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16. Abstract


This report is the first in a series outlining a major study of the behavior of post-tensioned concrete box girder bridges with post-tensioning tendons external to the concrete section. It presents an extensive literature review tracing the history and development of the technology connected with the use of external post-tensioning systems for bridges. Many of the references summarized have previously been available only in French or German language versions.

The report summarizes the historical development of external post-tensioning, the advantages and disadvantages of external post-tensioning, and highlights the design and construction details for deviators. It presents the general basis for a detailed investigation of deviator design and briefly describes development of a complex testing rig for external tendon deviator tests.


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# STATE OF THE ART EXTERNALLY POST-TENSIONED BRIDGES WITH DEVIATORS

by

**L. C. Powell, J. E. Breen, and M. E. Kreger**

Research Report No. 365-1

Research Project 3-5-85/8-365

"Evaluation of Strength and Ductility of Precast Segmental Box Girder Construction with External Tendons"

Conducted for
Texas
State Department of Highways and Public Transportation
In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

June 1988

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

# PREFACE

This report is the first report in a series which summarizes an investigation of the behavior of precast segmental box girder bridges with external tendons. This report gives a state-of-the-art overview of external tendon bridges with emphasis on the post-tensioning details and particularly the method of attachment of the tendons to the box girders at intermediate points or deviators.

This work is part of Research Project 3-5-85-365 entitled "Evaluation of Strength and Ductility of Precast Segmental Box Girder Construction with External Tendons." The research was conducted by the Phil M. Ferguson Structural Engineering Laboratory as part of the overall research programs of the Center for Transportation Research of The University of Texas at Austin. The work was sponsored jointly by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration under an agreement with The University of Texas at Austin and the State Department of Highways and Public Transportation. Important financial support to augment the main program and in particular to develop the complex testing rig utilized for the subsequent deviator tests was provided by the National Science Foundation through Grant ECE- 8419430, "Seismic Behavior of Prestressed Concrete Segmental Box Girders with External Tendons." The University of Texas at Austin contributed additional direct salary support to Ms. Powell under the auspices of the Nasser I. Al-Rashid Chair in Civil Engineering.

Liaison with the State Department of Highways and Public Transportation was maintained through the contact representative, Mr. Alan Matejowsky. Mr. Peter Chang was the contact representative for the Federal Highway Administration.

This portion of the overall study was directed by John E. Breen, who holds the Nasser I. Al-Rashid Chair in Civil Engineering. He was assisted in this position by Michael E. Kreger, Assistant Professor of Civil Engineering who was co- investigator on the overall TSDHPT and NSF projects. The development of the state-of-the-art report and of the deviator testing rig was the direct responsibility of Lisa Carter Powell, Assistant Research Engineer.

## SUMMARY

This report is the first in a series outlining a major study of the behavior of post-tensioned concrete box girder bridges with post-tensioning tendons external to the concrete section. It presents an extensive literature review tracing the history and development of the technology connected with the use of external post-tensioning systems for bridges. Many of the references summarized have previously been available only in French or German language versions.

The report summarizes the historical development of external post-tensioning, the advantages and disadvantages of external post-tensioning , and highlights the design and construction details for deviators. It presents the general basis for a detailed investigation of deviator design and briefly describes development of a complex testing rig for external tendon deviator tests.

# IMPLEMENTATION

This report provides a background overview of the development of external tendon post-tensioned concrete bridges. It outlines many of the special aspects of the technology connected with the use of external post-tensioning systems for bridges and provides a summary of a great deal of contemporary foreign experience with such systems. Its primary purpose is to provide detailed background to designers, constructors, and district level field engineers so that they can better understand the general nature and the importance of details of this relatively new technology. Detailed deviator design criteria are presented in the second report of this series.

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# CHAPTER 1. INTRODUCTION

## 1.1 Background

The development of post-tensioned concrete box girder bridges in the U.S. has progressed at a remarkable rate. The introduction of segmental technology, with its time-saving and economic advantages, has resulted in widespread use of segmental prestressed box girder construction for medium to moderately long span bridges. An important recent development in U.S. box girder bridge construction is the use of external post- tensioning tendons (tendons external to the concrete cross section), as opposed to traditional internal tendons which are contained in ducts within the webs or flanges. The United States' first externally post-tensioned concrete box girder structure, the Long Key bridge, was designed by Figg and Muller Engineers, Inc. and was completed in 1980. Long Key was one of four externally post-tensioned bridges linking the Florida Keys. Since 1980, a significant number of these bridges have been built and many more are in design and planning stages. At the present time, the Texas Department of Highways and Public Transportation is involved in a four-part project to construct several miles of elevated highway through San Antonio. Segmental precast box girders with external tendons were the lower cost alternates bid by the contractors and are being used throughout that project.

"Internal post-tensioning" refers to the practice of embedding tendon ducts, in straight or draped patterns as required by design, within the webs and flanges of the box girder section. This practice requires time-consuming placing and securing of the ducts inside the box girder reinforcing cage. The presence of multiple ducts often results in congestion and interference with the reinforcing cage (Fig. 1.1). After the concrete is placed and cured, or after precast segments are assembled, the tendons are pulled through the embedded ducts and then stressed. After post-tensioning, the ducts are normally cement grouted. The grout bonds the tendon to the duct and the concrete along the full length of the tendon, and, if the ducts are completely filled with a dense grout, should improve corrosion protection for the tendon.

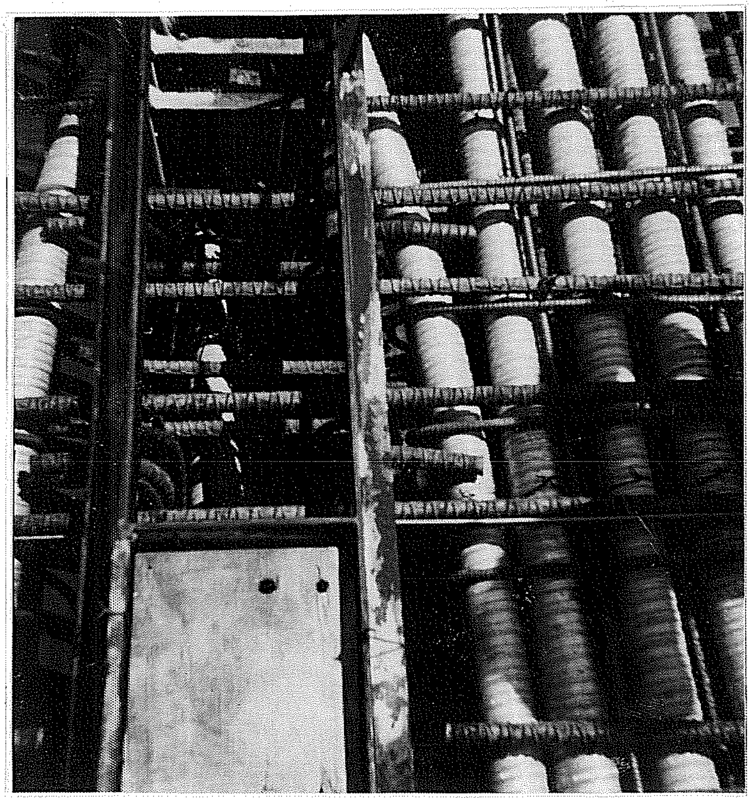


Fig. 1.1 Congestion of Ducts for Internal Tendons

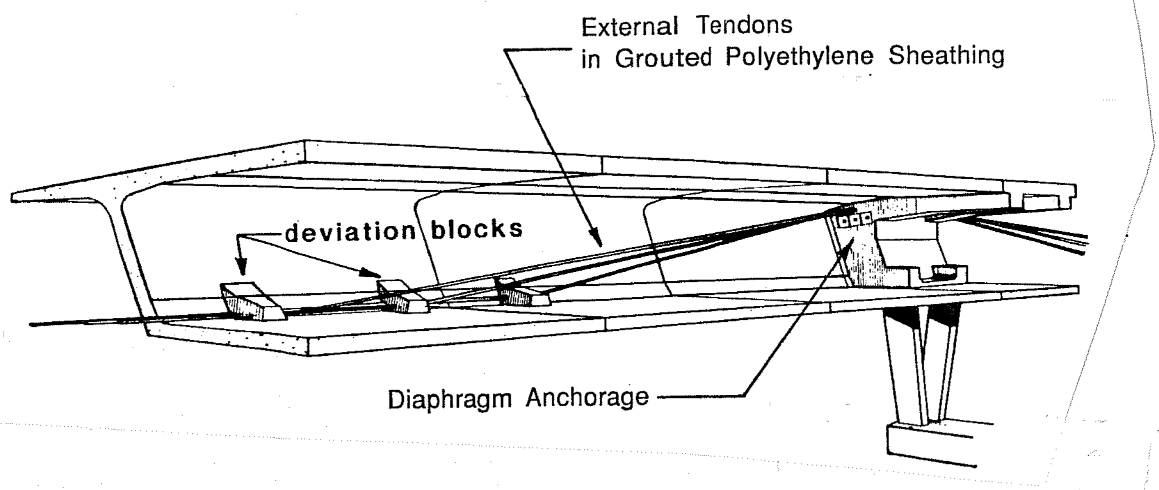



Fig. 1.2 External Post-tensioning in Long Key Bridge

"External post-tensioning" implies that the tendons are removed from the webs and flanges of the concrete section, and are relocated inside the void of the box girder, or between the webs of non-box girders. The draped profile is maintained by passing the tendons through deviation devices cast monolithically with the webs and/or flanges at discrete points along the span. These "deviators" vary in shape and size, though the most common form is a small block or saddle located at the junction of the web and flange of the box girder section. Anchorages for the external tendons are usually placed in thick diaphragms situated over the piers, although blister anchorages are sometimes used at intermediate points within a span. Tendons typically overlap at diaphragm anchorages for continuity. The cut-away view of the Long Key bridge in Fig. 1.2 clearly illustrates the concept of external post-tensioning. The only positive connection of the external tendon to the concrete section occurs at anchorages and deviators. Between attachment points, the exposed tendon is enclosed in sheathing, typically polyethylene tubing. The tendon is usually grouted along its entire length for protection against corrosion. External tendons are considered unbonded since the majority of the tendon is not bonded to the concrete section and the strains in the tendon are independent of the strains in the adjacent concrete sections.

The behavior of bridges constructed using external tendons and subjected to overload has not been thoroughly documented. Uncertainty also exists concerning the proper design criteria and methodology for deviation details. A study is currently being conducted in the Ferguson Laboratory to explore these topics. The scope includes testing of a scale model of a three-span externally post-tensioned precast segmental box girder bridge and related programs to evaluate current deviation details, to develop design criteria for deviators, and to evaluate the efficiency of epoxy joints. These subjects will be treated in successive reports in this series.

## 1.2 Objective and Scope

The primary objectives of this report are twofold. The first is to report on the current state of the art of external tendon details in order to provide general background for the designer contemplating use as well as to provide direction for the main project and satellite studies. The second is to report on the development of specialized testing facilities for evaluating external tendon deviator performance.

1.2.1 Literature Review. Experimental research in the area of external post-tensioning for bridges is just beginning in this country, although research efforts in Europe, especially in France, have been underway for some time. In fact, although external post-tensioning is a very recent development in the United States, the concept has been incorporated in a number of European structures over the last several decades. The object of the literature review is to trace the development of the use of external tendons, citing both successes and problems that have been experienced from inception to the present. Such information provides insight to the current state of the art and points to uncertainties that could benefit from experimental investigation.

1.2.2 Experimental Setup. The scope of this report's treatment of the experimental program is to briefly outline the design and construction of a generalized test setup for evaluating external tendon and deviator details. The objective is to provide a versatile facility for a series of model tests that may eventually result in the refinement of deviator design and detailing, and lead to more accurate modelling of deviators for analysis. The setup is also intended to be adaptable for future studies of external tendon systems and variables for topics such as minimizing fretting fatigue, improving corrosion resistance and improving localized bonding to the section.

## 1.3 Summary of Following Chapters

The body of this report is organized in the following way:

Chapter 2 contains an extensive review of available literature on the general subject of external post-tensioning. It includes descriptions of the earliest uses of external tendons, and follows the development through modern structures. Advantages and disadvantages of external post-tensioning are discussed in Sec. 2.3. Chapter 3 specifically treats the state of the art for the deviators and includes descriptions of typical deviation details and problems that have been experienced in actual structures. Chapters 2 and 3 also summarize all of the completed and ongoing research efforts regarding external tendons and deviators that are known to date.

Chapter 4 covers the development of the special experimental facility for external tendon and deviator testing.

The final chapter presents a brief summary of the work contained herein.

# CHAPTER 2. STATE–OF–THE–ART EXTERNAL POST–TENSIONING

## 2.1 Introduction

In undertaking an investigation into a new area, the first logical step is to assess the developments that have led up to the current state of the art. Such assessments provide direction both for a research program and for designers interested in applying the new technology.

An attempt has been made to glean information from all possible sources. This included detailed and in-depth review of all available literature on the subject of external post-tensioning, personal interviews with key figures in the industry and academia who have been directly involved with the subject, and first-hand visits to project sites where external tendons are currently being used. Most of the advancement and development of external post-tensioning technology has originated in France in the short span of a couple decades. The authors gained much insight through direct communication with the outstanding French engineer, Monsieur Michel Virlogeux, chief engineer for the Service d'Etude Techniques des Routes et Autoroutes (S.E.T.R.A.) of France and a leading professeur at L'Ecole des Ponts et Chaussees. Considerable technical information was also obtained from Monsieur Jacques Trinh and the laboratory at Saint-Rémy-Les-Chevreuse where related research is currently ongoing. The firm of Figg and Muller Engineers, Inc. provided a great deal of information and guides for visits to projects in the Florida Keys and San Antonio.

In order to clearly denote sources of information, the following system has been devised: Groups of sentences or entire paragraphs that are attributed to one source will be followed by the reference number enclosed in brackets [ ]. Sources for individual statements will be enclosed in conventional parentheses ( ).

## 2.2 Historical Development of External Post-Tensioning

In assessing the current state of the art, it is important to examine the history of the developments that have taken place. Knowledge of concepts and details envisioned by early engineers, and their subsequent successes or failures, gives a much clearer understanding of the task at hand.

2.2.1 Early German Bridges. The mention of early historical developments in prestressed concrete tends to bring to mind the name of Eugene Freyssinet. Often referred to as the "father of prestressed concrete," Freyssinet in fact patented his invention in 1928 (1). It was not until after the massive destruction of World War II that he actually applied his own patent to prestressed bridges (1).

In pre-war Germany, however, the eminent engineer Franz Dischinger was pioneering the idea of prestressed concrete (2). Dischinger designed several bridges using prestressing cables external to the cross section of the structure, perhaps a natural progression of thought from the concept of cables in suspension bridges (3). The prestressing steel was actually large diameter rod with butt welded threaded couplings (7). The rods remained ungrouted and therefore unbonded to the concrete (2).

Dischinger's first attempt, the Saale-brucke, holds claim as the first prestressed bridge in the world. It was also the earliest application of external prestressing. This structure, completed in 1928, spanned 200 feet over the Saale River in Alsleben. Two more bridges of this type followed; one, a highway bridge at Aue in Saxony built in 1936, and a third, the Warthe-Brucke in Poznam. The Warthe-Brucke had a considerable span (for a concrete bridge of that era) of 245 feet, but it was never finished due to the outbreak of the war in 1939 [2].

The road bridge in Aue, a three-span cantilevered structure, had a main span of 210 feet. Though carefully designed and well constructed, this bridge did not perform satisfactorily because of limitations in the state of the art at that period. The "high-strength" rods that Dischinger used for these early projects had a tensile strength of about 60 ksi, only normal grade non-prestressed reinforcing steel by modern standards. Creep, shrinkage and relaxation losses were unknown and not considered in design. Consequently, after 25 years in service, the loss in prestress accumulated to about 75% of the original prestress level. The concrete structure underwent large displacements and cracking [7].

The cracking in the Aue bridge made Dischinger aware of the deformation of concrete due to creep and shrinkage, and the considerable effect of those deformations on prestress loss, especially with low strength prestressing tendons. German engineers began to realize the impact of plastic deformation on the structural behavior of concrete. Further development with unbonded prestressing continued without much success for some time to come. It was not until 40 years later that unbonded prestressing was used successfully in Germany, and then with high strength, highly stressed tendons [7].

2.2.2 Post-War Development in Belgium and France. The distinguished Belgian engineer Gustave Magnel reported in considerable detail about numerous early experiments and actual bridge applications involving external tendons [30]. In his 1954 text he showed examples of I-beam and box girder beams with external tendons deviated by rods or pins projecting from the web. Several tests were reported in detail of beams made with precast concrete blocks jointed with cement mortar and stressed with external tendons attached only at the end anchorages and passing through openings in one or two interior diaphragms to ensure that the cables deflected with the beam and that the eccentricity of the cable remained constant during the deflection. Final failure was sudden with the concrete in the top flange crushing with only a relatively modest increase in the effective stress in the unbonded tendons. Magnel reported that such external tendon segmental block beams were used in 1944 by the Canadian Royal Engineers for a wartime bridge to carry pipelines over the Terneuzen Canal.

Magnel also reports on a number of actual bridges constructed under his supervision prior to 1954 and using external cables. Notable among these were the Eecloo Bridge of 66 ft span which had a series of both straight and draped cables in sheaths which passed through slots in the diaphragms and the Canal Bridge at Baudour, Belgium, which had polygonal cables whose directions were changed by cross-stiffeners inside the box girders. One of the most interesting of these early examples was the continuous two span girder bridge constructed across the river Meuse at Sclayn, Belgium. Each span was 206 ft and the three-cell box girder cross section had rectangular openings provided in the diaphragms for inspection and passage of the external tendons. After post- tensioning, the tendons were encased in fine concrete placed in formwork around the tendons. The numerous examples cited by Magnel indicated wide acceptance of this construction type in Belgium around 1950.

The devastation wrought by the war left France with 103 destroyed or damaged major bridges, of which 32 were bridges over the Seine, the Marne, and the Oise rivers which were totally destroyed [4]. The tremendous task of reconstruction, plus shortages of essential materials (especially steel) inspired innovation. During this period, Freyssinet was responsible for six major reconstruction projects in prestressed concrete over the Marne [2]. Also at this time, interest revived in external post-tensioning, possibly due to a desire to evade the Freyssinet patents [3]. Four innovative bridges were built using external tendons: the Vaux-Sur-Seine in 1951, the Port-à-Binson bridge over the Marne, the bridge at Villeneuve-Saint-Georges and the Can Bia bridge, all three completed in 1953 [3].

In a recent report, French engineers studied these four early examples of external post-tensioning, citing observations made during their construction and also evaluating their performances after 30 years in service [4].

Vaux-Sur-Seine, 1951. The bridge at Vaux-Sur-Seine, (See Fig. 2.1), replaced a reinforced concrete bowstring structure which "disappeared" on June 10, 1940. The construction of the new three span two-celled box girder bridge (main span of 80 ft) employed an interesting technique for stressing the tendons as illustrated in Fig. 2.2. Bundles of prestressing wire, a pair at a time, were pushed apart with relatively low capacity jacks, and held in place by a large concrete chock. Anchorages for the bundles, the subject of a previous study, were relatively cumbersome. Two different systems were employed (see Fig. 2.3); the first wrapped individual wires in a doubled hook fashion around anchor bars. The second configuration, descriptively dubbed "pig tail" anchorages, comprised pairs of wires ending in spirals. Concrete partitions served as deviators to impart a classical draped profile to the tendons. In order to minimize friction at the deviations, the passage holes were fitted with well greased metal pieces curved to the shape of the final tendon profile. Wirope No. 2, a grease product for protecting suspension cables, coated the prestressing steel to prevent oxidation. Observations made during construction revealed that no problems occurred with the stressing method. Also noted was the ability to measure loss of prestress by registering the frequency of vibration of the exposed prestressing cables. It was proposed that any losses that should occur could be easily corrected. Some concerns were expressed regarding corrosion of the cables, but since the tendons would be protected from the elements by being enclosed within the box girder, concerns were dismissed [4].

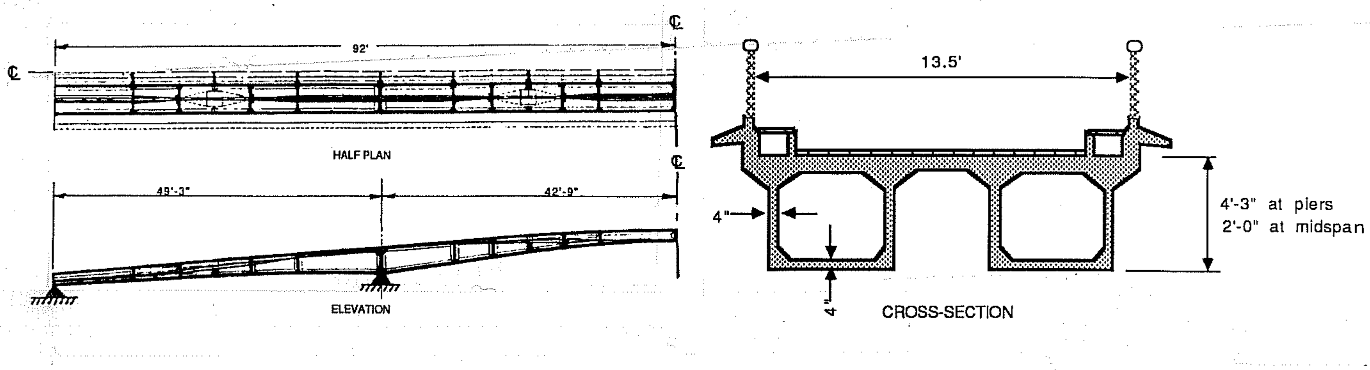



Fig. 2.1 Vaux-Sur-Seine Bridge

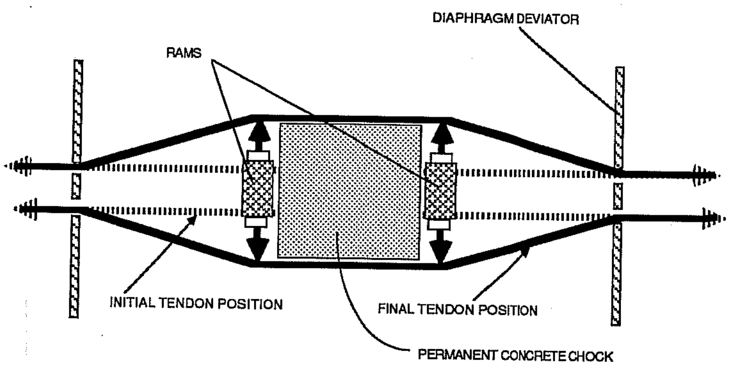



Fig. 2.2 Vaux-Sur-Seine Bridge, Stressing Method

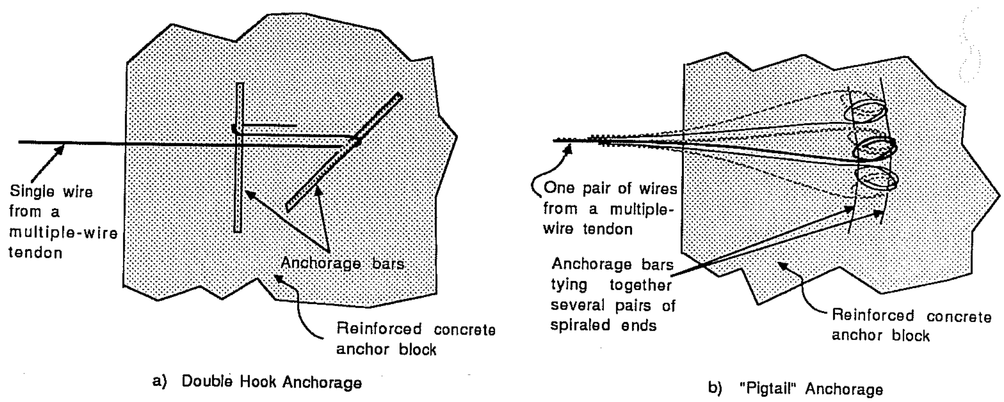



Fig. 2.3 Vaux-Sur-Seine Bridge, Anchorages

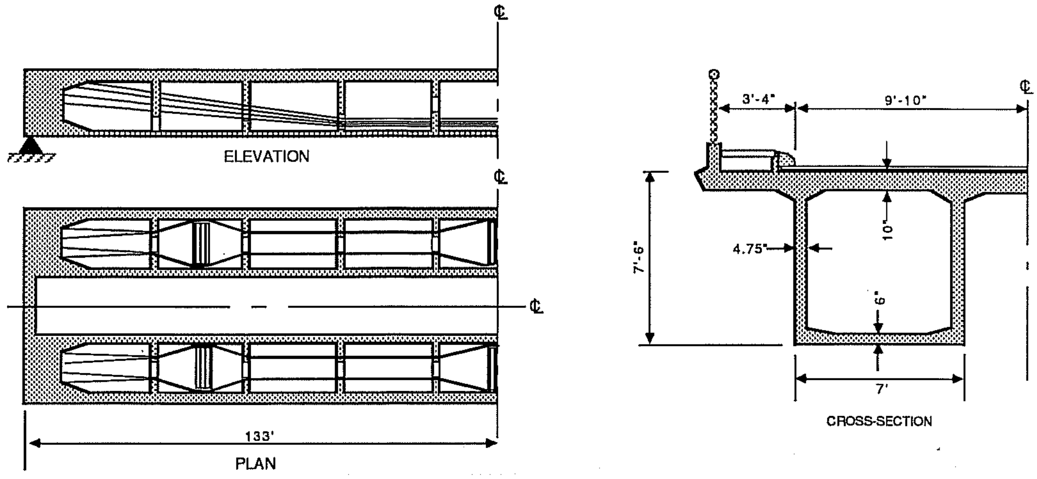



Fig. 2.4 Port-à-Binson Bridge

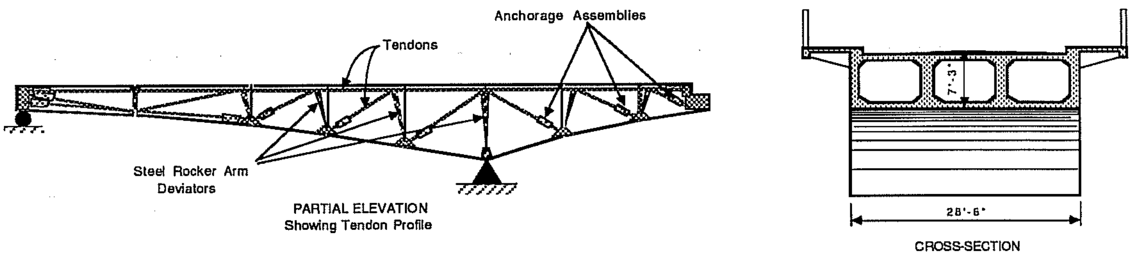



Fig. 2.5 Villeneuve-Saint-Georges Bridge

An inspection of Vaux-Sur-Seine in 1981 revealed an absence of cracking in the concrete, but several ruptured wires in the prestressing cables showed evidence of corrosion caused by condensation within the enclosed box. It seems that ventilation had been practically eliminated due to nests built by birds blocking the aeration ports. As a safety measure against further loss of capacity from the corroded wires, additional external post-tensioning, protected in greased PVC ducts, was added shortly after the inspection [4].

Port-à-Binson, 1953. The bridge at Port-à-Binson, a double box girder spanning 133 feet, utilized a prestressing system similar to Vaux-Sur-Seine (Fig. 2.4). Thirty years after construction, the interior of the girder was found to be perfectly dry, well ventilated, and the concrete structure free of visible cracking. The bituminous material which had been applied for tendon protection (and reapplied in 1960), had worked well. There was one note of concern: pigeons had roosted inside the box girder, evidently making a perch of the topmost cables since the lower cables were covered with droppings! Simple grills were recommended to resolve the problem [4].

Villeneuve-Saint-Georges, 1953. The bridge at Villeneuve-Saint-Georges, with a cantilevered center span of 250 feet over the Seine, was a three-celled variable depth box girder. The interiors of the cells were equipped with electric lighting systems to facilitate inspection and maintenance of the suspension bridge cables that made up the external tendons. The tendon profile was somewhat complicated (Fig. 2.5). Inclined tendons were deviated by means of specially fabricated hinged rockers supported on reinforced concrete bosses cast monolithically with the bottom flanges (Fig. 2.6a). Anchorages for the cables, composed of 193 spiraled 4.1 mm high-strength wires, consisted of molded steel trumpets filled with a lead-tin alloy. Through the base of the trumpet passed four threaded shafts which were anchored back to the concrete section by means of embedded, hooked reinforcement welded to their extremities (Fig. 2.6b)[6].

Though an innovative, pleasing and efficient structure, the Villeneuve-Saint-Georges bridge was uneconomical compared to conventional alternatives due to the complexity of the details envisioned for

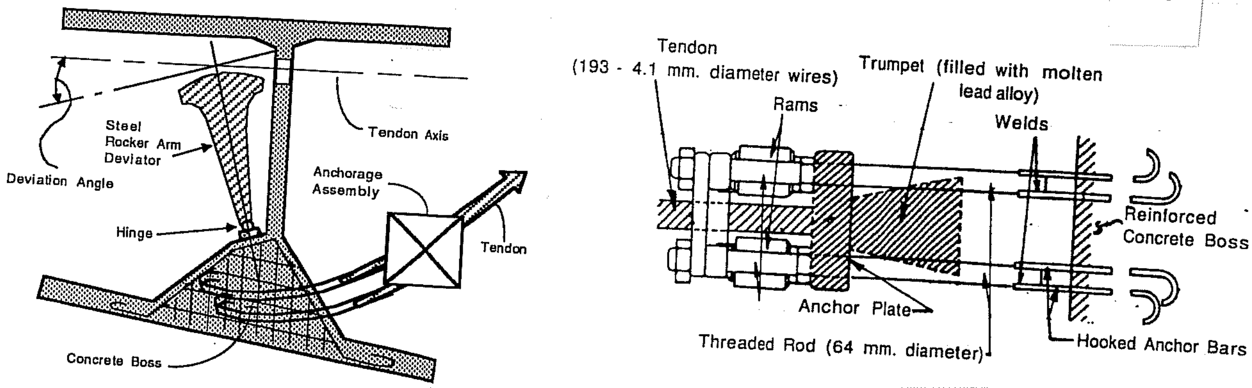



Fig. 2.6 Villeneuve-Saint-Georges Bridge, Details

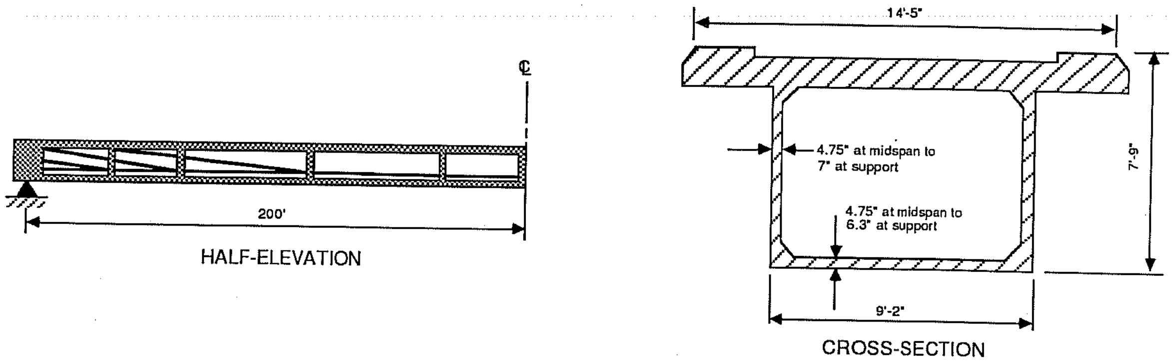



Fig. 2.7 Can Bia Bridge

the external tendons. The post-tensioning method did have the advantage , however, that the stress in the tendons could be easily monitored and corrected if necessary [6].

Can Bia, 1953. The bridge at Can Bia was a box girder structure spanning 190 feet. External tendons were deviated through transverse diaphragms and anchored in the top slab (Fig. 2.7). The tendons were covered with a bituminous paint. Of the four bridges studied, this structure sustained the most deterioration over the 30 year service period. Cracks in the diaphragms and ends of the girder were apparent and were attributed to the diffusion or spreading out of the concentrated tendon forces. More serious, however, was the rupture of several wires, approximately 8% of the total area of the post-tensioning tendons. Most of these ruptures were located in the first compartments of the bridge and were not related to the cracking of the concrete. Corrosion was responsible for the damage to the tendons which were poorly waterproofed and ventilated [4].

Summary. In review, after 30 years in service these four pioneering bridges showed few cracks in the concrete. The main problems that were encountered were loss of prestress and failure of tendon protection measures to adequately guard against corrosion.

### 2.2.3 Other Early Bridges with External Tendons.

Rio Caroni, 1965. In the early sixties, the German structural engineer Fritz Leonhardt designed the bridge at Rio Caroni in Venezuela [3,5]. Rio Caroni was one of the first bridges built by the incremental launching or pushout method. Both internal and external tendons were used (Fig. 2.8). The external tendons served two purposes. First, during the launching procedure, the tendons were straight and applied an axial compression to the section being launched to counteract flexural stresses produced during the launching procedure. After all sections were launched and the superstructure set in its final position, special devices were used to deviate the straight tendons, downward near midspan, and upward near the piers in order to achieve a parabolic profile. The deviators were reinforced concrete ribs fitted with formed steel slats which allowed the tendons to be jacked up or down. The wires were enclosed within boxes that served to deviate as well as to separate the concentration of strands into

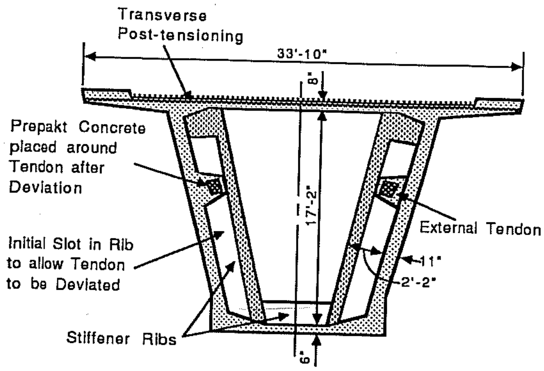
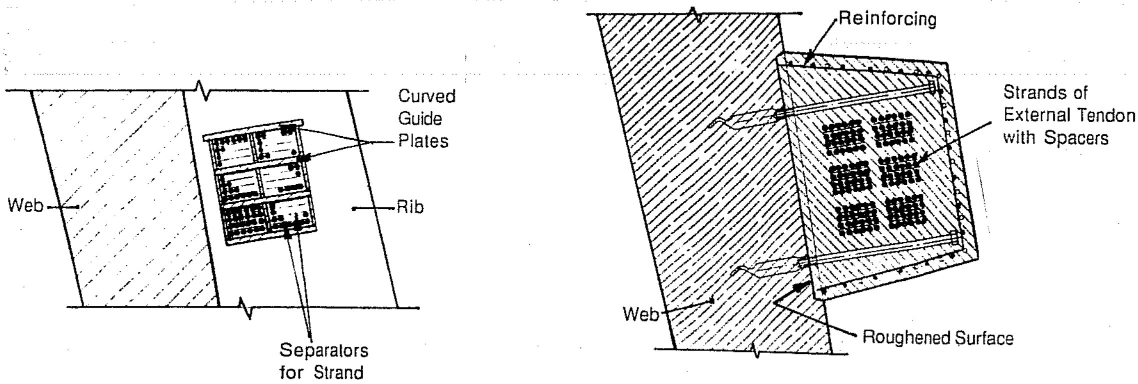



Fig. 2.8 Rio Caroni Bridge, Cross Section




a) Deviator

b) Concrete encasement for external tendon

Fig. 2.9 Rio Caroni Bridge, Details

tendons (Fig. 2.9a). After deviating the tendons, they were finally fixed into position by the concreting of the rib opening. Between ribs the tendons were enclosed in a concrete encasement which was placed by pumping mortar through pre-packed aggregate. The encasement was bonded to the webs with the aid of anchor bolts (Fig. 2.9b). This encasement provided an ideal corrosion protection, since under the effect of creep the concrete became prestressed [5].

English Attempts. English engineers also experimented with the concept of external post-tensioning for bridges. Under English practices, the external tendons were anchored in blisters such as those sometimes used for conventional internal tendons. This technique complicated construction and produced undesirable local effects. In general, the idea of external post-tensioning did not meet with much success in Great Britain [3].

2.2.4 Experience Gained From Bridge Repairs Using External Tendons. Much of the development of external prestressing technology can be traced to repair and retrofit of prestressed concrete bridges [3]. Additional prestressing has proved to be an efficient method for bridge strengthening and repair. Two methods have been suggested: differential jacking of supports, or adding prestressing tendons. The first method has the advantage of being relatively easy, economical and time-saving. The measure is most likely short term, though, since creep induces force redistributions over time; also, care must be taken to avoid overstressing other parts of the structure. Lecroq presents this suggestion for differential jacking in connection with repairs to French bridges but does not cite specific examples. The second and prevailing method involves additional prestressing with external tendons. While a more permanent solution, it entails several technical problems and considerations concerning tendon layout, deviation devices, anchorages, and choice of duct type and tendon protection scheme [8].

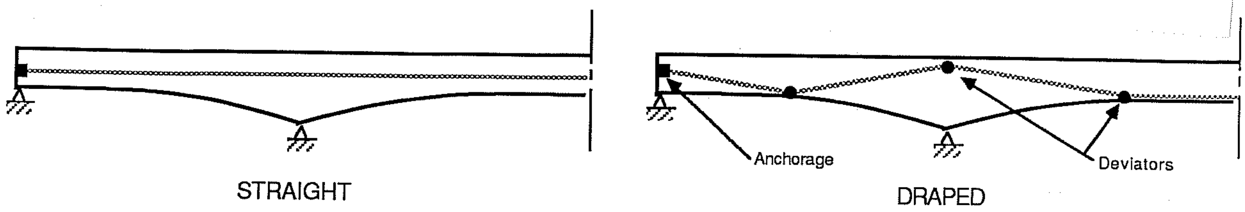



Fig. 2.10 Retrofit External Tendons, Profiles

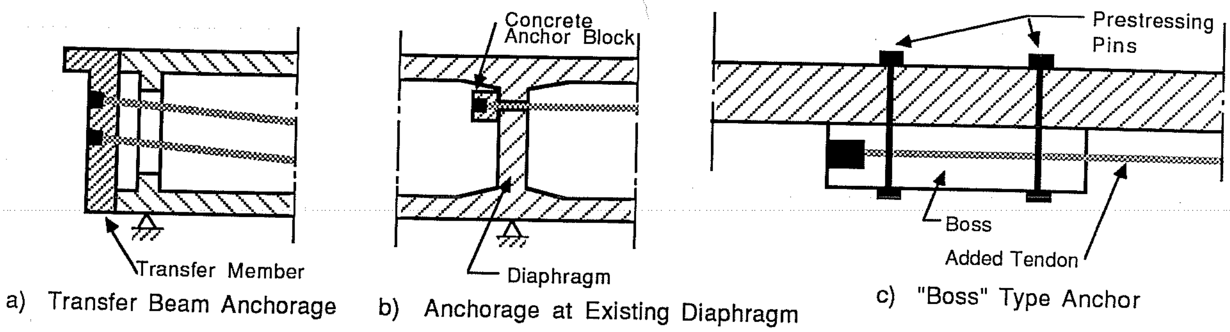



Fig. 2.11 Retrofit External Tendons, Anchorage Schemes

Both straight and draped tendons have been used successfully for retrofit and repair (Fig. 2.10). While straight tendons eliminate the need for deviators and reduce friction losses to practically nil, the result is not usually efficient for flexural resistance and will not increase shear resistance. A polygonal profile, which follows more closely the moment diagram, offers more efficient use of prestress plus increased shear resistance from the inclined tendon. However, the deviators required to achieve the tendon profile, which must be somehow clamped to existing webs or flanges, introduce stress concentrations and increase friction losses [8].

Anchoring the added tendons onto the existing structure presents the most difficult problem. Several alternatives have been devised. A large transfer beam may be cast against the ends of the bridge section (Fig. 2.11a). The transfer member can be designed to uniformly distribute concentrated forces to the existing structure. This procedure however, necessitates using tendons as long as the bridge. Also, the abutments must be demolished and reconstructed, a project that will most likely disrupt traffic flow for a considerable period of time [8]. Alternatively, tendons may be anchored at existing diaphragms, if analysis verifies that the diaphragms can provide adequate resistance. A core must be drilled through the diaphragm and anchorage hardware is embedded in a concrete anchorage block cast against the face of the diaphragm (Fig. 2.11b) [8].

In the absence of existing or adequate diaphragms, a concrete boss can be prestressed to the web or flange of the existing section (Fig. 2.11c). This system is advantageous since the prestress from the added external tendons can be distributed fairly well over the length of the structure. On the other hand, effects of force diffusion create considerable localized stresses which are additive to the existing state of stress. Localized effects have been reduced by locating anchors in the gusset areas at flange-web junctions [8].

Experience has shown that design of anchorage bosses should be conservative. The clamping force should be at least two times the prestress force. Stressing the boss to the web in two stages can minimize losses in the short prestressing pins. First the pins are stressed to the desired force level. Then, just before stressing the added tendon, the pins are stressed again to recover losses from creep and seating. Tests have shown that the tendon force does not distribute uniformly over the contact surface, but concentrates in two zones at the extremities of the boss. Therefore, pins will be more effective if located in these zones. Light roughening to remove laitance has proven the best contact surface preparation. Application of epoxies may improve bond, but may be a trouble source if the epoxy fails to cure [8].

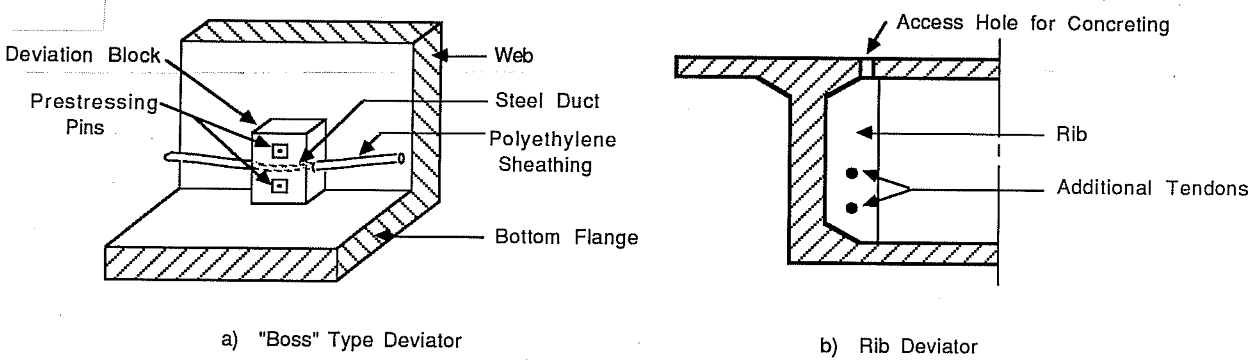



Fig. 2.12 Retrofit External Tendons, Deviators

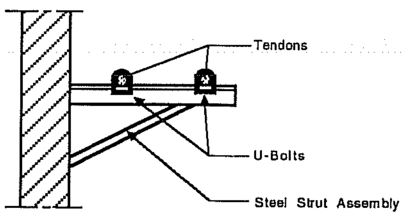



Fig. 2.13 Antivibratory Device

Similar considerations apply to the design of deviators for draped tendons. The forces on deviators are generally small and no problems have yet been encountered with retrofit deviators. Two alternatives have been developed. The first is similar to the anchor boss (Fig. 2.12a). An alternate scheme, (Fig. 2.12b), attaches a reinforced concrete rib to the web with epoxied dowels. The benefit of this method is the distribution of the deviation force along the entire web. One inconvenience is that a hole must be cored in the top flange and deck in order

Ducts located within anchorages and deviators serve two purposes. The duct isolates the tendon from the concrete and assures that the tendon is guided properly. Cold bent rigid metal tubes have generally been used for construction convenience. For protection against corrosion between attachment points, several types of sheathing have been used with varying degrees of success. Early repair projects employed corrugated sheathing, such as that typically used for internal post-tensioning tendons. This type of sheathing was found to be insufficiently watertight. Metal conduit and high density polyethylene tubing have both worked well, with polyethylene the most economical choice today. Experience with galvanized cables has disclosed certain problems. There is a possibility that galvanized cables under tension may be susceptible to corrosive agents such as salts, bird droppings, etc. [8]. On a particular project, zinc accumulated on the anchorage wedges, resulting in the severe detensioning of a tendon [3].

Another problem occurring in repair with external tendons is vibration of external cables unsupported over extended lengths. Natural frequencies for typical prestressed bridge structures range from 3 to 5 hertz. In comparison, the natural frequency of a 12-0.5 in. diameter strand tendon is about 5 hertz for a free length of about 90 feet. Maximum recommended free length has been about 60 feet. Otherwise, intermediate supports, (Fig. 2.13), which can be very light, have been suggested [8].

2.2.4.1 <u>Repairs to French Bridges - 1970's</u>. A number of prestressed bridges built in France prior to 1975 suffered distress (cracking, excessive deflections) due to deficiencies in design and construction. Several factors can be held responsible. Competition in an open bid market spurred minimization of concrete and steel. Loads were sometimes underestimated or ignored (eg. weights of toppings or hardware). A major source of error was the miscalculation of the effects of plastic deformation of concrete. Redistribution of moments due to creep and shrinkage were not considered and loss of prestress due to creep deflections was underestimated. The effects of thermal gradients were ignored. Friction losses of internal tendons in curved or misaligned ducts were underestimated [3]. In some of

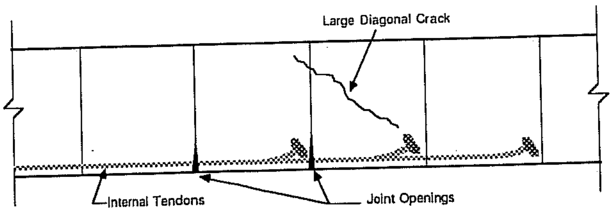



Figure 2.14a Distress in a French Bridge

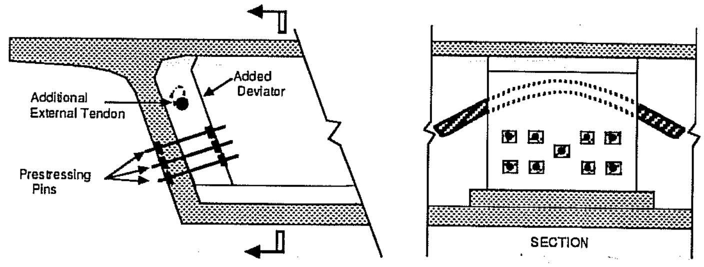



Figure 2.14b Repair Scheme

Fig. 2.14 French Bridge, Distress and Repair

the later bridges, a possible need for future additional prestress was foreseen, and empty ducts and extra anchorages were added during construction for this purpose (8). Still, in some cases even more prestress had to be added than was initially provided for by extra ducts. In one instance, in a precast segmental structure, the empty ducts could not be made perfectly watertight. Rather than risk damage to the structure due to corrosion of the duct or expanding ice, the empty ducts were grouted [3].

The accounts that follow are case-in-point examples of lessons learned through experience and are reported so that these lessons can be incorporated into future design provisions:

A long approach viaduct consisted of a single cell trapezoidal box girder. The precast, prestressed webs were inserted into slip forms in which the remaining parts of the 10 ft long segments were cast-in-place in cantilevered construction. The internal longitudinal prestress was provided by 5 tendons per web; 4 tendons of 12-0.6 in. diameter strands each and 1 tendon of 12-0.375 in. diameter strands each. This particular job was a lump sum bid, so there was a tendency to eliminate any "waste" of materials. In this vein, "exact" quantities required by design were used. Not long after completion, the combined effect of creep, thermal gradients, inadequate shear area and minimal prestress resulted in joint openings and formation of large diagonal cracks in some segment webs (Fig. 2.14a).

Remedial measures included the addition of 19-0.6 in. diameter strands per web in external tendons, corresponding to an augmentation of prestressing steel area of 37%. The draped tendons were deviated and anchored in wide ribs that were cast against and prestressed to the existing webs (Fig. 2.14b). The total cost of the repair was reported to amount to about 12% of the original cost.

The importance of prestress loss due to friction was well illustrated during the construction of the viaduct at Marne-La-Vallee, the first precast segmental railroad bridge. Using jacks equipped with pressure gages to measure applied force at both extremities of a tendon, the considerable friction loss over the length of the tendon became apparent. Additional prestressing steel was required to make up for losses. The French railroad commission, SNCF, had foreseen a possible future need to augment the prestress force, and had requested that provisions be made (extra ducts, anchorages, etc.) to add a supplementary 20%. Unfortunately, an increase in prestress force beyond that figure was required. Six external tendons of 12T15 (metric equivalent to 12-0.6 in. diameter strands) per web were deviated through existing diaphragms over the piers and through attached "boss" type deviators. In light of the repairs made, the efficiency of external prestressing with regard to friction losses was recognized [3].

During load tests on the completed viaduct at Marne-La-Vallee it was observed that the external tendons were easily set to vibrating, necessitating the addition of antivibratory devices. It was also this project that precluded further use of traditional corrugated sheathing for external tendons, as it

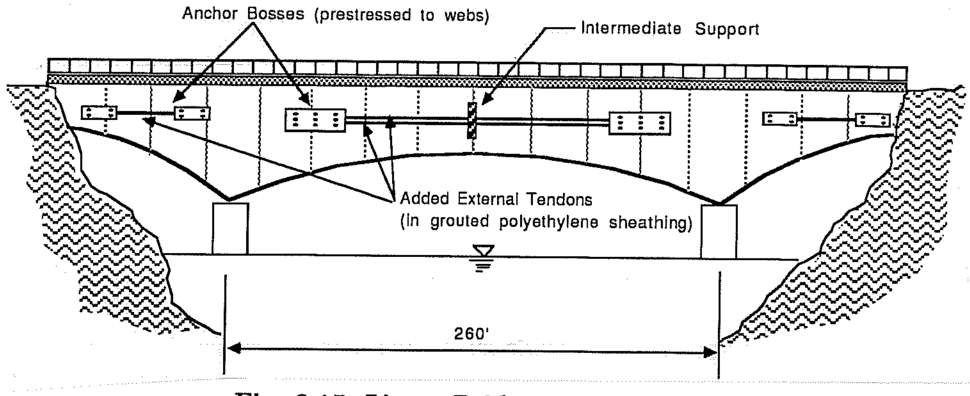



Fig. 2.15 Lievre Bridge, Repair Scheme

became apparent during grouting procedures that this type of sheathing was not sufficiently watertight. The Marne-La-Vallee Viaduct was completed in 1977 [3].

2.2.4.2 Repair of North American Bridges. The North American continent's first precast segmental bridge, an internally prestressed structure with a main span of 260 feet over the Lièvre River in Quebec, Canada, dates to 1967 [9]. The Lièvre bridge was plagued by the same problems experienced by the french bridges, notably the effect of plastic deformation of the concrete and neglect of thermal gradient effects. In addition, insufficient development length contributed to a lack of fixity at the piers. The structure eventually suffered noticeable cracking in the positive moment regions, joint openings, and deflections.

The repair scheme, shown in Fig. 2.15, consisted of straight external tendons, located in the positive moment regions only, anchored in bosses prestressed to the webs of the box girder. An intermediate support was required in the long center span.

Several other Canadian bridges required similar reinforcement as have the cast-in-place segmental bridges at Vail Pass in Colorado.

The lessons learned from these early bridges have been carefully reviewed and form a major basis for the comprehensive design and construction recommendations for post-tensioned concrete segmental bridges developed for consideration by AASHTO by a special Post-Tensioning Institute ad hoc committee in a major NCHRP study (28). Design and construction to these standards should eliminate the chance of such problems.

2.2.5 Modern Bridges with External Tendons. Within the last decade, a number of bridges with external tendons have been built in various parts of the world. These structures have been built with rapid construction times and very competitive costs. To illustrate the diversity of cross sections, tendon profiles, construction methods, etc. of modern externally post-tensioned bridges, this section provides a brief description of several notable structures which have been recently completed or are still in progress.

2.2.5.1 Florida Keys, U.S.A. The first modern bridges to employ external tendons exclusively were a string of four structures linking the Florida Keys: Long Key, Seven Mile, Channel Five, and Niles Channel bridges. The most notable of these bridges, Long Key and Seven Mile, were designed by the consulting firm of Figg and Muller Engineers, Inc.(3) Long Key Bridge, comprising 11,960 total feet of roadway, is of precast segmental construction erected by the span-by-span method. The structure is continuous over eight spans with expansion joints at the pier segments and slender precast V-piers. In addition to the two-lane roadway, the structure was designed to carry a 30 in. diameter water main within the void of the box section [9].

An underslung erection truss was utilized to support the 65 ton segments in a typical span of 118 feet, until the entire span could be post-tensioned. Anchorages are located in thick diaphragms within the pier segments. A typical span could be erected in two working days. Joints between the typical match-cast segments were left dry [9].

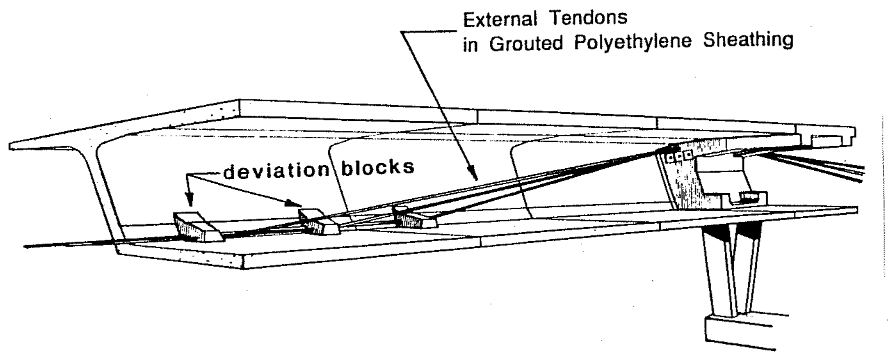



Fig. 2.16 Long Key Bridge, Cut-away View

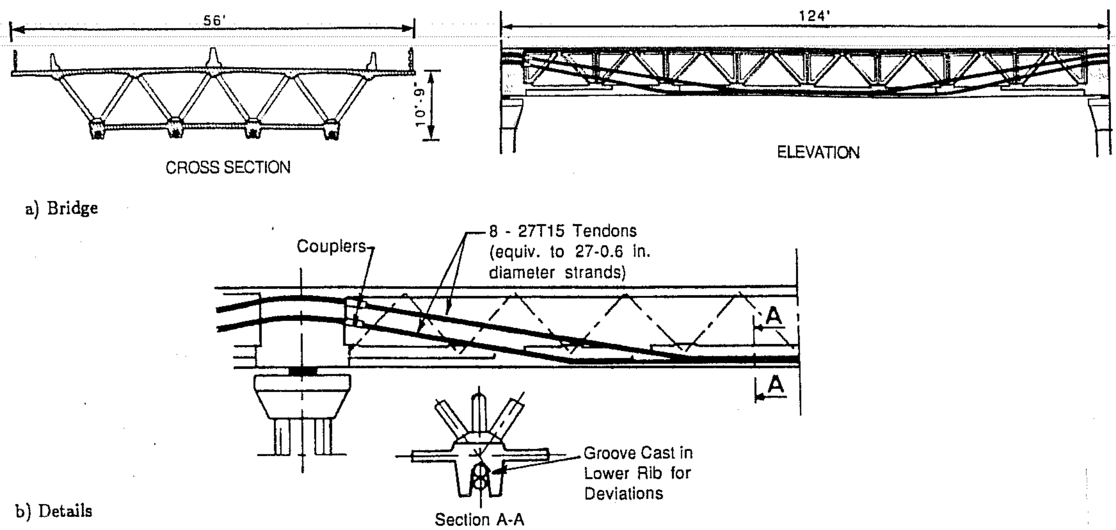



Fig. 2.17 Bubiyan Bridge

Figure 2.16 shows a schematic of the tendon layout for Long Key. Multi-strand external tendons pass through ducts embedded in small reinforced concrete deviation blocks or saddles cast monolithically with the single-cell box girder. The tendons overlap at anchorage diaphragms for continuity. Corrosion protection provided for the tendons between anchorages and deviators consists of grouted polyethylene sheathing [9].

The Seven Mile Bridge is a similar structure with a cross section almost identical to Long Key. The alignment includes both horizontal and vertical curvatures. A different pier design employs hollow box segments post-tensioned together. The erection scheme was changed to include an overhead erection truss instead of the underhung truss used previously. A total of 264 spans of 135 feet were erected in record time [9].

2.2.5.2 Bubiyan, Kuwait. The French firm of Bouygues proposed the design for the Bubiyan bridge in Kuwait. The Bubiyan superstructure incorporates an interesting space truss configuration in which an open lattice system of concrete truss elements replaces the long solid webs of conventional concrete box girders (Fig. 2.17a). The advantages of this open system include a reduction of dead weight and an ability for load redistribution in the unlikely event of a member failure [9].

The external longitudinal post-tensioning is strung in four vertical planes with couplers at the pier segments for continuity. Between anchorages and couplings, tendons are enclosed entirely in polyethylene sheathing. Deviations are accomplished by means of grooves cast in the underside of the bottom ribs (Fig. 2.17b). To avoid damage to the polyethylene in the deviations, the sheathing was reinforced on the inside with flexible metal tubing [3].

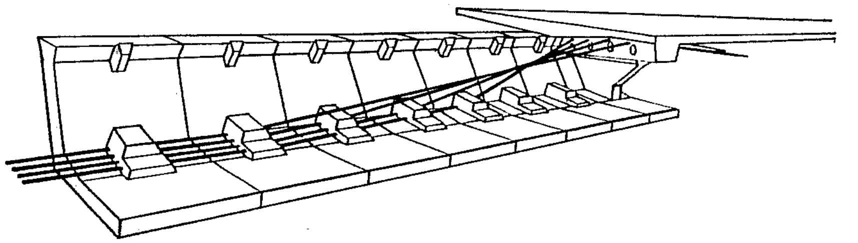


Fig. 2.18 Vallon-des-Fleurs Viaduct

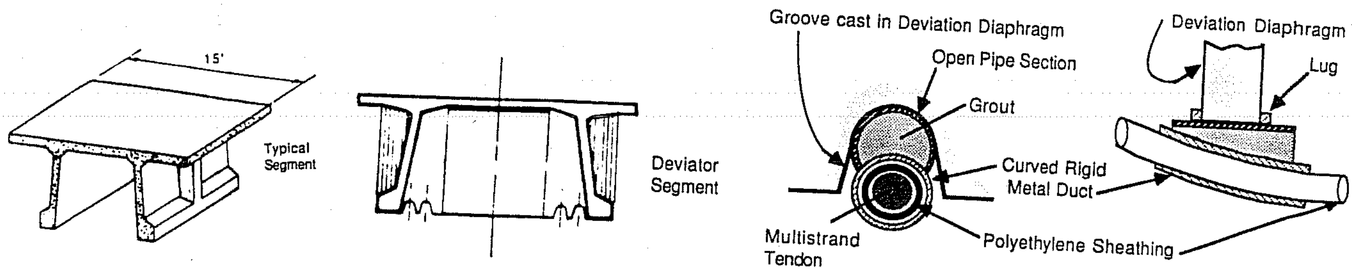



Fig. 2.19 Saint-Agnant Viaduct, Details

2.2.5.3 Banquiere and Vallon-des-Fleurs Viaducts, France. The aforementioned structures all were constructed using span-by-span techniques. French constructors, Campenon-Bernard, wished to utilize the method of progressive placement with provisory cable stays for the construction of two similar viaduct structures. Experience had shown that large friction losses are encountered when traditional internal post-tensioning is used in conjunction with progressive placement construction, due to the complex tendon profile required. The Banquiere and Vallon-des-Fleurs viaducts are both precast segmental parallel box girders post-tensioned solely with external tendons. The tendons are anchored in pier segment diaphragms and deviated in large deviation blocks cast monolithically with the lower web-flange gussets in the center of each segment (Fig. 2.18). Projections at the upper gussets serve as anchorages for temporary dywidag bars used during construction. Between attachment points, tendons are enclosed in high density polyethylene sheathing injected with grease. This practice allows possibility of future tendon replacement [3].

2.2.5.4 Saint-Agnant Viaduct, France. In an effort to maximize efficient use of precast segmental technology applied to short-span bridge structures, Campenon-Bernard devised an open cross section for the Saint-Agnant viaduct (Fig. 2.19a). The open profile was designed to facilitate forming, casting and stripping procedures, as well as economize use of materials. The relatively light segments (less than 16 tons) resulted in substantial savings for the erection appurtenances and equipment required for span-by-span construction. Aiming at a standardization of elements, an external post-tensioning scheme was adopted and a new concept for deviators was devised (Fig. 2.19b) [10].

2.2.5.5 Sermanez Viaduct, France. The Sermanez Viaduct crosses the Rhone River just east of Lyon. Five spans of two parallel variable depth box girders were proposed for the project. The peculiarity of the Sermanez bridge lies in the use of external tendons in conjunction with a cantilever method of construction. The external tendon profile closely matches that of a classical internal prestressing profile for cantilevers (Fig. 2.20). The tendons are anchored and deviated in stout ribs cast monolithically with the precast web segments [11].

The single-cell box girder consists of a pair of precast webs, joined by top and bottom slabs of cast-in-situ concrete. A reduction in web width from 40 cm (15.75 in.) (required for an identical structure with internal tendons) to 32 cm (12.6 in.) was achieved; the difference of 8 cm (3.15 in.) corresponds to the diameter of an internal duct. A further reduction was theoretically possible (24 centimeters would have corresponded to the minimum required for shear), but the dimension was left conservatively at 32

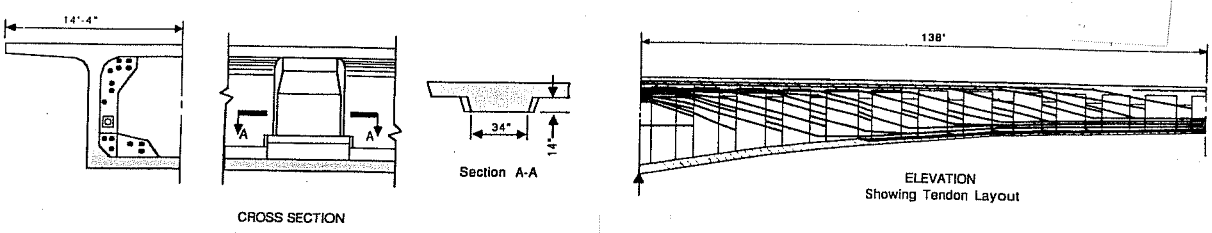



Fig. 2.20 Sermanez Viaduct

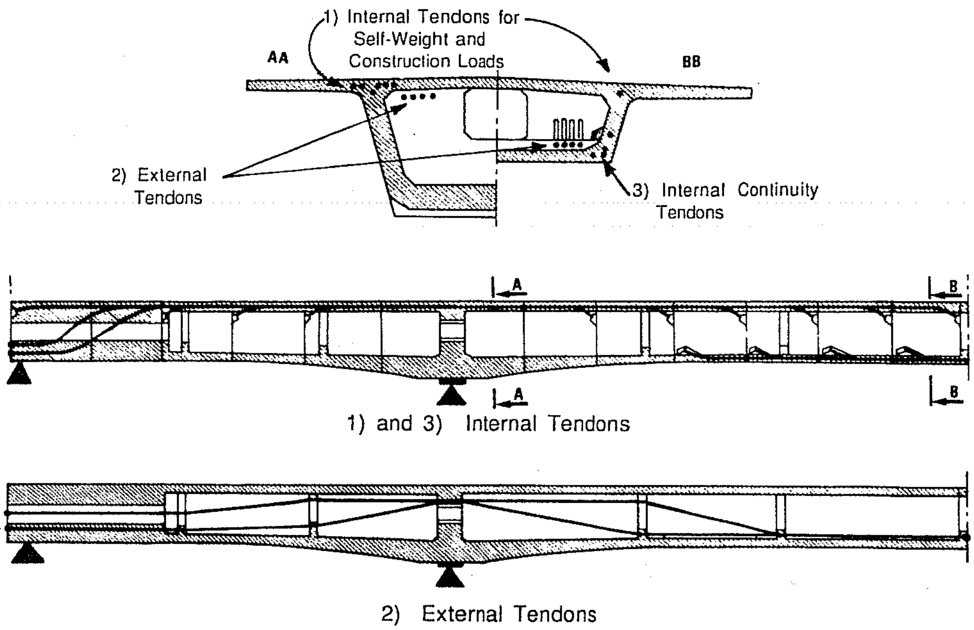
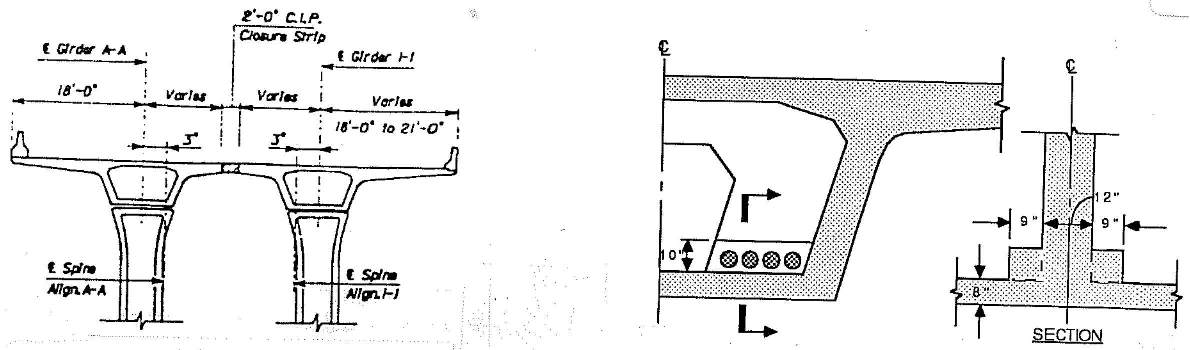



Fig. 2.21 French Bridges, Mixed Tendon Layout, Example

cm. As a result, the volume of concrete required for both alternates was the same; the savings offered by thinning the web being offset by the ribs [11].

Another deviation from common practice on the Sermanez project was the use of galvanized strand for the external cables. The justifications for the decision were numerous and precautions were taken to prevent corrosion or other problems that might stem from the use of galvanized material. Several tests ensured that the galvanized coating was sufficiently thick and uniform, and would not flake or crack due to bending or rubbing in the deviations. To prevent the mishap that had occurred on a previous project due to accumulations of zinc in the wedges, a procedure was devised so that fresh wedges would always be used for final seating of the strands. Anticorrosive measures for anchorages included galvanized trumpets and anchor plates, and chrome plated wedges. The anchorage hardware and tendons were left bare, so that the tendons are completely replaceable [11].

2.2.5.6 <u>French Bridges with Mixed Tendons</u>. The span-by-span method of construction lends itself particularly well to the exclusive use of external post-tensioning. For other methods of construction such as balanced cantilever, progressive placement, or incremental launching, however, internal tendons are advantageous to support the weight of the structure during construction. In France, a synthesis of these two ideas has resulted in a number of projects with "mixed tendons", a combination of internal and external post-tensioning. In general, three separate "families" of tendons can be distinguished (Fig. 2.21). First, internal tendons in the cantilever are proportioned to carry only the self weight of the structure and construction loads. These tendons are typically straight and are anchored in the upper gusset areas of the box girder section. The second set is comprised of draped external tendons for continuity. These tendons carry most of the live and superimposed dead loads and the inclination of the tendons increases shear resistance. Typically, the external tendons are stressed near completion




a) Cross section

b) Diaphragms

**Fig. 2.22 San Antonio Y**

of the structure. The final set consists of internal continuity cables, usually straight, anchored in the lower gussets. Three prominent examples of mixed tendon construction, each built using a different construction technique, are the Fleche Bridge, the viaduct at Pont-a-Mousson, and the Cergy-Pointoise Bridge [3].

2.2.5.7 Recent U.S.A. Projects. The newly opened Sunshine Skyway bridge crosses over four miles of Tampa Bay in Florida. The precast segmental structure has cable stayed main spans, the largest of which is 1200 feet. The externally post-tensioned approaches consist of 8 and 10 span continuous twin box girder units with 135 ft spans. The designers are Figg and Muller Engineers, Inc. [20].

San Antonio Y is a multi-phase project which will add an upper deck to interstate highways that pass through the downtown section of San Antonio, Texas. Figg and Muller's design for Phase I of the project includes a precast segmental single-cell box girder superstructure (Fig. 2.22a). The 90 to 100 ft spans are erected span-by-span using underhung erection trusses, and are post-tensioned with external tendons. Tendons are deviated with full-height diaphragms (Fig. 2.22b). Construction of the first phase of the project as well as generally similar second and third phases is presently underway.

## 2.3 Advantages and Disadvantages of External Post-Tensioning

2.3.1 Advantages. Several advantages are offered by the use of post-tensioning tendons external to the concrete section, as opposed to classical internal post-tensioning. Most benefits stem from the fact that tendons and ducts are removed from the webs and flanges of the bridge girder, so that they do not take up space in the concrete cross section and do not interfere with the mild reinforcement in the section. Access to external tendons can facilitate and speed up construction as well as allow for possible replacement of damaged tendons. Problems with fatigue and friction loss can also be reduced. For precast segmental bridges, external post-tensioning can simplify and speed construction.

Section Free of Ducts. Removing tendons and ducts from the concrete section allows reduction in web thickness. Thinner sections mean a possible reduction in dead load and the savings that implies (Fig. 2.23a).

The absence of ducts in the webs allows more efficient use of web area for shear resistance. Tests have shown that the effective web width for shear capacity calculations should be the actual width minus half the diameter of the grouted post-tensioning ducts contained therein (Fig. 2.23b) [3].

No Interference with Passive Reinforcement. Displacing ducts to outside the section eliminates the time consuming operations of placing, positioning, and securing the ducts inside the reinforcing cage. Interference with passive reinforcement is eliminated. Mild reinforcing cages can be assembled without fear of later interference with post-tensioning ducts. This benefit has the greatest impact in segmental construction since "assembly line" efficiency can be achieved.

Reduced Congestion. The absence of ducts within the mild reinforcing cage relieves congestion, resulting in better conditions for concrete placement and consolidation. Incidents of poorly consolidated concrete occur routinely when ducts are grouped together in closely spaced bundles, which is often the case with traditional internal tendons.

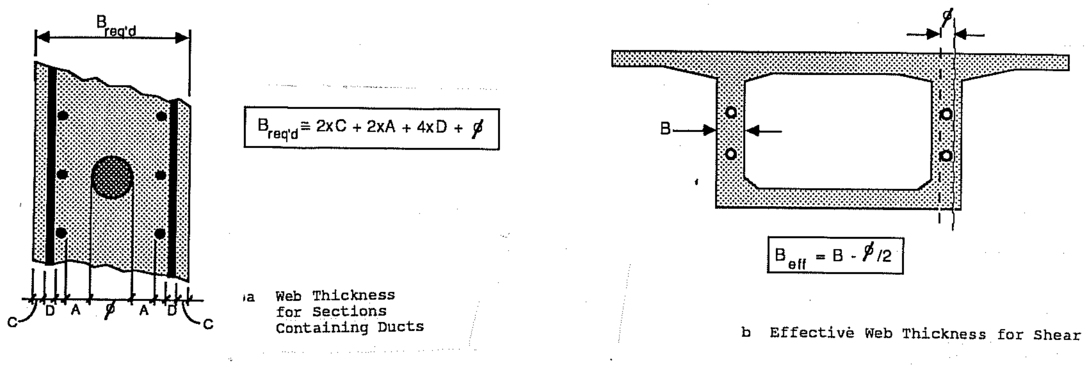



Fig. 2.23 Web Thickness

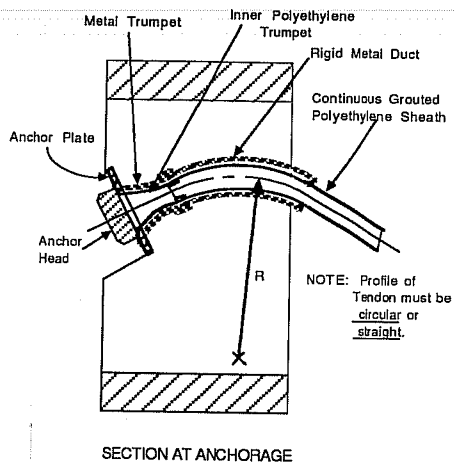



Fig. 2.24 "Double duct" for Replaceable Tendons

Accessibility of Tendons. The ease of access to external tendons facilitates installation of the tendons and sheathing. Open access to the full length of the tendons ameliorates grouting conditions. Preplaced grout ports become unnecessary, and danger of obstructed, collapsed, or otherwise damaged ducts disappears.

Replaceability of Tendons. The issue of replaceability of post-tensioning tendons has not been addressed much in this country. In France, however, there is considerable concern regarding this subject. The French federal transportation administration has taken the position that all external tendons should be replaceable (3). The French have patented a method of "double ducts" that allow removal and replacement of tendons, even if they are grouted (Fig. 2.24) (12). The accessibility of external tendons means that, with a few precautions and relatively little additional cost, tendons can be replaced without major effort or disruption of the use of the bridge (3).

Reduced Friction Losses. The use of external tendons reduces overall loss of prestress due to friction. Since the tendon is straight between attachment points and the duct is unrestrained, the effect of wobble is effectively nullified. Since the minimum acceptable radius of curvature for post-tensioning ducts is about the same for both internal and external tendons, curvature friction remains about the same.

Fatigue Advantages. Risk of fatigue failure of unbonded external tendons should be substantially less than that of bonded internal tendons because an unbonded tendon should experience substantially lower stress fluctuation under live load. However, there is a possible concern in that unbonded tendons are more susceptible to failures in the grips and there is a possibility of fretting at the deviators. This latter phenomenon is being studied in a carryon project at The University of Texas at Austin.

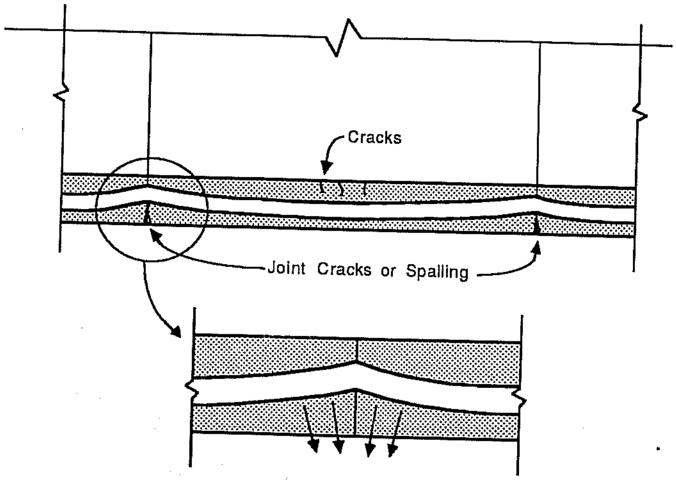



Fig. 2.25 Misalignment of Ducts at Segment Joints

Particular Advantages for Segmental Construction. In segmental construction with internal tendons, the tendon ducts are interrupted at each joint. The continuous sheathing (between attachment points) used for external tendons can possibly result in better corrosion protection.

A problem common to internally prestressed segmental construction which is eliminated with external tendons is the misalignment of ducts at joints (Fig. 2.25). Positioning short lengths of duct within the segment forms requires tight tolerances in order to ensure exact matching of the location and slope of two ducts coming together at a joint. In addition, the flexible sheathing commonly used for ducts may deform under the weight of the concrete. Slight misalignments can increase friction losses significantly. When severe, the condition may cause spalling of the concrete at stressing, requiring costly "surgical" realignment of the offending ducts.

One of the principal functions of epoxy in segmental construction is to seal around the ducts of two adjoining segments. Since the use of external tendons eliminates this need, claims have been made that epoxy can be omitted altogether, resulting in savings of both time and money.

An advantage afforded by the use of external post- tensioning in conjunction with segmental span-by-span erection is the great rapidity in construction possible. This achievement has been demonstrated by recent American projects. For the Long Key bridge, the contractor averaged 2.25 spans per week. As many a six spans were erected per week at the Seven Mile bridge site.

2.3.2 Disadvantages. Ironically, some of the same characteristics that create advantages for systems with external post-tensioning can also work to their disadvantage. Since tendons are external to the concrete cross section, eccentricity may be decreased and the free tendons can vibrate. External tendons are considered unbonded tendons, which behave differently from internal, bonded post-tensioning. Lack of bond can result in a reduction in efficiency and ductility, and a concentration of forces at attachment points.

Vibration of Unrestrained Tendons. Attachments of external tendons to the concrete section are often spaced quite far apart. The problem of vibration of long expanses of free cable has been experienced several times. Special precautions are required in these cases.

Reduced Eccentricity. Reduction in available eccentricity of prestress force is another negative aspect of external post-tensioning. For the typical closed box girder section, the tendon usually lies inside the inner surfaces of the flanges. The limited eccentricity decreases flexural efficiency.

Reduced Efficiency of Unbonded Tendons. Reduced efficiency at ultimate is characteristic of beams with unbonded tendons. Instead of the large numbers of small, well-distributed cracks which occur with internal bonded tendons, a few large cracks appear in unbonded beams [15]. Tendon strain is averaged over the length of the tendon, so the stress in the tendon at ultimate is only slightly above stressing levels. If the number of tendons required is based on ultimate strength considerations, such loss of potential tension force capacity could be a serious economic problem. However, most prestressed

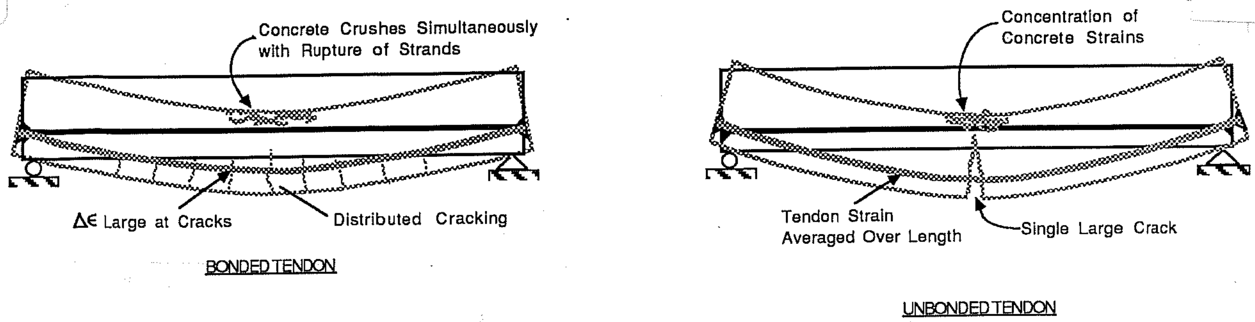



Fig. 2.26 Reduced Efficiency for Unbonded Tendons

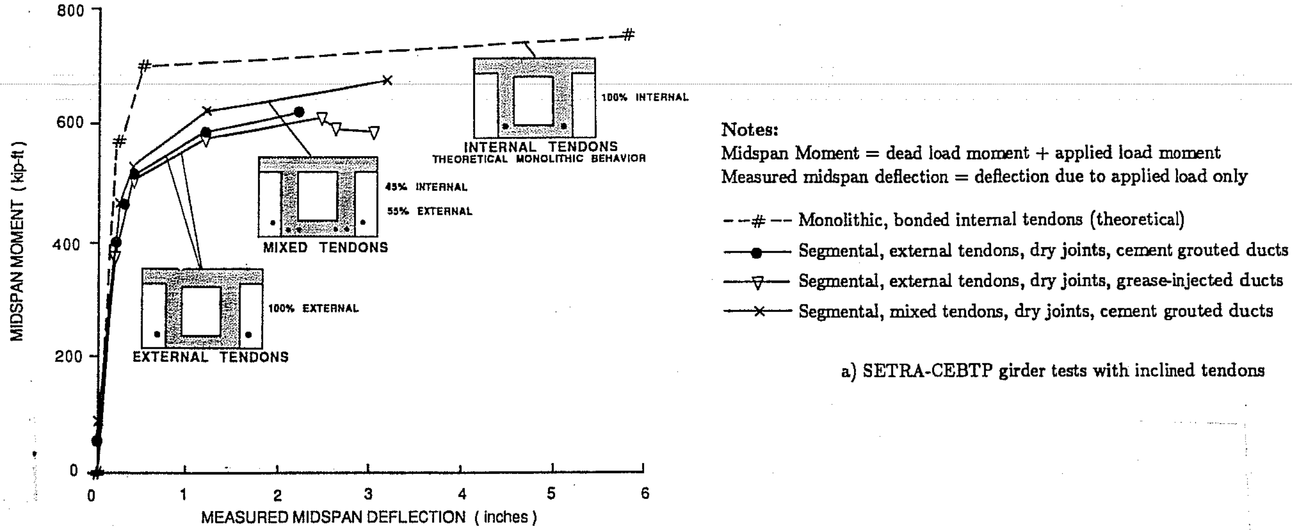



Fig. 2.27 Moment-deflection Behavior

bridges designed under AASHTO have the tendon sizes governed by allowable stress considerations under service load. The reduced ultimate stress of unbonded tendons is not a factor at service load levels and there is not an economic penalty. If ultimate conditions control, partial bonding of the tendon at deviators and a few intermediate locations could greatly improve tendon efficiency. Of some concern is the fact that with unbonded tendons, the concentration of rotation at a single crack often results in an early compressive type failure (Fig. 2.26). The absence of passive reinforcement across these large cracks (or the open joints in the case of segmental bridges) can worsen the condition.

In addition, there may be a reduction of shear transfer with external tendons due to the absence of dowel action in the web. This is probably a minor effect, though.

Reduced Ductility. A major concern in developing design criteria is the possible reduction in ductility for externally post-tensioned bridges. This is currently the subject of experimental and analytical study and will be reported in detail in the third report of this series. Figure 2.27a shows a theoretical moment-deflection curve for a simple beam model analyzed assuming monolithic construction with internal bonded tendons. It also shows test results for the same member with a mixture of internal bonded tendons and external bonded tendons, as well as test results with external bonded tendons alone. This comparison illustrates the loss of tendon strength development and possible reduction in ductility for the external tendon case. Experimental studies on segmentally cast I-beams with bonded and unbonded tendons have confirmed this trend (Fig. 2.27b) [17].

Concentrated Forces at Attachments. Another area under experimental investigation concerns the diffusion of concentrated forces at anchorages and deviations. These elements represent the only physical attachment of the external tendon to the concrete girder. Distress or failure of such elements could be catastrophic.

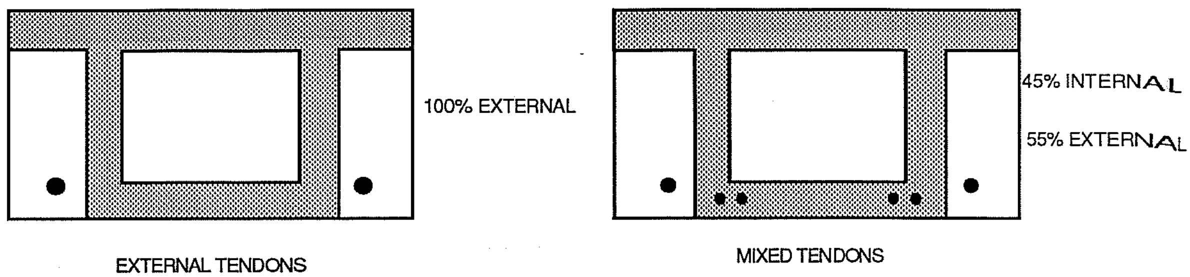



Fig. 2.28 Saint-Rémy Laboratory, Test Girders

## 2.4 Related Research

Experimental research with external post-tensioning may be regarded as in its infancy. The most significant projects known are briefly described below.

### 2.4.1 Girder Studies with External Tendons

Laboratory at Saint-Rémy, France. Under the combined auspices of the French organizations SETRA (Service d'Etude Technique des Routes et Autoroutes) and CEBTP (Centre Experimentale de Recherce et D'Etudes du Batiment et des Travaux Publics), engineers at the laboratory at Saint-Rémy conducted tests of four externally post-tensioned segmental box girders. The primary objective of the study was to examine the ultimate behavior of girders with external tendons in order to furnish or justify the assumed criteria used for ultimate strength calculations [16].

The test girders were match cast and erected with dry joints. The reduced-scale cross section used for all four tests was a simple, compact prismatic box girder (Fig. 2.28). The girders differed in tendon profile and type of tendon protection (grouted, hot wax injected). The fourth test girder also included some internal tendons. The behavior of the deviators, and their effect on overall behavior, was not a parameter in this study [16].

The simply supported girders were loaded symmetrically with point loads at outer quarter points. All girders experienced the same failure mode, independent of tendon profile or protection. First, the central joints opened and continued to open up to the level of the bottom surface of the top flange. At the same time, diagonal cracks propagated upward from the shear keys in the compressed region. The stress in the tendons did not rise significantly until the applied load was within approximately 10 to 15% of the failure load. The girders failed by crushing of the top flange in the presence of concentrated strains [16].

The experimental results agreed reasonably well with calculations performed according to methods presented previously by M. Virlogeux (16,3).

Construction Technology Laboratories, Illinois. At the request of Figg and Muller Engineers, Inc., Construction Technology Laboratories conducted tests of three simply supported segmental girders with differing post-tensioning systems. One girder had conventional bonded internal tendons, a second had unbonded external tendons, and a third included external tendons enclosed with a secondary cast, making them modified unbonded. The primary objectives were to verify the theoretical analyses and to compare the behavior of the three types of post-tensioning systems [17].

Figure 2.29 shows the I-shaped cross section used for each girder. Deviator behavior was not of interest in this test series, so deviators were conservatively designed. The match-cast segments were assembled with dry joints and the ducts containing all tendons were cement-grouted [17].

The girders were statically loaded in two cycles. The first cycle loading increased incrementally until the girder reached a midspan deflection of about 3 inches. The girders were subsequently unloaded and, in attempt to simulate an anchorage loss in the case of an earthquake, the wedges for some of the strands were burned and removed. The girders were then reloaded incrementally to failure [17].

The failure mode for the bonded tendon girder was flexural; concrete in the compression zone crushed simultaneously with the fracturing of strands in the tensile zone. The unbonded and modified

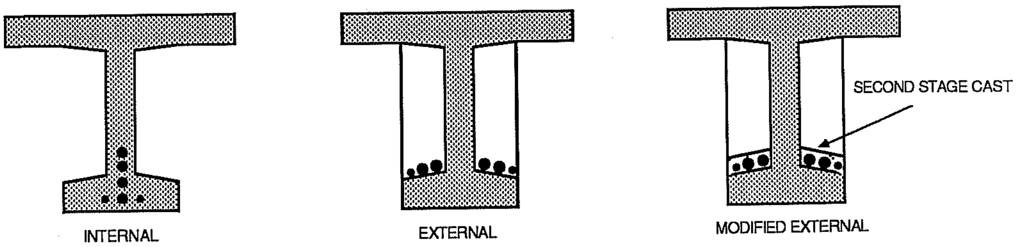



Fig. 2.29 PCA Test Girders

bonded tendon girders both experienced a shear- compression failure in the top flange. Joints opened and shear keys progressively broke, concentrating strain in the top flange [17].

Ferguson Structural Engineering Laboratory. Results of an experimental program involving external tendons presently underway at the University of Texas at Austin's Ferguson Laboratory will be reported in the third report of this series. Sponsored by the Texas Department of Highways and Public Transportation with financial support for some aspects also provided by the National Science Foundation, the program includes construction and testing of a quarter-scale three span continuous bridge model and development of a computer program to analyze externally post-tensioned systems. As with the other girder studies, behavior of the deviators is not a parameter in the bridge test. Preliminary evaluation of test data indicates the three span bridge performed very well at both service and factored load levels.

# CHAPTER 3. STATE-OF-THE-ART DEVIATORS

## 3.1 Introduction

One detail that is unique to external post-tensioning is the deviation device. The deviator is a critical detail since, other than at anchorages, it is the only positive attachment of the external tendon to the concrete section. Deviators provide the hold-down points necessary to achieve the draped tendon profiles required by flexural and shear design, and by geometry. Since the first externally post-tensioned structures were built, designers have devised a variety of alternatives for deviation devices.

## 3.2 Types of Deviators

The most common configurations of deviation devices for external tendons can be categorized by shape into three basic types: (1) the diaphragm (see Fig. 3.1) [2] the rib or stiffener (see Fig. 3.2) [3] the saddle or block (see Fig. 3.3). Each is comprised of a reinforced concrete projection cast monolithically with the bridge section which contains curved rigid ducts through which pass the external tendons. Block, rib or diaphragm deviators work well with the box girder cross section typically used for bridges with external tendons. Departures from the box-shaped section can require innovative conceptions for deviators. The Bubiyan bridge and the Saint-Agnant viaduct deviators, for example, include grooves cast strategically into the lower parts of the girder segments.

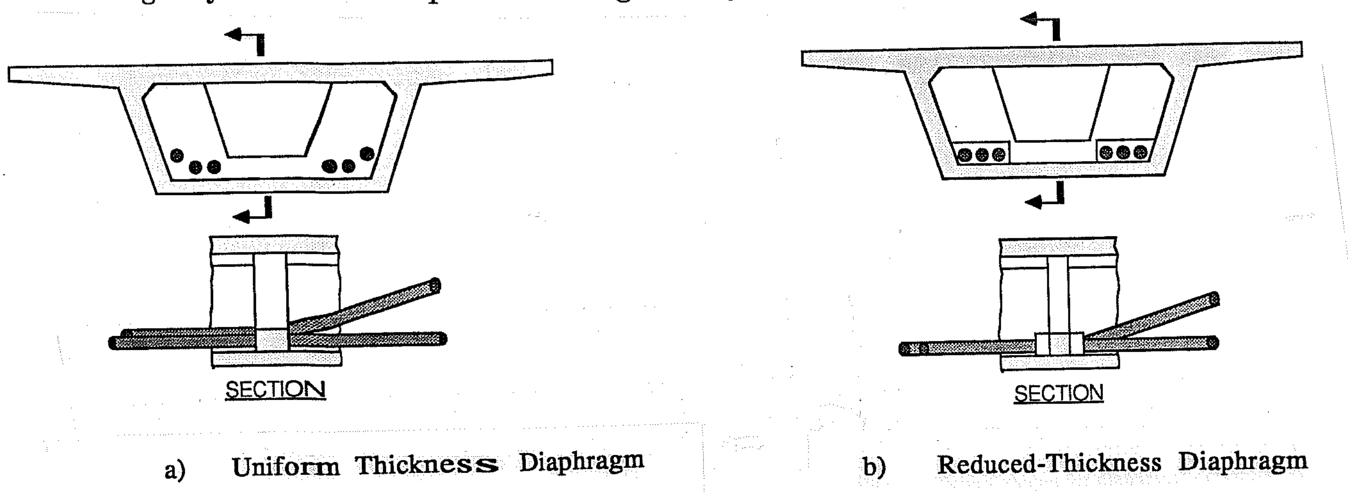



Fig. 3.1 Typical Shapes for Diaphragm Deviators

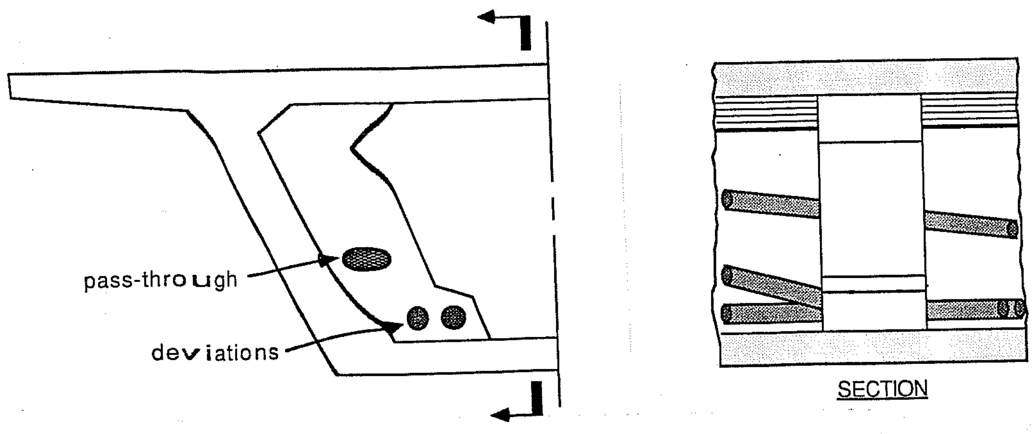



Fig. 3.2 Typical Shape for Rib Deviators

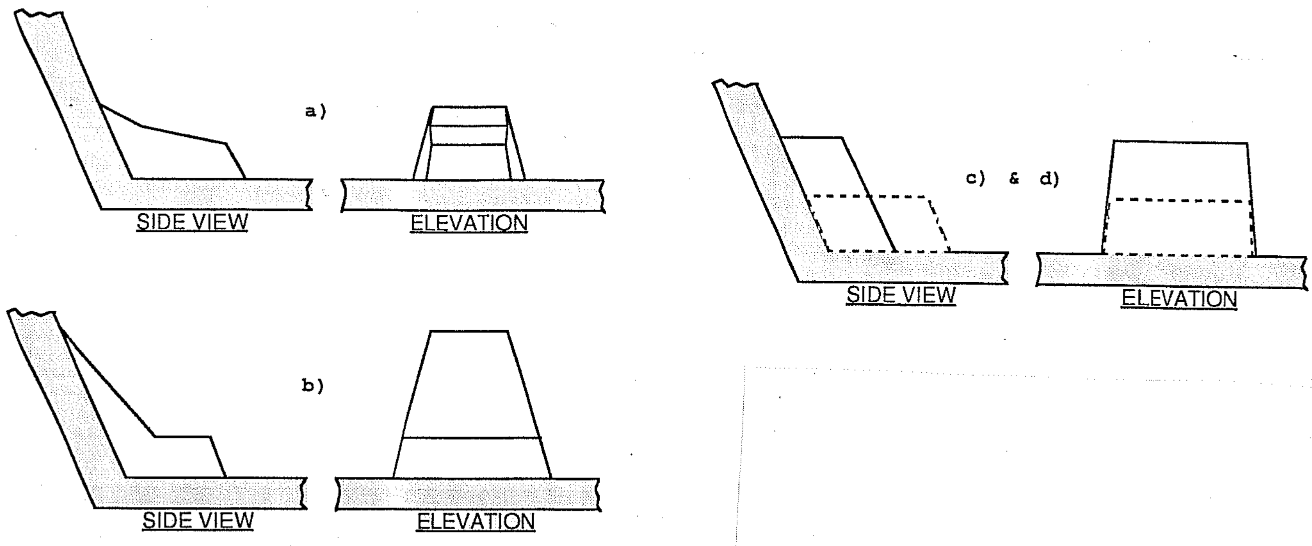



Fig. 3.3 Typical Shapes for Deviation Blocks

3.2.1 Diaphragm. Diaphragm type deviators are usually "U"-shaped, (Fig. 3.1a), or the "U" may be inverted. The opening allows passage through the void of the girder. The thickness of the diaphragm, typically ranging from 12 to 30 inches, is normally dictated by the length required to achieve a particular deviation. In some cases the upper part of the diaphragm is thinner; the lower section containing the deviated tendon extends out from either side, as in Fig. 3.1b.

3.2.2 Rib. The stiffener or rib shape is normally employed for retrofit measures where external tendons are added, although it has also been incorporated in new bridge designs. The ribs are typically somewhat shallow, extending perhaps a foot or so from the web of the box section. The depth is usually increased at the toe and the head of the rib (Fig. 3.2). A common difficulty with both the rib and the diaphragm type deviators is the need for locating pass-through openings to allow non-deviated tendons to pass through without deviation. This location process becomes very complex particularly on spans with horizontal curvature.

3.2.3 Block. Jean Muller is credited with the conception of the saddle or block type deviator (3). This relatively small block is located at the junction of the web and lower flange of the box section. Figure 3.3 illustrates some typical shapes for deviation blocks.

## 3.3 Ducts

Ducts located within deviators guide the external tendons and separate them from the concrete. Typically, the duct is formed by embedding a smooth, curved, rigid metal pipe (usually standard steel pipe) in the concrete section of the deviator.

3.3.1 Duct Details. Detailing the ducts requires care to minimize friction between the tendon and the duct and to avoid problems due to misalignment of the duct. Since the deviation of an external tendon usually involves both horizontal and vertical components, precise positioning of the curved pipe within this spatial geometry is often difficult.

Usually, the ends of the embedded pipe extend four to eight inches beyond the face of the deviator. This extension provides a place for attaching the protective sheathing that is generally used for external tendons. Figure 3.4 shows a typical detail for attaching polyethylene pipe to metal duct. The sheathing and duct are typically injected with cement grout (conventional American practice) or with grease or wax (European practice). If cement grout is used, the tendon is effectively locally bonded at the deviator. Grease or wax injection is used when the possibility of tendon replacement is to be retained.

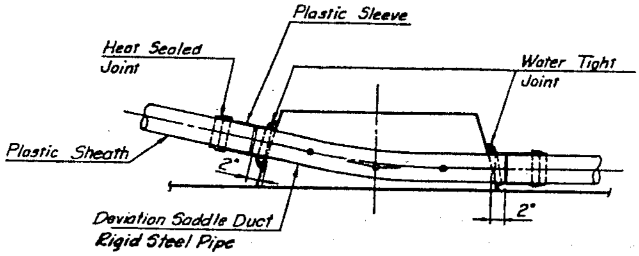



Fig. 3.4 Duct-sheathing Attachment

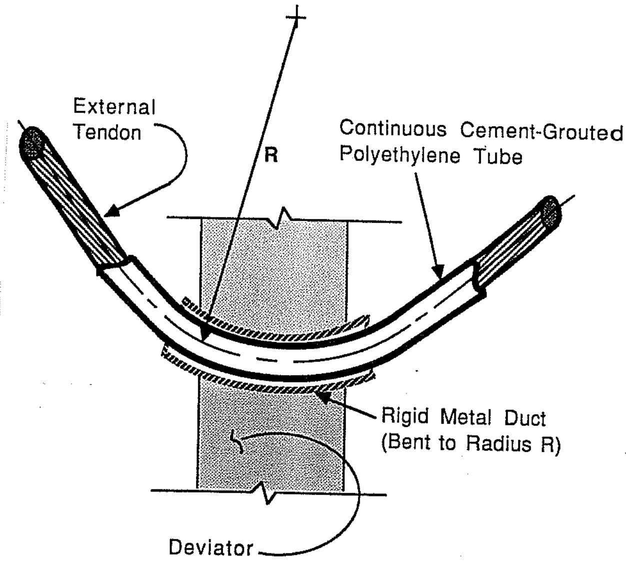



Fig. 3.5 "Double Duct" Configuration for Deviators

Extending the metal duct can also ameliorate slight misalignment problems since the duct can deform (see Sec. 3.5.2).

A "double duct" configuration has been used in several French bridges. In this case, the tendon sheathing remains continuous over the full length of the tendon (Fig. 3.5). This detail enables replacement of the tendon but prevents local bonding at the deviator.

Another type of metal duct which has been used in some externally post-tensioned bridges has a flared or "bugle" shape. This shape minimizes misalignment problems (refer to Sec. 3.5.2).

3.3.2 Duct Size. The minimum nominal duct area for a multiple-strand post-tensioning tendon is specified as two times the area of the tendon [21,22,23]. For external tendons, however, a desirable criteria of 2-1/2 times the area of the tendons has been recommended [24].

3.3.3 Radius of Curvature. Rigid metal ducts are cold-bent to a radius required to conform to the geometry of the external tendon profile. The radius is chosen to achieve the total deviation angle within the length of the deviator. Ordinarily, the duct is actually bent to a slightly smaller radius in order to avoid misalignment problems (refer to Sec. 3.5.2) If the radius of curvature is small, however, the friction loss due to curvature can be considerable. This must be taken into consideration during the conception of the tendon profile and the design of the deviator. Neither AASHTO nor PTI specify a minimum radius of curvature for post-tensioning ducts [22,23]. In standard American practice 15 feet is normally the smallest radius used for internal tendons. The 15 foot dimension corresponds to the minimum radius that traditional corrugated post-tensioning duct can be bent without kinking [25]. For external tendons, the following has been recommended as a desirable criteria [24]:

| | |
|---|---|
| Minimum radius at piers: | 20 ft for 19-strand tendons |
| | 15 ft for 12-strand tendons |
| | 10 ft for 7-strand tendons and smaller |
| Minimum radius at deviators: | 10 ft for all tendon sizes |

Pending results from tests to be conducted at the Saint-Rémy Laboratory, the French federal transportation administration specifies a minimum radius for external post-tensioning ducts of 3 meters (9.2 ft) for smaller tendons, and 4 meters (12.3 ft) for larger tendons (eg. 19-0.6 in. diameter strands or larger). The same requirements apply to conventional internal tendon ducts [3].

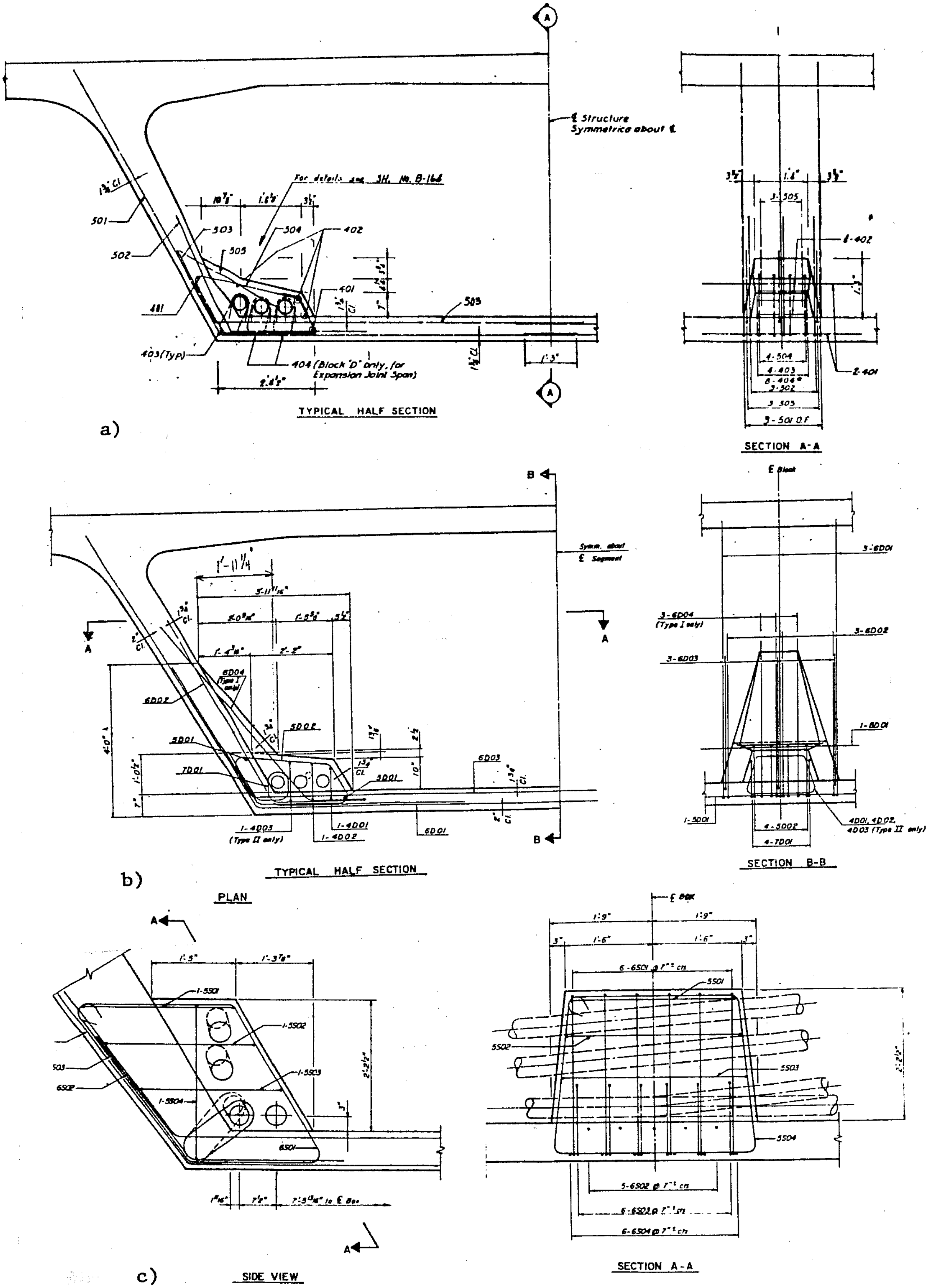



Fig. 3.6 Deviation Block Reinforcement

### 3.4 Reinforcing Schemes

To illustrate the types of typical reinforcement currently being used in deviators, several reinforcing diagrams from existing structures are shown.

3.4.1 Saddle or Block – Typical Reinforcement. Reinforcement for deviation blocks is usually mild reinforcing steel in the form of links and bent bars anchored back to the web and bottom flange of the box girder section. Figures 3.6a through c present several examples from existing bridges.

3.4.2 Diaphragm - Typical Reinforcement. Diaphragm type deviators typically include a mesh of horizontal and vertical reinforcing bars which tie into the flanges and webs of the box girder section. Additional bent bars provide confinement in the region of the deviation ducts. Transverse post-tensioning has also been used. Figure 3.7 shows some examples of diaphragm reinforcement.

3.4.3 Design Criteria and Procedures. The exact criteria that have been utilized for the design of deviation devices in existing bridges are not well known. That is to say, no open literature has been found where such criteria have been published. Regarding deviation saddles, personal conversations with engineers in the industry have revealed that deviators are designed to resist direct tension and shear friction using basic ACI Building Code procedures. The service load is taken as the deviated force component of the tendon at a stress level of 80 percent of ultimate, and the maximum service stress of the mild reinforcement is taken as 40 percent of yield stress.

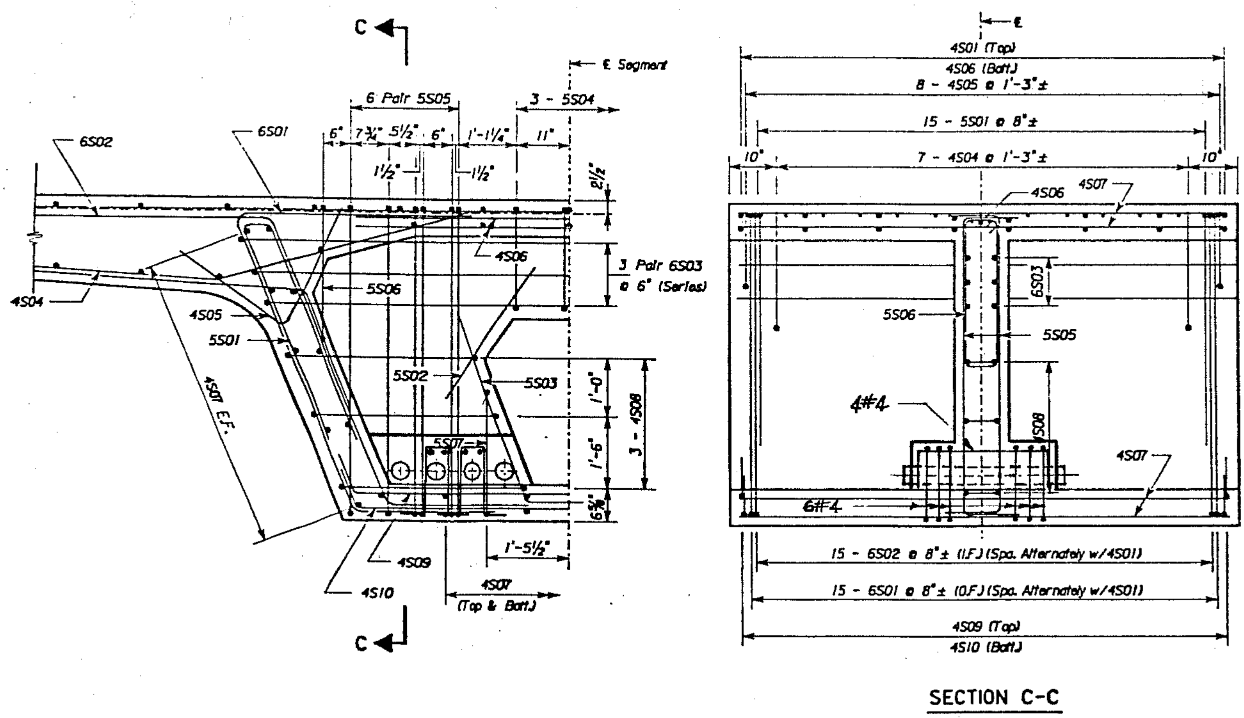



Fig. 3.7 Diaphragm Reinforcement, Example

A recently published document from the Florida Department of Transportation cites similar criteria [18]. The following is an excerpt from Chapter 4 (*Design Criteria for Segmental Bridges*) of that document, Sec. 10. *Deviation Saddles*:

"10.2 Design

Reinforcement shall be provided in the form of links and bent bars to take the full resultant pullout force from the deviated tendon(s) at a service stress of 0.4 fy. Additional reinforcement shall be provided to take any out of balance longitudinal forces by shear action according to the latest A.C.I. Standard 318, Article 11.7. Reinforcement shall also be provided to take any localized bending effects transmitted from the deviation saddles to the webs and/or flanges.

10.3 Detailing

All reinforcement shall have a full effective development length measured from the tendon axis or shall otherwise be fully mechanically anchored around longitudinal reinforcement located at the outside of the (box) section. Consideration shall be given to constructability and clearances between reinforcement for adequate concrete compaction. As a guide, not more than two reinforcing bars shall be bundled and the clear distance between reinforcement should be at least one-half inch (1/2 in.) greater than the maximum aggregate size and in no case less than 1-1/2 in."

## 3.5 Problem Areas Concerning Deviators

3.5.1 Deviator Type. The configuration of deviators has been a controversial issue (3). One design philosophy advocates the use of deviation blocks. The other more conservative school of thought prefers the full height diaphragm for deviating external tendons. The rib type deviator represents a compromise between the block and the diaphragm. Each of the solutions has definite advantages as well as disadvantages.

One point of contention between the design philosophies is the relative strength of each deviator type. Clearly, the strength of any design depends to a large extent on the detailing of the reinforcement. Even with adequate detailing, however, it is reasonable to expect a greater risk of catastrophic failure for a deviation block than for the rib or diaphragm deviators. Generally, in a diaphragm or rib, a compression strut forms above the deviated tendon. This compression strut contributes to the post-cracking strength of the deviator. (This is not always the case, however, as evidenced by the problems with the Pont-a- Mousson diaphragms; See Sec. 3.5.4). In a cracked deviation block, there is no compression strut. The deviator reinforcement must resist the pullout forces with little contribution from the concrete.

The block or saddle deviator has seen more prevalent use in U.S. bridges. Long Key, Seven Mile, Glenwood Canyon, Dauphin Island, and Sunshine Skyway bridges all include this type of deviation detail. Two french structures, the Banquiere and Vallon-des-Fleurs viaducts, included a slightly larger version of the typical American deviation block. With regard to construction, the saddle or block configuration presents considerable advantages. The formwork can be greatly simplified, since only a small blockout in the interior form is required. When casting a segment without a deviator, the blockout is simply sheathed over. In addition, the small block adds an almost insignificant amount to the self-weight of the superstructure [3].

On the other hand, the use of deviation blocks imposes limitations on the tendon profile. The points of deviation are, in effect, predetermined by the geometry of the structure. The external tendons cannot be aligned in a single vertical plane; the profile must therefore include otherwise unnecessary horizontal displacements [3].

Full-height diaphragms allow the designer complete freedom when laying out the tendon profile [3]. Ordinarily, efforts are made to reduce the number of diaphragms required since the diaphragms add substantial weight to the structure. Reducing the number of deviations increases the magnitude of the deviation angles, creating larger deviation forces on the diaphragm. The resulting diaphragm required usually has a "U" shape. The bottom part of the "U" is designed to stiffen the lower flange to carry localized bending from the deviation forces.

From a construction point of view, the full-height diaphragm is cumbersome. Obviously, the formwork becomes more complicated. For segmental construction, segments with diaphragms must often be made shorter, due to weight restrictions for segment handling. This results in further modification of forms. Alternatively, the diaphragms may be placed in a second stage cast, in which case dowels must be provided and an opening left in the upper flange for subsequent concreting [3]. Moreover, the diaphragms can add considerably to the dead load, easily offsetting the savings offered by web reduction. Both rib and diaphragm deviators must be provided with pass-through ports for passage at non-deviated tendons. Their location becomes time consuming, especially with horizontal curvature.

3.5.2 Geometry Errors of Ducts Embedded in Deviator. The difficulty of properly aligning the ducts within the deviator has caused recurrent problems. There are three principal sources of error: bending of the duct to an incorrect radius of curvature, improper positioning of the ends of the duct at the faces of the deviator, and pivoting of the duct around the axis joining the points of exit. Care must be taken to ensure that the duct is adequately secured so that it will not shift during concreting [12]. The misaligned duct creates angular bends or kinks in the tendon at the point of exit from the concrete. The kink, in turn, creates concentrated forces which are applied to the unreinforced cover at the face of the deviator. Damage, in the form of cracking or spalling, is usually noticed during stressing procedures when deviation forces are highest. Repair measures may become necessary. Undetected damage may lead to corrosion of the mild reinforcement in the deviator, possibly resulting in further distress or disfunction of the deviator.

Not only the concrete deviator, but the tendon as well, is subject to damage. Localized bending of strands in the tendon, and rubbing under the action of loading, can result in wear and fatigue of the tendon.

Several preventive measures have been devised to alleviate problems due to geometry errors of ducts in deviators. Extending the duct a few inches beyond the face of the concrete allows the metal duct to deform somewhat, softening the kink. In addition, the duct is made larger than necessary and bent to a slightly smaller radius than that required for the exact geometry of the tendon profile. Under these conditions, the tendon is less likely to bear against the face of the concrete (Fig. 3.8a). Another method inserts some flexible material around the end of the duct at the face of the concrete. This isolates the duct from the unreinforced cover and transfers the concentrated force to the reinforced concrete (Fig. 3.8b) [12]. A different idea used in some French bridges employs a bugle shaped rigid metal duct (Fig. 3.9). The radius of the flare is smaller than that required by the tendon geometry for all deviators in the bridge. In this manner, the same duct can be used for all deviators, and location of the ducts is simplified.

3.5.3 Force Diffusion at Deviator. There are two areas of uncertainty concerning force diffusion at the deviators: How do the concentrated force components from the deviated tendon distribute within the deviator? This question will be answered in detail in the second report of this series. How does the deviator transfer forces to the box girder section? The second question is of less interest; there have been no reports of damage to the concrete in the girder section. Most of the problems that have occurred have involved cracking or spalling of the deviator itself.

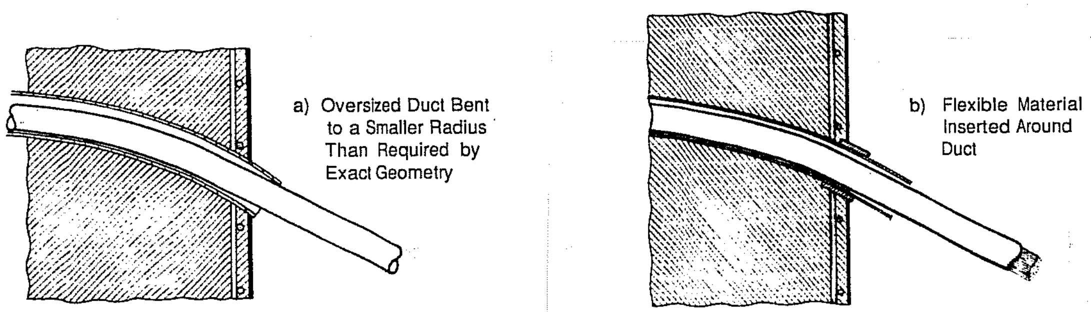



Fig. 3.8 Preventive Measures for Duct Geometry Problems

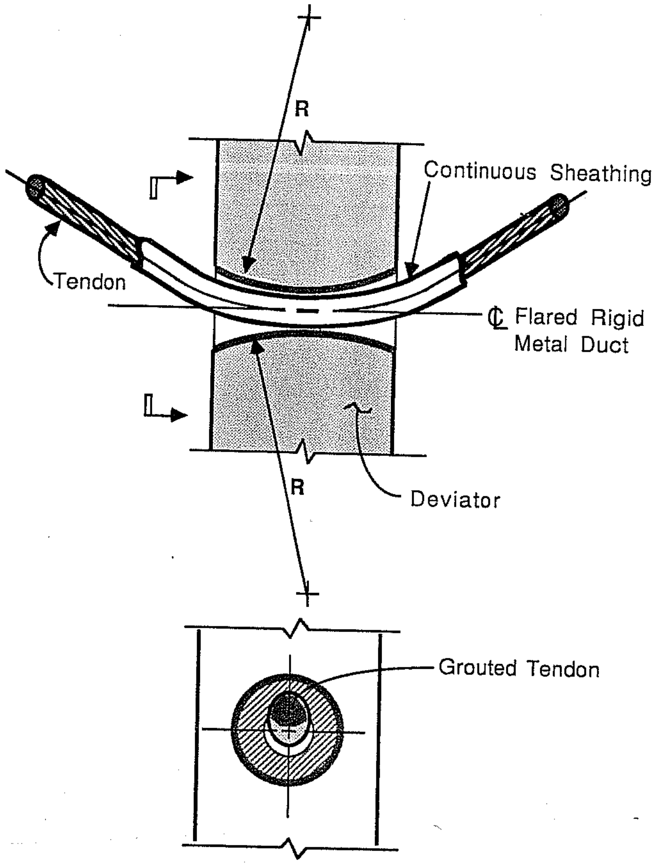



Fig. 3.9 "Bugle"-shaped Duct for Deviators

3.5.4 Damage to Deviators in Existing Structures. Several incidences of damage to deviators in existing structures have occurred. Most of the problems that have been published have originated from errors in geometry of the ducts within the deviators. Other problems, more structural in nature, have also been reported. Unfortunately, when problems occur, parties involved with the distressed structure are naturally reluctant to publicly disclose information about the problems, so as not to call attention to them. The authors have learned of the following accounts through published reports, personal conversations with eyewitnesses, and from first-hand observations.

The Sermanez viaduct (See Sec. 2.2.5.5) is one of the few externally post-tensioned bridges built by the cantilever method. The complicated tendon profile required a large number of ducts in each of the closely spaced deviation ribs. Imperfections in the alignment of several ducts (the ends of which were flush with the face of the concrete) provoked minor spalling around the ducts. The galvanized external tendons were carefully checked, fortunately revealing no damage to the galvanized coating at the kinks [11].

A similar incident involved a segmental box girder bridge with deviation saddles. During stressing of the first few spans, the concrete around the ducts in several deviators spalled and cracked. The spalled concrete was chipped away and patched. For the remaining segments which were waiting for erection, the contractor rechecked the alignment of each duct using surveying equipment. Unacceptable deviators were repaired by chipping away the concrete around the end of the duct, bending the duct to the proper position, and then patching.

The Pont-a-Mousson bridge (See Sec. 2.2.5.6) experienced more serious cracking in the diaphragm type deviators. The external tendon profile included some sharp horizontal deviations which were directed inward toward the void of the box section (Fig. 3.10a). Interferences with anchorages for

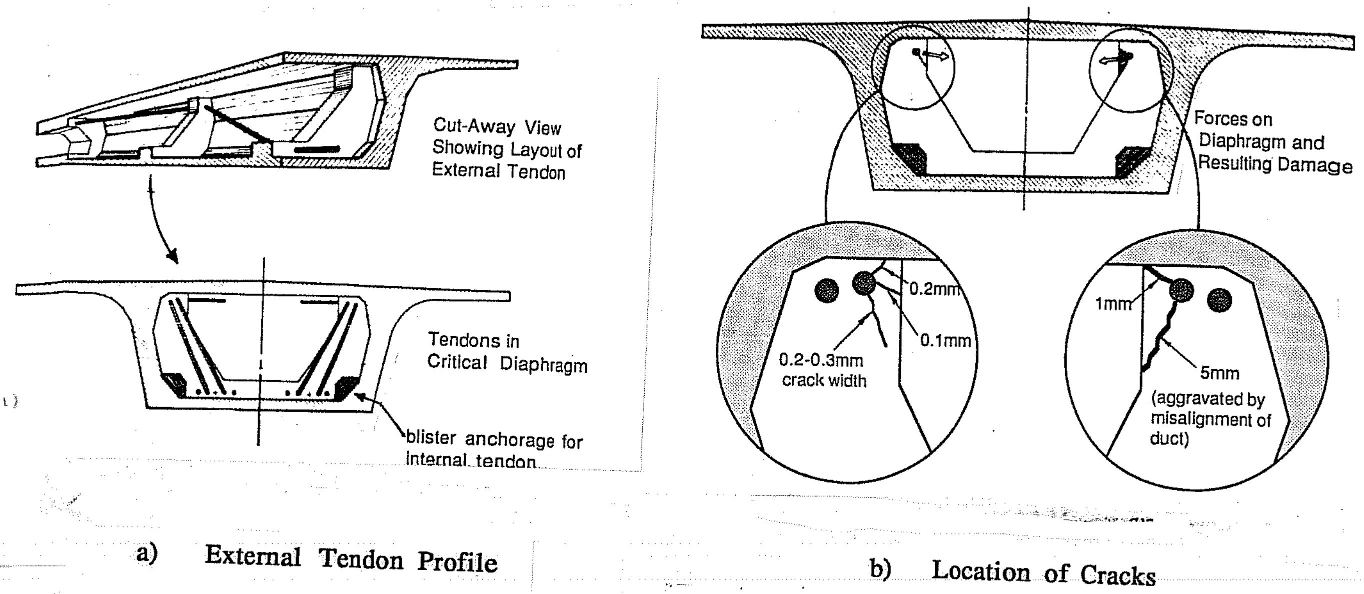



Fig. 3.10 Cracking of Pont-à-Mousson Diaphragms

internal continuity tendons necessitated these horizontal deviations. Large cracks and spalling appeared during stressing operations (see Fig. 3.10b), requiring extensive repairs [12].

In another case, deviation diaphragms were highly congested with reinforcing steel and ducts of multiple tendons which were deviated or passing through the diaphragm. A combination of the shape of the diaphragm and congestion in the reinforcing cage resulted in poor consolidation in the area of the deviations. For a large number of segments, the deficient concrete had to be chipped away and repaired with an epoxy-based cement patch.

In other structures, there have been reports of structural cracking in the deviation saddles. One such report involved a large number of relatively small cracks in several deviation blocks. Most of these cracks appeared on the top faces of the blocks and ran perpendicular to the tendons.

3.5.5 Lack of Consistent Design Philosophy. Because little is known about the behavior of deviators under overload, universally accepted models for designing deviators do not exist. When using design procedures for proportioning direct tension and shear friction reinforcement, the load path of the deviation forces is left up to the designer. Where complete reinforcement details and tendon layouts have been available, the senior author has attempted to back-calculate from the given loads and areas of steel, in order to determine the rationale used for designing a particular deviator. In most cases, a rational approach for distributing forces to the appropriate reinforcement was not apparent. One objective of this research program is to establish representative models for behavior of deviation blocks. Such models are reported in the second report of this series.

## 3.6 Deviator Research Studies

The only other known experimental program involving deviators is currently ongoing at the laboratory at Saint-Rémy-Les-Chevreuse in France. Sponsored by SETRA-CEBTP, this study concerns not the behavior of the deviator itself, but the behavior of the deviated tendons. Parameters in the study include characteristics of the deviation (e.g. radius of curvature, deviation angle, duct type and size, etc.), nature of the tendon (e.g. number of strands, degree of entanglement of the strands), tendon protection (duct material and injection product), and loading. The Saint-Rémy laboratory does have future plans to study the deviator itself, using the same test setup used in the tendon tests [19].

# CHAPTER 4. TEST FACILITY DESIGN AND CONSTRUCTION

## 4.1 Test Objectives

To begin a program of deviator research, a representative prototype was chosen from existing U.S. structures. Based on that prototype a versatile testing rig was developed and several scale models were constructed for testing. The test facility was designed to provide flexibility in testing many deviated tendon patterns with varied horizontal and vertical deviation angles. In design of this facility and the deviator test program the basic objectives were to:

1. Investigate the behavior of the deviator through a full range of loading. Is the mode of failure of the deviator relatively brittle or ductile? Is the failure zone confined to the deviator itself, or are there significant local effects in the web and flange surrounding the deviator? The test facility had to have the capacity to take representative deviators to failure.
2. Evaluate the design of the prototype deviator with respect to details and overall performance. Is adequate anchorage provided for tying the deviator to the concrete section? Do splices and laps develop? How much reserve capacity over service load level is provided by this typical design? Are similar current designs safe? The test facility had to have the ability to model a reasonable piece of the surrounding segment as well as the deviator.
3. Determine a generalized model of behavior for this particular case. How does each type of reinforcement contribute to the overall load carrying system? What are important considerations (e.g. shear friction, local bending, confinement) for design and detailing of this type of deviator? The test facility had to be equipped with sophisticated instrumentation to determine the contribution of any piece of reinforcing steel.
4. Suggest a safe, rational design approach. If a reasonable model can be established from the limited test data, a more rational approach for design than currently exists can be suggested. Based on results of the first two tests, a limited approach was suggested by Carter [27]. After further study, the set of design procedures was refined [29] and is presented in the second report in this series.
5. Evaluate the test setup. Does it function well? Can it be modified to facilitate testing, or to improve test conditions? Initial testing showed that the facility worked well and a few minor improvements were made to simplify operation.

## 4.2 Test Specimens

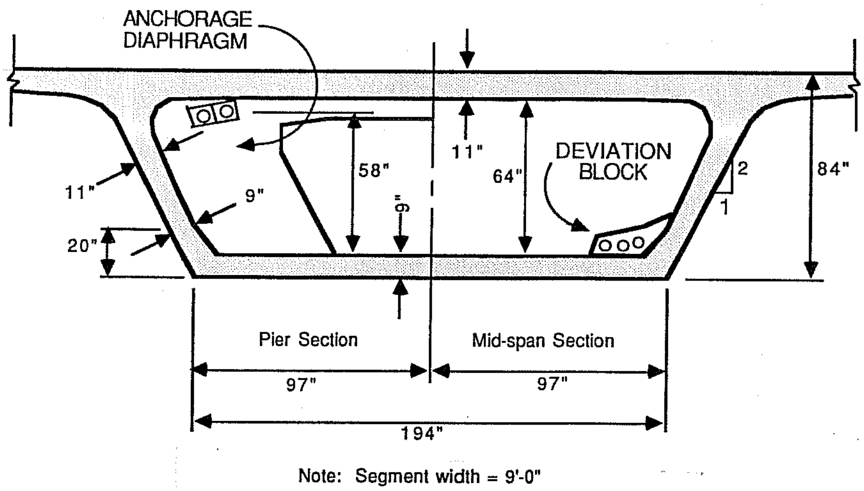
4.2.1 Basis for Models. A thorough survey of existing externally post-tensioned bridge structures revealed that the deviation block is the most prevalent type of deviation detail. Common sense dictates that the block type deviators are generally weaker than diaphragms or rib deviators. But the deviation block also has advantages over other alternatives (See Sec. 2.5.1), and could have even more appeal if testing confirms the safety of this type of detail. Several of the structures that were examined had very similar deviator reinforcing details. One structure from this set was chosen as a prototype for the scale model on which to base design of the testing facility.

In order to represent typical stiffness of supporting elements and to determine the adequacy of reinforcement anchorage details, it was decided that the typical model should comprise a complete typical segment of a single-cell box girder bridge, excluding the cantilever wing portions. Two deviators could be located in the model segment and be tested one at a time. Deviators could have identical or varied reinforcement. Blocks could deviate one, two, or three tendons. In the initial specimen, one block deviated two tendons, while the other contained three. This was done to see how a change in the relative magnitude and direction of the total force component on the deviator would affect its behavior.

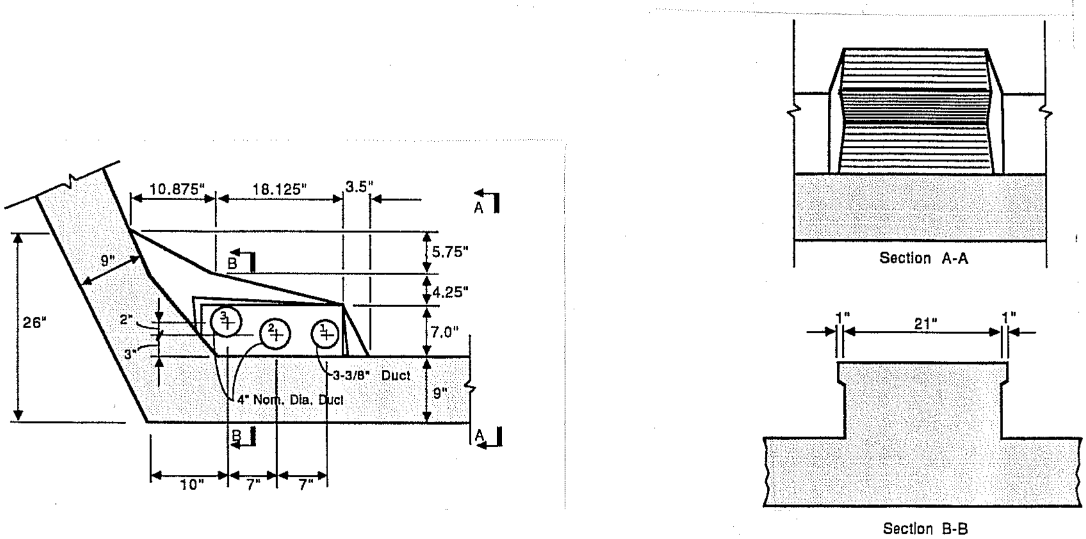
The basic box section of the specimen had several advantages. It eliminated torsional problems and was easy to hold in place. The single cell box is a typical section and makes an adaptable basis for future testing. Reinforced according to AASHTO minimum requirements, the basic box can remain the same regardless of the type of deviator to be tested. Only the deviator reinforcement and the inside forms need to be replaced. Complete discussion and dimensions for the first series of models is presented herein so that the development of the testing facility and procedures can be illustrated.

4.2.2 Scale Factor. Several considerations influenced the choice of scale factor. On one hand, the specimen needed to be large enough so that scaling would not appreciably alter local effects. In addition, too small a specimen would complicate construction, since tolerances for placement of reinforcing bars and ducts would be critical. On the other hand, too large a specimen would require unrealistically large test appurtenances and equipment in order to develop the tendon reactions. Safety was also a factor in limiting the size to a reasonable level. For the initial tests, a linear scale factor of 1/3 was chosen. For subsequent tests the scale factor was reduced to 1/4 to allow more margin for force development.

4.2.3 Dimensions of Prototype and Initial Model. The prototype cross section is shown in Fig. 4.1. For modelling, the shape of the box and of the deviation block were simplified somewhat. The total width of the prototype box as well as the length of the segment were reduced by 2/3 before applying the 1/3 scale factor to facilitate anchorage and handling of the specimen. Fig. 4.2 shows the model cross section and dimensions.




a) Cross Section




b) Deviator

Fig. 4.1 Protype Cross Section

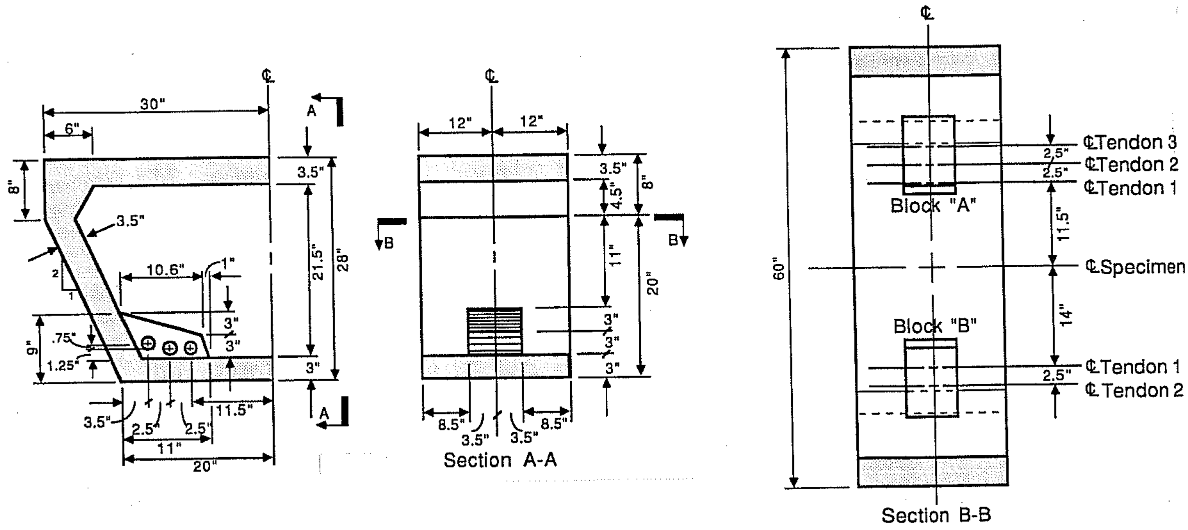
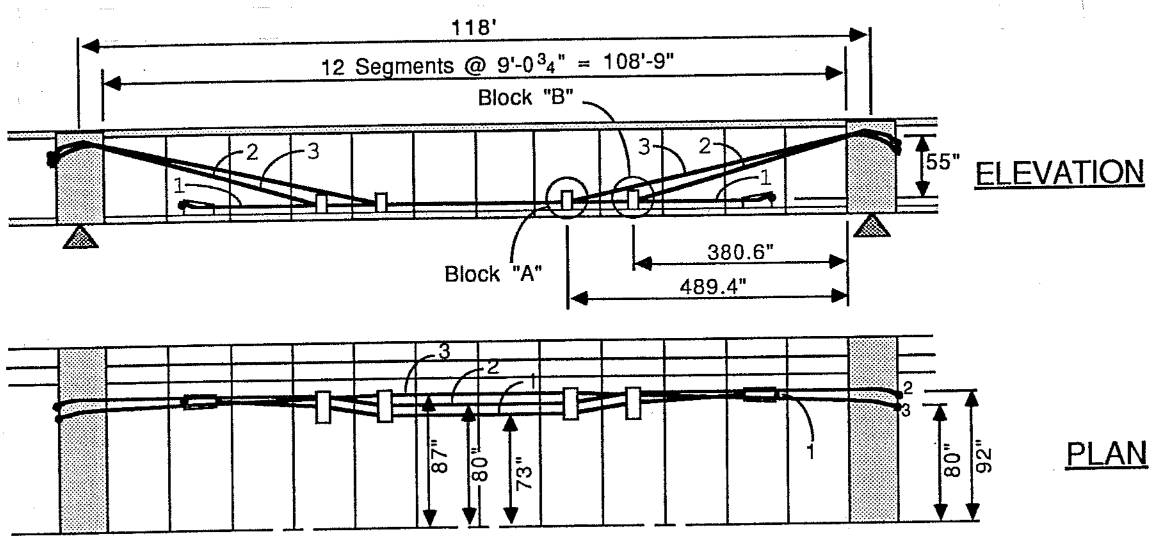



Fig. 4.2 Specimen Dimensions

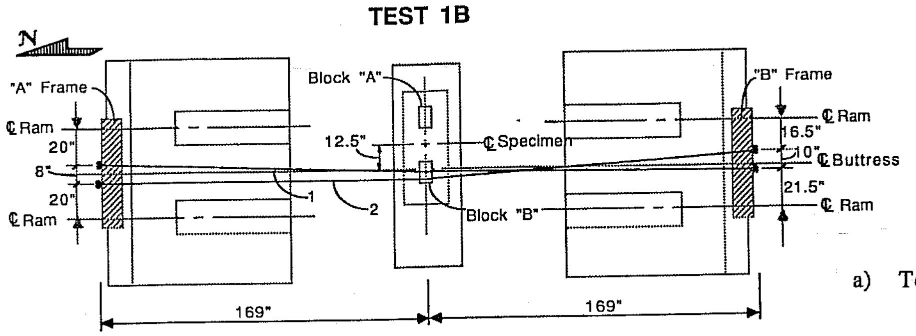



DEVIATION ANGLES:

| | Tendon | Horiz. Angle | Vert. Angle |
|---|---|---|---|
| Block "A" | 3 | 0.82° ↓ | 6.41° |
| | 2 | 3.68° ↑ | 0 |
| | 1 | 3.68° ↑ | 0 |
| Block "B" | 2 | 2.93° ↓ | 8.22° |
| | 1 | 1.84° ↓ | 0 |

Tendon 1 : 12-0.5" dia. strands
2 : 19-0.5" dia. strands
3 : 19-0.5" dia. strands

Fig. 4.3 Prototype Tendon Layout

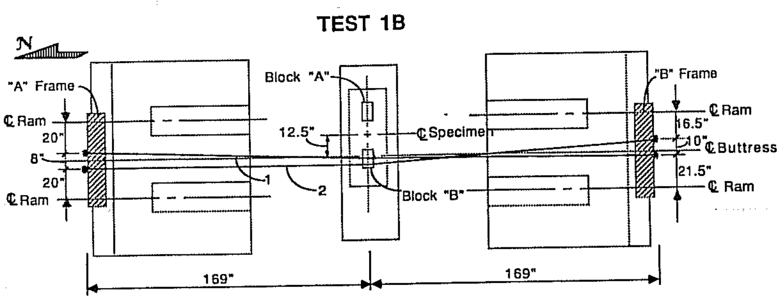



a) Test 1A tendon layout, plan

Tendon 1 : 9 - 3/8 in. dia. strands
2 : 12 - 3/8 in. dia. strands

DEVIATION ANGLES:

| | Tendon | Horiz. Angle | Vert. Angle |
|---|---|---|---|
| Block "B" | 2 | 3.87° | 8.1° |
| | 1 | 1.51° | 0 |

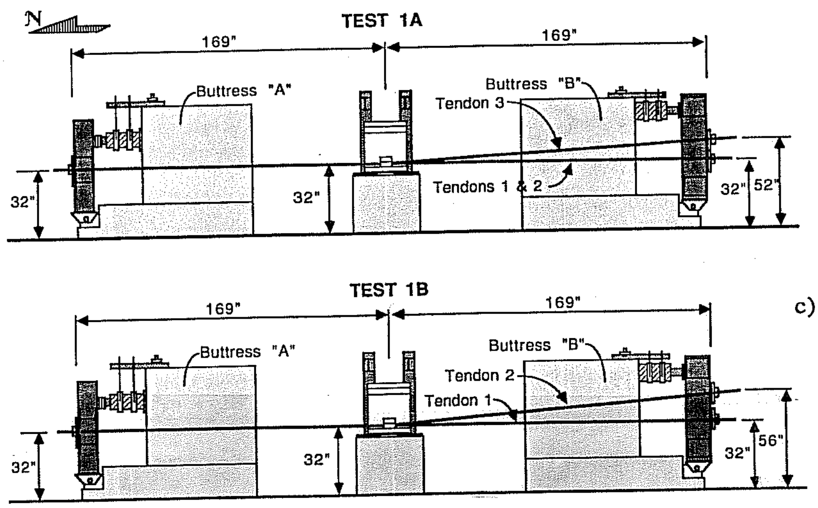



b) Test 1B tendon layout, plan

Tendon 1 : 9 - 3/8 in. dia. strands
2 : 12 - 3/8 in. dia. strands

DEVIATION ANGLES:

| | Tendon | Horiz. Angle | Vert. Angle |
|---|---|---|---|
| Block "B" | 2 | 3.87° | 8.1° |
| | 1 | 1.51° | 0 |




c) Tendon layouts, elevation

Fig. 4.4 Tendon Layout

4.2.4 Tendon Layouts. The prototype tendon layout, and resulting deviation angles, are presented in Fig. 4.3. The modelled deviations are denoted as "Block A" and "Block B" in the figure. Figures 4.4a through 4.4c show the tendon layouts used for each test. Geometry constraints of the test setup prohibited exact reproductions of the prototype deviation angles for each tendon. However, the layouts were manipulated in such a way that the total relative horizontal and vertical components on the deviator are approximately the same.

4.2.5 Choice of Materials

4.2.5.1 Concrete. A nominal concrete strength of 6000 psi is representative of strengths used for segmental post-tensioned construction. 3/8 in. pea gravel was used for coarse aggregate to preclude the need for micro-concrete. Retarder was used to improve workability which was an important criteria since good consolidation in and around the deviation block was very important. A local ready-mix plant supplied the concrete.

4.2.5.2 Mild Reinforcing Steel. The smallest conventional reinforcing bars (No. 3's) were too large for use in scale models. Undeformed wire was unacceptable, as it was felt that bond characteristics might play an important role in the behavior of the deviator. Fortunately, deformed microbars (No. 1's, 1.5's and 2's) were located. When tested, the microbars demonstrated a relatively brittle behavior so the bars were annealed to produce a stress-strain behavior very similar to conventional GR 40 reinforcing steel. The range of available sizes allowed a close representation of the prototype deviator reinforcement; in most cases the same number of bars were used. The microbars also had the advantage of being easy to bend, a desirable quality since much of the reinforcing in the deviator had complex shapes.

4.2.5.3 Ducts. A smooth walled duct with a representative scaled wall thickness was desired. Thin-walled galvanized metal conduit was chosen for this purpose.

4.2.5.4 Tendons. Seven-wire strand with a nominal diameter of 3/8 inches was a convenient choice for the multi-strand tendons, since this size was compatible with standard anchorage hardware and stressing equipment. The low relaxation strand had a nominal ultimate strength of 270 ksi.

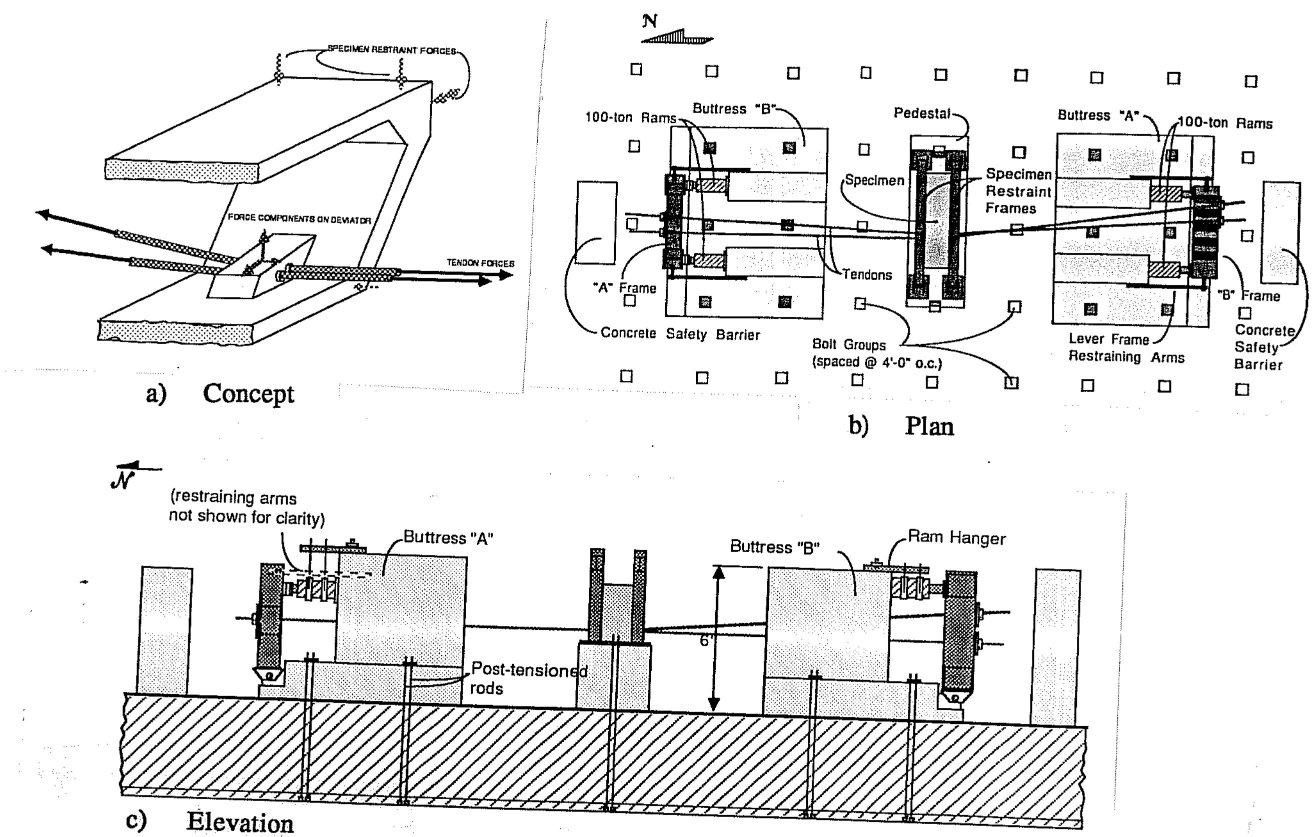
### 4.3 Test Setup

The primary concerns governing the design of the test setup were safety and adaptability. In any application, the concentrated energy stored in highly stressed post-tensioning tendons presents a potentially dangerous situation which must be carefully and conscientiously controlled. The danger is magnified tenfold with external tendons, since, after stressing, the tendon is not contained inside a beam or slab of concrete, but is exposed. Several precautions were taken to ensure the safety of the testing crew and general laboratory personnel.

Since it was difficult to foresee the direction and scope that the deviator research program would take, adaptability was important. Adaptability of the setup meant incorporating in the test structures and equipment the means to test a variety of specimens including flexibility in specimen size, the number of tendons, the layout of the tendons and the loading method.

4.3.1 General Layout. Figure 4.5a illustrates the basic testing concept while Figs. 4.5b and c show the structures and equipment that make up the testing assembly. Two large concrete reaction buttresses are anchored to the structural laboratory floor with high strength post-tensioned threaded rods. The normal clamping force from the tensioned rods provides shear friction resistance capable of developing a total horizontal reaction of up to 460 kips from the post-tensioning tendons.

Heavy steel lever frames are pin-connected to the end of each buttress. Restraining arms at the sides of the frames fasten to the buttresses to hold the frames in position between tests. The external tendons anchor at the back side of each frame. Anchor plates for the post-tensioning tendons bolt to the frame members. Located at the north end of the setup, frame "A" is designed to accommodate anchorages in a number of positions along a horizontal line (Fig. 4.6). Stressing operations take place at this end. The tendons run from the "A" frame, pass through the specimen deviator, and are deviated to the "B" frame, located at the south end of the setup. The "B" frame is designed as a sort of "grillage" of deep vertical members (Fig. 4.7). Anchor plates can be bolted to the vertical members in a number of positions in order to achieve the desired deviation angle at the deviator.




a) Concept

b) Plan

c) Elevation

Fig. 4.5 Test Setup

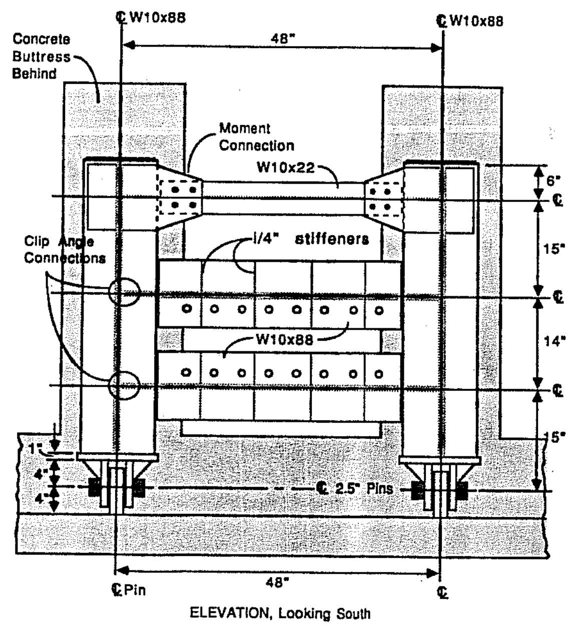



Fig. 4.6 Lever Frame "A"

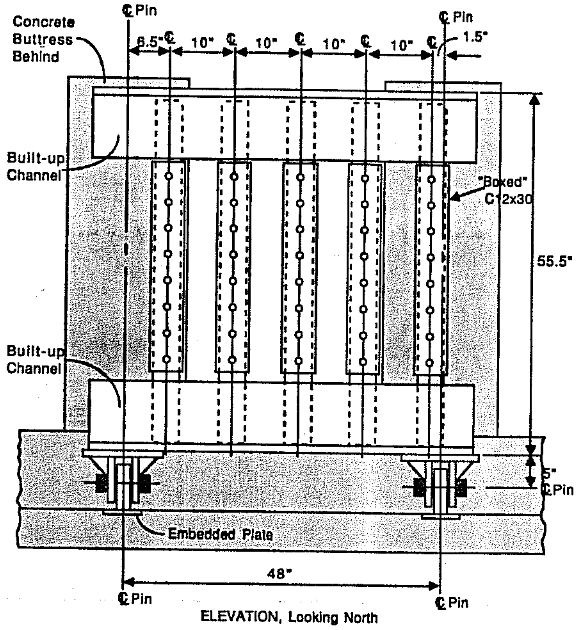



Fig. 4.7 Lever Frame "B"

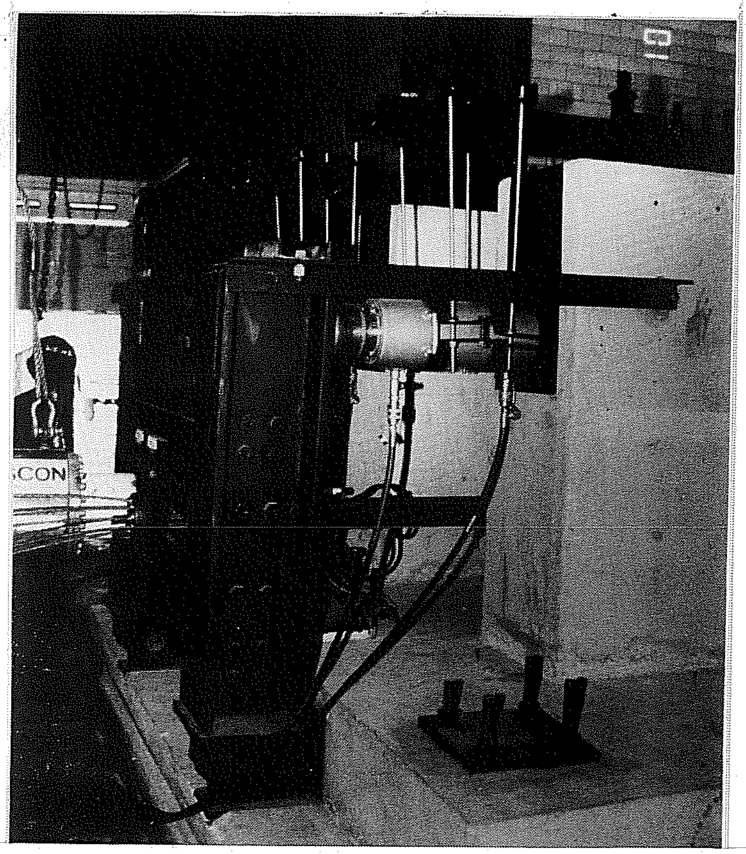


Fig. 4.8 100-Ton Loading Rams With Supports

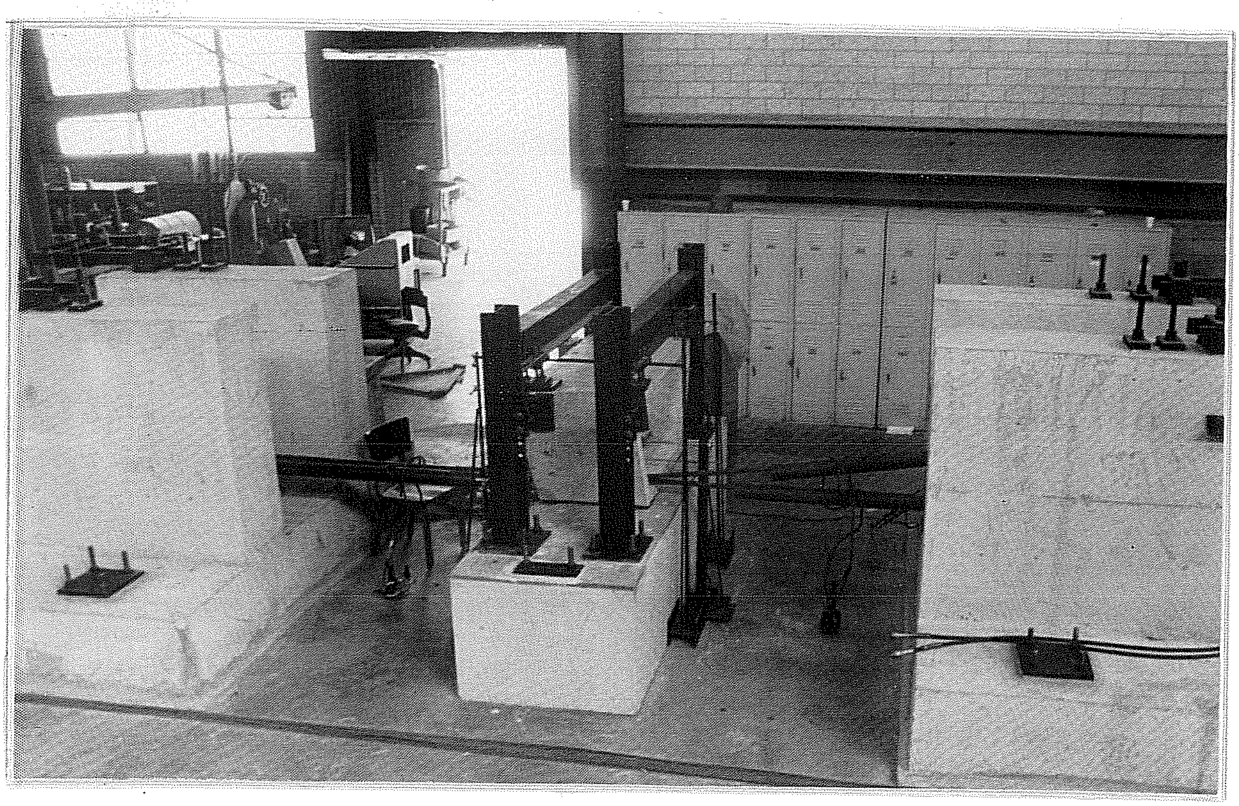


Fig. 4.9 Pedestal

A pair of 100-ton capacity hydraulic rams supply the upper reaction between the lever frame and the buttress. The rams are suspended by adjustable rods from small cantilever frames fixed to the tops of the buttress uprights. Spherical heads attached to the rams ensure good bearing between the ram pistons and the steel frame members (Fig. 4.8).

Between the two buttresses, the specimen is anchored to a long concrete pedestal (Fig. 4.9). Like the buttresses, the pedestal is also prestressed to the structural floor. The pedestal serves two purposes. First, it elevates the specimen to the appropriate height. Secondly, a set of open steel moment frames bolted to the pedestal provide vertical and horizontal (i.e. east-west) restraint for the specimen. Additional devices attached to the pedestal provide restraint in the longitudinal (i.e. north-south) direction.

Additional equipment includes pumps for the hydraulic rams, the data acquisition computer system, surveying instruments for reading dial gages and remote monitoring of the specimen, and a video camera for recording the tests. This equipment is located on the east side of the setup. Large concrete barriers were installed at the extremities of the setup as a safety precaution.

4.3.2 Safety Features. As previously mentioned, safety of the laboratory personnel was of utmost importance during this test program. The main concern was to provide a system of containment for the stressed tendons in the event that a wire or strand should break or an anchorage should fail. Large concrete barriers are situated directly behind the anchorage zones. Each external tendon is encased in a 3-1/2 in. diameter standard pipe. Several steel cables, looped around the pipes and anchored to the structural floor or to the buttresses, provided restraints for the tendons.

The entire test area was roped off to general lab traffic during the tests. Surveying equipment provides a remote optical system for monitoring cracking in the deviator and for reading dial gages located around the specimen.

4.3.3 Specimen Restraints. A system of restraints to anchor the specimen within the test setup was designed to limit as much as possible the rigid body motion of the box section, while keeping the deviation zone unrestrained. The restraint system is also fully adjustable to accommodate changes in the positioning of the specimens from test to test and the possibility of different sizes of specimens.

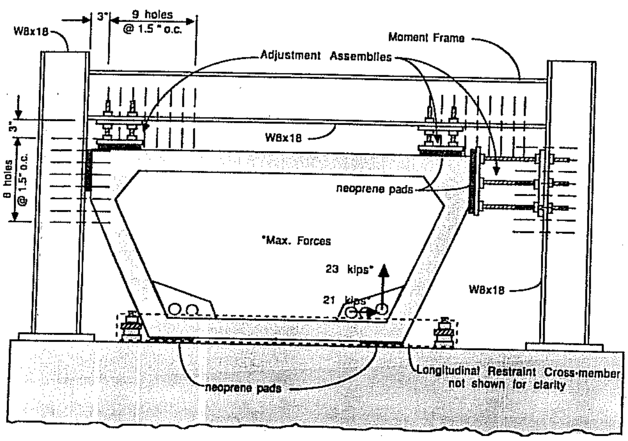



Fig. 4.10 Specimen in Restraint Frames

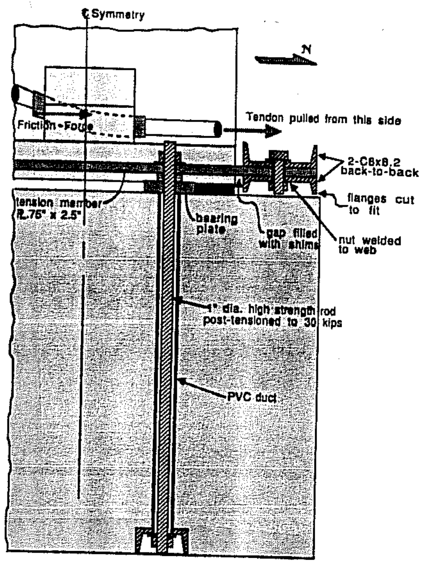



Fig. 4.11 Longitudinal Restrains

The pair of moment frames which are fixed to the pedestal were designed to carry the maximum foreseeable deviation force components. Attached to these frames are a set of small adjustment assemblies. The adjustment assemblies bear on neoprene pads at the upper corners of the specimen box section. The base of the specimen rests on four small neoprene pads located at each corner of the base. By tightening the adjustment assemblies against the outside of the box the specimen is clamped into position (Fig. 4.10).

During the first test it became apparent that the larger than anticipated capacity of the specimens resulted in longitudinal forces on the specimen. Slight damage to the pedestal required a more substantial system for longitudinal restraint. Figure 4.11 shows the revised detail, a double channel section which is prestressed to the pedestal. Steel spacer plates driven between the channel flanges and the bottom flange of the specimen ensure a tight fit.

4.3.4 Loading Concept. Forces on the deviator are increased by increasing the stress in the deviated tendons. This may be accomplished in a variety of ways. By adjusting the piston extensions on the pairs of 100-ton rams, then tightening the bolts on the frame restraining arms, the frames can be locked in any desired position (vertical, or inclined in or out). Then, using a standard jack for multi-strand tendons, any one tendon can be stressed incrementally to load the deviator. Or, for multiple tendons, the tendons can be stressed, one at a time, to a desired preliminary stress level. Then, the stress in all tendons can be increased as the 100-ton rams are activated, pushing out the lever frames thereby extending the tendons. In this manner, the force in the tendons can be increased from one side only, by pushing out on only one frame. Or, the force can be increased from both sides, by alternately or simultaneously pushing out both lever frames. The movement of the frames can be controlled by monitoring either the deformation of the frame or the pressure in the hydraulic rams. Retracting the rams unloads the deviator. The variability in the loading scheme allows the test setup to be used to model many possible prototype loading scenarios. In addition, with some modification to the hydraulic system, the setup can be used for dynamic as well as static loading.

4.3.5 Anchorages. A combination of standard and specially fabricated hardware was utilized to anchor the multi-strand tendons at each lever frame. Freyssinet-type multi-strand anchor blocks were donated by Prescon Corp. The forged steel anchor blocks, type 12K5P, have 12 tapered holes, drilled at correct angles to eliminate unwanted kinks in the strands, to accommodate up to 12-0.5 in. diameter strands and wedges. For the 3/8 in. diameter strands used in the tests, Prescon also supplied special conversion wedges that allow anchorage of the smaller strand in the holes for 0.5 in. diameter strands.

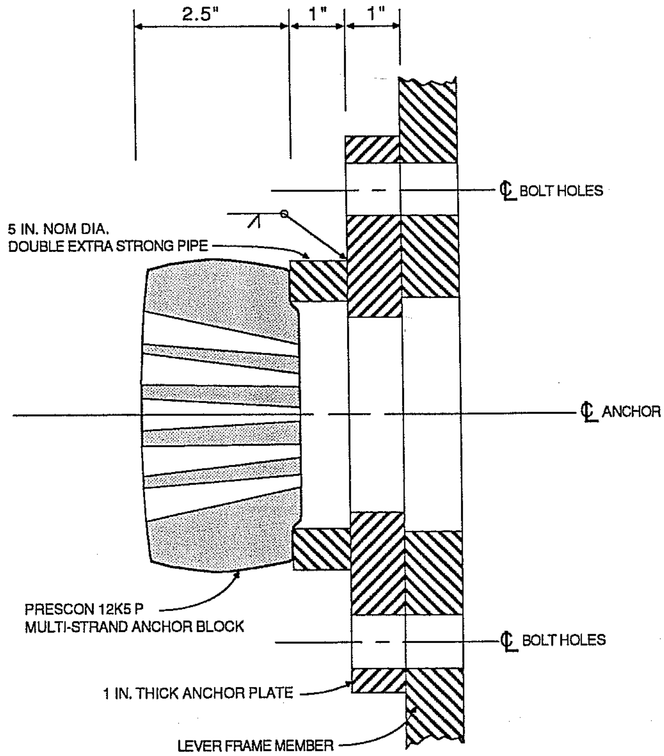



Fig. 4.12 Anchorage Detail

Anchor plates for "A" end anchorages consist of 1 in. thick rectangular steel plates with center holes for the tendons, and bolt holes spaced to correspond with the holes drilled in the horizontal members of the "A" frame. Bevel welded to the anchor plate are 1 inch long sections of 5 in. nominal diameter double extra strong steel pipe. The surface of the 3/4 in. thick pipe wall serves as the bearing surface for the anchor block (Fig. 4.12).

Anchorages at the "B" frame posed a special problem. The tendons deviated to the "B" frame are inclined vertically and horizontally at the anchorage point. A bearing surface perpendicular to the direction of the tendon is required. To accomplish this, a section of the 5 in. nominal diameter double extra strong pipe was flame cut at an angle calculated to correspond to the angle of the tendon, and bevel welded to the 1 inch thick rectangular steel plate. Bolt holes, drilled in the plate, match holes in the vertical members of the "B" frame.

4.3.6 Instrumentation Concept. Several types of electronic and mechanical instrumentation are utilized to collect key information during the tests. The most important measured quantity is the force in the external tendons from which the deviation force components are calculated. Two different systems were selected to measure tendon force. Pressure transducers connected to the 100-ton rams read hydraulic pressures that are converted to force units. The total force in the tendons is calculated from equilibrium conditions. For tests with only two tendons, equilibrium equations give the relative force in each tendon as well, since the point of application of the loads from the tendon and the geometry of the statically determinate frame are well known. A second system uses strain gages on several strands in each tendon.

Strain gages were placed on selected reinforcement within the deviation block. The objective of measuring strains in the reinforcement is to try to determine the participation of each reinforcing bar.

Linear potentiometers and dial gages monitor the rigid body motion of the concrete box and the displacement of the deviation block relative to the box. Potentiometers and dial gages are also used to monitor the movement of the lever frames.

A Hewlett-Packard data acquisition/control unit, hooked to an IBM XT PC collects and stores the electronic data.

# CHAPTER 5. SUMMARY

## 5.1 Literature Review

An extensive literature review traced the history and development of the technology connected with the use of external post-tensioning systems for bridges. The first known use of tendons external to the concrete section was in a German road bridge designed by Dischinger in 1928. The unavailability of high strength post-tensioning steel, and the then unknown effects of creep, shrinkage and relaxation rendered early post-tensioned structures ineffectual. Several externally post-tensioned bridges were built in France and Belgium in the post WWII era. The use of external tendons evaded the Freyssinet prestressing patents and resulted in a number of successful and innovative structures. In the 1970's in France, considerable refinements in external post-tensioning technology were made through extensive use of external tendons for retrofit and repair of bridges. External tendons were first introduced in American concrete bridge construction in the Florida Keys bridges in 1980.

The use of external tendons poses inherent advantages as well as disadvantages. Advantages include the omission of tendon ducts from the concrete cross section which reduces congestion and avoids interference of the ducts with the reinforcing cage. The accessibility of external tendons simplifies stressing and grouting procedures and allows the possibility of tendon replacement. Friction due to wobble is eliminated with external tendons and some fatigue advantages are apparent as well. The use of external tendons in segmental span-by-span construction has proven to save time and materials. Some disadvantages associated with the external post-tensioning include potential vibration problems for long unrestrained tendon lengths and limitations on available tendon eccentricity. In addition, unbonded, external tendons may result in reduced flexural efficiency and reduced ductility relative to comparable systems with internal tendons. Distribution of concentrated forces produced by external tendons at attachment points pose additional problems.

Deviators are required to create draped profiles for external tendons. Typical deviators for box girder bridges can be classified by shape into three categories: blocks, ribs and diaphragms. The detailing of external tendon ducts embedded in deviators (and in anchorage zones) warrants special attention. The recommended minimum duct diameter corresponds to a duct area of 2.5 times the tendon area, an increase over requirements for internal tendons. The minimum radius of curvature for external tendon ducts varies from 10 to 20 feet depending on location (i.e, in anchorages or deviators) and tendon size. In addition, since aligning ducts to meet the spatial geometry of the deviated external tendon is often difficult, special measures must be taken to avoid problems due to duct misalignment.

Design criteria which have been used to proportion and detail deviator reinforcement are not well known. Traditional design approaches do not directly treat the design or required safety of saddle or block type deviators. No generally accepted models exist for determining the diffusion of concentrated forces at deviators.

Problems have occurred with deviators in several existing bridges. The most common form of damage has been spalling and local cracking around misaligned external tendon ducts. Structural cracking has also been reported for both block and diaphragm type deviators.

The most significant experimental research projects concerning external post-tensioning currently known include two simple-span girder test series. One of these was performed at the Laboratory at Saint-Rémy-les-Chevreuse, France, by SETRA-CEBTP, while the other was performed at the PCA Construction Technology Laboratories in Illinois. A scale model test of a three-span box girder bridge with external tendons has recently been completed at the Ferguson Structural Engineering Laboratory. The only other known experimental research concerning deviators is currently ongoing at the Saint-Rémy laboratory, although the scope of that project seems limited to the behavior of the tendon in the deviator and not to the behavior of the deviator itself.

## 5.2 Experimental Facility for Deviator Studies

Experimental investigation of deviator behavior required that a generalized test setup be designed and constructed. The setup adapts to accommodate a variety of specimen sizes, tendon layouts and loading alternatives. A typical structure was chosen from among existing U.S. bridges with external tendons to serve as a prototype for the test facility design. A basic box girder section (minus the cantilevered wings), modelled geometrically after the prototype cross section, was designed and reinforced in accordance with AASHTO minimum specifications. The 1/3 scale box section was chosen as a basic test unit which would be adaptable to a variety of deviation details.

Various deviation blocks from segments in the prototype bridge were chosen for the initial 1/3 scale model tests. The reinforcement in the initial two deviators was identical but was varied in subsequent tests. The scaled reinforcement in the test deviators was provided by microbars having an average yield strength of 45 ksi. The primary reinforcement, which consisted of various bent bar types, were instrumented with a number of embedded strain gages. Secondary deviator reinforcement, which included additional web and flange reinforcement for the box section, was not instrumented. The tendon layout can differ in test deviators and can include from one to three deviated tendons.

The deviation blocks can be subjected to a full range of loading by incrementally increasing the stress in the deviated tendons. Deviator deflections, deviator reinforcing strains, and cracking patterns can be monitored continuously throughout the tests.

Based on test data and model assumptions, the effectiveness of each pattern of primary deviator reinforcement can be evaluated and basic behavioral models developed to assist in future deviator design.

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