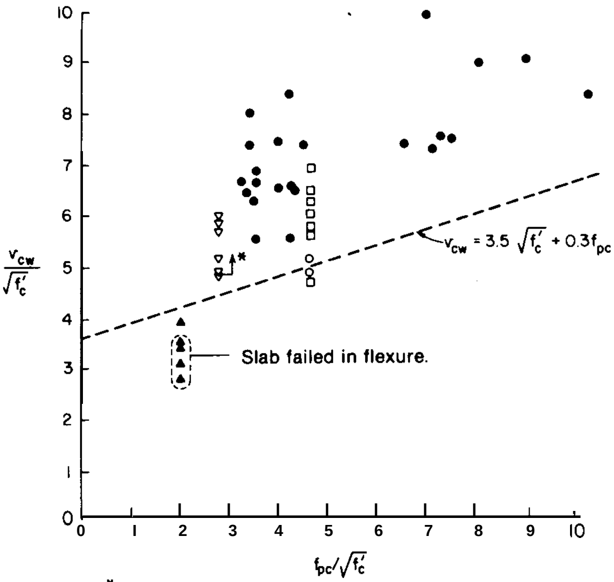
Critical-Section Properties

$$A_c = (c_x + c_y + d)\, d; \qquad c_{AB} = \frac{(c_x + d/2)^2 d}{2A_c}, \qquad c_{AD} = \frac{(c_y + d/2)^2 d}{2A_c}$$

$$J_{c_x} = \frac{d(c_x + d/2)^3}{12} + \frac{(c_x + d/2)d^3}{12} + (c_y + d/2)\,d\,c_{AB}^2 + (c_x + d/2)\,d\left(\frac{c_x + d/2}{2} - c_{AB}\right)^2$$

$$J_{c_y} = \frac{d(c_y + d/2)^3}{12} + \frac{(c_y + d/2)d^3}{12} + (c_x + d/2)\,d\,c_{AD}^2 + (c_y + d/2)\,d\left(\frac{c_y + d/2}{2} - c_{AD}\right)^2$$

Fig. 5.20 — Moment-Shear Interaction-Relationships for Corner Column Connections .



*Did not fail in punching shear; maximum stress attained

Fig, 5.21 — Shear Test Data Versus Equation (11-13) of ACI 318-83

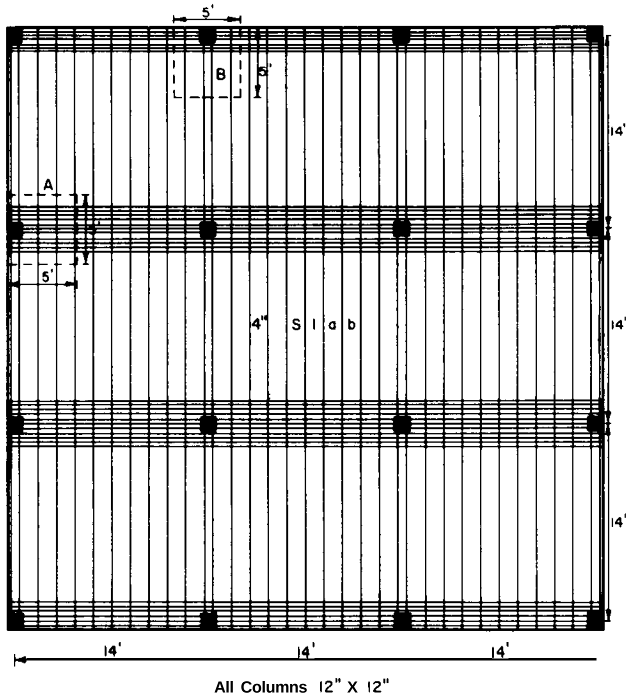


Fig. 5.22 — Two-Thirds Scale Model of Prestressed Concrete Slab

the long side to the short side of the column, and b_o is the perimeter of the critical section. However, tests of edge column — plate connections at the University of Illinois at Champaign — Urbana published in 1982* indicate that equation (11-13) is also satisfactory for design of exterior columns. Fig 5.22 provides details of the two-thirds scale model used for the University of Illinois tests. Two specimens were tested with the banded tendons in the direction of moment transfer ("A" in Fig. 5.22), and two specimens were tested with the banded tendons perpendicular to the direction of moment transfer ("B" in Fig. 5.22). A comparison of the measured and predicted shear strength is presented in Table 5.3. As indicated by Table 5.3, the measured strength of the four test specimens was greater than the strength predicted by equation (11-13), and much greater than the upper limit of $4\sqrt{f'_c}$ for equation (11-36). On the basis of these tests, equation (11-13) is considered to be satisfactory for design of exterior columns of post-tensioned flat plates. Equation (11-13) is permitted for use for this purpose in jurisdictions covered by the Uniform Building code, and a revision of ACI 318-83 to permit use of equation (1 l-l 3) for exterior columns is under consideration by ACI Committee 318.

Table 5.3 Comparison of Measured and Predicted Shear Strength Using ACI Model

	PREDICTED SHEAR STRENGTH				MEASURED STRENGTH		
	$v_{cw} = 3.5\sqrt{f_c} + 0.3f_{pc}$		$v_c = 4\sqrt{f_c}$				
SPECIMEN	V_u	M_u	V_u	M_u	V_u	M_u	MODE OF FAILURE
S1	10.58	40.02	7.76	20.74	12.98	49.62	Flexure
s2	10.17	44.93	12.38	30.45	18.71	46.27	Shear
S3	13.47	33.17	12.13	29.83	15.10	37.24	Flexure
s4	22.80	34.90	20.68	31.72	25.59	3908	Shear

Ultimate shear V_u in units of kips.
Ultimate moment m_u in units of k-ft.

Tests at the University of Washington have confirmed the applicability of the above approach for calculations of the shear capacity of interior slab-column connections transferring moment as well as shear. The shear stress, v_{cw}, is compared to the maximum total vertical plus torsional shear stress, and F/A is the average pre-compression in the direction of moment transfer. Exterior column connections must be detailed to control cracking due to torsion and to prevent loss of stiffness. This

'Sunidja. Harianto, Foutch, Douglas A., and Gamble, William A., "Response of Prestressed Concrete Plate — Edge Column Connections," Structural Research Series No. 498, Report No. UILU-ENG-82-2006, University of Illinois at Urbana-Champaign, Urbana, Illinois. March 1982.

requires that some tendon anchorages be placed within the column width and that auxilliary closely spaced bonded reinforcement be placed through the column and as close as practicable to the column in each direction. Theflexural capacity of the tendons and the bonded reinforcement within lines 3h/2 either side of the column must exceed the fraction of the moment not transferred by shear, $(1 - \alpha) M_t$.

5.3.8.3 Details of Tendons and Bonded Reinforcement in Flat Plates

Many arbitrary distribution arrangements have been successfully used, proportioning from 50% to 100% of the total bay force in "column strips" and from 50% to 0% of the total bay force in so-called "middle strips." Most of these arbitrary values are adaptations from design tables for reinforced concrete slabs, and fail to recognize one of the most important actions of post-tensioned tendons. It is important that the designer recognize the fact that tendons exert downward loads at the "high points" that join adjacent parabolic profiles, and these downward reactions should be, as nearly as practicable, resisted by columns, walls, and/or upward tendon loads to achieve minimum deflections and maximum shear capacity. Then for a statically rational distribution of tendons, all the tendons in one direction wpuld be placed thru or immmdiately adjacent to the columns, and the tendons in the perpendicular direction would be spaced uniformly across the bay width. Where irregular column layouts must be used, the advantages of being able to easily trace the path of balanced loads to supports, as discussed in Section 5.3.8.1, becomes an obvious and powerful design tool, but the rationale applies to all two-way systems. In addition to the obvious structural advantages, this "banded" tendon arrangement simplifies the construction sequence, reducing labor cost and construction time. Tendon distribution using the banded system is illustrated by Fig. 5.22. See also, the discussion in Section 6.2.4 and the related figures. Recommended details of reinforcement for banded tendon distribution are as follows:

a) The number of tendons required in the design strip (center to center of adjacent panels) may be banded close to the column in one direction and distributed in the other direction. At least two tendons should be placed through columns in the distributed direction.

b) Minimum A, at columns is A, = 0.00075hl *(equation 18-8 in AC/ 378-83)* where l is the length of the span in direction parallel to that of the reinforcement being determined. At least four bars shall be provided in each each direction in negative moment areas at columns. For determining the amount of bonded reinforcement parallel to the slab edge at exterior columns, the amount obtained from equation (18-8) should be multiplied by the ratio l_3/l_4 where l_3 and l_4 are as defined in Fig. 5.23. (Note: this provision for reinforcement parallel to the slab edge at exterior columns anticipates a revision of ACI 318-83 now under consideration by ACI Committee 318. Also, it reflects the basis of proportioning this reinforcement shown to be satisfactory in model tests of post-tensioned flat plates.)

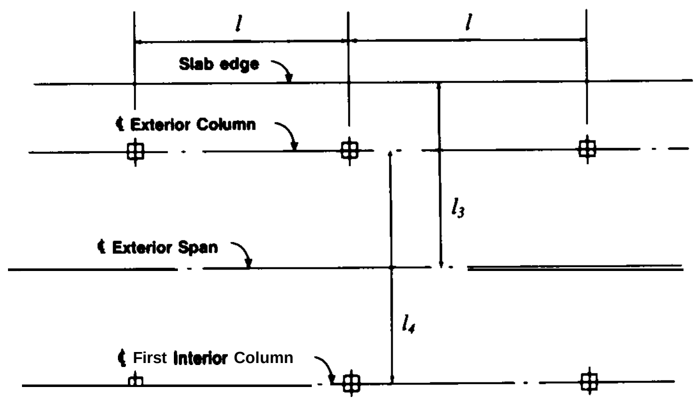


Fig. 5.23 — Adjustment coefficients of ACI 318-83 equation (18-8) for bonded reinforcement parallel to slab edge at exterior columns

c) Place the bonded reinforcement within a slab width between lines that are 1.5h outside opposite faces of column supports *(ACI* 318-83 *Section* 18.9.3.3). Maximum spacing of these bars is 12 in.

d) Minimum length for the negative moment bars shall extend 1/6 the clear span on each side of support.

e) Where service load positive moment stresses are in excess of $2\sqrt{f'_c}$, minimum bonded reinforcement is specified by equation *(18-7) of AC/ 318-83:*

$$A_s = N_c/0.5f_y$$

where N_c is the tensile force in the concrete due to unfactored dead load plus live load.

f) Min. A_s for positive moment (when not considered in plate strength calculations) shall have length at least 1/3 the clear span with the bars centered in the positive moment area.

g) Where bonded reinforcement is used along with unbonded tendons to meet strength requirements (rather than minimum A_s) the bar cut off points should be as specified in Chapter 12 of ACI 318-83.

h) For generally uniform loading, the maximum spacing of single tendons or groups of tendons in one direction should not exceed 8 times the slab thickness, with a maximum spacing of 5 ft. In addition, the spacing of tendons should provide a minimum average prestress of 125 psi on the local slab section tributary to the tendon or tendon group (the section 1/2 of the spacing either side of the center of the tendon group). Special consideration of tendon spacing may be required to accommodate heavy concentrated loads.

5.4 SPECIAL DESIGN AND DETAILING CONSIDERATIONS

5.4.1 Anchorage Zones

Anchorage bearing area must be provided for the tendon anchorages in accordance with *Section*3.7.7. Manufacturers'standard bearing plates and other standard anchorage hardware should be used in all but special applications.

Anchorage plates apply the prestress force in a concentrated manner which involves high local stresses requiring a certain length to spread out over the cross-section of the member. The length required for these localized stresses to distribute is generally called the transmission zone. Depending on the size and location of the anchorages relative to the concrete section, significant tensile stresses can be generated in a direction perpendicular to the tendon in the transmission zone.

If the anchorages are located in a massive section of concrete, the tensile stresses can dissapate without causing distress to the concrete. On the other hand, if the **ancorages** are located in thin section of concrete, reinforcement is usually required to carry the tensile stresses and prevent concrete from splitting.

Leonhardt discusses the distribution and calculation of the tensile bursting stresses in his textbook.* Fig. 5.24, taken from Leonhardt's text, shows the distribution of the tensile stresses in a direction perpendicular to the tendon path. These distribution curves were plotted assuming the width of the bearing plate equal to the width of the concrete section. Therefore, they closely approximate the stress distribution of anchorages on thin members. The stresses imposed by the anchorage immediately behind the plate are

* Leonhardt, F., "Prestressed Concrete-Design and Construction," Second Edition, Wilhelm, Ernest & Sohn. Berlin, Munich, 1964.

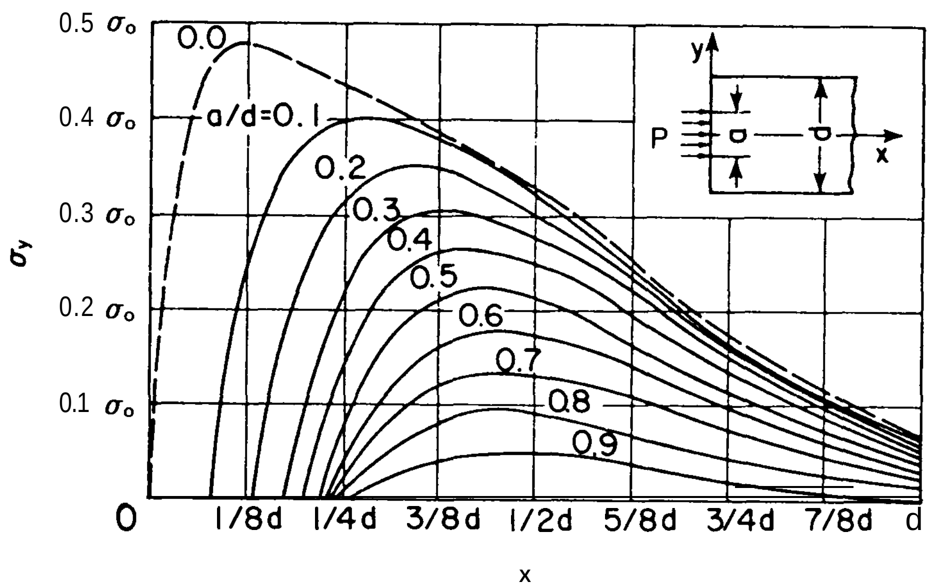


Fig. 5.24

compressive but become tensile at some distance away from the plate depending on the ratio of the depth of the anchor plate, a, to the depth of the concrete section, d. Since only tensile forces are of concern in this context, only that portion of the diagram is shown by Fig. 5.24.

The magnitude of the tensile splitting force can be determined from the following equation:

$$Z = 0.3\,P(1 - a/d)$$

Where:
Z = total splitting or busting force
P = tendon force
a = depth of the anchor plate
d = depth of the concrete section

In Fig. 5.24, $\sigma_0 = P/bd$, where b is the width of the anchor plate.

Sufficient vertical reinforcement acting at a unit stress of 0.6 x reinforcement yield stress to resist the computed value of Z should be distributed within the distance of d/2 of the anchorage location. Normal shear reinforcement may be included in the total area required. A comprehensive investigation of post-tensioned girder anchorage zones was concluded in 1981 at the University of Texas at Austin.** While this research was directed towards larger capacity tendons utilized in webs of box girder bridges, the results are of interest relative to the smaller single strand unbonded tendon anchorages as well.

54.2 Volume Change Restraints

The relative stiffness of post-tensioned beams and columns must be carefully analyzed. Such restraining items as walls which frame into columns should be given special attention, and if necessary, properly designed control joints provided. When a post-tensioned beam is framed into the top of a relatively stiff column, the elastic, shrinkage, and creep shortening of the beam and slab will cause high moments and shears in the column. Furthermore, the prestress force is reduced by the amount that is diverted to column bending. In multi-story and multi-bay frames, these effects are distributed throughout the frame. However, for multi-story frames, the significant effects have been shown to be limited to the first floor columns, and to the post-tensioning forces in the first two or three floors.

As discussed in Section 6.2.8, the restraint forces can be reduced by use of pour strips to temporarily isolate the post-tensioned floor system from restraining columns or walls. In this regard, Fig. 5.25 provides an approximate indication of the proportion of final shrinkage or creep movements vs. time. The figure indicates that about 40 percent of shrinkage or creep takes place within the first 28 days (after casting and after loading, respectively), and that 60 percent takes places within the first 90 days. Accordingly, pour strips left open for these time periods would substantially reduce creep and shrinkage restraint forces. Of course, the pour strip would eliminate the development of any restraint from the isolated vertical elements due to elastic shortening of the floor system at the time of post-tensioning.

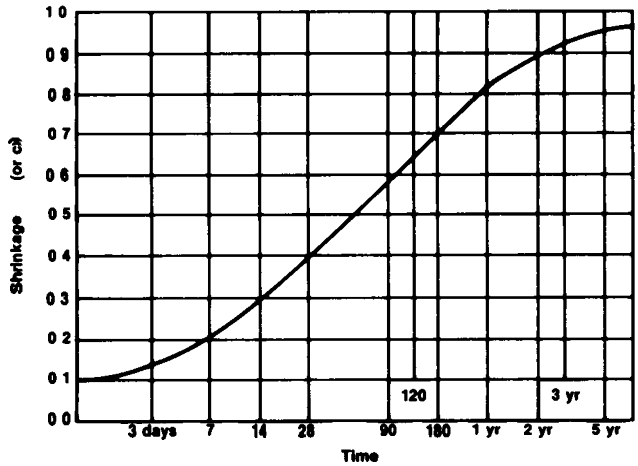


Fig. 5.25 — Approximate proportion of final shrinkage or creep vs. time

A research paper entitled, "Time-Dependent Forces Induced by Settlement of Supports in Continuous Reinforces Concrete Beams"* provides useful design insights and information relative to restraint forces due to bending of columns, and possibly, to some extent, restraint forces from shear walls. In this research, two-span beams were subjected to sudden and progressive deflections of the center supports. The qualitative results are indicated in Fig. 5.26. As indicated, the force, P, required to cause a fixed instantaneous deflection, δ, at the center support reduced substantially with time due to creep of the concrete. Also with reference to Fig. 5.26, when the same

** "Design of Post-Tensioned Girder Anchorage Zones" by W.C. Stone and J.E. Breen. Research Report 208-3F, Center for Transportation Research, The University of Texas at Austin, Austin, Texas 78712. The report is available through the National Technical Information Service, Springfield, Virginia 22161.

*Ghali, Amin, Dilger, Walter, and Neville, Adam M., "Time-Dependent Forces Induced by Settlement of Supports in Continuous Reinforced Concrete Beams", Journal of the American Concrete Institute, November, 1969.

displacement of δ was induced in the beam over a period of time, the maximum force required was reduced to only a fraction of the force required for the same instantaneous deflection.

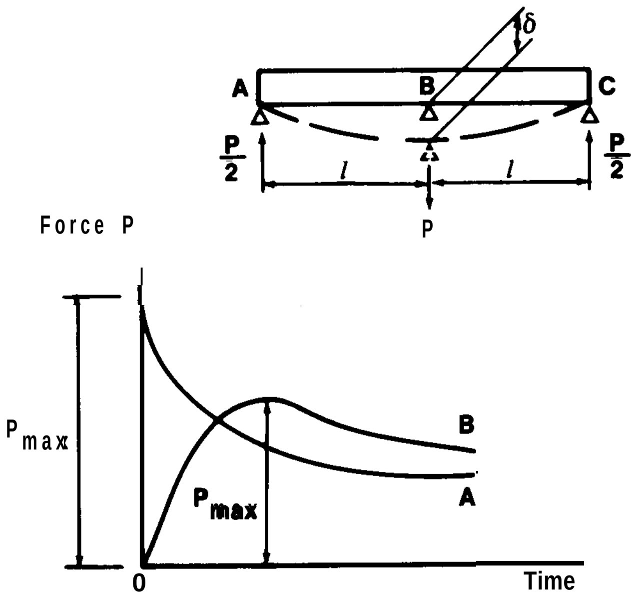


Fig. 5.26 — Time-dependent forces caused by differential settlement of a support in a continuous reinforced concrete beam: A — when settlement occurs suddenly; B — when it occurs progressively

The results of this research are shown on a quantitative basis in Fig. 5.27. As indicated by the dashed line, the force due to an instantaneous deflection without creep relief was a little less than 24 kips. When the same settlement occurred over a time period of 30 days or more, the induced force was 12 kips or less.

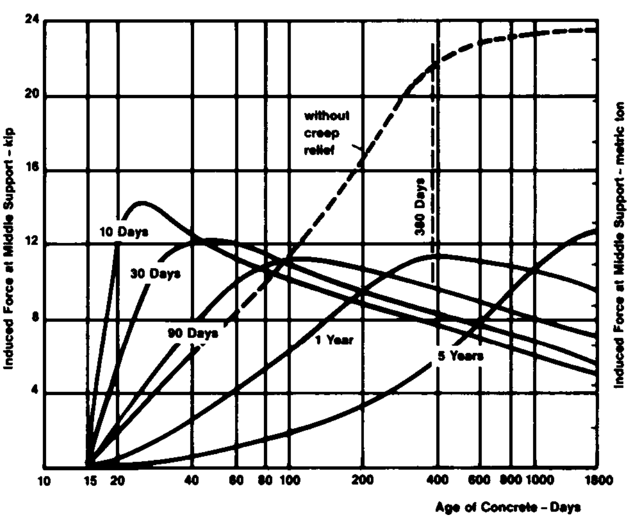


Fig. 5.27 — Change in reaction due to a settlement of 0.92 in. occurring during 10 days, 1 month, 6 months, 1 year, and 5 years

Although only approximate generalizations can be drawn from this specific research, it does suggest that concrete creep in restraining columns (and possibly in walls), may reduce the ultimate restraint moments and forces due to both elastic and creep displacements to approximately 50 percent of the value that would be expected if the total elastic shrinkage and creep displacement occured instantaneously (Creep displacement is about twice the elastic displacement). Accordingly, the total creep restraint moments and forces may be approximately 50 percent of those resulting from the displacement due to elastic and creep shortening due to post-tensioning, and due to concrete shrinkage.

A detail that has been used successfully to reduce restraint forces in post-tensioned slabs supported by walls and still provide a final structural connection between the slab and the wall is shown in Fig. 5.28. The use of the "NMB Splice Sleeve" provides for a delay of 60 days or more before the bent dowel connecting the slab and the wall (which is cut at the middle of the splice sleeve) is connected by filling the splice sleeve with high strength grout. The use of the heavier than usual bent bar connection reduces the number of splice sleeves required. Details on the "NMB Splice Sleeve" may currently (1985) be obtained from: Splice Sleeve North America, 1107 Ninth Street, Suite 700, Sacramento, CA 95814.

See Section 6.2.8 for discussion of other details and procedures to avoid or minimize development of restraint to dimensional changes.

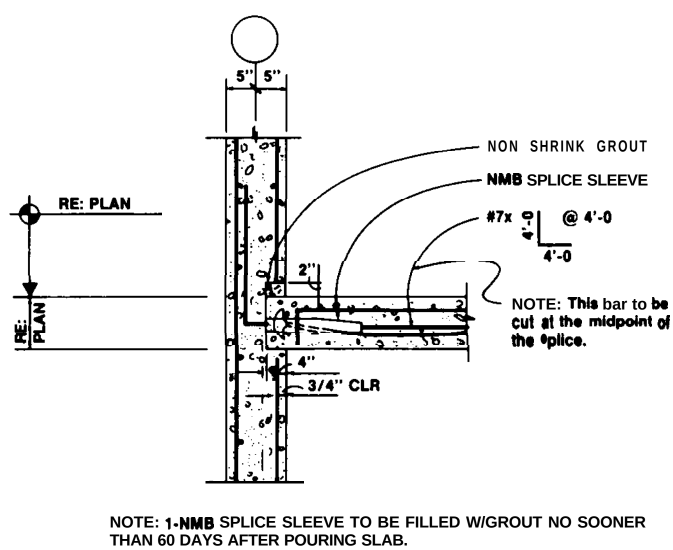


Fig. 5.26 — Connection of post-tensioned slab to wall by use of "NMB Splice Sleeves" (Detail provided courtesy of Richard Weingardt Associates, Denver).

5.4.3 Catastrophic Loading of Beams and One-Way Slabs with Unbonded Tendons

A catastrophic loading such as might occur from an explosion or a severe earthquake which resulted in a failure in one bay of a beam or one-way slab with unbonded tendons could result in a progression of the failure throughout all bays of a multi-bay building. Concern about this potential failure mode under abnormal loadings led to the following provision of Section 2678(j) of the Uniform Building Code:

"One-way, unbonded, post-tensioned slabs and beams shall be designed to carry the dead load of the beam plus 25 percent of the unreduced superimposed live load by some method other than the primary unbonded post-tensioned reinforcement. Design shall be based on the strength method of design with a load factor and capacity reduction factor of one. All reinforcement other than the primary unbonded reinforcement provided to meet other requirements of this section may be used in the design."

This provision provides a secondary means of carrying loads in one-way structures with unbonded tendons under abnormal or catastrophic loading conditions. A provision of this nature is not considered necessary for applications such as two-way flat plates where the system of tendons in the,second direction automatically provides substantial load carrying capacity in event unusual or abnormal circumstances should result in loss of post-tensioning force in one direction.

The amount of bonded reinforcement required by the above Uniform Building Code provision may exceed the amount required for one-way slabs and beams by *Section 78.9 of ACI 378-83.*

5.4.4 Effective Flange Width of Post-Tensioned Tee Beams

The current provisions of *Section 8.70 of ACI* 378-83 have been in the code in the same format for many years. Due to the exclusion of *Sections* 8.70.2, 8.70.3, and 8.70.4 relative to application to prestressed concrete in *Section 78.7.3 of ACI 378*-83, there are no explicit provisions for effective flange width of cast-in-place post-tensioned T-beams. The discussion presented below is presented to provide guidance for flange width determination for cast-in-place post-tensioned T-beams. In addition, a procedure is presented for calculation of the diffusion of the post-tensioning force throughout the effective section at sections adjacent to end anchorages.

The effective width of the compression flange of a T-beam is not constant along the length of the beam and depends:

on the nature of the beam considered (simply supported spans or continuous spans);

on the method of load application (distributed or concentrated);

on the ratio of the thickness of the flange to the depth of the beam and on the existence of any connecting fillets;

on the ratios of the length of the beam between points of zero moment to the width of the web and to the distance between webs.

Tables based on elastic analyses have been published which may be used as a means of determining the effective flange width of isolated or multiple cast-in-place T-beams.* Alternatively, flange width determinations may be made by finite element analysis, or by any other generally accepted elastic analysis procedure. In lieu of these more rigorous analytical procedures, the generally conservative approximations of effective flange width of *Sections **8.10.2, 8.10.3,*** and *8.70.4* of ACI 318-83 may be *used.*

Comparative stress calculations for post-tensioned T-beams based on precise elastic analyses and *Section* 8.70 *of AC/* 378-83 indicate that *Section* 8.70 generally provides conservative results. However, the wider flange widths which normally result from more precise analyses are more realistic, particularly from the standpoint of strength calculations.

The diffusion of the prestressing force from the anchorages, which progressively affects the entire section, may be recognized in an approximate way as follows: †

At the end sections of post-tensioned T-beams adjacent to tendon anchorages, the prestressing moment may be considered effective at the beam end. The prestressing force may beconsidered to be distributed into the main plane of a slab or web from the area of the beam end covered by anchorage hardware within an angle bounded by straight lines above and below the tendon trajectory each inclined at an angle of 33 degrees to the direction of the force.

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* "Recommendations for an International Code of Practice for Reinforced Concrete", American Concrete Institute and the Cement and Concrete Association, 52 Grosvenor Gardens, London S.W. 1 pp **65-70.**

† "International Recommendations for the Design and Construction of Concrete Structures", Second Edition Cement and Concrete Association, 52 Grosvenor Gardens, London S.W. 1 p. 40.

Application of the above procedure is illustrated in Fig. 5.29.

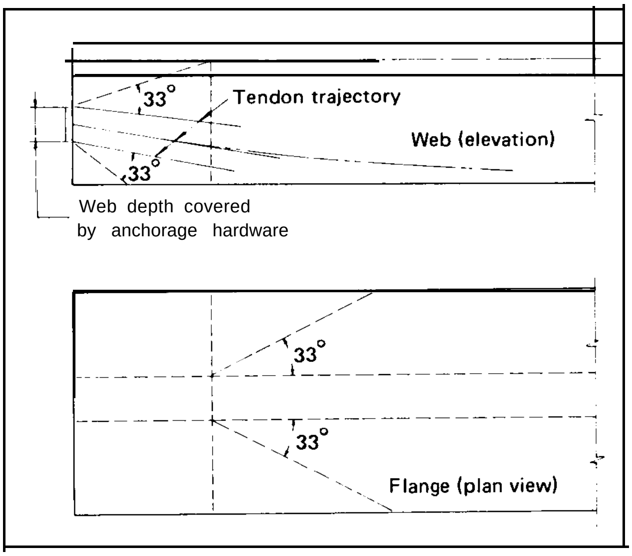


Fig. 5.29 — Recommended procedure for calculation of diffusion of post-tensioning force into tee beams adjacent to anchorage

5.4.5 Stage Post-Tensioning*

Stage post-tensioning may be defined as a prestressing technique in which prestress is applied incrementally to counteract successive loading stages. The applied load may be due to external forces or loads, or due to internal volume changes (elastic shortening, shrinkage and creep). The concept of stage post-tensioning is not new and has been used in conventional post-tensioning work.

The principle and mechanics of stage post-tensioning are illustrated in *Fig. 5.30* for design of a girder to span a subway and to support the loads due to progressive construction of a high rise building above the girder. *Fig. 5.30* shows the construction steps involved in the project and the pertinent stress conditions in the girder corresponding to each construction stage.

Further details on the various steps and post-tensioning stages in the project as shown in *Fig.* 5.30 are as follows:

* For further discussion of stage post-tensioning applications, see "Stage Post-Tensioning — A Versatile and Economic Construction Technique" by William M. Slater in the January/February 1975 Journal of the Prestressed Concrete Institute, Vol. 20. No. 1, pp. 14-27. The discussion presented in this section is extracted from Mr. Slater's article.

Step 1 (Day 1)
Girder construction

Assume that the girder to be constructed is of minimum height H and span Land will eventually support a 12-story building. The compressive strength f'_c of the concrete is specified to be 5000 psi at 28 days.

The formwork, which is supported by scaffolding from below or by a truss spanning between the supports, carried the dead weight D_o of the girder.

Inside the girder there are several ducts which have a predetermined profile. Post-tensioning tendons will subsequently be threaded through these ducts.

Step 2 (Day 5)
Application of Stage 1 post-tensioning (P,)

When the compressive strength of the concrete reaches a minimum of 3000 psi the first post-tensioning force P_1 (Stage 1) is applied. This prestress balances (with W_{B1}) the dead weight D_o of the girder, plus the first stage of applied load W_1 (in this case the dead load of the first three floors).

Note that at the completion of Step 2, a maximum compressive stress of 0.45 f_{ci} is induced at the bottom fiber of the girder at midspan while the maximum permissible tensile stress at the top fiber is $3\sqrt{f'_{ci}}$.

Step 3 (Day 25)
Application of Stage 1 loading W,

The construction of three floors takes place, say from Day 5 through Day 25, applying a total load of W_1 to the girder.

By this time the compressive strength of the concrete has reached almost the specified value of 5000 psi. The top fiber stress at midspan is now 0.45 f'_c compression and the bottom fiber stress is $3\sqrt{f'_c}$ tension.

These stresses are almost the reverse of the stress state at Step 2. Also, the vertical deflection has changed from an upwards induced camber (at a relatively low concrete modulus E_2) to a small downwards deflection (at a concrete modulus E_3).

Step 4 (Day 25)
Application of Stage 2 post-tensioning

Stage 2 post-tensioning P_2 is now applied to balance (with W_{B2}) the additional expected load W_2 from Floors 4 and 5.

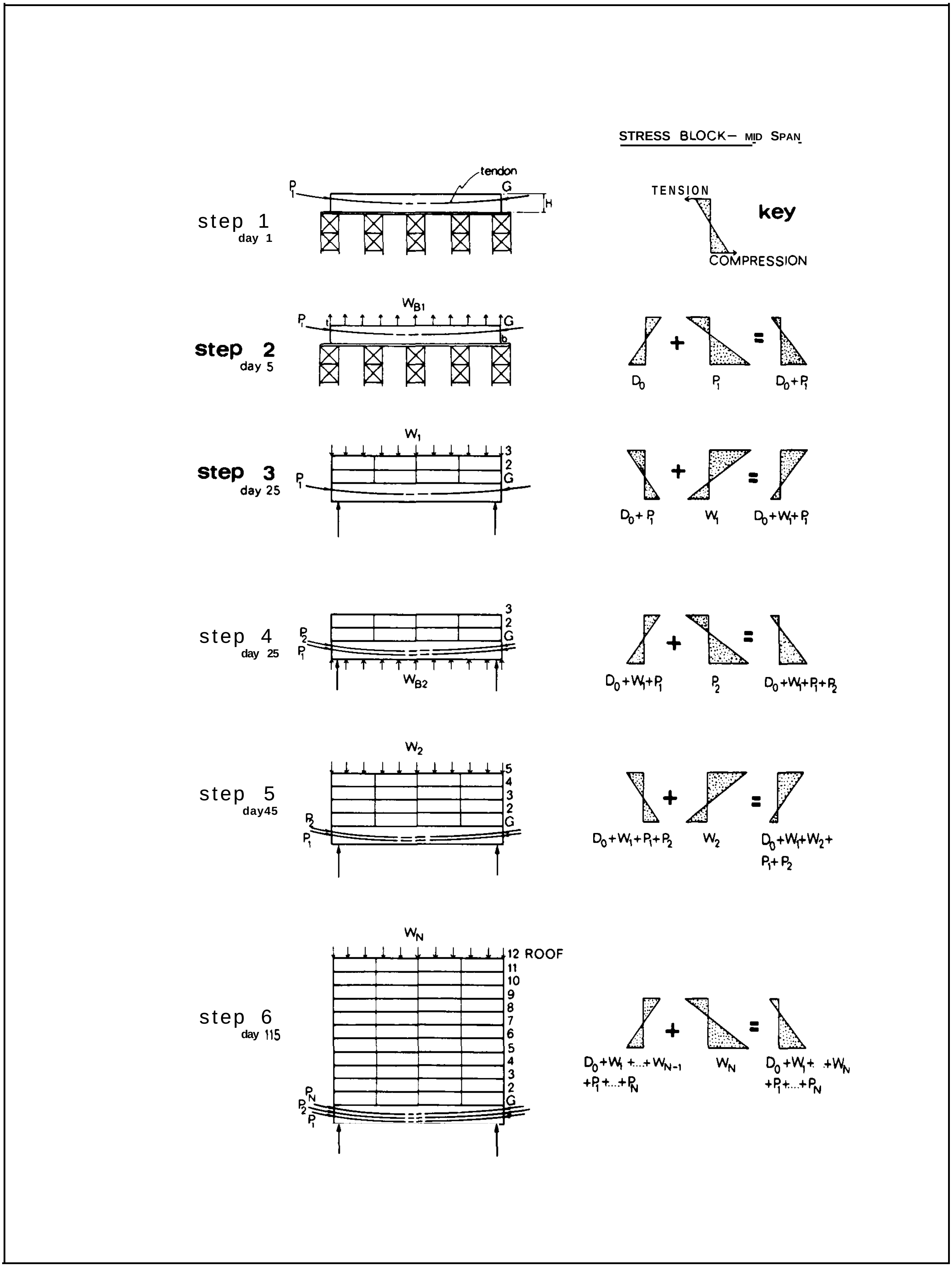


Fig. 5.30 — Principle and mechanics of stage post-tensioning showing stress conditions at corresponding construction steps

Again, the girder stresses are reversed so that the top fiber stress $3\sqrt{f'_c}$ is kept safely below the cracking value and likewise the compressive stress in the bottom fiber is controlled at a safe level. Note that the shear resistance of the girder also increases with each application of stage post-tensioning.

Step 5 (Day 45)
Application of Stage 2 loading

Construction is completed to Floor 5 and the full load from Floors 4 and 5 is applied to the girder with the resulting change in stress state similar to that described in Step 3 above.

Step 6 (Day 115)
Application of Nth Stage post-tensioning (restressing of tendons)

Stage post-tensioning and construction continues through (N-l) stages until the 12th level (roof) is reached at Day 115 after girder concreting. During this 4-month period (in which only approximately uniform normal prestress, P/A, has been applied), prestress losses have been taking place in the tendons. (Note that at this stage the tendons, still remain ungrouted.) the following volume changes occur:

1. Shrinkage of concrete.
2. Elastic shortening of concrete.
3. Creep of concrete.
4. Relaxation of prestressing steel.

These losses are represented by terms relevant to each stage $\Delta P_1, \Delta P_2 \ldots \Delta P_{N-1}$ over a given time internal (days) from T_1 to T_{115}.

However, very significantly, all the tendons can now be retensioned to their original forceduring the Nth and final stage of post-tensioning. At the level of the original forces it is found that additional elongations ΔE_1, ΔE_2 . . . E_{N-1} are attained in each tendon group, more in the earliest stages and less in the later stages.

During this Nth stage of post-tensioning, all the prestress losses in the tendons are recovered or "recaptured." (Note that at this time the tendons can be anchored and grouted.) This "gain" in prestress losses may in many cases amount to 15,000 psi, or provide a savings of over 10 percent in the prestressing steel.

From a structural viewpoint, stage post-tensioning has several important advantages:

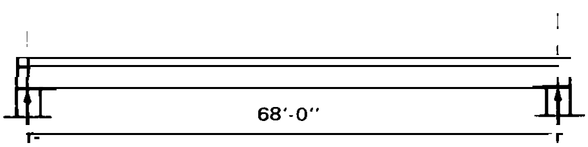
1. The depth of beams and girders can be kept to an absolute minimum affording substantial savings in the cost of materials and providing slender, attractive members.
2. The working stresses can be controlled as each successive loading is applied.
3. Deflection and camber may becontrolled.
4. Because fairly large prestress losses can be recovered, the amount of prestressing steel may be significantly reduced. (However, it is to be noted that recovery of prestress losses requires that tendons remain in the ducts in a stressed and ungrouted state for a period of four or five months. In some environments, it may be preferable or necessary to grout the tendons within a more limited time period after initial stressing or use some effective means to inhibit development of corrosion).

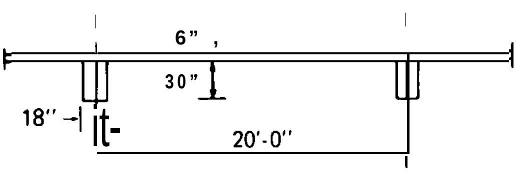
EXAMPLE 5.5.1

Cast-in-place T-Beam — Single Span

1. Given.

Design a simply supported, cast-in-place T-Beam to span 68 ft. Beams are spaced 20 ft. on centers and support a superimposed dead load of 10 psf and live load of 40 psf. Normal weight concrete (150 pcf) with compressive strength of 4000 psi at transfer (f_{ci}) is used. The design is based on the use of grouted tendons (bonded construction). This example design utilizes temporary overstress of the tendons (above 0.66 f_{pu} and less than 0.75 f,,) to compensate for prestress losses due to elastic shortening. The calculations on anchor zone bearing stresses are included only for illustrative purposes. These stresses are normally maintained within allowable levels by use of the standard commercially available bearing plate sizes.





2. Section Properties.

Assume T-beam with a slab 9 ft. 6 in. wide (per Sec. 8.10 of ACI 318-83):

A_g = 1224 in.^{2}
I_g = 140,300 in.^{4}
y_t = 25.06 in.
Z_b = = 560010.9in.^{3}in.

Z_t = 12,820 in.^{3}

3. Flexural Calculations.

Stresses due to external loads may be calculated as follows:

Dead Load	Load, klf	Moment $wl^2/8$ ft-kips	Stresses, ksi Top	Stresses, ksi Bottom
Slab (0.5 x 20 x 0.150)	1.50	867	+0.812	-1.858
Beam (1.5 x 2.5 x 0.150)	0.56	324	+0.303	-0.694
Superimposed (20x0.010)	0.20	116	+0.109	-0.249
Live Load (20 x 0.040)	0.80	462	+0.433	-0.990
Totals			+1.657	-3.791

4. Approximate Post-Tensioning Requirements.

From experience (or by a trial design) the assumed location of the prestressing steel at midspan will be taken at 3.75 in. above the bottom of the beam. Therefore, the maximum eccentricity is 25.06 − 3.75 = 21.31 in.

Calculate the required post-tensioning force with an allowable tensile stress of $6\sqrt{f'_c}$
$6\sqrt{5000}$ = 424 psi:

$$\frac{P}{A_g} + \frac{Pe}{Z_b} - 3791 = -424 \text{ psi}$$

$$\frac{P}{1224} + \frac{21.31\ P}{5600} = 3367$$

P (0.000817 + 0.003805) = 3367

P = 3367/0.004622 = 728,473 lb

Use 1-3/8 in. diameter 150k threadbars.

$P_{pu} = A_s \times 150 = 1.58 \times 150 = 237k$

P, = 0.66 x 237 kips = 156.4k

Prestress losses = 25 ksi x 1.58 = 39.5k

Approximate P_{pe} = 156.4 − 39.5 = 116.9k

Approximate number of bars required =

$$\frac{728{,}473}{116{,}900} = 6.23$$

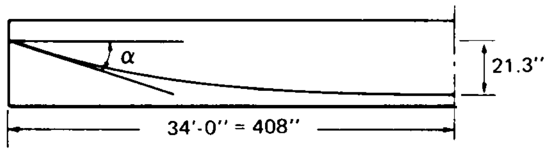
Prestress losses will be reduced due to temporary overstressing to compensate for elastic shortening loss.

Try six 1-3/8 in. diameter Grade 150 threadbars

A_{ps} = 6 x 1.58 = 9.48 sq. in.

5. Effect of Friction (two-end tensioning).

With a parabolic drape and the c.g.s. at the centroid of the section at the end of the beam.



$$a = \tan^{-1} \frac{2 \times 21.3}{408} \cong 0.104 \text{ radians}$$

$$P_o = P_x e^{(kl + \mu a)}$$

where l is the length in ft.

With k = 0.0002 and μ = 0.15,

$$P_o/P_x = e^{(0.002 \times 34 + 0.15 \times 0.104)} = e^{0.0224}$$

$$P_o/P_x = 1.0224$$

Approximate friction loss to center of span assuming initial jacking stress = 0.75 f,,:

P_o = 0.75 x 237 = 177.8k

P_x = 177.8/1.0224 = 173.9k

Friction loss = 177.8k – 173.9k = 3.9k

6. Estimate initial tendon stress.

Maximum force in tendon after anchorage: $0.66P_{pu}$ = 0.66 x 237 = 156.4k. At the time this prestressing force is effective, prestress losses due to friction and elastic shortening have taken place. No anchor seating loss need be assumed for the threadbar system in this application. In this case, the maximum tendon force at the anchorage becomes $0.66P_{pu}$ plus the elastic shortening loss. Elastic shortening losses may be estimated at 7.0k per tendon (using procedures presented in the following material with a tendon force of 156.4k).

On the above basis, the maximum force per tendon at the anchorage becomes 156.4k + 7.0k = 163.4k. The maximum effective force at mid-span becomes 156.4k – 3.9k (friction loss) = 152.5k.

Calculate actual elastic shortening loss based on tendon forces after friction losses (163.4k at the anchors, 163.4k – 3.9k = 159.5k at midspan).

Stresses induced in the concrete if all six bars were tensioned simultaneously:

$$\text{at end} = \frac{6P_o'}{A_g} = \frac{6 \times 163.40}{1224} = 0.801 \text{ ksi}$$

$$\text{at midspan} = \frac{6P_x'}{A_g} + \frac{P_x'e^2}{I_g} - \frac{M_{dl}e}{I_g}$$

$$= \frac{6 \times 159.5}{1224} + \frac{6 \times 159.5 \times 21.3'}{140{,}300} - \frac{1191(12)(21.3)}{140{,}300}$$

$$= 0.782 + 3.095 - 2.170 = 1.707 \text{ ksi}$$

Average loss on any previously stressed tendon due to stressing one additional tendon =

$$\Delta P = \frac{1}{n}\left(\frac{0.801 + 1.707}{2}\right)\frac{E_s}{E_c} A_s$$

$$= \frac{1}{6}\left(\frac{0.801 + 1.707}{2}\right)\frac{30}{3.8} (1.58)$$

ΔP = 2.61 k

The first tendon stressed will have a loss of 5 x 2.61 = 13.05k, the second tendon 4 x 2.61 = 10.44k, and the last tendon stressed will have no elastic shortening loss. It is normally accrate enough to consider the average loss for all tendons:

$$\left(\frac{n-1}{2}\right) \Delta P = \left(\frac{6-1}{2}\right) \times 2.61 = 6.53k$$

(Note: the average elastic shortening loss of 6.53K is close enough to assumed value of 7.0k).

7. Tendon forces.

On the basis of the previous calculations, the initial tendon forces (after elastic shortening and friction loss) are:

P_o = 156.4k

P_x = 156.4 – 3.90 = 152.50k

Total prestress losses = 25 ksi x 1.58 = 39.50k

- Elastic shortening = 6.53k

Prestress loss due to creep, shrinkage and steel relaxation = 32.97k

Effective prestress force per tendon at end = 156.40

– Prestress losses = 32.97

P_{pe} = 123.43

Effective prestress force per tendon
center of span = 152.50

− Prestress losses = 32.97

P_{pe} = 119.53

Total effective
Prestress force at end = 6 x 123.43 = 740.58k

Total effecftive prestress
force at midspan = 6 x 119.53 = 717.18k

8. **Check midspan stresses at critical load stages.**

(Compression = +, Tension = -)

	Stresses, ksi Top	Bottom
Dead load, beam and slab	+ 1.115	− 2.552
Post-Tensioning, initial		
P/A, = 152.50 x 6/1 224	+ 0.748	+ 0.748
Pe/Z_t = 152.50 x 6 x 21.3/12,820	− 1.520	
Pe/Z_b = 152.50 x 6 21.3/5,600		+ 3.480
1. At transfer =	+ 0.343	+ 1.676
Dead load beam, slab and permanent dead load	+ 1.224	− 2.801
Post-Tensioning, final		
P/A, = 717.18/1224 =	+ 0.586	+ 0.586
Pe/Z_t = 717.18 x 21.3/12,820	− 1.192	
Pe/Z_b = 717.18 x 21.3/5,600		+ 2.723
2. Under permanent loads	+ 0.618	+ 0.508
Live load	+ 0.433	− 0.990
3. Under full service loads	+ 1.051	− 0.472

$472/\sqrt{5000} = 6.68\sqrt{f'_c}$
Maximum allowable tensile stress = $12\sqrt{f'_c}$. So, tensile stress OK although greater than $6\sqrt{f'_c}$

Allowable compression,
initial = 0.6 x 4000 = 2400 psi

Allowable compression,
final = 0.45 x 5000 = 2250 psi

All of the stresses are less than allowable and are therefore satisfactory.

Since there are no moments at the beam ends in this case due to either applied loads or post-tensioning, the stresses at beam ends are also satisfactory (0.748 ksi uniform compression at transfer, and 0.586 ksi uniform compression after all losses). Note: Moments due to applied loads and post-tensioning do occur when single span beams are framed monolithically to columns.

9. **Flexural Capacity.**

Applied ultimate loads =

1.4 D + 1.7 L

Applied M_u = 1.4 (1307) + 1.7 (462)

= 2615 ft. kips.

Determine location of neutral axis,

$1.4\, d\, \rho_p f_{ps}/f'_c$

estimate f_{ps} = 144,000

$$= 1.4\,(32.25)\ \frac{6 \times 1.58}{114 \times 32.25} \times \frac{144{,}000}{5{,}000} = 3.35 \text{ in.}$$

Neutral axis is located in the flange.

$$f_{ps} = f_{pu}\left(1 - 0.5\,\rho_p \frac{f_{pu}}{f'_c}\right)$$

$$f_{ps} = 150\left(1 \quad 0.5\ \frac{9.48}{114 \times 32.25} \times \frac{150{,}000}{5{,}000}\right)$$

f_{ps} = 150 (0.961) = 144.15 ksi

$M_u = \phi\,[A_{ps}\, f_{ps}\, d\ (1 - 0.59\,\omega_p)]$

$$\omega_p = \frac{A_{ps}}{bd}\ \frac{f_{ps}}{f'_c} = \frac{9.48}{114 \times 32.25} \times \frac{144.15}{5}$$

ω_p = .0743

$$M_u = 0.9\left[\left(9.48 \times 144.15 \times \frac{32.25}{12}\right)\right.$$

$$\left.(1 \quad 0.59 \times .0743)\right]$$

M_u = 3,160 ft-kips > 2615 ft-kips required

Ultimate moment capacity is more than required. Amount of post-tensioning could be reduced by permitting a higher midspan tensile stress (say = $9\sqrt{f'_c}$).

10. Shear.

For convenience, the shear carried by the concrete, V_c, will be taken as specified by equation *11-10 of ACI 318-83* =

$$V_c = \left(0.6\sqrt{f'_c} + 700\ \frac{V_u d}{M_u}\right) b_w d$$

$$2\sqrt{f'_c}\ \ b_w d < V_c \leq 5\sqrt{f'_c}\ \ b_w d$$

$$2\sqrt{5000} = 141 \text{ psi},\ 5\sqrt{5000} = 354\text{psi}$$

V_u = 1.4D + 1.7L = 1.4 (2.26) 34

+ 1.7 (0.80) 34

V_u = 153.82 k at center of support

Shear Calculation Table (See below)

Only positive shear requirement of 34 psi occurs at 1/4 span. Minimum shear reinforcement will suffice.

Base shear reinforcement requirements on equation 1 *1-15 of ACI 318-83* =

$$A_v = \frac{A_{ps}}{80}\ \frac{f_{pu}}{f_y}\ \frac{s}{d}\ \sqrt{\frac{d}{b_w}}$$

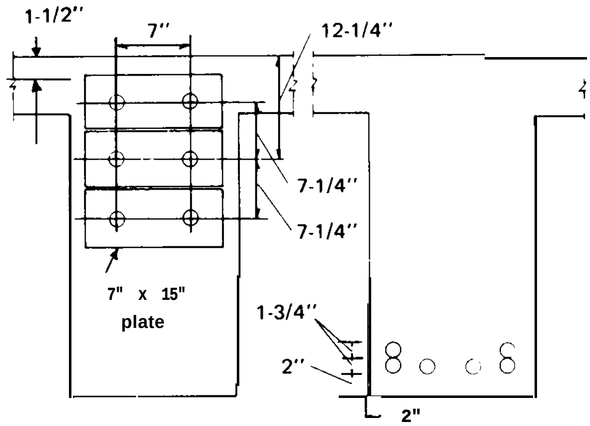
$$\text{at 1/4 Span, } A_v = \left[\frac{9.48}{80} \times \frac{150}{60} \times \frac{s}{2.24 \times 12} \times \sqrt{\frac{2.24 \times 12}{18}}\right] = 0.135\ s$$

Using No. 4 Grade 60 stirrups: a, = 0.40

0.40 = 0.135 s s = 29.6 in.

AC/ 378-83 Section *11.5.4* specifies maximum stirrup spacing of 0.75 h $\leq$24. In this case, 0.75 h = 0.75 x3.0 = 2.25 ft., so maximum spacing is 24 in. Use No. 4 Grade 60 stirrups at 24 in. centers throughout beam length.

11. Anchorage Zone Stresses.



Assume three plates 7 in. x 15 in. as shown.

Allowable bearing stresses

At service load (See Section 3.1.7)

Allowable $f_{cp} = 0.6\ f'_c \sqrt{A'_b/A_b} \leqq f'_c$

$= 0.6 \times 5.0 \sqrt{(24.50 \times 18)/(3 \times 7 \times 15)}$

= 3.55 ksi

The effective force per bar at'the anchorage under service loads is calculated in the preceding as 123.43 kips:

Bearing stress = 2 x 123.43/(7 x 15) =

2.37 ksi < 3.55 ksi OK

At transfer (f'_{ci} = 4 ksi)

allowable $f_{cp} = 0.8\ f'_{ci} \sqrt{(A'_b/A_b) - 0.2} \leqq 1.25\ f_{ci}$

$= 0.8 \times 4.0 \sqrt{(24.50 \times 18)/(3 \times 7 \times 15) - 0.2}$

= 3.51 ksi

The prestressing force per bar is calculated in the preceding is 162.93 (156.4 + 6.53) kips before elasftic shortening loss.

Bearing stress = 2 x 162.93/(7 x 15) = 3.10 ksi

Bearing stress at time of anchorage is less than 3.51 ksi, so the 7 x 15 in. bearing plates are satisfactory, both at time of anchorage and under service loads.

Shear Calculation Table

Location	V_u kips	M_u ft.kips	d ft.	vc ksi	v_u ksi	$v_u - v_c$ ksi
h/2 from face of support	147.04	226	1.062	.354	.334	-0.020
1/4 Span	76.91	1961	2.240	.141	.175	+0. 034
1/3 Span	52.30	2324	2.480	.141	.115	-0.036
Midspan	0	2615	2.690	.141	0	-0.141

12. Deflection.

Live load deflection = $\frac{5}{384} \times \frac{wl^4}{EI}$

$$= \frac{5 \times 20 \times 40 \times 68 \times (68 \times 12)^3}{384 \times 4.3 \times 10^6 \times 140{,}300}$$

$$= 0.64 \text{ in } = \frac{68 \times 12}{.64} = \frac{1}{1275} \times \text{Span}$$

Permissible live load deflection = 1/360 x Span

= 1/360 x 68 x 12 = 2.27 in.

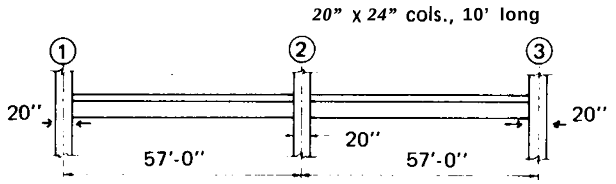
So the beam as designed is more satisfactory for live load deflection. Bilinear deflection calculations unnecessary due to small deflection calculated above, and small excess of design tensile stress (6.68 $\sqrt{f'_c}$) above 6 $\sqrt{f'_c}$.

EXAMPLE 5.5.2

Two-Span Cast-in-place T-Beam

1. Given.

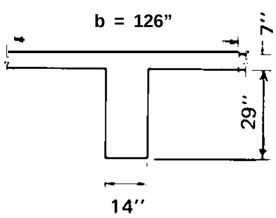
Design a two-span cast-in-placeT-Beam parking garage floor of normal weight concrete. Use $f'_c = 5000$ psi, $f'_{ci} = 4000$ psi, and $f_{pu} = 270{,}000$ psi (1/2 inch diameter strand post-tensioning tendons). Beams are spaced 26.9 ft. on centers. The design is based on use of unbonded tendons.



Using a 14 in. stem thickness, the clear slab span is 26.9 − 1.17 = 25.73 ft.

$$\text{Required slab thickness} = \frac{\text{Span}}{45} = \frac{25.73}{45} \times 12$$

Required slab thickness = 6.86 in. <u>Use 7" slab.</u> (Note: Slab thickness would have to be increased in locations where de-icer chemicals are used and minimum top cover = 2 inches.) Use 3 ft. section depth (arbitrarily).



Effective slab width in accordance with Section 8.70 of *ACI 318-83* = stem width + 8 x slab thickness each side = 14 in. + 8 x 7 x 2 = 126 in.

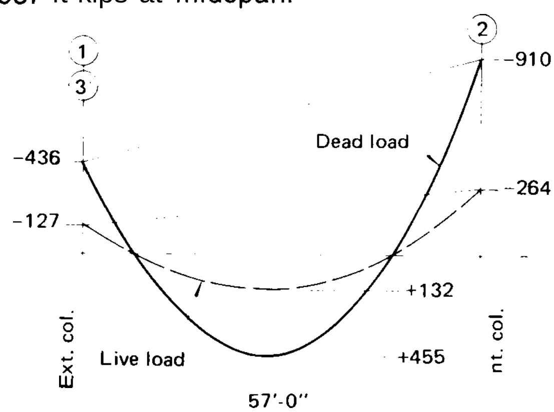
2. Section properties. (See table below)

3. Dead load and live load moments

Dead load = 2777 lb/ft (including full slab)

Live load = 50 psf reduced to 30 psf on columns and beams (30 x 26.9 = 807 lb./ft.)

From the above information, the dead load moments were calculated (by moment distribution) to be -436 ft-kips at the exterior columns, -910 ft.-kips at the interior column and +455 ft-kips at midspan. The corresponding live load moments (not considering alternate panel loading) are -127, -264 and +132 ft-kips, respectively. Thus, the sum of the dead and live load moments are -563 ft-kips at the exterior columns, -1174 ft-kips at the interior column, and +587 ft-kips at midspan.



Section Properties

Section	A	y	Ay	Ay²	I_o
Top slab 126 x 7	882	- 3.5	- 3,087	10,804	3,601
Stem 29x 14	406	-21.5	- 8,729	187,674	28,453
	1,288		-11,816	198,478	32,054

$$y_t = \frac{-11{,}816}{1{,}288} = -9.17 \text{ in.}$$

$y_b = 36 - 9.17 = 26.83$ in.

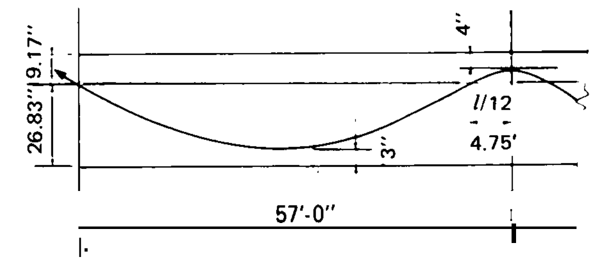
$Z_t = 122{,}179/9.17 = 13{,}324$ in.³

$Z_b = 122{,}179/26.83 = 4{,}554$ in.³

32,054
230,532
$-1288 \times 9.17^2 = -108{,}353$
$I = 122{,}179$ in.⁴

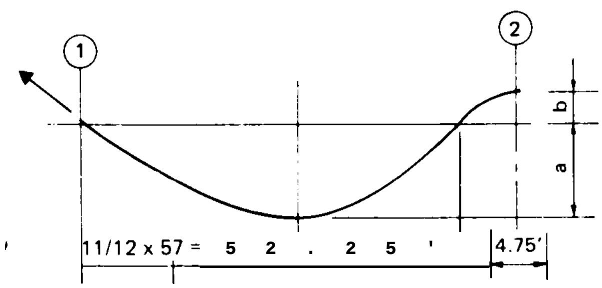
4. Post-tensioning force selection

The effects of post-tensioning are treated separately from the effects of loads and are dependent on the magnitude and position of the post-tensioning force and on frame action. Assume two-end stressing. A tendon profile is selected:



a) Secondary Moments:

Secondary moments induced by post-tensioning are accounted for by considering the effects of restrained rotations. For simplicity, a post-tensioning force of 1 kip is used.



a = (26.83 − 3) x 1 = 23.83 in.-kips
b = (9.17 − 4) x 1 = 5.17 in.-kips

Using the conjugate beam method:

Rotation @ 1:

$$EI\theta_1 \left[\frac{2}{3} \times 23.83 \times 52.25 \times \left(\frac{52.25}{2} + 4.75\right) - \frac{2}{3} \times 4.75 \times 5.17 \times \frac{3}{8} \times 4.75\right] \frac{1}{57}$$

$$= \mathbf{-458.59}$$

Rotation @ 2:

$$EI\theta_2 = -\left[\frac{2}{3} \times \frac{(52.25)^2}{2} \times 23.83 - \frac{2}{3} \times 4.75 \times 5.17 \times \left(52.25 + \frac{5}{8} \times 4.75\right)\right] \frac{1}{57}$$

$$= \mathbf{-378.89}$$

Fixed end moments (per kip of force)

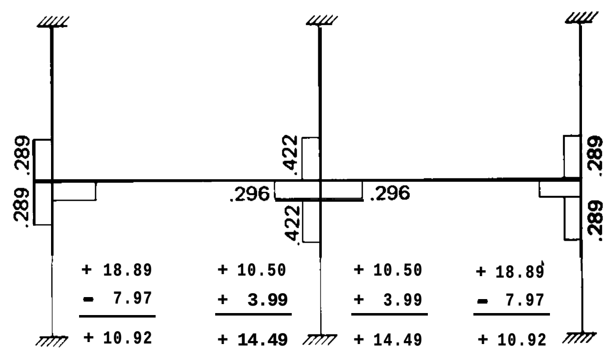
$$\text{F.E.M. @ 1} = \frac{\theta_2 - 2\theta_1}{l/2} = \frac{538.29}{28.5}$$

$= 18.89$ in-kips

$$\text{F.E.M. @ 2} = \frac{\theta_1 \quad 2\theta_2}{l/2} = \frac{299.19}{28.5}$$

$= 10.50$ in-kips

Distribute fixed-end secondary moments:



The resulting secondary moments are + 10.92 in. kips at exterior columns, + 14.49 in. kips at the interior column, and + 12.71 in. kips at midspan. These secondary moments may be considered as effective changes in the tendon profile in accordance with the equation:

$M_s = P\,(\Delta e)$

so the resulting Ae values in this example are 10.92 in. at columns 1 and 3, 14.49 in. at column 2 and 12.71 in. at midspan.

b) Required post-tensioning force:

Allow a tensile stress of $6\sqrt{f'_c} = 424$ psi

This permissible tensile stress can be expressed in terms of moments at midspan and at columns as follows:

$$\text{Span } \Delta M = 424 \times \frac{4554}{12{,}000} = 160.91 \text{ ft.-kips}$$

$$\text{Support } \Delta M = 424 \times \frac{13{,}324}{12\ 000} = \mathbf{470.78\ ft.\text{-}kips}$$

The required post-tensioning force at columns can be calculated as follows:

$$\frac{P}{A_g} + \frac{Pe}{Z_t} + \frac{P\Delta e}{Z_t} = f_t - (\text{f allowable})$$

Multiplying both sides by Z_t and solving for P provides:

$$P = \frac{M_t l - \Delta M_{supp}}{\frac{Z_t}{A_g} + e + Ae}$$

Based on the above equation, the required post-tensioning force at exterior columns is:

$$P = \frac{[-563 - (-470.78)]\,12}{\frac{13324}{1.288} + 0 + 10.92} = 52.05 \text{ kips}$$

The required post-tensioning force at interior column is

$$P = \frac{[-1{,}174 - (-470.78)]\,12}{\frac{13324}{1.288} + 5.17 + 14.49} = 281.29 \text{ kips}$$

The required post-tensioning force at midspan is:

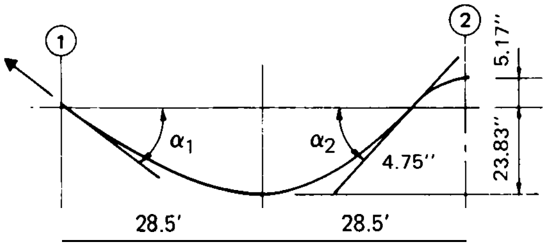
$$P = \frac{M_t l - \Delta M_{span}}{\frac{Z_b}{A_g} + e - Ae}$$

$$P = \frac{(-587 - 160.91)\,12}{\frac{4554}{1.288} + 23.83 - 12.71} = 348.78 \text{ kips}$$

Thus the necessary post-tensioning force at midspan governs the required amount of post-tensioning steel. Note that a significant decrease in the post-tensioning force is possible by allowing a higher service load tensile stress in the positive moment area.

5. Friction Losses.

Calculate friction losses using equation 18-1 (ACI 318-83), $P_s = P_x e^{(kl + \mu\alpha)}$. (Stressing sequentially at support 1 and 3).



$$\tan \alpha_1 = \frac{2 \times 23.83}{28.5 \times 12} = 0.139$$

$$\alpha_1 = 7.92 \text{ degrees} = \frac{7.92}{57.29} \text{ radians} = 0.138$$

$$\tan \alpha_2 = \frac{2 \times 23.83}{23.75 \times 12} = 0.1672$$

$$\alpha_2 = 9^\circ\, 30' = \frac{9.50}{57.29} = 0.166 \text{ radians}$$

For unbonded tendons values of μ and K of 0.08 and 0.0014, respectively, will be assumed. Using these values, friction losses may be calculated as follows:

For midspan =

$$\mu\alpha = 0.08 \times 0.138 = 0.0110$$
$$Kl = 0.0014 \times 28.5 = 0.0399$$
$$Kl + \mu\alpha = 0.402$$

$$P_s = P_x e^{(Kl + \mu\alpha)} = P_x e^{0.0509} = 1.04 P_x$$
$$P_x = P_s / 1.04$$

For interior support =

$$\mu\alpha = 0.08\ (0.138 + 2 \times 0.166) = 0.0376$$
$$Kl = 0.0014 \times 57 = 0.0798$$
$$Kl + \mu\alpha = 0.1174$$

$$P_s = P_x e^{0.1174} = 1.12 P_x$$

$$P_x = P_s / 1.12$$

6. Tendon Stress Diagram.

Prestress losses = 35,000 psi. Estimate elastic shortening loss at 7,000 psi and temporarily stress tendons 7,000 psi plus friction loss to midspan in excess of 0.7 f_{pu} to compensate for elastic shortening loss and friction loss at midspan.

Initial tendon stress =

0.7 x 270,000 x 1.04 + 7000 = 203,560 psi

203,560/270,000 x 100 = 75.4 percent of f_{pu}

Initial effective tendon stress at midspan:

= 0.7 x 270,000 =	189,000 psi
− shrinkage, creep & steel relaxation	− 28,000
Effective tendon stress at midspan=	161,000 psi

initial effective tendon stress at interior columns:

= 0.7 x 270,000 x 1.04/1.12 =	175,500 psi
− creep, shrinkage & steel relaxation =	28,000
Effective tendon stress at interior column =	147,500 psi

Initial effective tendon stress at exterior columns:

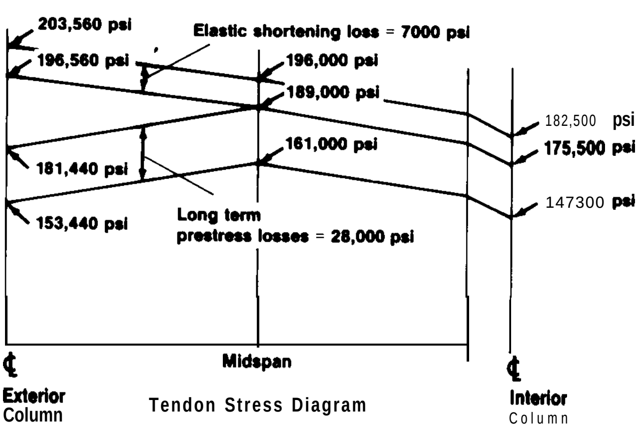
Friction loss to midspan = 7,560 psi

189,000 − 7560 = 181,440 psi

181,440/270 = 67.2% OK < 70%

	181,440
creep, shrinkage & steel relaxation	− 28,000
Effective tendon stress at exterior column =	153,400 psi

Note that anchor seating losses were considered to reduce the tendon stress at the anchorage to 70/1.04 = 67.2 percent so that tendon stress between the anchorage and midspan does not exceed 70 percent. The final force diagram in the tendon is presented in the following sketch.



Tendon Stress Diagram

7. Number of 1/2" ϕ 270 k strand required.

A_{ps} = 0.153 sq. in/strand

At midspan $= \dfrac{348,780}{0.153 \times 161,000} = 14.15$

Use 15 1/2" ϕ 270 k strands

Midspan effective force =

15 x .153x 161,000 = 369,495 lb.

Interior column effective force =

15 x .153 x 147,500 = 338,512 lb.

Exterior column effective force =

15 x .153 x 153,440 = 352,145 lb.

Average force = 353,384 lb.

8. Stresses at service loads.

Allowable stresses = 424 psi tension, 0.45 x 5000 = 2250 psi compression.

a) At exterior column:

P = 352,145 lb.

$$f_t = +\frac{P}{A} + \frac{P(e+\Delta e)}{Z_t} - \frac{M_t l}{Z_t}$$

$$f_t = +\frac{352.145}{1,288} + \frac{352.145\,(0 + 10.92)}{13,324} - \frac{563,000 \times 12}{13,324}$$

$$f_t = +273 + 288 - 507$$

= + 54 psi compression OK

$$f_b = +\frac{352,145}{1,288} - \frac{352,145 \times 10.92}{4,554} + \frac{563,000 \times 12}{4,554}$$

$$f_b = +273 - 844 + 1,483$$

= + 912 psi compression OK

b) At midspan =

$$f_t = +\frac{369,495}{1,288} - \frac{369,495\,(23.83 - 12.71)}{13,324} + \frac{587,000 \times 12}{13,324}$$

$$f_t = +287 - 308 + 529$$

= + 508 psi compression OK

$$f_b = +\frac{369,495}{1,288} + \frac{369,495\,(23.83-12.71)}{4,554} - \frac{587,000 \times 12}{4,554}$$

f_b = + 287 + 902 − 1,547 = -358 psi tension OK *c) At interior column:*

P = 338,512

$$f_t = +\frac{338,512}{1,288} \quad \frac{338,512\ (5.17 + 14.49)}{13,324} +$$

$$+\frac{1,174,000 \times 12}{13,324}$$

f_t = + 263 + 499 1,057 = -295 psi tension OK

$$f_b = +\frac{338,512}{1,288} - \frac{338,512\ (5.17 + 14.49)}{4,554}$$

$$+\frac{1,174,000 \times 12}{4,554}$$

f_b = + 263 − 1,461 + 3,094

= 1,896 psi compression OK

All service load stresses are well within the allowable stresses.

9. Stresses under initial post-tensioning force, considering only dead load present.

Allowable stresses = 0.6 x 4000 =

2400 psi compression

$3\sqrt{4000}$ = 190 psi tension

a) At exterior support:

P = 181,440 x 15 x .153 = 416,405 lb.

$$f_t = +\frac{416,405}{1,288} + \frac{416,405\ (0 + 10.92)}{13,324}$$

$$-\frac{436,000 \times 12}{13,324}$$

f_t = + 323 + 341 − 393

= + 271 psi compression OK

$$f_b = +\frac{416,405}{1,288} - \frac{416,405\ (10.92)}{4,554}$$

$$+\frac{436,000 \times 12}{4,554}$$

f_b = + 323 − 998 + 1,149

= + 474 psi compression OK

b) At midspan:

P = 189,000 x 15 x 0.153 = 433,755 lb.

$$f_t = +\frac{433,755}{1,288} - \frac{433,755\ (23.83 - 12.71)}{13,324}$$

$$+\frac{455,000 \times 12}{13,324}$$

f_t = + 337 − 368 + 410

= + 379 psi compression OK

$$f_b = +337 + \frac{433,755\ (23.83 \quad 12.71)}{4,554}$$

$$\frac{455.000 \times 12}{4,554}$$

f_b = + 333 + 1,059 − 1,199

= + 193 psi compression OK

c) At interior column:

P = 15 x 175,500 x 0.153 = 402,772 lb

$$f_t = +\frac{402,772}{1,288} + \frac{402,772\ (5.17 + 14.49)}{13,324}$$

$$\frac{910,000 \times 12}{13,324}$$

f_t = + 312 + 594 − 819

= + 87 psi compression OK

$$f_b = +312 - \frac{402,772\ (5.17 + 14.49)}{4,554}$$

$$+\frac{910,000 \times 12}{4,554}$$

f_b = + 312 − 1,739 + 2,398

= + 971 psi compression OK

All stresses at the time the tendons are stressed are also well within the allowable stresses.

10. Flexural Capacity.

Design load = 1.4(2,777) + 1.7(807)
= 5,260 lb./ft.

Dead + live = 2,777 + 807 = 3,584 lb./ft.

a) Required moment capacity (including secondary moments due to post-tensioning).

At exterior columns:

$$M_u = \frac{5,260}{3.584}\ (563) = -826 \text{ ft. kips}$$

+ Secondary post-tensioning moment

$$= +\frac{10.92}{12} \times 353.38 = +321 \text{ ft. kips}$$

Total moment capacity required to be provided by tendon =

$-826 + 321 = -505$ ft. kips

At midspan:

$$M_u = \frac{5{,}260}{3.584}(587) = +861 \text{ ft. kips}$$

secondary moment =

$$\frac{12.71}{12} \times 353.38 = +374 \text{ ft. kips}$$

$+1235$ ft. kips required

At interior column:

$$M_u = \frac{5{,}260}{3.584}(1174) = -1723 \text{ft. kips}$$

secondary moment

$$\frac{14.49}{12} \times 353.38 = +427$$

-1296 ft. kips required

b) Flexural Capacity Provided

At exterior columns:

Bonded reinforcement in top slab per Section 18.9.2 of ACI 318-83: A, = 0.004A

A, = 0.004 x 126 x 7 = 3.53 sq. in.

8 No. 6 bars = 3.52 sq. in.

Check amount of bonded reinforcement required for strength:

$$\rho_p = \frac{15 \times .153}{14 \times 26.83} = 0.0061$$

$$\text{Eq. (18-4) } f_{ps} = f_{se} + 10{,}000 + \frac{f'_c}{100\,\rho_p}$$

$$f_{ps} = 153{,}440 + 10{,}000 + \frac{5000}{100 \times 0061}$$

$$f_{ps} = 163{,}440 + 8197 = 171{,}637$$

M_u provided by tendons:

$$\phi\, M_u = \phi \left[A_{ps}\, f_{ps} \quad (d - \frac{a}{2})\right]$$

$$a = \frac{A_{ps}\, f_{ps}}{.85\, f'_c\, b} = \frac{0.153\text{x1 } 5\text{~}171{,}637}{.85\text{x}5000\text{x}14} = 6.62 \text{ in.}$$

$$\phi M_u = 0.90 \left[15\text{x.1 } 53\text{~}171{,}637\ (26.83 - \frac{6.62}{2})\right]$$

$\phi M_u = 8338$ in. kips/12 = 695 ft. kips

OK > 505 ft. kips required.

Since tendons alone provide more than required moment capacity, provide bonded reinforcement for rectangular section only.

A, = 0.004 x 14 x 18 = 1.008 sq. in.

Use 5 No. 4 Grade 60 bars A, = 1.00 sq. in.

$$d = \frac{.153\text{x}15\text{x}171{,}637\text{x}26.83 + 1.0\text{x}60{,}000\text{x}33}{.153\text{x}15\text{x}171{,}637 = 1.0\text{x}60{,}000}$$

$$d = \frac{10{,}571 + 1980}{394 + 60} = \frac{12{,}551}{454} = 27.65 \text{ in.}$$

a = 454/.85x5x14 = 7.56 in.

$$M_u = 0.9 \left[454\ (27.65 \quad \frac{7.56}{2})\right]$$

M_u = 9753/12 = 813 ft. kips

OK > 505 ft. kips

at midspan:

A, = 0.004 x 26.83 x 14 = 1.50 sq. in.

Use 4 No. 6 bars for detailing = 1.76 sq. in.

$$\rho_p = \frac{15\text{x}.153}{126\text{x}33} = 0.000552$$

$$f_{ps} = 171{,}000 + \frac{5000}{0.0552}$$

$= 171{,}000 + 90{,}579 = 261{,}579$

max $f_{ps} = f_{se} + 60{,}000 = 161{,}000 + 60{,}000$
$= 221$ ksi

$$d = \frac{2.295\text{x}221{,}000\text{x}33 + 1.76\text{x}60{,}000\text{x}32}{2.295\text{x}221{,}000 + 1.76\text{x}60{,}000}$$

$$d = \frac{10{,}571 + 1980}{507 + 106} = 32.82 \text{ in.}$$

$$a = \frac{613}{.85\text{x}5\text{x}126} = 1.14 \text{ in} \qquad \frac{a}{2} = 0.57 \text{ in.}$$

$$\phi M_u = 0.90\ [613\ (32.82 - 0.57)]$$

ϕM_u = 17,792/12 = 1483 ft. kips

OK 1498 ft. kips > 1235 ft. kips required *at interior columns:*

Use 8 No. 6 Grade 60 bars as per exterior column calculation

A, = 3.52 sq. in.

$$\rho_p = \frac{15x.153}{14x32} = 0.0051$$

$$f_{ps} = 147{,}500 + 10{,}000 + \frac{5000}{.51} = 167{,}300 \text{ psi}$$

$$\rho = \frac{3.52}{14x33} = .0076$$

$$\omega = .0076x60/5 = 0.091$$

$$\omega_p = .0051 \times 167{,}300/5000 = .171$$

$$\omega + \omega_p = 0.091 + 0.171 = 0.262 < 0.30$$

$$d = \frac{2.30x167{,}300x32+3.52x60{,}000x33}{2.30x1\ 67{,}3000+352x60{,}000}$$

$$d = \frac{12{,}313+6969}{385+211} = \frac{19{,}282}{596} = 32.35 \text{ in.}$$

$$a = 596/.85x5x14 = 10.02 \text{ in.}$$

$$\phi M_u = 0.90 \quad 596 \left[(32.35 - 10.02/2)\right] /12$$

$$\phi M_u = 1222 \text{ ft. kips} < 1296 \text{ ft. kips required}$$

$$\frac{1222}{1296} = 94.3\%$$

Add 2 No. 6 Grade 60 bar for a total of 10 No. 6 bars over interior column to meet moment requirement

$$\rho = \frac{10x.44}{14x33} = .0095$$

$$\omega = .0095x60/5 = .114$$

$$\omega + \omega_p = .114 + .171 = .285 < .30 \text{ OK}$$

Since $\omega_p + \omega > 0.24 \times .80 = .192$, *Section 18.10.4.3 of AC/* 378-83 does not permit redistribution of moments.

$$d = \frac{12{,}313 + 4.40 \times 60 \times 33}{385 + 4.40 \times 60}$$

$$d = \frac{12{,}313 + 8712}{385 + 264} = 32.85 \text{ in.}$$

$$a = \frac{649}{.85 \times 5 \times 14} = 10.91 \text{ in.}$$

$$\phi M_u = 0.90\left[649\,(32.85 - \frac{10.91}{2})\right] /12$$

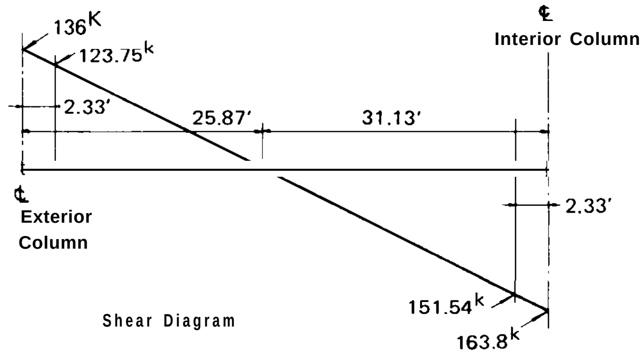
$$\phi M_u = 1336 \text{ ft/kips} > 1296 \text{ft. kips required OK}$$

Since bonded reinforcement is required for strength, it must be detailed for laps between positive and negative moment bars in accordance with Chapter 12 of ACI 318-83.

11. Shear

V_u at center of interior column = 163.8 k

V_u at center of exterior column =
149.9 − 13.9 = 136 k



$$\text{Point of zero shear} = \frac{136}{136 + 163.8} \times 57 = 25.87'$$

Design shear reinforcement at columns based on shear at h/2 from the face of columns *(ACI-318-83 Section 11.1.2.2).*

10" + 36/2 = 28"
28/12 = 2.33 ft. from center of columns

At h/2 from face of exterior columns:

$$\text{Shear} = \frac{23.54}{25.87} \times 136^k = 123.75 \text{ k}$$

At h/2 from face of interior column =

$$\text{Shear} = \frac{31.13 - 2.33}{31.13} \times 163.8 = 151.54 \text{ k}$$

Assume shear carried by the concrete is calculated by equation *11-10 of ACI 378-83.*

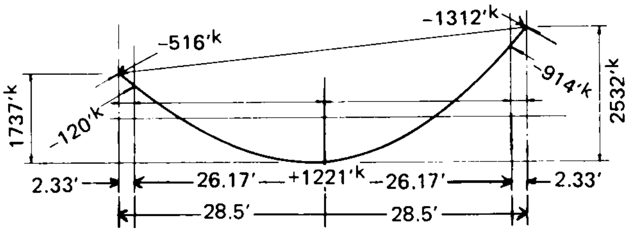
$$V_c = \left(0.6\sqrt{f'_c} + 700\frac{V_u d}{M_u}\right) b_w d$$

$$2\sqrt{f'_c}\, b_w d < V_c < 5\sqrt{f'_c}\, b_w d$$

$$2\sqrt{5{,}000} = 141 \text{ psi } 5\sqrt{5{,}000} = 353 \text{ psi}$$

Check stirrup requirement 2.33 from ℄ of exterior column

Approximate Ultimate Moment Diagram (Note midspan moment is assumed to be the maximum moment)



$$Mu = 1221 - \left(\frac{26.17}{28.5}\right)^2 \times 1{,}737 = -120'^k$$

$$d = 3.00 + \left[\left(\frac{26.17}{28.5}\right)^2 \times 23.831\right.$$

$= 23.02" = 1.92'$

$$v_c = \left[0.6\sqrt{5{,}000} + 700\left(\frac{123.75 \times 1.92}{120}\right)\right]$$

$v_c = 42 + 700\,(\max) = 742$ psi. Usemaximum value = 353 psi

$$v_u = \frac{123{,}750}{0.85 \times 14 \times 0.80 \times 36} = 361 \text{ psi}$$

Use No. 4 Grade 60 stirrups:

$$A, = 0.40 \text{ in.}^2 = \frac{(vu - vc)\, b_w s}{f_y}$$

$$0.40 = \frac{(361 - 353)\ 14 \times s}{60{,}000}$$

$$s = \frac{60{,}000 \times 0.40}{8 \times 14} = 214 \text{ in.}$$

In accordance with AC/ 318-83 *Section 11.5.4.1,* use No. 4 Grade 60 stirrups @ 24 inch centers in end of beams adjacent to exterior columns.

Check shear at 1/4 span

$$V_u = 136 \quad \frac{14.24}{25.87} \times 136 = 61.09 \text{ k}$$

$$M_u = 1221 \quad (.25 \times 1{,}737)) = 787'\text{k}$$

$$d = 9.17 + \left[23.83 - \left(\frac{14.25}{28.50}\right)^2 \times 23.83\right]$$

$= 27.04" = 2.25'$

$$v_c = \left[42 + 700\left(\frac{61.09 \times 2.25}{787}\right)\right] = 164 \text{ psi}$$

$$v_u = \frac{61{,}900}{0.85 \times 14 \times 0.8 \times 36} = 181 \text{ psi}$$

$$A, = 0.40 \text{ in.}^2 = \frac{(181 - 164)\ 14\, s}{60{,}000}$$

$$s = \frac{60{,}000 \times 0.40}{17 \times 14} = 101 \text{ in.}$$

So only minimum stirrups are required at 1/4 span from column. Use No. 4 Grade 60 stirrups at 24 inch centers.

Check requirement at 2.33 ft. from center of interior column.

$$V_u = 163.8 - \frac{2.33}{31.13} \times 163.8 = 151.54^k$$

$$M_u = 1221 - \left(\frac{26.17}{28.50}\right)^2 \times 2533 = -914$$

$$d = 26.83 + [5.17 - \left(\frac{2.33}{4.75}\right)^2 \times 5.171$$

$= 30.76" = 2.56'$

$$v_c = \left[42 + 700\left(\frac{151{,}540 \times 2.56}{914}\right)\right] = 339 \text{psi}$$

$$v^c = \frac{151{,}540}{0.85 \times 14 \times 0.80 \times 36} = 442 \text{ psi}$$

$$A_v = 0.40 = \frac{(442 - 339) \times 14 \times s}{60{,}000}$$

$$s = \frac{24{,}000}{103 \times 14} = 16.64 \text{ in.}$$

Use No. 4 Grade 60 stirrups at 15 in. centers in 1/3 of beam span either side of interior column.

12. Deflection.

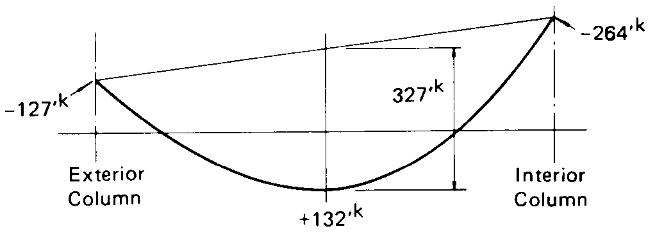
$I = 122{,}179 \text{ in.}^4$

$E = 150^{1.5} \times 33 \times \sqrt{5{,}000} = 4.29 \times 10^6$ psi

Liveload =30 psf x 26.9 = 807 plf

Permissible liveload deflection = 1/360 x span

Permissible liveload deflection = 1/360 x57 x 12 = 1.9 in.



Moment Diagram

Simple Beam Deflection at midspan:

$$\Delta = \frac{5wL^4}{384\ EI}$$

$$= \frac{5 \times 807 \times 57 \times (57 \times 12)^3}{384 \times 4.29 \times 122{,}179}$$

$$A = +0.37 \text{ in.}$$

Negative Moment Deflection at midspan:

$$A = -(M_1 l^2 + M_2 l^2)\frac{1.728 \times 10^6}{16\ EI}$$

$$A = -(127x57^2 + 264x57^2)\ \frac{1.728}{6x4.29x122{,}179}$$

$$\Delta = \frac{(412{,}623 + 857{,}736)\ 1.728}{8{,}386{,}366}$$

$$A = -0.26 \text{ in.}$$

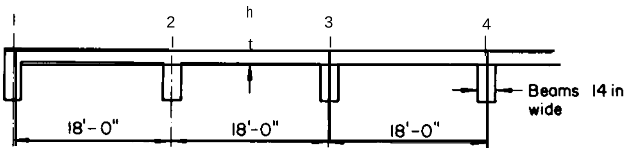
Net liveload deflection at midspan is approximately:

$$+0.37 \text{ in.} - 0.26 \text{ in.} = +0.11 \text{ in.}$$

Liveload deflection is much less than the 1.9 inches permissible. A shallower beam would be satisfactory from the standpoint of liveload deflection.

Note: For illustrative purposes, this design example includes many calculations that are not necessary in routine designs. However, a complete design investigation would require consideration of another loading case with design dead load on both spans and design live load on only one span.

5.5.3 One-Way Slab Parking Structure



Assume f'_c = 4000 psi

Design for slab dead load plus 50 psf live load.

1. Determine thickness for 50 psf live load.

   h, Parking deck @ L/48 = 18 x 12/48 =4½" For durability use, 1" additional cover = 5%".

2. Determine loads.

Dead 150 x 5.5/12	= 69 x 1.4 = 96 psf
Live	= 50 x 1.7 = 85 psf
Total	= 119 psf, service
	= 181 psf ultimate

3. Estimate balanced load. (Arbitrarily balance about 65 percent of the dead load.)

   w_{bal} = 0.65 dead load = 45 psf

4. Determine frame properties, EI is constant, no column stiffness.

   Thus:
   Say stiffness k interior = 1, k endspan = 3/4

5. Calculate moments due to:

   a) Dead loads.

   end = 0.69 (324)/12 = -1.86 ft. kips
   interior = 0.69 (324)/8 = -2.79 ft. kips

	2		3		4	
Dist. Factors	0.43	0.57	0.5	0.5	0.5	0.5
F.E.M.	-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. kips

b) 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):

2	3	4
+ 1.56 ft. kips	+ 1.12 ft. kips	+ 1.21 ft. kips

c) Live load on alternate spans and adjacent spans ("skipped" live load. FEM = 0.05 (324)/12 = – 1.35 ft. kips. Maximum negative moment at column 2: loads on first two spans.

1	2		3		4	
	0.43	0.57	0.50	0.50	0.50	0.50
-1.35	-1.35	-1.35	-1.35	0	0	0
+1.35	-0.67					
	-2.02	-1.35	-1.35	0	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		

Maximum positive moment in span 1: loads on first and third spans.

1	2		3		4	
	-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
	+0.15	-1.19	-0.45	+0.45	+0.17	-0.16
		+0.22	+0.10	-0.08	-0.22	
	+0.09	-0.13				
	-0.91	-0.91				

Positive moment at middle of first span = $wl^2/8$ = 0.91/2 = 2.02 – 0.45 = 1.57 ft. kips

Maximum negative moment at column 3: loads on second and third spans

1	2		3		4	
	0	-1.35	-1.35	-1.35	-1.35	0
	-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		

Maximum positive moment at middle of span 2; loads on spans 2 and 4.

0	-1.35	-1.35	0	0	-1.35	-1.35	0
-0.58	+0.77	+0.67	-0.68	-0.68	+0.67	+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	+0.03	-0.04				
-0.80	-0.80	-0.68	-0.68				

Maximum positive moment in span 2 =
= $wl^2/8$ – Average FEM
= 0.05x324/8 – 0.74 = 1.28 ft. kips

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".(')*

* Reference presented at the end of the example

6. Calculate post-tenstoning force required to balance assumed loads.

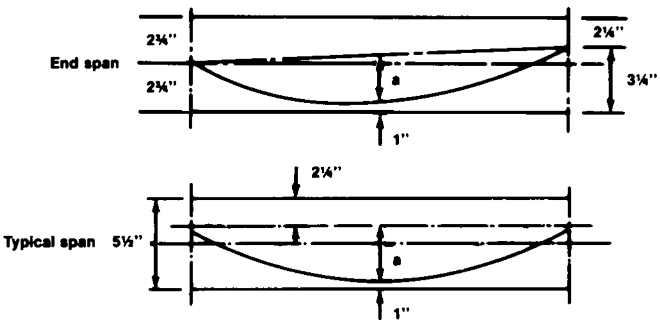
   a) Determine sag

   Assume cover for 2 hour fire rating; i.e.: 3/4" typical bottom cover, but 2" typical top cover in consideration of traffic and exposure to weather and deicing chemicals.

   Then @ endspan

   $$\text{sag } a = \frac{2\% + 3\%}{2} - 1 = 2''$$

   Typ-Span a = 5% — 2% — 1 = 2.25"



   b) Force $= \dfrac{wL^2}{8\,a}$

   End span F = 0.045 (324) 12/(8 x 2.0)

   $= 10.94^k$/ft.

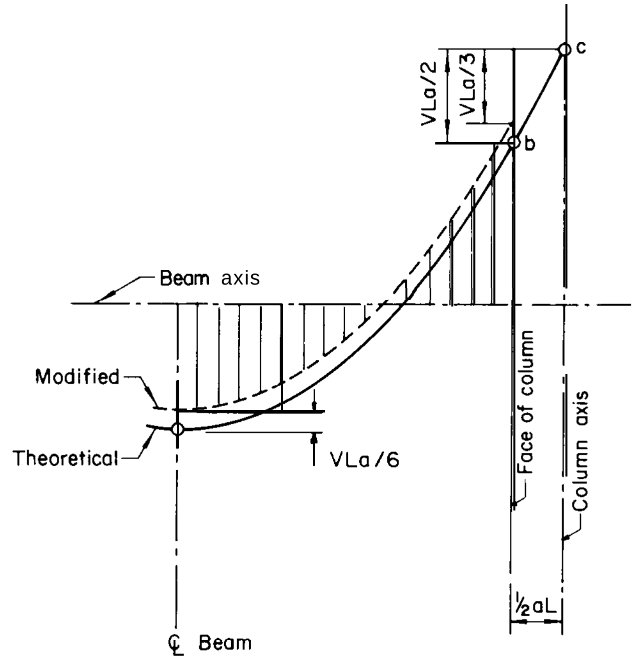
   F/A = 10.94/(5.5 x 12) = 166 psi

   Int. span F = 0.045 (324) 1.5/2.25 $= 9.72^k$/ft.

   F/A = 147 psi

7. Combine moments and adjust to critical section at face of support.

The PCA publication 'Continuity in Concrete Building Frames" [1] Suggests modification of design moments to the face of columns or supports in accordance with the following diagram:



The VLa/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 correction VLa/3 may also be expressed as Vc/3, where c is the support width.

Min V @ support = 0.074 x 18/2 = 0.666 kips
Section Modulus S = $t^2/6$ = 5.04 in^2/ft.

Loading	Magnitude psf	Moments (ft. kips at support) 2	3	4
Dead Load	0.69	-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 at Support (kips)		0.67		0.67
Vc/3 0.67 x $\frac{14}{12}$ x $\frac{1}{3}$		+ .26		+ .26
Face Moment		-2.44		-2.10
M/S		+.484		+.417
P/A		-.166		-.147
Stress at top of slab-ksi		+.318		+.270

Maximum net tensile stress = 0.318 ksi = 5.03 $\sqrt{f'_c}$. Design could be recycled with slightly lower balanced load. However, this will be considered satisfactory for illustrative purposes. Use post-tensioning force of 10.94 k/ft. in the end spans and 9.72 k/ft. in the typical interior spans.

8. Calculate secondary moments and design moments by factoring and combining results from previous moment distributions.

a) Calculate secondary moments due to post-tensioning. The power of the load balancing method is that it directly 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:

$$M_{bal} = M_1 + M_2 \text{ or } M_{bal} = Fe + M_2$$

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:

$$Vc/3 \text{ at support 2} = 0.045 \times 9 \times \frac{14}{12} \times \frac{1}{3} = 0.158$$

Supports	2	3	4
Balanced Moment at Supports − Vc/3	+1.56 -0.16	+1.12 -0.16	+1.21 -0.16
Balanced Moments at Face	$1.40	+0.96	+1.05
Fe = 10.94k x 0.5/1 2 =	-0.46		
Fe = 9.72k x 0.5/12 =		-0.41	-0.41
Secondary Moments, M_2 =	+0.94	+0.55	+0.64

(Moments shown have units in ft. kips)

b) Combine 1.4 dead + 1.7 live + 1.0 M_2 (units in ft. kips)

	Midspan	2	Midspan	3	Midspan	4
1.4D	+2.22	-3.35	+1.04	-2.41	+1.30	-2.60
1.7L	+2.67	-3.18	+2.18	-2.91	+2.18	-2.91
1.0M	+0.47	+0.94	+0.75	+0.55	+0.60	+0.64
M	+5.36	-5.59	+3.97	-4.77	+4.08	-4.87
V	1.63	1.63	1.63	1.63	1.63	1.63
Vc/3		+ .63		+ .63		+ .63
Vc/6	.31		- .31		- .31	
M_{face}	+5 05	-4.96	i3.66	-4.14	+3.77	-4.24

9. Calculate capacity utilizing minimum bonded steel area = 0.002 bh in accordance with ACI 318-83 equation (18-6).

a) Calculate ultimate tendon force from equation (18-5) of ACI 318-83.

$$f_{ps} = f_{se} + 10{,}000 + \frac{f'_c}{300\,\rho_p}$$

Stressing tendons to 0.7 x 270 = 189 ksi, and allowing 29,000 psi for losses provides an effective tendon force, f_e, of 160,000 x 0.153 = 24.5 kips for each 1/2 inch diameter 270 k strand (area of steel = 0.153 sq. in.)

$$\rho_p = A,, / bd$$

$$\text{end span } \rho_p = \frac{10.94}{24.5} \times .153/(12 \times d)$$

$$\text{Use average d} = \frac{3.25 + 4.5}{2} = 3.88 \text{ in.}$$

$$\text{end span } \rho_p = \frac{10.94}{24.5} \times .153/(12 \times 3.88) = .00147$$

$$\text{end span } f_{ps} = 160{,}000 + 10{,}000 + \frac{4000}{.440} = 179{,}090 \text{ psi}$$

$$\text{interior span } \rho_p = \frac{9.72}{24.5} \times .153/(12 \times 3.88) = .00130$$

$$\text{interior span } f_{ps} = 170{,}000 + \frac{4000}{.390} = 180{,}250 \text{ psi}$$

Use average value of 179,670 psi throughout for f_{ps}

$$F_{ps} = \frac{179.7}{160} \times 10.94 = 13.44 \text{ kips/ft. end spans}$$

$$F_{ps} = \frac{179.7}{160} \times 9.72 = 10.92 \text{ kips/ft. interior spans}$$

b) Calculate Design Capacity

(1) Capacity at exterior midspan section: Exterior midspan A, = 0.002 (5.5) 12 = 0.132 sq. in. No. 4 @ 18 in. = 0.133 sq. in.

F_{su} = 13.44 kips end span
$A_s f_y = 0.133 \times 60 =$ 7.97 kips
T_u = 21.42 kips end span

depth of compression block,

$$a = \frac{T_u}{0.85 f'_c b}$$

$$a = \frac{21.42}{0.85 \times 4 \times 12} = 0.53 \text{ in.}$$

$$(d - \frac{a}{2}) = 4.50 \quad 0.27 = 4.23 \text{ in.}$$

$$M_u = \phi T_u (d - \frac{a}{2}) = 0.9\ (21.42)\ \frac{4.23}{12} =$$

6.80 ft. kips

OK M_u provided = 6.80 ft. kips > 5.05 ft. kips required

(2) Capacity at first interior support

A, = 0.133 – No. 4 @ 18 in. centers
T_u = 21.42 kips as for exterior midspan.
depth of compression block:

$$a = \frac{21.42}{.85 \times 4 \times 12}$$

a = 0.52 in.

$$(d \quad \frac{a}{2}) = 3.25 - .26 = 2.99 \text{ in.}$$

$$Mu = \phi T_u (d - \frac{a}{2}) = 0.9\ (21.42)\ \frac{2.99}{12} =$$

4.80 ft. kips

Moment provided = 4.80 ft. kips <
Moment required = 4.96 ft. kips

Check redistribution permitted by Section 18.10.4 of ACI 318.83:

$$R = 20\ (1 - \frac{\omega + \omega_p - \omega'}{0.30}) \text{ per cent}$$

provided $(\omega + \omega_p - \omega') < 0.20$

$$(\omega + \omega_p) = (\frac{0.133}{3.25 \times 12} \times \frac{60{,}000}{4{,}000} + .00147 \times \frac{179{,}670}{4{,}000})$$

$(\omega + \omega_p) = (0.0512 + 0.0660) = 0.117 <$ 0.20 OK.

$$R\ \% = 20\ (1 - \frac{0.117}{0.30}) = 12.2\%$$

With redistribution, M_u required = (4.96) (1 0.112). M_u required = 4.40 ft. kips < M_u provided = 4.80 ft. kips. Note that end span positive moment section has enough excess capacity to take the additional moment resulting from this redistribution.

(3) Check typical midspan moment capacity with #4 at 18 inch centers:

F_{su} = 10.92 kips
$A_s f_y$ = 7.98 kips
T_u = 18.90 kips

$$a = \frac{18.90}{0.85 \times 4\ 12} = 0.46 \text{ in.}$$

$$(d - \frac{a}{2}) = 4.55 - 0.23 = 4.27 \text{ in.}$$

$$\frac{4.27}{12} = 0.356 \text{ ft.}$$

$$M_u = \phi T_u (d - \frac{a}{2}) =$$

0.9 (18.9) (0.356) = 6.06 ft. kips

OK M_u provided = 6.06 ft. kips > 3.77 ft. kips required.

(4) Check typical interior support moment capacity with No.4 bars at 18 in. centers.

T_u = 18.90 kips and a = 0.46 in. from midspan section calculations above.

$$(d - \frac{a}{2}) = \frac{3.25 - 0.23}{12} = 0.252 \text{ ft.}$$

$$M_u = \phi T_u (d - \frac{a}{2}) =$$

0.9 (18.9) (0.252) = 4.29 ft. kips

M_u provided = 4.29 ft. kips > M_u required = 4.24 ft. kips OK.

(5) Check total capacity of typical interior span assuming full redistribution:

M_u required = 0.181 $(18 - 1)^2/8$ = 6.54 ft. kips

$M_{u\ provided}$ = 4.29 + 6.06 = 10.35 ft. kips > 6.54 ft. kips.

Capacity provided is more than adequate. Critical sections have been checked above using ACI Code limits on redistribution of moment, and have been found to be adequate. 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.

10. Check shear capacity

$$V_u = (9 - 0.58)\ 0.181 = 1.52 \text{ kips}$$

$$v_u = \frac{1.52}{0.85 \times 12 \times 3.25} = 0.046 \text{ ksi}$$

$$2\sqrt{f'_c} = 0.126 \text{ ksi X.046 ksi}$$

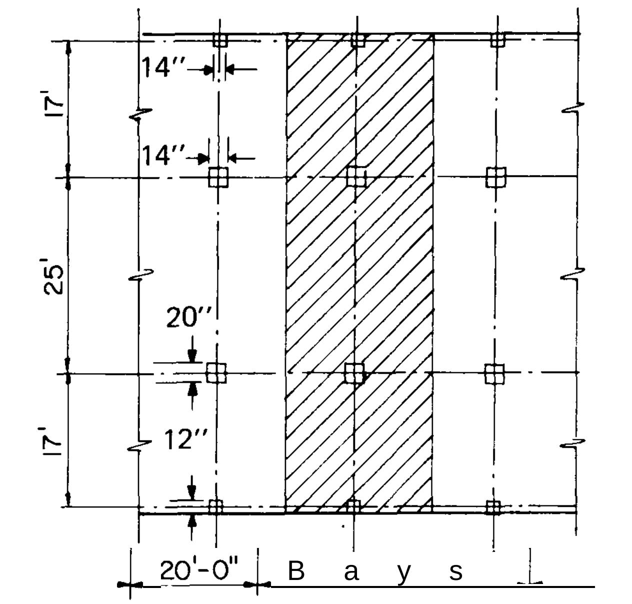
Shear strength is more than adequate without checking further since v_u need not be taken as less than $2\sqrt{f'_c}$ under the provisions of ACI 318-83. In cases where applied shear stresses are higher (rare for one-way slabs) permissible shear stress may be evaluated by use of equations (11-11) and (11-13) of ACI 318-83.

11. Reference

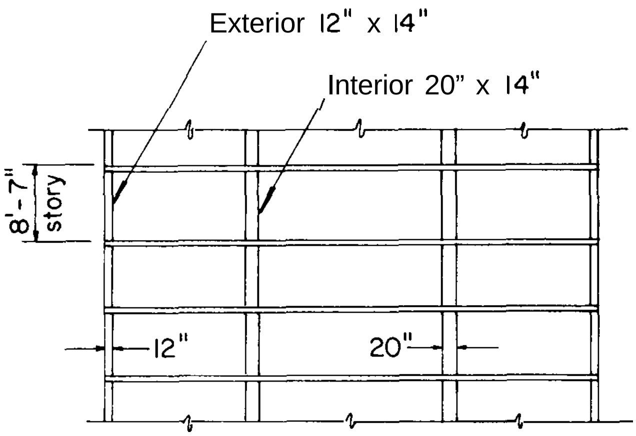
(1) "Continuity in Concrete Building Frames", Portland Cement Association, Skokie, Illinois, 1959.

5.5.4 Flat plate Apartments

I. Design typical transverse strip as described in drawings and calculations below.



PART PLAN



SECTION

Slab Thickness @ L/45

$$\text{Longitudinal} = \frac{20 \times 12}{45} = 5.3''$$

$$\text{Transverse} = \frac{25 \times 12}{45} = 6.7''$$

Use 6%" Slab

Loads

6½" Slab = 81

Partitions = 15

Dead = 96 x 1.4 = 134

Live = 40 x 1.7 = 68

Total = 136 psf = 202 psf, ultimate

Assume hardrock concrete f'_c = 4000 psi in slabs and columns.

2. 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 318-83, Section 13.7. Check flexural stresses at critical sections and revise load balancing tendon forces as required to obtain net flexural tension stresses in accordance with ACI 318-83.

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 318-83 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 318-83 Section 11 .1 1.2.

3. Load Balancing

Arbitrarily, a force corresponding to an average compressive stress of 175 psi, with maximum parabolic tendon profile, will be used for the initial estimate of balanced load.

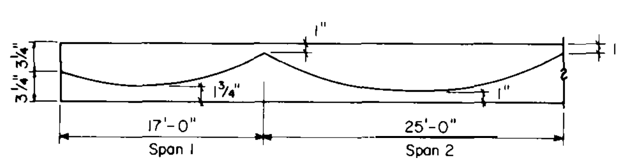
Then F_e = 0.175 x 6.5 x 12
= 13.65 kips/foot

Assuming 1/2" diameter, 270 ksi strand tendons and 30 ksi long term losses, effective force per tendon is 0.153 x (0.7 x 270 − 30) = 24.33^k.

For a 20 foot bay, 20 x 13.65/24.33 = 11.2 tendons, say eleven.

Then F_e = 11 x 24.33/20 = 13.38^k/ft.
F/A = 13.38/78 = 0.172 ksi

4. Tendon Profile



For spans 1 and 3:

$$a = \frac{3\% + 5\%}{2} - 1\% = 2.625"$$

$W_{bal} = 8 \times F \times a/(12 \times L^2) = (13.38 \times 2.625)/(1.5 \times 289) = 0.081$ ksf.
Net load causing bending = W_{net} = $0.136 - 0.081 = 0.055$ ksf

For span 2:
$a = 6\% - 1 - 1 = 4\%"$
$W_{bal} = 0.064$ ksf
$W_{net} = 0.072$ ksf

5. Equivalent Frame properties
(see ACI 318-83, Section 13.7)

a. Equivalent Columns

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"*

$$K_c = \frac{EI}{L'}\left(1 + 3\left(\frac{L}{L'}\right)^2\right)$$

Where I is taken at the column, L is center to center height and L' is clear height.

Carryover factors are approximated by $-1/2\ (1+3h)$ 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"[(2)]

$$K_c = \frac{4EI}{L - 2h}$$

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, ibid, for a comparison of approximate and classical methods.

Exterior column - 14 x 12

$$I = \frac{14 \times 12^3}{12} = 2016 \text{ in.}^4$$

$$\frac{E_{col}}{E_{slab}} = 1.0$$

$$K_c = \frac{4 \times 1.0 \times 2016}{103 - 2 \times 6.5} = 90 \times 2 =$$

180 joint total

Torsional stiffness of slab in column line, K_t, is calculated as follows:

$$C = (1 - 0.63\ \frac{x}{y})\ \frac{x^3y}{3}$$

$$= (1 - 0.63\ \frac{6.5}{12})\ \frac{(6.5)^3 \times 12.0}{3}$$

$$= 724$$

$$K_t = \frac{\Sigma 9 \times C \times E}{L_2 \times (1 - c_2/L_2)^3}$$

$$= \frac{9 \times 724 \times 1}{20 \times 12(1.0 - 1.17/20)^3} + \frac{9 \times 724 \times 1}{20 \times 12(1.0 - 1.17/20)^3} = 65$$

Equivalent column stiffness is then obtained:

$$\frac{1}{K_{ec}} = \frac{1}{K_t} + \frac{1}{K_c}$$

$$K_{ec} = (1/65 + 1/180)^{-1} = 48$$

Interior column = 14 x 20

$$I = \frac{14 \times (20)^3}{12} = 9333 \text{ in.}^4$$

$$K_c = \frac{4 \times 1 \times 9333}{103 - 2 \times 6.5} = 415 \times 2$$

$$= 830 \text{ joint total}$$

$$C = (1 - 0.63 \times \frac{6.5}{20}) \times \frac{(6.5)^3 \times 20}{3}$$

$$= 1456$$

$$K_t = \frac{9 \times 1456}{240(1 - 1.17/20)^3} + \frac{9 \times 1456}{240\ (1 - 1.17/20)^3} = 130$$

$$K_{ec} = \left(\frac{1}{830} + \frac{1}{130}\right)^{-1} = 112$$

* References are listed at the end of the example.

b. Slab stiffness (see Rice and Hoffman, ibid.) width of slab-beam = 20/2 + 20/2 = 20 ft.

$$K_s = \frac{4EI}{L_1 - c_1/2}$$

where L_1 is centerline span
c_1 is column depth
at exterior column

$$K_s = \frac{4 \times 1 \times 20 \times (6.5)^3}{12 \times 17 - 12/2} = 111$$

at interior column
spans 1 and 3

$$K_s = \frac{4 \times 1 \times 20 \times (6.5)^3}{(12 \times 17 - 20/2)} = 113$$

$$K_s = \frac{4 \times 1 \times 20 \times (6.5)^3}{(12 \times 25 - 20/2)} = 76$$

c. Distribution factors for analysis by 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

6. Moment Distribution — Net loads

Since the nonprismatic section causes only very 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 = 0.055 x 289/1 2 = 1.32. Span 2 FEM = 0.072 x 625/12 = 3.75.

Moment Distribution:

0.70	0.37	0.25 symm.
-1.32	-1.32	-3.75 FEM
+0.92	-0.90	+0.61 dist
+0.45	-0.46	-0.30 C.O.
-0.32	+0.06	-0.05 dist
-0.27	-2.62	-3.49

7. Check net tensile stresses.

a. At face of column

Moment at column face is center line moment + Vc/3

$$-M_{max} = -3.49 + \frac{20}{3 \times 12}(12.5 \times 0.072)$$

= -3.49 + 0.50 = − 2.99 ft.-k

$S = bt^2/6 = 12 \times 6.5 \times 6.5/6 = 84.5 \text{ in.}^3$

$$= 84.5 \text{ in.}^3 \frac{\text{ft}}{12 \text{ in}} = 7.04 \text{ in.}^2 \text{ ft.}$$

then $f_t = \frac{2.99}{7.04} - 0.172 = 0.425 \quad 0.172$

$= +.253$ ksi OK since $6\sqrt{f'_c} = .380$ ksi

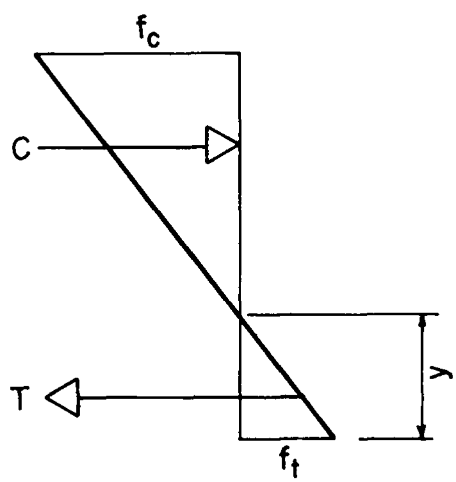
b. Check midspan tensile stress:

$+M_{max} = 5.62 - 3.49 =$
+2.13 ft.-kip

$f_t = \frac{2.13}{7.04} - 0.172 = 0.130$ ksi >

$2\sqrt{f'_c} = 0.126$ ksi

By requirements of ACI 318-83 Section 18.9, when tensile stress of $2\sqrt{f'_c}$ is exceeded, the entire tensile force must be replaced by mild reinforcing at a stress of $f_y/2$.



$f_c = -0.302 - 0.172 = -0.474 < 0.45 \times (4000)$

then $y = 6.5(0.130/(0.130 + 0.447)) = 1.40$ in.

$T = 0.130 \times 1.40 \times \frac{12}{2} = 1.09^k/\text{ft.}$

$A_s = \frac{1.09}{60/2} = 0.036 \text{ in.}^2/\text{ft.}$

Say 4 No. bars at 60" o.c. bottom of midspan 2, $A_s = 0.04 \text{ in.}^2/\text{ft.}$

This completes the service load portion of the design, but the strength in flexure and shear must be verified to complete the design. Based on the service load flexural stresses, the design is conservative.

8. Flexural Capacity

a. 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 x 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 condtion with appropriate load factors. For this example, the balanced load moments are determined by moment distribution as follows:

Span 1, 3 balanced load FEM = 0.081 x 289/12 = 1.95

Span 2 FEM = 0.064 x 625/12 = 3.33

Moment Distribution Balanced Loads:

0.7	0.37	0.25 symm.
+1.95	+1.95	+3.33 FEM
-1.37	+0.51	-0.35 dist
-0.25	+0.68	+0.17 C.O.
+0.18	-0.19	+0.13 dist
+0.51	+2.95	+3.28

Since load balancing accounts for both primary and secondary moment directly, secondary moments can be found from the following relationship:

$M_{bal} = M_1 + M_2$ then $M_2 = M_{bal} - M_1$

The primary moment, M_1, is F x e at each support. Thus at exterior columns, the secondary moment, M_2

is $M_2 = 0.51 - 11\ 3.38^k \times \frac{0 \text{ in}}{12 \text{ in./ft.}}$

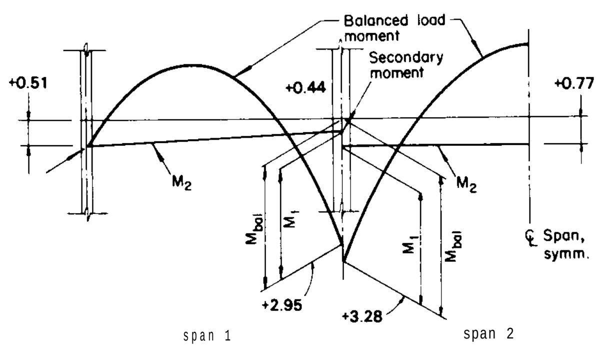
= 0.51 ft.-k

At interior column, span 1, 3

$M_2 = 2.95 - 13.38 \times (3.25 - 1.0)/12 = 0.44$ ft.-k

Span 2:

$M_2 = 3.28 - 13.38 \times (2.25)/12 = 0.77$ ft.-k



Factored load moments

Span 1, 3:

1.4 dead + 1.7 live FEM
= 0.202 x 289/12 = -4.86 ft.-k

Span 2:

FEM = 0.202 x 625/12 = -10.52 ft.-k

Moment Distribution Factored Loads:

0.7	0.37	0.25 symm.
-4.86	-4.86	-10.52 FEM
+3.40	-2.09	+ 1.42 dist
+1.05	-1.70	− 0.71 C.O.
-0.74	+0.37	− 0.25 dist
-1.15	-8.28	-10.06

Combine factored load and secondary moments to obtain design moments:

	Span 1		Span 2
Factored load Moments	-1.15	-8.28	-10.06
Secondary Moments	+0.51	+0.44	+0.77
design moments at col	-0.64	-7.84	-9.29
Moment reduction to face Vc/3	+0.42	+1.19	+1.40
design moments at critical section	-0.22	-6.65	-7.89

Calculate design midspan moments

Span 1

$$V_{exterior} = \frac{.202 \times 17}{2} - \frac{(8.28 - 1.15)}{17}$$

$= .172 - 0.42 = 1.30$ kips/ft.

$V_{center} = .172 + 0.42 = 2.14$ kips/ft.

Point of zero shear and maximum

moment: $x = \frac{1.30}{.202} = 6.45$ ft. from

exterior column.

End span positive moment =

$1.30 \times 6.45 = +.840$

$-\frac{.202\ (6.45)^2}{2} = -4.20$

End moment $= \underline{-1.15}$

$+3.05$

$M_2 = \underline{0.48}$

$+M$ max. $= +3.53$ ft. kips/ft.

Span 2

$V = \frac{.202 \times 25}{2} = 2.52$ k/ft.

+ Moment $= .202 \times 25^2/8 =$

15.78 ft. kips/ft.

− End Moment $= \underline{-10.06}$

$+ 5.72$

$M_2 = \underline{+\ .77}$

+ M max. = + 6.49 ft. kips/ft.

b. Calculation of flexural capacity

Check interior support section

Section 18.9 of ACI 318-83 requires a minimum amount of mild steel at the immediate column zone regardless of service load stress conditions, or strength, unless more than the minimum is required for flexural capacity. This minimum amount is to help insure the integrity of the punching zone so that full shear capacity can be developed, and is determined by the following expression:

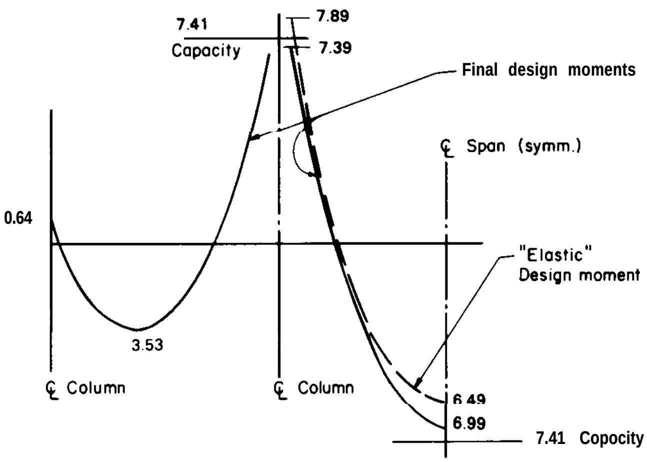
$A_s = 0.00075h \times L$

The initial check of flexural strength will be made considering this steel.

$A_s = 0.00075 \times 6.5 \times (17 + 25)/2 \times 12$
$= 1.22$ in.^{2}

Say 6 No. 4 Bars x 9 ft. Space at maximum 6" o.c. so that bars are placed within a width of column plus 1-1/2 slab thickness each side of column. Then for average one foot strip,

$A_s = 6 \times 0.2/20 = 0.06$ in.^{2}/ft.



Calculate design stress in tendon, use ACI 318-83 equation (18-5):

$$f_{ps} = f_{pe} + \frac{f'_c}{300\rho_p} + 10 \text{ ksi}$$

since we have 1 1-1/2 in. dia. tendons in a 20 ft. bay, each with area = 0.153 in.^{2}:

$$\rho_p = \frac{A_{ps}}{bd} = \frac{11 \times 0.153}{20 \times 12 \times 5.5} = 0.00127$$

f_{se} = 0.7 x 270 – 30 ksi losses = 159 ksi

$$f_{ps} = 159 + 10 + \frac{4.000}{0.00127 \times 300} = 169 +$$

10.5 = 179.5 ksi

$$F_{su} = \frac{179.5 \times 0.153 \times 11}{20} = 15.10 \text{ k/ft.}$$

F_u = 60 x 0.06 = 3.60

F = Total Force = 18.7 k/ft.

depth of compression block $a = \dfrac{F}{.85\, b\, f'_c}$

$$a = \frac{18.7}{0.85 \times 12 \times 4} = 0.46 \text{ in.}$$

since bars and tendons are in the same layer:

$$(d - \frac{a}{2}) = (5.5 - \frac{0.46}{2})/12 = 0.44 \text{ ft.}$$

moment capacity at column centerline=

M_u = 0.9 x 0.44 x 18.7 = 7.41 ft. kips/ft.

This value is less than the required capacity of 7.89 ft.-kips/ft.

Calculate available capacity at midspan and allowable inelastic moment redistribution at column. See ACI 318-83 Section 18.10.

Allowable redistribution:

$$= 20\% \times \left(1 - \frac{\omega_p + \omega}{0.30}\right)$$

$$\Sigma\omega = \frac{18.7}{5.5 \times 12 \times 4} = 0.071 < 0.20$$

R = 20 (1 - E) = 15.2%

M_R = 0.152 x 7.41 = 1.13 ft.-k

Since the midspan of span 2 requires 4 No. 4 bars from service load considerations, the flexural strength is:

F_{su} = 15.10 k/ft.

F_u = 60 x 0.04 2.40

F = total force = 17.50

$$a = \frac{17.50}{0.85 \times 12 \times 4} = 0.43 \text{ in.}$$

$$(d - \frac{a}{2}) = (5.5 - \frac{0.43}{2})/12 = 0.44 \text{ ft.}$$

Moment capacity at center of span:

M_u = 0.9 x 0.44 x 17.50 = 6.93 ft. kips/ft.

The required moment capacity is 6.49 ft-k, which leaves 0.44 ft-k available to accommodate moment redistributed from the support section. If 0.44 ft-k is redistributed:

-M = -7.89 + 0.44 = -7.45 > 7.41 NG

+M = +6.49 + 0.44 = 6.93 = 6.93 OK

Thus minimum rebar and tendons are not adequate for strength at the column. Addition of 2 No. 4 bars at midspan will make midspan strength identical to strength at column (which also has6 No. 4 bars) = 7.41 ft. kips/ft. In this case 7.41 ft. kips – 6.49 ft. kips/ft. or 0.92 ft. kips/ft. are available for redistribution. Redistributing 0.50 ft. kips:

-M = -7.89 + 0.50 = 7.39 < 7.41 OK

+M = +6.49 + 0.50 = 6.99 < 7.41 OK

Therefore, with addition of 2 No. 4 bars at midspan and redistribution of 0.50 ft. kips from the negative moment area to midspan, both sections are adequate for strength. Midspan sections of spans 1 and 3 have more than adequate capacity by comparison with span 2.

The flexural capacity at exterior columns is goverened by moment transfer requirements. Since moment transfer also involves shear stresses, the two aspects will be treated under the heading of shear.

9. Shear Capacity

a. Shear at exterior Column
Vertical shear at exterior column calculated above at 1.30 kip/ft.
Total shear at exterior column = 20 x 1.30 = 26.0 kips

Assume exterior skin is masonry and glass averaging 0.4 klf

$V_u = 1.4 \times 0.4 \times 20 = 11.2$ kips
Slab shear = 26.0
Total Shear = 37.2 kips

b. Moment Transfer:

At exterior columns, a portion of the total bay moment is transferred to the column by the eccentricity of the critical section relative to the column center. In order to evaluate these cases the properties of the critical section must first be calculated.

Shear Section Properties

The critical shear section is taken at d/2 from the face of the column as per Fig. 5.19. Referring to Fig. 5.19:

Assume d = 0.8 x 6.5 = 5.2 in.

$c_1 = 12$ in.
$c_2 = 14$ in.

$c_m = 14 + 5.2 = 19.2$ in.

$$c_t = 12 + \frac{5.2}{2} = 14.6 \text{ in.}$$

A, = 5.2(19.2 + 2 x 14.6) = 252 in.^{2}
$c_{AB} + 14.6^2 \times 5.2/252 = 4.40$ in.
$c_{CD} = 14.6 - 4.40 = 10.2$ in.
g = 10.2 − 12/2 = 4.2

$$\alpha = 1 - \frac{1}{1 + 2/3\left(\frac{c_t}{c_m}\right)^{1/2}}$$

$$\alpha = 1 - \frac{1}{1 + 2/3\left(\frac{14.6}{19.2}\right)^{1/2}}$$

$$\alpha = 1 - \frac{1}{1 + .58} = 1 - .63 = 0.37$$

$$J_c = \frac{dc_t^3}{6} + \frac{c_t d^3}{6} + c_m d c_{AB}^2$$
$$+ 2\,ctd\left(\frac{c_t}{2} - c_{AB}\right)^2$$

$$J_c = \frac{5.2 \times 14.6^3}{6} + \frac{14.6 \times 5.2^3}{6}$$
$$+ 19.2 \times 5.2 \times 4.0^2$$
$$+ 2 \times 14.6 \times 5.2\left(\frac{14.6}{2} - 4.40\right)^2$$

J_c = 2697 + 342 + 1937 + 1277 = 6249 in.^{4}

Total bay moment at column centerline:

M_u = 20 x (-0.64) = -12.8 ft. kips

Moment transferred by eccentricity of shear reaction:

$$Vg = 26.0 \times \frac{4.2}{12} = -9.10 \text{ ft. kips}$$

Net moment to be transferred = M_t

= -12.8 − (-9.10) = -3.70 ft. kips

Amount to be transferred by shear = αM_t

αM_t = 0.37 x -3.70 = -1.37 ft. kips

$$v_c = \frac{37{,}200}{252 \times .85} \quad \frac{1{,}740 \times 12 \times 4.40}{6249 \times 0.85}$$

v_c = 173 + 17 = 190 psi

Shear stress allowable according to Equation (1 l-36) of ACI 318-83.

$$v_c = 4\sqrt{f'_c} = 4\sqrt{4000}$$

v_c = 253 psi

Allowable shear stress exceeds calculated stress. Note the discussion in Section 5.3.8.2 concerning the use of Equation (1 l-37) of ACI 318-83 for shear design at edge columns. In this case, equation (11-37) would permit an allowable shear at the exterior column:

$$v_c = 3.5\sqrt{4000} + 0.3 \times (172)$$

(ignoring v_p/A_c)

$$v_c = 221 + 52 = 273 \text{ psi}$$

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 thecolumn plus 1-1/2 slab thicknesses each side. Assume that of fthe eleven 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 thestructural drawings. Besides providing flexural capacity, this prestress 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 x 6.5 x 12 x 17 = 1.0 in.^{2}

Say 5 No. 4 bars x 5 ft. including standard nook
A_s = 1.0 in.^{2}

Calculate stress in strand tendon

b = 14 + 3 x 6.5 = 33.5 in.

$$f_{ps} = \frac{33.5 \times 3.25 \times 4}{300 \times 0.153 \times 3} + 169 = 172 \text{ ksi}$$

$$F_p = \frac{172}{159} \times 24.33 \times 3 = 79.0^k$$

F_y = 60 x 5 x 0.2 = 60.0^k
T_u = 139.0 kips

$$a = \frac{139.0}{0.85 \times 4 \times 33.5} = 1.22 \text{ in.}$$

tendon $j_u d$ = (3.25 − 1.22/2)/12 = 0.22 ft.

rebar $j_u d$ = (5.5 − 1.22/2)/12 = 0.41 ft.

ϕM_u = 0.9 x (0.22 x 79 + 0.41 x 60) = 37.38 ft. kips. Much **greater than** moment transfer requirement.

c. Shear at Interior Column

Direct shear left and right of interior columns is calculated in section 8 above.

Total direct shear = (2.14 + 2.52)20 = 93.20 kips
Moment transfer M_t = 20(9.35 − 7.86) = 29.80 ft. kips

Shear section properties (See Fig. 5.18)

d = 6.5 − 1.0 = 5.5 in.

d + c_1 = 5.5 + 20 = 25.5 in.

at torsion faces, d = 0.8 x 6.5 = 5.2 in.

d + c_2 = 5.2 + 14 = 19.2 in.

$b_o d$ = 2 x (25.5 x 5.2 + 19.2 x 5.5) = 476 in.^{2}

Polar moment of inertia:

$$J = 2\left(5.2 \times (25.5)^3/12 + 25.5 \times (5.2)^3/12 + 19.2 \times 5.5 \times \left(\frac{25.5}{2}\right)^2\right) = 49{,}300 \text{ in.}^4$$

$$\frac{J}{12 \times c} = \frac{49{,}300}{12 \times 25.5/2} = 322.2 \text{ in.}^2\text{-ft.}$$

Portion of moment to be transferred by torsional shear:

$$a = 1.0 - \frac{1.0}{1.0 + 2/3\left(\frac{25.5}{19.2}\right)^{1/2}} = 43.3\%$$

M_{vt} = 0.433 x 29.80 = 12.90 ft. kips
M_{vf} = 29.80 − 12.90 = 16.90 ft. **kips**

Shear Stresses:

direct shear = $v_u = \frac{93.20}{476}$ = 0.196 ksi

torsional shear $= v_t = \frac{12.90}{322.2} = 0.040$

total shear = **0.236 ksi**
divided by ϕ = 0.85 = 0.276 ksi

Calculated allowable shear stress by ACI 318-83 equation (1 l-37):

$$v_{cw} = 3.5\sqrt{f'_c} + 0.3\ F/A + V_p/A_c$$

$$v_{cw}\ 3.5\sqrt{4000} + 0.3 \times 0.172 = 0.273 \text{ ksi}$$

0.2734 < 0.2765 but Vp component of tendon force crossing the critical section will make up the slight deficiency.

M_{vf} = 16.90 ft. kips

Say f_p = 79.0 kips as per exterior column.

A_s = 0.00075 x 6.5 x 21.0 x 12 = 1.22 in.^{2}

Use 6 No. 4 bars A_s = 1.20 in.^{2}

F_y = 1.20 x 60 = 72.0 kips

T_u = 79.0 + 72.0 = 151.0 kips

depth of compression block, a = $\frac{151}{0.85 \times 4 \times 33.5}$ = 1.33 in.

$(d - \frac{a}{2}) = (5.5 - \frac{1.33}{2})/12 = 0.40$ ft.

ϕM_u = 0.9 x 0.40 x 151 = 54.4 ft. kips

Moment transfer capacity in flexure of 54.4 ft. kips is much greater than the required flexural moment transfer of 16.90 ft. kips

It is strongly emphasized that tendons in each direction should pass directly through the critical shear perimeter of the column. In addition to shear contribution at design loads, these tendons provide a residual shear capacity after complete failure of the punching section by acting in pure tension or "catenary" action. Shear reinforcement may be designed using ACI 318-83 Section 11.11.

10. Deflection

Calculate live load deflection of a 1-ft. strip in 25-ft. span by area moment procedure.

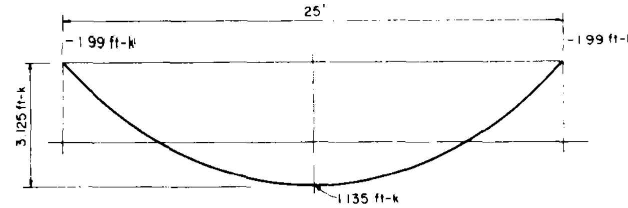
Live load = 0.040 ksf,

I = 12 x $6.5^3/12$ = 275 in.^{4}

E = 3.8 x 10^3 ksi

EI = 1045 x 10^3 k-in.^{2}

Support moments = $\frac{0.040}{0.202}$ x 10.06 = 1.99 ft. kips



Slope at A and B is $\Sigma \frac{Mdsx}{EI}$

$$-\text{Moment} = \frac{1.990 \times 12 \times 25 \times 12 \times .5}{1045 \times 10^3} = -.00342$$

$$+\text{Moment} = \frac{3.125 \times 12 \times 25 \times 12 \times 2/3 \times .5}{1045 \times 10^3} = +.00359$$

Slope at A and B = +.00017

Positive slope means tanget slopes downward from left to right from Support A. This slope is increased by negative moment and decreased by positive moment.

Deflection at midspan

+ 0.00017 x 12 x 12.5 + 1.99 x 12^3 x 12.5^2 x 0.5/(1045 x 10^3) = 0.283

− 3.125 x 12^3 x 12.5^2 x 2/3 x 0.375/(1045 x 10^3) = -0.202

total deflection = +0.081

Approximate live load deflection in 20 ft. interior spans is similar to fixed end beam:

$$= \frac{wL^4}{384EI} = \frac{0.04 \times 20^4 \times 12}{384 \times 1045 \times 10^3} = 0.027 \text{ in.}$$

total deflection at center of panel = 0.108

Calculated live load deflection = 25 x 12/0.108 = span/2778.

The net dead load in this span is 0.032 ksf, so by comparison with the live load deflection calculated above, the long term dead load deflection, assuming a creep factor of 2, is approximately 0.17 inch and the total long term dead plus instantaneous live load deflection at the center of the panel would be about 0.28 inches. Permissible long term plus short term deflection = span/480 = 0.63 in. The calculated deflections of this structure are satisfactory.

11. Distribution of Tendons

In accordance with ACI 318-83, Section 18.12, the 11 tendons per 20 ft. bay in this design will be distributed in a group of 3 tendons directly through the column with the remaining 8 tendons spaced at 2 ft. 3 in. centers (about 4 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.

12. References

(1) Cross, Hardy, and Morgan, Newlin Dolbey, "Continuous Frames of Reinforced Concrete" John Wiley & Sons, Inc. New York, 1954.

(2) Rice, Paul F., and Hoffman, Edward S., "Structural Design Guide to the ACI Building Code", Van Nostrand Rheinhold, New York.

Chapter 6

Detailing and Construction Procedures

6.1 GENERAL

The primary emphasis of this chapter is on detailing and construction procedures for buildings. While much of the material presented is also applicable to post-tensioned bridge construction and other applications of post-tensioning, these applications may require specialized considerations beyond the scope of this chapter. Detailing procedures for cast-in-place post-tensioned bridges are presented in a separate PTI Publication entitled "Post-Tensioned Box Girder Bridge Manual."

6.2 DETAILING

In accordance with the procedures followed for most types of construction, details for post-tensioned buildings are first developed by the design engineer or architect. Following award of the contract more specific detail drawings, called "shop drawings" are developed by the post-tensioning subcontractor as well as by the subcontractors for the other materials to be embedded in the project. The shop drawings for post-tensioning materials are normally prepared in much more detail than the design drawings and are submitted for review and approval by the design agency before fabrication of tendons is initiated.

In cases where construction difficulties, problems or accidents develop on a project, litigation often develops relative to liability for financial losses. Such litigation sometimes evolves around questions as to whether the material supplied in accordance with approved shop drawings conforms to the design concept of the project and complies with the project specifications and drawings as prepared by the design engineer. In a "Commentary on Contract Documents" published by the National Society of Professional Engineers in 1974, the following position is taken relative to the engineer's responsibility in such a case:

"In approving Shop Drawings, catalog data, schedules and samples, the Engineer should indicate that these items conform to the design concept of the Project and comply with the Specifications and Drawings prepared by him. "

"Once the submission is given to him, the Engineer approves or disapproves of it to the extent that it is or is not responsive to the data given in the contract documents. The Owner and the Contractor are entitled to reply upon the Engineers approving or disapproving these submissions. "

The above quotations are presented in this discussion relative to detailing and shop drawings to emphasize the importance of this step in the construction process as well as to suggest the fundamental responsibilities of the design engineer. While post-tensioning materials fabricators do routinely review projects from an engineering perspective in development of shop drawings, their awareness of the intent of the design engineer is usually limited to the information communicated by the design drawings and contract documents. The post-tensioning subcontractor normally does not, have design calculations or other information on the project necessary for complete determination of structural adequacy. For these reasons, the post-tensioning subcontractor as well as the general contractor must rely on the engineer to determine that the information submitted on the shop drawings is structurally adequate and responsive to the intent of the contract documents.

It is essential that details of the post-tensioning tendons, nonprestressed reinforcement, underfloor ducting for electrical or telephone services, and any other embedded items be reviewed for conflicts during the detailing stage. It often develops that final details for these different materials are shown on separate shop drawings produced by different subcontractors, and that these drawings show two or more items occupying the same space. The ultimate responsibility for eliminating such conflicts between shop drawings is considered to rest with the engineer, architect or other agency responsible for construction supervision. In most cases details can be rather easily adjusted at the shop drawing stage to accommodate all embedded items. When conflicts do arise during the development of shop drawings, or during construction, preferential consideration should be given to the primary structural system which normally includes the post-tensioning tendons. Special procedures for detailing beam-column joints to avoid conflicts or undue conjestion are described in *Section 6.2.7.*

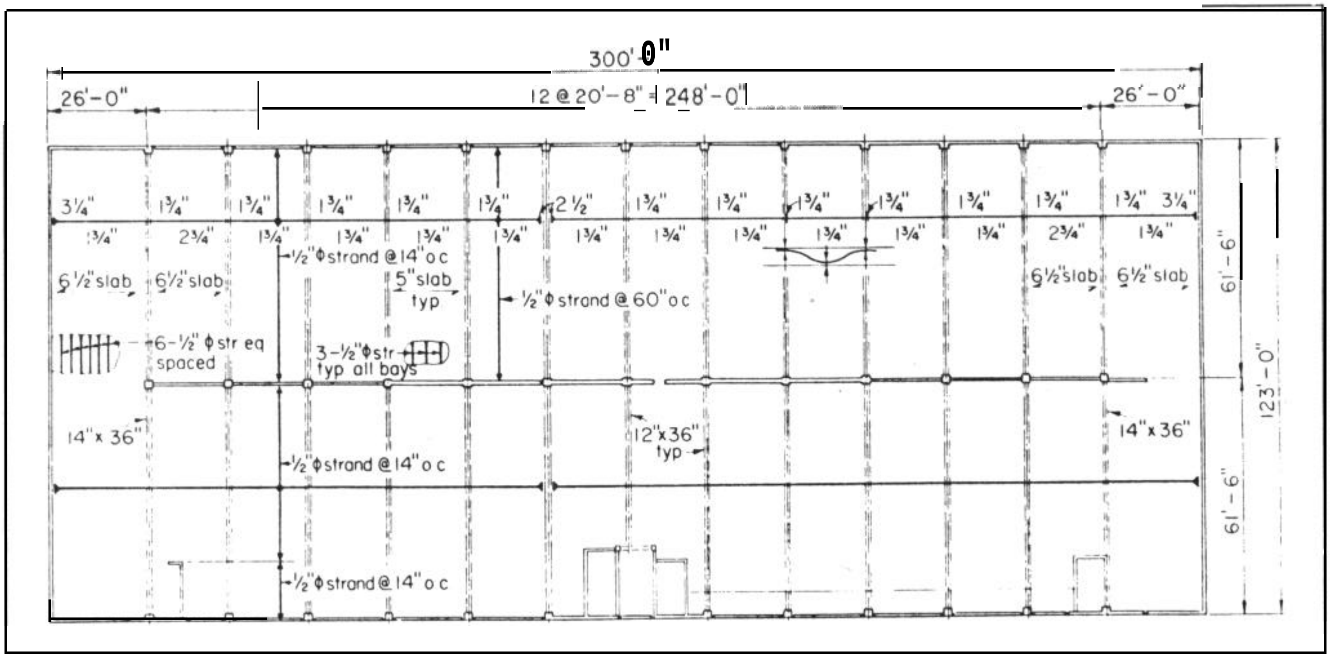


Fig. 6.1 — Framing plan for a one-way beam and slab parking garage

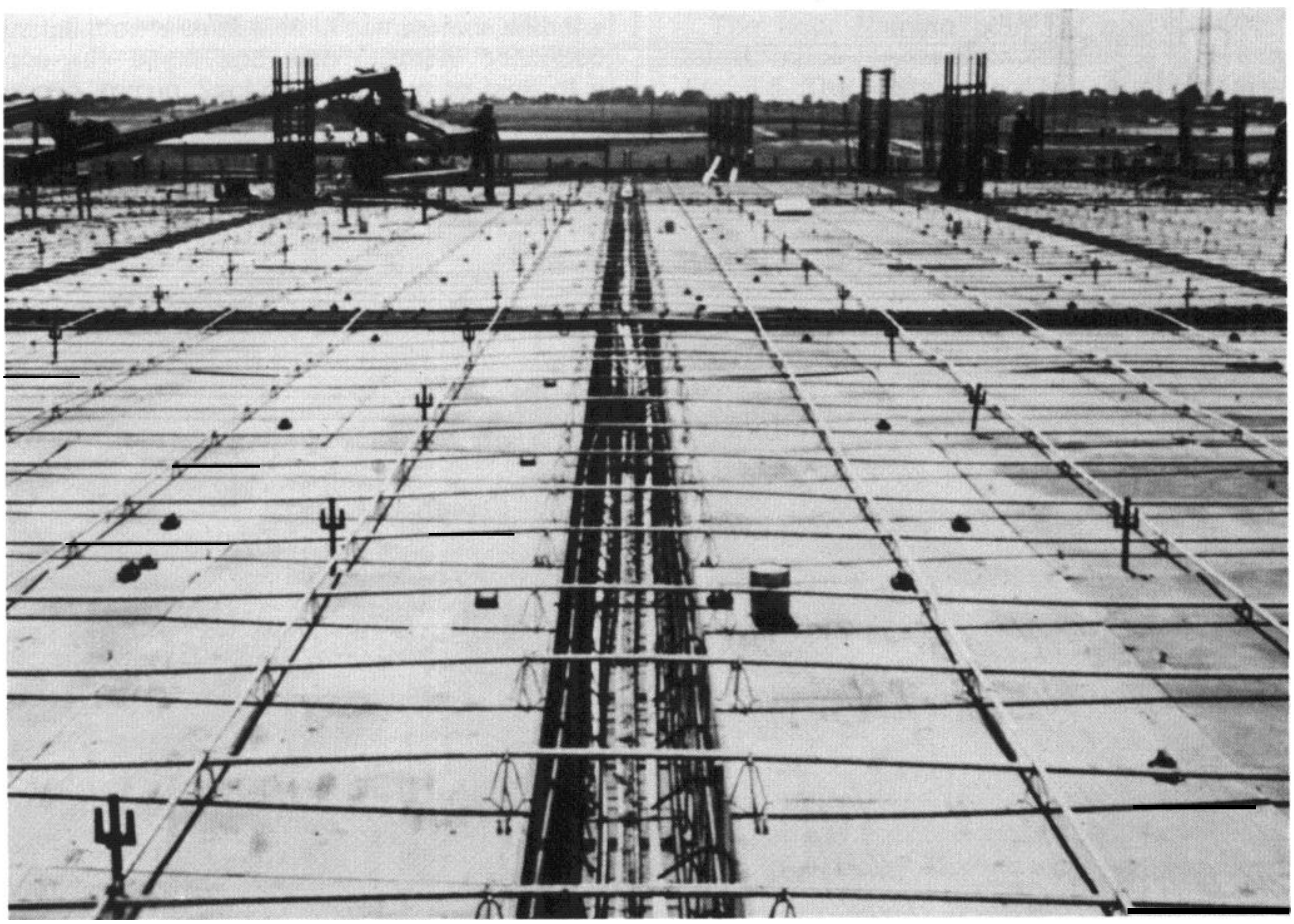


Fig. 6.2 — A typical one-way beam and slab parking garage with formwork and tendons in place.

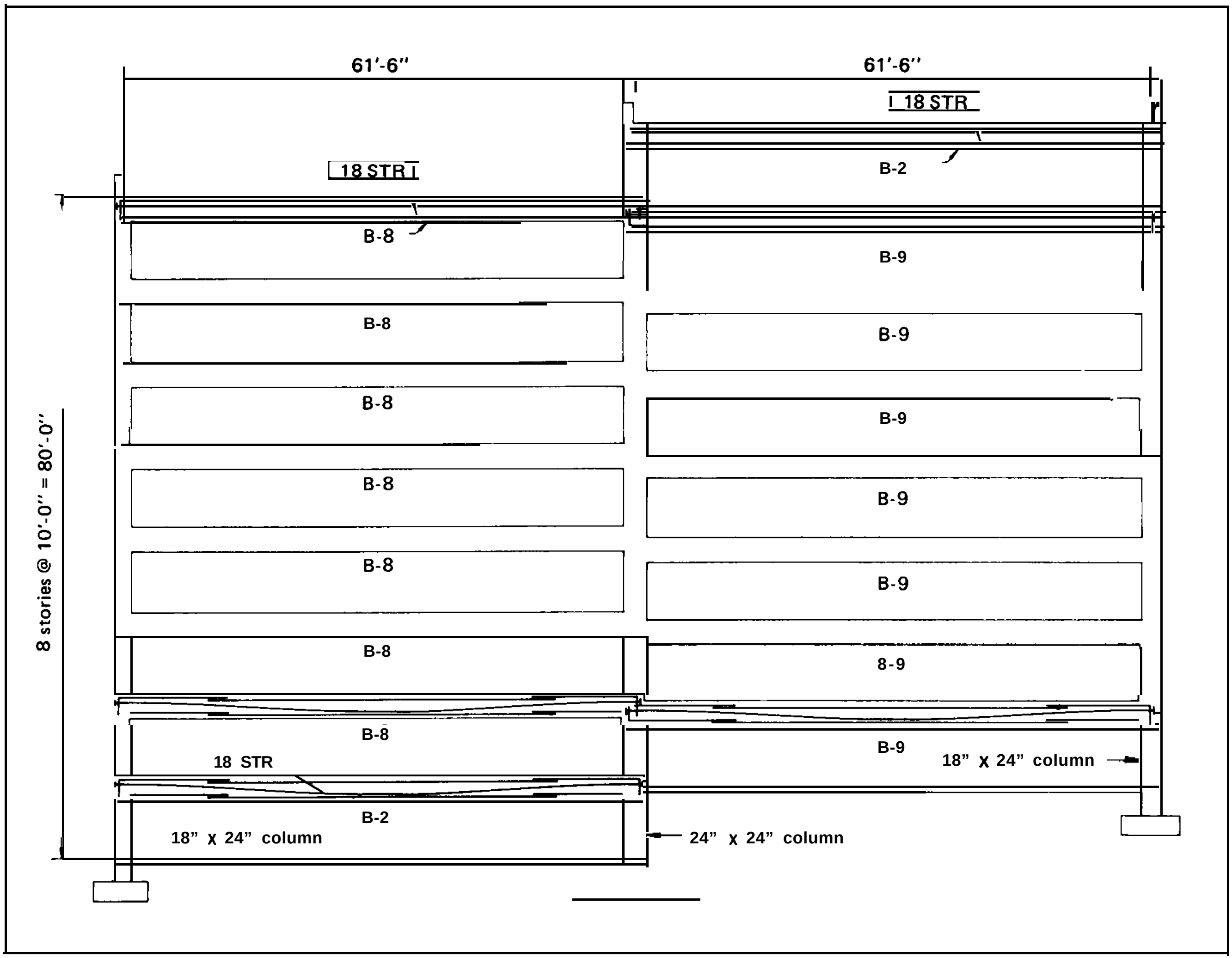


Fig. 6.3 — Frame elevation, one-way slab parking garage

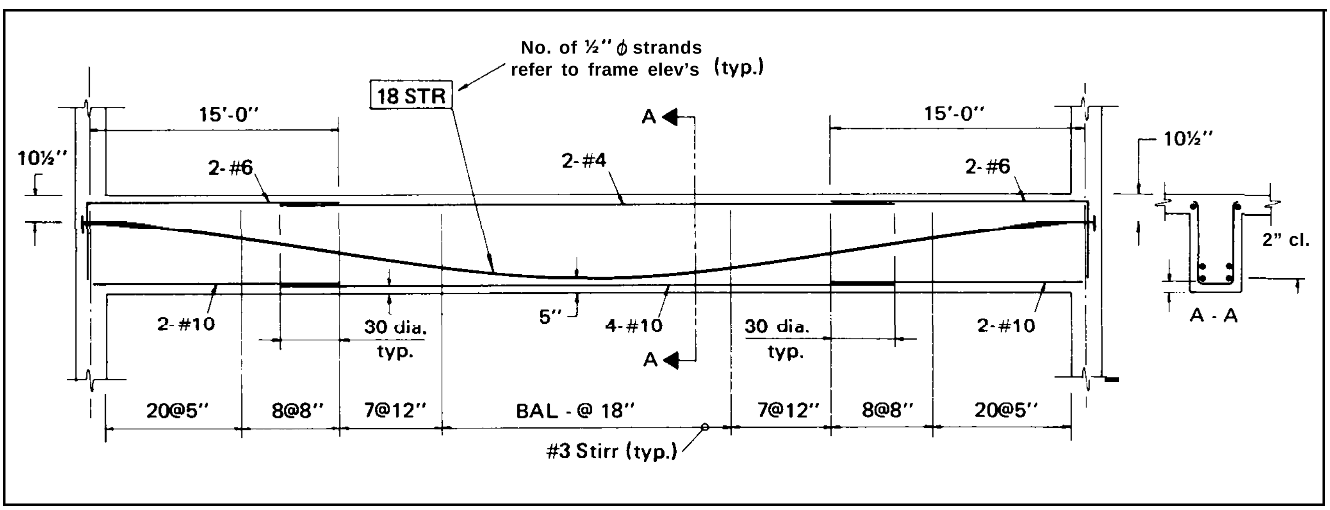


Fig. 6.4 — Construction details, post-tensioned cast-in-place beam.

The following sections illustrate typical design detailing practices for most common applications of post-tensioning in building construction. In most cases, photographs showing tendon installation are provided for construction projects of the type illustrated in the drawings.

6.2.1 One-Way Slab

The framing plan for a one-way beam and slab parking garage is shown in Fig. 6.7. In a typical panel, the primary slab tendons are 1/2 in. 270k strand at 14 in. center to center. Additional 1/2 in. 270k strand tendons are used at 60 in. center to center in the end panels to allow for the increased moments due to lack of continuity at one end. fig. 6.2 is a photograph of a job of this type with formwork and tendons in place, In this picture the primary structural slab tendons run from left to right across the picture. Temperature tendons occur at wider spacing normal to the primary tendons. Detailing of beam tendons for this teype of construction is described in Section 6.2.2.

One-way post-tensioned slab construction is also widely used for slabs supported on cast-in-place bearing walls. In this case, tendon installation is simplified in comparison with the one-way beam and slab example described above and in Section 6.2.2 due to lack of a 'requirement for detailing the beam tendons.

6.2.2 Beams

The beam schedule for one frame of a one-way slab parking garage is shown in Fig. 6.3. Details for beam B-2 in this frame are presented in Fig. 6.4. fig. 6.5 shows fabrication of nonprestressed reinforcement and post-tensioning tendons in beams similar to those detailed in Fig. 6.4. However, in the specific case shown in Fig. 6.5, the post-tensioning tendon and beam are continuous over two spans, whereas Fig. 6.4 shows details for a beam for a single span. Detailing of the beam-column intersection in an application such as this should be carefully developed to avoid conflicts between the post-tensioning tendon and anchorage and the main vertical column reinforcement as discussed in Section 6.2.7. Adequate clearance must be provided to permit access by stressing equipment Stressing equipment clearance requirements for various tendon sizes are typically detailed for the different post-tensioning systems in the systems data presented in Section 2.2.

6.2.3 One-Way Joists

The floor framing plan for one-way joist construction for an office building is shown in fig. 6.6. The joists span 47-ft., 4 in. between exterior columns with a total floor depth of 24-

Fig. 6.5 — Fabrication of conventional reinforcement and post-tensioning tendons in beams as detailed in Fig. 6.4.

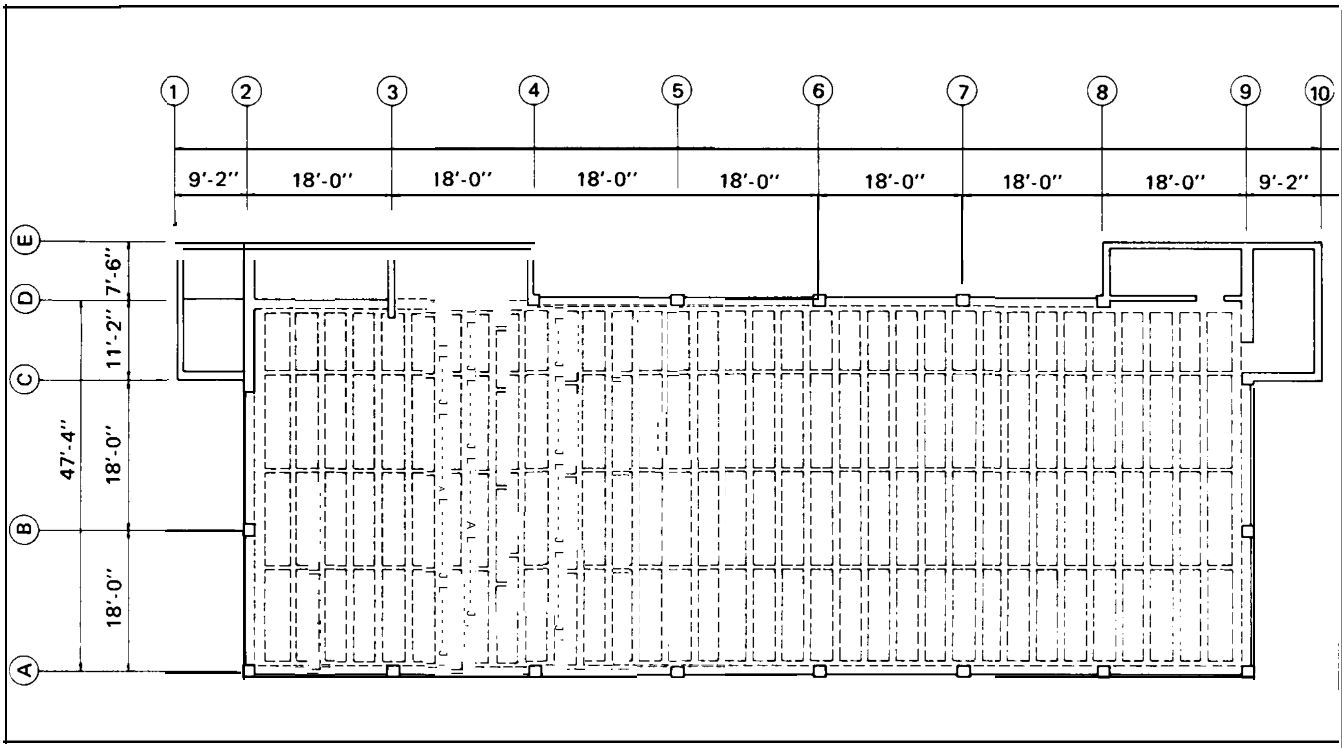
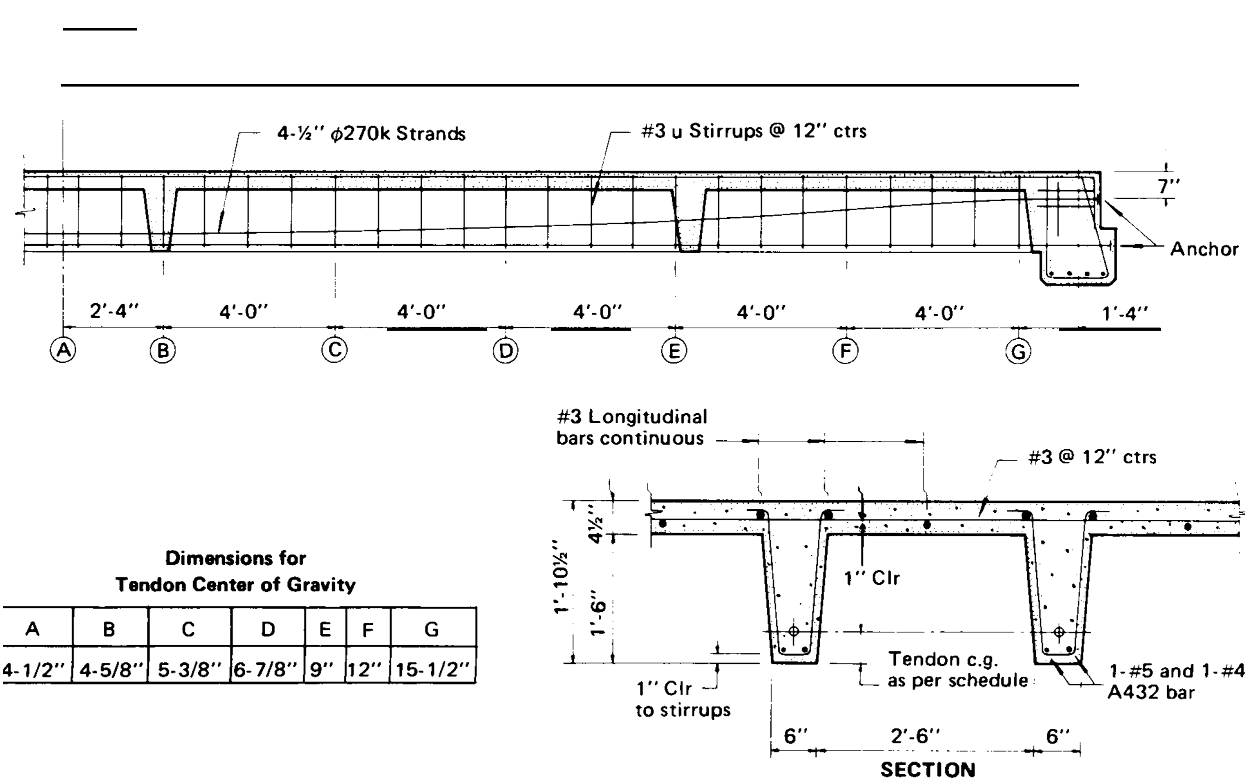


Fig. 6.6 — Floor framing plan for one-way joist office building



Dimensions for Tendon Center of Gravity

A	B	C	D	E	F	G
4-1/2"	4-5/8"	5-3/8"	6-7/8"	9"	12"	15-1/2"

Fig. 6.7 — Joist details.

1/2 in. Details of the typical joist section are shown in fig. 6.7. Typical post-tensioning in each joist stem consists of 4-1/2 in. 270k strand. Nonprestressed reinforcement in joist stems consisted of one No. 5 and one No. 4 A 432, 60 ksi yield strength bars. A construction view of a one-way joist building is presented in fig. 6.8. In this case, steel columns were utilized to minimize column size for architectural purposes

6.2.4 Two-Way Fiat Plate

Fig. 6.9 shows the tendon layout for a 7 in. flat place apartment building as presented on the shop drawing prepared by a post-tensioning material fabricator. The tendons delivered to the job site will be color coded as indicated on the drawing. Groups of two, three, or four strands are bundled together along each tendon line as indicated by the legend. Tendon hands are not required over the exterior columns at the top and bottom of the drawing because the slab is supported by beams. The drawing also shows the stressing and dead ends of the tendons, as well as the elongation of the tendons during stressing. Detail F2 showing the location of tendons and rebar over the interior columns is shown in Fig. 6.10. Except for the bundle of uniform tendons directly through the column centerline, tendons in both directions have the same eccentricity in the negative moment area. All reinforcement, tendons and rebar, is placed in two layers.

Fig. 6.77 shows an installation of a banded tendon flat plate. The tendon bands run vertically, and the uniformly spaced tendons run horizontally in the photo. *Fig.* 6.72 shows reinforcement details over a column similar to that detailed in *Fig. 6. IO.*

Sometimes it is desirable to make a structural connection between flat place (and other) post-tensioned floors and walls. When the walls are cast in advance of the floor system, post-tensioning anchors can be cast in the wall with sleeves provided for the insertion of the tendons through the wall and the anchor when the floor slab is constructed. This detail is illustrated by *Figs.* 6.13, 6.14, and 6.15.

Flat plate buildings often use more complicated geometric shapes which in turn inherently require more complicated tendon geometry. The use of banded tendon installations may provide a practical tendon layout in such cases, as discussed in Chapter 5. In some circumstances more complicated tendon geometry is required, as, for example, the layout of tendons and nonprestressed reinforcement for a typical apartment floor of the Edmonton House in Edmonton, Canada shown in ***Fig. 6.76.*** A construction view is provided in ***Fig. 6.77*** showing layout of tendons and nonprestressed reinforcement as well as placement of concrete.

Fig. 6.8 — Construction view of forming and tendon Installation for a one-way joist building.

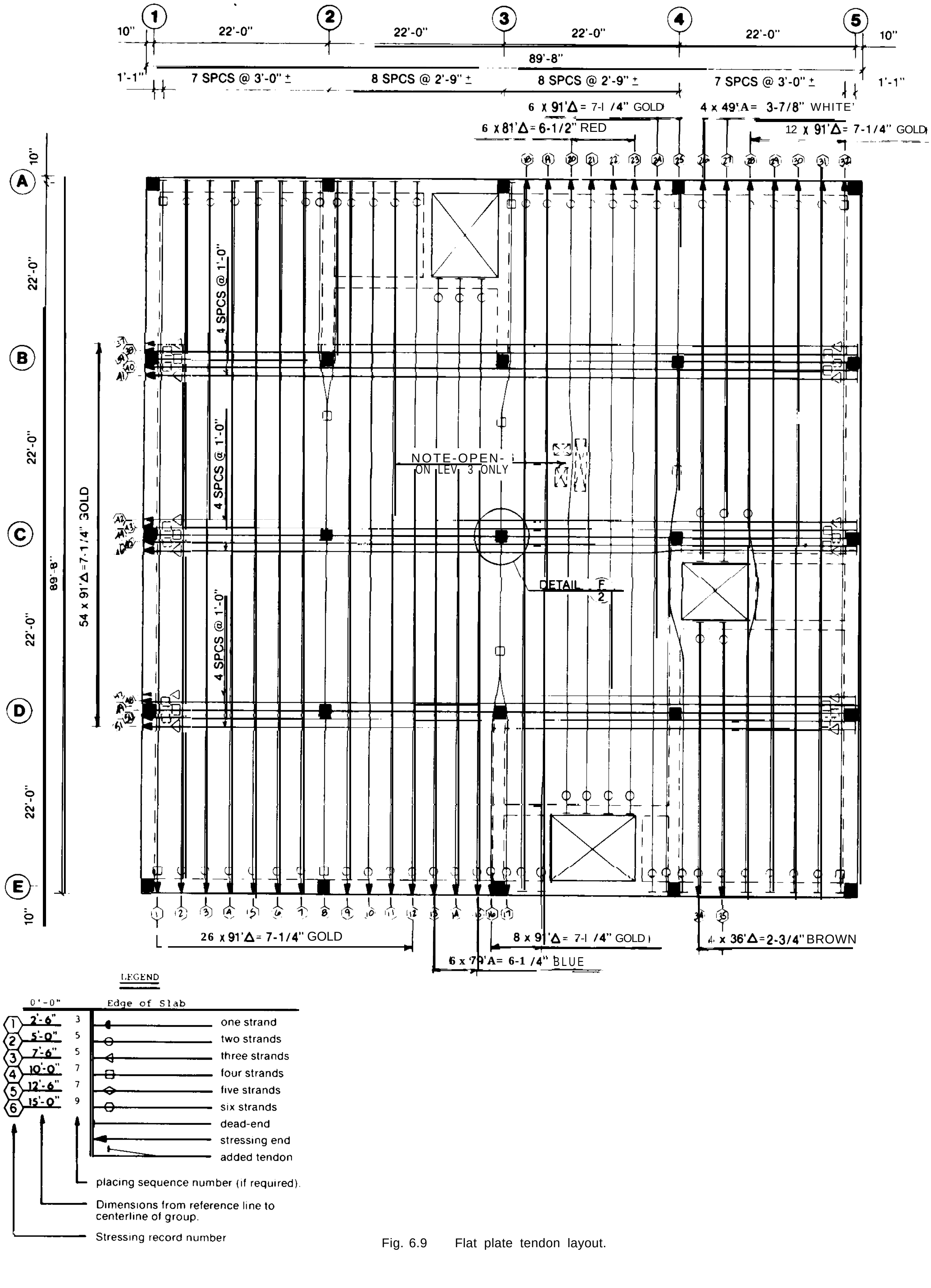


Fig. 6.9 Flat plate tendon layout.

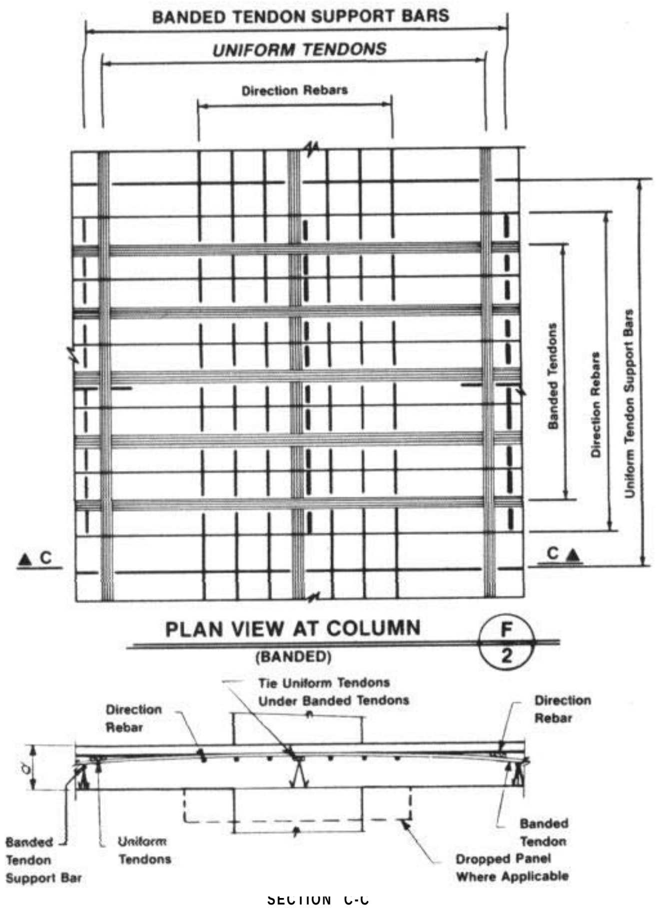


Fig. 6.10 — Flat plate reinforcement details at column

Fig. 6.11 — Flat plate banded tendon distribution.

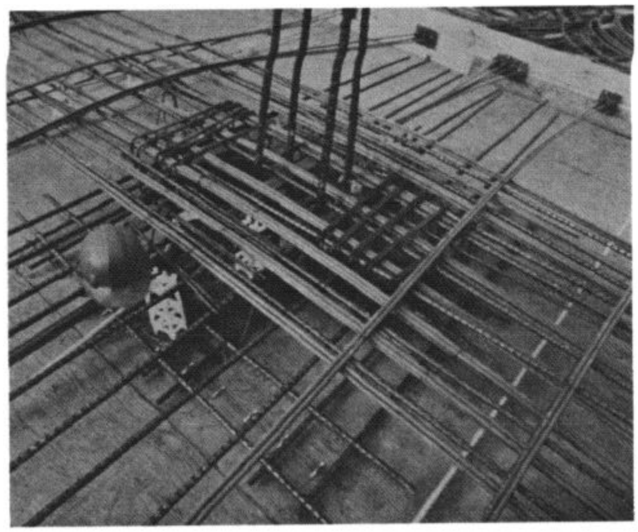


Fig. 6.12 — Arrangement of reinforcement over column of banded flat plate.

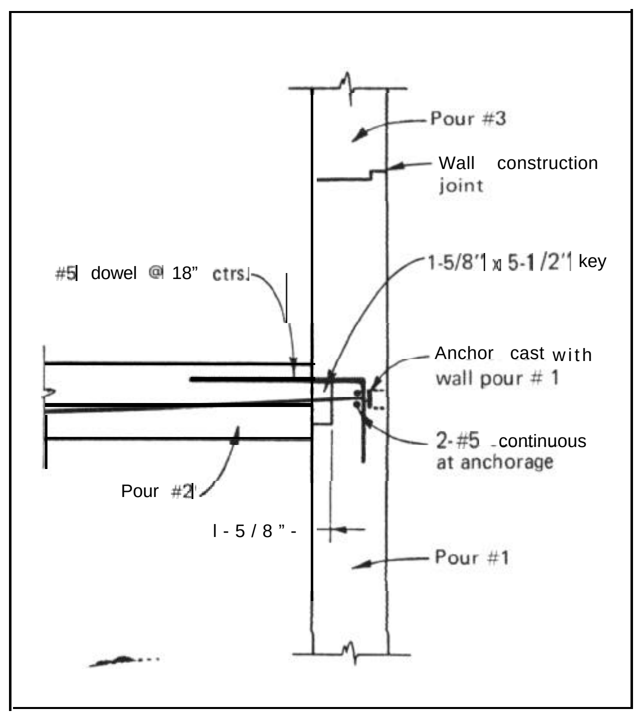


Fig. 6.13 -Two-way slab, wall to slab joist details.

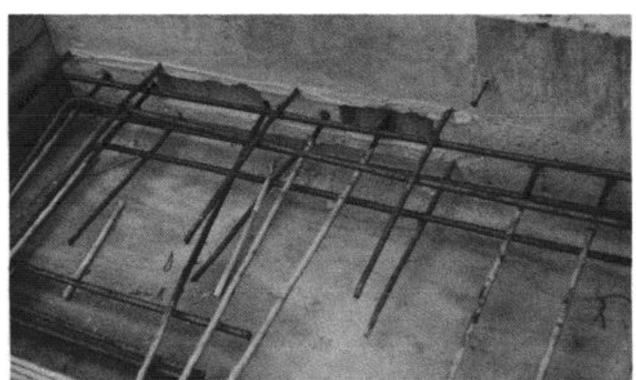


Fig. 6.14 — Details of post-tensioned joint between slab and the wall prior to placement of floor slab concrete.

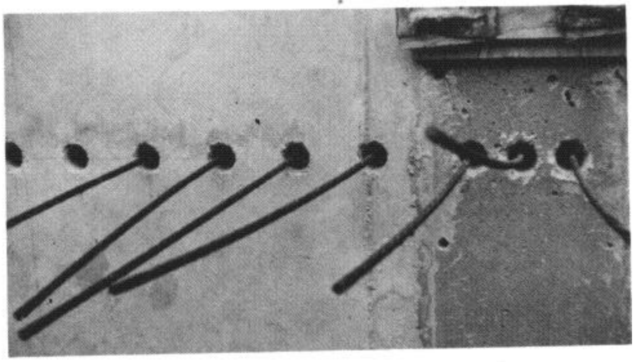


Fig 6 15 — Outside of wall prior to stressing the tendons shown in Fig 6.14

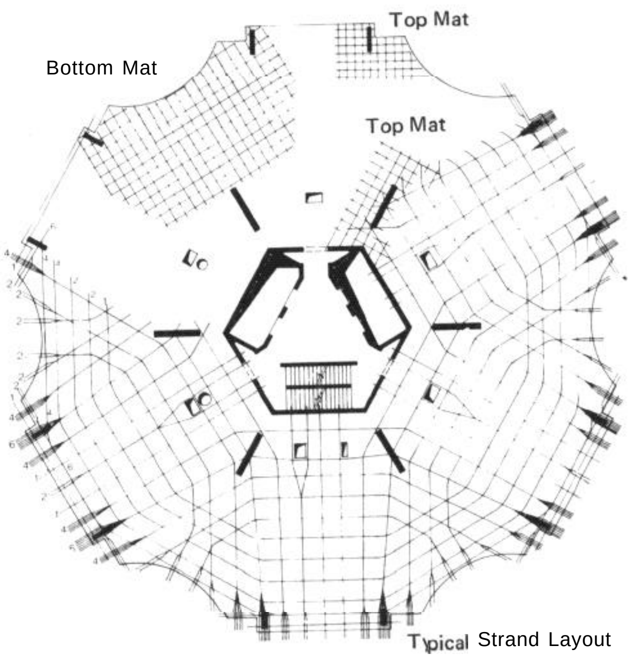


Fig. 6.16 — Layout of tendons and nonprestressed reinforcement for a typical apartment floor of the Edmonton House.

6.2.5 Two-Way Slabs With Dropped Panels

Two-way flat plates have been widely used for spans of under 20 to over 30 ft. By adding dropped panels to increase the concrete thickness in the column area, spans in the 35 to 45 ft. range may be constructed with a relatively small increase in the quantity of concrete and tendons. Elimination of interior columns and longer spans are particularly important in parking garages to increase parking space and to make the garage area more convenient and efficient. A construction photograph of a flat slab with drop panels is presented in fig. 6.78. Forming for one of thedropped panels isvisible in the background and central stress boxes are in place throughout the slab. Stressing of central stress tendons is accomplished in the block-out in the interior of the slab as opposed to stressing at the exterior facing. Central stressing eliminates edge pockets and additional forming for the stressing operation around the outside of the slab. After stressing, the central stress block-outs are filled with concrete.

Detailing of tendons and nonprestressed reinforcement for two-way slabs with dropped panels is similar to detailing for two-way flat plates.

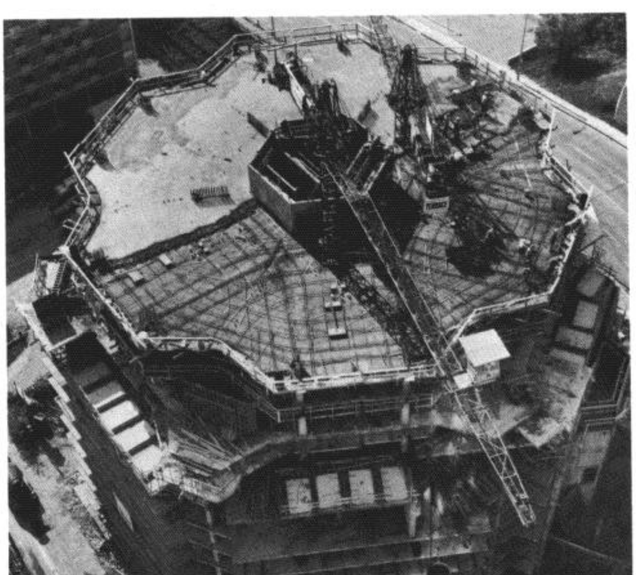


Fig 6 17 — Construction view of a typical apartment floor of the Edmonton House.

Fig. 6.18 — A flat slab with dropped panels and central stressing.

6.2.6 Two-Way Joists (Waffle Slab)

Waffle slabs or two-way joist construction may be economical for applications with spans as low as 35-ft] and as high as 65-ft] In special circumstances, even longer spans have been built with a modified (large pan size) waffle layout. In some cases waffle slabs are used for architectural purposes such as incorporation of lighting fixtures within the space created by the pans. This also provides a significant economic benefit due to reduced costs for lighting fixtures.

Detailing for a post-tensioned two-way joist building floor or roof is similar to that used for one-way joist buildings except that the tendon geometry and size must be specified for tendons in both directions of the slab. Tendon installation in a waffle slab is illustrated in fig. 6.79. ***Fig. 6.20*** presents typical details for tendon anchorage installation at a corner column of a waffle slab structure.

Fig. 6.19 — Tendons in place in a waffle slab as final preparations are made for concrete placement.

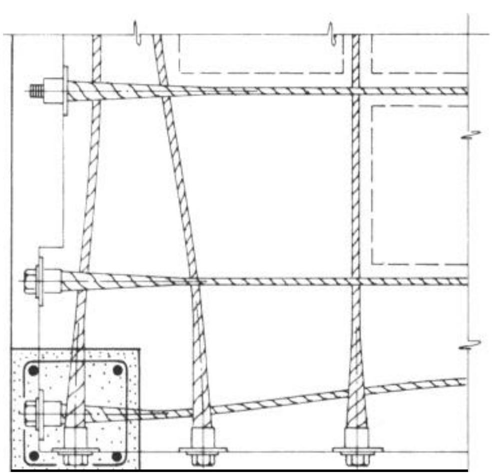


Fig. 6.20 — Typical details for tendon anchorage installation at corner column of waffle slab structure.

6.2.7 Detailing of Anchorage Zones

Design and detailing of the concrete member and reinforcement in the area of tendon anchorages requires special attention. Specifically, the following items should be investigated:

(1) Adequate bearing area must be provided for tendon anchorages which utilize bearing plates in accordance with Section 3.7.7. Some tendon anchorages described in Section 2.2 utilize rings of steel plate or spiral reinforcement to contain the concrete bursting forces and in these cases the projected concrete bearing area in front of the anchorage may be less than specified in Section 3.1.7. Details of anchorages presented in Section 2.2 will generally provide adequate bearing area. However, special anchor plate sizes or details may be required in some circumstances; for example, when tendons are stressed at relatively low concrete strengths, or where lightweight aggregate concretes are used.

(2) Horizontal and vertical reinforcement should be placed in front of the anchor plate to resist bursting and spalling stresses in the concrete. The amount and location of this reinforcement depends on the size, number and location of anchor plates with respect to size of the concrete section, and other variables. Often nominal reinforcement will suffice for bursting stresses. In other cases, detailed design of anchorage zone reinforcement will be required. Procedures for design of bursting reinforcement are discussed in Section 5.4: 7]

(3) Adequate space and clearance must be provided for anchors, flared ends of conduit, access for stressing equipment, and other post-tensioning system details. Particular care must be given to detailing of spacing and location of post-tensioning tendons, anchorages, and non-prestressed reinforcement to avoid interference at beam-column, beam-girder, and beam-slab intersections. The use of bundled column reinforcement as shown in ***Fig, 6.27*** may be particularly helpful at main beam-column joints. Note also in ***Fig. 6.27*** the additional horizontal ties and vertical reinforcement in front of the anchor plate which may be proportioned by the shear-friction concept to assure that the forces related to the ultimate (factored) column-beam moment can be transmitted between the beam and column.

(4) Reinforcement details in anchorage zone areas should be reviewed to assure that sufficient space is provided to permit careful place-

ment and vibration of concrete behind anchorage plates. Voids or honey-combing behind anchor plates resulting from overly congested reinforcement details, or lack of vibration, can result in movement of the anchor plate into the member at the time of stressing. This in turn could cause a more serious failure, or, at a minimum, necessitate repair and delay in construction.

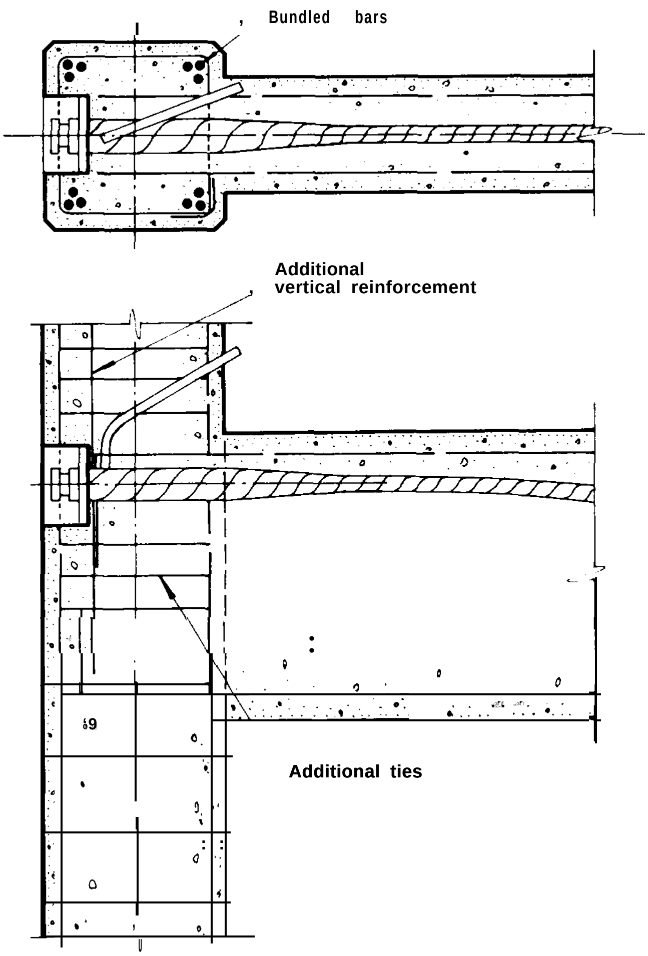


Fig. 6.21 — Typical reinforcement in anchorage zones

6.2.8 Detailing To Avoid or Minimize Development of Restraint to Dimensional Changes

A primary design rule for engineers working with prestressed concrete is to make provision for movement of the structure in the direction of stressing at the time the prestress force is applied. Additional movement in thedirection of stressing will result from plastic deformation or creep of concrete with time, as well as from shrinkage and thermal volume changes, and these movements must also be considered in detailing connections between flexural elements and supporting columns or walls. When a post-tensioned beam is framed monolithically into relatively stiff columns, the elastic, shrinkage and creep shortening of the beam will cause high moments and shears in the columns. Furthermore, the prestress force in the beam is reduced by the amount of force that is required to bend and translate the column sufficiently to accommodate the beam shortening. In multi-story and multi-bay frames, the effects of shortening of flexural elements are distributed throughout the frame.

The restraints due to dimensional changes can be accommodated in the following ways:

(1) Design or locate supporting elements to minimize restraint. Relatively long flexible columns may reduce restraint forces to the point where they can be easily accommodated by column reinforcement. Lateral load resisting elements can often be located near the center of movement so that no restraint develops.

(2) Segment the structure with pour strips or temporary joints to minimize the movement and restraint developed during post-tensioning and due to early volume changes.

(3) Detail the connection between the flexural elements and columns to permit movement. Joint details to permit movement between a slab and a stairwell or elevator core are shown in fig. 6.22. Fig. 6.23 shows two examples of joint details for concrete slab construction that provide for rotation and displacement during post-tensioning and a later structural connection. The joint details shown in Fig. 6.23 may reduce the shear capacity somewhat. Such joints require careful detailing, and the possible reduction of shear strength should be considered during the design stage.

In applications of post-tensioning such as one-way slabs and two-way flat plates, the prestressing stress is relatively small. For example, average stresses due to post-tensioning in these applications may be on the order of 150 psi or even somewhat less. Stresses of this magnitude do not produce large dimensional changes in normal sized buildings due to elastic shortening or due to concrete creep. In such cases, no special details may be required to minimize restraint forces. However, even in these applications, care should be exercised when the building dimensions, or thedimensions between joints become large, or when the flexural elements are supported by rigid elements which could produce substantial restraint forces if not properly detailed.

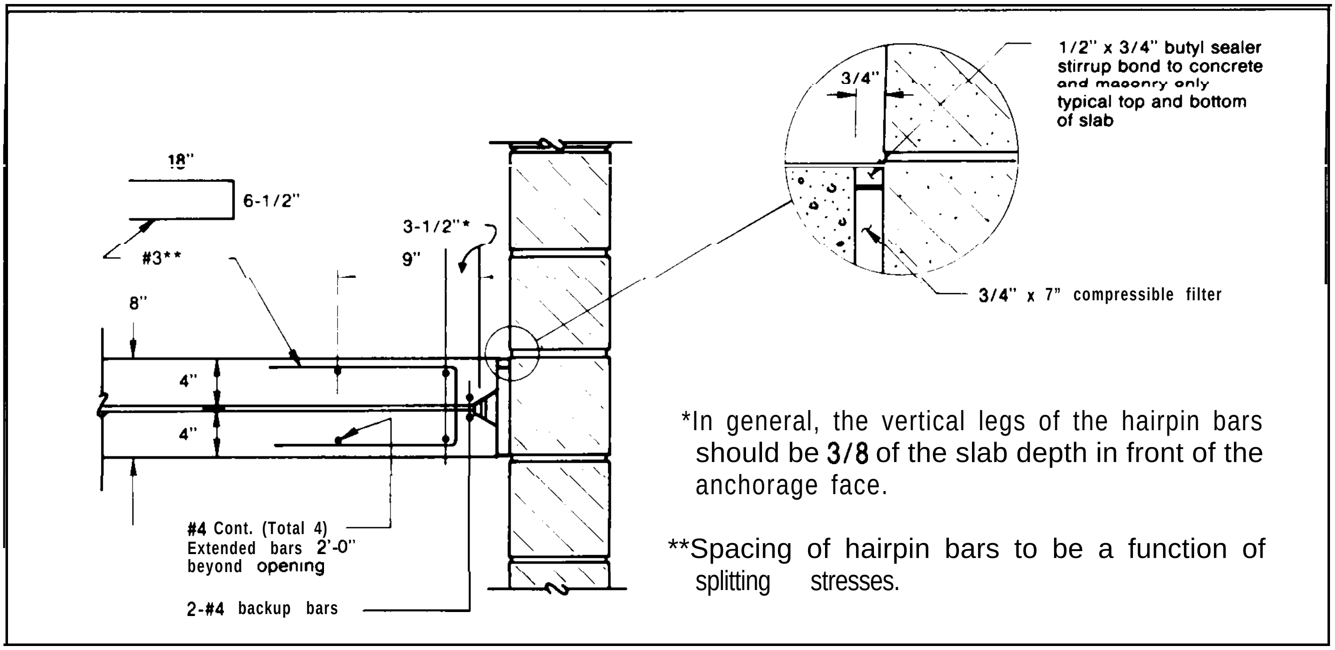


Fig. 6.22 — Details for separation of slab from elevator core or stairwell

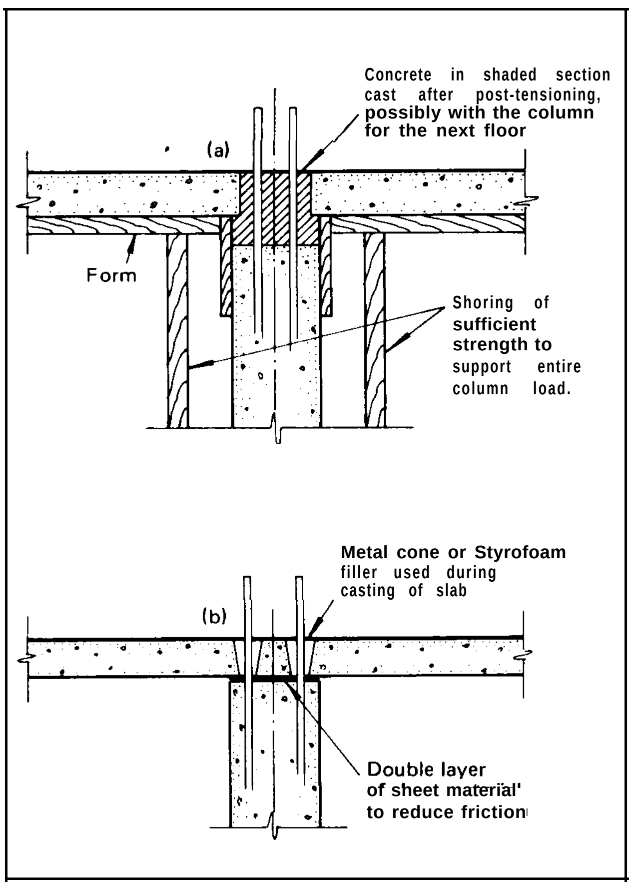


Fig. 6.23 — Examples of joint details for concrete floor construction.

6.3 CONSTRUCTION

6.3.1 Formwork

The forming system is a major factor in determining the economy and construction speed of cast-in-place post-tensioned construction. A five 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. This construction rate can be further reduced by use of precast bearing walls or precast columns. fig. 6.24 shows the flying forms used for the one-way post-tensioned slabs of the Roosevelt Island project in New York City. Flying forms have been developed to accommodate complicated structural geometries 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 dimension lumber forming, economy is achieved through repetition, simplicity of details, use of reasonable sized shapes, and provisions for easy installation and removal of bracing.

Forms should be drilled to receive tendon stressing hardwareand 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 deyond the tendon terminal to provide space for the stressing operations.

6.3.2 Tendon Placing

There are two methods of placing tendons. First, pre-assembled tendons may be placed as a unit prior to placing concrete. Second, bearing plates and duct sheathing may be installed prior to placing the concrete, and then after concreting, the prestressing steel and anchorages may be installed. Supporting ties for pre-placed tendons must be adequate to support the tendon weight. When only the duct is placed prior to concreting, ties must resist buoyancy forces.

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. Tendons are usually placed before reinforcing steel, electrical conduit, and mechanical work.

Pre-placed 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 duct tape.

The placing sequence number for flat plate 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 tehdon. After all tendons marked for the initial placing sequence have been uncoiled in their approximate position, tendons with the second placing sequence num-

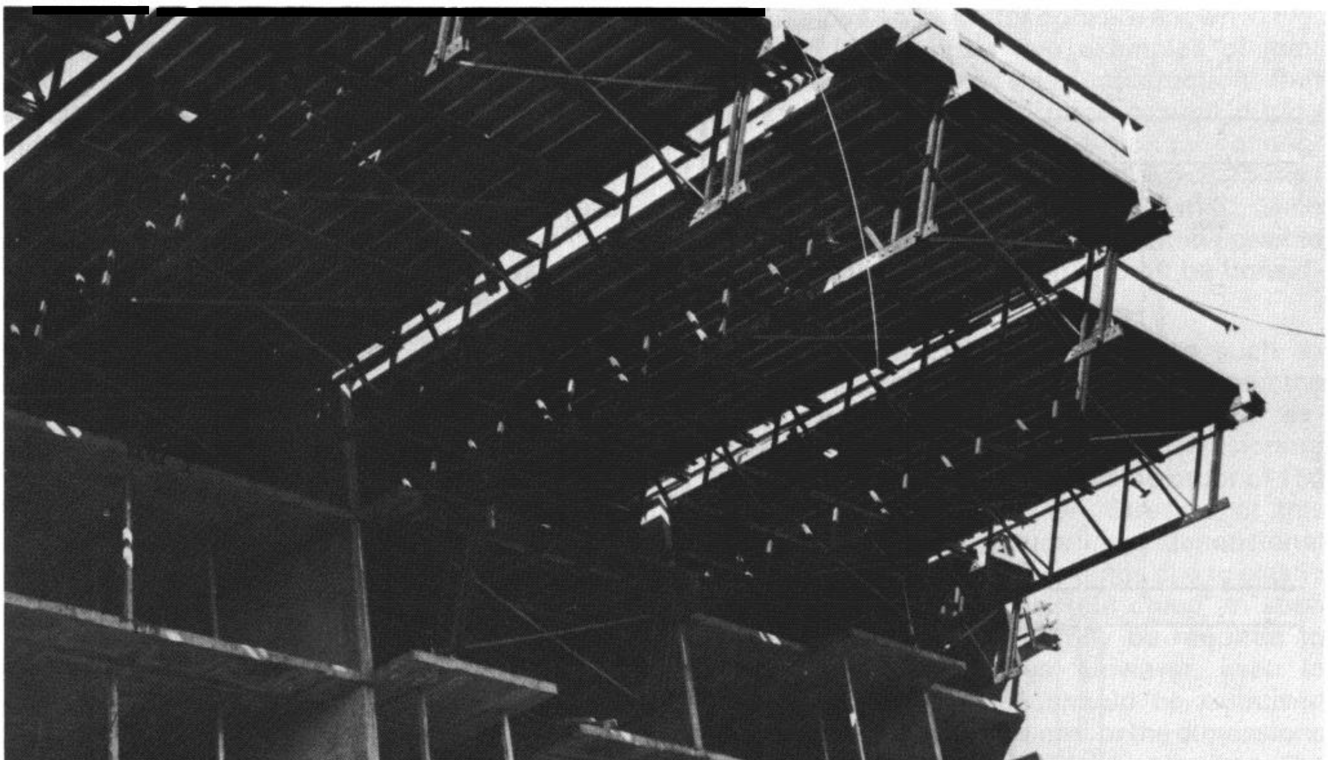


Fig 6 24 — Flying forms used for the Roosevelt Island project ber may be placed as described above. Remaining tendons should be placed in numerical sequence.

The structural engineer has determined the exact tendon location by careful calculations. Vertical deviations in tendon location should be kept to about 1/4 in. for slab thickness dimensions less than 8 in., and 3/8 in. to 1/2 in. for thicker slabs or beams. Horizontal plane deviations which may be necessary to avoid openings, ducts, chases, inserts, etc., should have a radius of curvature of not less than 21 ft.

Concrete cover between tendons and openings in slabs should normally be at least 6 in. Nonprestressed reinforcing steel may be required to control high concrete stresses near voids.

Tendon profiles are maintained by tying to reinforcing steel, chairs or other supports with wire ties. Small tendons (such as single strand) should be supported at about 4 ft. centers. Supports for large tendons which naturally maintain an approximately correct profile due to the stiffness of the tendon should be 10 ft. or less, as required by the tendon stiffness.

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, do not substitute, consult your post-tensioning materials fabricator.

With the "pull through" method, only a semi-rigid thin wall tubing is initially placed in the forms. Prestressing steel is cut to length at the site and pulled into the sheathing just prior to stressing. This method of installation was developed in the late 1960's in California and has subsequently found wide acceptance throughout thecountryfor multi-strand tendons because of its inherent advantages. The most important advantages of this method are -the greatly reduced friction losses due to the reduction of wobble in semi-rigid tubing — labor for installation is substantially reduced — eliminates the possibility of corrosion since prestressing steel is installed just prior to stressing — and, practically no limitation with respect to capacity and length of a single tendon.

Bearing plates and trumpet assemblies are installed prior to the reinforcing steel when the "pull through" method of tendon installation is used. Bearing plates are bolted to blockouts and shimmed so that they are located perpendicular to the tendon axis.

The tubing or duct to form the opening for the post-tensioning tendon is normally installed subsequent to placement of the reinforcing steel. It is made from either 24 or 26 gauge black or galvanized sheet metal strips formed spirally into diameters varying from 2 to 6 in. It is furnished in lengths of 20, 30 or 40 ft. and can be coupled in place with 18 in. long couplers of 1/8 in. larger diameter. Photographs of the tubing are presented in *Section* 2.2. Tubing location should not vary from the location shown on the contract plans by more than 1/4 in. at high and low points of the tendon profile. When the tendon is near the neutral axis of the member, variations of tendon location of up to 1 in. are acceptable. The tubing is normally tied with tie-wire to the stirrups at maximum spacings of about 6 ft. Ties must be sufficient to prevent the tubing from floating during placement of concrete.

After the concrete has sufficiently cured, prestressing steel in 12,000 ft. packs or coils is delivered to the site. The tendons are then pulled out of the packs, cut to length, and a "Kellums grip" is fitted over the end of the tendon. A tugger is attached to the Kellums grip to pull the tendon into the structure. Anchorages are then placed and the tendon is stressed. This process is illustrated for various systems in *Section 2.2.*

Consult ram clearance charts in *Section 2.2* for clear distance required for stressing equipment.

When welding or burning near tendons, care must be exercised to prevent the tendon from overheating, to keep molten slag from coming in contact with the tendon. Grounding of welding equipment to the tendon should not be allowed.

6.3.3 Concrete Placement

The American Concrete Institute's "Recommended Practice for Measuring, Mixing and Placing Concrete" 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. Much 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. CALCIUM CHLORIDE SHOULD NOT BE

USED IN CONCRETE FOR POST-TENSIONED CONSTRUCTION.

Prior to placing concrete, tendon profiles should be checked at critical locations such as midspan, inflection points, and in negative moment areas over columns or walls by measuring from the form soffit to the center of the tendon diameter. 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 insure 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 insure uniform compaction at these points. Voids behind the bearing plate, or insufficient concrete strength will cause concrete failure. Careful vibration and propercuring will eliminate most such difficulty. Voids behind the bearing plate should be repaired prior to the stressing operation.

Curing in accordance with American Concrete Institute recommendations should be followed to insure proper concrete strength.

6.3.4 Stressing Operations

When the stressing is not performed by the post-tensioning materials fabricator, all necessarystressing equipment is usually furnished by the post-tensioning materials fabricator and delivered to the job site according to prearranged schedule.

When tests of field-cured cylinders indicate that the concrete has reached the proper strength (usually 60-80 percent of the 28-day strength) the stressing operation may begin. IT IS ESSENTIAL THAT THE SHORING BE LEFT IN PLACE UNTIL THE STRESSING IS COMPLETED.

Tendons should be stressed only when proper data and experienced personnel are present. Refer to the stressing data chart for proper ram area, forces and gauge readings for each tendon.

Before beginning the stressing operation, make sure ‘that sufficient power is available, that the source is close enough, and/or the line is large enough to ensure that no appreciable power drop will occur.

The post-tensioning materials fabricator will supply simple rules and procedures to follow to insure that stressing is accomplished in a satisfactory manner.

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 materials fabricator. Second, the theoretical elongation of the tendon can be calculated using the formula:

$$\text{Elongation} = \frac{PL}{AE}$$

where
- P = Prestress force in pounds
- L = Length of tendon in inches
- A = Area of steel in square inches
- E = Modulus of elasticity of prestressing steel, psi

The modulus of elasticity of various post-tensioning tendon materials may be assumed as follows:

Seven-Wire Strand;	E = 28,000,000 psi
Wire;	E = 29,000,000 psi
Bars;	E = 29,000,000 psi

It is generally required that the tendon force measured by gauge pressureagree witin about 5 percent with the tendon force calculated by elongation measurements using the above formula. The modulus of elasticity of seven-wire strand varies somewhat from the 28,000,000 psi averge value suggested. Since a variation of 1 ,000,000 psi in the modulus of elasticity represents a difference of about 4 percent 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 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 back of or 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 exist, (the bearing plate begins to recede into concrete), release all pressure on the equipment at once; remove the faulty concrete and patch the void with suitable material that will attain the required strength before attempting to restress the tendon. Calcium chloride or admixtures containing calcium chloride should not be used in any patching operation.

Should equipment become inoperative or malfunction and a technical representative is not immediately available, any reputable hydraulic equipment repair shop can usually repair the equipment. ,

Exercise care in operating the pump to retract the ram. Do not allow pressure build-up on the return side, as this may blow the ram seal.

6.3.5 Inspection Guidelines

Three distinct construction phases are involved for all post-tensioning systems:

- Material manufacturing
- Tendon installation
- Tendon stressing

Table 6.1 lists important questions that should beconsidered by inspectors about each of these phases for both bonded (grouted) systems and unbonded (greased and sheathed) tendons.

Material manufacturing: Most fabrication plants have similar production facilities. Depending upon the magnitude of the project, plant inspection may be appropriate. If not, then jobsite material review is in order.

Tendon installation: An experienced inspector should review the process with the placer during installation of the first pour, and reach an understanding with the crew regarding critical elements.

Tendon stressing: Jobsite technical instruction on the proper operation of stressing equipment is normally provided in accordance with project specifications by the post-tensioning material fabricator. The inspector must be familiar with the various operations involved in stressing tendons.

6.3.6 Grouting

See Section 3.3 for recommended grouting procedures.

6.3.7 Form Removal and Reshoring

Shoring must be left in place until the stressing operation is completed.

Bolts at end anchorages should be removed within 24 hours after concrete is placed, otherwise corrosion will cause bolts to “freeze”.

Edge or pocket forms, and bulkheads should be removed well ahead of stressing operation.

Beam or side forms may be removed prior to stressing with permission from the engineer.

Removal of shoring and forms may follow immediately after the stressing operation. After stressing, reshoring may be required to prevent overloading during additional construction. Usually reshoring at midspan will suffice, however, standard construction reshoring practices are a precaution against overloading. Do not wedge shoring tightly against prestressed members.

6.3.6 Protection of End Anchorages

To insure neat placing of cover concrete in recesses, pockets, or edge strips, fit forms securely against the previously placed concrete.

It is essential that all exposed end anchorages and/or wires are protected by some adequate means, such as epoxy coating, asphalt base paint, mastic and/or concrete cover. If non-shrinking or expanding grout or concrete is required, the additive must be compatible with prestressing steel. Place cover concrete with as low a slump as possible to avoid excessive shrinkage. Vibrate well, or dry pack, to insure compaction around anchorage. DO NOT USE CALCIUM CHLORIDE IN CONCRETEOR MORTAR USED TO PROTECT END ANCHORAGES.

Table 6.1

GUIDELINES FOR INSPECTION OF POST-TENSIONING*

PHASE	UNBONDED TENDONS	BONDED TENDONS
MATERIAL MANUFACTURING	Are fixed end wedges evenly and adequately seated in the anchor? Is excessive sheathing stripped at the fixed end? Is the plastic sheathing of sufficient and uniform thickness? Is the grease evenly applied and of consistent texture? Does the strand appear to be of new quality, free of corrosion when sheathing and grease are removed? Are the anchors properly cast with smooth wedge holes? Are the wedges free of rust and steel shavings, and of consistent quality? Are mill reports and certifications available for the prestressing steel and other components, as required by the specifications?	Are the anchor heads properly machined, cleaned, and protected from corrosion? Are the wedges or threaded nuts free of rust and steel shavings, and of consistent quality? Is the duct manufactured from quality steel strip with specified thickness and watertight seams? Is the bare prestressing steel free of corrosion and debonding contaminants, and adequately protected during storage?
TENDON INSTALLATION	Are the tendon high and low points at the correct elevation? Are the tendon profiles smooth and correctly shaped (parabolic, circular, or straight) between reference points? Do the tendons have excessive horizontal wobble? Is the sheathing damaged, and if so, has it been repaired? Does the chair or support-bar system conform to contract documents? Are the stressing anchors securely fastened to the form with appropriate pocket formers? Is bursting steel installed behind the anchorages as required by the contract documents7 Has the method of concrete placement been reviewed as to its effect on tendon stability during placement? Has the conventional steel placement been reviewed?	Are high and low points of the center of duct at the correct elevation? Are duct profiles smooth and correctly shaped (parabolic, circular, or straight) between reference points? Are all duct joints properly mated and sealed with duct tape? Are there any holes in the duct, and if so, have they been repaired to prevent concrete intrusion? Are there any kinks in the duct which will prevent prestressing steel installation? Is the support system adequately tied to prevent displacement and floating of the duct during concrete placement? Are the bearing plates securely fastened to the form blockouts? Is bursting steel installed behind the anchorages as required by the contract documents? Has the method of concrete placement been reviewed as to its effect on duct stability during placement? Has the conventional steel placement been reviewed?
TENDON STRESSING	Are the stressing anchor wedge holes free of grout, dirt and plastic? Is a consistent dimension used for the elongation datum mark on the strand? Is the stressing equipment well maintained, and are calibration charts available? Is the stressing ram operator careful with the equipment and consistent from tendon to tendon? Are the tendons stressed slowly enough to allow the strand to overcome as much friction as possible prior to seating? Are the wedges seated evenly and under pressure? After elongation approval, are the tendon tails cut off well inside the pocket to allow proper grout cover? Are pocket surfaces sufficiently clean to allow good grout bond during and after patching?	Are the anchor heads, wedges and nuts free of corrosion, dirt and grease? Has the elongation datum mark for the Initial and final reading been logically and clearly located? Is the stressing equipment well maintained, and are calibration charts available? Is the stressing ram operator careful with equipment and consistent from tendon to tendon? Are the wedges, shims, or nuts properly seated after stressing? Are the tendon ends and stressing pockets properly prepared for patching?

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6.3.9 Slab Penatrations and Openings

To the fullest possible extent, 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. In anticipation of possible future slab penetration, it is very desirable that the tendon location be marked on the bottom of the concrete slab as illustrated in Fig. 6.25. In this case the concrete was marked by spraying the forms with paint along the tendon lines just prior to placement of concrete. Enough of the paint 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. If such a procedure is not followed, it is necessary to use a metal detector such as the James R-Meter (available from James Instruments, Inc., Chicago, Illinois) to locate tendons prior to cutting into the slab.

In some cases, it is necessary to make large openings in post-tensioned floors following construction. In the case of the Western Canadian Place Towers described in Section 1.4.3, it was necessary to provide stair openings to interconnect floors at 10 separate floor levels following completion of the project. 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. Interior designers were asked only to avoid beam locations and end spans when locating stair openings, and that did not unduly restrict their planning. The process of constructing the opening is illustrated in figs. 6.26, 6.27, and 6.28. 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. 6.26.** The metal clamps were used to provide a slow release of the tendon force after the tendons were cut. This process permitted detensioning of all tendons through the stairwell openings with no damage to the glazing which had already been installed around the perimeter of the buildings. Tendon anchorages were then reset as shown in Fig. 6.27, and after concrete placed in the area in front of the anchor reached the necessary strength, the tendons were restressed at the perimeter of the opening. Finally, forms were installed as shown in fig. 6.28, and concrete was placed around the forms to finish the openings.

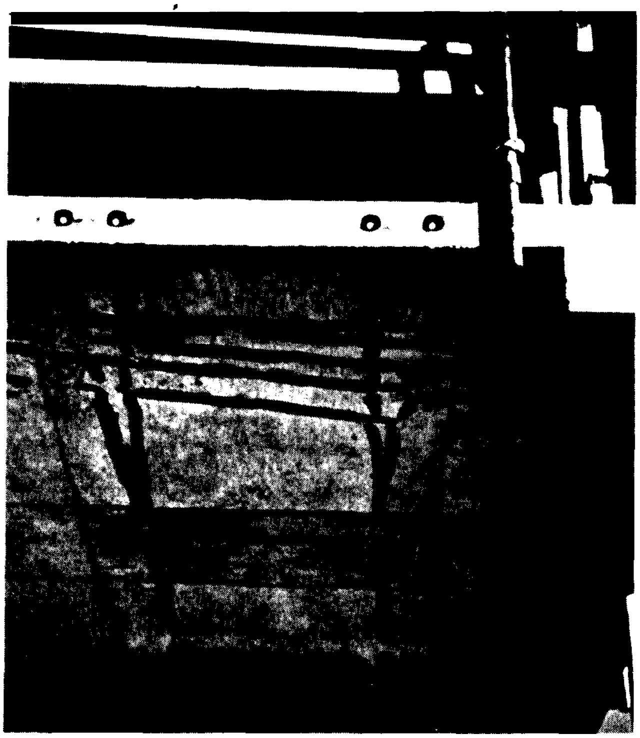


Fig. 6.25 Tendon locations marked on bottom of slab by use of spray paint on the forms prior to concrete placement.

Fig. 6.26 — Procedure for detensioning tendons at stairwell openings, Western Canadian Place.

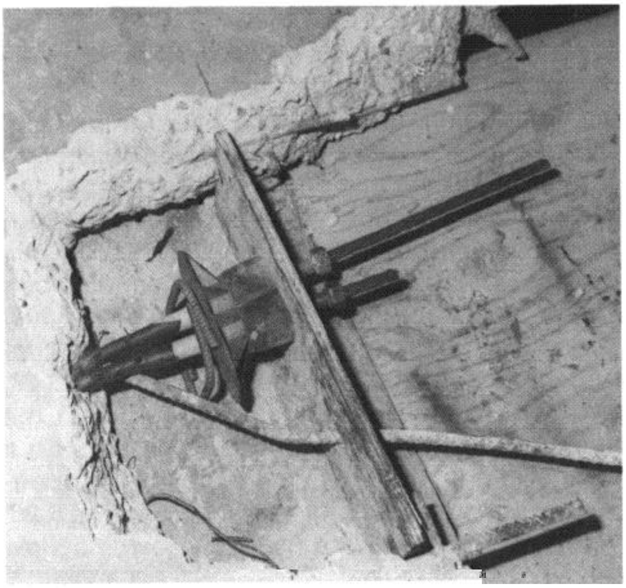


Fig. 6.27 — Anchorage reset at perimeter of opening ready for concrete placement.

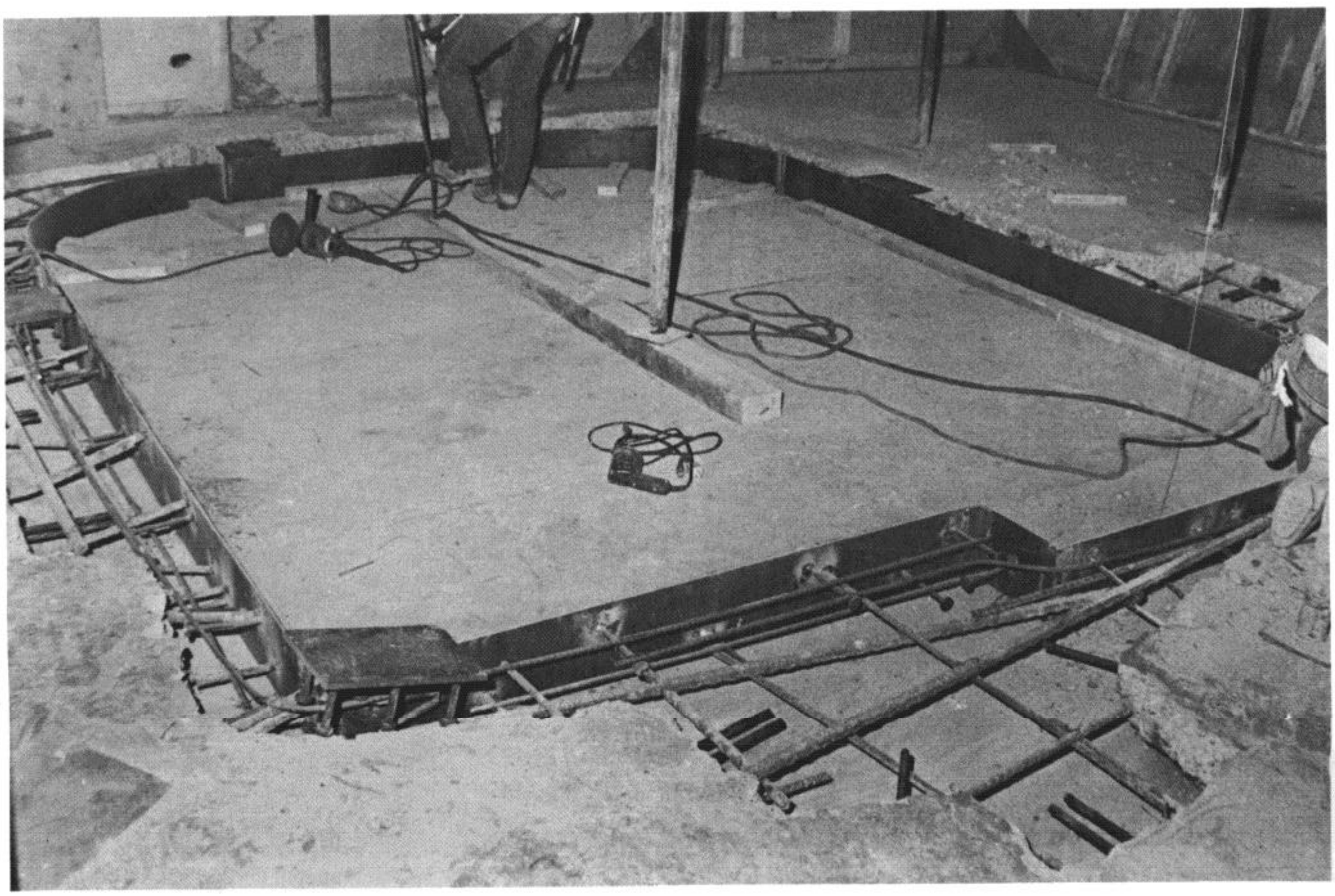


Fig. 6.28 — Forming for stairwell opening in place.

Chapter 1

Fire Resistance of Post-Tensioned Structures

7.1 INTRODUCTION

More than 140 full-scale fire tests of prestressed concrete structural components have been conducted in the United States. In addition, researchers have studied the properties of steel and concrete subjected to high temperatures. They have also measured and calculated analytically the temperatures that occur in structural components during fires. This physical and analytical research provides the basis for structural detailing recommendations for achieving various fire endurance ratings, and has more recently made possible the development of rational design procedures for determining the fire endurance of prestressed concrete structures.

7.1.1 Scope

The purpose of this chapter is to present an overview of pertinent information concerning the fire endurance of structures with post-tensioned reinforcement. Information has been gathered from a number of sources. Results of fire tests of 18 slabs and beams with post-tensioned reinforcement constitute a major source. These results were compared with other information, i.e., tendon temperature data were compared with data on temperatures within unreinforced slabs and beams, and then the resulting fire endurances were analyzed. Because the temperatures of the tendons were in all cases cooler than temperatures at comparable locations in unreinforced slabs, it was possible to make conservative recommendations on minimum dimensions for various fire endurances. In addition, results of high temperature tests of tendon-anchor assemblies make it possible to determine realistic cover thickness for anchors.

As an alternative to achieving fire endurance through the specified amount of concrete cover or use of an undercoat or ceiling, rational design procedures may now be utilized to evaluate the fire endurance of structural systems with varying amounts and locations of reinforcement. These procedures make it possible to analyze a number of options for obtaining a specified period of fire endurance, and the best solution based on cost and other considerations can then be selected.

Rational design procedures have been developed from analyses of fire tests. The procedures utilize basic data on the strength-temperature relationships for steel and concrete along with information on temperatures within concrete beams and slabs during standard fire tests. Rational design procedures are discussed further in Section 7.7, which includes example calculations of fire endurance of a post-tensioned flat plate floor.

7.1.2 Standard Fire Tests of Building Construction and Materials (ASTM E119) (1)*

The fire resistive properties of building components are measured and specified 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 1000F, at 30 minutes 1550F, at 1 hr 1700F, at 2 hrs 1850F, and at 4 hrs 2000F. The fire represents combustion of about 10 lb of wood (with a heat potential of 8,000 BTU per lb) per sq ft of exposure area per hr of test. Actually, the fuel consumed during afire test is dependent on the furnace design and on the heat capacity of the test assembly. For example, the amount of fuel consumed during a fire test of an exposed concrete floor specimen is likely to be 10 to 20 percent greater than that used for a test of a floor with an insulated ceiling, and considerably greater than that for a combustible assembly.

The standard, ASTM E119, 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, either loadbearing or non-bearing, the minimum specified area is 100 sq ft with neither dimension less than 9 ft, while for beams it is 12 ft.

During fire tests of floors, roofs, beams, loadbearing 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.

'Numbers in raised parentheses designate references listed in Section 7.9

End point criteria for floors and roofs are:

(a) Specimens must sustain the applied loading-collapse is an obvious end point.
(b) Holes, cracks, or fissures through which flames or gases hot enough to ignite cotton waste must not form.
(c) The temperature of the unexposed surface must not rise an average of 250F or a maximum of 325F at ony one point.
(d) For unrestrained assemblies, based on fire tests of restrained assemblies, the temperature of the steel must not exceed 1100F for structural steel or reinforcing bars or 800F for cold-drawn prestressing steel.
(e) For restrained beams spaced more than 4 ft on centers, the temperatures given in (d), above, must not be exceeded for the first half of the fire endurance period. For restrained beams spaced 4 ft or less on centers and for restrained slabs, the steel temperatures are disregarded.

"Restrained" in this case means that thermal expansion of a specimen is restricted during a fire test. Two classifications can be derived from fire tests of restrained specimens, "unrestrained" and "restrained". Only "unrestrained" assembly classifications can be obtained from tests of unrestrained specimens, in which case there is no limit on the steel temperature. The most recent versions of ASTM E119 include a guide for classifying construction as restrained or unrestrained. The guide indicates that either restraint to thermal expansion or continuity restraint results in greatly improved fire endurance and that nearly all cast-in-place concrete constructions are considered to be restrained.

7.2 PROPERTIES OF STEEL AND CONCRETE AT HIGH TEMPERATURES

Physical properties of steel and concrete are affected by the temperatures encountered in fires. Strength, modulus of elasticity, expansion, thermal conductivity, creep, stress relaxation, etc., are all affected to some degree. Insofar as ultimate capacity during fires is concerned, strength is of primary importance.

7.2.1 Steel Strength at High Temperatures

Fig. 7.7 shows typical relationships between temperature and strength for cold-drawn prestressing steel, i.e., wire or strand; hot-rolled steel, i.e., reinforcing bars; and high strength alloy steel bars. [5,6,7] Note that half of the strengths are retained at about 800Ffor cold-drawn steel, 1050F for alloy steel bars, and 1120 for hot-rolled steel.

7.2.2 Concrete Strength at High Temperatures

Fig. 7.2 shows the temperature strength relationships for three kinds of concrete. [8] Carbonate aggregates include limestoneand dolomite which undergo a chemical change at temperatures above about 1300F, i.e., carbon dioxide is given off from the calcium and magnesium carbonates. Heat is used up during the reaction so the temperatures within the concrete remain somewhat lower than for non-carbonate aggregates. Also, the resulting products are better insulators than the original aggregates. Siliceous aggregates include quartzite, granite, sandstone, etc. The data for sanded lightweight concrete shown in *Fig.* 7.2 represents concretes with a unit weight in the range of 105 to 115 pcf. Note that at 800F, concrete retains most of its original strength and at 1200F carbonate and lightweight concrete have nearly all of their original strengths. Siliceous aggregate concrete retains more than half its initial strength at 1200F.

7.3 RESULTS OF 18 STANDARD FIRE TESTS CONDUCTED IN THE UNITED STATES

Data have been published from a total of 18 fire tests conducted in the United States on post-tensioned prestressed concrete slabs and beams. Two fire tests of slabs were conducted by the Fire prevention Research Institute in Gardena, California. [9, 10] Underwriters Laboratories, Inc., Northbrook, Illinois, conducted three tests of post-tensioned specimens; one was a slab [11] and two were inverted tee beams. [12] The Portland Cement Association fire tested seven post-tensioned beams, [7] all of which were modified tee beams spanning 40 ft. Six tests conducted by the National Bureau of Standards in 1953 are of historical

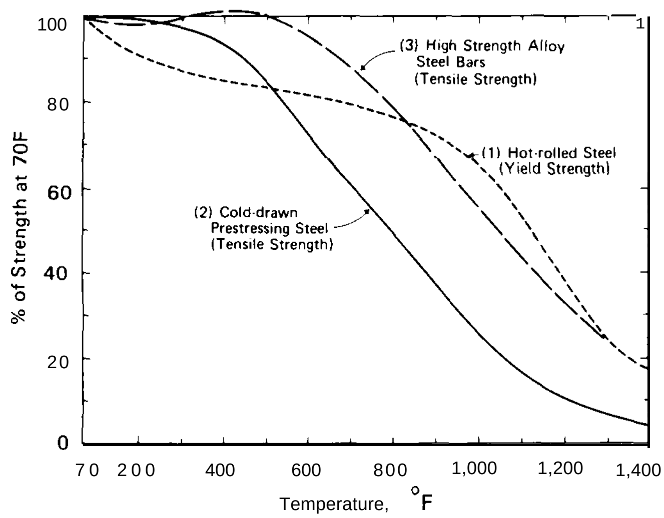


Fig. 7.1 — Temperature-strength relationships for hot-rolled, cold-drawn, and high strength alloy steels. (Curves 1. 2. and 3 from References 5, 6. and 7. respectively.)

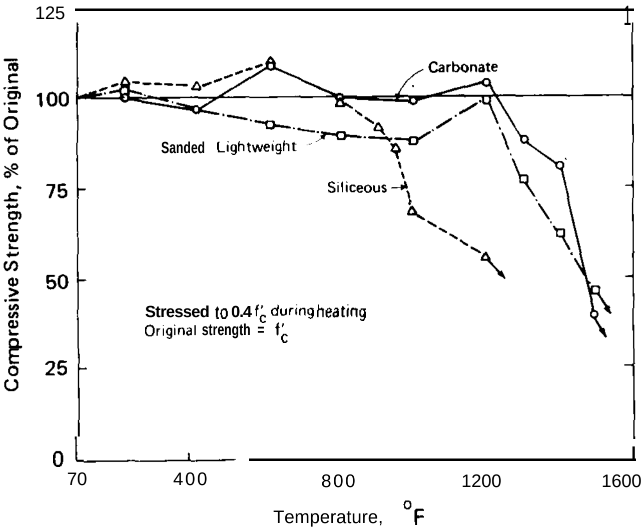


Fig. 7.2 — Compressive strength of concrete at high temperature. (Reference 8)

interest,[13] and were part of a series sponsored by the British Joint Fire Research Organization and the Building Research Station. Notes on each of the eighteen tests are included in Section 7.8.

7.3.1 FPRI Tests

Section 7.8.1 and 7.8.2 give pertinent data about these tests which are described in more detail in References 9 and 10. Both tests involved normal weight concrete slabs, 6 in. thick, made with siliceous aggregates and post-tensioned unbounded tendons. One of the specimens was an integral beam-and-slab assembly; the other was a flat plate floor. The beams were prestressed longitudinally and the slab was prestressed transversely. The minimum clear cover was 1-1/2 in. for the slab tendons and 2 in. for the beam tendons. In the other specimen, the 6-in. slab was prestressed with post-tensioned tendons in two directions. The minimum cover at midspan was 1-1/2 in.

Both assemblies were mounted in fixed restraining frames during the fire tests. Structural end points were not reached during the tests which lasted more than 4 and 3 hours, respectively. The end point for the first test occured at 3 hr 51 min when the unexposed surface temperature rose an average of 25OF. Although the second test was stopped before an end point was reached, the heat transmission end point would have been reached at about 3 hr 15 min.

7.3.2 UL Tests

Section 7.8.3 and Reference 11 give pertinent details of the fire test of a lightweight concrete post-tensioned flat plate floor. Duration of the test was 3 hr 45 min with no end point occuring. The specimen had been dried for seven months at high temperatures prior to the test and the moisture content of the concrete was low. Based on the correction procedure for non-standard moisture content (Appendix A5 of ASTM E119-83), the heat transmission end point would have occured at about 4 hr 40 min. No spalling of the specimen occurred.

Sections 7.8.4, 7.8.5 and Reference 12 refer to fire tests of inverted tee beams prestressed with post-tensioned tendons. In one specimen the tendon was bonded while in the other the tendon was unbonded. The superimposed load on the unbonded specimen was substantially lower than the load on the bonded specimen. Both tests were terminated at 4 hr 15 min even though no end point was reached. At the ends of the tests the midspan deflections were about 1 in. for the 17 ft 5 in. spans. A companion pretensioned specimen was also fire tested in the same series. The behavior of the pretensioned specimen was similar to that of the post-tensioned companions.

7.3.3 PCA Tests

As part of a broad series of fire tests, the Portland Cement Association fire tested seven 40-ft beams in which the reinforcement was post-tensioned.[7] Abstracts of these test results are presented in Sections 7.8.6 through 7.8.72, inclusive. Two types of reinforcement were used, high strength alloy steel bars and cold-drawn wires with button heads. Beams were essentially rectangular, 14 in.

TABLE 7.1 — Data from PCA Tests (Reference 7)

Beam NO.	Type of Reinforcement	Bonded or Unbonded	Type of concrete	Superimposed Load lb per ft.	Fire Endurance hr : min.
80	Bars	Unbonded	Normal weight	1040	502
82	Bars	Bonded	Normal weight	1535	4 29
83	Bars	Bonded	Lightweight	1680	5 01
76	Wires	Unbonded	Normal weight	1135	3 04
78	Wires	Bonded	Normal weight	1750	3 20
79	Wires	Bonded	Lightweight	1740	4 33
89	Wires	Bonded	Normal weight	1760	3 18

wide, 25 in. deep, with 6-in. x 4-in. flanges. Tendon cover at midspan was 2-1/2 in. *Table* 7.1 gives some pertinent data about the specimens and tests.

Beams were simply supported on rocker-roller supports to minimize restraint to thermal expansion. Included in the series of tests were companion specimens reinforced with Grade 40 and Grade 60 bars, and three specimens with pretensioned seven-wire strand. Among the conclusions reached from the PCA series of tests: (1) prestressed beams of lightweight concrete had longer fire endurances that their normal weight companions, and (2) beams with unbonded post-tensioned reinforcement had about the same fire endurances as their counterparts with bonded reinforcement.

7.3.4 NBS Tests

As noted above, the six tests conducted at the National Bureau of Standards in 1953 involved beams manufactured in England and fire tested in accordance with the 1932 edition of British Standard 476. The test procedure of BS476-32 is similar to that of ASTM El 19 except for one major difference. The loading requirement of BS476-32 called for a superimposed load of 1-1/2 times the design live load rather than one live load. Recent editions of NS476 have revised that requirement to one live load -the same as ASTM El 19. Thus the fire endurances of the six NBS tests were probably significantly shorter than might be expected if the normal loading had been applied. Abstracts of these tests are presented in Sections 7.8.73 through 7.8.78 inclusive. The beams were rectangular with or without a composite slab. The steel consisted of wires, 0.1 or 0.2-in. diameter, post-tensioned and grouted. Span was either 10 ft. or 16 ft. and beams were simply supported. Two beams were coated with an inch thickness of vermiculite concrete. Fire endurances (not adjusted for loading) ranged between about 1-1/2 and 6 hours.

7.4 ANALYSIS OF TEST DATA

The "Fire Resistance Directory",[14] published by Underwriters Laboratories, Inc., gives unrestrained and restrained assembly ratings for many designs of floors, roofs, and beams. Although many of the ratings are the results of standard fire tests, most of the ratings given for prestressed concrete assemblies are based on engineering studies.

In the UL studies that led to the classification criteria included in the "Fire Resistance Directory", measured steel temperatures were compared with each other and with published data on temperatures within concrete members during fire tests. For example, steel temperatures during full scale fire tests of 14 hollow-core slabs were compared with temperatures measured within plain concrete slabs. Data from four of the tests of lightweight concrete slabs are shown in *Fig. 7.3.* The basic slab temperature data on which the charts are based were developed in the test program described in PCA Research Department Bulletin 223.[15] Charts similar to *Fig. 7.3* for carbonate and siliceous aggregate concretes were also prepared and analyzed. Note that the measured steel temperature curves are roughly parallel to the slab temperatures curves. In each case the steel temperatures are somewhat lower than those estimated from the slab data for the same distance from the exposed surface. Thus the cover requirements based on the slab concrete temperatures are slightly conservative. Based On *Fig.* 7.3, the cover requirements for unrestrained lightweight prestressed concrete

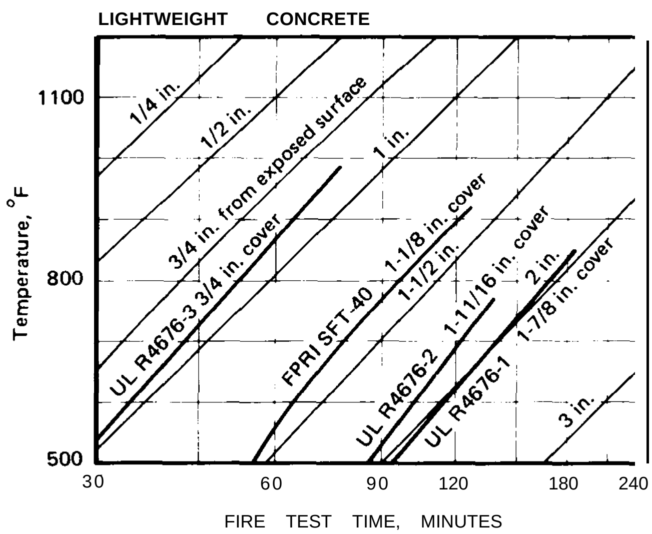


Fig. 7.3 — Temperatures within concrete slabs during fire tests — expanded shale aggregates — showing strand temperatures in hollow-core slabs.

slabs are approximately 1 in. for 1 hr, 1-5/8 in. for 2 hr and 2 in. for 3 hr.

A similar, though more complex, procedure was used in analyzing the data for stemmed units and inverted teel beams.

The procedure used by UL in developing their classification criteria are essentially those used below for analyzing slabs and beams.

7.4.1 Analysis of Slab Data

Pertinent steel temperature data from the three tests of slabs are shown in *Figs.* 7.4, 7.5, and 7.6. *Fig. 7.4* shows the temperatures of the tendons in the beam-and-slab assembly, FPRI SFT-1. Note that the temperature of the slab tendons with 1-1/2 in. cover reached an average of 800F after 3 hours of fire exposure. This temperature is far lower than would be expected for 1-1/2 in. cover based on the concrete slab temperatures data. In fact, the beam steel temperatures were lower than the 2-in. line on the plot. The low recorded temperatures might be due to either or both of the following items. First, the recorded temperatures reflect the average temperature of the tendon rather than the maximum temperature that would occur at the bottom off the tendon. Second, the tendons were greased and wrapped and the lubricant and wrapping materials might have kept the tendons cooler during fire exposure.

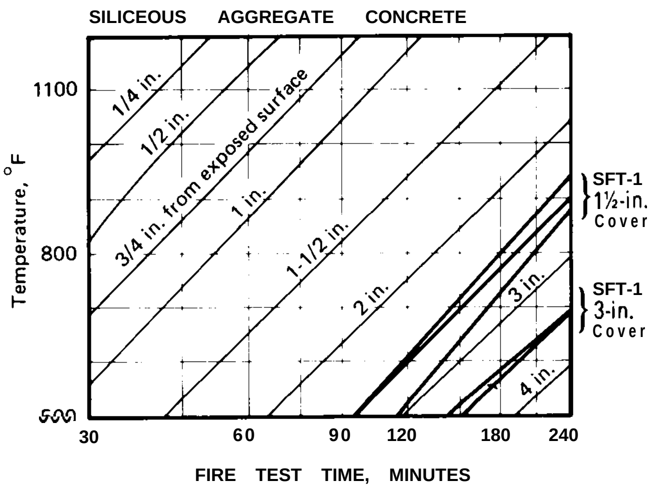


Fig. 7.4 — Temperatures within concrete slabs during fire tests — siliceous aggregates — showing slab tendon temperatures in FPRI-SFT-1.

Fig. 7.5 shows the tendon temperatures recorded during the test of the normal weight concrete flat plate floor, FPRI-SFT-2. The curvilinear shape of the temperature curves may be due to the furnace atmosphere temperatures which were somewhat low during the first 2-1/2 hours and high thereafter. Again the recorded steel temperatures were lower than would be expected from the data on which the curves are superimposed.

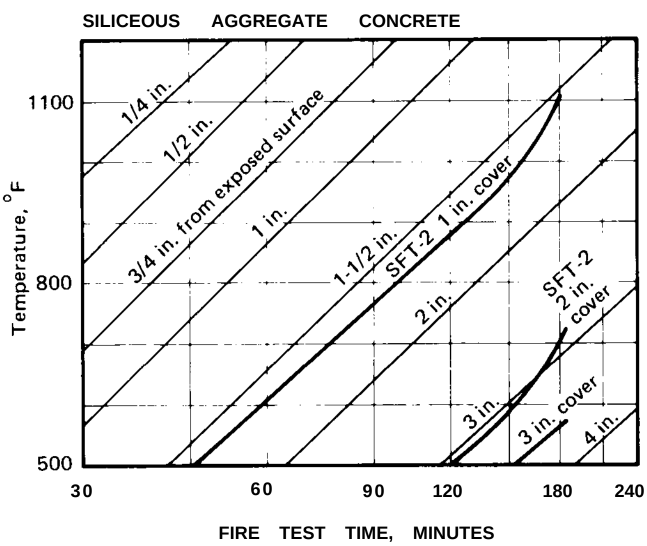


Fig. 7.5 — Temperatures within concrete slabs during fire tests — siliceous aggregate- showing tendon temperatures in FPRI-SFT-2. (Adjusted for furnace temperature lag.)

Fig. 7.6 shows the tendon temperatures recorded during the test of the lightweight concrete flat plate floor, UL R5084-3, superimposed on a plot of temperatures within lightweight concrete slabs during fire tests. Again the temperatures are lower, but to a lesser extent than those in *Figs. 7.4* and 7.5, possibly because the specimen was kiln-dried prior to the test.

In an attempt to determine required cover thickness for unrestrained slabs with post-tensioned tendons, an analysis can be made of test times at which the tendons reached 800F. These values can be compared with corresponding test times

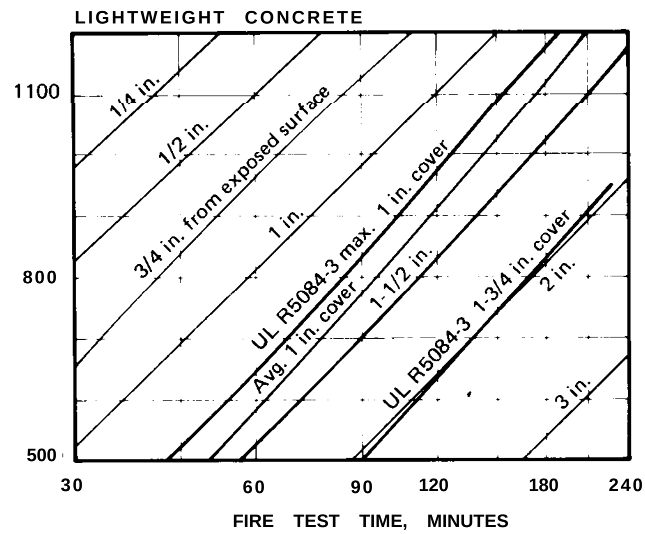


Fig. 7.6 — Temperature within concrete slabs during fire tests — expanded shale aggregates — showing tendon temperatures in UL R5084-3.

at which concrete at various levels reaches 800F during fire tests. *Table* 7.2 provides a basis for comparison.

The values of "B" in Table 7.2 are the distances from the exposed surface in plain concrete slabs

TABLE 7.2 — Summary of Data on Slab Tendons

Test	"A" Cover, in.	Test Time to Reach 800F, hr:min	"B" Corresponding Distance from Exposed Surface, in.	"B"—"A" in.
FPRI-1	1-1/2	3:12	2-5/8	1-1/8
FPRI-1	3	5:07*	3-5/8	5/8
FPRI-2	1	1:44	1-3/4	3/4
FPRI-2	2	3:30*	2-5/8	5/8
UL-R5084-5	1	1:31	1-3/8	3/8
UL-R5084-5	1-3/4	2:47	2-1/4	1/2

*Extrapolated

at which the temperature is 800F at the test time indicated in the third column. The values of "B" - "A" in the last column indicate the magnitude of reduction of cover possible. Note that those values range from 3/8 in. to 1-1/8 in. Thus a reduction of at least 3/8 in. is warranted. Resulting cover thickness for simply supported unrestrained slabs with post-tensioned reinforcement, based on temperatures within slabs with a 3/8 in. reduction, are given in *Table* 7.3.

Cover requirements shown in *Table* 7.3 apply to tendons 1/2 in. or larger in size. The values for 1 hr for carbonate and lightweight aggregate concretesaregoverned by minimum cover requirements for slabs, ACI 318-83.

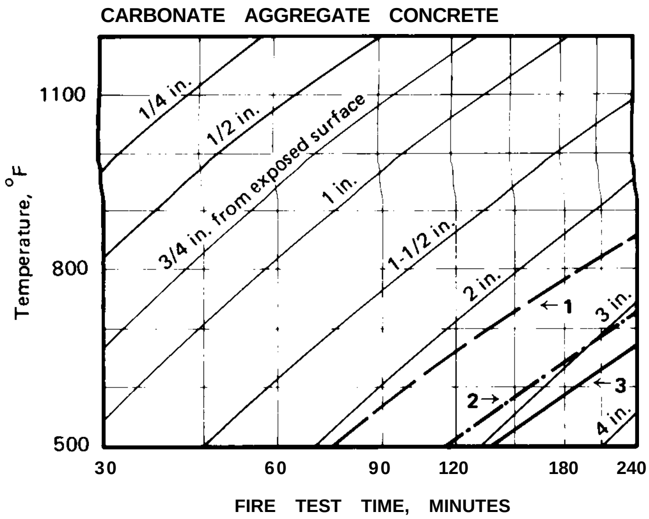
TABLE 7.3 — Cover Requirements for Unrestrained Slabs with Post-Tensioned Reinforcement

	Cover Thickness, in., for Fire Endurance of			
Aggregate Type	**1 hr**	**1-1/2 hr**	**2 hr**	**3 hr**
Carbonate	3/4	1-1/16	1-3/8	1-7/8
Siliceous	3/4	1-1/4	1-1/2	2-1/8
Lightweight	3/4	1	1-1/4	1-5/8

7.4.2 Analysis of Beam Data

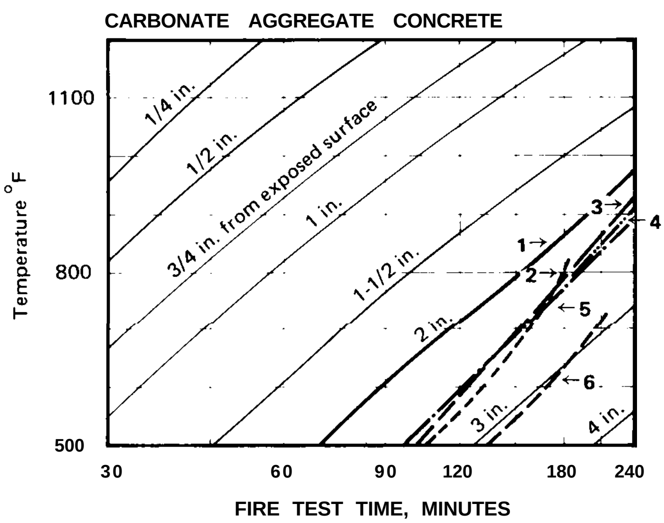
fig. 7.7 shows tendon temperatures at midspan of the three inverted tee beams fire-tested at Underwriters Laboratories. It is interesting to note that the temperatures of the pretensioned strand (UL R4213-12) correspond to the temperatures that might be anticipated for strand centered about 2-1/4 in. above the bottom of a slab even though the cover was only 1-3/4 in. and the strands were in a beam rather than in a slab. Temperatures of the post-tensioned tendons (UL R4123-12A) were lower yet, possibly because the bonded and unbonded tendons were centered 2-3/4 and 2-1/2 in. respectively above the bottoms of the beams. Nevertheless, the temperatures correspond to those of about 3-1/4 and 3 in. above the bottom of a slab. The low temperatures for the bonded tendon might result from the insulation afforded by the high water content of the grout within the duct.

Figs. 7.8 and 7.9 show temperatures of the corner bars or strands of the 40-ft. beams tested at PCA. The temperatures shown represent the maximum bar or strand temperatures because the corner bars or strands, i.e., those with 2-1/2 in. side and bottom cover, were the hottest in each of the tests. Note that in *Fig.* 7.8 (normal weight concrete) the time-temperature relationships for the post-tensioned bars and the unbonded post-tensioned wires are grouped closely together. Temperatures of the corner pretensioned strands were higher and those of the bonded post-tensioned wires were lower. The same approximate relationships are also true for the lightweight concrete specimens, Fig. 7.9.



1. Pretensioned 1-3/4 in. cover.
2. Post-tensioned unbonded 1-7/8 in. cover.
3. Post-tensioned bonded 1-15/16 in. cover.

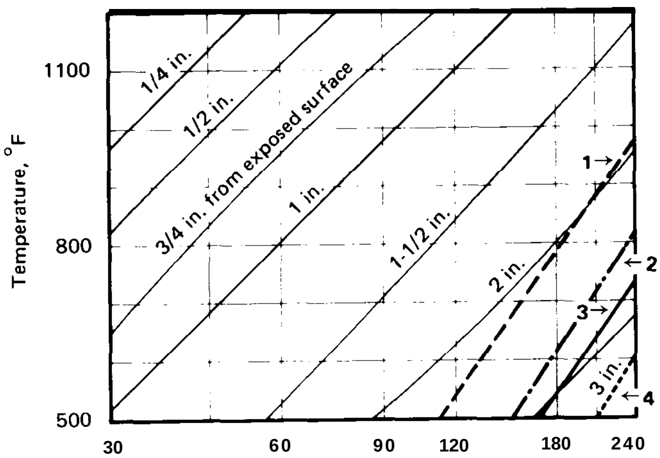
Fig. 7.7 — Temperatures within concrete slabs during fire tests carbonate aggregate — showing tendon temperatures in inverted tee beams, UL 4123-12-12A.



1. Pretensioned strand.	4. Post-tensioned bars, bonded.
2. Post-tensioned wires, bonded.	5. Post-tensioned bars, unbonded.
3. Reinforcing bars.	6. Post-tensioned wires, unbonded.

Fig. 7.8 Temperatures within concrete slabs during fire tests — carbonate aggregates -showing temperatures of corner bars, wires or strand during PCA tests of 40-ft beams.

From figs. 7.8 and 7.9 it can be seen that the corner bar temperatures of the post-tensioned units are essentially the same as (or lower than) slab temperatures at a distance of about 2-1/2 in. from the exposed surface. Since that was the cover of the corner bars, the cover requirements for slabs, as determined from the PCA concrete slab data, should be adequate for beams with dimensions roughly comparable to those tested. On this basis, for beams with post-tensioned reinforcement wider than about 12 in. the cover requirements for unrestrained classifications would be those shown in *Table* 7.4.



1. Pretensioned strand.
2. Post-tensioned bars.
3. Reinforcing bars.
4. Post-tensioned wires.

Fig. 7.9 Temperatures within concrete slabs during fire tests — expanded shale aggregates showing temperatures of corner bars, wires or strand during PCA tests of 40-ft beams.

TABLE 7.4 — Cover Requirements for Beams at Least 12-in. Wide and Prestressed with Post-Tensioned Reinforcement

Steel Type	Concrete Type*	For Beams at Least 12-in. Wide, Cover Thickness, in., for Fire Endurance			
		1 hr	2 hr	3hr	4 hr
Cold-Drawn	NW	1-1/2	2	2-1/2	3
Cold-Drawn	LW	1-1/2	1-3/4	2	2-1 /2
H.S.A. Bars	N W	1-1/2	1-1/2	1-1/2	2
H.S.A. Bars	LW	1-1/2	1-1/2	1-1/2	2

***NW = normal weight; LW = lightweight**

The above tabulation assumes that the minimum cover would be 1-1/2 in. for all beams and that tendons are 1/2 in. or larger in size. For narrower beams the cover would have to be somewhat greater in some cases. For beams 8 in. wide, comparable cover requirements could be those shown in *Table* 7.5. For beams with widths between 8 in. and 12 in., cover requirements can be obtained by direct interpolation. For example, for a 10 in. wide beam of lightweight concrete with cold-drawn steel, the cover for 3 hours would have to be 2-5/8 in.

TABLE 7.5 — Cover Requirements for 8-in. Wide Beams Prestressed with Post-Tensioned Reinforcement

Steel Type	Concrete Type	For Beams B-in. Wide, Cover Thickness, in., for Fire Endurance			
		1 hr	I-112 hr	2 hr	3 hr
Cold-Drawn	NW	1-3/4	2	2-112	4-1/2*
Cold-Drawn	LW	1-1/2	1-3/4	2	3-1/4
H.S.A. Bars	NW	1-1/2	I-112	1-1/2	2-1/2
H.S.A. Ban	LW	1-1/2	1-1/2	1-1/2	2-114

***Not practical but shown for interpolation purposes.**

The values for 8 in. wide beams were derived from the relationships of cover, beam width, and temperature based on results of tests at PCA and UL, some of which have not yet been published. For cold-drawn steel, a temperature limit of 800F was used. For high strength alloy steel bars, a temperature limit of 1000F was used, making the results somewhat conservative.

7.4.3 Protective Coatings

A 1972 report[16] gives analyses of fire tests of slabs, beams, and joists and concludes with recommended thicknesses of sprayed insulation for prestressed units. The thickness of sprayed mineral fiber, vermiculite cementitious material, or intumescent mastic are given for slabs and beams of various widths and concrete cover thickness.

Even though none of the tests analyzed in that report were of members with post-tensioned reinforcement, the data should be directly applicable to any member with cold-drawn prestressing steel. It should be noted that two of the NBS tests were of specimens coated with vermiculite concrete. In each case the fire endurance of the

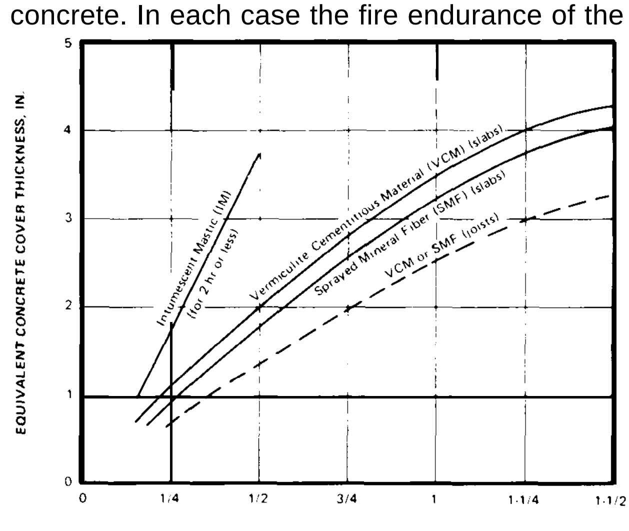


Fig. 7.10 — Equivalent concrete cover thickness for spray applied coatings.[16] Drawing used by permission of the Prestressed Concrete Institute.

coated specimen was more than double that of its uncoated counterpart. The data are not directly applicable to beams or slabs with high strength alloy steel bars, but would be conservative if applied directly.

7.5 ANALYSES OF RESULTS OF TESTS OF TENDON-ANCHOR ASSEMBLIES AT HIGH TEMPERATURES

A number of tests have been performed to determine if anchors commonly used in America for post-tensioning continue to function at temperatures that occur during fires. Reports of these tests are not readily available, so much of the pertinent data is included here.

7.51 Tensile Tests of Tendon-Anchor Assemblies

Three series of tests were conducted at the Portland Cement Association Laboratory. In two of the series, tests were performed at various temperatures between 600F and 1000F, and 70F. In the other series, twelve types of tendon-anchor assemblies were tested at 800F and at 70F. Results of these tests were compared with results of tensile tests of tendons in which the anchors were not heated.

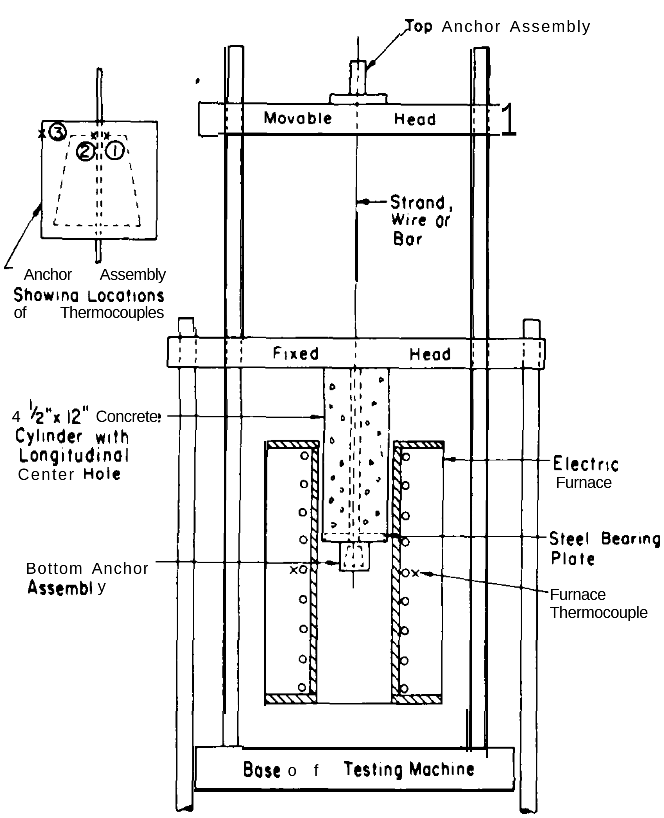


Fig. 7.11 — Arrangement for high temperature tests of *tendon*-anchor assemblies.

Fig. 7.*11* shows the test set-up. Note that the bottom anchor assembly was centered within the electric furnace. The concrete cylinder was used to provide uniform bearing for the anchor. In fact. several of the types of bearing plates must be cast into the concrete. The cylinder also served to locate the anchor within the furnace. Some of the cylinders were jacketed with steel pipes. In such cases, the bearing plates were machined to a maximum diameter of 3-7/8 in. to ensure that the plates did not bear on the steel jacket. A load of about 1000 lb. was applied to the specimen at the start of the test and maintained during the heating period. A period of 2 to 3-1/2 hours was required to heat the specimen to the desired test temperature. When the thermocouples 1, 2, and 3 *(Fig.* 7.7 *1),* located on the anchor housing and on the tendon, reached the test temperature with a variation of 15F or less, the tensile load was increased at a rate of about 8000 lb. per minute until failure occurred.

Results of the two series of tests conducted at 70F, 700F, 800F, 900F, and 1000F, are shown in *Fig. 7.72.* Series No. 1 consisted of 0.6-in. diameter strand and a rather massive anchor-bearing plate assembly. Both the strand size and the anchor are among the largest in use in America. They were selected for this series of tests because the investigators felt that large tendon anchor assemblies might be more vulnerable to heat than smaller ones. Series 12 consisted of 1/2-in. strand and small anchor-bearing plate assemblies. Duplicate tests were conducted

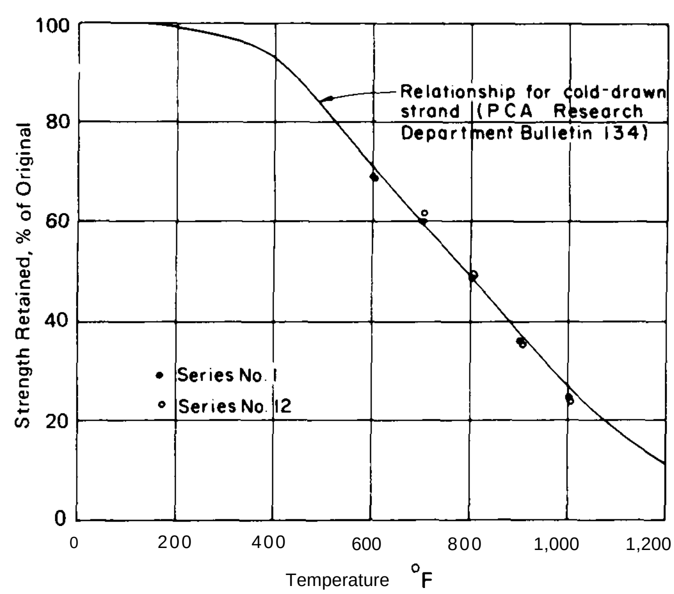


Fig. 7.12 — Relationship between temperatures and the tensile strength of cold-drawn strand (from Abrams and Cruz) together with results of tendon-anchor assemblies.

at 70F, 700F, and 900F and triplicate tests at 800F. Relatively small differences in the breaking loads occurred for duplicate and triplicate tests at a specific temperature. *Fig. 7.12* shows the results of these tests compared with the tensile strength-temperature relationship of cold-drawn steel strand determined by Abrams and Cruz[6] from tests in which the anchors were not heated. It can be noted that the test results compare favorably with those for strand, differing by 3 percentage points or less in all cases. Thus it appears reasonable to assume that the temperature-strength relationships of tendon-anchor assemblies are about the same as those for the tendon alone.

In the third series of tests, twelve types of tendon-anchor assemblies were tested at temperatures of 800F and 70F. Eight assemblies made use of 1.2-in. diameter seven-wire strand, two used 0.6-in. strand, one used 1/4-in. diameter button-headed wire, and one a 5/8-in. diameter deformed bar. *Fig. 7.13* shows the results of the twelve 800F tests compared with the comparable value reported by Abrams and Cruz[6] for strand at 800F. The value shown for Series No. 9 is not directly comparable to the others because the tendon was a 5/8-in. diameter hot-rolled bar having a tensile strength of 215 ksi. The other tendons were 270ksi cold-drawn strand or 240-ksi wire. Note that the average variation from that for strand was less than 2 percentage points and the maximum variation was about 4 percentage points.

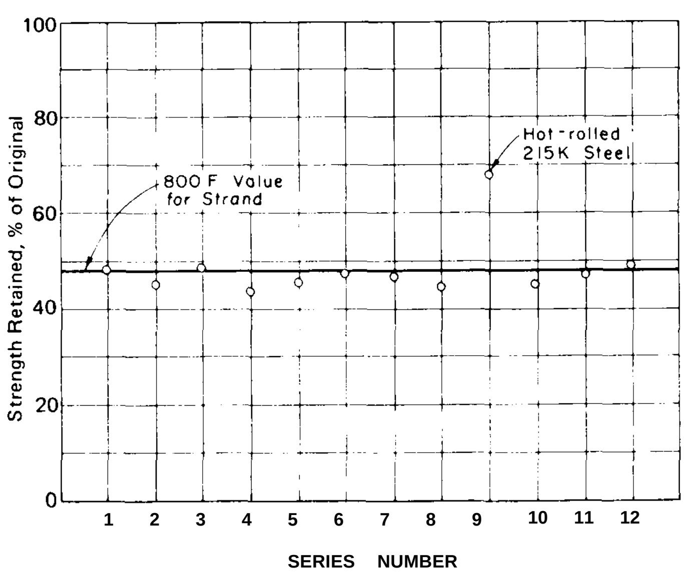
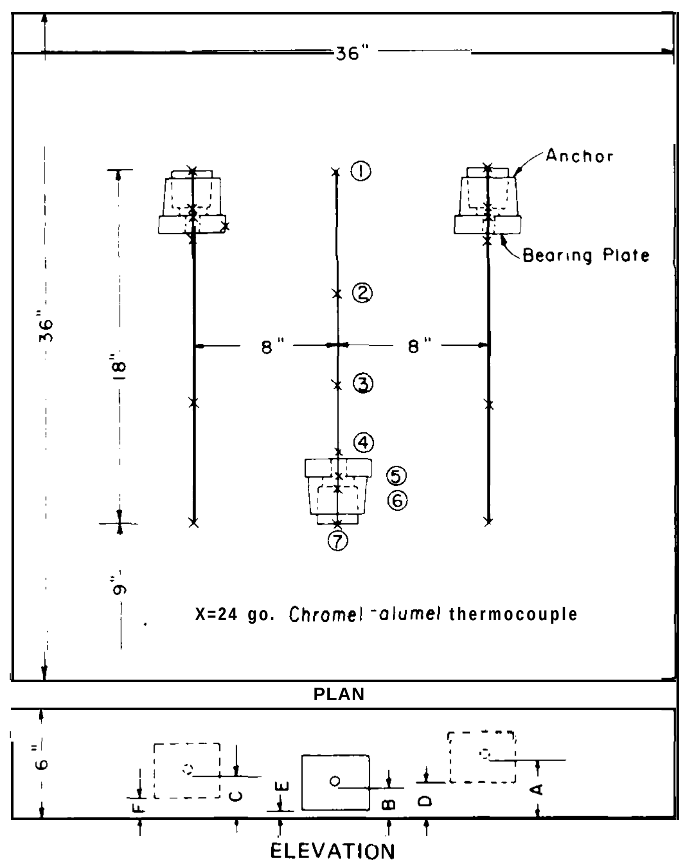


Fig. 7.13 — Results of tests made at 800F compared with value obtained for cold-drawn strand at same temperature.

As noted above, the results of Series No. 9 are not directly comparable to the others since the temperature-strength relationship for 215 ksi hot-rolled steel is probably different than that of cold-drawn strand. From *Fig. 7.7* it can be seen that high strength alloy steel bars (145 ksi) have about 80% of their 70F strength at 800F. Even though the value of 68% for Series No. 9 is lower than that for 145-ksi bars, it is considerably higher than the value for 270-ksi cold-drawn steel, and thus seems to be reasonable.

From these tests it appears that the anchor does not influence the temperature-strength relationship significantly. It should be noted that the tendon-anchor assemblies represented a wide spectrum of those in use in America. it does not appear that the mass of the anchor has a significant influence on the behavior at high temperatures.



Dimensions in inches

Dimension	A	B	C	D	E	F	Strand Size
Specimen A	3	1 1/2	2 1/4	1 7/8	3/8	1 1/8	0.6 in
Specimen B	1 1/2	2 1/2	3	3/4	1	1	1/2 in

Fig. 7.14 — Details of strand-anchor assemblies embedded in slab

7.5.2 Fire Tests to Study the Effects of Cover on Tendons and Anchors

In most post-tensioned structures, exposure to fire is likely to be less severe at the anchor than at other locations in the beam or slab. However, in some cases, the anchors are situated in vulnerable locations. Because the anchors represent concentrations of metal, it is likely that the strand temperature at the anchor can be different from that away from the anchor.

To study the magnitude of the temperature difference, fire tests were performed on two slabs in which three tendon-anchor assemblies were embedded. Slab specimens were 3 ft by 3 ft in plan and 6 in. thick. Strands in the tendon-anchor assemblies were horizontal throughout as shown in *Fig.* 7.74. In Specimen A, tendons were 0.6 in. diameter strand and the anchors were rather massive; concrete cover thicknesses beneath the strand were 1-1/2 in., 2-1/4 in. and 3-in. In Specimen B, tendons were 1/2 in. strand with small anchors, and cover thicknesses were 1-1/2 in., 2-1/2 in., and 3 in. Covers to the bearing plates were 3/8 in., 1-1/8 in., and 1-7/8 in. in Specimen A and 3/4 in., 1 in, and 1 in. in Specimen B. Six thermocouples were located on each tendon both at the anchor and away from the anchor.

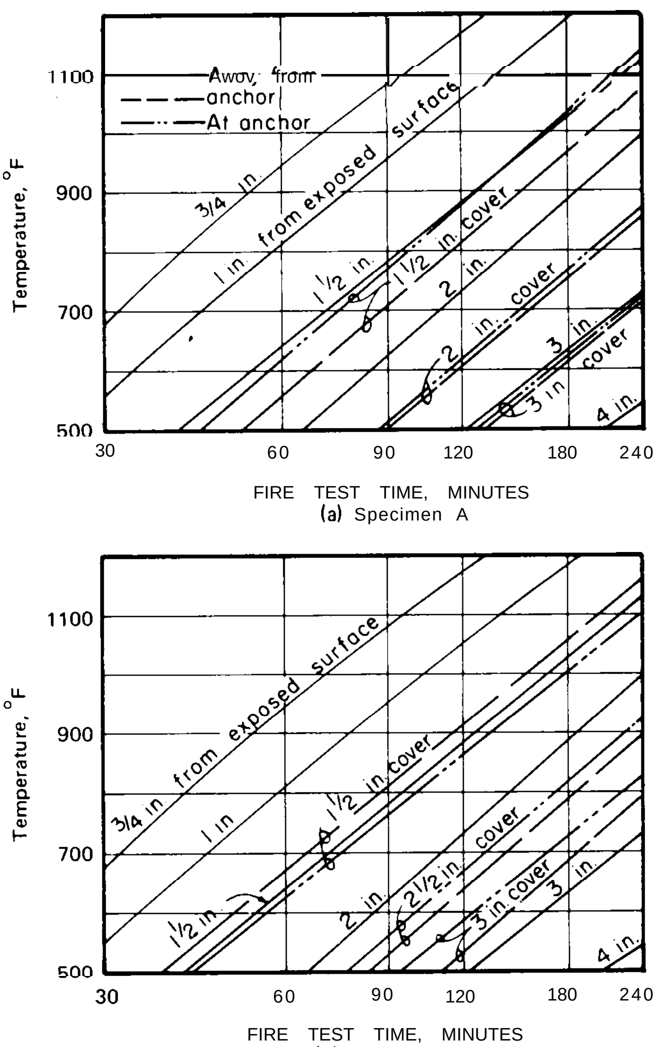


Fig. 7.15 Temperatures within concrete during fire tests compared with temperatures at and away from anchors.

Fig. 7.75 shows the results of the tests. Temperatures of the tendons at the anchors were higher than away from the anchor for three of the tendons. The difference was insignificant for two tendons, and for one tendon, the temperature of the tendon at the anchor was cooler than away from the anchor. Disregarding the tendon that was cooler at the anchor, the tendons were up to about 70 degrees warmer at the anchor than away from the anchor. To compensate for the higher temperature at the anchor, the cover to the tendon can be increased by about 1/4 in.

7.5.3 Fire Tests to Study the Effects of Different Sheathing Materials for Unbonding

In America, unbonded post-tensioned tendons have been greased and sheathed with either kraft paper or plastic. Paper sheathing is now obsolete. Plastic sheathing which is now used by all fabricators is in the form of a continuous tube. A fire test was conducted at the Portland Cement Association Laboratory to determine if the sheathing material affects the tendon temperature during exposure to fire.

The fire test specimen consisted of a concrete slab in which some strands were sheathed with paper and some with plastic. The 4 in. thick slab specimen, which was 3 ft by 3 ft in plan, contained two layers of sheathed strand. Four strands in the east-west direction had l-in. cover and four in the north-south direction had 2-in. cover. At each level, the first and third strands were sheathed with paper and the other two with plastic. Two thermocouples were positioned on each strand, one at midspan and the other 12 in. away. The thermocouples were located on the strand within the sheaths.

The slab specimen was exposed to a standard (ASTM E119) fire exposure for 2-1/2 hours. During the test, thermocouple readings were monitored and compared. With l-in. cover, strands with paper sheathing were about 15 to 30 degrees F cooler than those with plastic sheaths. With 2-in. cover, strands with paper sheaths were 10F cooler to 15F warmer than those with plastic sheaths. These differences are not considered to be significant because of the usual variations in temperature readings of embedded metal in concrete. Thus it appears that the type of sheathing material (paper or plastic) has only a minor influence on the strand temperature and does not affect the concrete cover requirements significantly.

7.6 RECOMMENDATIONS FOR MINIMUM DIMENSIONS FOR VARIOUS FIRE RESISTIVE CLASSIFICATIONS

7.6.1 Slabs

For heat transmission, i.e., temperature rise of 250F of the unexposed surface, the thickness requirements for concrete slabs should be the same whether the concrete is plain, reinforced, or prestressed. *Table* 7.6 gives slab thicknesses suggested in PCA Research Department Bulletin 223[15].

Cover thicknesses for post-tensioned tendons in unrestrained slabs are determined from results of fire tests of restrained specimens by the elapsed time during a fire test until the tendons reach a critical temperature. For cold-drawn prestressing steel that temperature is 800F. For restrained slabs there are no temperature limitations. Fire tests of restrained slabs indicate that slabs with post-tensioned reinforcement behave about the same as reinforced concrete slabs of the same dimensions. Accordingly the cover for post-tensioned tendons in slabs should be the same as the cover for reinforcing steel in slabs. Cover thicknesses are suggested in *Table* 7.7 for slabs with post-tensioned tendons made of cold-drawn steel.

TABLE 7.6 — Suggested Concrete Slab Thickness Requirements for Various Fire Endurances

Aggregate Type	Slab Thickness, in., for Fire Endurance Indicated				
	1 hr	1-1/2 hr	2 hr	3 hr	4 hr
Carbonate	3-1/4	4-1/8	4-5/8	5-3/4	6-5/8
Siliceous	3-1/2	4-1/4	5	6-1/4	7
Lightweight	2-5/8	3-1/4	3-3/4	4-5/8	5-1/4

TABLE 7.7 — Suggested Concrete Cover Thickness for Slabs Prestressed with Post-Tensioned Reinforcement

Restrained or Unrestrained	Aggregate Type	Cover Thickness, in., for Fire Endurance of				
		1 hr	1-1/2 hr	2 hr	3 hr	4 hr
Unrestrained	Carbonate	3/4	l-1116	1-3/8	1-7/8	—
Unrestrained	Siliceous	3/4	1-1/4	1-1/2	2-1/8	—
Unrestrained	Lightweight	3/4	1	1-1/4	1-5/8	—
Restrained	Carbonate	3/4	3/4	3/4	1	1-1/4
Restrained	Siliceous	3/4	3/4	3/4	1	1-1/4
Restrained	Lightweight	3/4	3/4	3/4	3/4	1

TABLE 7.6 — Suggested Cover Thickness for Beams Prestressed with Post-Tensioned Reinforcement

Restrained or Unrestrained	Steel Type	Concrete Type*	Beam Width,** in.	Cover Thickness, in., for Fire Endurance of				
				1hr	1-1/2 hr	2 hr	3 hr	4 hr
Unrestrained	Cold-drawn	NW	8	1-3/4	2	2-1/2	4-1/2***	— — —
Unrestrained	Cold-drawn	LW	8	1-1/2	1-3/4	2	3-1/4	---
Unrestrained	H.S.A. Bars	NW	8	1-1/2	1-1/2	1-1/2	2-1/2	— — —
Unrestrained	H.S.A. Bars	LW	8	1-1/2	1-1/2	1-1/2	2-1/4	---
Restrained	Cold-drawn	NW	8	1-1/2	1-1/2	1-3/4	2	2-1/2
Restrained	Cold-drawn	LW	8	1-1/2	1-1/2	1-1/2	1-3/4	2
Restrained	H.S.A. Bars	NW	8	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2
Restrained	H.S.A. Bars	LW	8	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2
Unrestrained	Cold-drawn	NW	>12	1-1/2	1-/34	2	2-1/2	3
Unrestrained	Cold-drawn	LW	>12	1-1/2	1-1/2	1-3/4	2	2-1/2
Unrestrained	H.S.A. Bars	NW	>12	1-1/2	1-1/2	1-1/2	1-1/2	2
Unrestrained	H.S.A. Bars	LW	>12	1-1/2	1-1/2	1-1/2	1-1/2	2
Restrained	Cold-drawn	NW	>12	1-1/2	1-1/2	1-1/2	1-3/4	2
Restrained	Cold-drawn	LW	>12	1-1/2	1-1/2	1-1/2	1-1/2	1-3/4
Restrained	H.S.A. Bars	NW	>12	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2
Restrained	H.S.A. Bars	LW	>12	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2

*NW = normal weight; LW = lightweight

**For beams with widths between 8 and 12 in., cover thickness can be determined by interpolation

"'Not practical for S-in. wide beam but shown for purposes of interpolation

7.6.2 Beams

Minimum dimensions for beams with post-tensioned reinforcement for various fire endurances are functions of the types of steel and concrete, beam width, and cover. For very wide beams, the cover requirements should be about the same as those for slabs.

For restrained beams spaced more than 4 ft on centers, the fire endurance is twice the elapsed time during a fire test at which the steel reaches the critical temperature. The suggested cover thicknesses in *Table* 7.8 are based on these criteria.

For beams or joists less than 8 in. wide, the UL requirements for pretensioned stemmed members can be used for members with post-tensioned cold-drawn steel. Beams or joists with post-tensioned high strength alloy steel bars and narrower than 8 in. should have the same cover as reinforced concrete joists of the same size and fire endurance.

7.6.3 Anchor Protection

The cover to the prestressing steel at the anchor should be at least 1/4 in. greater than that required away from the anchor. Minimum cover to the steel bearing plate should beat least 1 in. in beams and 3/4 in. in slabs.

7.7 RATIONAL DESIGN PROCEDURES

7.7.1 General

As background for the discussion of rational design procedures for evaluating the behavior of structures during fires, three types of flexural members will be discussed briefly.

a) **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 expansion can occur without restriction. 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 collapse will occur. Such behavior has been clearly demonstrated in prestressed as well as reinforced concrete members.[2]

   It is apparent from the above description that the moment capacity depends on the steel temperature, which in turn depends *on* (a) the stress in the steel, and (b) the type of steel. The stress in the steel depends on the load intensity on the member. For example, if the steel stress is 50 percent of the initial yield strength, the critical temperature will be about 1120F. However, if the steel stress is one-third of the yield strength, the critical temperature will be about 1220F. 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. Through rational design procedures, it is possible to estimate the increase in fire endurance due to a decrease in load intensity.

b) **Continuous slabs and beams —** Structures that are continuous or otherwise statically indeterminate, undergo changes in moments when subjected to fire.[3] It should be noted that this is different than simply supported members where the applied moments at a section remain 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 tend 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.

   During the course of a fire, the negative moment reinforcement remains cooler than the positive moment reinforcement because it is better protected from the fire. Thus the increase in negative moment can be accommodated. The resulting decrease in positive moment means that the positive moment steel can withstand a higher temperature before failure will occur. Thus 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.

c) **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 first occurs, but as the fire progresses the line of action of the thrust rises as the heated concrete softens.[4] 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 properties 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 generally great enough to increase the fire endurance significantly, In most fire tests of restrained assemblies, the fire endurance is determined by temperature rise of the unexposed surface rather than by structural considerations, even though the steel temperatures often exceed 1500F. Design procedures are discussed in References 17 and 18.

7.7.2 Design for Fire Endurance

Rational design procedures for fire endurance of concrete elements rely on basic structural engineering principles together with:

a) Application of the information on structural behavior outlined in Section 7.7.7.

b) Use of temperature-strength relationships for steel and concrete, as presented in *Figs.* 7.7 and 7.2, respectively.

c) Empirical data on temperatures within concrete elements at various intervals during exposure to a standard fire, as presented in *Figs.* 7.3 through 7.9.

Use of rational design procedures for post-tensioned concrete beams and slabs which are made with bonded post-tensioned tendons are essentially the same as those for pretensioned prestressed concrete elements.[17] Curved tendons rather than straight or deflected strands introduce only minor differences which do not change the design procedures.

As indicated in *Section 7.4,* tests of post-tensioned elements indicate that the temperature of the tendons at the end of a fire test can be considered to be essentially the same regardless of whether the tendons are bonded or unbonded. Further, these tests indicate that the prestressing steel stress at ultimate during fire tests, $f_{ps\Theta}$, can be estimated from the relationship:

$$\frac{f_{ps\Theta}}{f_{pu\Theta}} = \frac{f_{ps}}{f_{pu}}$$

where f_{ps} = stress in prestressed reinforcement at nominal strength, psi. This stress may be calculated for bonded tendons by equation (18-3) of ACI 318-83 and for unbonded tendons by equations (18-4) and (18-5), respectively, in ACI 318-83.

f_{pu} = specified tensile strength of post-tensioning tendons, psi.

$f_{ps\Theta}$ = stress in post-tensioning tendons at nominal strength at high temperatures, psi.

$f_{pu\Theta}$ = tensile strength of post-tensioning tendons at high temperatures, psi. The value of $f_{pu\Theta}$ at various temperatures can be obtained by multiplying f_{pu} by the percent of strength at 70 degrees Farenheit from *fig. 7.7.*

For continuous beams or slabs utilizing continuous draped unbonded tendons exposed to fire from below, the value of $f_{ps\Theta}$ in the negative moment regions must be taken as the same as that in the positive moment region. Since the tendon is unbonded, the capacity at any point along its length is limited by the capacity at the point where the steel temperature is highest.

On the basis of the above, it is possible to determine the retained theoretical moment strength at a specified period of fire endurance (say 2 hours) in the positive moment region and in both negative moment regions of a given panel in a building. It will usually be possible to select a critical panel for the fire endurance calculations as discussed in reference to the design example below. The maximum moment capacity at exterior columns must not exceed that which can be transmitted to the column. To evaluate retained theoretical moment strength, it may be assumed that if a fire occurs beneath the floor, a redistribution of moments will occur yielding the negative moment reinforcement. If the applied mid-span moment is less than the retained moment capacity after redistribution, the fire endurance will be adequate. That is:

$$M \leqq M^+_{n\Theta} + 1/2\,(M^-_{n1\Theta} + M^-_{n2\Theta})$$

where $M^+_{n\Theta}$ = retained midspan moment capacity

$M^-_{n1\theta}$ = retained negative moment capacity at column 1

$M^-_{n2\theta}$ = retained negative moment capacity at column 2

If however, the span moment is greater than the retained moment capacity, changes must be made in the design. Several options for improving the fire endurance are available. The options include:

(1) Increase the concrete cover in the positive moment region.

(2) Increase the number of prestressing tendons.

(3) Add positive moment reinforcing steel.

(4) Add negative moment reinforcing steel.

(5) Undercoat the slab with spray-applied insulation.

Of course there are other solutions such as the use of a thicker slab, the use of lightweight concrete, or the addition of a fire resistive ceiling. Also, combinations of the options listed above can be used. The most appropriate solution depends on in-place cost, architectural acceptability, and perhaps other considerations. For example, to upgrade thefireenduranceof an existing floor, options 1 through 4 are not applicable, so either an undercoat or a ceiling might be most appropriate.

7.7.3 Example Problem

To illustrate the use of the procedures described above, assume that a 2-hour fire endurance is required for the 6-inch thick flat plate floor shown in fig. 7.76. Assume that the floor is one of the typical floors in a multi-story building. Interior panels in the floor can be considered to be restrained against thermal expansion, and thus will readily qualify for a 2-hour fire endurance. The panels between column lines 5 and 6 will have little restraint to thermal expansion in the longitudinal direction, and because the panels are longer than those between lines 1 and 2, they are probably most critical from a fire endurance standpoint.

Reinforcement in the selected panel is shown in fig. 7.77. Longitudinal tendons are unbonded and are grouped together within the column strips. Transverse tendons are also unbonded but are equally spaced throughout the floor. Supplemental tendons are provided in the end spans. All tendons are 1/2 inch strands with a guaranteed ultimate strength of 270 ksi. Reinforcing bars are Grade 60, and the 28-day concrete strength is 4000 psi. Slabs are made of carbonate aggregate concrete and the weight of the 6-inch slab can be assumed to be 75 psf. The superimposed dead load is 20 psf and a 40 psf live load is specified.

The first step in the calculations is to determine the retained theoretical moment strength at 2 hr. in the positive moment region and in both negative moment regions (over column lines 5 and 6). The moment capacity over line 6 must be transmitted to the columns along line 6, so the maximum value that can be used in the calculations must not exceed that which can be transmitted to the column. It should be assumed that if a fire occurs between the floor, a redistribution of moments will occur yielding the negative moment reinforcement. Thus if the span moment is less than the retained moment capacity, after redistribution, the fire endurance will be adequate, i.e.,

$$M \leqq M^+_{n\theta} + 1/2\,(M^-_{n5\theta} + M^-_{n6\theta})$$

If the span moment is greater than the retained moment capacity at 2 hr., changes must be made in the design as discussed in Section 7.7.2.

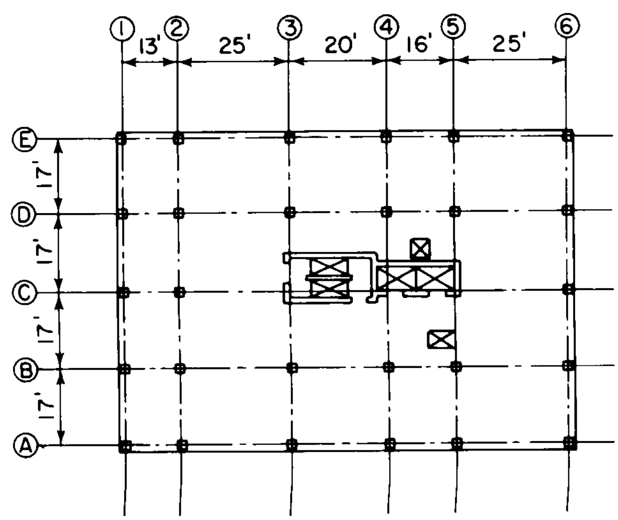


Fig. 7.16 — Typical Floor Plan

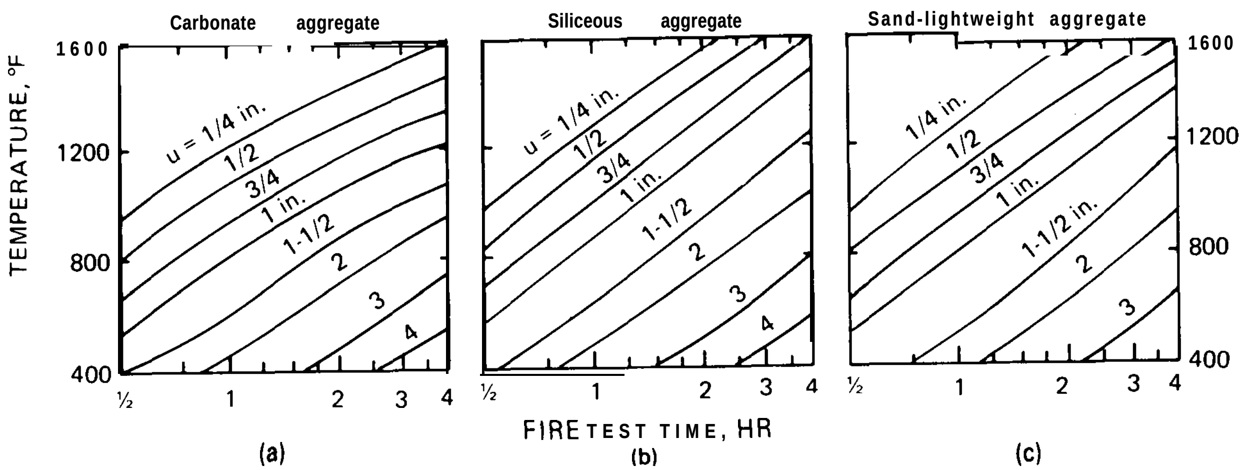


Fig. 7.18 -Temperatures within concrete slabs during fire tests.[15] Drawing used by permission of the Prestressed Concrete Institute.

Check span moment capacity

$$M_{n\Theta} = M^{+}_{n\Theta} + \frac{M^{-}_{n\Theta 5} + M^{-}_{n\Theta 6}}{2} =$$

$$72.6 + \frac{88.5 + 64.4}{2} = 149.0 \text{ ft-k} < 165.2 \text{ N.G.}$$

Option 1: Raise tendon profile in positive moment region by 1/2" so u = 2"

Θ_s = 720°F (Fig. 7.18a); $f_{pu\Theta}$ = 0.58 f_{pu} = 156.6 ksi (Fig. 7.1)

$f_{y\Theta}$ = 0.78 f_y = 46.8 ksi (Fig. 7.1)

$$f_{ps} = 159{,}000 + 10{,}000 + \frac{4000}{100\,\dfrac{(2.295)}{204 \times 4}}$$

= 183,200 psi

$$f_{ps\Theta} = \frac{183.2}{270}(156.6) = 106.3 \text{ ksi}$$

$$a^{+}_{\Theta} = \frac{2.245(106.3) + 0.8(46.8)}{0.85(4)\ (204)} = 0.40"$$

strand $M^{+}_{n\Theta} = 2.295(106.3)(4.0 - 0.20)/12 =$ 77.2 ft-k

rebar $M^{+}_{n\Theta} = 0.8(46.8)(4.0 - 0.20)/12 =$ 11.9 ft-k

$M^{+}_{n\Theta}$ = 89.1 ft-k

Values of $M^{-}_{n\Theta}$ remain the same

$$M_{n\Theta} = 89.1 + \frac{88.5 + 64.4}{2} =$$

165.5 ft-k > 165.2 OK

Note: Be sure to check behavior and moment capacities at normal temperatures. Also be sure thaft transverse steel can be installed without interference. Because d is reduced by 11 %, it is probable that two additional tendons will be needed.

Option 2: Add tendons to end span

Assume $M^{+}_{n\Theta}$ and $M^{-}_{n\Theta}$ are increased equally by $\dfrac{165.2 - 149.0}{2} = 8.1$ ft-k

strand $M^{+}_{n\Theta}$ = 60.0 ft-k; Approx. **No.** strands = $\left(\dfrac{60.0 + 8.1}{60.0}\right)$ (15) = 17

a^{+}_{Θ} = 0.32" ; $M^{+}_{n\Theta}$ = 80.4 ft-k

a^{-}_{Θ} = 0.46" ; @ Col. B5, $M^{-}_{n\Theta}$ = 96.1 ft-k;

@ Co. B6, $M^{-}_{n\Theta}$ = 68.7 ft.k

$$M_{n\Theta} = 80.4 + \frac{86.1 + 68.7}{2} =$$

162.8 ft-k < 165.2 close

By inspection 18 strands will be OK

Note: Be sure to check behavior and moment capacity at normal temperature.

Option 3: Add positive moment rebars

Increase $M^{+}_{n\Theta}$ by 165.2 − 149.0 = 16.2 ft-k

Estimate A^{+}_{s} (rebars) needed

Before adding rebars, $M^{+}_{n\Theta}$ = 72.6 ft-k

and a^{+}_{Θ} + 0.29 in.

New $M^+_{n\Theta} = 72.6 + 16.2 = 88.8$ ft-k

and $a^+_\Theta = \frac{88.8}{72.6}(0.29) = 0.36$ in.

strand $M^+_{n\Theta} = 2.295(72.1)(4.50 - 0.18)/12 =$ 59.6 ft-k

rebar $A^+_s = \frac{(88.8 - 59.6)(12)}{43.5(4.50 - 0.18)} = 1.87$ in.^{2}

Try 10 #4 Grade 60 bars, $A^+_s = 2.00$ in.^{2}
(Addition of 6 #4 Grade 60 bars)

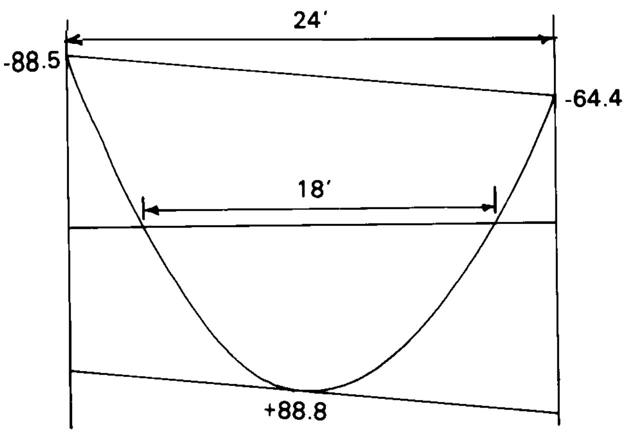
$a^+_\Theta = \frac{2.295(72.1) + 2.00(43.5)}{0.85(4)(204)} = 0.36$ in.

rebar $M^+_{n\Theta} = 2.00(43.5)(4.50 - 0.18)/12 =$ 31.3 ft-k

strand $M^+_{n\Theta}$ 59.6 ft-k

Total $M^+_{n\Theta}$ = 90.0 ft-k

90.9 ft-k > 88.8 ft-k OK



Length of rebars = 18 ft + 2l_d = 20 ft.

Wt of M^+ bars = 10(20)(0.668) = 133.3 lbs.

Orig. Wt of M^+ bars = 4(13)(0.668) = 34.7

Added rebar wt. = 98.9

Option 4: Add negative moment rebars over Col. B5

$M^-_{n\Theta}$ must be increased by

2(165.2 − 149.0) = 32.4 ft-k

$M^-_{n\Theta}$ must be 88.5 + 32.4 = 120.9 ft-k

$a^-_\Theta = \frac{120.9}{88.5}(0.43) = 0.59$ in.

strand $M^-_{n\Theta} = 2.295(72.1)(4.45 \quad 0.30)/12 =$ 57.2 ft-k

rebar $A^-_s = \frac{(120.9 - 57.2)/12}{52.2(4.45 - 0.30)} = 3.53$ in^2

Try 10 #5 + 4 #3 bars, $A^-_s = 3.54$ in.^{2}

check $a^-_\Theta = \frac{2.295(72.1) + 3.54(52.2)}{0.85(3.4)(204)} =$ 0.59 in. same as above

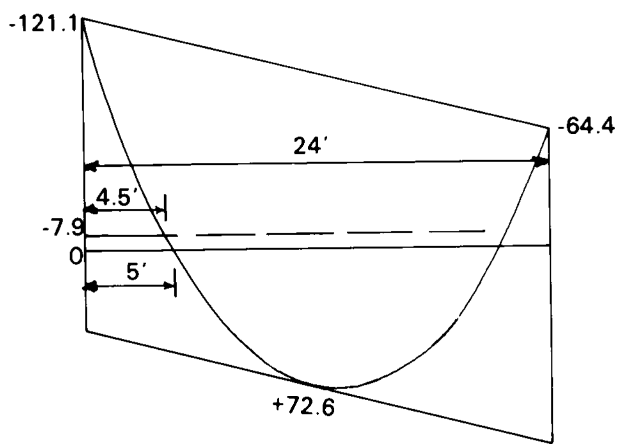
rebar $M^-_{n\Theta} = 3.54(52.2)(4.15)/12 = 63.9$ ft-k

strand $M^-_{n\Theta}$ = 57.2

$M^-_{n\Theta}$ = 121.1 ft-k

OK > 120.9 ft-k

Contribution of continuous #3 bars = $\frac{0.44}{3.54}(63.9) = 7.9$ ft-k



Added rebars must be 2(4.5) + 1 + 2(1) = 12 ft long

Wt of #5 bars = 10(12)(1.043)
= 125.2 lbs.

Wt of orig. #4 = 6(11)(0.668)
= 44.1 lbs.

Added Wt = 81.1 lbs.

Option 5: Undercoat slab with spray-applied insulation

From Option 1, 1/2"-in. additional cover is adequate. Thus an additional equivalent concrete cover thickness of 1/2 in. will be adequate. From *Fig. 7.70* for an equivalent concrete cover thickness of 1/2 in., the required thickness of spray mineral fiber or vermiculite cementitious material applied to slabs is less than 1/4 in. The minimum practical thickness is about 1/4 in. so use 1/4 in. of SMF or VCM.

Summary of Calculations

Option 1 requires that the tendons be raised 1/2-in. in the positive moment region. Although raising the tendons will result in a 2-hr. fire endurance, the behavior and moment capacity at room temperature may be inadequate, so it is likely that two or more additional tendons will have to be added to offset the reduction in d and e. Also, raising the tendons might create interference with the transverse tendons. These items must be checked before the design is completed.

Option 2 requires addition of three tendons. Behavior at normal temperature should be checked.

Option 3 requires addition of 98.9 pounds of reinforcing steel per panel on the positive moment region.

Option 4 requires addition of 81.1 pounds of reinforcing steel per panel on the negative moment region.

Option 5 requires a 1/4-in. undercoat of either sprayed mineral fiber or vermiculite cementitious material.

As illustrated by this example, the advantage of utilizing rational design procedures is that a number of options can be analyzed and the best solution based on cost and other considerations can then be selected. Prior to the use of rational design procedures, only one or two solutions were acceptable — increasing the concrete cover or using an undercoat or ceiling. Very often the best solution is the addition of some reinforcing steel which improves not only the fireendurance but also the overall strength and ductility of the floor.

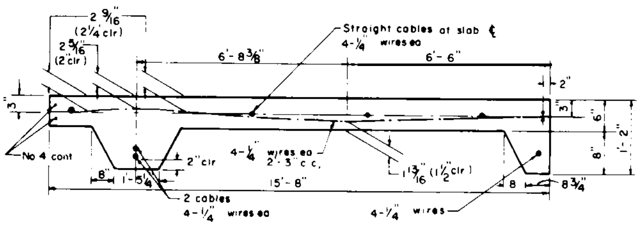
7.8 DIGESTS OF FIRE TEST RESULTS

7.8.1

DATE OF TEST: December 6, 1958

PLACE OF TEST: Fire Prevention Research Institute, Inc., Gardena, Calif.

SECTION TESTED: Beam-and-slab, post-tensioned, unbonded.



PRESTRESSING REINFORCEMENT: All cables composed of four %-in. diameter high tensile strength wires coated with mastic and wrapped. Post-tensioned, unbonded.

MINIMUM CONCRETE COVER: 1-½ in. for slab; 2 in. for beams.

AGGREGATES: Sand and gravel.

LOAD DURING TEST: One live load (75 psf).

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM E 119.

PERTINENT TEST DATA: Specimen was 15 ft. 8 in. by 16 ft. 0 in. out to out. Some spalling of beams occurred during test. Maximum deflection: 2.02 in. at 1 hr.; 2.56 in. at 2 hr.; 2.94 in. at 3 hr.; 3.57 in. at 4 hr. Cable temperature at 4 hr. 11 min.: in slab, maximum 971 F, average 948F; girder 913F.

REPORT: Fire Prevention Research Institute, February 1959, "Fire Test of Prestressed Concrete Floor Panel No. I," (SFT-I), by G. E. Troxell, FPRI, 19113 S. Hamilton St., Gardena, Calif.

DURATION OF TEST: 4 hr. 24 min. exposure.

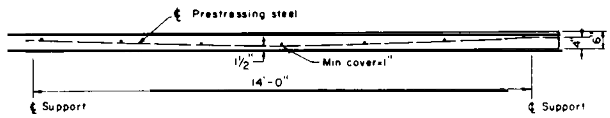
End point by heat transmission through slab was 3 hr. 51 min. Structural end point not reached.

7.0.2

DATE OF TEST: December 19, 1959

PLACE OF TEST: Fire Prevention Research Institute, Gardena, Calif.

SECTION TESTED: Two-way flat plate, post-tensioned, unbonded.



PRESTRESSING REINFORCEMENT: Each cable composed of six ¼-in. diameter high tensile strength wires, encased in a "stress-gard" slippage sheathing. Post-tensioned, unbonded.

MINIMUM CONCRETE COVER: 1 inch.

AGGREGATE: Normal weight aggregates.

LOAD DURING TEST: One design live load (120 psf).

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM E 119.

PERTINENT TEST DATA: Outside dimensions of slab were 15 ft. 8 in. by 16 ft. 0 in. with four supports located symmetrically at each corner and 14 ft. 0 in. on centers in each direction. Center point deflections: 2.4 in. at 1 hr.; 3.2 in. at 2 hr.; 4.2 in. at 3 hr. Cable temperatures at 3 hr: 545F minimum, 740F average 1046F maximum.

REPORT: Fire Prevention Research Institute, February 1960, "Fire Test of Six-Inch Deep Prestressed Concrete Flat Slab" (SFT-2), by G. E. Troxell.

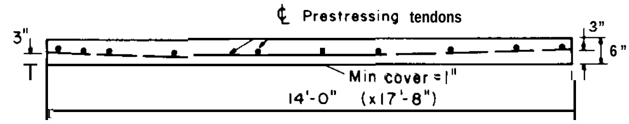
DURATION OF TEST: 3 hr 10 min. (Corrected for furnace temperature lag.) No end point reached.

7.8.3

DATE OF TEST: September 12, 1967

PLACE OF TEST: Underwriters Laboratories, Northbrook, Illinois.

SECTION TESTED: Prestressed post-tensioned flat plate floor slab.



PRESTRESSED REINFORCEMENT: Each tendon composed of five %-in. diameter high tensile strength wires. Post-tensioned, unbonded. Design stress 144,000 psi.

MINIMUM CONCRETE COVER: 1 inch.

AGGREGATE: Expanded shale.

LOAD DURING TEST: One design live load (100 psf).

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM E 1 19.

PERTINENT TEST DATA: The 14'0" x 17'8" slab was supported at the four corners only and restrained on all four sides by grout fill. No cracking or spalling occurred on the exposed surface during fire exposure. Hairline cracks diagonally across the four corners were observed on the unexposed surface. Center point deflections: 2.78 in. at 1 hr; 3.10 in. at 2 hr.; 3.80 in. at 3 hr.; 4.33 in at 3 hr. 45 min. Cable temperatures at 3 hr. 30 min.: 570F minimum, 1270F maximum. Slab withstood hose stream and double live load after standard fire test.

REPORT: Underwriters Laboratories, Inc., Retardant 5084-3.

DURATION OF TEST: 3 hr. 45 min. (No ASTM end point reached).

7.8.4

DATE OF TEST: April 13, 1965

PLACE OF TEST: Underwriters Laboratories, Northbrook, Illinois.

SECTION TESTED: Post-tensioned (bonded) inverted tee beam.

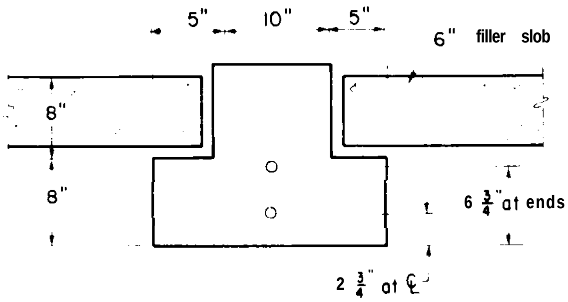
PRESTRESSING REINFORCEMENT: One bonded tendon, fourteen ¼-in. cold-drawn wires. Design stress 168,000 psi.

MINIMUM CONCRETE COVER: 1-15/16 in.

AGGREGATE: Natural sand and limestone.

LOAD DURING TEST: Beam+filler slabs+live load = 250+503+1295 = 2048 plf.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM El 19.



PERTINENT TEST DATA: 5-in. bearing area. 17 ft 5 in. c-c bearings. Beam ends restrained by grout fill. Lateral expansion provided. No cracking or spalling occurred on the exposed or unexposed surface during fire exposure. Maximum deflections: 0.79 in. at 30 min; 0.91 in. at 1 hr; 1.03 in. at 2 hr; 1.09 in. at 3 hr; 1.16 in. at 4 hr. Tendon temperature at centerline was approximately 5BOF at 3 hr; and 675F at 4 hr. Beam withstood hose stream and double live load after standard fire test.

REPORT: Underwriters Laboratories, Inc., Retardant 4 123-12A.

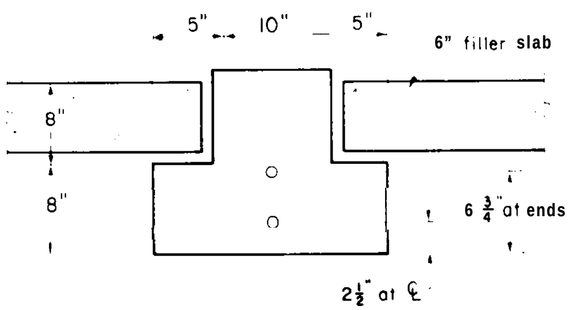
DURATION OF TEST: 4 hr 15 min. No ASTM end point reached.

7.8.5

DATE OF TEST: April 20, 1965

PLACE OF TEST: Underwriters Laboratories, Northbrook, Illinois.

SECTION TESTED: Post-tensioned (unbonded) inverted tee beam.



PRESTRESSING REINFORCEMENT: One unbonded tendon, fourteen ¼-in. cold-drawn wires. Design stress 168,000 psi.

MINIMUM CONCRETE COVER: 1-7/8 in.

AGGREGATE: Natural sand and limestone.

LOAD DURING TEST: Beam+filler slabs+live load = 250+503+880 = 1633 plf.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM El 19.

PERTINENT TEST DATA: 5 in. bearing area. 17 ft 5 in. c-c bearings. Beam ends restrained by grout fill. Lateral expansion provided. No cracking or spalling occurred on the exposed or unexposed surface during fire exposure. Maximum deflections: 0.65 in. at 30 min; 0.82 in. at 1 hr; 0.92 in. at 2 hr; 0.94 in. at 3 hr; 0.95 in. at 4 hr. Tendon temperature at centerline was approximately 640F at 3 hr; and 7 10F at 4 hr. Beam withstood hose stream and double live load after standard fire test.

REPORT: Underwriters Laboratories, Inc., Retardant 4123-12A.

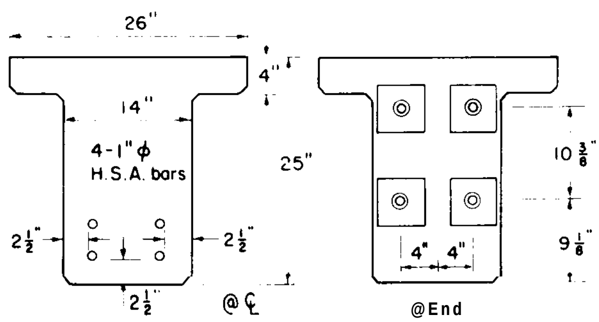
DURATION OF TEST: 4 hr 15 min. No ASTM end point reached.

7.8.6

DATE OF TEST: December 17, 1964

PLACE OF TEST: Fire Research Laboratory, Portland Cement Association, Skokie, I Il.

SECTION TESTED: Tee beam, post-tensioned, unbonded.



PRESTRESSING REINFORCEMENT: Four l-in. diameter high strength alloy steel bars, unbonded.

MINIMUM CONCRETE COVER: 2% in.

AGGREGATE: Carbonate sand and gravel from Elgin, Illinois.

LOAD DURING TEST: 1,040 lb per ft. superimposed load.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM E 119.

PERTINENT TEST DATA: Tested as a simply supported beam on a 40-ft span with no end restraint.* No spalling occurred. Average steel temperatures at midspan: 530F at 2 hr; 695F at 3 hr; 805F at 4 hr; 938F at 5 hr. Midspan deflections: 5.1 in. at 2 hr; 7.4 in. at 3 hr; 11.3 in. at 4 hr; and 21.5 in. at 5 hr.

REPORT: "Fire Resistance of Prestressed Concrete Beams. Study C. Structural Behavior During Fire Tests," by A.H. Gustaferro, M.S. Abrams, and E.A.B. Salse. PCA R&D Bulletin RD 009.01B, 1971.

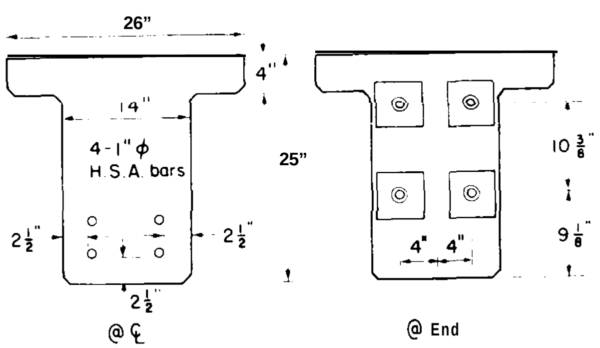
DURATION OF TEST: 5 hr 02 min.

7.8.7

DATE OF TEST: January 29, 1965

PLACE OF TEST: Fire Research Laboratory, Portland Cement Association, Skokie, I II.

SECTION TESTED: Tee Beam, post-tensioned, bonded.



PRESTRESSING REINFORCEMENT: Four l-in. diameter high strength alloy steel bars, bonded.

MINIMUM CONCRETE COVER: 2% in.

AGGREGATE: Carbonate sand and gravel from Elgin, Illinois

LOAD DURING TEST: 1,535 lb per ft. superimposed load.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM El 19.

PERTINENT TEST DATA: Tested as a simply supported beam on a 40-ft span with no end restraint.* Corner spalling occurred at 20 min. At about 3 hr 10 min flexural cracking began near midspan. Average steel temperatures at midspan: 500F at 2 hr; 690F at 3 hr; 820F at 4 hr. Midspan deflections: 6.6 in. at 2 hr; 9.4 in. at 3 hr; 16.2 in. at 4 hr.

REPORT: "Fire Resistance of Prestressed Concrete Beams. Study C. Structural Behavior During Fire Tests," by A.H. Gustaferro, M.S. Abrams, and E.A.B. Salse. PCA R&D Bulletin RD 009.01B, 1971.

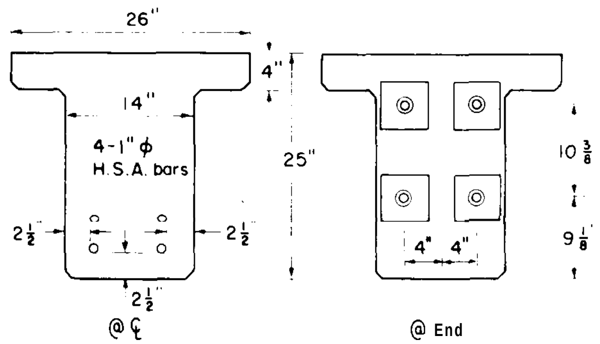
DURATION OF TEST: 4 hr 29 min.

7.8.8

DATE OF TEST: May 4, 1965

PLACE OF TEST: Fire Research Laboratory, Portland Cement Association, Skokie, I II.

SECTION TESTED: Tee beam, post-tensioned, bonded.



PRESTRESSING REINFORCEMENT: Four l-in. diameter high strength alloy steel bars, unbonded.

MINIMUM CONCRETE COVER: 2% in.

AGGREGATE: Expanded shale.

LOAD DURING TEST: 1,680 lb per ft. superimposed load.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM El 19.

PERTINENT TEST DATA: Tested as a simply supported beam on a 40-ft span with no end restraint.* Spalling occurred about 10 ft from one end at about 10 min. Spalling continued in an area about 2 ft long and 10 in. wide on the bottom of the beam to a maximum depth of about 6 in. Average steel temperatures at midspan: 350F at 2 hr; 540F at 3 hr; 730F at 4 hr; 895F at 5 hr. Midspan deflections: 4.2 in. at 2 hr; 6.4 in. at 3 hr; 10.4 in. at 4 hr; and 22.9 in. at 5 hr.

REPORT: "Fire Resistance of Prestressed Concrete Beams. Study C. Structural Behavior During Fire Tests," by A.H. Gustaferro, M.S. Abrams, and E.A.B. Salse. PCA R&D Bulletin RD 009.018, 1971.

DURATION OF TEST: 5 hr 01 min.

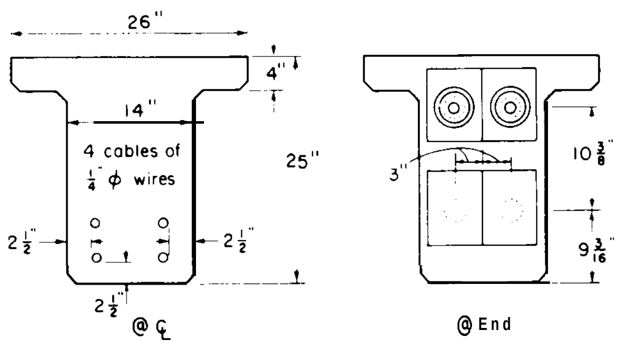
*Supported on rocker-roller bearings, thus end restraint was minimal.

7.8.9

DATE OF TEST: February 24, 1965

PLACE OF TEST: Fire Research Laboratory, Portland Cement Association, Skokie, Ill.

SECTION TESTED: Tee beam, post-tensioned, unbonded



PRESTRESSING REINFORCEMENT: Four cables each of which consisted of twelve ¼-in. button-headed cold-drawn wires, unbonded.

MINIMUM CONCRETE COVER: 2% in.

AGGREGATE: Carbonate sand and gravel from Elgin, Illinois.

LOAD DURING TEST: 1,135 lb per ft. superimposed load.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM El 19.

PERTINENT TEST DATA: Tested as a simply supported beam on a 40-ft span with no end restraint.* Average steel temperatures at midspan: 270F at 1 hr; 500F at 2 hr; 715F at 3 hr. Midspan deflections: 4.2 in. at 1 hr; 5.5 in. at 2 hr; 13.5 in. at 3 hr. Corner spalls occurred early in the test. Cracking near midspan began at about 2 hr 30 min.

REPORT: "Fire Resistance of Prestressed Concrete Beams. Study C. Structural Behavior During Fire Tests," by A.H. Gustaferro, M.S. Abrams, and E.A.B. Salse. PCA R&D Bulletin RD 009.01 B, 1971.

DURATION OF TEST: 3 hr 04 min.

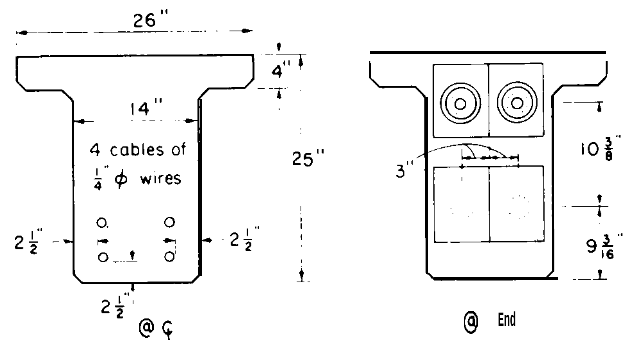
*Supported on rocker-roller bearings, thus end restraint was minimal.

7.8.10

DATE OF TEST: December 10, 1964

PLACE OF TEST: Fire Research Laboratory, Portland Cement Association, Skokie, I ll.

SECTION TESTED: Tee Beam, post-tensioned, bonded.



PRESTRESSING REINFORCEMENT: Four cables each of which consisted of twelve '/o-in. button-headed cold-drawn wires, bonded.

MINIMUM CONCRETE COVER: 2% in.

AGGREGATE: Carbonate sand and gravel from Elgin, Illinois.

LOAD DURING TEST: 1,750 lb per ft superimposed load.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM E 119.

PERTINENT TEST DATA: Tested as a simply supported beam on a 40-ft span with no end restraint. * Average steel temperatures at midspan: 235F at 1 hr; 410F at 2 hr; 590F at 3 hr. Midspan deflections: 5.3 in. at 1 hr; 6.3 at 2 hr; 14.1 at 3 hr. No spalling occurred. Cracking near midspan began at about 2 hr 20 min.

REPORT: "Fire Resistance of Prestressed Concrete Beams. Study C. Structural Behavior During Fire Tests," by A.H. Gustaferro, M.S. Abrams, and E.A.B. Salse. PCA R&D Bulletin RD 009.018, 1971.

DURATION OF TEST: 3 hr 20 min.

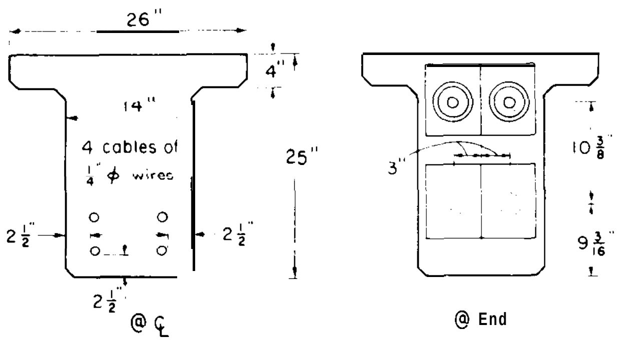
*Supported on rocker-roller bearings, thus end restraint was minimal.

7.8.11

DATE OF TEST: April 20, 1965

PLACE OF TEST: Fire Research Laboratory, Portland Cement Association, Skokie, Ill.

SECTION TESTED: Tee Beam, post-tensioned, bonded,



PRESTRESSING REINFORCEMENT: Four cables each of which consisted of eleven %-in. button-headed cold-drawn wires, bonded.

MINIMUM CONCRETE COVER: 2% in.

AGGREGATE: Expanded shale from Ottawa, Illinois.

LOAD DURING TEST: 1,740 lb per ft superimposed load.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM E 119.

PERTINENT TEST DATA: Tested as a simply supported beam on a 40-ft span with no end restraint.* Average steel temperatures at midspan: 230F at 2 hr; 360F at 3 hr; 520F at 4 hr. Midspan deflections: 4.3 in. at 1 hr; 4.5 at 2 hr; 5.6 at 3 hr; and 10.4 at 4 hr. Some minor spalling occured early in the test.

REPORT: "Fire Resistance of Prestressed Concrete Beams. Study C. Structural Behavior During Fire Tests," by A.H. Gustaferro, M.S. Abrams, and E.A.B. Salse. PCA R&D Bulletin RD 009.016, 1971.

DURATION OF TEST: 4 hr 33 min.

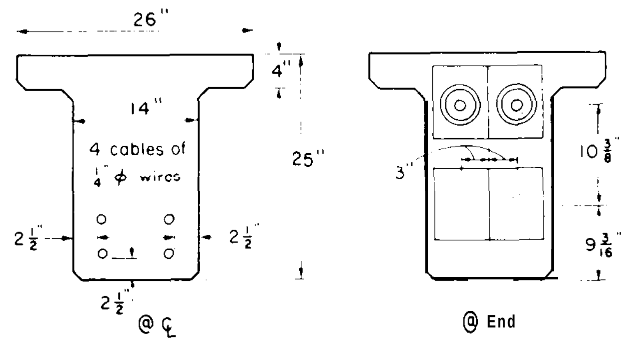
* Suppoı ted on rocker-roller bearings, thus end restraint was minimal.

7.8.12

DATE OF TEST: March 16, 1965

PLACE OF TEST: Fire Research Laboratory, Portland Cement Association, Skokie, Ill.

SECTION TESTED: Tee Beam, post-tensioned, bonded.



PRESTRESSING REINFORCEMENT: Four cables each of which consisted of twelve ¼-in. button-headed cold-drawn wires, bonded.

MINIMUM CONCRETE COVER: 2% in.

AGGREGATE: Carbonate sand and gravel from Elgin, Illinois.

LOAD DURING TEST: 1,760 lb per ft superimposed load.

TEST SPECIFICATIONS: Standard Methods of Fire Tests of Building Construction and Materials, ASTM E 119.

PERTINENT TEST DATA: Tested as a simply supported beam on a 40-ft span with no end restraint.* Average steel temperatures at midspan: 230F at 1 hr; 410F at 2 hr; 620F at 3 hr. Midspan deflections: 5.3 in. at 1 hr; 7.1 at 2 hr; 15.4 at 3 hr. Corner spalls occurred early in the test. Cracking near midspan began at about 2 hr 30 min.

REPORT: "Fire Resistance of Prestressed Concrete Beams. Study C. Structural Behavior During Fire Tests," by A.H. Gustaferro, M.S. Abrams, and E.A.B. Salse. PCA R&D Bulletin RD 009.01B, 1971.

DURATION OF TEST: 3 hr 18 min.

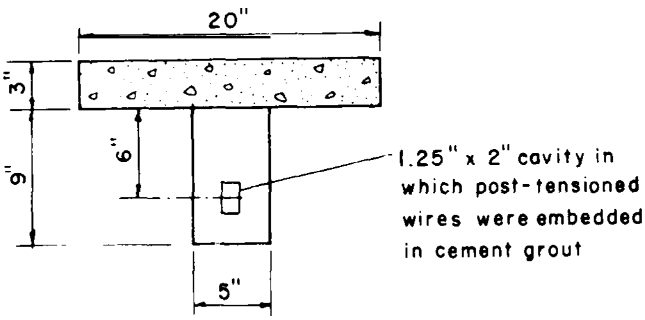
*Supported on rocker-roller bearings, thus end restraint was minimal.

7.8.13

DATE OF TEST: September 30, 1953

PLACE OF TEST: National Bureau of Standards, Washington, D.C.

SECTION TESTED: Rectangular beam with composite slab.



PRESTRESSING REINFORCEMENT: 32 wires 0.10 in. dia. in 4 vertical rows of 8 each, all embedded in cement grout. Post-tensioned.

MINIMUM CONCRETE COVER: 2 in. bottom and 1.9 in. side.

AGGREGATE: Sand and gravel.

LOAD DURING TEST: Dead load plus 1.5 live loads. Total load was 24,400 lbs. Live load concentrated equally at four points.

TEST SPECIFICATIONS: British Standard 476, of 1932. (This is comparable to ASTM E 119 except for loading. Time-temperature relationships are almost identical.)

PERTINENT TEST DATA: Beam was simply supported on 10-ft. span. Centerline deflections: 1.2 in. at 30 min.; 1.6 in. at 1 hr.; 3.8 in. at 1 hr. 28 min. Load was removed and test was stopped when failure appeared to be imminent by yield of prestressing steel. No spalling occurred. Twenty-four hours after load was removed deflection was 1.2 in. Beam was then reloaded, and at 1.5 live load deflection was 3.6 in.

REPORT: "Fire Resistance of Prestressed Concrete Beams," by L.A. Ashton and S. C. C. Bate., Journal ACI, May 1961.

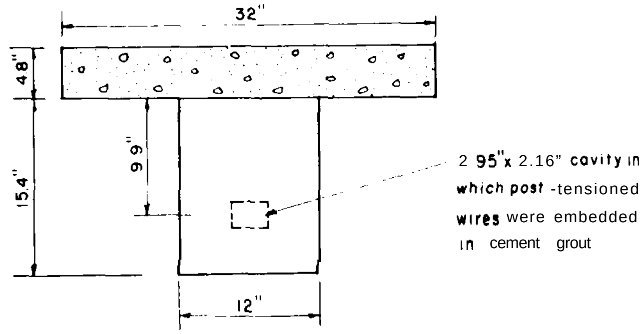
DURATION OF TEST: 1 hr. 28 min.

7.8.14

DATE OF TEST: October 12, 1953

PLACE OF TEST: National Bureau of Standards, Washington, D.C.

SECTION TESTED: Rectangular Beam with Composite slab.



PRESTRESSING REINFORCEMENT: 24 wires 0.20 in. dia. embedded in cement grout. Post-tensioned.

MINIMUM CONCRETE COVER: 4.7 in. bottom and 4.9 in. side.

AGGREGATE: Sand and gravel.

LOAD DURING TEST: Dead load plus 1.5 live loads. Total load was 58,200 lbs. Live load concentrated equally at four points.

TEST SPECIFICATIONS: British Standard 476, of 1932. (This is comparable to ASTM E 119 except for loading. Time-temperature relationships are almost identical.)

PERTINENT TEST DATA: Beam was simply supported on a 16-ft. span. End point was reached at 3 hr. 40 min. by collapse of the beam after concrete cover collapsed.

REPORT: "Fire Resistance of Prestressed Concrete Beams," by L.A. Ashton and S.C.C. Bate, Journal ACI, May 1961.

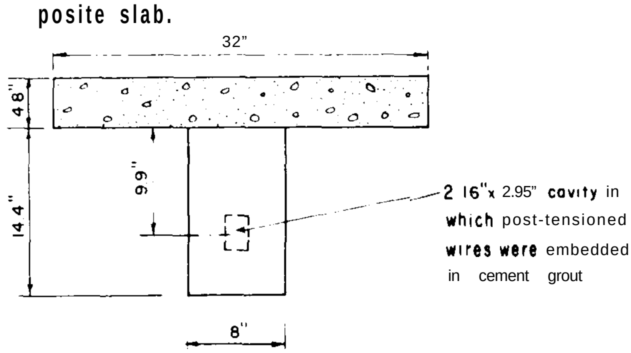
DURATION OF TEST: 3 hr. 40 min.

7.8.15

DATE OF TEST: October 19, 1953

PLACE OF TEST: National Bureau of Standards, Washington, D.C.

SECTION TESTED: Rectangular beam with composite slab.



PRESTRESSING REINFORCEMENT: 24 wires 0.20-in. dia. embedded in cement grout. Post-tensioned.

MINIMUM CONCRETE COVER: 3.4 in. bottom and 3.2 in. side.

AGGREGATE: Sand and gravel.

LOAD DURING TEST: Dead load plus 1.5 live loads. Total load was 59,600 lbs. Live load applied equally at four points.

TEST SPECIFICATIONS: British Standard 476, of 1932. (This is comparable to ASTM E 119 except for loading. Time-temperature relationships are almost identical.)

PERTINENT TEST DATA: Beam was simply supported on 16-ft. span. End point was reached at 2 hr. 35 min. after the concrete cover collapsed, at which time 15 of the 24 wires broke.

REPORT: "Fire Resistance of Prestressed Concrete Beams," by L. A. Ashton and S.C.C. Bate, Journal ACI, May 1961.

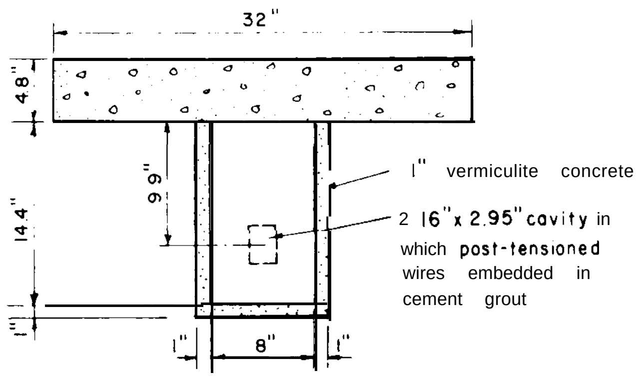
DURATION OF TEST: 2 hr. 35 min.

7.8.16

DATE OF TEST: October 30, 1953

PLACE OF TEST: National Bureau of Standards, Washington, D.C.

SECTION TESTED: Rectangular beam with composite slab coated with vermiculite concrete.



PRESTRESSING REINFORCEMENT: 24 wires 0.20-in. dia. embedded in cement grout. Post-tensioned.

MINIMUM CONCRETE COVER: 3.4 in. bottom and 3.2 in. side. Also 1 in. vermiculite concrete.

AGGREGATE: Sand and gravel.

LOAD DURING TEST: Dead load plus 1.5 live loads. Total load was 59,600 lbs. Live load applied equally at four points.

TEST SPECIFICATIONS: British Standard 476, of 1932. (This is comparable to ASTM E 119 except for loading. Time-temperature relationships are almost identical.)

PERTINENT TEST DATA: Beam was simply supported on a 16-ft. span. Beam was subjected to fire for over 6 hours and had not failed when test was stopped.

REPORT: "Fire Resistance of Prestressed Concrete Beams," by L.A. Ashton and S.C.C. Bate, Journal ACI, 1961.

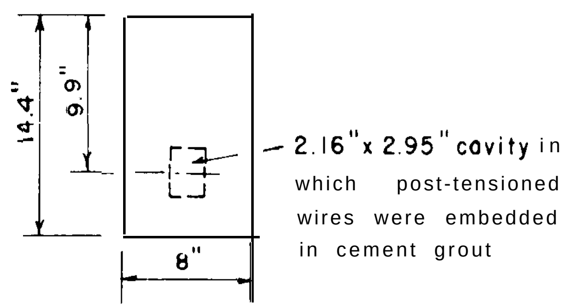
DURATION OF TEST: 6 hr. 3 min.

7.8.17

DATE OF TEST: November 9, 1953

PLACE OF TEST: National Bureau of Standards, Washington, D.C.

SECTION TESTED: Rectangular beam



PRESTRESSING REINFORCEMENT: 24 wires 0.20-in. dia. embedded in cement grout. Post-tensioned.

MINIMUM CONCRETE COVER: 3.4 in. bottom and 3.2 in. side.

AGGREGATE: Sand and gravel.

LOAD DURING TEST: Dead load plus 1.5 live loads. Total load was 26,200 lbs. Live load applied equally at four points.

TEST SPECIFICATIONS: British Standard 476, of 1932. (This is comparable to ASTM E 119 except for loading. Time-temperature relationships are almost identical.)

PERTINENT TEST DATA: Beam was simply supported on a 16-ft. span. End point was reached at 1 hr. 55 min., after concrete cover collapsed, at which time the beam collapsed.

REPORT: "Fire Resistance of Prestressed Concrete Beams," by L.A. Ashton and S.C.C. Bate, Journal ACI, May 1961.

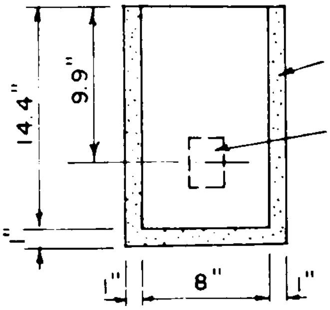
DURATION OF TEST: I hr. 55 min.

7.8.18

DATE OF TEST: November 18,1953

PLACE OF TEST: National Bureau of Standards, Washington, D.C.

SECTION TESTED: Rectangular beam coated with vermiculite concrete



PRESTRESSING REINFORCEMENT: 24 wires 0.20-in dia. embedded in cement grout. Post-tensioned.

MINIMUM CONCRETE COVER: 3.4 in. bottom and 3.2 in. side. Also l-in. vermiculite concrete.

AGGREGATE: Sand and gravel.

LOAD DURING TEST: Dead load plus 1.5 live loads. Total load was 26,200 Ibs. Live load applied,equally at four points.

TEST SPECIFICATIONS: British Standard 476, of 1932. (This is comparable to ASTM El 19 except for heading. Time-temperature relationships are almost identical.)

PERTINENTS TEST DATA: Beam was simply supported on a 16-ft. span. End point was reached at 4 hr. 39 min. when failure by yielding of prestressing steel appeared to be imminent.

REPORT: "Fire Resistance of Prestressed Concrete Beams," by L.A. Ashton and S.C.C. Bate, Journal ACI, May 1961.

DURATION OF TEST: 4 hr. 39 min.

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Appendix and Design Aids

A.1 MOMENT COEFFICIENTS FOR CONTINUOUS POST-TENSIONED STRUCTURES*

Tables are presented to simplify the computation of moments over the supports in continuous structures under post-tensioning loads. Coefficients are provided for two-span structures and for symmetric structures of three or more spans. Tendon profiles are parabolic segments. A procedure accounting for friction losses is included.

The bending moments in a beam continuous over several supports produced by post-tensioned prestressing tendons are usually computed by the equivalent load method as presented by Moorman[1]. All the forces between the tendon and the concrete are applied to the concrete beam, in effect as an exterior load assuming the tendons to be omitted. The elastic analysis of continuous beams under these loads presents no theoretical difficulties; however, it is tedious if performed manually by moment distribution, slope deflection or similar methods. Generally, these methods involve two steps: the computation of fixed end moments; and the elastic distribution of these moments. The second step is explained in any text on structural analysis[2]. The computation of fixed end moments is simplified by various charts and tables. Formulas and graphs for a variety of conditions are presented by Parme and Paris[3] and tables for beams of constant cross section are presented by Bailey and Ferguson[4].

This paper presents tables which simplify the bending moment computations for multispan beams with typical draped parabolic profile tendons. The restrictions are that the beams must be prismatic between supports; moreover, for three or more spans, the geometry of the structure must be symmetric. Except for the two-span case, coefficients are given only for tendon geometry that is symmetric about the centerline of the structure. Within these restrictions, the coefficients are given for a range of geometry parameters which covers the designs usually encountered. The determination of moments in beams with long tendons, where friction losses must be taken into account, is also considered, both for symmetric tensioning from both ends and for tensioning from only one end. The method presented here is not very cumbersome and should be suitable for general engineering use.

Problems beyond the scope of this paper, such as general variation of cross section or nonsymmetric structures with more than two spans, can be analyzed by slope deflection methods with the fixed end moments computed using the curves developed by Parme and Paris[3]. Fixed end moments for cubic paradola tendon profiles can be computed using formulas presented by Fiesenheiser[5] and those for sine curve tendon profiles can be computed using graphs presented by Parme and Paris. The moments due to post-tensioning can also be obtained by using a general digital computer program for frame analysis, such as STRUDL[6], if it allows members of the desired shape with the equivalent loads as the applied loading. None of these methods consider the continuity between the beam and its supporting columns, except for STRUDL where this effect may be taken into account.

Equivalent Loads

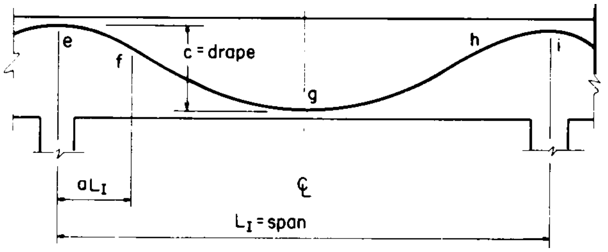
The equivalent vertical distributed tendon load imposed on any point of the beam is computed as the product of the curvature of the tendon profile and the horizontal component of the tendon force at that point. It is usually accurate enough to consider the horizontal tendon force at each point equal to the total tendon force, particularly if the drape of the tendon profile is less than 4% of the span length.

The tendon profile in an interior span is in the shape of three parabolic segments, shown in Fig. l(a) as ef, fgh, and hi. Segments ef and hi are the reversed parabolas. Points e, g and i are the horizontal points of the parabolas and points f and h are points of common tangency. The profile is assumed to be symmetric about the centerline of the span with its high point over the supports and low point at the span centerline.

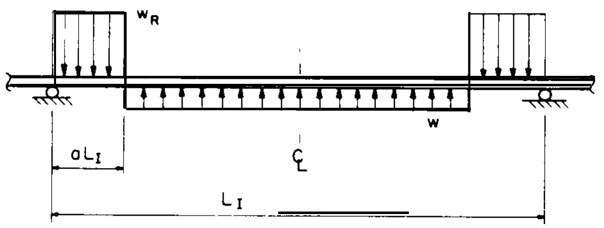
The corresponding idealized structure, with the equivalent loads applied, is shown in Fig. l(b). The magnitude of the upward distributed loads from the main portion of the tendon is

$$w = \frac{8Pc}{(1 - 2a)L_I^2} \qquad (1)$$

*From paper in Journal of the Prestressed Concrete Institute, January-February, 1972, by Peter Turula and Clifford L. Freyermuth.

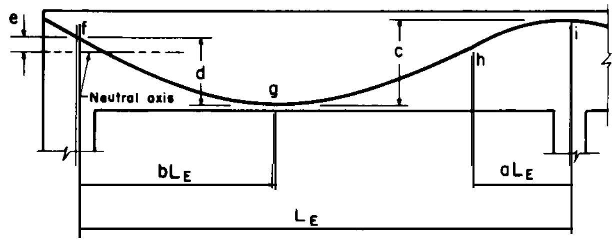


(a) Tendon profile geometry

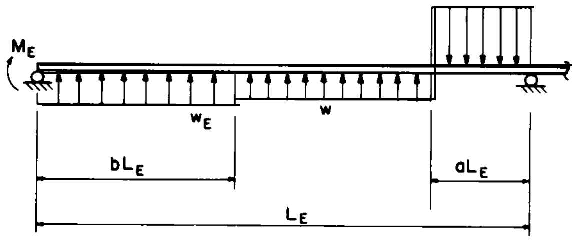


(b) Equivalent load

Fig. 1 — Typical interior span



(a) Tendon profile geometry



(b) Equivalent load

Fig. 2 — Typical exterior span

where P is the horizontal tendon force component. The downward load at the reverse parabola segment is

$$w_R = \frac{1-2a}{2a} w \tag{2}$$

However, it need not be considered separately when using the tables presented in this paper.

A typical exterior span, as shown in Fig. 2(a), has a tendon profile which consists of three parabolic segments: fg, gh, and hi. Segments fg and gh have a common horizontal low point at g; segments gh and hi have a common tangent at h; and the reversed parabola segment, hi, has a horizontal high point at i directly over the support.

The equivalent tendon load acting on the exterior span is considered in three parts, Fig. 2(b). First, the end moment

$$M_E = Pe \tag{3}$$

Second, the upward load due to the tendons in the external parabola segment where the upward prestress load is given by

$$w_E = \frac{2Pd}{b^2 L_E{}^2} \tag{4}$$

And, third, the load due to the remaining part of the tendons for which the upward segment of the prestress load is given by

$$w = \frac{2Pc}{(1-b)(1-b-a)L_E{}^2}$$

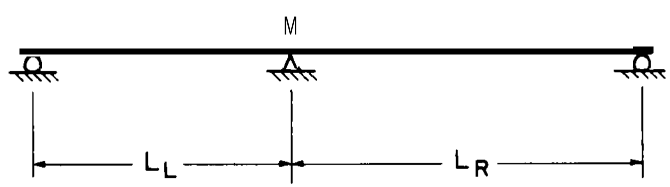
Moment Influence Coefficients

Tables I through VI are to be used in computation of beam moments at the support points. These tables are intended for beams of constant cross section over all spans, but may be used if the section changes from span to span, as explained later. Table I covers the 2-span beam for which the two spans may or may not be the same. Tables II to VI cover 3-, 4- and 5-span beams for which the geometry of the structure must be symmetric. A beam of more than five spans can be analyzed by taking the additional interior

TAPLF I INFLUENCE SEGMENT COFFFICIENTS FOR 2 SPANS - - I-ST INTERIOR SUPPORT

* NUMBERS REFFR TO PFR CFNT OF SPAN LOADING DESCRIPTION	REVERSE CURVF	0.650	0.700	0.750	0.800	0.850	0.900	0.950	1.000
		- - RATIO OF LEFT SPAN LENGTH TO RIGHT SPAN LENGTH - -							
- LOAD ON ONE SPAN ONLY -									
FIRST 30 OF FIRST SPAN	00	0.003576	0.004335	0.005180	0.006111	00007132	0.008244	00009447	0.010743
LAST 70 OF FIQST SPAN	05	0.014459	0.01'528	0.020943	0.024711	0.028839	0.033333	0.038197	0.043438
LAST 70 OF FIQST SPAN	10	0.011971	0.014511	0.017339	0.020458	0.023876	0~027596	0.031623	0.035962
LAST 70 OF FIQST SPAN	15	0.009751	0.011821	0.314124	0.016666	0.019450	0.022480	0.025761	0-029295
FIRST 40 OF FIQST SPAN	00	0.006124	0.007424	0.008871	0.010467	3.012216	3.014119	0.016180	0.018399
LAST 60 OF FIRST SPAN	05	3.017306	0.014918	3.017824	0.021031	0.024545	0.028369	0.032510	0.036970
LAST 60 OF FIQST SPAN	10	C.010173	0.312332	0.014735	0.017386	0.020290	0.023452	0.026075	0.030562
LAST 60 OF FIQST SPAN	15	0.008771	0.017077	0.011980	0.014135	0.016497	0.019067	0.021850	0.024848
FIRST 50 OF FIRST SPAN	00	0.009192	3.011034	3.013183	0.015555	0.018154	0.020982	0.024044	0.027343
LAST 50 OF FIRST SPAN	05	0.009724	0.011789	3.014085	3.016619	0.019396	0.022418	00025690	0.029214
LAST 5" OF FIRST SPAN	10	0.007947	0.000634	0.011511	0.013582	0.015851	0.018320	0.020994	0.023074
LAST 50 OF FIQST SPAN	15	0.006367	0.307712	0.009215	09010873	0.012689	0.014667	0.016807	0.019113
LAST 30 OF LAST SPAN	00	0.017072	0.011679	0.012278	0.011937	0.011614	0.011309	0.011019	0.010743
FIRST 70 OF LAST SPAY	05	0.052652	0.051103	0.349643	0.048264	0.046960	0.045724	0.044552	0.043438
FIRST 70 OF LAST SPAN	10	0.043590	0.042308	0.041099	0.039958	0.038878	0.037855	0.036884	0.035962
FIRST 70 OF LAST SPAN	15	0.035510	0.074465	0.033481	0.032551	0.031671	0.030837	0.030047	0.029295
LAST 40 OF LAST SPAN	00	0.022303	0.021647	0.021028	0.020444	0.019891	3.019368	0.018871	0.018400
FIRST 60 OF LAST SPAN	C5	0.044812	0.041494	0.042251	0.041078	0.039967	0.038916	0.037918	0.036970
FIRST 60 OF LAST SPAN	1"	0.037045	3.035955	0.034928	0.033958	0.033040	0.032171	0.031346	00030562
FIRST 60 OF LAST SPAN	15	0.030119	0.029233	0.028398	0.027609	0.026863	09026156	0.025485	00024848
LAST 50 OF LAST SPAN	00	0.033143	0.072169	0.031250	0.030381	0.029560	0.028782	0.028044	0.027343
FIRST 50 OF LAST SPAN	C5	0.035411	0.034370	0.033388	0.032460	0.031583	0.030752	0.029963	0.029214
FIRST 50 OF LAST SPAN	10	0.028939	0.028088	0.027285	0.026527	0.025810	0.025131	C.024487	0.023075
FIRST 50 OF LAST SPAN	15	0.073167	0.022486	0.071843	0.021237	0.023663	0.020119	0.019603	0.019113
UNIT MOMENT ON LEFT END		-0.196969	-0.205882	-0.214285	-0.222222	-0.229729	-0.236842	-0.243589	-0.250000
UNIT MOMENT ON RIGHT END		-0.303030	-0.294117	-0.285714	-0.277777	-0.270270	-0.263157	-0.256410	-0.250000
- - - APPLIED LOADS - - -									
UNIT DEAD LOAD ON 1-ST SPAN		-0.020804	-0.025220	-0.030133	-0.035555	-0.041494	-0.047960	-0.054959	-0.062500
UNIT DEAD LOAD ON 2-ND		-0.C75757	-0.073529	-0.071428	-0.069444	-0.067567	-0.065789	-0.064102	-0.062500
UNIT " A " LOAD ON BOTH SPANS		-0.096562	-0.098749	-0.101562	-0.104999	-0.109062	-0.113749	-0.119062	-0.125000

* For example, in the third line of coefficients thr 70 and 10 indicate 70% and 10% of the first span.

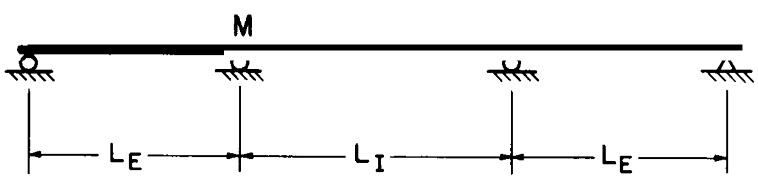


$$M = \left[\sum w \cdot \text{coef.} \right] \cdot L_R^2 + \sum M_E \cdot \text{coef.}$$

TABLE I INFLUENCE SEGMENT COEFFICIENTS FOR 3 SPANS - - I-ST INTERIOR SUPPORT

NUMBERS REFER TO PER CENT OF SPAN — — RATIO OF EXTERIOR SPAN LENGTH TO INTERIOR SPAN LENGTH — —

LOADING DESCRIPTION	REVERSE CURVE	0.650	0.700	0.750	0.800	0.850	0.900	0.950	1.000
- - SYMMETRIC PRESTRESS - -									
END 30 OF END SPANS (SYM)	00	0.002744	0.003350	0.004028	0.004703	0.005615	00006526	0.007519	0.008594
INNER 70 OF END SPANS	05	0.011096	0.013544	0.016289	0.019339	0.022703	0.026388	0.030402	0.034750
INNER 70 OF END SPANS	10	0.009187	0.011213	0.013485	0.016011	0.018796	0.021847	0.025170	0.028769
INNER 70 OF END SPANS	15	0.007484	0.009134	0.010985	0.013043	0.015311	0.017797	0.020504	0.023436
END 40 OF END SPANS	00	0.004700	0.005737	0.006899	0.008191	0.009616	0.011177	0.012878	0.014719
INNER 60 OF END SPANS	05	0.009444	0.011527	0.013863	09016459	0.019322	09022459	0.025875	0.029576
INNER 60 OF END SPANS	10	0.007807	0.009529	0.011460	0.013606	0.015973	0.018566	0.021390	0.024449
INNER 60 OF END SPANS	15	0.006347	0.007748	0.009318	0.011062	0.012987	09015095	0.017391	0.019878
END 50 OF END SPANS	00	0.006985	0.008526	0.010253	0.012173	0.014291	0.016611	0.019137	0.021875
INNER 50 OF END SPANS	05	0.007463	0.009109	0.010955	0.013006	0.015269	0.017747	0.020447	0.023371
INNER 50 OF END SPANS	10	0.006099	0.007444	0.000953	0.010629	0.012478	0.014504	0.016710	0.019099
INNER 50 OF END SPANS	15	0.004882	0.005959	0.007167	0.008509	0.009989	0.011611	0.013377	0.015290
CENTER SPAN	05	0.049709	0.044579	0.047499	0.046467	0.045478	0.044531	0.043622	09042749
CENTER SPAN	10	0.041860	0.040909	0.039999	0.039130	0.038297	0.037499	0.036734	0.035999
CENTER SPAN	15	0.034593	0.073806	0.033055	0.032337	0.031649	0.030989	0.030357	0.029750
UNIT MOMENTS ON THE ENDS		-0.151167	-0.159090	-0.166666	-0.173913	-0.180851	-0.187499	-0.193877	-0.200000
- ANTI-SYMMETRIC PRESTRESS -									
END 30 OF END SP. (ANTI-	00	0.005131	0.006141	0.007257	0.008462	0.009774	0.011188	0.012705	0.014324
INNER 70 OF END SP. -SYM)	05	0.020746	0.024932	0.029320	0.034215	0.039520	0.045237	0.051369	0.057917
INNER 70 OF END SP.	10	0.017175	0.020558	0.024274	0.028327	0.032719	0.037452	0.042528	0.047949
INNER 70 OF END SP.	15	0.013990	0.016745	0.019772	0.023073	0.026650	0.030506	0.034641	0.039056
END 40 OF END SP.	00	0.008787	0.010518	0.012419	0.014493	0.016740	0.019162	0.021759	0.024533
INNER 60 OF END SP.	05	0.017657	0.021134	0.024954	0.029121	0.033636	0.038501	0.043720	0.049293
INNER 60 OF END SP.	10	00014596	0.017471	0.020629	0.024073	09027806	0.031828	09036142	0.040749
INNER 60 OF END SP.	15	0.011867	0.014204	0.016772	0.019572	0.022607	0.025877	0.029385	0.033131
END 50 OF END SP.	00	0.017059	0.015631	0.018457	0.021538	0.024877	0.028476	0.032336	0.036458
INNER 50 OF END SP.	05	0.013953	0.016701	0.019719	0.023012	0-026580	0.030425	0.034548	0.038953
INNER 50 OF END SP.	10	0.011402	0.013648	0.016115	0.018806	0.021721	0.024864	0.028234	0.031833
INNER 50 OF END SP.	15	0.009130	0.010928	0.012903	0.015058	0.017392	0.019908	0.022607	0.025488
UNIT MOMENTS ON THE ENDS		-0.282608	-0.291666	-0.300000	-09307692	-0.314814	-0.321428	-0.327586	-0.333333
- - - APPLIED LOADS - - -									
UNIT DEAD LOAD ON I-ST SPAN		-0.027908	-0.027608	-0.032812	-0.038528	-0.044764	-0.051528	-0.058827	-0.066666
UNIT DEAD LOAD ON 2-ND		-0.058139	-0.056818	-0.055555	-01054347	-0.053191	-0.052083	-0.051020	-0.050000
UNIT DEAD LOAD ON 3-RD		0.006941	0.008120	0.009375	0.010702	0.012098	0~013560	0.015084	0.016666
UNIT DEAD LOAD ON ALL SPANS		-0.074106	-0.076306	-0.078993	-0.082173	-09085857	-0.090052	-0.094764	-0.100000
UNIT D.L. ON SPANS 1 AND 2		-0.081048	-0.084477	-0.088368	-0.092876	-0e097956	-0.103612	-0.109840	-0.116666
UNIT D.L. ON SPANS 1 AND 3		-3.015966	-0.019488	-0.023437	-0.027826	-0~032666	-0.037968	-0.043743	-0.050000



$$M = \left[\sum w \cdot \text{coef.} \right] \cdot L_I^2 + \sum M_E \cdot \text{coef.}$$

TABLE III INFLUENCE SEGMENT COEFFICIENTS FOR 4 SPANS -- I-ST INTER IOR SUPPORT

NUMBERS REFER TO PER CENT OF SPAN LOADING DESCRIPTION	REVERSE CURVE	0.650	0.700	0.750	0.800	0.850	0.900	0.950	1.000
		RATIO OF EXTERIOR SPAN LENGTH TO INTERIOR SPAN LENGTH							
-- SYMMETRIC PRESTRESS --									
END 70 OF EN@ SPANS (SYM)	00	0.004215	0.005082	0.006043	0.007097	0.008247	0.009493	0.010836	0.012278
INYER 70 OF END SPANS	05	0.017041	0~020550	0.024433	0.028697	0.033345	0.038383	0.043815	00049643
INYER 70 OF END SPANS	10	0.014108	0.017013	0.020228	0.023758	0.027606	0.031777	0.036274	0.041099
INNER 70 OF END SPANS	15	0.011493	0.013859	0.016478	09019354	0.022489	0.025886	0.029550	0.033480
END 40 OF END SPANS	00	0.007218	0.008705	C.010349	0.012155	0.014124	0.016258	0.018559	0.021028
INNER 60 OF ENDS SPANS	05	0.014504	0.017490	0.020795	0.024424	0.028380	0.032668	0.037290	0.042251
INNER 60 OF END SPANS	10	0.011990	0.014459	0.017191	0.020190	0.023461	0.027006	0.030827	0.034928
INNER 60 OF END SPANS	15	0.009748	0.011755	C.013977	0.016415	0.019075	0.021956	0.025063	0.028398
END 50 OF END SPANS	00	0.010727	0.012936	0.015380	0.018064	0.020990	0.024161	0.027580	0.031250
INNER 50 O F END SPANS	05	0.011461	0.013821	0.016433	0.019300	0.022426	0.025815	0.029468	0.033388
INNER 50 OF END SPAYS	10	C.009366	0.011295	0.013429	0.015772	09018327	0.021096	0.024082	0.027285
INNER 50 OF END SPAYS	15	0.007498	0.009042	0.010751	0.012627	00014672	0.016889	0.019279	0.021843
TWO MIDDLE SPAYS	05	0.038169	0.036853	0.035624	0.034475	0.033398	0.032386	0.031433	0.030535
TWO MIDDLE SPANS	10	0.032142	0.071034	0.029999	0.029032	0.028124	0.027272	0.026470	0.025714
TWO MIDDLE SPAYS	15	0.026562	0.025646	0.024791	09023992	0.023242	0.022537	0.021875	0.021250
UNIT MOMENTS ON T H E ENDS		-0.232147	-0.241379	-0.250000	-0.258064	-0.265625	-0.272727	-0.279411	-09285714
- ANTI-SYMMETRIC PRESTRESS -									
END 30 O F END SP. (ANTI-	00	0.003576	0.004335	0.005180	0.006111	0.007132	0.008244	0.009447	09010743
INNER 7 0 OF END SP. -SYM)	05	0.014459	0.017528	0.020943	0.024711	0.028839	0-033333	0.038197	0.043438
INNER 7 0 O F END SP.	10	0.011971	0.014511	0.017339	0.020458	0.023876	01027596	0.031623	0.035962
1 NNER 70 O F END SP.	15	0.009746	0.011815	0.014117	0.016657	0.019439	0.022468	0.025747	0.029279
END 40 OF END SP.	00	0.006124	0.007474	C.008871	0.010467	0.012216	0~014119	0.016180	0-018399
INNER 60 OF END SP.	05	0.012306	0.014918	0.017824	0.021031	0.024545	0.028369	0.032510	0.036970
INNER 60 O F END SP.	10	0.010173	0.012332	0.014735	0.017386	0.020291	0.023452	0.026875	0.030562
INNER 60 OF END SP.	15	0.008271	C.010027	0.011980	0.014135	0-016497	0.019067	0.021850	0.024848
END 5 0 OF END SP.	00	0.009102	0.011034	0.013183	0.015555	0.018154	0.020982	0.024044	0.027343
INNER 50 O F END SP.	05	0.009724	0.011789	C.014085	0.016619	0.019396	00022418	0.025690	0.029214
INNER 5 0 OF END SP.	10	0.007947	0.009634	0.011511	0.013582	0.015851	0~018320	0.020994	0-023874
INNER 50 OF END SP.	15	0.006367	0.007719	0.009223	0.010882	0.012700	0.014679	0.016821	0.019129
TWO MIDDLE SPANS	05	0.064772	0.062867	0.061071	0.059374	0.057770	0.056249	0.054807	09053437
TWO MIDDLE SPANS	10	0.054545	0.052941	0.051428	0.049999	0.048648	0.047368	0.046153	0.044999
TWO MIDDLE SPANS	15	0~045106	0.043779	0.042528	0.041347	0.040229	0.039171	0.038166	0.037212
UNIT MOMENTS O N THE ENDS		-0.196969	-0.205882	-0.214285	-0.222222	-0.229729	-0.236842	-0.243589	-0.249999
- - - APPLIED LOADS - - -									
UNIT DEAD LOAD ON 1-ST SPAN		-0.022662	-0.027394	-0.032645	-0.038422	-0.044736	-0.051593	-0.059001	-0.066964
UNIT DEAD LOAD ON 2-ND		-0.060200	-0.058316	-0.056547	-0.054883	-0.053315	-0.051834	-0.050433	-09049107
UNIT DEAD LOAD ON 3-RD		0.015557	0.015212	0.014880	0.014560	0.014252	0.013955	0.013668	0.013392
UNIT DEAD LOAD O N 4-TH		-0.001857	-0.002174	-0.002511	-0.002867	-0.003241	-0.003633	-0.004041	-0.004464
UNIT DEAD LOAD ON ALL SPANS		-0.369167	-0.072672	-0.076827	-0~081612	-0.087040	-0.093106	-0.099806	-00107142
UNIT D.L. ON SPANS 1,2 AND 4		-0.084720	-0.087885	-0.091703	-0.096173	-06101293	-0.107061	-0.113475	-0.120535
UNIT D.L. ON SPANS 7 AND 3		-0.044642	-0.043103	-0.041666	-0.040322	-0.039062	-0.037878	-0.036764	-0.035714
UNIT D.L. ON SPANS 1 AND 2		-0.007105	-0.012181	-0.017764	-0.023861	-0.030484	-0.037638	-0.045332	-0.053571
UNIT D.L. ON SPANS 7 AND 4		-0.062057	-0.060493	-0.059058	-0.057750	-0.056556	-0.055467	-0.054474	-0.053571

TABLE V INFLUENCE SEGMENT COEFFICIENTS FOR 4 SPANS -- 2-ND INTERIOR SUPPORT

NUMBERS REFER TO PER CENT OF SPAN -- RATIO OF EXTERIOR SPAN LENGTH TO INTERIOR SPAN LENGTH --

LOADING DESCRIPTION	REVERSE CURVE	0.650	0.700	0.750	0.800	0.850	0.900	0.950	1.000
-- SYMMETRIC PRESTRESS --									
END 30 OF END SPANS (SYM)	00	-0.002107	-0.002541	-0.003021	-0.003548	-0.004123	-0.004746	-0.005418	-0.006139
INNER 70 OF END SPANS	05	-0.008520	-0.010275	-0.312216	-0.014348	-0.016672	-0.019191	-0.021907	-0.024821
INNER 70 OF END SPANS	10	-0.007354	-0.008506	-0.010114	-0.011879	-0.013803	-0.015888	-0.018137	-0.020549
INNER 70 OF END SPANS	15	-0.005746	-0.006929	-0.008239	-0.009677	-0.011244	-0.012943	-0.014775	-0.016740
END 40 OF END SPANS	00	-0.003609	-0.304352	-0.005175	-0.006077	-0.007062	-0.008129	-0.009279	-0.010514
INNER 67 OF END SPANS	05	-0.007752	-0.008745	-0.010397	-0.012212	-0.014190	-0.016334	-0.018645	-0.021125
INNER 60 OF END SPANS	10	-0.005995	-0.007229	-0.008595	-0.010095	-0.011730	-0.013503	-0.015413	-0.017464
INNER 60 OF END SPANS	15	-0.004874	-0.005877	-0.006988	-0.008207	-0.009537	-0.010978	-0.012531	-0.014199
END 50 OF END SPANS	00	-0.005363	-0.006468	-0.007690	-0.009032	-0.010495	-0.012080	-0.013790	-0.015625
INNER 50 OF END SPANS	05	-0.005730	-0.006910	-0.008216	-0.009650	-0.011213	-0.012907	-0.014734	-3.016694
INNER 50 OF END SPANS	10	-0.004683	-0.005647	-0.306714	-0.007886	-0.009163	-0.010548	-0.012041	-0.013642
INNER 50 OF END SPANS	15	-0.003749	-0.004521	-9.005375	-0.006313	-0.007336	-0.008444	-0.009639	-0.010921
TWO MIDDLE SPANS	05	0.087789	0.088448	0.089062	0.089636	0~390175	0.090681	0.091157	0.091606
TWO MIDDLE SPANS	10	0.073929	0.074482	3.074999	0.075403	0.075937	0.076363	0.076764	00077142
TWO MIDDLE SPANS	15	0.061094	0.061552	0.061979	0.062379	0.062754	0.063106	0.063437	0.063750
UNIT MOMENTS ON THE ENDS		0.116071	0.120689	0.125000	0.129032	0.132812	0.136363	0.139705	0.142857
- ANTI-SYMMETRIC PRESTRESS -		0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000	0.000000
--- APPLIED LOADS ---									
UNIT DEAD LOAD ON 1-ST SPAY		0.006130	0.007392	0.008789	0.010322	0.011994	0.013806	0.015760	0.017857
UNIT DEAD LOAD ON 2-ND		-0.051339	-0.051724	-0.052083	-0.052419	-0.052734	-0.053030	-0.053308	-0.053571
UNIT DEAD LOAD ON 3-RD		-0.051339	-0.051724	-0.052083	-0.052419	-0.052734	-0.053030	-0.053308	-0.053571
UNIT DEAD LOAD ON 4-TH		0.006130	0.007392	0.008789	0.010322	0.011994	0.013406	0.015760	0.011057
UNIT DEAD LOAD ON ALL SPANS		-0.090418	-0.088663	-3.086588	-0.084193	-0.081479	-0.078446	-0.075096	-0.071428
UNIT D.L. ON SPANS 1 ,2 AND 4		-0.039079	-0.036939	-0.034505	-0.031774	-0.028745	-0.025416	-0.021707	-0.017057
UNIT D.L. ON SPANS 2 AND 3		-0.102678	-0.103448	-0.104166	-0.104838	-0.105468	-0.106060	-0.106617	-0.107142
UNIT D.L. ON SPANS 1 AND 3		-0.045209	-0.044331	-0.043294	-0.042096	-0.040739	-0.039223	-0.037548	-0.035714
UNIT D.L. ON SPANS 2 AND 4		-0.045209	-0.044331	-0.043294	-0.042096	-0.040739	-0.039223	-0.037548	-0.035714

$$M = \left[\sum w \cdot \text{coef.} \right] \cdot L_I^2 + \sum M_E \cdot \text{coef.}$$

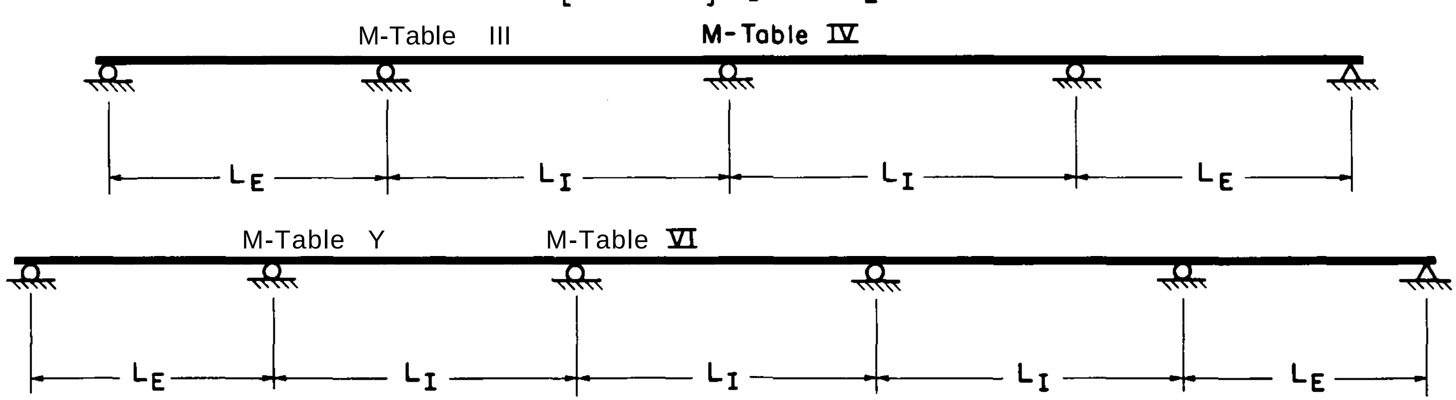


TABLE V INFLUFNCE SEGMENT COEFFICIENTS FOR 5 SPANS • • 1-ST INTERIOR SUPPORT

NUMBERS RFFFR TO PFR CFNT OF SPAN ••RATIO OF EXTERIOR SPAN LENGTH TO INTERIOR SPAN LENGTH ••

LOADING DESCRIPTION	REVERSE CURVF	0.650	0.700	0.750	0.800	0.850	09900	0.950	1.000
••SYMMETRIC PRESTRESS ••									
END 30 OF FND SPANS (SYM)	00	0.003807	0.004606	0.005493	0.006471	0.007540	0.008702	0.009958	0.011309
INNFR 70 OF END SPANS	05	0.015392	0.018624	0.022212	0.026165	0.030487	0.035184	0.040262	08045724
INNFR 70 OF FND SPAN5	10	0.012743	0.015418	0.018389	0.021662	0.025240	0.029129	0.033333	0.037855
INNFR 70 OF FND SPAN5	15	0.010381	0.012560	0.014980	0.017646	0.020561	00023729	0.027154	0.030037
FND 40 OF END SPANS	00	0.006520	0.007888	0.009409	0.011083	0.012914	0.014903	0.017054	0-019360
INNFR 60 OF FND SPANS	05	0.013100	0.015850	0.019905	0.022269	0.025947	0.029945	0.034267	0.030916
INNFR 60 OF END SPANS	10	0.010829	0.013103	0.015628	0.018409	0.021450	0-024755	0.028328	0n032171
INNFR 60 OF FND SPANS	15	0.008005	0.010653	0.012706	0.014967	0.017439	0.020127	0.023031	0.026156
FND 50 OF END SPANS	00	0.009689	0.011773	0.013982	0.016470	0.019191	0.022140	0.025344	0.028782
INNFR 50 OF FND SPAN S	05	0.010352	0.012525	0.014939	0.017597	0.020504	0.023663	0.027078	0.030752
INNFR 50 OF FND SPAYS	10	0.008460	0.010236	0.012208	0.014381	0.016756	0.019338	0.022129	0.025131
INNFR 50 OF END SPAYS	15	0.00677)	0.008194	0.009773	0.011513	0.013414	0.015481	0.017716	0.020119
2-ND AND 4-TH SPANS	05	0.055161	0.053437	0.051818	0.050294	0.048857	0.047499	0.046216	0.044999
2-ND AND 4-TH SPANS	10	0.046451	0.044999	0.043636	00042352	0.041142	0.039999	0.039910	0.037894
2-ND AND 4-TH SPANS	15	0.039387	0.037187	0.036060	0.035000	0.034000	0.033055	0.032162	0.031315
CFNTFR SPAY	05	-0.013790	-0.013359	-0.012954	-0.012573	-0.012214	-0.011874	-0.011554	-0.011249
CFNTFR SPAN	10	-0.011612	-0.011249	-0.010909	-0.010588	-0.010205	-0.009999	-0.009729	-0.009473
CFNTFR SPAN	15	-0.009596	-0.009296	-0.009315	-0.008750	-0.008500	-0.009263	-0.009040	-0.007020
UNIT MOMENTS ON T H F FNDS		-0.709677	-0.210750	-0.227272	-0.235294	-0.242057	-0.249999	-0.256756	-0.263157
• ANTI-SYMMETR IC PRFSTRESS •									
FND 30 OF FND SP.(ANTI-	00	0.003978	0.004806	0.005725	0.006735	0.007839	0.009037	0.010330	0.011720
INNFR 70 OF FND SP. -SYM)	05	0.016084	0.019433	0.023147	0.027233	0.031694	0.036538	0.041767	0.047387
INNFR 70 OF FND SP.	10	0.013316	0.016089	0.019164	0.022546	0.026240	0.030249	0.034579	0.039231
INNFR 70 OF FND SP.	15	0.010847	0.013106	0.015611	0.018366	0.021375	0.024642	0.028168	0.031958
FND 40 OF FND SP.	00	0.006813	0.008231	0.009805	0.011535	0.013425	0.015477	0.017692	0.020072
INNFR 60 OF FND SP.	05	0.013609	0.016540	0.019701	0.023178	0.026975	0.031097	0.035540	0.040331
INNFR 60 OF FND SP.	10	0.011316	0.013673	0.016206	0.019160	0.022300	0.025707	0.029307	0.033340
INNFR 60 OF FND SP.	15	0.009200	0.011116	0.013241	0.015578	0.018130	0.020901	0.021092	0.027107
FND 50 OF FND SP.	on	0.010124	0.012233	0.014571	0.017142	0.019951	0.023000	0.026292	0.029829
INNFR 50 OF FND SP.	05	0.010817	0.013073	0.015568	0.018315	0.021316	0.024574	0.020091	0.031870
INNFR 50 OF FND SP.	10	0.008840	0.010681	0.312722	0.014968	0~017420	0.020082	0.022956	0-026045
INNFR 50 OF FND SP.	15	0.007077	0.008551	0.010185	0~011983	00013946	0.016077	0.019370	0.020051
2-ND AND 4-TH SPANS	05	0.048033	0.046467	0.044999	0.043622	00042326	0.041105	0.039953	0.038863
2-ND AND 4-TH SPANS	10	0.040449	0.079130	0.037894	0.036734	0.035643	00034615	0.033644	0.032727
2-ND AND 4-TH SPANS	15	0.031475	0.032335	0.031314	0.030355	0.029454	0.028604	0.027802	0.027044
UNIT MOMFNTS ON THF FNDS		-0~219101	-0.229260	-0.236842	-0.244897	-0.252475	-0.259615	-0.266355	-0.272727
• • • APPLIFD LOADS •••									
UNIT DFAD LOAD ON 1-STS PAY		-0.022644	-0.027379	-0.032633	-0.038415	-0.044734	-0.051598	-0.059013	-00066905
UNIT DFAD LOAD ON 2-ND		-0.060347	-0.058423	-0.056618	-0.054921	-0.053323	-0.051816	-0.050391	-0.049043
UNIT DFAD LOAD ON 3-RD		0.016129	0.015625	0.015151	0.014705	0.014285	0.013888	0.013513	0.013157
UNIT DFAD LOAD ON 4-TH		-0.004168	-0.004076	-09003987	-0.003901	-0.003818	-0.003739	-0.003662	-0.003588
UNIT DFAD LOAD ON 5-TH		0.000497	0.000582	0.000672	0.000768	0.000868	0.000973	0.001002	0.001196
UNIT DFAD LOAD ON ALL SPANS		-0.070534	-0.073671	-0.077414	-0.081764	-0.086723	-0.092291	-0.090471	-09105263
UNIT D.L. ON SPANS 1,2 AND 4		-0.087160	-0.089879	-0.093239	-0.097238	-0.101877	-0.107154	-0.113067	-0.119617
UNIT D.L. ON SPANS 2,3 AND 5		-0.043771	-0.042216	-0.040794	-0.039447	-0.038169	-0.036953	-0.035795	-0.034688
UNIT D.L. ON SPANS 1,3 AND 5		-0.006018	-0.011171	-0.016800	-0.022941	-0.029580	-0.036736	-0.044417	-0.052631
UNIT D.L. ON SPANS 7 AND 4		-0.064516	-0.062500	-0.060606	-0.058823	-0.057142	-0.055555	-0.054054	-0.052631

TABLE V I INFLUENCE SEGMENT COEFFICIENTS F O R 5 SPANS — 2-ND INTERIOR SUPPORT

NUMBERS REFER TO PER CENT OF SPAN LOADING DESCRIPTION	REVERSE CURVE	0.650	0.700	0.750	0.800	0.850	0.900	0.950	1.000
		— — RATIO OF EXTERIOR SPAN LENGTH TO INTERIOR SPAN LENGTH — —							
— — SYMMETRIC PRESTRESS — —									
END 30 OF END SPANS (SYM)	00	-0.000761	-0.000921	-0.001090	-0.001294	-0.001508	-0.001740	-0.001991	-0.002261
INNER 70 OF END SPANS	05	-0.003078	-0.003724	-0.034442	-0.005233	-0.006097	-0.007036	-0.000052	-0.009144
INNER 70 OF END SPANS	10	-0.002540	-0.003083	-0.003677	-0.004332	-0.005040	-0.005825	-0.006666	-0.007571
INNER 70 OF END SPANS	15	-0.002076	-09002512	-0.002996	-0.003529	-0.004112	-0.004745	-0.005430	-0.006167
END 40 OF END SPANS	00	-0.001304	-0.001577	-0.001001	-0.002216	-0.002582	-0.002980	-0.003410	-0.003073
INNER 60 OF END SPANS	05	-0.002620	-0.003170	-0.003781	-0.004453	-0.005109	-0.005909	-0.006053	-0.007703
INNER 60 OF END SPANS	10	-0.002165	-0.0112620	-0.003125	-0.003601	-0.004290	-0.004951	-0.005665	-0.006434
INNER 60 OF END SPANS	15	-0.001761	-0.002130	-0.002541	-0.002993	-0.003407	-0.004025	-0.004606	-0.005231
END 50 OF END SPANS	00	-0.001937	-0.002344	-0.002796	-0.003294	-0.003838	-0.004429	-0.005060	-0.005756
INNER 50 OF END SPANS	05	-0.007070	-0.002505	-0.002907	-0.003519	-0.004100	-0.004732	-0.005415	-0.006150
INNER 50 OF END SPANS	10	-0.001692	-0.002047	-0.002441	-0.002876	-0.003351	-0.003867	-0.004425	-0.005026
I NNER 50 OF END SPANS	15	-0.001354	-0.001630	-0.001954	-0.002302	-0.002682	-0.003096	-0.003543	-0.004023
2-ND AND 4-TH SPANS	05	0.031717	0.032062	0.032386	0.032691	0.032970	0.033249	0.033506	0.033749
2-ND AND 4-TH SPANS	10	0.026709	0.026999	0.027272	O-027529	0.02 7771	0.027999	0.028216	0.028421
2-ND AND 4-TH SPANS	15	0.077072	0.022312	0.022537	0.022750	0.022950	O-023130	0.023317	0.023486
CENTER SPAN	05	0.045507	0.045421	0.045340	0.045264	0.045192	0.045124	0.045060	0.044999
CENTER SPAN	10	0.038322	0.038249	0.030101	0.030117	0.030057	08037999	0.037945	O-037094
CENTER SPAN	15	0.071669	0.031609	0.031553	0.031500	0.031450	0.031402	0.031350	0.031315
UNIT MOMENTS O N THE ENDS		0.041935	0.047750	0.045454	0.047058	0.048571	0.049999	0.051351	0.052631
— ANTI-SYMMETRIC PRESTRESS —									
END 30 OF END SP.(ANTI-	00	-0.001326	-0.001602	-0.001908	-09002245	-0.002613	-0.003012	-0.003443	-0.003906
INNER 70 OF END S P . -SYM)	05	-0.005361	-0.036477	-0.007715	-0.039077	-0.010564	-0.012179	-0.013922	-0.015795
INNER 70 OF t-Y" SP.	10	-0.004438	-0.005363	-0.006388	-0.007515	-0.008746	-0.010083	-0.011526	-0.013077
INNER 70 OF END SP.	15	-0.003615	-0.004368	-0.005203	-0.006121	-0.007124	-0.008213	-0.009300	-0.010651
END 40 OF END SP.	00	-0.002271	-0.002743	-0.003268	-0.003845	-0.004475	-0.005159	-0.005097	-0.006690
INNER 60 O F END SP.	05	-0.004563	-0.005513	-0.006567	-0.007726	-0.008991	-0.010365	-0.011049	-0.013443
INNER 60 OF END SP.	10	-0.003772	-0.004557	-0.005428	-0.006386	-0.007433	-0.000569	-0.009795	-0.011113
INNER 6 0 O F END SP.	15	-0.003066	-0.003705	-0.004413	-0.005192	-0.006043	-0.006967	-0.007964	-0.009035
END 5 0 OF END SP.	00	-0.003374	-0.004077	-0.004857	-0.005714	-0.006650	-0.007666	-0.000764	-09009943
INNER 50 OF END SP.	05	-0.003605	-0.004356	-0.035109	-0.006105	-0.007105	-0.008191	-0.009363	-0.010623
I NNER 50 OF END SP.	10	-0.0'17946	-0.033560	-0.004240	-0.004989	-0.005806	-0.006694	-0.007652	-0.008681
INNER 50 OF END SP.	15	-0.002359	-0.002850	-0.003395	-0.003995	-0.004649	-0.005360	-0.006127	-0.006951
2-ND AND 4-TH SPANS	05	0.055238	0.055760	0.056249	0.056709	0.057140	0.057547	0.057932	0.058295
2-ND AND 4-TH SPANS	10	0.046516	0.046956	0.047360	0.047755	0.048118	0.048461	0.040704	0.049090
2-ND AND 4-TH SPANS	15	0.038446	0.038810	0.039150	0.039470	0.039770	0.040053	0.040321	0.040574
UNIT MOMENTS ON THE ENDS		0.073033	0.076086	0.070947	0.081632	0.084158	0.086538	0.000705	0.090909
— — — APPLIED LOADS — — —									
UNIT DEAD LOAD ON 1-ST SPAN		0.006071	0.007340	0.008747	0.010295	0.011907	0.013824	0.015009	0.017942
UNIT DEAD LOAD ON 2-ND		-0.050851	-0.051358	-0.051834	-0.052280	-0.052701	-0.053090	-0.053473	-0.053027
U N I T DEAD LOAD ON 3-RD		-0.053225	-0.053125	-0e053030	-0.052941	-0.052857	-0.052777	-0.052702	-0.052631
UNIT DEAD LOAD O N 4-TH		0.013754	0.013058	0.013955	0.014045	0.014130	0.014209	0.014203	0.014354
UNIT DEAD LOAD ON 5-TH		-0.001642	-0.001900	-0.002354	-0.002765	-0.003213	-0.003699	-0.004222	-0.004784
UNIT DEAD LOAD ON ALL SPANS		-0.085893	-0.085265	-0.004517	-0.083647	-0.082655	-0.001541	-0.000305	-0.078947
UNIT D.L. ON S P A N S 1,2 AND 4		-0.031024	-0.030159	-0.029131	-0.027939	-0.026584	-0.025064	-0.023300	-0.021531
UNIT D.L. ON SPANS 2,3 AND 5		-0.135719	-0.106464	-0.107219	-0.107987	-0.100772	-0.109575	-0.110390	-0.111244
UNIT D.L. ON SPANS 1,3 AND 5		-0.048796	-0.047765	-0.046638	-0.045411	-0.044083	-0.042652	-0.041116	-0.039473
UNIT D.L. ON SPANS 2 AND 4		-0.037096	-0.037500	-0.037878	-0.030235	-0.030571	-0.030000	-0.039109	-0.039473

spans as equivalent to the center span of the 5-span beam. For the 3-span case coefficients are given only for the first interior support moment, and for the 4- and 5-span cases they are given for the first two interior supports. The moment over any other support is the same as the corresponding symmetric support moment if the loading is symmetric. If the loading is not symmetric, e.g., due to friction losses in a long beam tensioned from one end only, the moment is obtained by reversing the sign of the corresponding anti-symmetric component load coefficient as explained in the full PCI Journal article.

Within each of the tables, the loadings, considered consist of either uniformly distributed load segments or applied end moments. In the first case, the moment M is obtained by multiplying the coefficient by both the intensity of the load w and the square of the interior span length L_I; that is, the coefficients are moments for a unit load intensity applied to a structure with unit interior span length. In the second case the moment M is obtained by mulitplying the coefficient by the applied end moment M_E.

The algebraic signs of all moments follow the beam convention: positive for a moment giving compression in the top fiber. The moments obtained from the tabulated influence coefficients will follow this sign convention provided the sign of the distributed load is positive if it is applied in its usual direction. That is, a distributed dead load acting downward is positive and a distributed prestress load acting upward over the major portion of the tendons is also positive. The distributed load due to the prestressing tendon is always expressed as that of the major (upward curvature) portion. The effect of the reverse curvature portion is already included in the tabulated moment coefficients.

Four types of distributed loads are considered in the tables:

1. Loads applied to the end portion of the exterior spans over segment fg denoted by bL_E in Fig. 2(a). Coefficients are given for a b of 30%, 40% and 50%.
2. Loads applied to the remaining (interior) portion of the exterior spans. The reverse curvature portion is segment hi in Fig. 2(a), denoted by aL_E in Fig. 2(a). Coefficients are tabulated for an a of 5%, 10% and 15%.
3. Loads applied to the interior spans. Here the reverse curvature segments ef and hi are denoted by aL_I in Fig. l(a), with coefficients given for the above percentages for a. Note that the tendon profile is assumed to be symmetric within each interior span.
4. Uniform loads applied to specific spans for use in computing moments due to dead load as well as live load.

In this discussion L and I refer to span length and moment of inertia, respectively; subscripts E, I, L, R and C denote exterior, interior, left, right and center, respectively.

Coefficients for reverse curvature segment lengths, other than those tabulated, can be obtained by linear interpolation. For reverse curvatures of less than 5% the coefficient for the 0% case can be extrapolated by taking the corresponding 15% coefficient plus three times the difference between the 5% coefficent and the 10% coefficient. Similarly, the 20% case can be obtained by adding to the 5% case three times the difference between the 15% and the 10% cases. These extrapolations give results accurate to about 0.1%. The error due to linear interpolation is at most 0.3%.

Each table for the moment coefficients over a support is developed from the influence line for the moment in the beam over that support. The coefficients are obtained by computing the area under the influence line over the segment that is loaded by a constant distributed load. If the coefficient represents the effect of several load segments, then it is the sum of the area under each of the segments multiplied by the ratio of the equivalent loads.

Example 1

Consider the 4-span structure shown in Fig. 3. The moment at support C is to be computed for each of the following loadings:

a = distributed prestress load as shown

b = a 1000 k.-ft. end moment acting on both ends

c = a 3 k./ft. uniform dead load

The end to interior span ratio is 0.75. From Table III, the coefficient for the end 40% of the end span is 0.0103. The coefficient for the inner 60% with 13.3% reverse curve is interpolated as 0.0150. The coefficient for the middle spans is 0.0300. So the moment at support C due to the distributed prestress load a is

$$M = (0.0103 \times 6 + 0.0150 \times 4 + 0.0300 \times 4)\ 100^2 = 2424 \text{ k.-ft.}$$

The coefficient for end moments is -0.25 so the desired moment due to loading b is M = -250

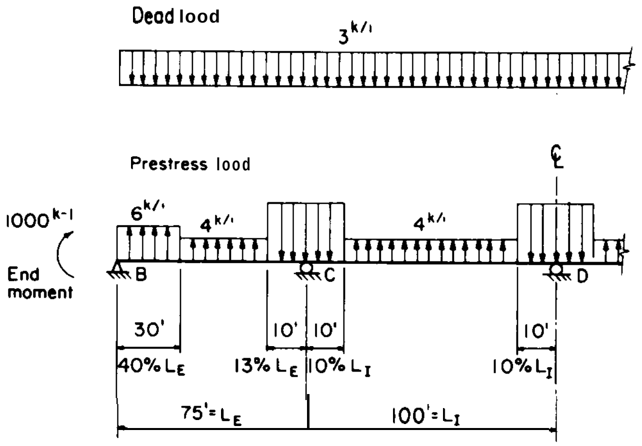


Fig. 3 — 4-span structure for Example 1.

k.-ft. The coefficient for unit dead load on all spans is -0.0768, so the moment due to the 3 k./ft. dead load is M = -2304 k.-ft.

Non-Symmetric Loadings

As pointed out previously, symmetry is not a consideration in the 2-span case so only structures of three or more spans are considered in this section. As long as a structure is symmetric, any loading can be separated into two loadings, one of which is symmetric and the other anti-symmetric. This division is usually obvious. However, if not, it can be bbtained by reversing the original loading, taking half of the sum of the original and reversed loadings as the symmetric part, and half of the difference as the anti-symmetric part. The moments are then computed for both parts using the appropriate coefficients. The moments for the left half of the structure are equal to the sum of the computed moments; those for the right are equal to the difference.

Example 2

The moments at the supports of a beam of constant cross section with four equal spans are to be computed. A typical non-symmetric equivalent prestress loading as produced when tensioning long beams from one end is considered. End moments, as considered in this example, would appear only if the tendons are anchored away from the neutral axis of the cross section, or if the beam is cantilevered. The complete loading diagram is shown in Fig. 4a. Load diagrams (b), (c) and (d) show the reversed, the symmetric portion, and the anti-symmetric portion respectively. Note that only the left half of the beam need be considered for these three loadings. Load diagram (b) was derived by folding the right part of the structure about its centerline. Load diagram (c) is half the sum of (a) and (b). Load diagram (d) can be computed either as the difference between (a) and (c) or as half the difference between (a) and (b). The moment at support B caused by the symmetric part of the loading is computed by using the coefficients in the symmetric prestress portion of Table III.

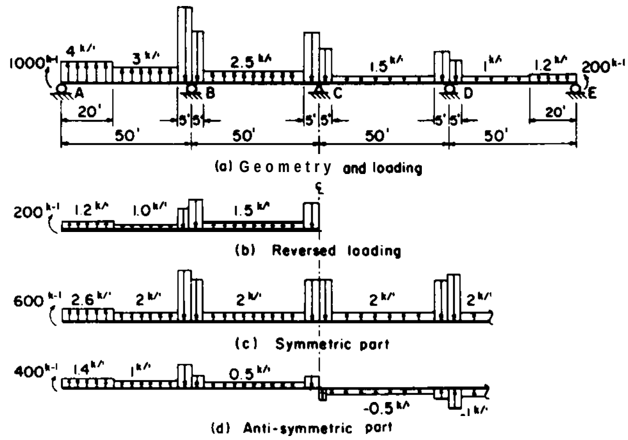


Fig. 4 — 4-span structure with non-symmetric loading for Example 2.

$$M_{BS} = (2.6 \times 0.02103 + 2.0 \times 0.03493 + 2.0 \times 0.02571)50^2 + 600 \times (-0.2857) = 268.5 \text{ k.-ft.}$$

The moment at support B due to the anti-symmetric prestress is

$$M_{BA} = (1.4 \times 0.01840 + 1.0 \times 0.03056 + 0.5 \times 0.04500)50^2 + 400 \times (-0.2500) = 97 \text{ k.-ft.}$$

The moment at support C due to the symmetric load (Table IV) is

$$M_{CS} = (-2.6 \times 0.01051 - 2.0 \times 0.01746 + 2.0 \times 0.07714)50^2 + 600 \times 0.1429 = 316 \text{ k.-ft.}$$

The moment at support C due to anti-symmetric prestress is zero.

$$M_{CA} = 0$$

Finally, the moments at the three supports are

Left

left: $M_B = M_{BS} + M_{BA} = 365.5$ k.-ft.

$M_C = M_{CS} + M_{CA} = 316$ k.-ft.

right: $M_D = M_{BS} - M_{BA} = 171.5$ k.-ft.

$M_C = M_{CS} - M_{CA} = 316$ k.-ft.

Spans With Different Moments Of Inertia

Two cases of spans with different moments of inertia may be analyzed using the tables. First, the cross section of the end spans may be different (e.g. the cross sections of the spans in a 2-span beam). Second, the center span cross section of a 5-span beam may be different from the other two interior spans.

If the cross sections of the end spans differ, replace the end span ratio computation L_E/L_I by $L_E I_I/L_I I_E$ for selecting coefficients in the tables. Then multiply each distributed load applied to the end spans by $(I_E/I_I)^2$. For the 2-span case, the ratio is $L_L I_R/L_R I_L$ and the multiplying factor is $(I_L/I_R)^2$ applied to a distributed load on the left span.

The center span section of a 5-span beam may be different only to the extent that its stiffness remains the same as for the other interior spans. That is $I_C/L_C = I_I/L_I$ where C refers to the center span and 1 refers to the other interior spans. Any distributed loading applied to the center span must then be multiplied by $(I_C/I_I)^2$.

Example 3

The moment at point C of the symmetric beam shown in Fig. 5 is to be computed. The end span ratio to be used is

$$100 \times I_I/100 \times 1.25\, I_I = 0.8$$

The distributed load factor $(I_E/I_I)^2$ is 1.5625. The center span stiffness requirement is satisfied since $I_C/L_C = I_I/L_I$. Its distributed load factor is 4.0. Hence, the required moment is obtained by using Table VI.

$$\begin{aligned} M_C &= 0.04706 \times 1000 + (-0.00222 \\ &\quad \times 5 \times 1.56 - 0.00368 \times 4 \\ &\quad \times 1.56 + 0.02753 \times 5 + 0.03812 \\ &\quad \times 2 \times 4.0)\,100^2 = 4070 \text{ k.-ft.} \end{aligned}$$

Similarly, from Table V, the moment at point B is

$$M_B = 3048 \text{ k.-ft.}$$

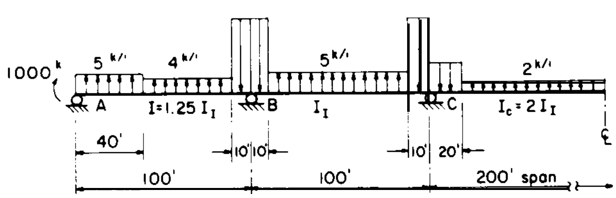


Fig. 5 – 5-span structure with varying moments of inertia for Example 3.

Bending Moments Between The Supports

Bending moments between the supports can be computed by two methods. The first method is simply to compute the moment at any point by statics using the applied equivalent loads and the computed moments at the supports. The second method, which requires considerably less computation, is to compute a primary moment which is the moment that would be present if the beam spans were free to rotate at their ends, and a secondary moment which is the moment produced by restoring beam continuity over the supports.

M'_x, the primary moment at any point x, is the horizontal component of the prestress force at that point times the eccentricity of the tendon profile from the neutral axis

$$M'_x = P_x e_x \qquad (6)$$

The secondary moment is linear between the supports and, for a typical span AB

$$M''_x = M''_A \left(1 - \frac{x'}{L}\right) + M''_B \frac{x'}{L} \qquad (7)$$

where x' is the distance from support point A to the point x, L is the length of span AB, M'' is the secondary moment at the point indicated by the subscript. The secondary moment at a support, as required in Equation (7), is computed by subtracting the primary moment from the total moment obtained by using the moment influence coefficients. So the total moment at any point x is obtained from Equations (6) and (7) as

$$\begin{aligned} M_x &= P_x e_x + (M_A - P_A e_A) \\ &\quad \times \left(1 - \frac{x'}{L}\right) + (M_B\; P_B e_B)\frac{x'}{L} \end{aligned} \qquad (8)$$

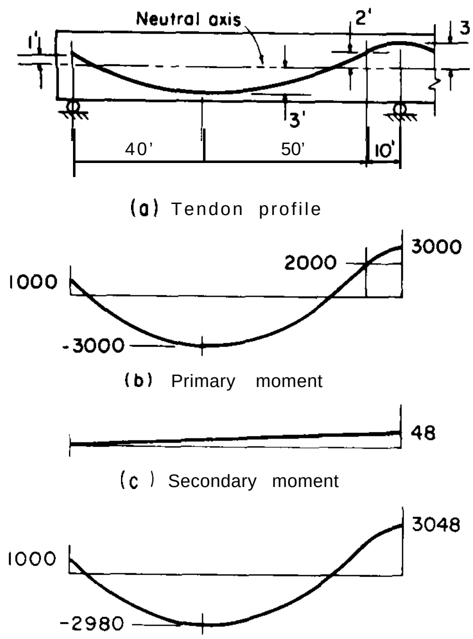
Example 4

The moment in the first span of Example 3 is to be computed. The tendon profile is shown in Fig. 6(a), and the horizontal component of the tendon force is 1000 k. The primary moments as computed by Equation (6) are shown in Fig. 6(b). The secondary moments at the ends are

$$\begin{aligned} M''_A &= M_A - M'_A = 1000 \text{ k.-ft.} \\ &\quad - 1000 \text{ k.-ft.} = 0 \text{ (free rotation)} \end{aligned}$$

$$\begin{aligned} M''_B &= Me - M'_B = 3048 \text{ k.-ft.} \\ &\quad - 3000 \text{ k.-ft.} = 48 \text{ k.-ft.} \end{aligned}$$

and the secondary moment for the span as computed by Equation (7) is shown in Fig. 6(c). The sum of the primary and secondary moments gives the total moment as shown in Fig. 6(d). The moment at any point of this curve can be computed directly from Equation (8).



(a) Tendon profile

(b) Primary moment

(c) Secondary moment

(d) Total prestress moment • k-ft.

Fig. 6 - Bending moments between supports for Example 4.

Friction Losses

The equivalent load due to prestressing, as given by Equations (1) to (5), is proportional to P, the horizontal component of the force in the prestressing tendons. However, P is not constant along the beam since it is reduced by friction losses along the tendons. A further variation of force is caused by anchor set as the load is transferred from the jacking device. Anchor set causes a reversal of friction forces in the end sections.

For short prestressing tendons the friction losses can usually be neglected provided the total angular change of the tendon profile is small. However, anchor set losses may be large. In this case both effects may be accommodated by using a reduced constant value of P for the length of the beam.

For long post-tensioned tendons, the friction losses cannot be neglected in the final analysis. An ACI Building Code[7] formula gives the following value for P at any section x in the beam:

$$P_x = P_o e^{-(KL + \mu\alpha)} \tag{9}$$

If the value of KL + $\mu\alpha$ is below 0.3, in accordance with the ACI Code, Equation (9) may be replaced by

$$P_x = \frac{P_o}{1 + KL + \mu\alpha} \tag{10}$$

Equations (9) or (10) may also be used to compute friction losses through any segment of the beam[8] in which case the reference section is that end of the segment at which the tendon force P_o has already been computed, For reasonable accuracy in this case, Equation (10) should not be used if the value of KL + $\mu\alpha$ for the segment is greater than about 0.1.

The computed tendon force at various sections along the beam can now be plotted. If the slope of the tendon is large, the horizontal component can be computed by multiplying the tendon force by (1 – ½s^2) where s is the tendon slope. A linear approximation for the tendon force variation with distance along the beam is sufficiently accurate for most cases, and can be obtained by a straight line approximation of the plotted tendon force.

The loss of prestress force at the anchor section due to anchor set is

$$AP, = 2\sqrt{rAE\Delta L} \tag{11}$$

However, if the computed AP, is greater than 2 x ($P_o - P_{min}$), where P_o is the jacking force and P_{min} is the lowest computed prestress force in the beam – either at the non-jacking end for post-tensioning from one end, or near the midpoint for post-tensioning from both ends – then

$$AP, = P_o - P_{min} + \frac{rAE\Delta L}{P_o - P_{min}} \tag{12}$$

This value will be greater than the AP, computed by Equation (11). The prestress force plot can be revised to include the anchor set loss by noting that the friction losses will be reversed in the regions affected. The prestress force at the anchor will be P_o – AP,, and will increase with distance from the anchor at a rate of r.

NOTATION

A	= cross-sectional area of the prestressing tendons
E	= elastic modulus of the prestressing tendons
I_E, I_I	= moment of intertia of the cross section in the span indicated
K	= friction loss factor related to length
L	= length of the segment over which friction loss is computed
L_E, L_I	= length of the span indicated
AL	= tendon movement at the anchor due to anchor set
M_E	= end moment due to eccentricity of the tendon over the exterior support
M_x, M_A	= bending moment at the point indicated .

M' and M'' = primary and secondary bending moments respectively

P_0 = jacking force

ΔP_o = loss of prestress force at the jacking end due to anchor set

P_{min} = lowest prestress force considering friction losses

P_x, P_A = horizontal component of the prestressing tendon force at the point indicated

a = ratio of the reverse curve length to the span length

b = ratio of the end segment length to the span length in an exterior span

c = drape of the tendon profile, high point to low point

d = drape of the tendon profile in the end segment of an exterior span

e = eccentricity of the tendon profile above the neutral axis at the exterior support

e_x, e_A, etc. = eccentricity of the tendon profile above the neutral axis at the point indicated

r = loss of prestress force per unit length of beam

s = slope of the tendon profile

w = equivalent upward distributed load over the major segment of the tendon profile

w_E = equivalent upward distributed load in the end segment of an exterior span

w_R = equivalent downward distributed load over the reverse curvature segment of the tendon profile

x' = distance from the end of a span to a point x

α = angular change of the tendon profile in the segment over which friction loss is computed

μ = friction loss factor related to angular change of the tendon profile

Subscripts:

A and S designate anti-symmetric and symmetric, respectively.

x, A, B, C, etc. designate points along the beam.

L, R, E, I and C designate left, right, exterior, interior and center spans, respectively.

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4. Bailey, D. M. and Ferguson, P. M., "Fixed-End Moment Equations for Continuous Prestressed Concrete Beams," *Journal of the Prestressed Concrete Institute,* Vol. 11, No. 1, Feb. 1966, pp. 76-94.
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A.2 FRICTION LOSSES

The friction along a cable is calculated according to the formula:

$$T_x = T_o \cdot e^{-(\mu\alpha + kx)} \qquad (1)$$

or if ($\mu\alpha$ + kx) is not greater than 0.15, according to the following simplified formula:

$$T_x = \frac{T_o}{1 + \mu\alpha + kx} \qquad (2)$$

where T_x = prestressing force at point x
T_o = prestressing force at jacking end
e = base of Naperian logarithms
μ = coefficient of angular friction
α = total angular change in radians from jacking end to point x
k = wobble factor
x = length of cable from jacking end to point x in feet

The nomogram **on page 315 is provided for use with formula (1) above. It includes scales for T_x, T_a, T_o or f_{sx}, f_{se}, f_{so}, and ($\mu\alpha$ + kx). The dimensionless value for the friction ($\mu\alpha$ = kx) can be taken from page 314. When one of the three forces or stresses in 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.**

Friction coefficients: Applicable code requirements must be observed, but in the absence of these the following values may be applied:

Type of duct	Range of values		Recommended for calculations	
	μ	K	μ	K
flexible tubing non-galvanized	0.18-0.26	5-10 · 10^{-4}/ft.	0.22	7.5 · 10^{-4}/ft.
flexible tubing galvanized	0.14-0.22	3-7 · 10^{-4}/ft.	0.18	5.0 · 10^{-4}/ft.
rigid thin wall tubing non-galv.	0.20-0.30	1-5 · 10^{-4}/ft.	0.25	3.0 · 10^{-4}/ft.
rigid thin wall tubing galv.	0.16-0.24	o-4 · 10^{-4}/ft.	0.20	2.0 · 10^{-4}/ft.
greased and wrapped	0.05-0.15	5-15 · 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 also be considered.**

Diagram for determining $\mu\alpha$

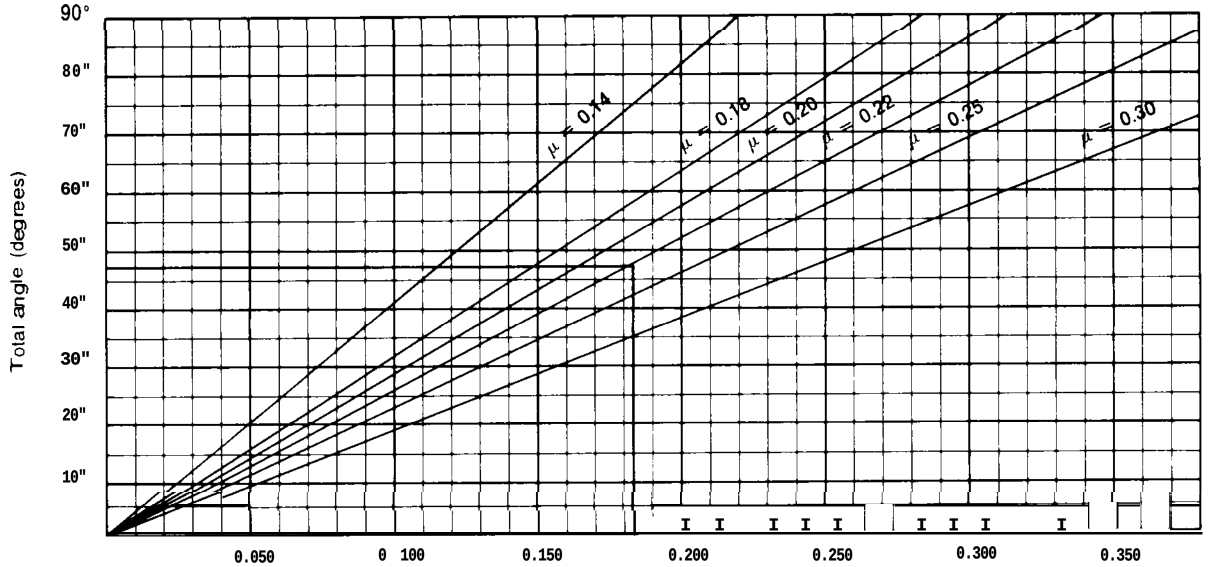
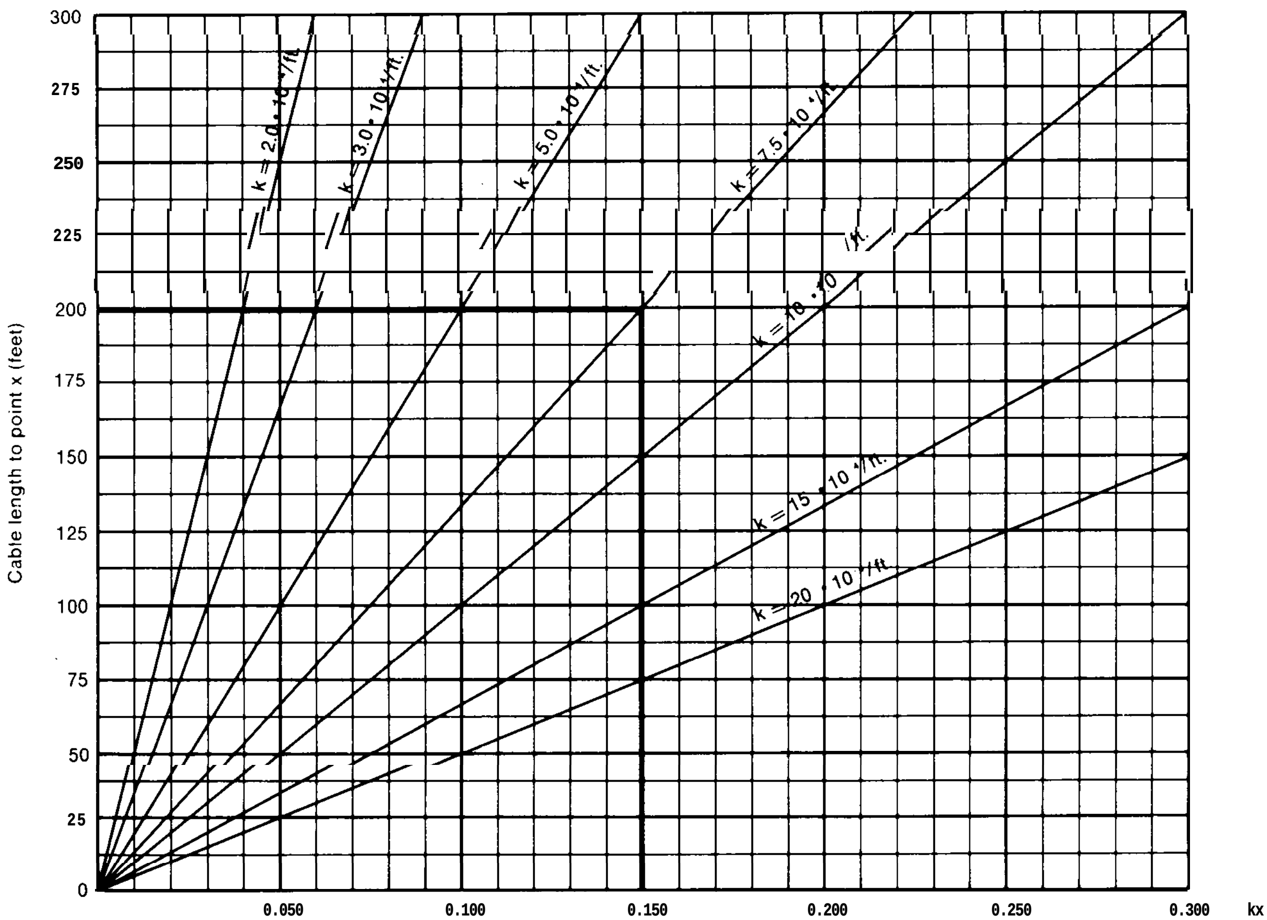
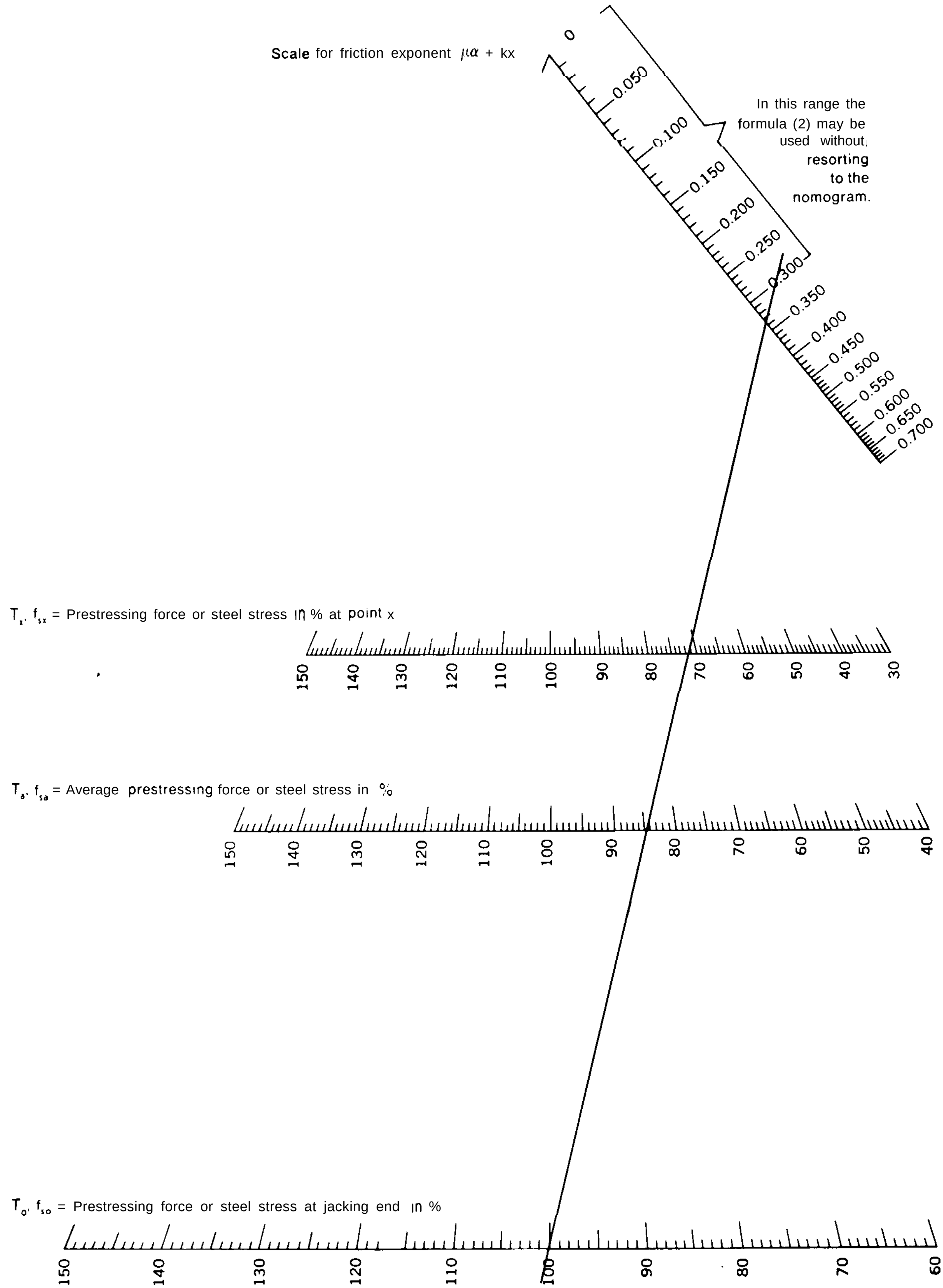


Diagram for determining kx

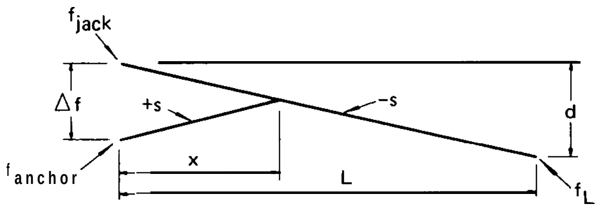


Nomogram for determining the prestressing forces and steel stresses along a tendon



A.3 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.



Af = 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
AL = Anchor set, in.
E = Modulus of elasticity, ksi

$$E = \frac{\text{Unit Stress}}{\text{Unit Strain}} = \frac{favg}{\Delta L/x} = \frac{favg\ x}{AL}$$

$$favg = \frac{EAL}{x}$$

$$\frac{Af}{2} = \frac{EAL}{12x}$$ Units Correct

$$\Delta L = \frac{Pavg\ x}{AE} = \frac{favg\ x}{E}$$

$$favg = \frac{EAL}{x}$$

$$\frac{Af}{2} = \frac{EAL}{12x}$$ Units Correct

$$Af = \frac{EAL}{6x}$$ <u>ΔL & x Known</u>

by similar triangles

$$\frac{x}{\Delta f/2} = \frac{L}{d}$$

$$Af = \frac{2xd}{L}$$ <u>x Known</u>

$$x = \frac{Af\ L}{2d}$$

$$x = \frac{EALL}{6x\ 2d}$$

$$x^2 = \frac{E(\Delta L)L}{12\ d}$$

$$x = \sqrt{\frac{E(\Delta L)L}{12\ d}}$$ <u>AL Known</u>

Also from $Af = \frac{EAL}{6x}$ & $\Delta f = \frac{2xd}{L}$

$$x = \frac{EAL}{6Af} = x = \frac{L\Delta f}{2d}$$

$$\Delta f^2 = \frac{E\Delta L d}{3L}$$

$$\Delta f = \sqrt{\frac{E\Delta L d}{3L}}$$ <u>AL Known</u>

When measuring anchor set, the tendon elongation within the jack must be considered:

Jacking to .75f′s = .75(270) = 202.5 ksi
5/8″ required anchor set
4′jack used (AL)
E = 27x 10^3 ksi

$$\Delta L = \frac{f\ L}{E} = \frac{202.5\ \ 4\times12}{27\times10^{3}} = .36''$$

Total elongation lost during anchor set
= AL + 5/8″
= .36″ + .625″
= 1″ ±

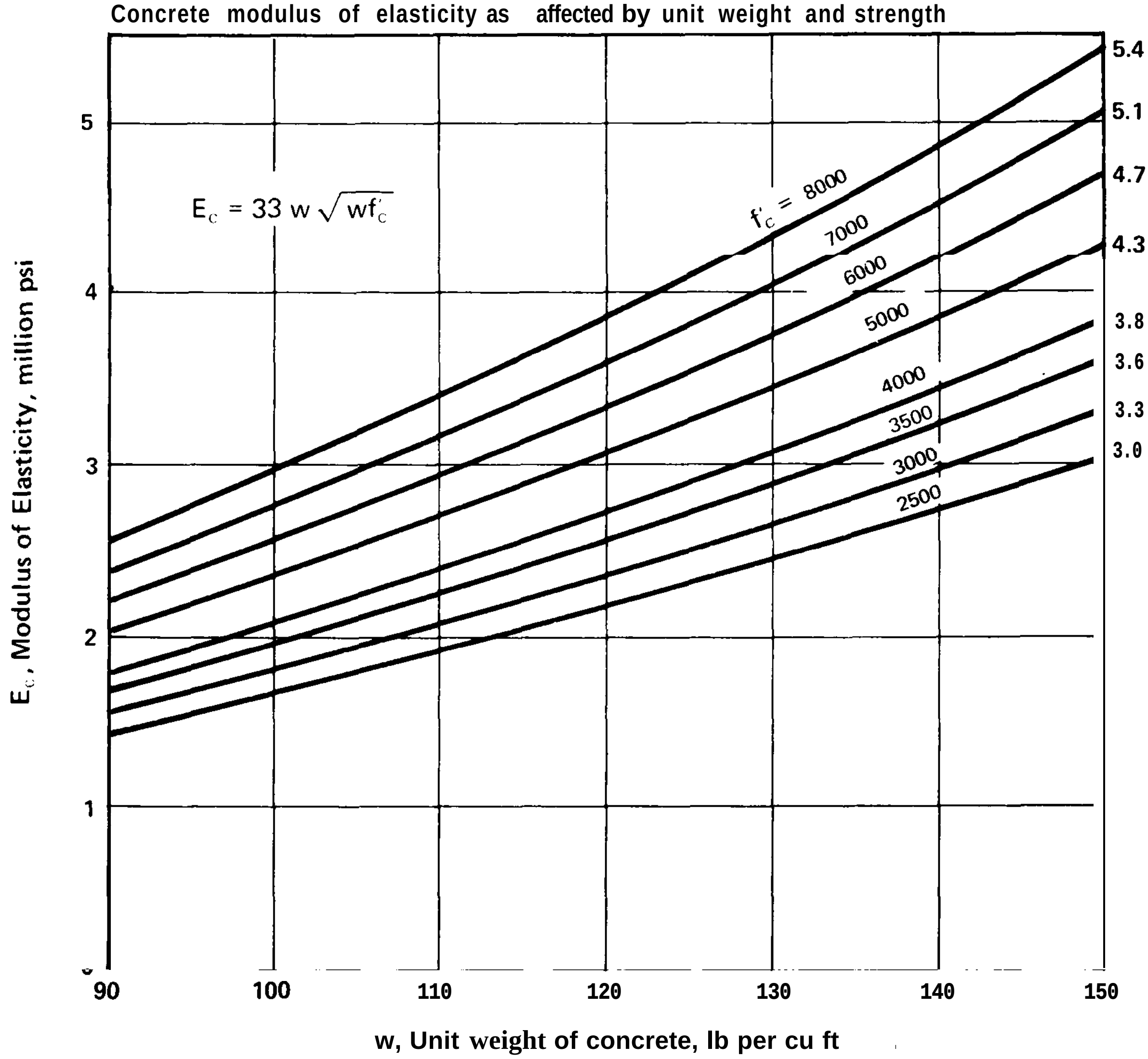
For simplicity use AL
= 1/12″ per foot of jack.

A.4 CONCRETE MATERIALS PROPERTIES

Table of concrete stresses

f'_c	$0.45\ f'_c$	$0.6\ f'_c$	$\sqrt{f'_c}$	$0.6\sqrt{f'_c}$	$2\sqrt{f'_c}$	$3\sqrt{f'_c}$	$3.5\sqrt{f'_c}$	$4\sqrt{f'_c}$	$5\sqrt{f'_c}$	$6\sqrt{f'_c}$	$12\sqrt{f'_c}$
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

Concrete modulus of elasticity as affected by unit weight and strength



A.5 MATERIALS PROPERTIES PRESTRESSING STEEL

Properties and design strengths of prestressing strand, wire and bar

Seven-Wire Strand, f_{pu} = 270 ksi

Nominal Diameter, in.	3/8	7/16	1/2	0.600
Area, sq in.	0.085	0.115	0.153	0.217
Weight, plf	0.29	0.39	0.52	0.74
0.7 f_{pu} A_{ps}, kips	16.1	21.7	28.9	41.0
0.8 f_{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

Seven Wire Strand, f_{pu} = 250 ksi

Nominal Diameter, in.	1/4	5/16	3/8	7/16	1/2	0.600
Area, sq in.	0.036	0.058	0.080	0.108	0.144	0.215
Weight, plf	0.12	0.20	0.27	0.37	0.49	0.74
0.7 f_{pu} A_{ps}, kips	6.3	10.2	14.0	18.9	25.2	37.6
0.8 f_{pu} A_{ps}, kips	7.2	11.6	16.0	21.6	28.8	43.0
f_{pu} A_{ps}, kips	9.0	14.5	20.0	27.0	36.0	54.0

Prestressing Wire

Diameter	0.192	0.196	0.256	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
0.7 f_{pu} A_{ps}, kips	5.05	5.28	8.25	9.84
0.8 f_{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

Prestressing Bars

Nominal Diameter, in.	5/8"	1"	1"	1-1/4"	1-1/4"	1-3/8"
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
0.66$f_{pu}A_{ps}$, kips	28.7	84.4	90.0	123.8	132.0	154.4
0.75$f_{pu}A_{ps}$, kips	32.6	95.9	102.2	140.6	150.0	175.5
$f_{pu}A_{ps}$, kips	43.5	127.8	136.3	187.5	200.0	234.0

A.6 MATERIALS PROPERTIES WELDED WIRE FABRIC

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 l00 sq ft
		Longit.	Trans.	Longit.	Trans.	Longit.	Trans.	
Two-Way Types	2 x 2-10/10	2	2	10	10	.086	.086	60
	2 x 2-14/14*	2	2	12	12	.052	.052	37
	2 x 2-12/12*	2	2	14	14	.030	.030	21
	3 x 3-8/8	3	3	a	8	.082	.082	58
	3 x 3-10/10	3	3	10	10	.057	.057	41
	3 x 3-12/12*	3	3	12	12	.035	.035	25
	3 x 3-14/14*	3	3	14	14	.020	.020	14
	4 x 4-4/4	4	4	4	4	.120	.120	85
	4 x 4-6/6	4	4	6	6	.087	.087	62
	4 x 4-8/8	4	4	8	8	.062	.062	44
	4 x 4-10/10	4	4	10	10	.043	,043	31
	4 x 4-12/12*	4	4	12	12	.026	.026	19
	6 x 6-0/0	6	6	0	0	.148	.148	107
	6 x 6-2/2	6	6	2	2	.108	.108	78
	6 x 6-4/4	6	6	4	4	.080	.080	58
	6 x 6-4/6	6	6	4	6	.080	.058	50
	6 x 6-6/6	6	6	6	6	.058	.058	42
	6 x 6-8/8	6	6	8	a	.041	.041	30
	6 x 6-10/10	6	6	10	10	.029	.029	21
One-Way Types	2 x 12-0/4	2	12	0	4	.443	.040	169
	2 x 12-2/6	2	12	2	6	.325	.029	124
	2 x 12-4/8	2	12	4	8	.239	.021	91
	2 x 12-6/10	2	12	6	10	.174	.014	66
	2 x 12-8/12	2	12	8	12	.124	.009	46
	3 x 12-0/4	3	12	0	4	.295	.040	119
	3 x 12-2/6	3	12	2	6	.216	.029	87
	3 x 12-4/8	3	12	4	8	.159	.021	64
	3 x 12-6/10	3	12	6	10	.116	.014	46
	3 x 12-8/12	3	12	8	12	.082	.009	32
	4 x8-8112	4	8	8	12	.062	.013	27
	4 x8-10/12	4	8	10	12	.043	.013	20
	4 x 12-0/4	4	12	0	4	.221	.040	94
	4 x 12-2/6	4	12	2	6	.162	.029	69
	4 x 12-4/8	4	12	4	8	.120	.021	51
	4 x 12-6/10	4	12	6	10	.087	.014	36
	4 x 12-10/12	4	12	10	12	.043	.009	19
	6 x 12-00/4	6	12	00	4	.172	.040	78
	6 x 12-0/4	6	12	0	4	.148	.040	69
	6 x 12-2/2	6	12	2	2	.108	.054	59
	6x 12-4/4	6	12	4	4	.080	.040	44
	6 x 12-6/6	6	12	6	6	.058	.029	32

'Usually furnished only in galvanized wire

A.7 PROPERTIES OF ASTM STANDARD REINFORCING BARS

BAR SIZE DESIGNATION	AREA* SQ. INCHES	WEIGHT POUNDS PER FT.	DIAMETER" INCHES
#3	.11	.376	.375
#4	.20	.668	.500
#5	.31	1.043	.625
#6	.44	1.502	,750
#7	.60	2.044	.875
#8	.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.