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**MINIMIZING CONSTRUCTION PROBLEMS IN SEGMENTALLY  
PRECAST BOX GIRDER BRIDGES**

by

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and  
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Research Report No. 121-6F

Research Project No. 3-5-69-121

"Design Procedures for Long Span Prestressed Concrete  
Bridges of Segmental Construction"

Conducted for

The Texas Highway Department  
In Cooperation with the  
U. S. Department of Transportation  
Federal Highway Administration

by

CENTER FOR HIGHWAY RESEARCH  
THE UNIVERSITY OF TEXAS AT AUSTIN

August 1975

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## P R E F A C E

This report is the sixth and final report in a series which summarizes the detailed investigation of the various problems associated with design and construction of long span prestressed concrete bridges of precast segmental construction. An initial report in this series summarized the general state of the art for design and construction of this type bridge as of 1969. The second report outlined requirements for and reported test results of epoxy resin materials for joining large precast segments. The third report summarized design criteria and procedures for bridges of this type and included two design examples. One of these examples was the three-span segmental bridge constructed in Corpus Christi, Texas, during 1972-73. The fourth report summarized the development of an incremental analysis procedure and computer program to analyze segmentally erected box girder bridges. The fifth report summarized structural performance data obtained from a realistic one-sixth scale model of the structure and compared these data with analytic results from several computer analyses. This report summarizes many lessons learned from actual construction operations. Many of these points may seem insignificant or obvious. However, in almost all cases they produced concern or uncertainty in this pioneering effort in United States bridge construction. It is a high tribute to the designers and constructors involved that they gladly shared these problems so that those who undertake similar projects can reap the benefits of their experience.

This work is a part of Research Project 3-5-69-121, entitled "Design Procedures for Long Span Prestressed Concrete Bridges of Segmental Construction." The studies described were conducted as a part of the overall research program at The University of Texas at Austin, Center for Highway Research. The work was sponsored jointly by the Texas Highway Department and the Federal Highway Administration under an agreement with The University of Texas at Austin and the Texas Highway Department.

This report concludes the official record of a project which spanned seven years and witnessed the development of a new form of bridge construction in the United States. The project assisted in the transfer and development of precast segmental box girder technology from foreign countries to a pattern adaptable to United States highway construction. This development was greatly assisted by the decision of the Texas Highway Department to support this study and especially to undertake construction of the Intracoastal Waterway structure in Corpus Christi, Texas. A special thanks are due to those involved in the conceptualization, design, approval, and construction of this structure. Everyone involved shared their knowledge and experience willingly with the project staff. Everyone involved was in his own way a pioneer. Ralph Waldo Emerson said "A great part of courage is the courage of having done the thing before." The thousands of miles between Corpus Christi, Texas, and any similar project ensured that none of the participants had "done the thing before". Theirs was true courage.

As representatives of the many who contributed to this study, special thanks are due to:

(a) Texas Highway Department - Highway Commissioner D. C. Greer and State Highway Engineer J. C. Dingwall for their support at the policy level; Bridge Engineer Wayne Henneberger and Contact Representative Robert L. Reed for continued interest and support in all phases of the study; Bridge Planning Engineer Farland Bundy, Bridge Construction Engineer Clarence Rea, and District Engineer Travis Long for suggesting, approving, and supervising the Corpus Christi application; Messrs Jim Dunlevy and Alan Matejowsky for development of the final design of the structure. Actual construction was supervised by Resident Engineer Donald E. Skewis and District Construction Engineer Rodger Welsch. Project personnel were aided immensely by the Corpus Christi staff and especially by James E. Allen and Gerald R. Scalf. Mr. Andy Seely and Mr. H. D. Butler of the Bridge Division were extremely helpful in coordinating field and laboratory operations.

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## S U M M A R Y

The cantilever construction of the first segmental precast prestressed concrete box girder bridge in the United States has been completed on the John F. Kennedy Memorial Causeway, Corpus Christi, Texas. The segments were precast, transported to the site, and erected by the balanced cantilever method of post-tensioned construction, using epoxy resin as a jointing material.

This report documents major steps in construction of such a structure with emphasis on detailing lessons learned during the construction which might facilitate or improve similar projects. Many of these are seemingly trivial or obvious, but they were not necessarily such when first encountered. Suggestions involving design and detailing to improve constructability are made based on the Corpus Christi bridge experience. Similarly, recommendations for various steps and procedures in fabrication, casting, erection, and closure are made.

An extensive summary is provided of an exploratory investigation of the cause of, and suitable control measures for, web cracking which occurred in some of the units. Cracks similar to those noted in the prototype construction were reproduced in laboratory tests. Several crack prevention or control measures, such as extra stirrups, concentric spirals, and vertical post-tensioning were investigated. The model tests revealed that the cracking load could be increased to a level substantially above the design prestress forces.

## I M P L E M E N T A T I O N

This report presents a series of observations of minor and major problems which occurred in construction of the first precast segmental prestressed concrete box girder bridge erected in the United States. This type of construction is becoming increasingly popular. These observations represent a fairly comprehensive listing of special factors to be considered and should be valuable in acquainting design, construction, and contractor personnel with the special types of problems which may be encountered in this type of construction.

Extensive documentation is provided of the probable cause and possible solutions for the web cracking which occurred and interrupted construction of the prototype bridge at Corpus Christi. Consideration of these solutions would be valuable in future designs of this type.

A number of very specific recommendations are made for development of improved design, construction, and inspection procedures. Implementation of these recommendations should result in economic cost savings in future projects of this type through increased designer and contractor awareness and confidence. A number of the procedures highlighted during the field observations are already being implemented in future bridges of this type.

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## C H A P T E R    1

### INTRODUCTION

#### 1.1 General

Construction of longer span bridges is increasing in the United States to satisfy requirements of function, economics, safety, and aesthetics. The long span potential of prestressed concrete cannot be fully developed in pretensioned I-girder and composite slab systems. These systems have practical limits in the 120 ft. span range. However, substantially longer span prestressed concrete bridges have been built by utilizing precast and cast-in-place box girder bridges erected using cantilevering techniques.<sup>35</sup> This study will treat principally the segmental precast prestressed concrete box girder bridge erected by the cantilever method.

In this construction method, precast segments (Fig. 1.1) are cast and transported to the bridge site. The precast segments are erected, as shown in Fig. 1.2, as balanced cantilevers from the pier segment which is rigidly connected to the pier either temporarily or permanently. In some applications temporary props are used to provide the cantilever moment capacity. In the first applications of this construction technique, concrete or mortar was used as a jointing material between segments. However, the French used epoxy resin successfully as a jointing material in 1964.<sup>35</sup> Because of the rapid setting of the epoxy resin, this type of jointing shortened the construction period appreciably and became widely accepted. As each pair of segments is positioned at the ends of the balanced cantilever, negative moment tendons are inserted and tensioned. These tendons must provide moment capacity for the full cantilever moment. Erection continues until the last cantilevered sections are placed at the center of the span and at the end supports, as shown in Fig. 1.2(e). The positive

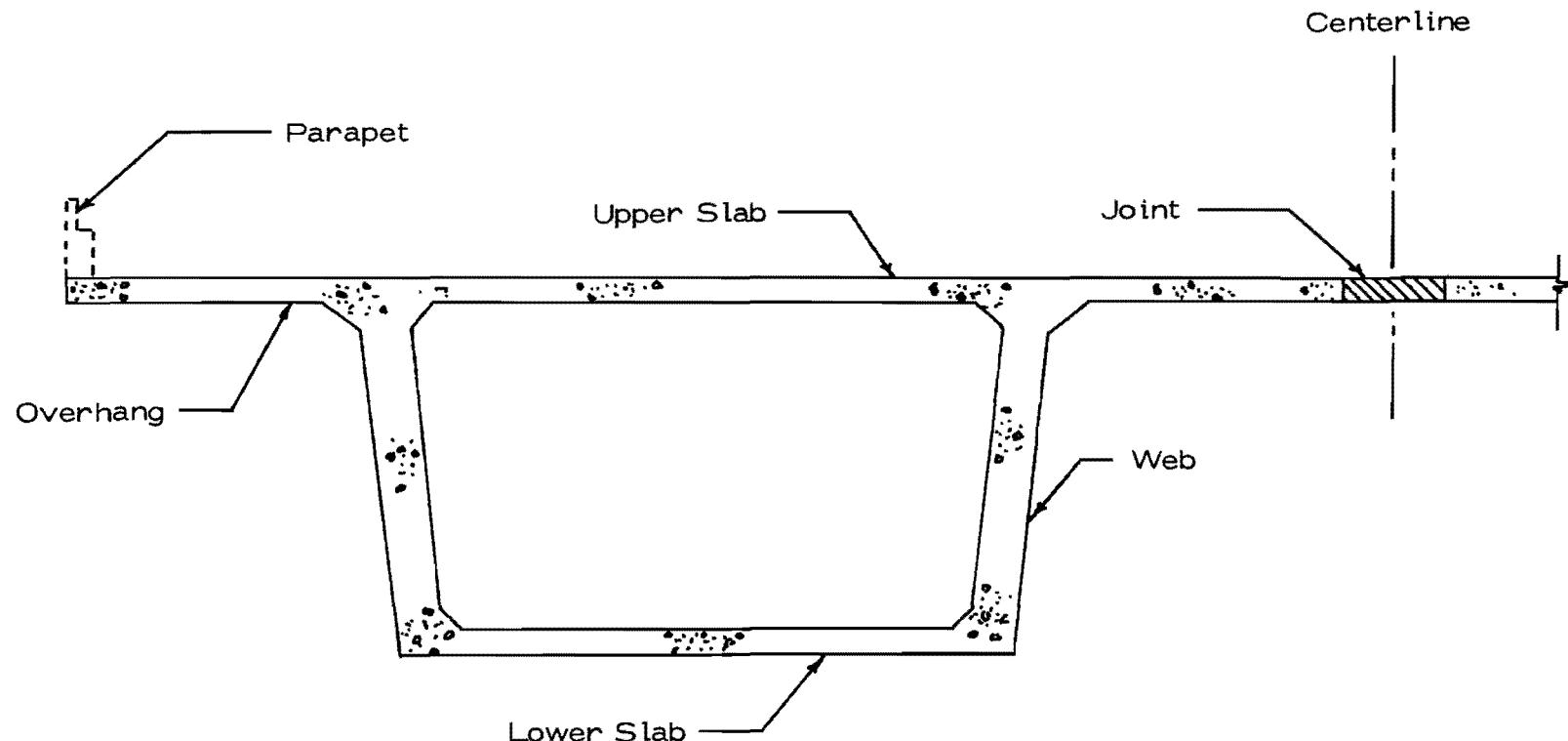


Fig. 1.1. Section through box girder bridge

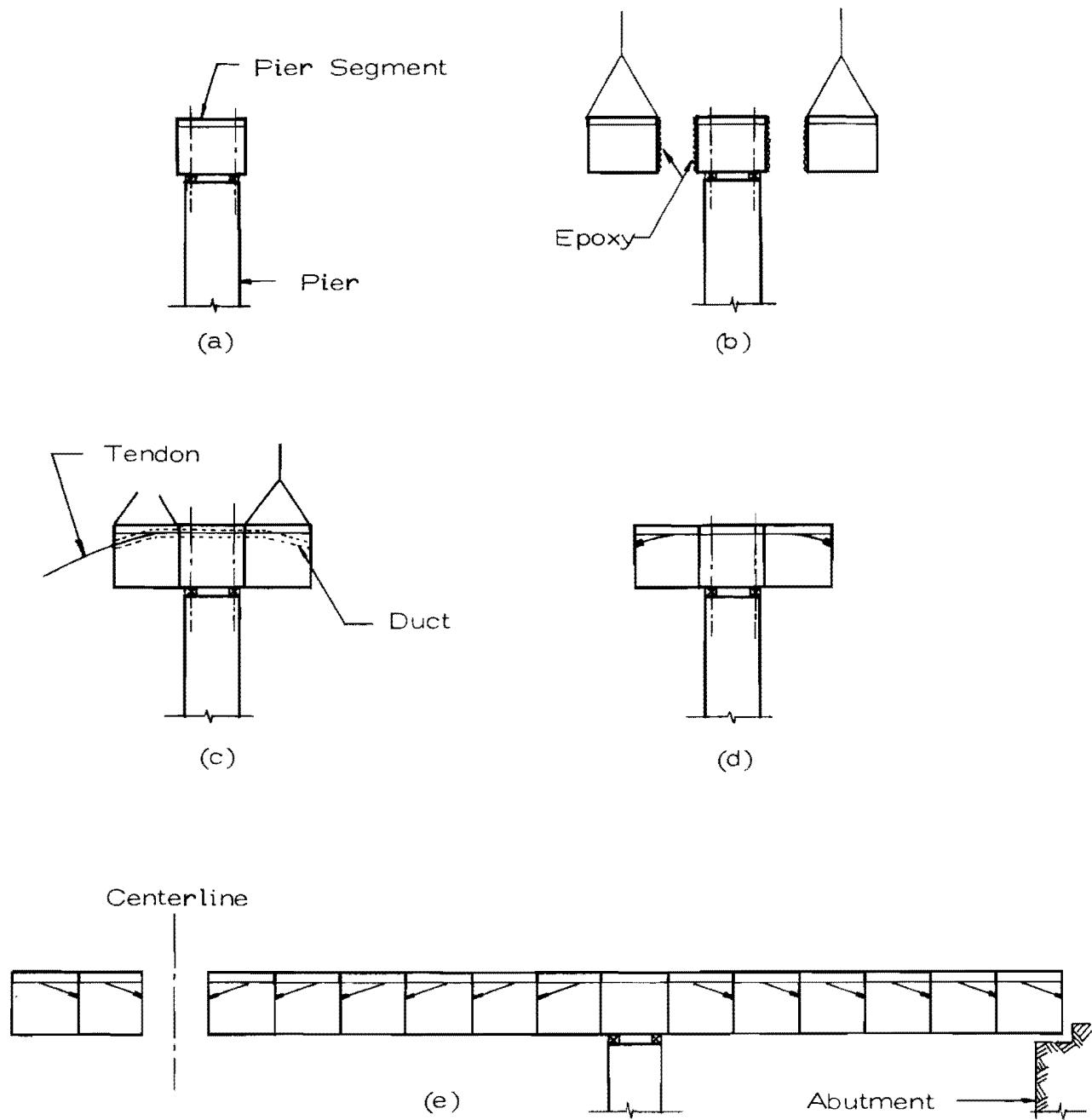


Fig. 1.2. Cantilever erection

moment tendons in the end span are prestressed and the end segments are seated on their supports prior to or during stressing of prestressing cables in the main span positive moment region. At midspan, the gap between the two cantilever arms is closed with cast-in-place concrete. Prestressing cables to resist live load in the central span positive moment region are inserted and stressed. The completed structure is shown in Figs. 1.3 and 1.4.

The use of such precasting and cantilevering techniques has the following advantages:<sup>18,20,27,32</sup>

- (1) Convenient to split large structures into manageable components to simplify transportation, handling, and erection.
- (2) Maintenance of navigational or traffic clearance during construction.
- (3) Units can be produced under industrialized conditions rather than in-situ, especially important when working over water or high in air.
- (4) High quality control of segments and control of deflection.
- (5) Flexible choice of the segment length depending on the capacity of transportation and lifting equipment.
- (6) Encourages "fast-tracking" through simultaneous start of segment casting and pier construction.
- (7) Significantly reduced erection time at the site. This is extremely important where construction seasons are limited.
- (8) Highly efficient use of forms.
- (9) Shrinkage can largely take place before erection and creep is minimized due to later age at stressing.
- (10) Full inspection and testing of individual parts can be carried out before they are incorporated in structure. Faulty units can be rejected with less interference with work progress.
- (11) Efficient labor use from simplicity and repetition.

However, as in all alternate solutions the method also has some disadvantages:

- (1) Materials are handled twice--first when units are made and then when units are placed in the structure.
- (2) Construction tolerances and alignment are critical. Tendon ducts must be clean and match perfectly or the job can be seriously delayed.

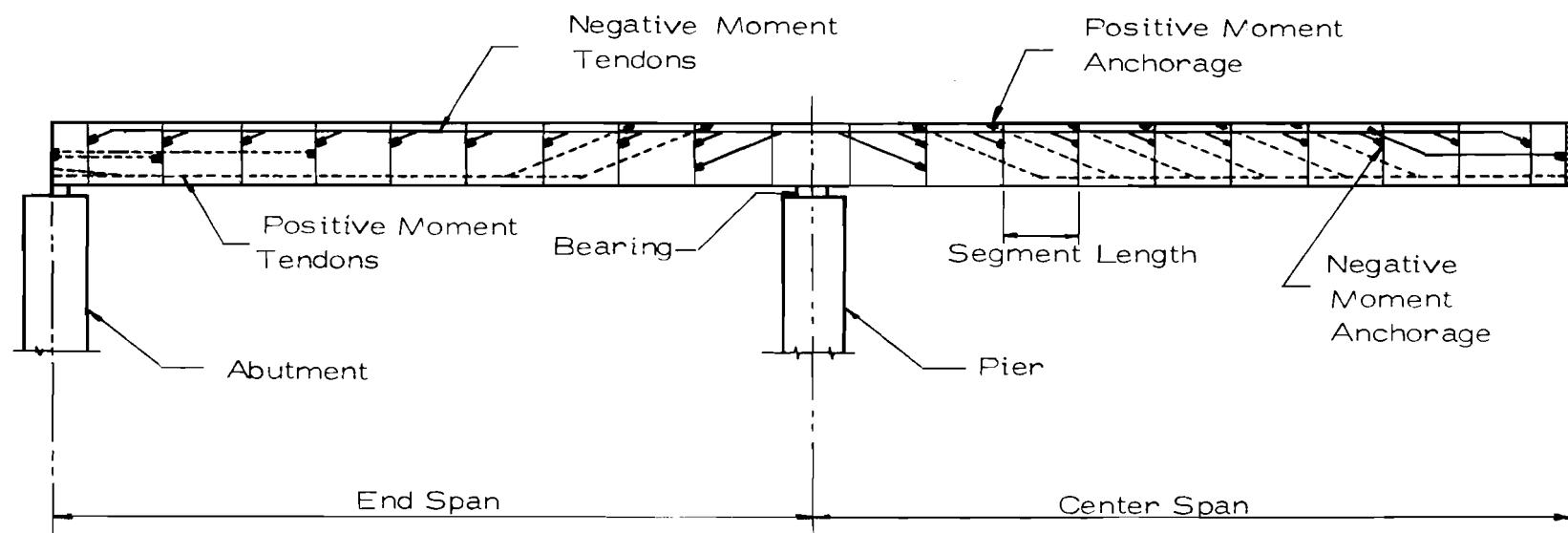


Fig. 1.3. Completed structure

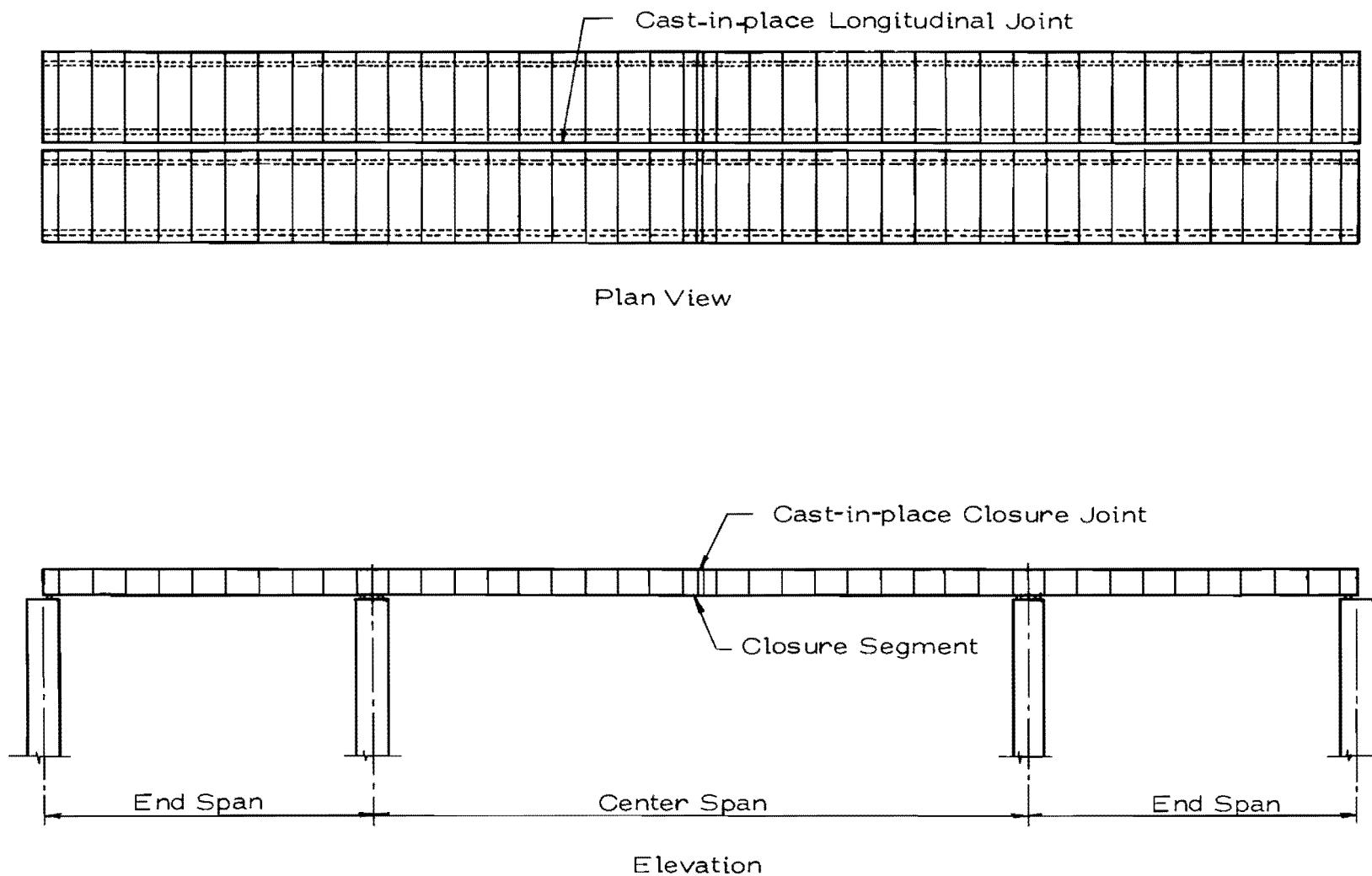


Fig. 1.4. Three-span bridge

- (3) Larger capacity lifting equipment will be required.
- (4) Handling and erection stresses may be critical.
- (5) Pier design may be greatly influenced by the moment capacity required to permit cantilevering.
- (6) Use of modularized units and balanced cantilevering requires greater standardization in span lengths.

Because this type of bridge had never been built in the United States, a cooperative research project with the Texas Highway Department and the Federal Highway Administration to investigate the various problems associated with design and construction procedures for long span precast prestressed concrete box girder bridges of segmental construction was undertaken by The University of Texas at Austin, Center for Highway Research, in 1968.<sup>8</sup>

The Texas Highway Department utilized a preliminary design developed as part of the project by The University of Texas at Austin researchers in developing plans for a long span bridge on the John F. Kennedy Memorial Causeway, Park Road 22, Corpus Christi, Texas. The requirement to maintain navigational clearance during construction as well as the highly corrosive environment on the Texas coast led to the choice of a precast prestressed concrete box girder bridge built in cantilever.

In order to study the applicability and accuracy of the design criteria, analytical methods, construction techniques, and shear performance of the epoxy resin joints, an accurate one-sixth scale model of the three-span continuous bridge was built and tested at the Civil Engineering Structures Research Laboratory of The University of Texas at Austin's Balcones Research Center. Subsequently, the prototype structure was constructed in Corpus Christi. This report summarizes observations during construction of the prototype which suggest improvements in design, construction, and inspection. It represents a summary of minor and major lessons learned during prototype construction.

## 1.2 Related Research

This construction observation program was only one element in a comprehensive investigation. Related research which has been completed includes:

(1) State-of-the-Art Survey.<sup>23,25</sup> In the initial stages of the research program a comprehensive literature survey was completed.

(2) Design and Optimization.<sup>24,26</sup> Criteria were developed for design procedures and preliminary designs of several example structures were made. The Texas Highway Department adopted one of these preliminary designs for the Corpus Christi structure and developed final plans. The bridge was largely designed by the Ultimate Strength Design method assuming that beam theory was applicable. Allowable stresses during construction and under service loads were also checked using beam theory. Service load behavior of the completed structure was then checked using folded plate theory to determine the effect of warping. Optimization of the cross section was studied by unconstrained nonlinear programming, although the optimal cross section was not used in the final design.

(3) Segmental Analysis.<sup>9,10</sup> In order to investigate the various critical stages during the cantilevering procedures, an incremental box girder analysis computer program utilizing the Finite Segment Method was developed by Brown. This program (SIMPLA2) treats staged construction with realistic prestressing forces and determines effects in both longitudinal and transverse directions.

(4) Experimental Study of Epoxy Resin Jointing.<sup>20,21</sup> While epoxy resins have been widely used with concrete as coating or patching materials and although a guide<sup>5</sup> existed for the general use of epoxy resin with concrete, at the inception of the study there was no U.S. specification for the specific use of epoxy resins in jointing a segmental precast prestressed concrete box girder bridge. A program of evaluations of various epoxy resin properties and requirements was undertaken which resulted in development of a Texas Highway Department tentative specification. Prior to the model bridge construction, a number of different epoxy resins were evaluated and the epoxy resin which came closest to meeting the specifications was selected for construction of the model. A revised specification has been suggested in Ref. 21.

(5) Model Study.<sup>22</sup> An extremely comprehensive structural model study of the Corpus Christi bridge has been completed. A one-sixth scale

microconcrete model was constructed by the balanced cantilever procedure. Comprehensive measurements of construction stresses and deformations were made. Upon completion of the model it was subjected to a wide program of service load, factored load, and failure load tests. The model study verified the general design procedure and provided many ideas for improvements in construction procedures.

(6) Field Study. The Corpus Christi bridge was instrumented with strain gages which were mounted on the reinforcement in various segments. Readings were taken at the time of cantilever erection. Structural behavior under actual service loads was observed for various loading conditions upon completion of the prototype construction. A summary of the field observations and a comparison with the model test results were given in Report 121-5. Summaries of major observations of construction procedures are given in this report. Details of an exploratory investigation of anchorage zone cracking are also given in this report.

### 1.3 Objective and Scope of This Study

A considerable body of literature exists which details design, fabrication, erection, and completion procedures for segmentally constructed precast box girder bridges. The first report in this series summarized the general state of the art as of 1969.<sup>23</sup> An excellent and comprehensive updating of advances in segmental construction has recently been published by the world's leading practitioner of design and construction of this type bridge.<sup>32</sup> A Prestressed Concrete Institute committee has also recently published<sup>37</sup> a Recommended Practice for Segmental Construction in Precast Concrete. However, both of these documents suffer from a common problem in that they are fairly general and do not contain specific examples of successful and unsuccessful details and procedures. In developing this construction method for application to a specific local situation, the owners' agents must first ensure availability of knowledgeable constructors. It became apparent from numerous prebid conferences with potential bidders for the Corpus Christi bridge project that local contractors were concerned that little was known of the pitfalls of the

procedures. Most articles published focused on the successful aspects and glossed over the difficulties.

The primary objective of this report is to document the improvements in design and construction operations suggested by the experience of building the Corpus Christi bridge. Publication of such experience might minimize field construction problems in subsequent construction of this nature. Wide dissemination of such information can be of tangible value to prospective designers and constructors of future projects.

This report is not an attempt to criticize any individual or organization for performance or lack of performance on this project. It is a measure of the quality of the individuals and organizations involved that most of the remarks, recommendations, and criticism came directly from those involved in the design and construction. Every individual contacted for suggestions was most cooperative and most enthusiastic about providing information to keep the next designer or constructor from struggling to learn the lessons learned by a select few in the Corpus Christi project.

Original special specifications for the Corpus Christi bridge and recently updated special specifications for a proposed segmental bridge are included in Appendix B. These specifications embody many of the ideas learned through the Corpus Christi experience.

While the field of segmental precast box girder construction is a very wide one, this report is primarily tied to the experience from the modest bridge erected in Corpus Christi. As in most reporting it tends to concentrate on the few things which went wrong so that future efforts may avoid this difficulty. This in no way should detract from the overall excellent performance by all involved. The bridge was completed on schedule and as visualized. (See Fig. 1.5.) Both the Texas Highway Department and the contractor deserve praise for this pioneering effort.

#### 1.4 Report Contents

Chapter 2 provides detailed background on the structure and construction operations, suggests design improvements, and discusses detailing



Fig. 1.5. Completed structure spanning  
the Intracoastal Waterway in  
Corpus Christi, Texas

changes. Copies of the major design drawings for the structure are included in Appendix A. Detailed problems and suggested solutions which occurred in formwork, fabrication, casting, and segmental unit inspection are covered in Chapter 3. Chapter 4 discusses erection and closure operations. The major unexpected problem which occurred in construction was the appearance of web cracking in the vicinity of the anchorages along the tendons. Chapter 5 summarizes the exploratory investigations undertaken to determine the probable causes and possible effects of the cracking. Considerable knowledge of anchorage zone behavior in this type member was obtained in this study which involved both field and laboratory studies. Recommendations are given which should help to minimize or eliminate such cracking in future applications.

Chapter 6 gives a brief summary of the most important conclusions and recommendations of the report. A more detailed list of conclusions and recommendations is given at the end of each of the subsequent chapters.

## 1.5 Industrialization

1.5.1 Introduction. With increased construction costs caused by increases in both labor and material prices, there comes a natural tendency to offset the inflationary rises through industrialization; i.e., the concentration of efforts and resources toward the common goal of improved production and reduced cost. The building industry has shown this trend with the systems approach and the application of modular principles. Many of the concrete bridges in use today exhibit some industrialization through the use of standardized pretensioned girders as part of the structural system. These girders are used extensively on bridges in the 40 to 120 ft. range.

One of the more common principles associated with the idea of industrialization is the application of mass production techniques which involve the use of specialized labor and machinery for the making of large quantities of a single product. Unfortunately, large quantities of completely identical units cannot be used for building a box girder bridge. However, the cells can be designed such that they are sufficiently alike to be able to make repeated uses of casting and erection equipment. This can

be made possible through carefully coordinating the entire program and standardizing as many items and details as possible.

#### THE KEY TO EFFECTIVE INDUSTRIALIZATION IS COORDINATION AND STANDARDIZATION

This includes such items as:

- (1) The selection of span lengths.
- (2) Types of construction.
- (3) Design loads.
- (4) Cross sections.
- (5) Reinforcement cages and details.
- (6) Stressing systems.

In general, the overall approach would be to have as many pieces of bridges built as nearly identical as possible. Often a single project can approach the practical volume of pieces required for effective precasting. A minimum of about 100 pieces and more desirably 200 pieces permit efficiency. In other cases, with shorter spans or smaller bridges, standard box girder designs can provide the economy of scale better than special designs, much like the use of standard length prestressed girders. This report discusses the coordination and standardization involved in all of the aforementioned areas.

Attention is focused on procedures required for economical construction of bridges by use of precast units. The two basic types of construction commonly used for box girder bridges (construction on false-work and construction in cantilever) are compared. Many of the problems and details associated with a casting program are discussed, including the reinforcing cages, concrete forms, stressing systems, and other details. Most of the solutions presented are based on the experience from the scale model construction program and the Corpus Christi prototype construction.

Segmental bridges can be classified into two types by their method of erection: (1) those where the segments are assembled in balanced cantilevers, and (2) those where the segments are temporarily supported by falsework during construction.

As opposed to bridges built on falsework, cantilevered bridges must fully support all design loads at any stage of construction. For this reason, the progress of some of the earlier cantilevered bridges was quite slow, since most of them utilized cast-in-place concrete as jointing between segments. The concrete had to be at full strength before cables were tensioned and erection of the next piece became possible, usually two to three days later. This slow rate of construction has probably been the primary reason for the absence of segmental construction in the United States, since the high daily cost for labor and equipment cannot be justified. However, techniques developed using epoxy joints, multiple shear keys, and temporary tendons, allow erection rates to be independent of the setting time of the jointing material. Some contractors are now reporting erecting eight segments daily (80 lineal feet of bridge) with the use of epoxy and improved construction techniques.

The design and erection of bridges constructed on falsework differs somewhat from those constructed in cantilever, but the casting of the segments is essentially the same in both cases. During erection the segments are usually not tensioned into a complete structure until all have been attached to or supported by a falsework assembly. After forming and curing joints between the cells with either concrete or epoxy, tendons are inserted and tensioned to provide the additional strength required before the units can act as a structure.

The combination of high interest rates and continuing inflation have made rapidity of construction an important factor. One of the great advantages of the precast segmental construction is that it lends itself to the "fast-track" construction operation. The contractor begins work on the substructure and concurrently, the precaster can work simultaneously with the superstructure. Capital requirements are reduced, since the basic forms can be reused more times and on-site time for erection and handling equipment is greatly reduced. American contractors tend to avoid heavy capital investments, preferring to rent lifting equipment for the period needed. This indicates that it is unlikely that the costly big erection gantries used in European applications will be immediately

introduced into the U.S. Design must consider the availability and economics of lifting units.

The particular cross section of a bridge is naturally dependent on the design conditions but will conform to one of three types, as shown in Fig. 1.6:

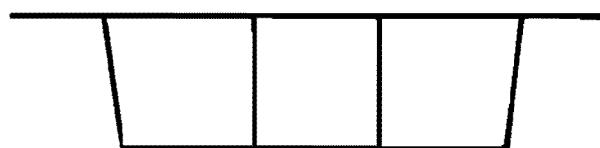
- (1) Single cell
- (2) Multi-cell
- (3) A series of single cell units with connected deck slabs

In the latter case, the segments are usually cast separately and joined after erection has been completed. This type will probably be the most frequently used cross section because of the minimization of handling loads. Lacey's optimization studies<sup>26</sup> show that single cell units apply primarily to narrow bridges and multicell segments tend to waste the concrete in the bottom slab. Muller<sup>32</sup> reports more of a trend to the multicell unit to reduce pier costs. For longer spans (300 ft. or more), the depth of the cross section usually varies along the length of the bridge from a maximum at the pier to a minimum at midspan.

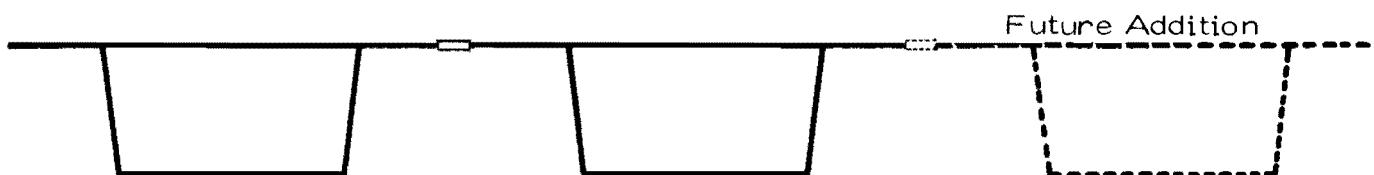
1.5.2 Organization and Management. Probably one of the major factors delaying the introduction of precast segmental box girder construction into the United States is our traditional division of responsibility in design and construction of concrete bridges. In this country the designer has traditionally thought of the completed structure and its behavior and the contractor has been responsible for the choice of construction method and the safety of the structure during erection. In steel bridges the designers have given more consideration to erection conditions and there has been more interplay between the two groups. Cantilevered precast and cast-in-situ segmental box girder construction is more engineering intensive than falsework supported construction. In Europe, there has been a much closer relation between the designer and the builder than has been traditional in the U.S. This has resulted in more rapid development of this engineered construction system in Europe.



Single Cell



Multi-Cell



Single Cells Connected with Joint

Fig. 1.6. Box girder cross sections

Since the purpose of this program was to assist the development of this type of construction in the United States, and not to revolutionize U.S. construction and engineering profession relationships, the Corpus Christi bridge was developed completely within the traditional framework for prestressed concrete construction in Texas. The bridge design was finalized by the Texas Highway Department Bridge Division and was bid by the normal qualified contractors who chose to submit bids. The design carefully considered cantilever segmental construction but left options open to the contractor as to the type of erection equipment and procedures to be used. No specific post-tensioning system was indicated. Rather specified tendon forces and centroidal locations were given. Selection of specific hardware systems and their details were left to the constructor subject to final approval. The contractor elected to precast the units himself but could have used a precast subcontractor. Tendons and stressing appurtenances were subcontracted, but actual tensioning was performed by the contractor. Thus, the construction process can be versatile and fitted into traditional systems of U.S. bridge building. It simply requires more educational effort to make all concerned--designer, inspector, precaster, materials supplier, erector, and owner--aware of their role in the process and the special nature of this type of construction. Prebid and postbid conferences for familiarization and training of personnel will pay high dividends in alleviating construction problems.

Otherwise, the organization and management required for construction of a segmentally constructed bridge is hardly different from any normal type of structure involving precast concrete. Managing the construction is essentially the same as with some buildings or even normal slab and girder bridges; it is coordinating the construction and the joining together of certain precast pieces into a useful structure. Of course, there are hundreds of items to be individually recognized and scheduled, including a considerable amount of logistics problems. When all factors are considered a network of problems exists. Network models, such as PERT and CPM have been developed which show the time and cost relationships of each item to the project as a whole.

1.5.3 Standardization and Coordination. The two items most important in the industrialization of a product are standardization and coordination. These are both necessary for the optimum use of mass production procedures. As mentioned earlier, box girder units are not presently designed as completely standard units. However, certain "standard" changes can be made in each unit. These changes usually involve the deletion or addition of one tendon in each segment and may be easily handled with proper concrete forms and tendon templates.

Standardization can involve almost any number of things; a considerable amount was accomplished in the planning of the Corpus Christi bridge. For instance, the cross section of each segment is identical with the exception of the first 25 ft. from the centerline of the two main pier supports. Over this distance the thickness of the bottom slab is reduced in two steps from 10 in. to 6 in. This is one case where a completely standard unit would be quite uneconomical; if each unit had a 10 in. lower slab, as required by pier negative moment, the additional dead load would require a still thicker slab at the pier, and so on. The problem would soon become unreasonable.

Another item of standardization was the constant depth of the bridge, adding to the simplicity of construction. Also, the design used only one size of post-tensioning strand, varying the number of strands in any one tendon as required for different tendon forces. As a direct result of the model program, a standard tendon layout and a standard scheme for bringing each tendon to its respective anchorage were developed. A common location for anchorages was also incorporated in each segment. Through coordination in the design, reinforcement spacings were selected so that the web reinforcement would match and thus could be tied to the bars in the top and bottom slabs. Each reinforcement cage could also be made completely identical, except for the pier sections. Spacings can be maintained identical in various units and bar sizes changed as steel requirements diminish with lessened shears away from the piers.

Although shear varies along the bridge, erection shears are quite similar. Standard shear keys were developed and provided in each section.

With these standardized cross sections, one form can be used to cast any of the segments. This can amount to a considerable savings in labor costs associated with forming, since complicated adjustments are not necessary. Finally, the procedure for erection of the segments is the same for each of the intermediate units, the pier sections and closure sections being different. All in all, this type bridge can be quite simply built and in Corpus Christi the rate of casting and erection improved steadily as experience was gained. Unfortunately, that project was so small (84 segments) that the project was half over before these benefits were attained. In construction of the Palacio del Rio Hotel in San Antonio using box segments of the approximately same weight, the labor time required per box decreased 20 percent after the first fifty units were completed, because of efficiency of repeated modular construction.

In undertaking segmental construction, all concerned should be brought up the learning curve as quickly as possible. There are a number of excellent reports, collections of slides, movies, and speakers available who can assist in this educational process. Not only designers but contractors, first and second line supervisors, and inspectors will benefit from such familiarization. A series of dry runs in the erection yard before trying seating and stressing on site will likewise pay important dividends in coordinating the crew and highlighting potential trouble areas.

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## C H A P T E R      2

### DESIGN

#### 2.1 General

Full documentation of the design procedure was given in Report 121-3. This report treats only factors concerned in the constructability.

There are certain basic assumptions made as the initial step in segmental bridge design; each person involved with the bridge should be made aware of the relation of these assumptions to one another. The assumptions have to do with such items as:

- (1) Whether the structure is to be continuous or where joints will be located.
- (2) Whether the span lengths are to be uniform or variable.
- (3) Whether the segment depth is to be uniform or variable.
- (4) Whether segments are to be laterally and vertically prestressed or normally reinforced.
- (5) Whether the bridge is to be constructed in cantilever or on falsework.
- (6) Whether the permanent piers will have to resist all erection moments or whether temporary supports will be used.

The main items affecting these assumptions are the location and loading of the structure, the types of construction equipment available, and the materials available. Each of these items and some of their effects are discussed in this chapter.

#### 2.2 Considerations

Since there are many factors affecting the final design of any structure, only those considered as most common are discussed.

2.2.1 Location. The particular site chosen for a bridge can have more effects than any other consideration. For instance, the crossing

proposed for Park Road 22 in Corpus Christi is over a navigable body of water; the minimum span length and vertical height is then set by the clearance requirements for the navigation channel. In other water crossings deep water channels can have an influence on the span lengths. Likewise, continuously deep water with difficult pier conditions will require maximum span lengths, as in the viaduct linking Oleron Island to the mainland in France. In Switzerland, foundation problems and a requirement that a minimum of trees be cut forced designers into building twin continuous structures with vertical curves, horizontal curves, superelevation, and three different span lengths. The result was a beautiful pair of bridges which give the appearance of having been molded into the landscape. Such structures are probably going to become more and more a part of our daily life, i.e., structures which enhance and improve the surrounding area.

If a bridge is to be located such that falsework is prohibited, then cantilever construction is denoted. Some segmental bridges with spans over 250 ft. have been constructed using falsework trusses; however, cantilevered units will probably prove to be more economical. Extremely long span lengths will undoubtedly call for prestressed cells as a means of reducing section weight. Generally, all structures will be continuous up to about five spans, since there is inherent economy and efficiency in continuous long span structures.

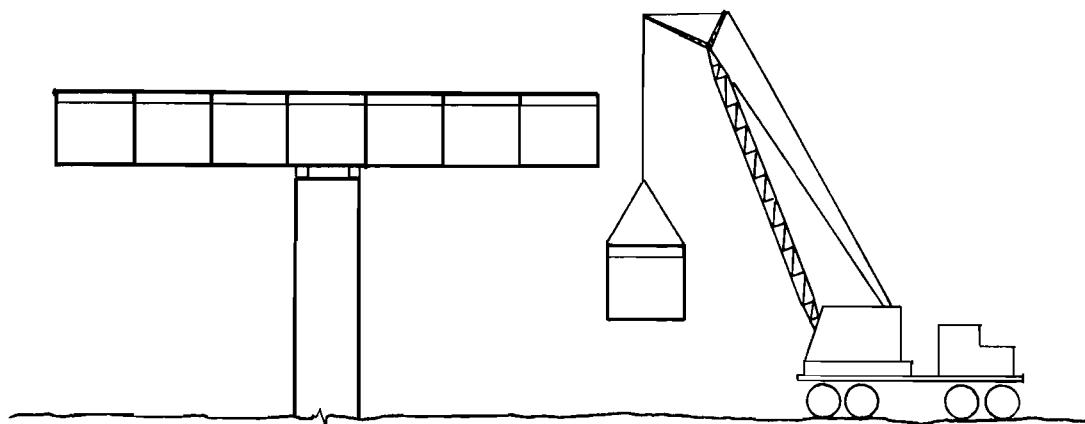
2.2.2 Construction Equipment. The type of construction equipment available in a particular area can have a considerable influence on the design of a segmental bridge. As an example, one of the major reasons for the use of two single cell boxes as the cross section of the Corpus Christi bridge was to limit weight to about 30 tons and enable any contractor with normal handling equipment to be able to construct the bridge. On the other end of the scale, the Oosterschelde Bridge in The Netherlands was designed assuming the use of a large overhead truss and lifts up to 600 tons were accomplished. Generally, the size and weight of segments will be determined by that size which can be transported from the casting site to the bridge site. If falsework assemblies are available and other conditions permit, bridges may be more economical if designed to be on falsework due to the potential savings in pier costs.

For cantilevered bridges, design loads can vary considerably dependent on whether or not a segment being placed has to be supported by the cantilever. The bridge with which this report deals was initially designed to have all segments suspended by mobile cranes operating from barges below until tensioning of the negative moment cables was completed. Since this would require the use of two cranes and their continuous operation during erection and stressing, the concept was changed to permit use of one crane. A check of the calculations showed that the bridge could safely support the additional 30 ton load caused by the weight of a unit coming onto the cantilever prior to tensioning, and so the unit could be hoisted and suspended from the structure using temporary hardware until stressing was completed. From experience gained with this and with other segmental bridges around the world, all bridges constructed in cantilever should be designed to support the unbalanced weight of a full-sized segment on either end of the double cantilever. This is true since many construction schemes or an accident could place the full segment weight on the structure. By the same token, the pier or temporary support and the moment connection between pier and pier segment should be sized to withstand the one unbalanced segment condition at any stage.

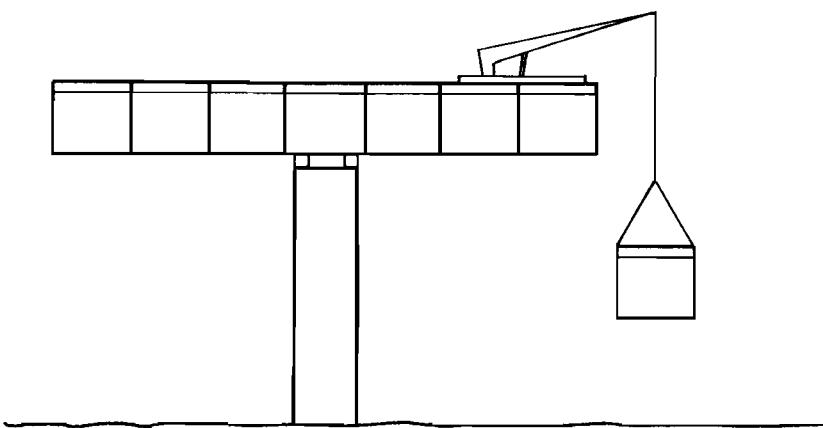
In many cases where bridges to be constructed are several miles long, European contractors have successfully used large overhead trusses which are slightly more than two spans in length. They are used on cantilevered bridges to place the segments one at a time for tensioning, creating the unbalanced condition mentioned above.

There are basically three methods for getting segments from the casting yard to the final position on the bridge, each affecting the design. They are as follows:

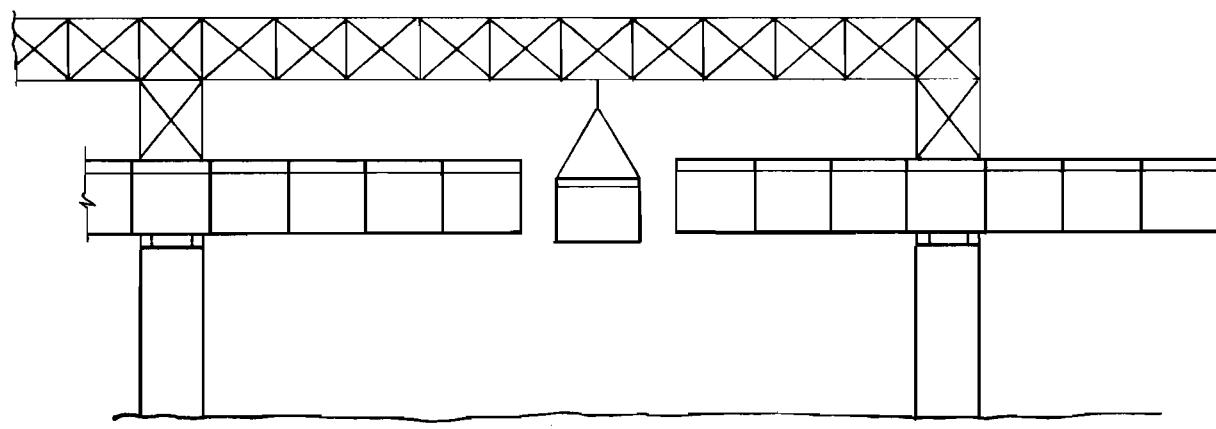
- (1) Transport the segment by barge or truck to a point beneath the bridge and lift it into position with a crane from below [Fig. 2.1(a)].
- (2) Transport the segment to a point beneath the bridge and lift it into position with a device on the structure [Fig. 2.1(b)].
- (3) Transport the segment over the completed structure and position it with an overhead truss on the structure or crane on an adjacent structure [Fig. 2.1(c)].



(a)



(b)



(c)

Fig. 2.1. Methods of placing segments

The overhead truss previously mentioned works very well for the third method, especially in inaccessible areas such as heavy timber, swamps, or shallow water. As far as the Corpus Christi bridge and any other similar designs are concerned, mobile cranes could be most economically used. For instance, both halves of the entire structure could be erected with the use of mobile cranes mounted on barges beneath the structure. If only one crane were to be used, a simple method of holding one segment in position while the corresponding segment was placed and tendons stressed has been developed. Some of the devices which have been used will be discussed in Chapter 4. Similarly, the entire structure could be erected with the use of a simple hoist mounted on the bridge, with the exception of the pier segments which have to be handled from below. Another way would be to assemble half of the bridge by one of the methods mentioned and to erect the second half with the aid of one or two mobile cranes located on the completed first half-structure. Segments could be carried by truck on the completed half or lifts could be made from barges below.

Since the pier section with its diaphragms is usually the heaviest section, its weight will ordinarily govern the size of lifting equipment. Should this weight be excessive, several ways are available to cope with this problem. The pier segment could be cast-in-place, between two temporarily supported segments. A shorter length could be chosen for the pier segment, which would reduce weight. The unit could be cast as two half-length units, lifted into position and bolted or post-tensioned together. Efforts should be made to hold down the weight of this unit as it will be the key to selection of lifting equipment.

In all cases, designers should be made well aware of construction techniques which are likely to be utilized, and contractors should be informed as to what techniques are allowed by the design.

2.2.3 Materials. Since the segmental bridges use precast segments, all specifications pertinent to precasting should be required and only the best quality raw materials allowed. Because long spans are inherent in segmental bridges, dead load of the bridge itself is tremendously

important; therefore, design practice should be directed toward reducing weight to a reasonable extent. The concrete cylinder strengths of 6000 to 7000 psi common in precasting yards aid considerably in reducing required cross section, since higher design stresses are allowed. The general economics of precasting indicate that economic benefits are usually attained when the high strength plant quality materials are required and used. Size should be optimized thinking of  $f'_c = 6000$  psi or more. Otherwise the design may be economically unfeasible when compared to cast-in-place. Some yards are frequently using 8000 to 10000 psi concrete, and research work now being done indicates even higher possible concrete strengths. Such practices in the future may lead to long span reinforced concrete structures consistently more economical than steel.

The now common practice of designing for full use of 60 ksi reinforcing steel also helps in reducing dead load and has the added benefit of sometimes providing extra clearance for tendon ducts. Both the model study and the prototype construction has shown that clearances are particularly small in areas near anchorages and in areas where tendons turn down from the top slab into the web. The extra distance provided between reinforcing bars due to the use of 60 ksi steel is a very beneficial detail.

Almost any system of post-tensioning can be adapted to segmental box girder construction. Designers should be aware, however, that there are physical limitations on the use of post-tensioning equipment; these include some of the following:

- (1) minimum radius of curvature
- (2) anchorage size
- (3) duct size
- (4) splices
- (5) stressing equipment

A minimum radius of curvature of 25 ft. for tendons is usually specified. This specification can become important when there are conflicting directions of tendons, such as when positive moment cables are anchored in the top deck of cantilevered bridges. Since the anchorage usually falls in the area occupied by the negative moment cables, proper

planning must be done to ensure that there are no intersections of cable paths.

The anchorages may be designed to fit within the cross section proper, usually in the web, or special pockets may be provided externally. Web stiffeners with stressing pockets are now being widely used according to Muller.<sup>32</sup> These have the advantage that when paired with a system of temporary tendons or bolts, the stressing operation is taken out of the critical path and may be accomplished later after several segments have been placed. Some bridges have been constructed using anchorages which are accessible from outside the structure, but they are usually found on the interior portions. If each segment is to be stressed on erection, it may be more economical to anchor cables within the web area, since the blockouts required are more easily formed than the external pockets. Sometimes the physical area required for bearing of the anchor simply cannot be provided in the normal cross section; in such cases the necessary area must be provided by a special built-up section. For negative moment anchorages, such a buildup is usually provided on the interior wall of the web or on a web stiffener; for positive moment anchors, on the upper surface of the lower slab or in web stiffeners (Fig. 2.2). In the design of the Corpus Christi bridge, the minimum thickness of webs was set at 12 in. so that anchorages could be located in recessed pockets of the web. Experience indicated this was theoretically but not practically sufficient. Tolerances on the confining reinforcement, the congestion near anchorage pockets, and the general crowding of the web indicated that the minimum thickness should have been about 14 in. and that the entire area near the web and top flange intersection should have been widened as indicated in Fig. 2.2 to provide more area for tendons and reinforcement. The problems were particularly difficult in the units in the vicinity of the pier, which had two tendons anchored in each web.

The congestion in the lower webs in the midspan sections with large numbers of positive moment tendons was particularly acute. Movement of some of these tendon anchorages to pockets on the lower flange would be of great assistance. This means provision must be made for

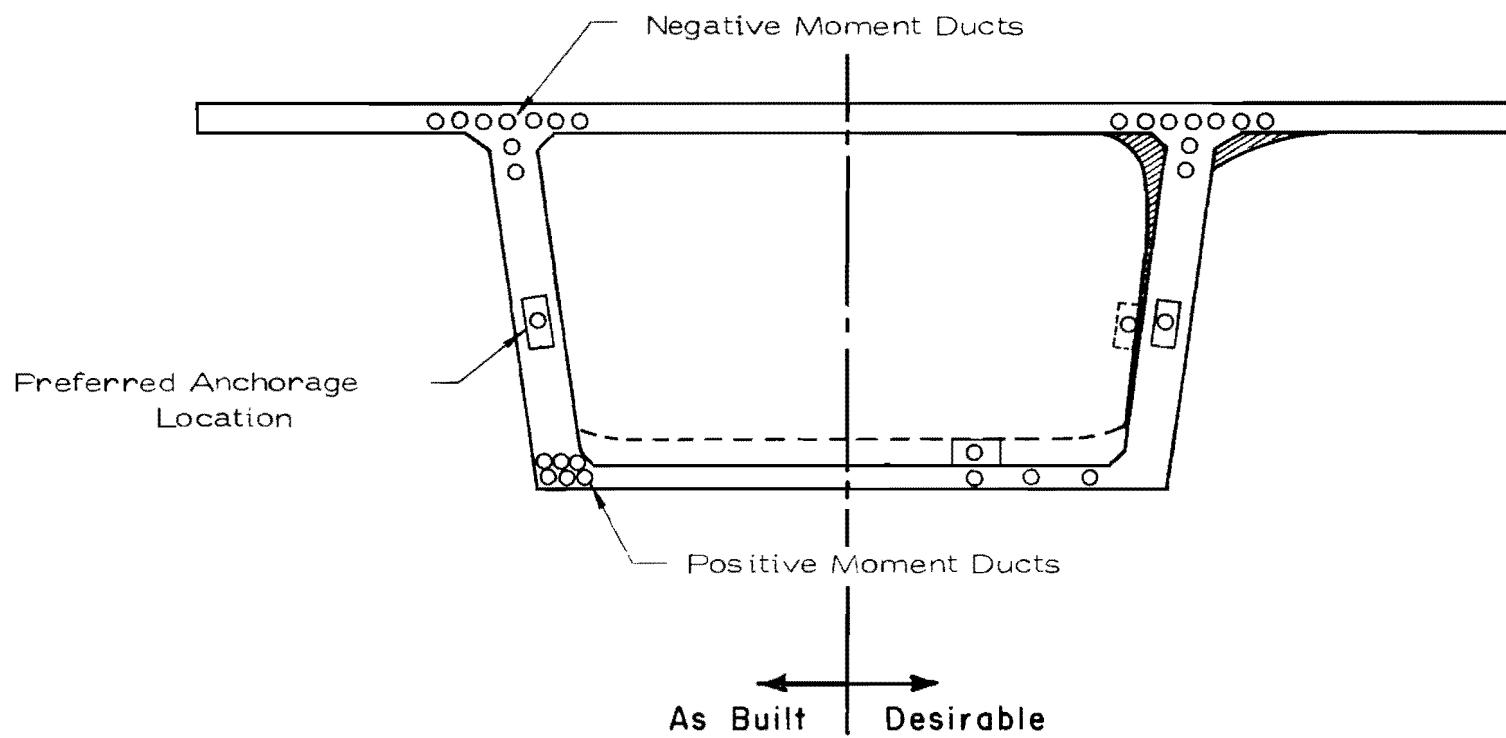


Fig. 2.2. Tendon ducts and anchorages

access to the interior for stressing equipment after closure, since all of these tendons are not stressed until closure has been made.

Web congestion could also be reduced by use of smaller diameter but closer spaced web reinforcement. The contractor suggested this would be an acceptable improvement. The wider web at the top would also be useful in providing anchorage for lifting equipment. A wider web would also provide increased cover for corrosion protection.

Since the tensioning cables have to be located within the reinforcing cage, careful attention must be paid to the size of ducts required to house the tendons. The negative moment cables must be located as near as possible to the top of the section for greatest efficiency, usually being between the tension and compression reinforcement of the top slab (Fig. 2.2). In the longer span bridges, the duct size can control the distance between the reinforcing bars since the ducts range up to about 4 in. in diameter. The most common ducts are made of steel, either rigid or flexible, rigid tubes having a slight design advantage because of their lower coefficient of friction and greater resistance to placement forces. Steel ducts are needed since most tendons are grouted after tensioning, and the steel is required to transmit strains between the tendons and surrounding concrete. Some bridges have been constructed with ducts formed by removable tubes, leaving a duct slightly smaller in diameter than steel ducts. However, even if the high friction losses are compensated for in the design, there are still physical problems which outweigh advantages offered by plain concrete ducts. These are discussed in Chapter 3.

Some tensioning systems and designs require splices in the tendons; these have been easily handled in previous bridges. The splice area usually amounts to nothing more than an enlarged area of the duct and can be solved with relatively simple blockouts in the cross section. Manufacturers of stressing equipment can provide the necessary dimensions for blockouts at external anchorages.

Assurance must be made that adequate clearances are provided at all stages of erection for the stressing equipment. The main areas of

concern are at the closure joints and abutments. The closure joint for the Corpus Christi bridge is only 18 in., but the negative moment cables for the last precast segments adjacent to the cast-in-place closure section are short enough to be stressed from the anchorage in the top slab (see Fig. 1.3). For some bridges, the length of cable required prevents stressing from one end only, and necessary clearances must be provided at the closure section. The same principles apply at the abutments.

### 2.3 Design Procedures

2.3.1 Bridges. Specific procedures have been established for the complete design and optimization of the superstructures of segmental bridges in Report 121-3.<sup>26</sup> The two types of bridges, those constructed on falsework and those erected in cantilever, are covered. This material will not be repeated herein.

2.3.2 Piers and Abutments. The design of piers and abutments for segmentally constructed bridges is essentially no different from other types of bridges except that extra moment capacity must be provided for cantilevered bridges. There are several ways of providing this capacity, including the following:

- (1) Design full capacity into the pier [Fig. 2.3(a)].
- (2) Design for use of temporary struts or tension ties [Fig. 2.3(b)].
- (3) Provide temporary shoring [Fig. 2.3(c)].

The Corpus Christi bridge piers were designed to withstand the full moment produced by unbalanced conditions, as discussed in Sec. 2.2.2, and the pier segment was temporarily rigidly attached to the pier by high strength bolts. The primary reason for this type of design was the fact that the piers were to be in the water next to a heavily traveled ship channel. Any method of providing temporary support which might withstand an accidental impact would be difficult. The additional moment capacity did add to the massiveness of the normal piers.

In retrospect, substantial economies may have been realized with alternate pier designs. The actual bid costs (1970 dollars) based on the

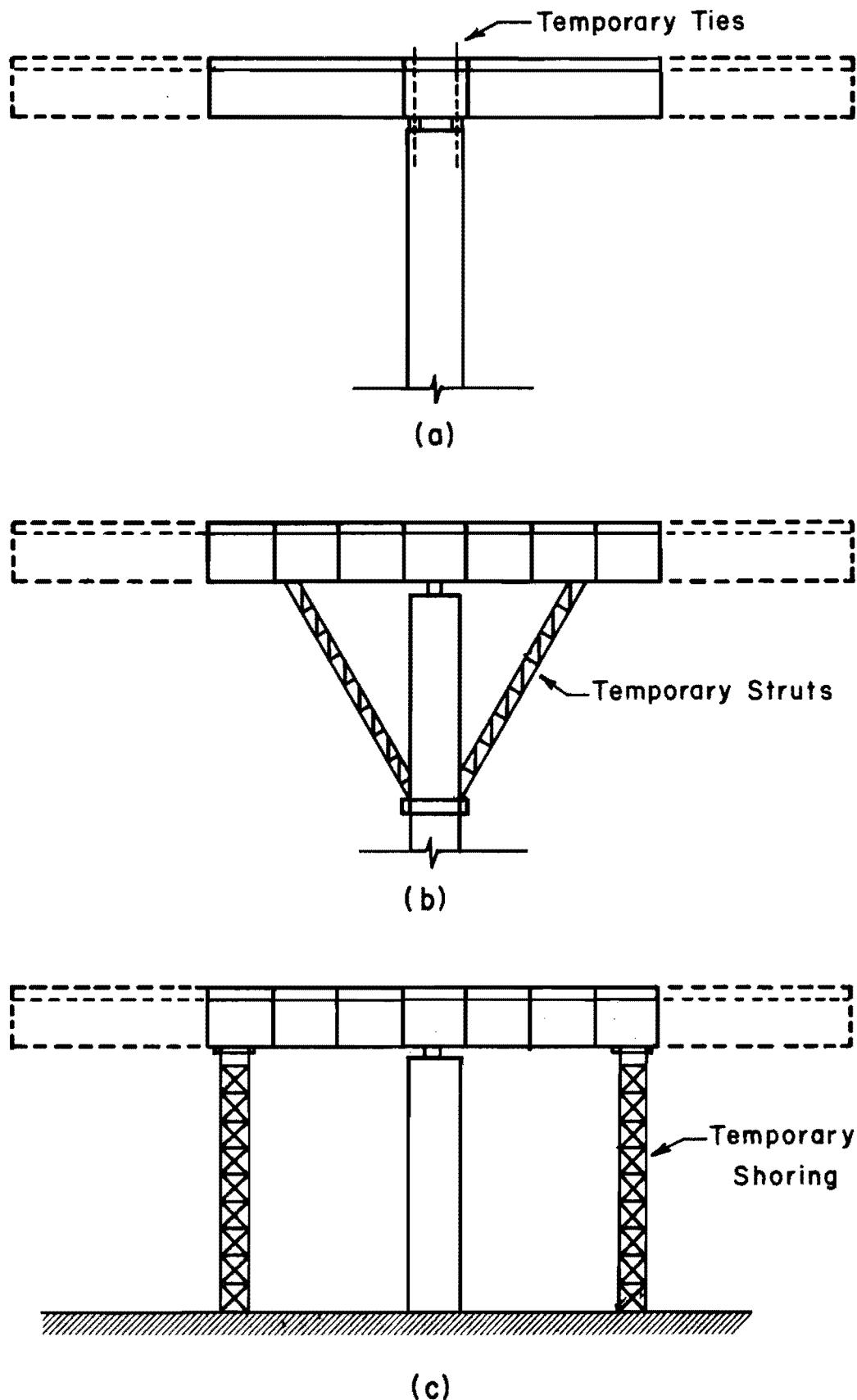


Fig. 2.3. Provisions for unbalanced moment

cost per square foot of the 400 ft. by 56 ft. superstructure are shown in Fig. 2.4. Pier costs were of the same magnitude as superstructure costs. Considerable attention was given to such items as minimizing forming costs in the box girders. As can be seen from Fig. 2.4, such costs were insignificant compared to the pier costs.

If the pier section above the waterline and danger zone from ship impact were redesigned as a hollow box or series of interlaced columns as shown in Fig. 2.5, substantial savings would be possible in materials and piling, even though full stiffness and moment capacity could be maintained. Pier design demands careful attention.

The model test program<sup>22</sup> also indicated that the temporary connection of the pier segment to the pier should be a fully prestressed one. In the Corpus Christi bridge the bolts were not initially stressed. Continual checks had to be made by the erection crews, since the nuts would loosen on each application of new segments. The test program indicated substantial erection deflections due to the low stiffness of the connection. As shown in Fig. 2.6, this has the effect of greatly reducing pier stiffness. This could be corrected by prestressing the temporary ties.

Many bridges have been erected with temporary struts which lead from the soffit of the bridge to a point on the pier stem near the footing. Over land, the simplest solution to providing the moment capacity required is to use shoring towers under the soffit of the segments on either side of the pier. However, some moment restraint still must be provided in the first few sections erected. There are many items to consider in each individual bridge and the designer must consider the various problems before deciding how to provide moment capacity for the unbalanced condition. A general knowledge of equipment available to likely contractors can be of considerable help.

2.3.3 Corpus Christi Bridge Design. The preliminary design for the crossing was a three-cell box girder; however, the final design calls for two single-cell boxes joined with a cast-in-place longitudinal joint,

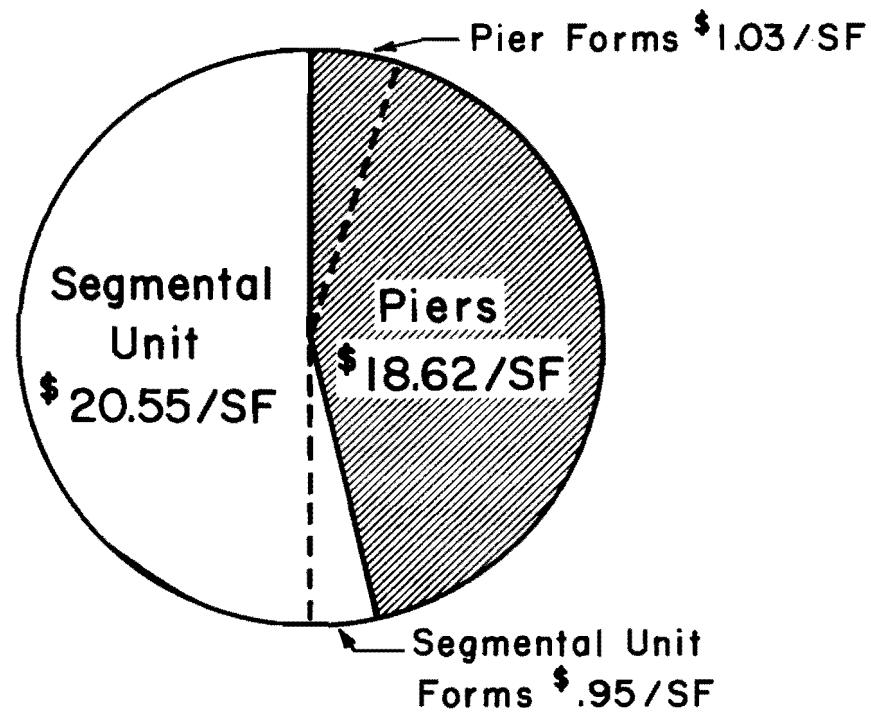
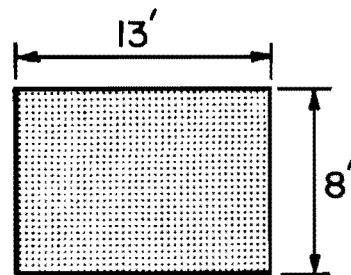


Fig. 2.4. Cost perspective for Corpus Christi bridge

## PIER DESIGN

Reduce Pier Mass



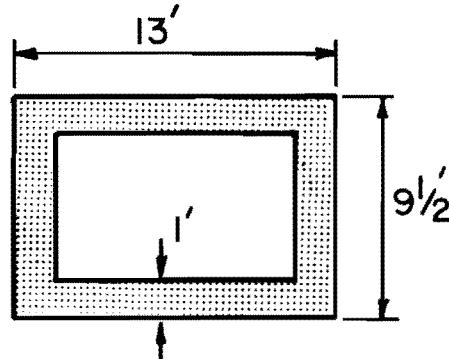
385 cu.yd.

Pier D 1560 K

Superstructure 1600 K  
D+L+I

Total 3160 K

100%



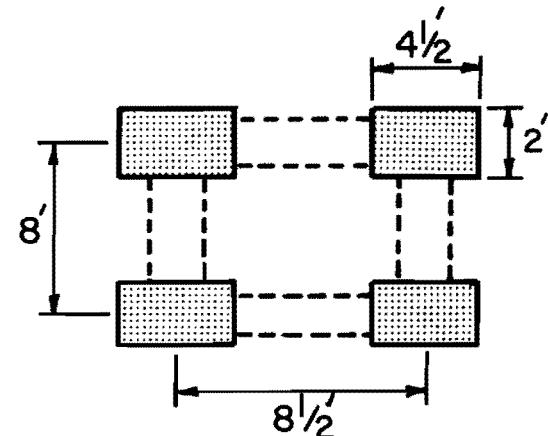
166 cu.yd.

672 K

1600 K

2272 K

72 %



150 cu.yd.

608 K

1600 K

2208 K

70 %

Fig. 2.5. Alternate pier designs

# PIER DESIGN

Stiffen Cantilever Connection

Design for Strength and Stiffness

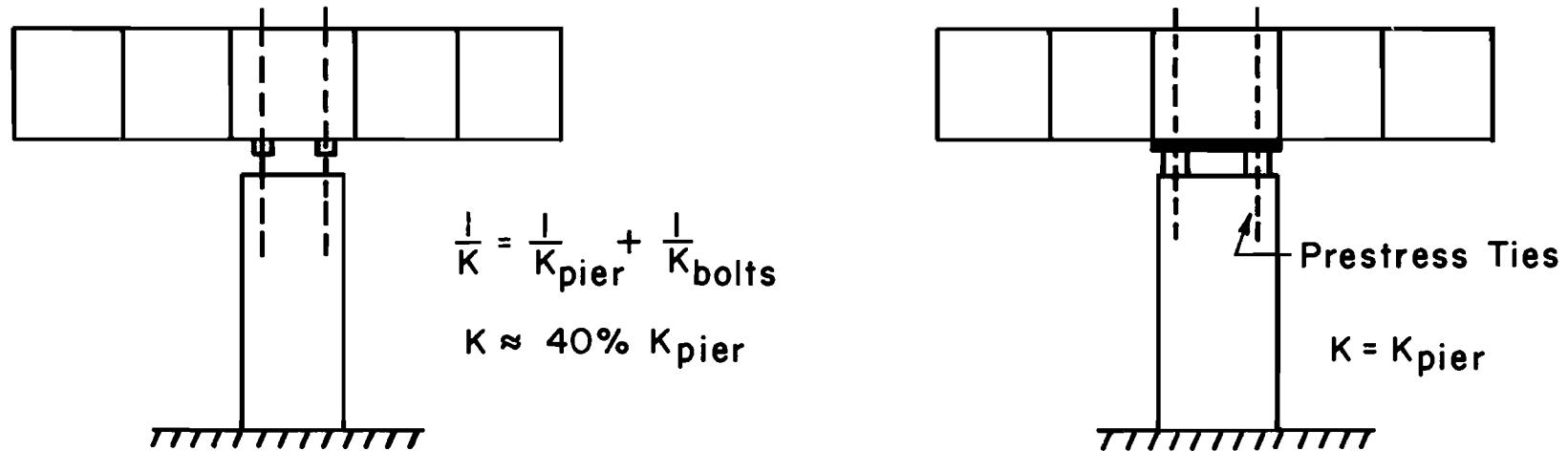


Fig. 2.6. Improved pier segment connection design

as noted in Fig. 1.4. The three basic reasons for changing the design were as follows:

- (1) Since the bridge was to be the first so constructed in the U.S., the bridge should be as simple to construct as feasible.
- (2) To better assure that a wide range of bridge contractors could erect the bridge, the individual segment weights and sizes should be as small as practical.
- (3) The multiple single cell approach allows easier expansion of the bridge at a later date.

The final design satisfied the three requirements quite well. The single cell boxes could be constructed with only one form and the difficulties in either casting or erection are minor. Also, the design was simple in appearance and should be appealing to both prospective contractors and the public alike. The weight of individual segments was reduced to less than 30 tons, which is not at all unreasonable for either erection or transportation. The size is not prohibitive either, since the width of 10 ft. and length of 28 ft. is not unusual. As far as widening is concerned, the only work required would be to erect a parallel structure with the same cross section and join it to the original structure with a longitudinal joint (Fig. 1.6).

Navigation clearances dictated a 200 ft. span over the heavily traveled Intracoastal Waterway in Corpus Christi, with approximately 100 ft. adjacent side spans (see Fig. 2.7). This was ideal for a segmental bridge erected in balanced cantilever. Precast, prestressed concrete was very desirable in this extremely corrosive environment. The bridge was designed for each stage of the segmental cantilevered construction, as well as for its completed state. With the span requirements fixed, and the decision to make the structure fully continuous over the 400 ft. length, the overall width of the superstructure was set as 56 ft., providing four 12 ft. traffic lanes, two 2 ft. shoulders, and a 2 ft. median. An 8 ft. depth was selected to give a span-to-depth ratio of 25. Smaller span-to-depth ratios have been suggested by some to provide less tendon area requirement and lessen congestion. Since the number of tendons is tied to the number of segments, and only the area of the

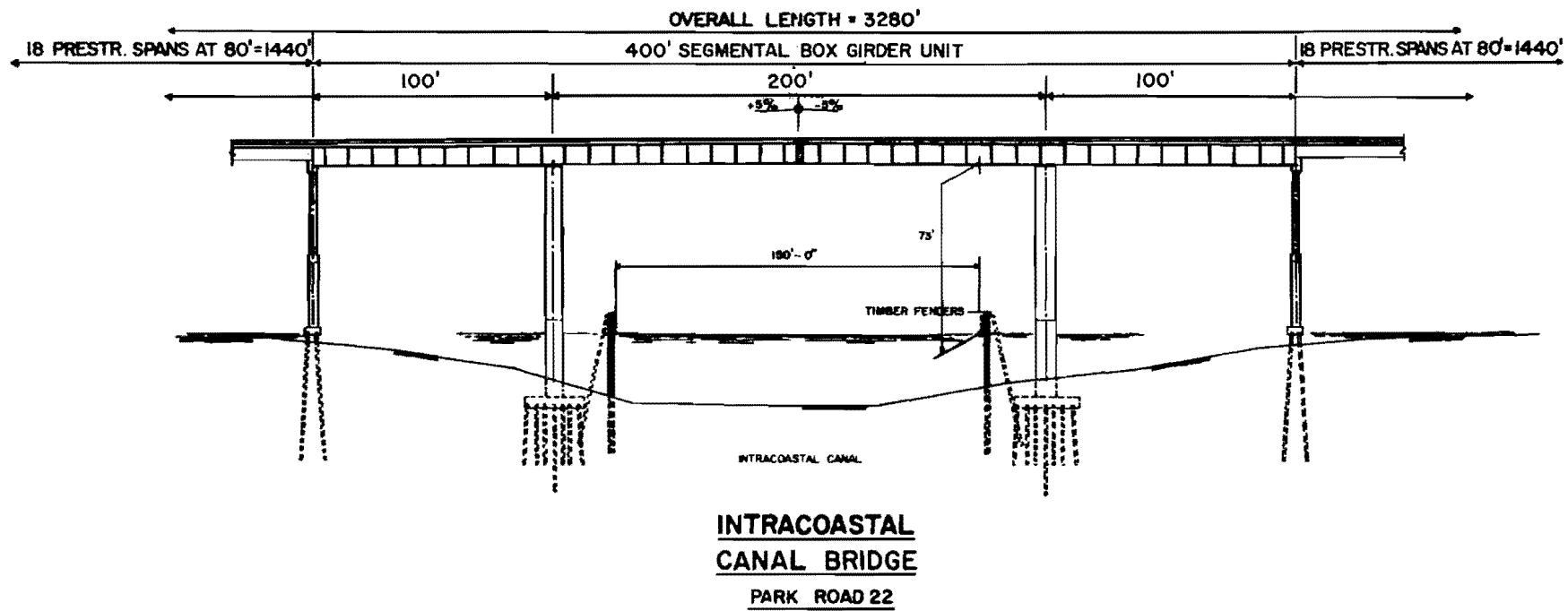


Fig. 2.7. Longitudinal section of Corpus Christi bridge

tendons would change, it did not appear that a deeper section would be beneficial. Dead load would increase and additional tendons would be required to carry it. Economic optimization studies indicated lesser span-to-depth ratios would increase costs.

Approximate cross section dimensions were developed for segments with a length of 10 ft. Much longer lengths complicated tendon layouts. The final cross section shown in Fig. 2.8 was selected. Maximum segment weight was approximately 30 tons, except for the four main pier units with diaphragms which weighed 50 tons. To minimize formwork and reinforcement variations, all 84 units required for the bridge had essentially the same cross section. The only major exceptions were the tendon profiles and the bottom flange thickness, which was increased near the main piers to provide for compressive forces. The tendon profile is shown in Fig. 2.9. Complete plans for the superstructure are included in Appendix A.

The segments were normally reinforced using concrete with minimum compressive strength of 6000 psi and Grade 60 reinforcement (Fig. 2.10). The tendons were designed to be 270 ksi steel with 162 ksi final effective design stress. The 2-ft. wide longitudinal joint was not to be cast until each half of the bridge had been completed. Neoprene pads were provided for pier supports after the completion of the bridge and provision of correct reactions. The optimization studies<sup>26</sup> showed that certain dimensions, such as the overall depth and the overhang portion of the top slab, can be varied considerably from optimum with only a small cost differential.

#### 2.4 Detailing

Based on the experience with the model and the prototype construction, a number of revisions in details are desirable.

2.4.1 General Section Dimensions and Layout. As mentioned previously, the minimum web thickness would be more appropriate as 14 in. The throat at the intersection of the web and top flange should be widened as indicated in Fig. 2.2. Some positive moment tendons should be anchored in the lower flange and some negative moment tendons in the top flange or side anchorages on the web.

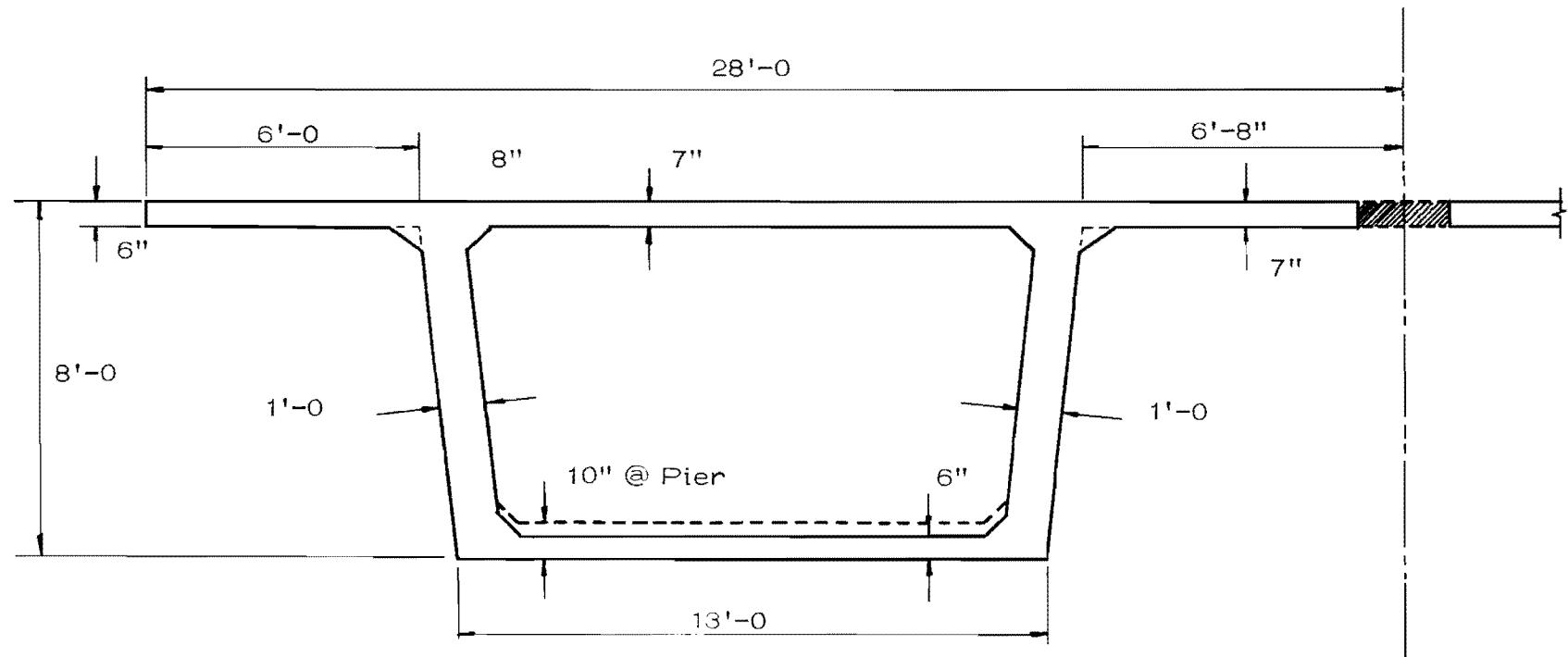


Fig. 2.8. Cross section of Corpus Christi bridge

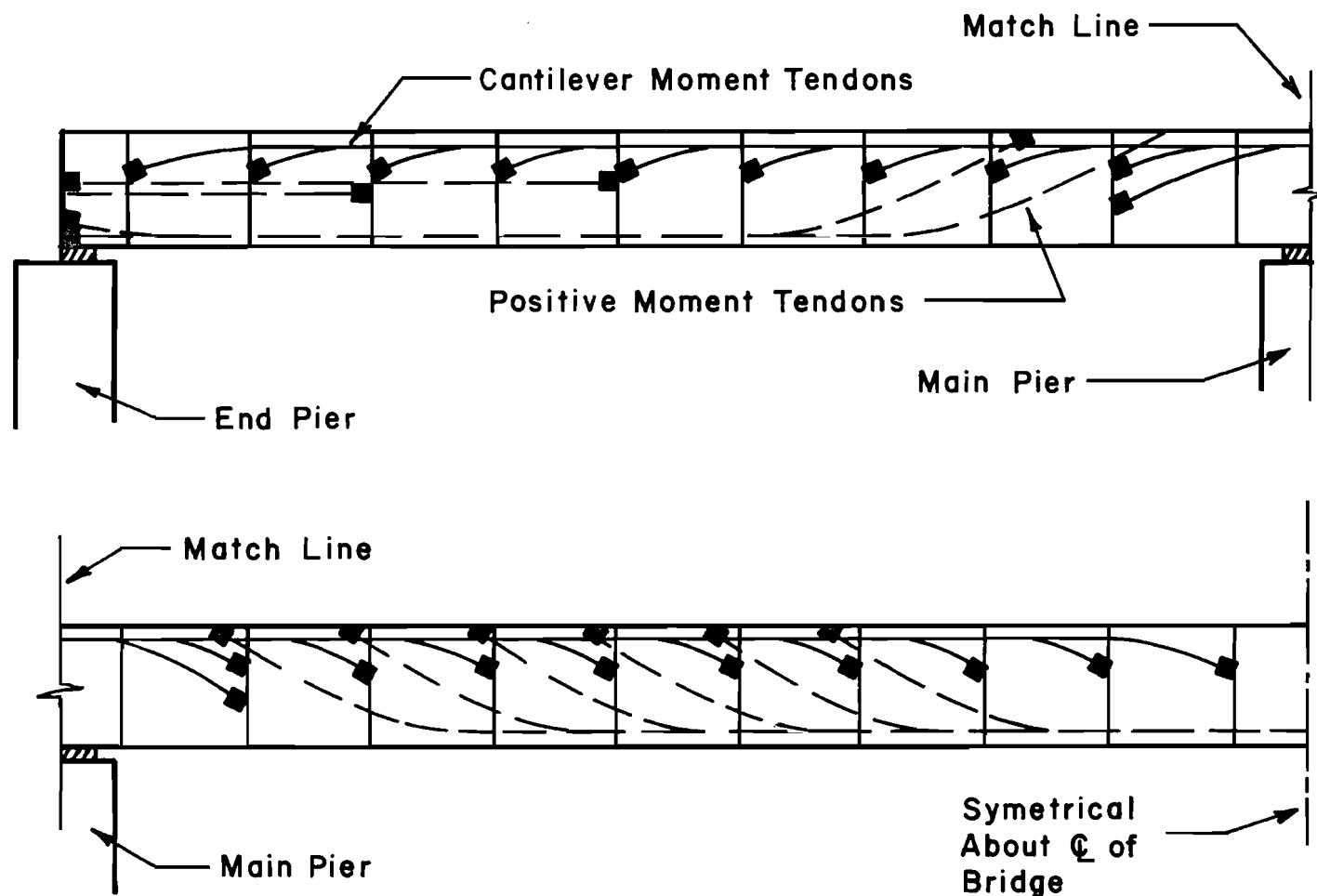


Fig. 2.9. Elevation showing tendon profile

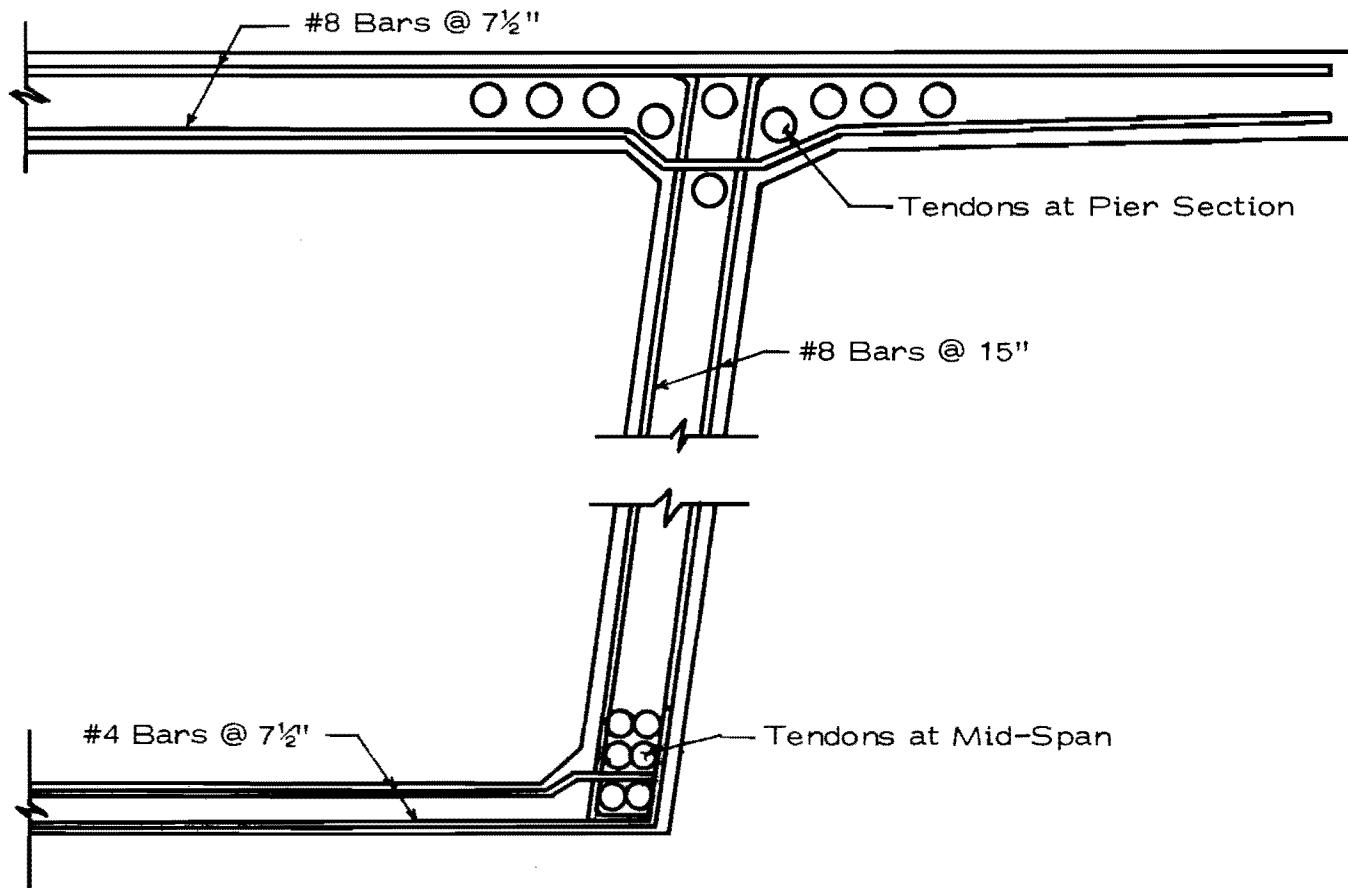


Fig. 2.10. Reinforcement and tendon details

All tendons must be enclosed within the reinforcement cages. Preliminary tests showed that tendons not contained within the main reinforcement cages would split fillets off at relatively low tendon forces.

The number of bars spliced and welded would be greatly reduced and the congestion greatly improved if transverse post-tensioning was used in the top slab. Vertical post-tensioning of the webs would be very useful in constant depth sections. In long span structures with constant depth, multiple shear keys should be considered.

In preparation and submission of working drawings, conventional reinforcement, anchorage fixtures, and tendon layouts should be shown on the same sheets. Numerous problems which arose during fabrication could have been avoided with better coordination at the working drawing stage.

**2.4.2 Reinforcement Details.** The contractor reported that the web bar diameter (#8 U @ 15 in.) was too large. He had continual difficulty with bends and tolerances. He suggested smaller, more closely spaced bars would be desirable. Alternatively, prestressed bars could be considered. He also felt that the reinforcement as detailed did not permit easy fabrication of complete cages. He suggested that stirrup bars should be lengthened and bent to make a more rigid connection with top mat steel and simplify cage tieing.

Since shear drops off rapidly away from the piers, lighter bars could be substituted in many of the units away from the piers.

A universal request was that the shear key reinforcement (Bars F on Sheet 2 in Appendix A) which were #8 be reduced in size to improve bond with the concrete and prevent spalling in the keys. The large reinforcement was particularly unsatisfactory where double shear keys or anchorages were used.

**2.4.3 Anchorage Zone Details.** As outlined in Chapter 5, considerable difficulty was experienced when inclined cracks developed along some tendon paths in the anchorage zone region during stressing. Note B on Sheet 2 of Appendix A states:

Reinforcing in precast units is subject to adjustments as required by post-tensioning systems. All required reinforcement at anchorages to resist bursting and compressive forces, as well as all adjustments in reinforcing from that shown on this sheet shall be submitted for approval to the engineer by the post-tensioning contractor.

This requirement is not meaningful in view of the present state of the art on anchorage zone design. The contractor should provide details plus calculations or test data to support these details. The revised specifications in Appendix B2 require that design calculations be provided for "supplementary reinforcement to resist tendon bursting, splitting, and spalling". This, coupled with the requirement that "prior to casting any permanent segments, the stressing system shall be successfully demonstrated on a segment designated on the plans," should greatly alleviate the problem. A simple mockup can find major flaws in a system. Always troubleshoot while change is practical before all units are cast.

**2.4.4 Erection Stresses.** Although the design criteria specified that no flexural tension stress would be allowed on the concrete during or subsequent to erection, the actual design violated this criterion. During the early cantilevering stages, checks of erection stresses and results from the model test indicated tensile stresses of small magnitude on the epoxy joints. Temporary lower flange prestressing was applied to control these stresses. This procedure was satisfactory in application. The tensile stresses vanish after a number of box units are placed.

## 2.5 Summary

There are many items for both designers and contractors to consider. Before establishing design criteria, designers should be informed of the erection equipment which contractors will likely have available and should weigh the particular problems associated with a proposed crossing. Undoubtedly, the environmental aspects are going to become more and more prevalent in bridge design, and designers must be able to furnish bridges which enhance rather than detract from the surrounding area. Pier design requires careful consideration and the possibility of temporary supports during cantilevering should be studied.

Every effort should be made to reduce costs through standardization and use of high quality materials. Standardization is one method of increasing productivity from laborers, who comprise an ever-increasing portion of the total cost of structures. Particular attention should be given to cross sections, tendons and their locations, ducts, and anchorages. Cross section design should enhance rather than restrict anchorage placement.

Objectives should be established in the beginning stages of design, and periodic checks made to ensure that the objectives are being satisfied. Pretests should be run on initial units to verify anchorage design.

Contractors should be made aware of the many and varied methods of construction available. Full use of existing equipment should be planned as an effort toward reducing total cost. Likewise, they should be encouraged to innovate and to not be afraid to depart from old established practices in order to meet the challenges of this new field of construction.

## C H A P T E R    3

### PRECASTING

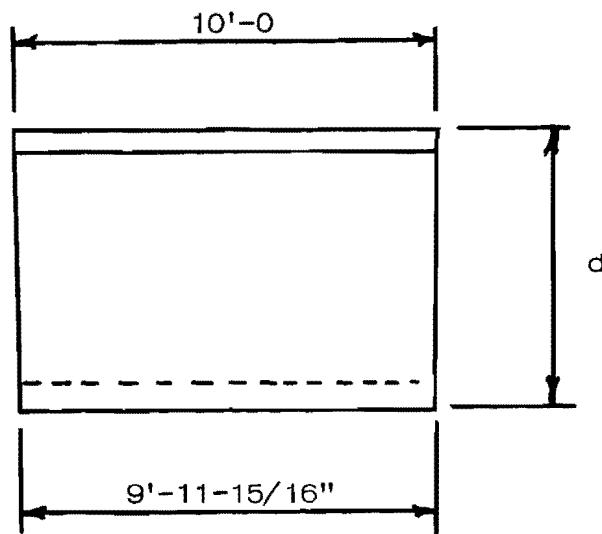
"To err is human, and so is trying  
to avoid correcting it."  
--R. Reycraft

The precasting procedures discussed in this chapter are generally applicable to bridges constructed either on falsework or in cantilever. In most cases, the casting procedures are identical for both types, but specific areas such as anchorages and tendon ducts tend to be simpler for the falsework type.

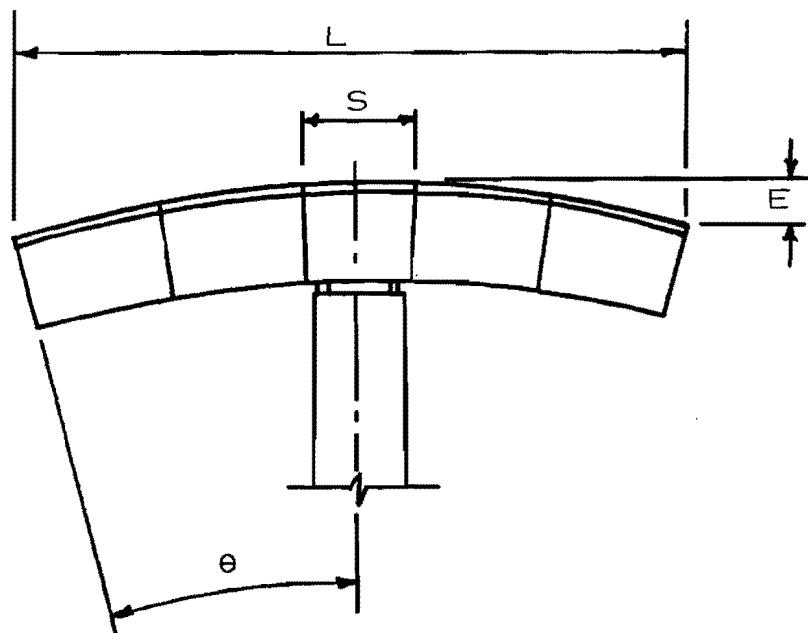
#### 3.1 Forms

Precast units of any type usually mean that dimensional tolerances are small; cantilevered segmental precasting calls for even better control of tolerances. This is because the units are placed end-to-end, and any error in dimensions of units near the pier are greatly multiplied by the lever arm of the cantilever. Because of the tolerance problems, the forms in which segments are cast and the manner in which they are used can mean either success or failure.

3.1.1 Alignment and Tolerances. If segments are cast separately, the tolerances required become extremely tight. The ACI Manual of Concrete Practice states that overall dimensions of precast concrete elements shall be as shown on the plans  $\pm 1/16$  in. per 10 ft. This was the general level of tolerance obtained in the Palacio del Rio Hotel box construction. However, this is not nearly accurate enough for the normal segmental bridge. Consider a form for a 10 ft. long segment being built to a tolerance of  $\pm 1/16$  in.; the form would be within tolerance if the top was 10 ft. long and the bottom of the form (soffit) was 9 ft.,  $11\frac{15}{16}$  in. [Fig. 3.1(a)]. Concrete cast in the form would take a slight wedge shape, however. When several units were assembled together end-to-end, they



(a)



(b)

Fig. 3.1. Error caused by imperfect segments

would begin to form an arc of a circle [Fig. 3.1(b)]. The same would be true if the top of the form were 1/16 in. longer than the bottom, but the arc would be in an upward direction. When the cantilever reached the centerline of the span between two piers, the possible error in alignment ( $E$ ) would be a maximum [Fig. 3.1(b)].

From the properties of a circle and assuming small deflection theory:

$$E = 144(d)(S)(\text{exsec } \theta)/t = 144(d)(S)(\sec \theta - 1)/t$$

where  $\theta = \frac{L}{2S} \tan^{-1} \frac{t}{12d} \approx 0.0417(L)(t)/(d)(S)$

$E$  = possible error, in.

$L$  = span, ft.

$t$  = overall dimensional tolerance per segment, in.

$d$  = girder depth, ft.

$S$  = segment length, ft.

Table 3.1 shows values of  $E$  for a  $(d)(S)$  product of 100 (girder depth 10 ft. and segment length 10 ft.). It can be seen that with a segment tolerance of 1/16 in. and a span of 200 ft., the possible error in vertical alignment is 3.13 in. Since the error varies inversely with the product of  $(d)(S)$ , the error possible for the Corpus Christi bridge would be:

$$\frac{(100)(3.13)}{(8)(10)} = 3.91 \text{ in.}$$

These errors quickly become unreasonable, as can be seen by the fact that a bridge similar to the Corpus Christi bridge ( $L/d = 25$ ) but with a span of 400 ft. could produce an error of:

$$\frac{(100)(12.56)}{(16)(10)} = 7.85 \text{ in.}$$

The table can be used similarly to find the horizontal alignment error induced by a segment being more narrow on one end than the other, in plan view.

There is a relatively easy solution to these types of tolerance problems, however, and that is to cast the segments end to end. This also seems to be the only practical solution. Assume that a segment has been cast with one face misaligned [Fig. 3.2(a)]. Then, if the next

TABLE 3.1. ALIGNMENT ERROR

Overall Tolerance, in.	Possible Alignment Error, E, in. *											
	Span, ft.											
	50	100	150	200	250	300	350	400	450	500	550	600
1/4	0.78	3.12	7.03	12.52	19.57	28.21	39.44	50.26	63.70	78.76	95.48	113.83
1/8	0.39	1.56	3.52	6.26	9.78	14.10	19.72	25.13	31.85	39.38	47.74	56.92
1/16	0.19	0.78	1.76	3.13	4.89	7.05	9.86	12.56	15.92	19.69	23.87	28.46
1/32	0.10	0.39	0.88	1.56	2.45	3.53	4.93	6.28	7.96	9.85	11.94	14.23
1/64	0.05	0.20	0.44	0.78	1.22	1.76	2.46	3.14	3.98	4.92	5.97	7.11

\* with a (d) (S) product of 100

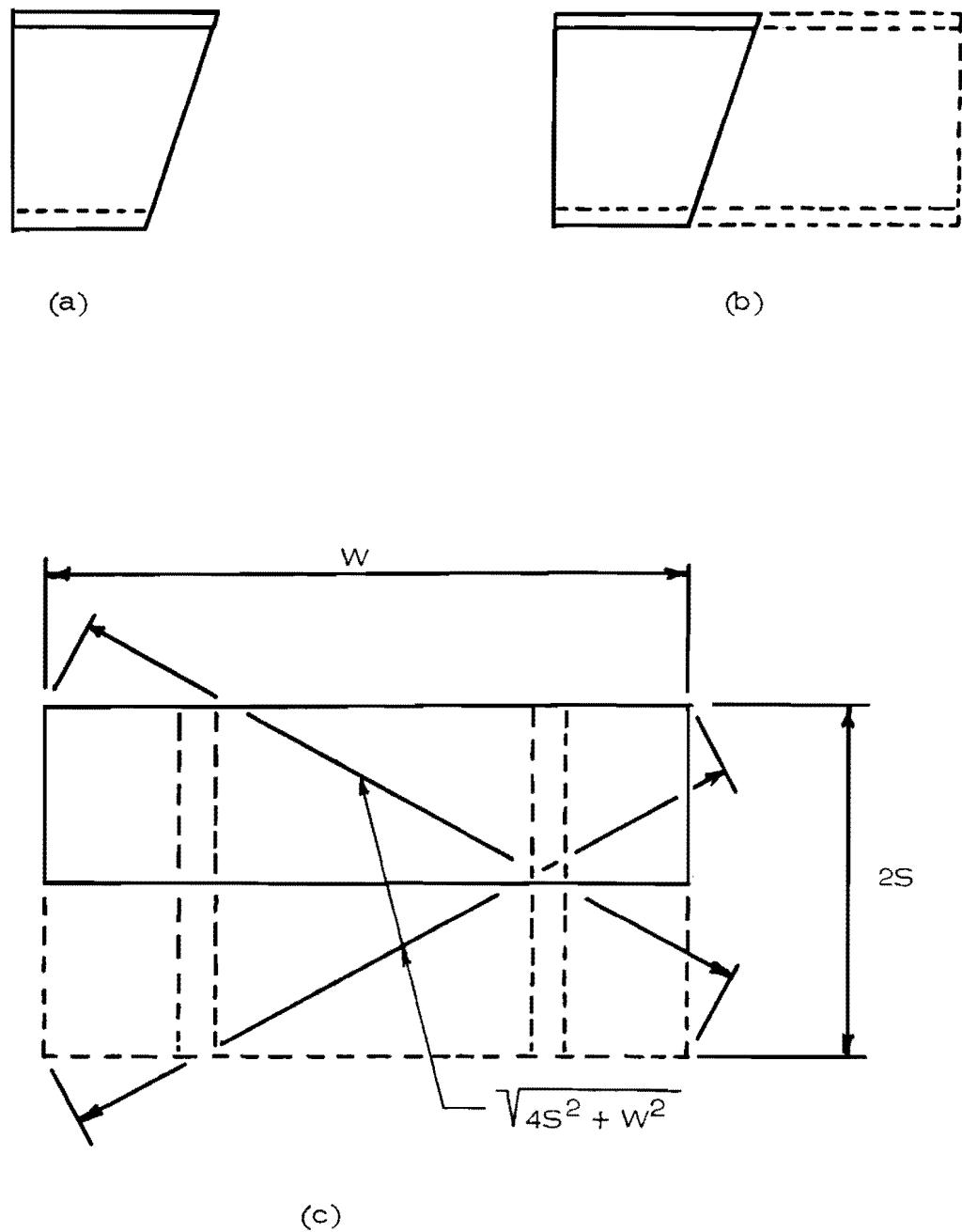


Fig. 3.2. Segment error corrections

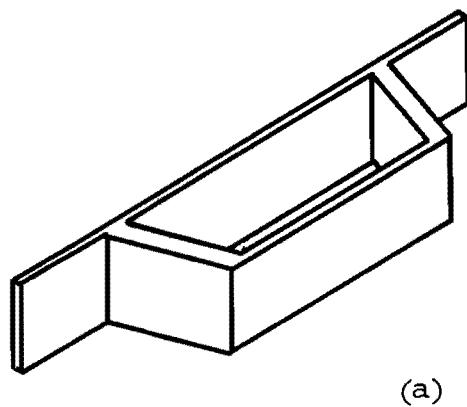
segment is cast directly against the first one, the misalignment can be totally corrected [Fig. 3.2(b)]. If the mistake had been left alone, a serious vertical alignment error would become evident during the erection stage. Similar problems in horizontal alignment can also be corrected in the casting stage. Computed deflections in the final structure can be corrected by building the correct offsetting deformations (camber for construction stresses) into the casting bench or soffit.

In practice, the first segment cast requires a bulkhead in each end of the form. From then on, the segment most recently cast is stripped and moved to one end of the form to act as a bulkhead for the succeeding segment. By simply keeping the soffits of the form and the previous segment properly aligned, vertical alignment problems become negligible. Horizontal alignment can be maintained by keeping the combined overall length of the two segments at the correct value, making sure that diagonal distances check [Fig. 3.3(c)]. Some contractors have assembled the soffit forms for the entire bridge before commencing casting operations; this assures proper alignment with a minimum of corrections involved. Others have used partial soffits with interim alignments. With high levels of control, a single segment soffit or "casting bench" can be used.

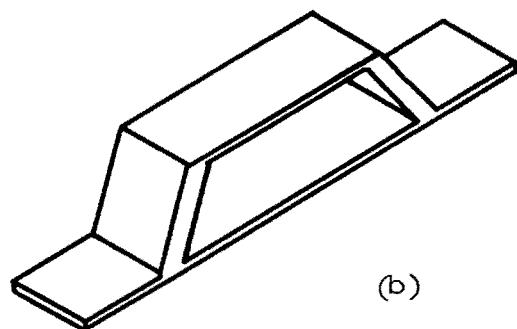
3.1.2 Methods of Forming. Since the segmental bridges are to be precast, maximum use and efficiency should be obtained from any forming system utilized. Steel forms will probably be specified for use on all outside surfaces, as is usual for precast items. Some European contractors have successfully cast with steel outside forms and wooden forms for the interior portion. There are several items to consider before selecting formwork, e.g., orientation of forms, method of forming joining faces, manner of casting, curing time.

There are basically three ways to orient forms for casting the segments; the top slab may be vertical [Fig. 3.3(a)], inverted [Fig. 3.3(b)], or upright [Fig. 3.3(c)]. Each method has been tried at one time or another and each has its own advantages and disadvantages.

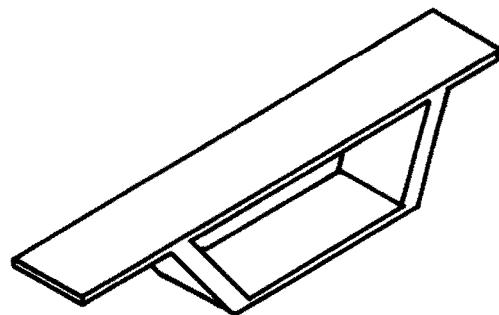
The main advantage of casting with the top deck in a vertical position is easier compaction of the concrete being placed. However, the



(a)



(b)



(c)

Fig. 3.3. Orientation of forms

difference is small since the maximum aggregate size will probably be 3/4 in. Another small item in its favor is that it saves space when casting end-to-end. More problems seem to be created, though, since there is a more involved process of stripping and preparing for placement. This method also calls for a maximum amount of formwork, per segment, meaning a higher initial form cost and higher labor costs associated with forming.

Casting with the top deck in an inverted position required less area of form than the previous position, and requires less finishing than the upright position. But the problem of turning the segment over after casting becomes prevalent. Also, since air bubbles in the concrete would tend to rise vertically, the finish of the concrete on the exterior web walls would be inferior.

The third method proposed, with the deck in an upright position seems to be the most advantageous. This position requires less area of forms than any other especially when the top side of the lower slab is left open (Fig. 3.4). Tests run on forms for the model showed that the forms for the interior web wall of the prototype would only have to extend about 1 ft. out over the top of the lower slab. This type detail was successfully used. Some of the European bridges have been cast in similar forms. The open area of the top slab is not at all objectionable since that is the traffic wearing surface, and it should have a roughened finish whether an asphalt overlay will be applied or not. The concrete finish produced by this method will be the best of all methods, since any air bubbles will go unnoticed on the interior wall of the web.

All of the above discussion would apply equally as well to a multicell box girder, the only difference being the number of web walls.

As far as the method of forming the joining faces goes, the system of casting end-to-end seems to be the only practical means; especially since tolerance control is so critical. Casting end-to-end does require that a bond breaker be applied to the previously cast piece. Casting end-to-end also assures that segments will match perfectly when joined together in the field. Matching faces are very important because of the

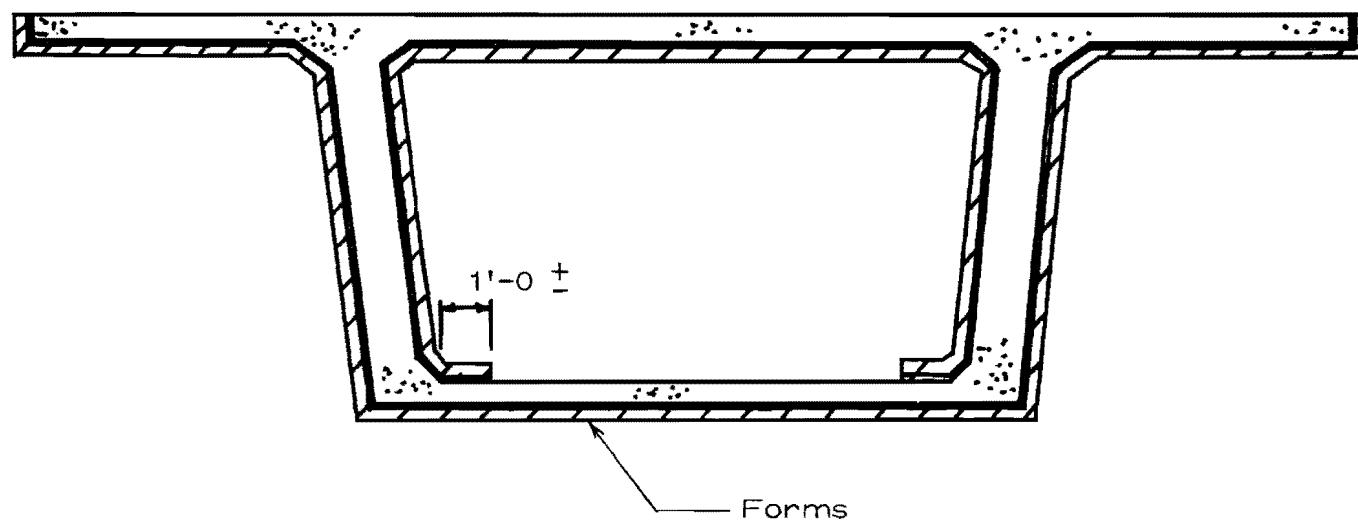


Fig. 3.4. Forms for box girder

extremely high forces produced by the post-tensioning strands, over three million pounds in each web of the pier segment for the Corpus Christi bridge. It is easily seen how stress concentrations resulting from improperly matching faces can produce spalling.

The manner in which a contractor plans to cast segments influences the formwork, especially if he were to do something such as cast the lower slab separately and later join it to the webs with a cold joint. Some bridges have been built with a hollow overhang being cast with a cold joint, but problems associated with it have prevented a repetition of the method.

The curing time of the concrete can be an important factor in the casting operation. The setting time of the concrete has often been increased by either chemicals or heat. For the economical mass production of segments, reduced setting time is essential; as in other areas, the equipment available can be most influential. Heating of the aggregate and mixing water seems to be common during winter months. Other forms of heat used involve heating the forms with either steam or electricity, and some contractors have placed electrical heating wires within the concrete. Some of the latest bridges have been built with segments produced at the rate of one segment every four hours from the same form, using a four or five man crew and heating the forms.

In addition to the forms being well engineered, substantially constructed units which are manufactured to close tolerances, the basic casting bed must be capable of maintaining desired tolerances. When first constructed in Corpus Christi, the casting bed for the segmental forms was erected on beams supported on a fill, as shown in Fig. 3.5(a). The bed consolidated when concrete was placed in the metal units and considerable difficulty in maintaining tolerances ensued. This was finally overcome by corrective measures. The forms used were substantial and versatile, as shown in Fig. 3.5(b) and (c).

3.1.3 Flexibility. Even though the industrialization of segmental bridges stresses standardization, it should be remembered that

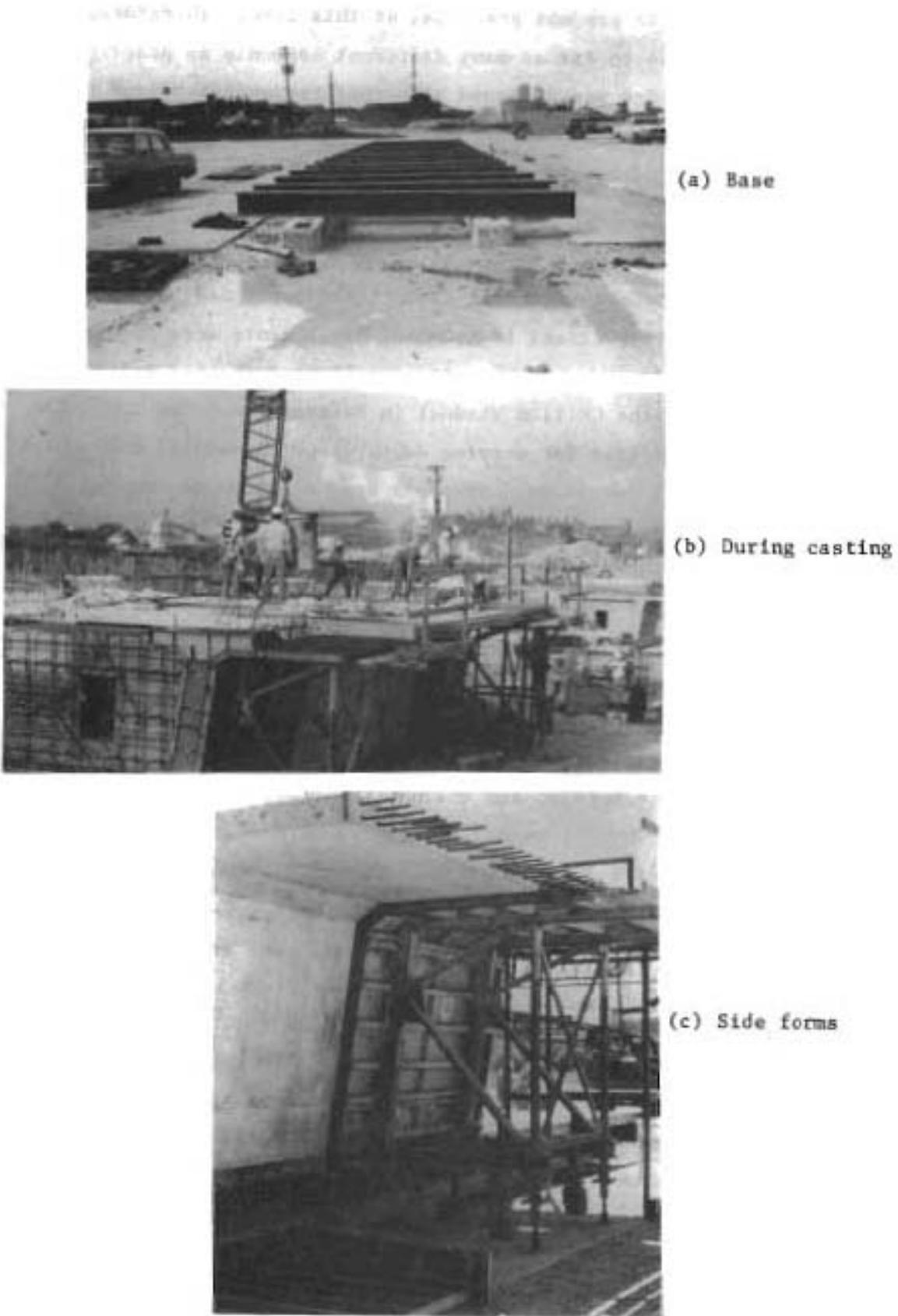


Fig. 3.5. Prototype forms

completely standard units are not practical at this time. Therefore, forms should be designed to fit as many different segments as practical. The Corpus Christi bridge was designed such that the entire bridge could be cast with one form. That form only had to adjust to the different duct locations in the end faces, and allow for the casting of diaphragms in the pier and abutment segments. Form cost amounted to \$0.95 per sq. ft. of superstructure deck area.

Forms could be designed to fit several similarly designed bridges, but would probably be impractical if too many adjustments were required. Even forms for one bridge can become quite complicated, as was pointed out by the construction of the Chillon Viaduct in Switzerland. The forms for those segments had to adjust for varying depth, superelevation, horizontal curvature, and vertical curvature. Segments were cast end-to-end and jacks adjusting the forms helped produce accuracies of 0.02 in. This case is an example of the "casting bench". Such forms are best used on large projects. Efforts should always be directed toward the use of forms made as simple as possible.

3.1.4 Required Number. The number of forms required for casting will vary with the time schedule of the bridge, the contractor's equipment, the cross section of the bridge, and the number of segments to be cast. Even though the Corpus Christi bridge could have been cast in one form, two forms with 90 ft. of soffit were used. The main reason for this was so that the segments in opposite cantilevers of the bridge would be relatively near the same age, since the creep characteristics will affect the deflection at midspan. There are 84 segments in the structure and a casting time of eight to ten months would probably have been required if only one form were available. Although the difference in strength would probably be minor, there would be a considerable difference in the age and creep coefficients between the first and last segments, especially considering the relatively young ages. Two forms would much be preferred, since the casting time would be cut almost in half, even with the use of the same crew. One of the forms could be used to cast segments for one side of a cantilever, while the other was being used to cast segments for the balancing side of the cantilever.

Four forms would have been even better, since the forming time would again be cut in half; the cost per segment would be nearly constant. With this many forms, the contractor could reduce the difference in age of concrete to a near minimum.

Of course, the cross section of the bridge may designate some minimum number of forms, although the segments will hopefully be as nearly identical as possible. Also, extremely large number of segments will require more than the normal number of forms or change to a "casting bench". Variable depth members are better suited to "casting bench" production.

3.1.5 Stripping. As was established in Sec. 3.1.1, it is imperative that the segments be cast end-to-end, unless extremely tight tolerance control can be maintained. With this in mind, it should be clear that some means of breaking the bond between segments must be provided. Commercial bond breakers appeared adequate in the model program when used properly. For instance, in the casting phase of the model program it had been found that grout leakage between the form and the previously cast segment was enough to provide considerable bond between the segments. Hydraulic rams had to be used in some cases to break the bond. As a result, the bond breaker normally applied only to the joining faces was later applied to all surfaces which were likely to come in contact with the segment being cast. Although the magnitude of the problem is related only to modeling, it does point out one of the problems that can develop with this type of casting.

Standard shear keys are provided in the web of each section; another pair of smaller alignment keys is also provided in the top and bottom slab (Fig. 3.6). These act as a means of making sure the segments are aligned in the field exactly as they were in the forms. These keys can present a problem if the segments which have been cast against each other are not stripped by pulling them straight apart in the longitudinal direction. As was theorized and later shown to be true in both model and prototype any rotation of the segments during stripping will cause the more massive male shear key to fracture the relative thin wall of the corresponding female pocket (Fig. 3.6, Sec. BB). Due to the vulnerability

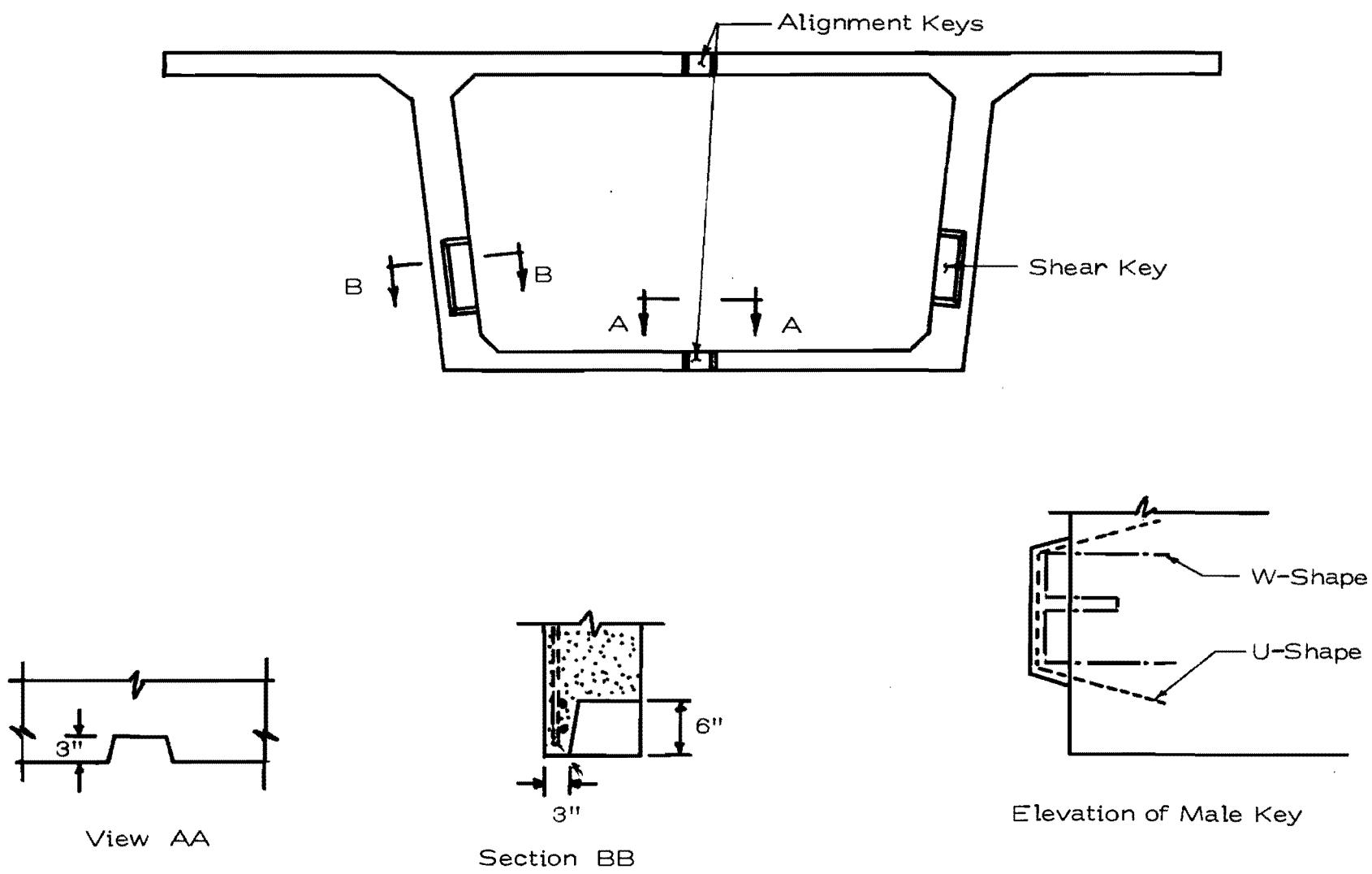


Fig. 3.6. Shear and alignment keys

of this thin wall, extra care must be taken with design of reinforcement in that area. Both horizontal and vertical bars were placed in the wall and anchored in the surrounding mass of the web wall.

A commercial bond breaker often used in lift slab operations was first used in the prototype construction. This proved unsuitable. As shown in Fig. 3.7, while the commercial bond breaking agent did break bond, it did not provide cushioning and lubrication when segments were separated. Considerable spalling occurred at keys and corners. During the early stages of construction, a foreign visitor suggested the use of flax soap and talc. The precaster switched to a mixture of

5 pts. flax soap (manufactured by Sherman Williams Co.)  
1 pt. talc (tire talc, baby powder or cosmetic base or foundation)

The formulation was approximate. They added talc to the soap and mixed until the soap turned whitish and stringy. This mixture was used in all the key ways and corners. Plain flax soap was rubbed by hand on all flat surfaces, since the talc produced a roughened surface. In some cases, duct tape was also used as a cushion in all key way reentrant corners. These changes, coupled with more careful separation procedures, using rams at the centroids, eliminated the spalling problem. The soap must be removed prior to application of epoxy. This was done by a light sandblasting of the mating surfaces.

Methods for forming ducts are discussed in Sec. 3.3.2; other than those items specifically covered in that section, there are no problems peculiar to segmental casting.

### 3.2 Cages

As for most precasting operations, the cages for the segments can be fabricated in mass production fashion. This can mean substantial savings to contractors, since more productivity can be attained.

3.2.1 Fabrication. If designers and detailers have properly planned the reinforcement details, fabrication of the rebar cages can be



(a) Top flange near alignment key



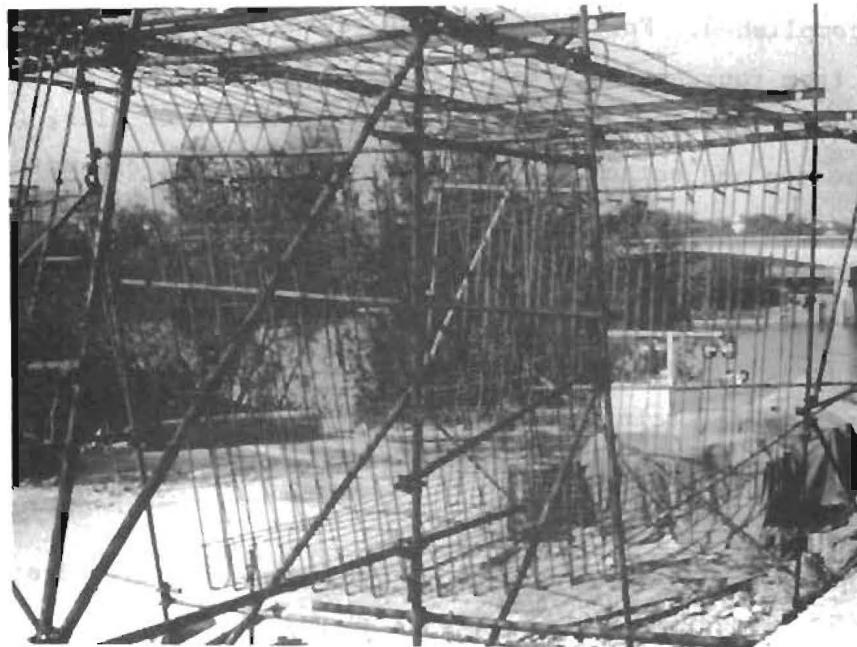
(b) Shear key

Fig. 3.7. Localized spalling

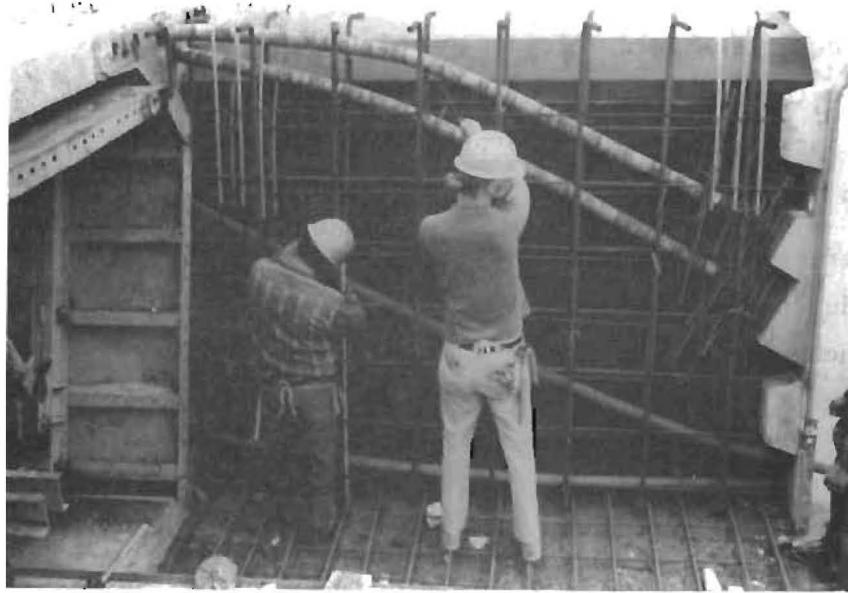
rapidly accomplished. For instance, the cages for the model bridge were fabricated from four basic rectangular mats: (1) one mat for the top of the upper slab, (2) one mat for the bottom of the upper slab, (3) one mat for both the top and bottom of the lower slab, and (4) one mat for either the left or right web. The reinforcement cages for the Chillon Viaduct in Switzerland were fabricated similarly. Since there were almost 1400 segments in the entire bridge, the cages were built with the use of jigs. Vertical jigs were made for each of the different mats required and intersections of the reinforcing bars were simply tack-welded. The mats went into the forms as separate units with only a minimum of tieing required in the forms.

Other contractors have used jigs to tie an entire cage together before placing it in the form, as shown in Fig. 3.8(a); this was the procedure established for the model bridge. In contrast, as shown in Fig. 3.8(b), all reinforcement for the Corpus Christi bridge was assembled in the forms. This put cage assembly in the critical path and increased the complexity of inspection. It made it particularly difficult to adjust bars when top steel was placed after side and top forms were moved into position. Details should allow cage and tendon assembly at a different work station than the forms. As in other cases, the type of fabrication the contractor uses is largely dependent on the number of segments to be cast and on the available labor and equipment.

There are certain areas of the cages which will require special attention during the fabrication stage. These are items such as the tendons, anchorages, and shear keys. The path of the tendons is the most complicated of the problem areas, since the pattern is established as a design criterion and cannot be more than slightly altered. At points where tendons have to turn, the reinforcing bars must be routed around the ducts to give priority to the ducts. However, care must be taken to assure that the routing of the reinforcement provides adequate residual strength. As a result of tests made with model segments, one important rule to remember is to always keep the tendons between planes of reinforcement. During the preliminary model studies, in one area it seemed to be easier to locate one of the tendons in the concrete fillet at the



(a) Reinforcement cage preassembled in France



(b) Reinforcement assembled in forms at Corpus Christi

Fig. 3.8. Reinforcement fabrication

intersection of the top slab and the web wall, below the reinforcement in the upper slab. However, when the tendon was stressed, the fillet fractured, leaving the bare tendon exposed. After that, it became a strict practice to always keep the tendons surrounded by reinforcement.

Anchorage are more of a normal item, since they amount to hardly more than a typical blockout. But the designers must be careful in routing reinforcement around anchorages to make certain that tendon paths are not intercepted and that flexural stresses can be satisfied. Additional reinforcement is usually required immediately under the bearing plate or bearing cone of anchors to take care of the localized stresses (see Chapter 5).

Reinforcement of the wall of the female shear key has already been emphasized; the male key also requires additional reinforcement. The reinforcement for the model bridge consisted of a pair of parallel bars (equivalent to #6 bars, Grade 60) which have a W shape and follow the outline of the key [Fig. 3.6(a)]. This shape provides twice the shear area of U-shaped bars used in some of the earlier test segments. Earlier segments also had the shear key centered in the web, with a thin wall on either side of it. But the fracturing problem mentioned earlier caused the change to the layout shown in Fig. 3.6, which provides a somewhat thicker wall. The shear key is also easier to form in this manner, since one side of the blockout fits flush against the inside face of the web wall form.

**3.2.2 Placement.** The placement of the cage into the forms will be dependent on the contractor's planned use of the formwork; two methods were mentioned in the previous section. The main difference between reinforcing cages for segmental precasting and more typical precasting is the requirement for tendon ducts. There are many ways by which the passages may be formed; several are covered in Sec. 3.3.2. In any case, care must be taken to assure that the ducts are securely attached to the cage in their proper locations.

### 3.3 Special Items

There are several things which seem to stand out as most important in the precasting of units for segmental construction. They are the result of experience gained in planning and construction of the model and prototype bridges and by contractors in other areas of the world.

3.3.1 Tendon Layout. The various tendon patterns required for cantilevered bridges are the major items which prevent complete standardization of the more typical segmental bridges. Since the tendon pattern cannot be identical throughout the length of a bridge, the changes in the pattern should be minimized and simplified as much as possible. A simple scheme for the path of each tendon was developed for the Corpus Christi bridge. Figure 3.9 shows an elevation and plan view of the typical tendon patterns in one web of the bridge. The basic approach was to provide a standard curvature pattern in the turndown area of the negative moment tendons by letting each tendon begin its turndown from a standard drop position. The path from the drop position to the anchorage is standard in all segments and consists of a left-hand and a right-hand curvature. Since the tendon must actually move laterally from the outboard drop position to the center of the web and simultaneously downward to the anchorage which is located at the centroid of the segment, the path reduces to a pattern similar to a helix. Since left and right drop positions are standard, the process of bringing tendons to one of the drop positions is simply to alternate from one side of the web to the other, as shown in the plan view of Fig. 3.9. This means that all tendons anchored in even numbered segments are located on one side of the web and tendons anchored in odd numbered segments are located on the other side of the web. Obviously, the problem becomes more complicated as the number of tendons increases. The Corpus Christi bridge had a total of only 52 tendons (13 per web) at the piers; however, a similar tendon pattern works just as well for the Johanniter Bridge over the Rhine in Switzerland, which has a 450 ft. main span requiring a total of 152 tendons at each pier.

The curve from the drop position to the tendon anchorage could be further simplified if the drop position could be located in the center of the web. However, for most cantilevered bridges with positive moment

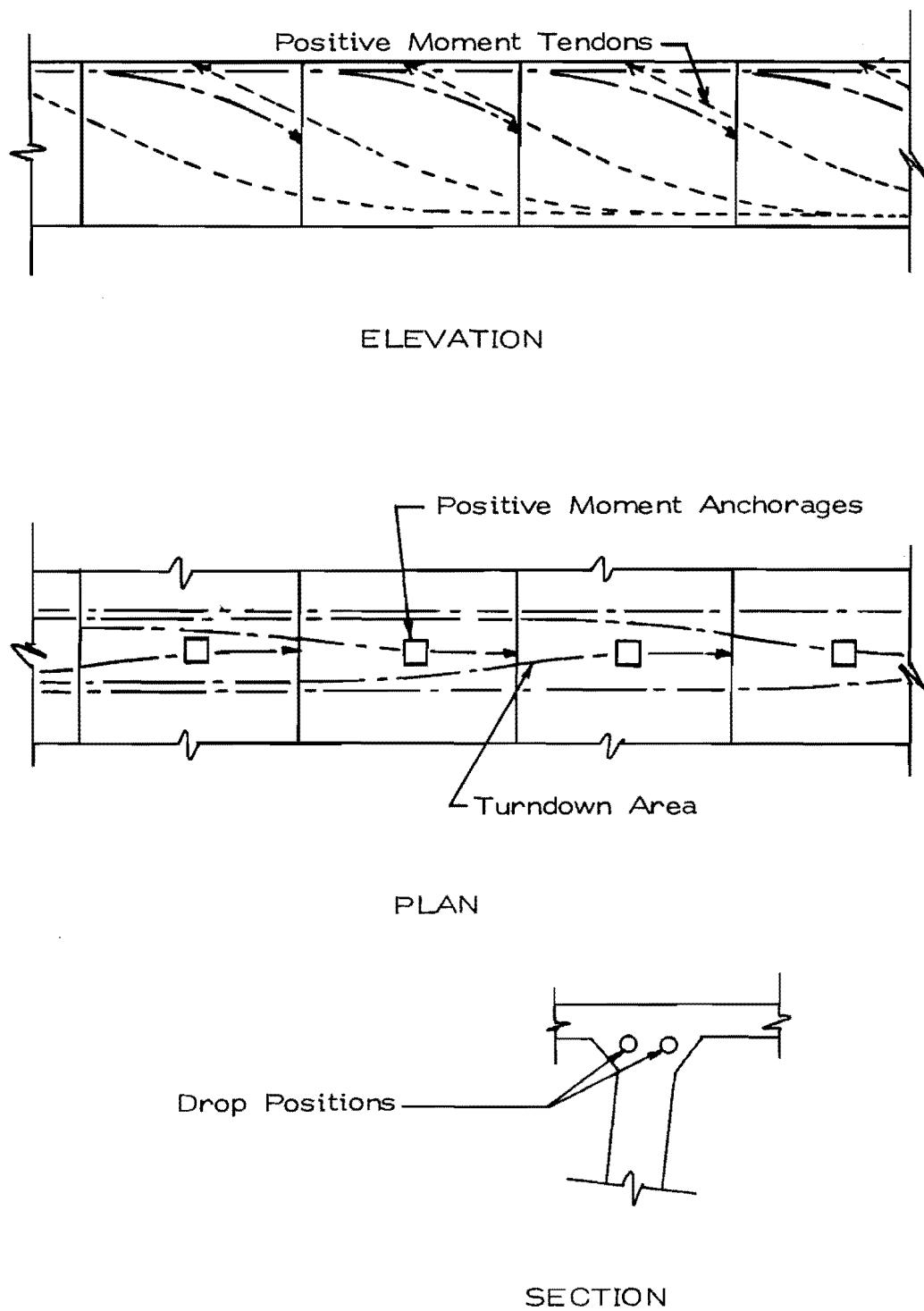


Fig. 3.9. Tendon profiles

cables anchored in the top slab this is not possible, since the negative and positive moment cables would have to cross in the web. When ducts cross, either the radius of curvature at the intersection is reduced below the 25 ft. minimum or the ducts block the passage of concrete, leaving a void under the ducts; this was proven during the model study. To prevent the ducts from crossing in the web, the tendon pattern used in the Corpus Christi bridge was designed such that the negative moment cables were routed laterally around the positive moment anchorages before dropping down into the web (Fig. 3.9). This problem is reduced if a substantial number of positive moment cables are anchored in the lower flange.

Similar to the turndown curve for the negative moment cables, a standard curve for the positive moment cables can be easily developed. For the Corpus Christi bridge, the positive moment cables must turn upward from their horizontal position at the bottom of the web, move laterally slightly, and proceed in a near straight line through the center of the shear key to the anchorage, located in the top slab over the center of the web.

Similar cantilevered bridges can be planned in a like manner; planners should always bear in mind the economic advantages of standardization. Although bridges to be constructed on falsework have different patterns, the tendon paths can be standardized even more easily.

3.3.2 Ducts. Ducts, which house the tendons, must be accurately located during the casting phase. As stated earlier, there are several materials available for forming the ducts, steel being the most common. It should be reiterated that serious problems can result from the use of plain concrete ducts, i.e., the forming of ducts with a material that is removed after casting is completed. Some European contractors have experienced cases where tendons were able to cross freely from one duct to another during installation due to voids between concrete ducts. Problems such as this could severely hamper field operations, since repairs would have to be made to realign the ducts. Inspection procedures could be established to assure that concrete ducts were properly located and adequately separated from one another, but the cost of repair would be prohibitive.

Steel ducts can be relatively inexpensive but must be strong. The primary requirements are that they maintain a prescribed path, withstand the hydrostatic pressure of the concrete during casting, and serve as a guide for the tendon during its installation. The duct must not unravel while the tendons are being pulled through. The main problems associated with ducts are their proper location and their alignment. A standardized tendon pattern can help simplify the problem of duct location, leaving alignment as the principle obstacle.

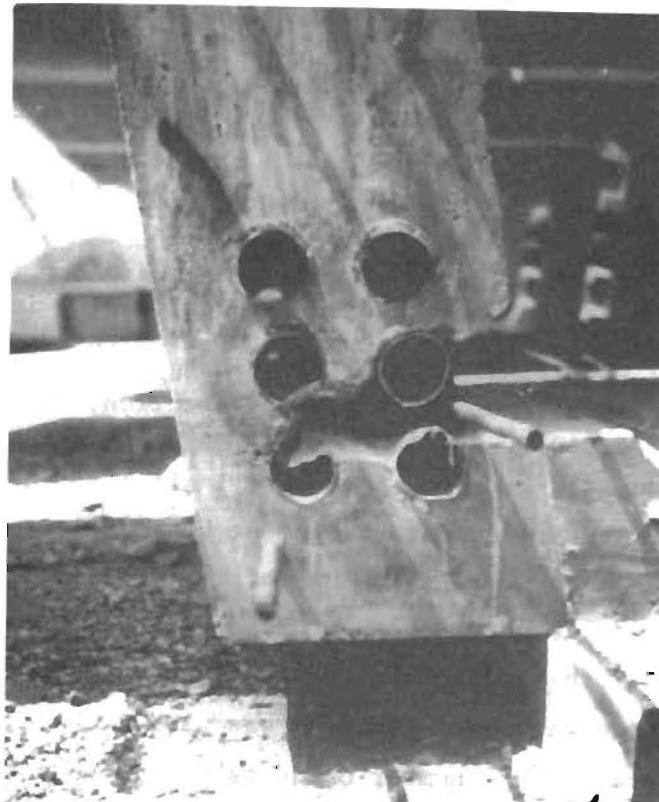
Since the basic principles of segmental construction involve a process of threading both positive and negative moment tendons through the entire length of a bridge, the ducts through which these tendons pass must be in near perfect alignment at all joining faces. The technique of casting segments end to end simplifies tolerance problems immeasurably; even if ducts are slightly out of position they can be in perfect alignment by casting segments end-to-end. The process of aligning the ducts is simple:

- (1) The first segment is cast with all ducts properly positioned and secured to bulkheads at each end of the form.
- (2) The segment is stripped and positioned to form one bulkhead for the next pour.
- (3) The reinforcing cage is placed in the form with ducts tied in place.
- (4) The ducts in the cage are aligned with the ducts in the previous segment and the bulkhead and an internal device is installed to keep the ducts aligned. Probably the simplest device would be a rubber tube extending completely through the duct which can be inflated with air to form a rigid connection. After casting, the rubber tubes can be deflated and removed [see Fig. 3.10(a)].

This method assures that tendons will align in the field, especially since the shear keys and alignment keys will reposition the segments exactly as they were cast. Use of the inflated ducts will stiffen the tendon ducts and keep them from getting misshaped during casting. Figure 3.10(b) indicates a series of damaged ducts in the Corpus Christi bridge where no stiffening was used. Casting in this manner also acts as a check on the number of ducts provided in each segment, since every duct must be continuous across the joining faces.



(a) Inflated duct stiffening and alignment tubes  
extending through far end of form



(b) Defective positive moment tendon ducts

Fig. 3.10. Tendon ducts

There are various methods for positioning ducts in the reinforcing cages and checking the accuracy of their placement. At least one contractor has made use of a frame inside which the reinforcing cage is placed; the frame contains a series of small lights laid out in the prescribed paths of different tendons. When the path for a particular tendon is required, the corresponding circuit switch is thrown, lighting the bulbs for that tendon. Workers can then position the ducts visually simply by covering the lights with duct material. The ducts for the model segments were positioned similarly, by using a piece of wood with the various tendon paths painted on the surface. Mechanical means could also be used to hold ducts in the required position while they are being secured to the reinforcing cage.

Most designers will probably require the use of bonded tendons, meaning that all tendons will have to be grouted after they have been stressed. Depending upon the design, the grouting could be done either immediately after each tendon has been stressed or after the entire structure has been completed. Because of the corrosion concern, the Corpus Christi bridge was grouted within 48 hrs. after stressing. In either case, provision must be made for placing the grout and for venting the ducts. The various manufacturers have different means of placing grout in the ducts, each having its own merits. No matter what method is used, proper planning must be done to ensure that grouting connections will be accessible at the time required and to ensure that no grout tubes will interfere with tendon ducts. It is just as important to ensure that provision has been made to prevent leakage of grout from one duct into another duct; this problem has occurred in other bridges.

3.3.3 Stressing Systems. There are many different methods in use for post-tensioning; all of them serve the common purpose of providing proper stress control within the concrete. Any type of tendon should work equally as well, whether it is composed of strands, wires, or bars, if it is a properly designed application for that type; the basic requirements are that the tendon conform to the design path and design strength. The type tendon path used in precast segmental units favors wires or strands. The anchorages serve to transfer the stress at the end of the tendon into

the concrete; this is done by direct bearing, by wedge action, or by a combination of the two. In each case, the anchorage is used to grip the end of the tendon and distribute the load as uniformly as possible into the concrete; extra reinforcement is usually provided in the area immediately surrounding the anchorage to prevent bursting. The curved tendons add radial stresses to normal bursting stresses. Bearing stresses and anchorage zone design are discussed in Chapter 5.

Each manufacturer of post-tensioning equipment usually has a special jacking system. Hydraulic jacks are used to stress the tendons to their required load and bring them to bear on the anchorage. The stressing operation involves:

- (1) Initially tensioning the tendon to remove all "slack" from the tendons; this is usually on the order of 10 percent of the tendon strength.
- (2) Stressing the tendon to its proof load, or to about 80 percent of its strength in order to help overcome friction.
- (3) Relieving the stress to a predetermined value, usually to about 70 percent of the ultimate strength, and setting the anchor. Double checks are made to ensure that the proper loads are in the tendons by checking the pressure readings from the hydraulic jacks against measured elongation of the tendons.

Stressing can usually be done from either end of the tendons. In long ducts, it is required that stressing be done from each end due to frictional losses. Friction along the length of the cable comes from two sources: (1) wall contact in sections not perfectly straight (known as wobble effect), and (2) wall contact in curved sections. The wobble effect varies with the rigidity of the duct material, with the smooth steel ducts being most favorable. Naturally, the friction coefficient varies with the type of duct material, type of steel tendon, and lubricant. Normal friction losses may be on the order of 20 to 30 percent of the force in the tendon. Friction will be accentuated if tendon ducts do not join smoothly at joints and if the ducts are dented or deformed during concrete placement.

**3.3.4 Special Segments.** In each bridge, there will probably be some segments which require some special attention. In the Corpus Christi bridge, the pier segments, end segments, and closure segments required a special modification of one form or another. The pier segments required a

diaphragm, which is typical for all long span bridges; this generally meant that the interior of the forms would have to be adaptable to allow for diaphragms. The end segments were different in the fact that they were only 5 ft. long and the fact that they required anchorages in different locations, as shown in Fig. 1.3. The segments near the piers also required extra anchorages, but this was considered a minor difference. The closure segments were cast only 4 ft., 3 in. long, in order to have an 18 in. cast-in-place joint at the centerline. In addition, the longitudinal reinforcement protruded from the face of the segment to help form the splice. Similar changes would probably be typical, but were not considered as unusual requirements.

### 3.4 Casting Procedure

There are a few items peculiar to segmental precasting which need to be pointed out.

3.4.1 Sequence. The order in which segments are cast can affect the economics of segmental construction. Since casting segments end-to-end has been shown to be the most ideal method of casting, it follows that segments should be cast in a sequence relative to their final position in the bridge structure; furthermore, they should be cast in the order of their erection.

Using the 90 ft. of soffit, as shown in Fig. 3.11, the contractor cast the pier segment centered on the soffit and then cast sequentially the next four segments on each side of the pier segment. The outermost segments were then moved in adjacent to the center, used as an end form and three additional segments on each side were cast. This process was then repeated to complete casting of a quadrant. Precise alignment was ensured by use of permanent bench marks. Elevation readings were taken on 1/32 in. rule at 5 ft. intervals along both sides of the members. Using the modified line procedure allowed work to proceed on several units at once. Continuity on several units could easily be checked before removal from the casting line.

3.4.2 Concrete. Experience at Corpus Christi indicated that a reasonably high cement factor (6.5 scy) using Type III cement with a

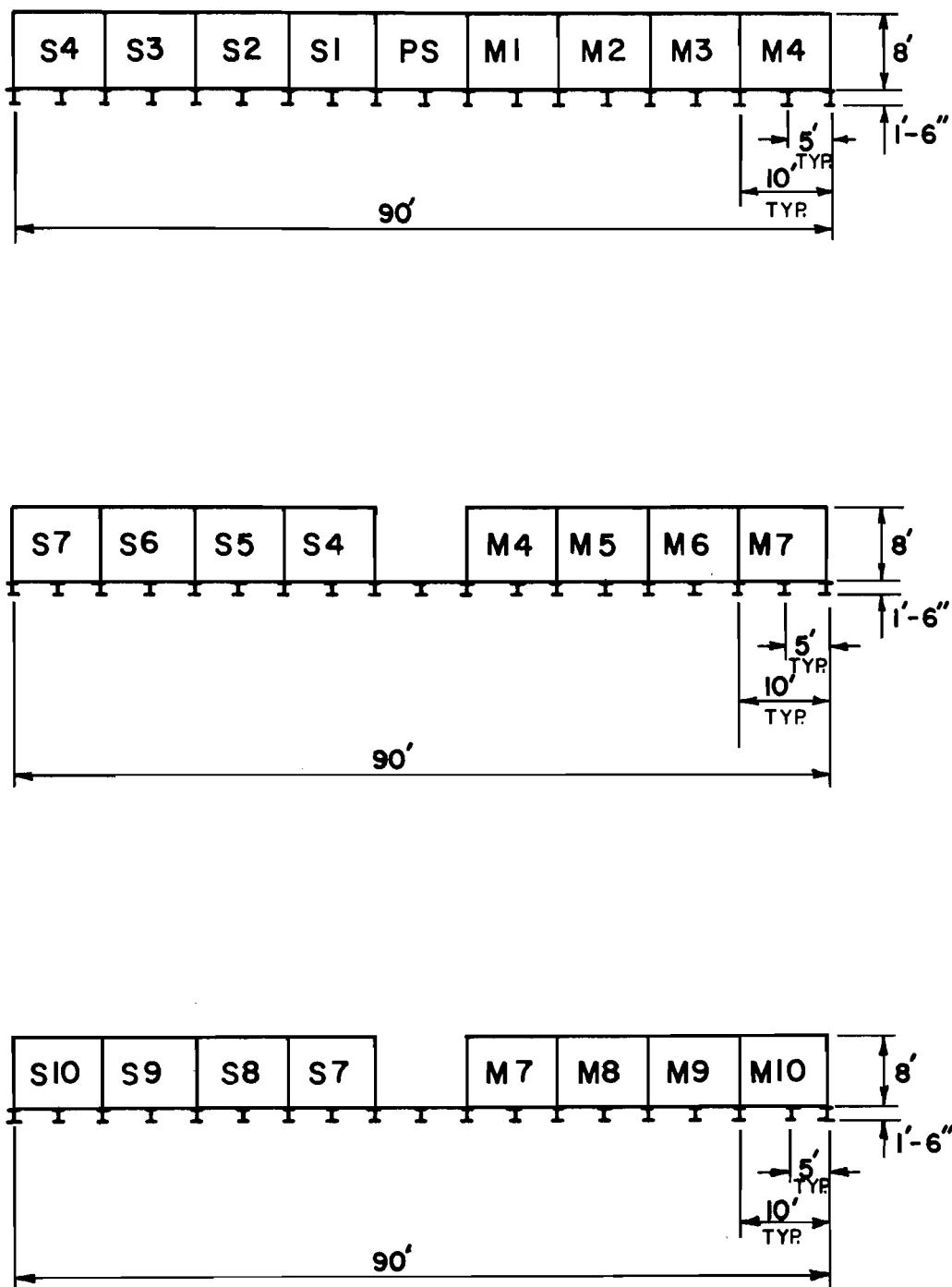


Fig. 3.11. Casting bed schedule

water-cement ratio of 0.52 produced a good quality, workable concrete. Slump ranged from an average 4 in. to 5 in. maximum. Compressive strength was specified as 6000 psi, but actually attained 4000 psi in 1 day and 7000 to 8000 psi in 14 to 28 days. A set retarder cement disperser admixture was used to improve workability. The contractor recommends use of an aggregate which does not tend to splinter on contact as some of the crushed limestone and sandstone do. He suggested river gravel would be more desirable, since their experience indicates it tends to fracture less when spalling occurs. They recommend a full 5 in. slump for web placement.

Industrialized, precast construction also demands availability of quality concrete. The concrete source should be near the casting bed so that unnecessary delay is avoided.

3.4.3 Inspection. The inspection procedure is really no different from that required for other types of precasting. There are a few important items which may need special attention, however.

- (1) The forms should be properly aligned with the previously cast segment to ensure that alignment errors will be minimized at the construction stage.
- (2) The tendon layout should be checked to make certain all ducts are properly located, accurately aligned, and adequately secured.
- (3) All ducts should be sealed to prevent the entrance of grout during the casting operation.
- (4) All required anchorages should be rigidly secured in their proper locations, along with any other necessary reinforcement.

A detailed check list suggested by the Chief Inspector at Corpus Christi is included as Appendix C.

3.4.4 Concrete Placement. It was shown in Sec. 3.1.2 that the most advantageous method of forming is to cast each segment with the top deck in its normal position. As shown in Fig. 3.4, the top of the bottom slab can be left open. The following sequence of concrete placement makes use of that fact:

- (1) Place concrete in the lower slab and begin finishing if desired.
- (2) Place concrete in the webs until they are completely filled. The concrete in the bottom slab should prevent loss of concrete from the web.

## (3) Place concrete for the top slab.

Special care should be taken during placement of concrete in the webs, since passage of the concrete is impaired by the tendon ducts, anchorages, and shear key blockouts. The Texas Highway Department required external vibration after the contractor had difficulty getting proper consolidation of web concrete. The contractor claimed that external vibration is undesirable because it overvibrates webs and causes concrete to move underneath positive moment tendon areas. This difficulty can be minimized by thicker webs and relocation of some tendons.

Methods of accelerating the hardening of the concrete were mentioned in Sec. 3.1.2. Particular methods employed are usually the decision of the contractor. As in so many other instances, the available equipment may have the greatest bearing on that decision.

3.4.5 Storage. Forms can usually be stripped the day following casting. The segments should be given a permanent identification mark and moved to a storage area (see Fig. 3.12). The segments will usually be quite strong and may be stacked to any necessary height in order to reduce the area of the storage yard. Some contractors have used cranes mounted on circular tracks to move the segments around a circular storage area. Others have used overhead cranes to move segments from the casting area to the storage area. Still others have moved them with vehicles designed especially for use with the segments.

Storage problems are not difficult, since special treatment of the segments is not required. Proper planning can make maximum use of storage areas with a minimum of handling required.

Match cast units should not have minor damage on their matching faces repaired. Any such damage should be cleaned and not repaired. Attempts to dress the faces may result in alteration of the segment match. The damaged area can be filled with epoxy bonding material at the time of erection.



(a) Interior



(b) Exterior

Fig. 3.12. High quality precast segments awaiting erection

### 3.5 Summary

It has been established that the method of casting end-to-end is almost essential in order to provide sufficiently accurate segments for normal bridges. Tolerances are too tight for other methods. Forms should be designed to cast segments in their usual orientation in order to get the most from the forms. The number of forms required will vary with the particular conditions. Two form units with 90 ft. of soffit were used on the Corpus Christi bridge.

The tendon layout for each bridge will differ somewhat, but the patterns can usually be simplified into a small number of variables. Proper planning is required to prevent conflicts in the paths of the tendons. Careful attention must be paid to areas where tendons are located in close proximity. The designer must provide adequate web and flange area to permit concrete placement around ducts and anchors.

In order to ensure that tendons can be installed in ducts during the erection stage, the ducts must be accurately aligned at all joining faces. This can best be accomplished by casting segments end-to-end and rigidly connecting duct tubes between the segment being cast and the previously cast segment. Internal support should be provided to prevent pinching or denting of ducts during concrete placement. Care must also be taken in placing the grout tubes.

The whole precasting operation can easily be made into an assembly line type of production. It might break into the following areas:

- (1) Reinforcing cage area: reinforcing bars could be fabricated into mats and then into complete cages.
- (2) Duct area: tendon ducts, grout tubes, and anchorages could be installed in the cages.
- (3) Casting area: all casting done in whatever forms were available.
- (4) Concrete area: all concrete required for casting could be supplied from large stockpiles.
- (5) Storage area: storage of the segments could be provided until they were required at the erection site.

The problems associated with precasting box girder segments may seem difficult, but simple solutions have been found to work satisfactorily.

The Corpus Christi bridge is an example of a simple operation completed successfully by local forces with little outside guidance.

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## C H A P T E R      4

### ERECTION

#### Murphy's Law

- (1) Nothing is as easy as it looks.
- (2) Everything takes longer than you think
- (3) If anything can go wrong, it will

The specific type of erection equipment used on any particular project will depend largely on the type of available equipment and the design criteria utilized for segmental analysis. Many methods which have been used successfully are discussed in this chapter along with details of the Corpus Christi erection procedures.

#### 4.1 Methods of Erection

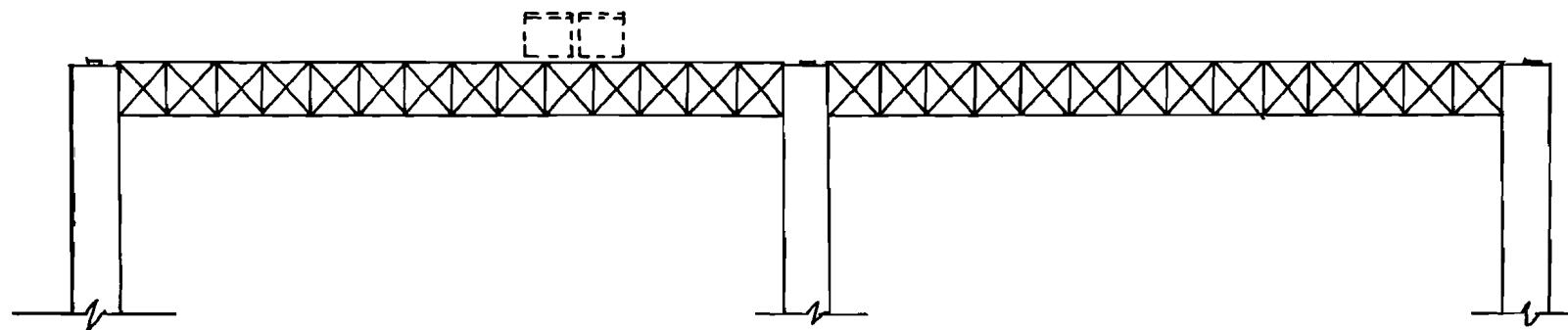
The two basic methods of erecting a segmental bridge are on falsework and in cantilever.

4.1.1 Erection on Falsework. The use of falsework for the erection of a bridge constructed by the segmental method depends chiefly on the location of the bridge and the cost of the required falsework. The cost of the precast units will be usually slightly less than for those cast for cantilever construction, with the difference being the cost of installing the extra ducts and anchorages required in cantilever construction.

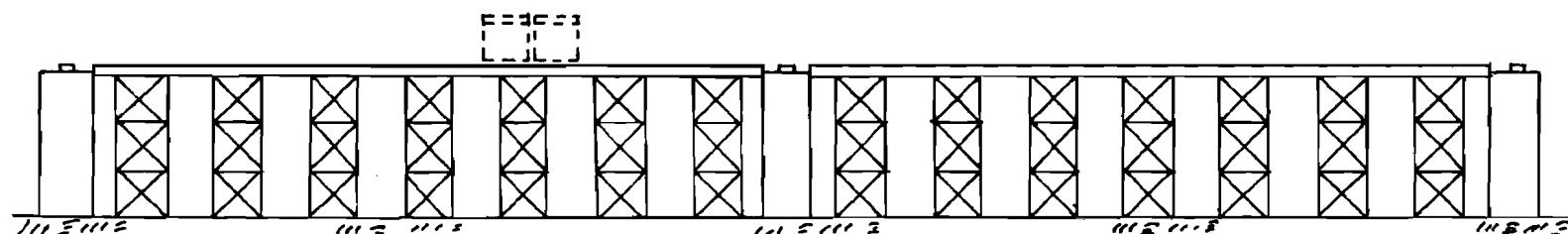
The location of the proposed bridge is quite an important consideration since there are many instances in which the vertical clearance requirements are such that the use of falsework would be prohibited. Locations such as crossings of highways and waterways where the flow of traffic must be maintained are examples; other places might be the construction of bridges or walkways over buildings or other structures. Remote locations can prevent the use of falsework because of their inaccessibility,

although such locations sometimes require the use of cast-in-place box girders for the same reason.

If the location and clearances provided indicate that falsework may be used, the next most important consideration is the cost involved. This would include the cost of falsework materials and equipment, the cost of transportation, the cost of erection, and the cost of removal. The initial investment in material and equipment can be quite varied, since so much depends on the location and on the load requirements.<sup>30</sup> A bridge constructed over water or a deep gorge would mean that a false-work truss would be required to span from one pier to another. Other areas may permit the use of something such as scaffolding (Fig. 4.1). Trusses could be designed to support the segments either from below or from above. It is easily seen that the trusses could become quite expensive with the long spans encountered in most of the more common segmental bridges. The use of supports such as scaffolding is relatively inexpensive. If the cost of the support system is low, the use of bridges constructed on this type of falsework may be warranted. Construction on falsework can provide important economies in permanent pier cost. A major consideration is the time involved in the erection, since some type of cast-in-place joints are often required. Epoxy joints are generally not feasible unless coupled with temporary stressing, since deflection in the falsework could produce joint movement prior to stressing of the tendons. The curing time requirements for the cast-in-place joints make falsework construction slower than cantilever construction. Muller<sup>32</sup> has reported on successful use of a combination of limited cantilevering and extensive falsework being used on freeway construction. Extensive use of temporary tendons is made to support the unit before stressing. It has been reported that the contractor on the 2230 ft. long, 250 ft. maximum span Napa River Bridge in California elected to build the cast-in-place segmental superstructure on falsework rather than in cantilever. The exercise of this option and the extremely low bid prices as compared to the engineer's estimates on both steel and conventional concrete box girders indicate the competitiveness of the construction method.



Trusses



Scaffolding

Fig. 4.1. Types of falsework

If a complete study of costs involved indicate that falsework may be utilized on a particular bridge, the general procedure for erection would be as follows:

- (1) Erect the piers and falsework.
- (2) Place each precast segment on the falsework in its approximate final position; segments may be placed in any order. A distance of 3 to 4 in. is usually maintained between segments if cast-in-place joints are to be used. Temporary stressing should be used if epoxy joints are to be used.
- (3) After all segments have been correctly positioned, insert the post-tensioning tendons in the ducts provided in the segments.
- (4) Make any adjustments necessary for final alignment. Provide interior and exterior forms at each joint and cast-in-place joints required.
- (5) After the joint curing has been completed, stress the tendons as required by the design.
- (6) When the stressing has been completed, the falsework may be removed, leaving a completed bridge structure. As in the case of bridges constructed by the cantilever method, jacks may be utilized to produce correct reactions at the piers and abutments.<sup>33</sup>

Since the assembly of segments on falsework is a relatively simple procedure, more detail is unnecessary. Other items associated with the falsework method of segmental construction are similar to or identical to the cantilever method and will be discussed in this chapter.

4.1.2 Erection in Cantilever. Due to the high expense associated with falsework and their application in long span situations, the majority of precast segmental bridges have been constructed by the cantilever procedure. This is a more involved procedure and requires more attention to detail.

## 4.2 Cantilever Erection Procedures

Some of the details which need to be considered are the pier segments, the methods of placing segments, the jointing problems, the stressing procedures, the closure joints, and the final completion of the structure.

4.2.1 Pier Segments. It was pointed out in Chapters 1 and 2 that there must be sufficient moment capacity in the piers or temporary support system to counter possible unbalanced cantilever moments applied during

erection either intentionally or accidentally. The magnitude of the moment depends on the length of the cantilever arms, the size of the segments, and the support system being used. Even though some temporary support system may be planned for the erection stage, there still needs to be moment capacity in the pier itself because temporary supports cannot become effective until at least one segment has been placed on each side of the pier segment.

There are several methods which have been used to provide the temporary moment capacity required between the pier and the pier segment. Usually the segment is secured to the pier by stressed rods or strands and rests on bearing pads which have been set to the prescribed grade. In some cases, the rods or strands extend through the section and are stressed from the top deck; in others, they are anchored in the diaphragm or the lower slab. Each case constitutes a different design, since the possible moment varies depending on the segment weights, cantilever length, and any temporary supports used.

Care must be taken to ensure that the pier segment is set to the correct grade, since any error in the setting of the pier segment is greatly multiplied at the end of the cantilever arm. In most cases, the deck of the individual segments is set at some slope, if for no other reason than to provide drainage. The soffit is really the control plane, and the deck is usually a troweled surface. Because it is difficult to measure elevations on the soffit, carefully controlled reference points must be built into the top slab of each segment; this would usually be done during the casting operation. For instance, metal inserts could be cast in the slab and adjusted so that they were parallel to the soffit. As the erection progressed, these inserts could be used to determine whether or not the bridge was properly aligned. If the pier segment is to rest on bearing pads during the construction, the elevation of the inserts could be used to check the accuracy of the elevation of the bearing pads and any necessary corrections could be made with the use of metal shims installed between the bearing pad and the soffit of the pier segment.

If the tiedowns are left ungrouted, they may remain stressed while the structure is being leveled. Since the adjustments will undoubtedly be small, the tiedowns will readily extend the required distance.<sup>40</sup>

On the Corpus Christi bridge the design engineers specified another type of connection to eliminate the need for any shims during the construction stage and to allow easy alignment of the units. The design utilized twelve bolts cast vertically in each pier for anchoring the pier segments (Fig. 4.2). The pier segment was, in effect, clamped between two rows of plates on the 3-in. diameter bolts and secured to the pier throughout the construction stage. After stressing was completed, hydraulic jacks were inserted to carry the load of the structure while the lower set of nuts was loosened; the permanent bearing pads were then installed and the box girders lowered onto the pads. Before the asphalt overlay was applied, the bolts were cut off just below the surface of the top slab. The bolt stubs were left in place to act as dowels. If the bolts were not required as dowels, they could best be utilized by designing them so that the bolts could be recovered from the pier after completion of the structure; holes remaining in the pier could be grouted to prevent entry of moisture. As detailed in Report 121-5, this connection was not completely satisfactory. While adequate in strength, its stiffness was low, resulting in large construction deflections under unbalanced loads. Stressing of the connection and placement of bearing blocks is recommended in subsequent bridges.

The weight of the pier units dictated that they be placed first at Corpus Christi. Figure 4.3 shows all pier units in place.

**4.2.2 Placement of Segments.** Various methods for moving the segments from the casting area to their final positions in the bridge structure were treated in Chapter 2. These reduce to two categories: (1) utilization of the structure, and (2) nonutilization of the structure.

Since segmental bridges should all be designed to withstand the weight of one segment on the end of a cantilever arm, they can usually be utilized for the placement of segments if equipment involved is not unreasonably heavy. For example, the Corpus Christi bridge could safely support the weight of a full segment in addition to a 25 kips allowance for lifting equipment. This opens the possibility of using a hoist

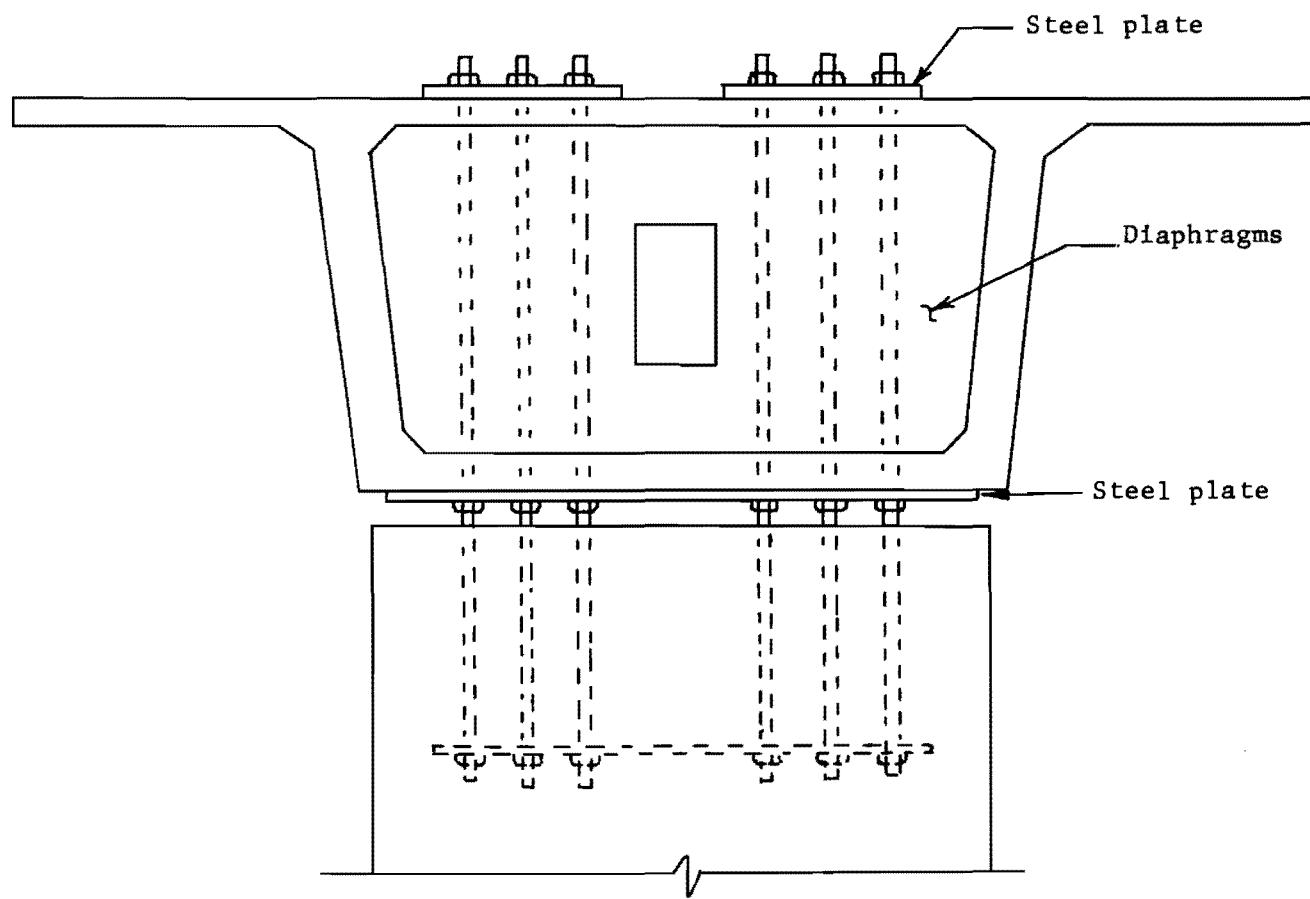


Fig. 4.2. Pier connection for Corpus Christi bridge



Fig. 4.3. Early erection stage at Corpus Christi  
(Note pier segments in place in  
background)

mounted on the cantilever. Similar devices have been used on many bridges in other countries.<sup>31</sup> They have ranged from simple structures with a pair of small winches to much more sophisticated mechanisms with integral work platforms (Fig. 4.4). The basic requirement of the hoists is to lift segments from a position below the bridge to the final position in the structure and to hold them in that position until stressing has been completed. The hoists usually have to be tied down to the structure, which calls for the casting of holes or tiedown devices in the segments. Counterweights add an unnecessary additional moment which has to be provided for in the design. If a temporary stressing mechanism is used to hold segments while they are being stressed, one hoist capable of traveling from one end of the structure to the other is adequate; otherwise, one is required at each end of the cantilever.

Temporary holding equipment is used quite frequently when lifts are made from below. It serves the general purpose of holding a segment while the tendons are being placed and stressed. The devices can be quite simple and they allow more efficient use of the lifting cranes. If the bridge can carry the unbalanced moment of a single segment, one crane can be used to place a segment at one end of a cantilever arm, have it temporarily supported on the structure, release and lift the balancing segment to the other end of the cantilever. Typically, the equipment consists of some stressing equipment which pulls the segment being placed against the completed portion; the shear keys carry a portion of the vertical load (Fig. 4.5).

Normally, mobile cranes cannot be utilized on the cantilever during erection, since they are considerably heavier than the hoist frames. They can be used on the adjacent half of the bridge if it has been completed. The most common use of mobile cranes will be to lift segments from the ground to a temporary holding device located on the structure. The Corpus Christi bridge was erected with the use of a single mobile crane mounted on a barge. However, problems associated with barges led to development of several innovations. Because of the weight of the segments, spuds generally would have to be reset for each lift to make sure that the barge was sufficiently stable while positioning the segments.

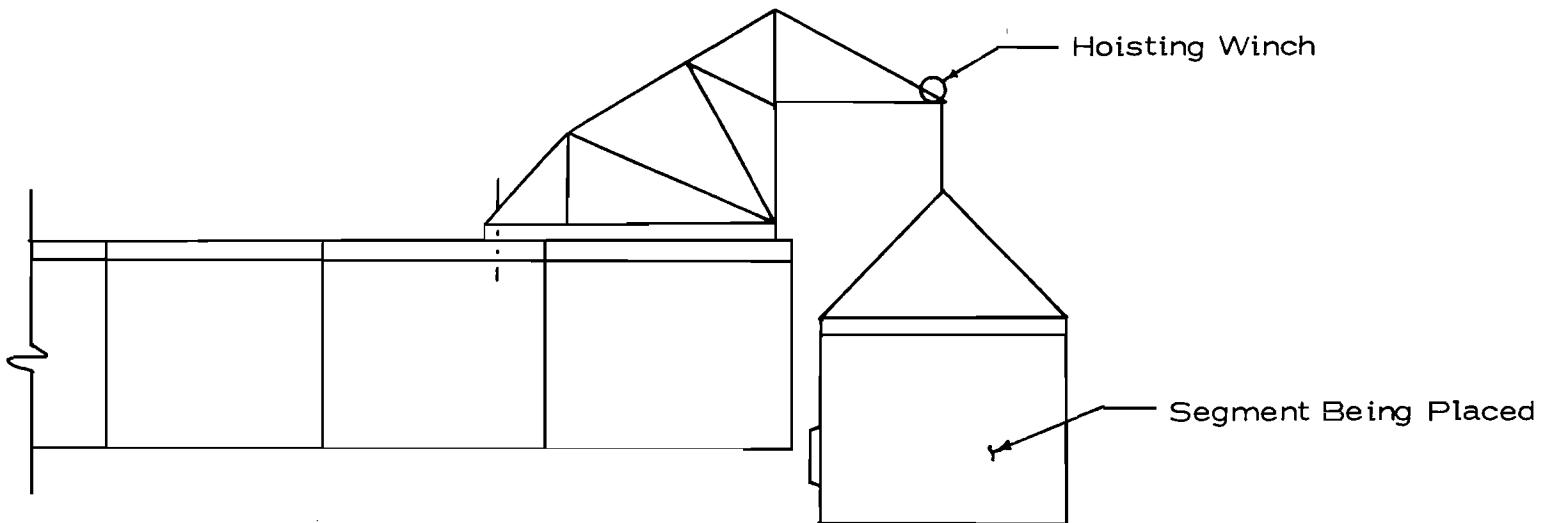


Fig. 4.4. Hoisting device

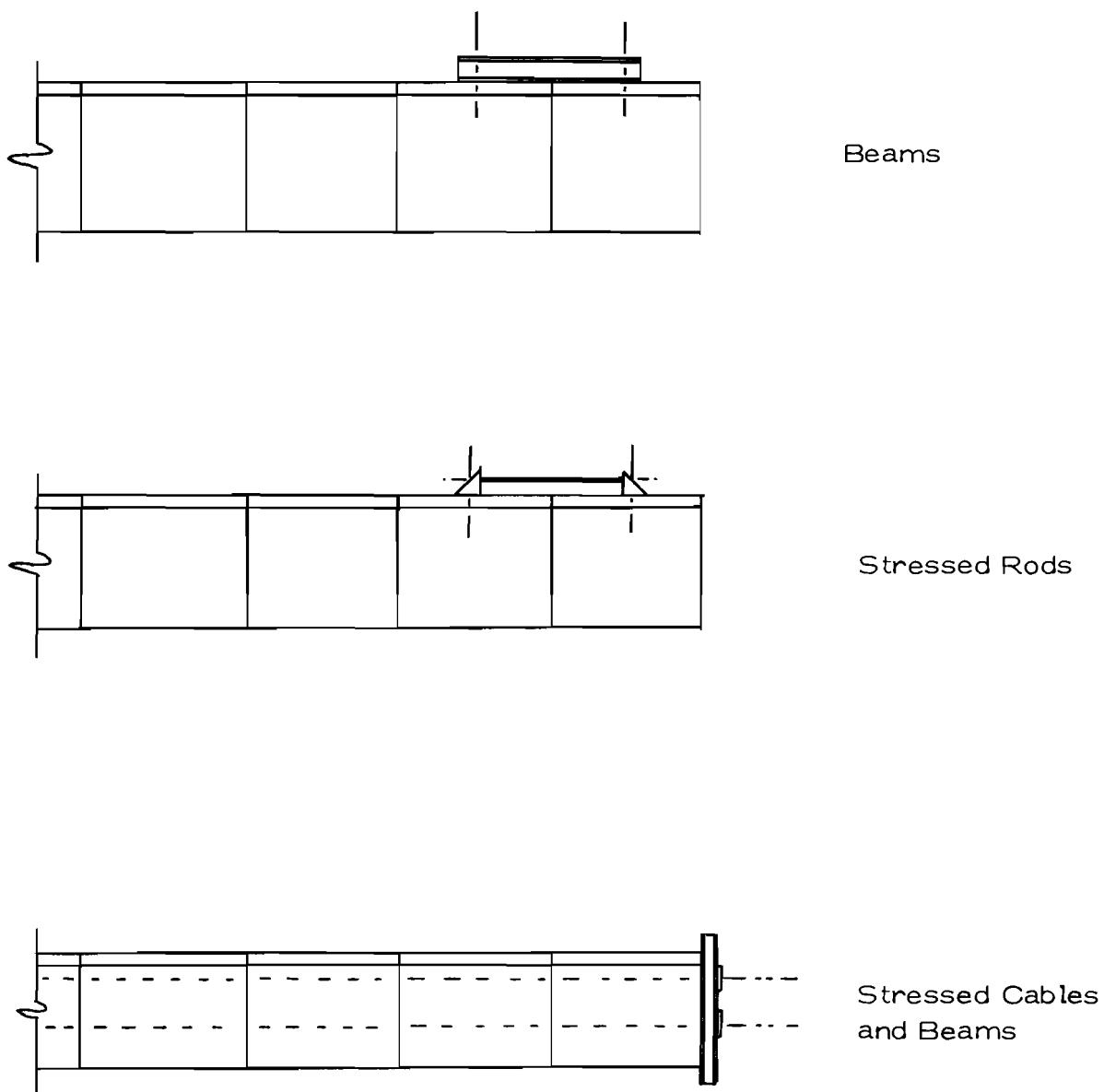


Fig. 4.5. Temporary holding equipment

Frequent barge traffic on the waterway also required moving the barge on which the crane was mounted. These problems pointed toward the use of a hoist mounted on the cantilever if segments could be brought to a point underneath the hoist and lifted from the barge deck. However, all arms were not over the water suitable for barge access.

The problem was solved in an ingenious way. The consulting engineering firm of Ogletree and Gunn, acting for the contractor, developed an erection procedure which utilized simple temporary holding devices and a single barge-mounted crane. The danger of the segment being lifted accidentally striking the in-place mating unit was minimized by design of the erection hardware. Minimization of this danger made the use of a barge-mounted crane more practical. In the Heldenfels Bros.-Ogletree and Gunn system, lifting beams [see Fig. 4.6(a)] equipped on one end with hooks which extend past the face of the segmental units are temporarily post-tensioned to the units using prestressing strands embedded in the webs. The unit is then picked up at a 21° cant by careful location of the single pickup point and connections to the lifting beam, as shown in Fig. 4.6(b). The lifting beam on the previously erected unit is equipped with a matching receiver bolt on the outboard end. Surfaces surrounding this bolt are tapered to guide the hook into easy mating in the receiver. Figure 4.7(a) shows a segment at the time of the engagement of the hook and bolt. Note the extreme separation of all contact faces. Upon engagement, the crane line is slowly released and the unit pivots about the hook-bolt connection closing the gap [see Fig. 4.7(b)] and coming into final position butting flush against the in-place unit. The receiver was equipped with hydraulic rams to allow fine adjustments to be made to match between the units. The tapered shear and alignment keys were useful in guiding the units into correct position. Temporary prestress was applied using rams in the lifting beam receivers and temporary strands in the lower flanges to ensure positive contact of all faces during epoxy setting and post-tensioning. The crane was then released as shown in Fig. 4.7(c).

In practice, the lifting system worked extremely well. Minor difficulties and subsequent adjustments were made during casting yard "dry runs", as shown in Fig. 4.6(b). Inserts were cast in the units to



(a) Lifting beam in place



(b) Unit picked up at  $21^{\circ}$  cant

Fig. 4.6. Lifting equipment



(a) Mating



(b) Closing the gap

Fig. 4.7. Segment erection



(c) Crane-released-unit carried by temporary lifting equipment  
Fig. 4.7 (Continued)

receive the temporary lifting equipment. Close tolerances must be maintained on such inserts and the lifting equipment carefully attached. After a series of "dry runs" the system was used for erection of the precast units under a wide range of wind and weather conditions. Good communication must be maintained between the crane operator and the person directing erection. Walky-Talky units are extremely useful. Experience indicated that the complete lifting process from pickup to release of crane could be completed within fifteen minutes with minimal danger of impact on previously placed units.

Such hardware can be developed economically. The 1971 cost of development and fabrication of erection hardware for six segments amounted to \$0.55/sq. ft. of superstructure deck, or about 2-1/2 percent of superstructure cost. A major erection cost was in the rental of the barge-mounted 165 ton American crane used for lifting units. Use of temporary unbalanced construction resulting in elimination of a second crane was a substantial cost-saving. This type of erection system is particularly useful on small to medium size projects. Amortization of large launching girders is only practical with projects of approximately ten times the scope of the Corpus Christi bridge (400 ft. x 56 ft., or 22400 sq. ft.).

Overhead trusses or launching girders have been widely used on extremely large construction projects, especially where access to the job site is limited. The contractors have sometimes used the trusses to lift segments from trucks or barges below the structure and have sometimes delivered segments to the truss by trucks traveling on that portion of the structure which has already been completed. Since many trusses set only one segment at a time, temporary holding equipment is generally utilized.

In order to construct a bridge in cantilever without using the already completed portion for bearing the load of an unstressed segment, plans must be very carefully made to ensure that the weight of a segment cannot come to bear on the structure. For the Corpus Christi bridge this would have been a very difficult task because of the planned use of barges; any motion induced by the barge or the mobile cranes could accidentally load the structure. It is generally impractical to develop construction methods which can support the segments completely when atmospheric and

human error factors are considered. Substantial progress has been made in developing launching gantries which can move from pier to pier without loading the superstructure.

In erection of segmental bridges of variable depth, the shear key plays an even more important role in erection. The compressive force in the lower flange of such a structure has a vertical component and the key must resist the resultant upward force, since the epoxy acts as a lubricant at this stage. Analysis for the magnitude of such forces is discussed by Muller.<sup>32</sup>

A rapidly developing construction procedure is the use of temporary towers above the piers and cable stays to support the cantilevers. In this way pier moments are greatly reduced. However, the superstructure must be designed for stress reversal and numerous deck anchorages must be used.

Continued development in forthcoming projects in the U.S. will undoubtedly lead to a better assessment of the relative economics of varied erection procedures. The ideal procedure will be simple, rapid, and require low capital investment. The scheme used at Corpus Christi goes far to meeting these requirements.

4.2.3 Alignment Errors. Hopefully, there will not be a time when it is necessary to correct the alignment of a bridge during the erection stage; any errors in the dimensions of segments should be found and corrected in the casting operation, as was done when it was discovered that the casting bed soffit had deflected on casting the initial segments. Corrections were made in subsequent segments to counteract this effect. Continual line and deflection checks should be made during erection and compared with calculated values. If very young precast units are being stressed, deflections should consider the age of the concrete at time of stressing. In the Corpus Christi bridge the units were quite mature at the time of stressing. Furthermore, the extremely small deflections made measurement almost impossible. No corrections were required during erection. However, there are means to correct alignment errors during

the construction stage. One means is to use metal shims on the face of a segment. For instance, if the cantilever is beginning to curve to the right side, thin metal shims could be placed on the right-hand face of the segment after the face has been coated with epoxy. The epoxy will hold the shims in place and help minimize stress concentrations. This method has been successfully used in bridges; however, care must be taken to see that the compression load produced by the tendons is distributed as uniformly as possible. The same method can be used to correct the direction of a cantilever which is turning either up or down from the proper path. If errors are such that shims are impractical, the use of a cast-in-place joint is probably the only means of correction. This can be a very expensive operation, since the segment must be held in position while the joint gains strength.

There may also be other errors which will not show up until the erection stage; e.g., grout could leak from a faulty duct during the grouting of an adjacent tendon and block entry of another tendon. This type of error can usually be corrected by drilling through the blocked area. Specifications should require that all ducts be swabbed after stressing and grouting operations to minimize such blockage. More serious problems may require that a replacement segment be cast; that would be the case if a segment were fractured beyond repair during handling operations. Casting a replacement segment is not a particularly difficult problem, although it is time-consuming and costly. Since the segments should match joining faces as closely as possible, the two segments against which the damaged segment was cast should be used as bulkheads in the forms. Of course, one of the segments could very easily already be in place on the cantilever and, therefore, not available for use as a bulkhead. However, the bulkhead used to form the face of the segment already in place should be adequate with a thick joint. Very careful measurements would have to be made to ensure that the bulkhead was set at the proper slope. Items such as shear keys, alignment keys, and ducts would also have to be very carefully located. Any errors in alignment of the replacement segment could be corrected with the use of shims, as noted above. Although it is probably unnecessary, a very nearly exact duplicate

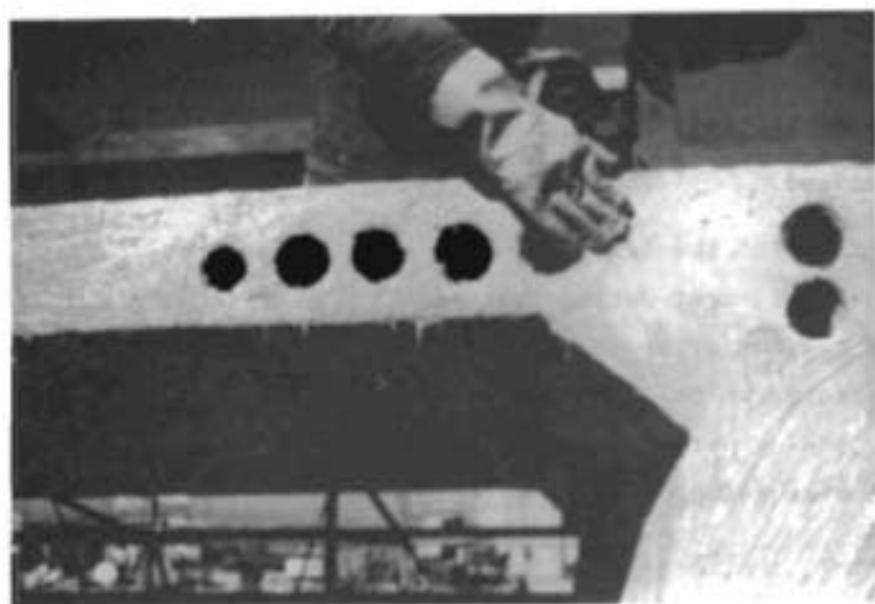
of the face of the segment already in place on the cantilever could be made by casting a thin section against it; the thin section could be used to form a bulkhead for the replacement segment. Similarly, a thin section could be cast against the face of the damaged segment, if the face was not damaged.

Because of the seriousness of such corrections, before correcting any error in alignment during the erection stage it should be determined whether or not the error can be corrected by jacking at closure. Routine vertical errors can be corrected by jacking at the piers, and horizontal errors by jacking with connecting cables at the ends of cantilevers to be joined. A discussion of alignment procedures may be found in Report 121-5.

4.2.4 Joints. The use of epoxy resins as a jointing compound has proved to be the most practical means of constructing segmental bridges. There are still some areas which will require a cast-in-place joint, such as the closure joint.

The primary function of the epoxy is to act as a moisture seal for the protection of the tendons; it may also serve to transfer some of the shear and tensile stresses across the joints. Required properties and performance of epoxies is discussed in detail in Report 121-2. Current developments in Europe are toward complete transfer of shear forces by multiple or serrated keys. Strength of epoxies is not being relied on for long term applications.

Typically, the application of the epoxy is quite simple. The epoxy must be mixed according to the manufacturer's instruction in a suitable container. The guide specification in Report 121-2 requires pre-packaging and color changes so misproportioning becomes immediately apparent. Surfaces must be dry and clean. Light sandblasting with a #4 blast sand was used to prepare all surfaces in the Corpus Christi project. Then the mixture is applied to one or both of the surfaces to be joined in a thin layer. The best means for applying the mixture has been found to be with a rubber glove. A worker simply dips his gloved hand into the epoxy and smears it over the face of the segment, as shown in Fig. 4.8(a). This was done immediately before lifting. Usually after only a few minutes the two



(a) Application



(b) Removal of excess after stressing

Fig. 4.8. Epoxy jointing procedures

segments are joined and temporary contact pressure applied. The tendons are then inserted in the ducts. As soon as possible, the tendons are stressed and anchored. Any excess epoxy will be squeezed out in the stressing operation [see Fig. 4.8(b)] and should be immediately cleaned off. Immediately after the units have been placed in contact and again when the stressing has been completed, a wiper of some sort should be pushed into each duct to remove or smooth out any epoxy that is in the duct and to seal any pockets that form at the joint. If for some reason erection is interrupted, immediate steps must be taken to remove the epoxy from the mating faces. The units must be reprepared for jointing. This is a difficult process.

Cast-in-place joints require that something be placed in the ducts to make them continuous across the joint, since the joints are usually 3 or 4 in. long. Epoxied joints do not require any such item. Of course, the most important reason for the use of epoxy is the much greater speed of erection; construction sequences are greatly distorted by cast-in-place joints which require too much time to cure properly before stressing can be carried out. Epoxy joints are stressed immediately and the epoxy then cures. A thin glue line is highly desirable. The contractor would prefer an epoxy with a moderate pot life and about three hours open contact time. A reasonable balance must be made, since strength development is important. Contact times of one to one and one-half hours seem to be a reasonable compromise.

There have been a few cases where dry joints have been used with no jointing material between the two segments being joined. Some local spalling has been common with the dry joints since local stress concentrations are more prevalent. If perfect matching of joining faces can be accomplished, the use of dry joints may become more common, even though the integrity of the joints with weathering in time is questionable.

4.2.5 Stressing System. The importance of having all ducts properly aligned and fully inspected became painfully evident during the erection stage. Since tendons must be threaded through the entire length of the cantilever, it is easily seen that all ducts must be in near perfect alignment at each joint and should have a minimum of pinches, kinks, or dents.

There are different ways in which tendons may be installed in the ducts. They usually involve the use of a smaller cable or wire which is passed through the duct first and is then used to pull the larger tendon through. In some cases, a neoprene ball has been blown through the ducts with air. A small wire attached to the ball is used to pull a larger cable.<sup>11</sup> Sometimes a rod or stiff wire is pushed through the ducts by hand, although friction losses become very large in the longer ducts. The cable which is passed through the ducts was attached to a small winch which pulled the tendons through the ducts. Tendons can be furnished with a special connection which can be attached directly to the cable.<sup>7</sup> In this application strands were pulled through the ducts with the cable sock or sleeve, shown in Fig. 4.9(a), which binds them tighter as the cable is pulled harder.

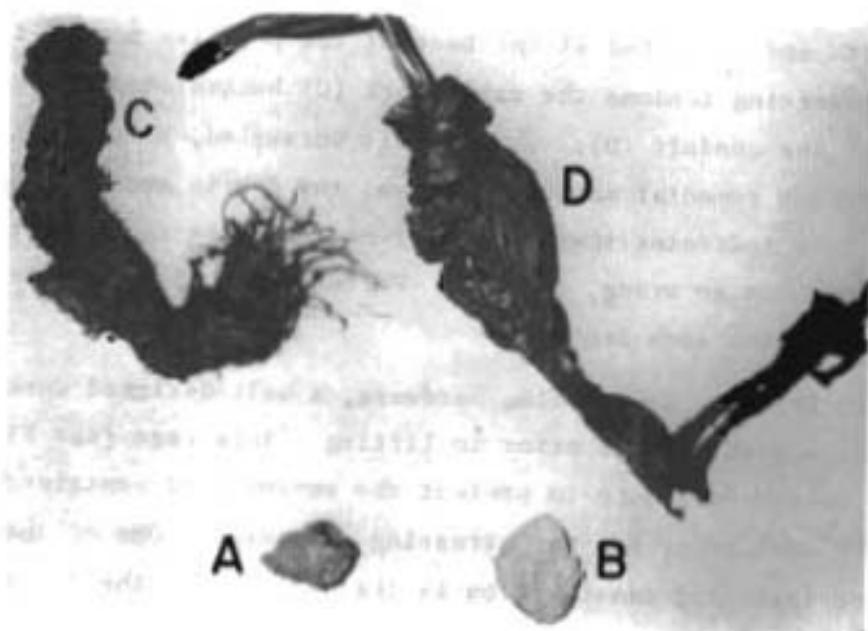
Substantial difficulty was encountered in inserting some tendons and in matching calculated and actual tendon elongations. Particularly in the earlier cast segments, high friction was noted. This was attributed to several factors but principally to kinking, denting, and pinching of the ducts during fabrication and tendon placement. The contractor indicated that his choice of a light duct material was a poor saving and the decision to not internally brace the ducts with inflated tubes was probably a costly one. Several tendons had to be removed and replaced because of wear during the insertion or stressing process in particularly difficult ducts.

After experiencing this difficulty with a number of tendons, remedial measures were taken. Friction was reduced with a water-soluble lubricant. This lubricant was sprayed on the tendon as it was being inserted [see Fig. 4.9(a)] to lessen the friction during the installation and stressing operations. It was later flushed out with water and air was then used to blow the water out before the grout was pumped into the ducts.

In a few cases major difficulties were encountered. In spite of precautions taken to check ducts after epoxy applications and grouting, a crossover did occur between two adjacent tendons at an epoxy joint. Grout flowed into an adjacent tendon and blocked a duct. The blockage



(a) Strand being sprayed with lubricant



(b) Duct residue

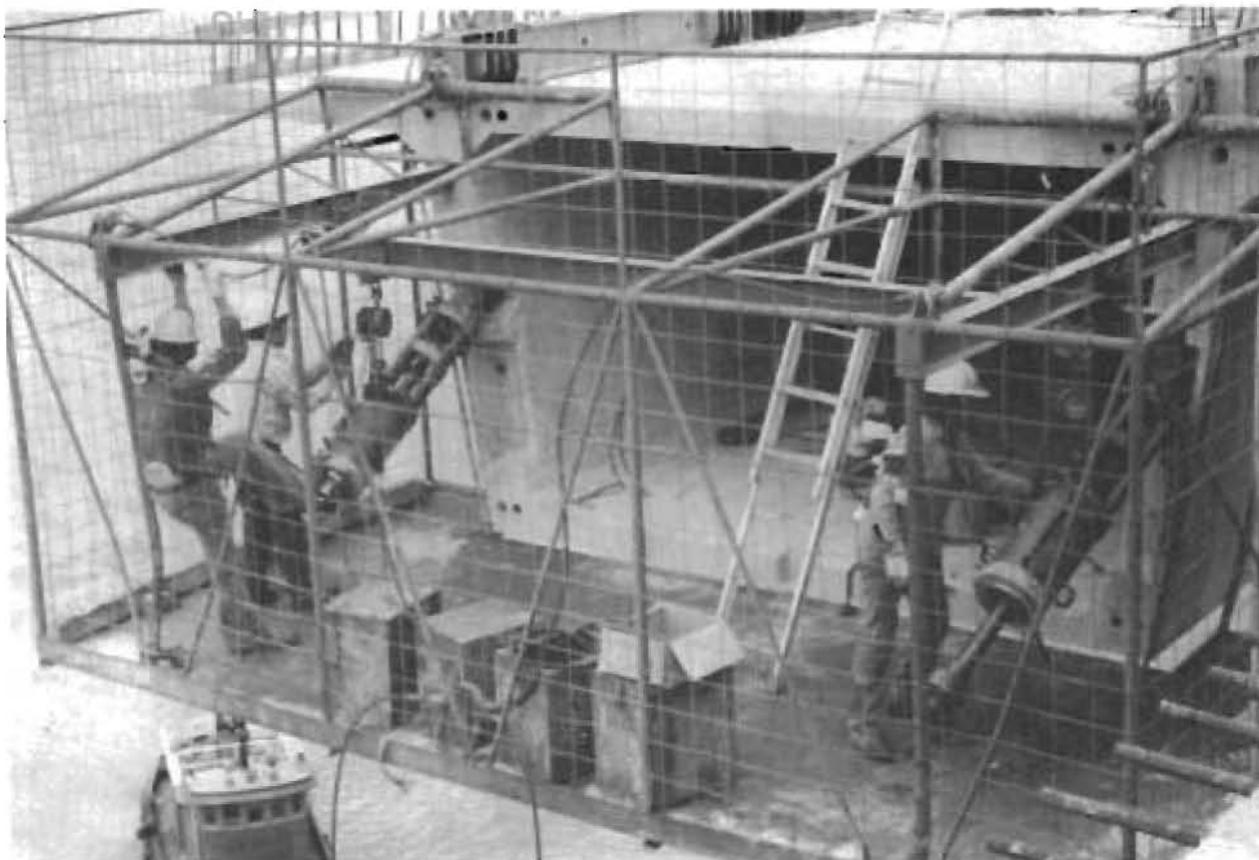
Fig. 4.9. Tendon and duct system

was isolated and the slab cut to expose the tendon duct. The blockage was removed and stressing continued. Such crossovers can be prevented by ensuring that joints have a thorough coating of epoxy so that no voids exist between tendon ducts. Excess epoxy which flows in the ducts can be easily swabbed out immediately after stressing.

In cantilever construction the basic difference in the negative and positive moment cables is in the order in which they are installed. The negative moment tendons must be installed as each pair of segments is added to the cantilever. When all the segments are in place on the cantilever and the closure joint has been completed, there may be a definite sequence specified by the designer for the installation and stressing of the positive moment tendons. In the Corpus Christi bridge, the tendons in the end span were stressed first (Fig. 1.3). Then the tendons in the main span were stressed in succession, beginning with the ones anchored nearest the pier.

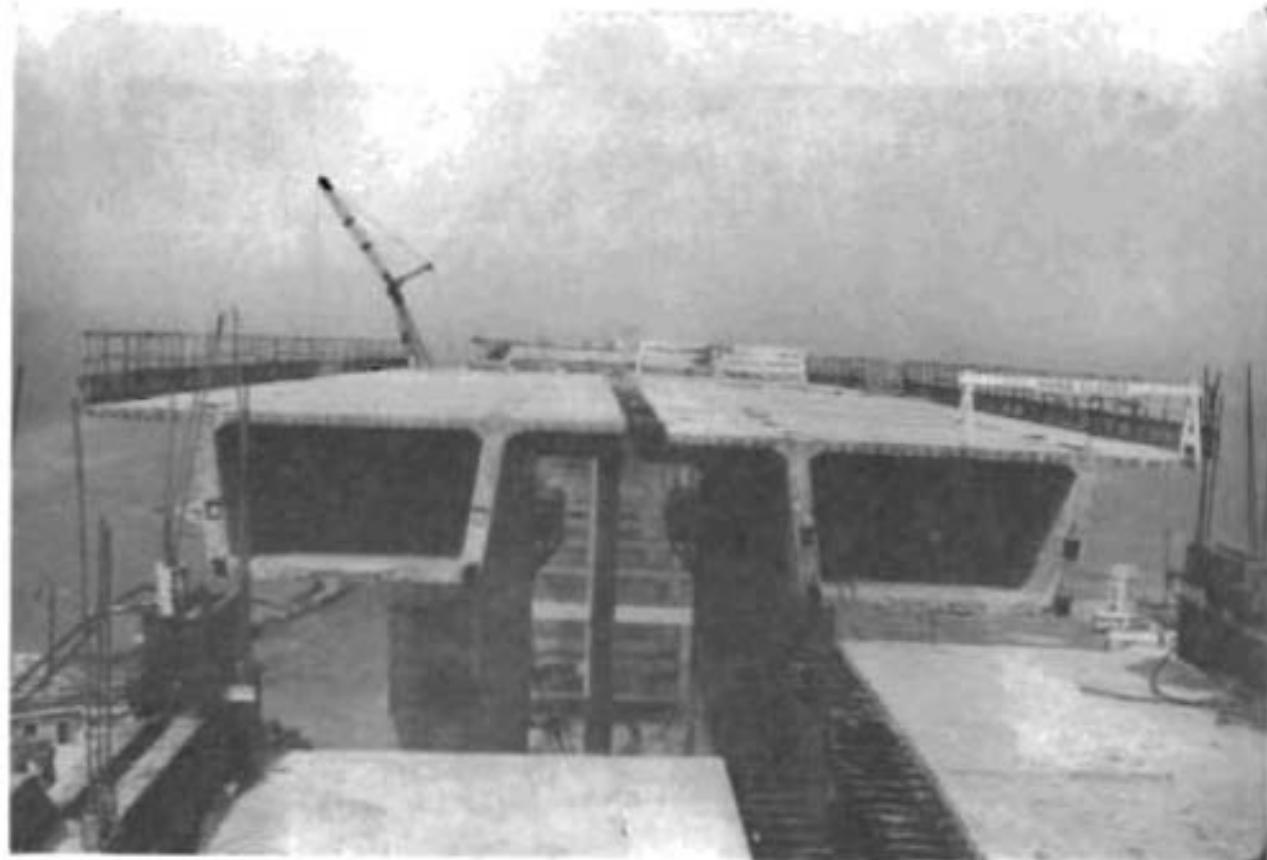
Unexpected difficulty was encountered in inserting positive moment tendons after casting of the closure. Some tendon blockage occurred from hardened epoxy [see A in Fig. 4.9(b)], and residue from the grouting operations (B), which apparently worked its way into the top surface tendon pockets and collected at the base of the positive moment tendon paths. In inserting tendons the cable sock (C) became entangled on a rough edge of the conduit (D). The conduit unraveled, producing a massive blockage. Prompt remedial measures removed the debris and stressing was completed. This indicates that even well-managed jobs can expect problems and if anything can go wrong, it will. Fortunately, the construction method can cope with such factors.

In addition to the lifting hardware, a well-designed work cage was attached to each segment prior to lifting. This cage [see Fig. 4.10(a)] served as a safety enclosure to protect the workmen and contained handling equipment and work space for the stressing equipment. One of the main advantages of segmental construction is that as soon as the first cantilevering sections are in place extremely good work surfaces are available. Grouting equipment, a small mobile crane, a compressor, and other items



(a) Stressing equipment

Fig. 4.10. Construction operations



(b) Nearing closure

Fig. 4.10 (Continued)

were hoisted to the bridge surface and used conveniently throughout cantilevering. The construction superintendent indicated that this advantage made this bridge one of the safest and most convenient he had worked on.

4.2.6 Closure. As noted previously, the closure joints have always been cast in place joints. This is mainly to allow for any slight errors in construction. Sometimes the closure joint is slightly more than the length of one segment. In those cases the closure segment is held in position while a grouted joint is formed at either one or both ends of the segment.<sup>41</sup> Since the grout usually takes at least two days to cure, the problem of holding the segment in position becomes the predominant problem. Probably a more common type of closure will be the one used for the Corpus Christi bridge (Fig. 4.11). After the two opposing cantilever arms were completed, aligned, and lowered to their permanent bearings, the temporary holding device was installed, forms were set, and concrete was placed. In effect, the closure segment was cut in half, with each half being attached to one of the cantilevers by stressing. An 18 in. joint was cast between them to complete the closure.

Although the two opposing cantilevers were not in exact alignment with each other, they were rather easily brought into alignment. Since the cantilever was balanced on the pier, rotation in either a vertical or horizontal direction was relatively easily accomplished. The units were brought into position by a series of successive approximations.

Temporary holding devices for the closure joints do not have to be elaborate by any means. Their purpose is to restrict relative movement of the cantilevers to a minimum while the concrete joint is curing. They can be either external or internal. If they are internal, provision may have to be made for their removal. A pair of beams on the top deck and the bottom of the soffit connected by stressed rods or strands should be sufficient.

Since the positive moment ducts are all continuous across the closure joint, tubes connecting each duct must be provided in the joint to prevent grout entry. Sometimes the tendons themselves are installed

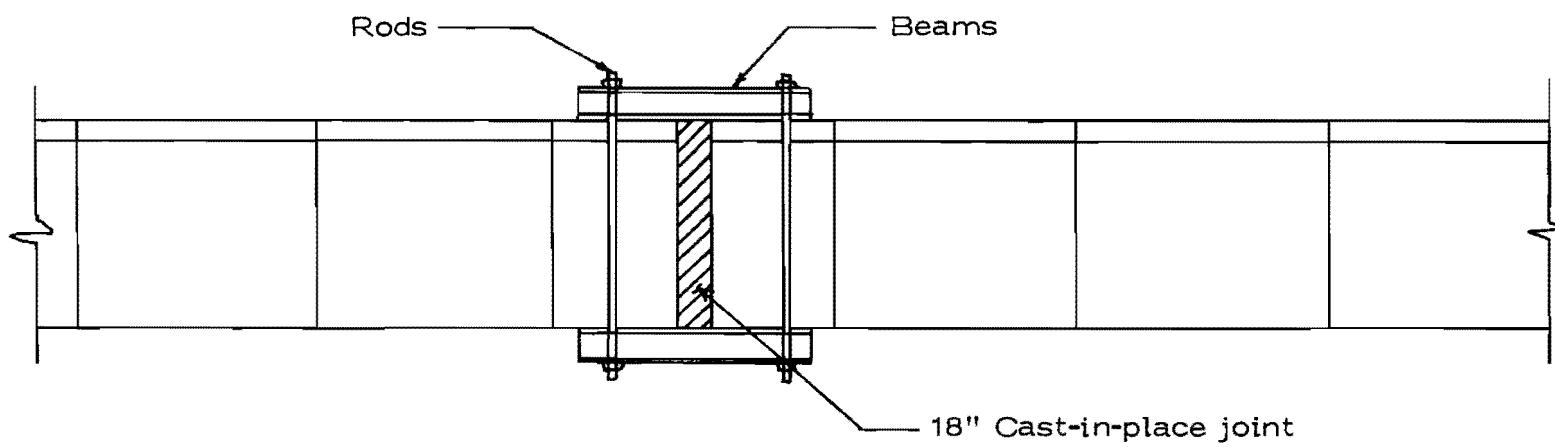


Fig. 4.11. Closure joint

before concreting the joint. Interior forms may be designed to be left in place or to be removed after the joint has cured. Both interior and exterior forms should be similar in cross section to the forms for casting the segments and should lap the other segments enough to prevent grout leakage.

The closure segments for the Corpus Christi bridge were designed to have all longitudinal reinforcement protrude from the face which is to form the cast-in-place joint. The reinforcing from each of the segments laps in the joint and provides additional strength. This is not always necessary or even practical, especially for the joints which are only a few inches in length. This detail made insertion of the last cantilever segments a delicate operation.

Expansion joints were not required in the Corpus Christi bridge but are provided in longer structures. The joints are a special design item but have usually been quite simple. Most of the joints are of a stepped design with the location of the joint being at midspan or at a point of inflection. In some details two half segments are temporarily joined for cantilevering and then freed to allow hinge rotation after completion of construction.

4.2.7 Completion of Structure. After the closure joint cured, the positive moment cables were installed and stressed. When all tendons were stressed, the two cantilevers may be considered a single structure. In the case of the Corpus Christi bridge, each of the longitudinal halves shown in Fig. 4.10(b) is considered as a separate structure until they are joined by the longitudinal cast-in-place joint.

Before the structure can be considered complete, it must be positioned on the end bearing pads as required by the design. This usually consists of weighing the bridge and adjusting the supports with jacks until the correct reactions are provided. Alternatively, adjustment of the supports may be specified by elevation. The distribution of weight is a design item and will always be specified by the designer. Discussion of the various trade-offs in end reactions is given in Report 121-5. Also, most designers will require that all tendons be grouted before positioning the structure. The design required that tendons be grouted within

48 hours of stressing. This caused considerable discussion but was technically feasible.

The installation of the longitudinal cast-in-place joint marks the last phase in the completion of a bridge structure. The joint is usually cast after both halves of the structure have been set in their final positions. Transverse reinforcement protruding from the segments is made continuous across the joint and extra longitudinal reinforcement is provided. In some instances the joint may be prestressed, where lateral post-tensioning is used.

After the longitudinal joint is completed, a wearing surface is usually applied. Asphaltic surfaces seem ideal for the segmentally constructed bridges, since their thickness may be varied to correct any irregularities in the top deck of the bridge--e.g., positive moment anchorages and to compensate for construction tolerances. Bridge railings and lighting are installed as required.

#### 4.3 Summary

The use of falsework for the construction of segmental bridges allows more simplified erection procedures and reduces the pier requirements, but the generally high cost of support systems and longer construction times may limit their use. Local conditions may also restrict the use of falsework. Erection by the cantilever method requires more attention to detail but rapid completion of bridges can be accomplished with proper planning.

Capacity for an unbalanced moment must be provided by the piers or by temporary supports. Several means for providing the moment connection between pier and pier segment have been used successfully. Segments can best be placed with full utilization of the structure. This also makes more efficient use of the structure, since it should always be designed to withstand the full weight of a segment on either end of the cantilever. Temporary holding equipment for securing the segments while tendons are being stressed can be simple in design and should be used if bridge-mounted hoists are not used.

Errors in alignment can be corrected during the erection phase, but careful inspection during the casting operation should eliminate serious errors. Many errors can be corrected by rotation of the cantilever on the pier. This method should always be considered before expensive repairs are commenced.

Epoxies are the most common and practical jointing materials and will undoubtedly continue to be so. Closure joints require cast-in-place concrete; there are relatively inexpensive means of accomplishing the closure between two cantilevers, especially if some small misalignment is present.

Careful attention should be paid to application of epoxy material to make sure that area between adjacent ducts is adequately coated to prevent later crossover of grout.

Tendon ducts should be carefully protected by internal stiffeners such as inflated hoses and supported during concreting operations to prevent damage. Duct joints between segments should be smooth and continuous to minimize tendon friction. Water soluble lubricants should be used when necessary to reduce friction.

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## C H A P T E R    5

### EXPLORATORY INVESTIGATION OF ANCHORAGE PERFORMANCE

#### 5.1 General

Prior to the construction of the prototype bridge in Corpus Christi, an accurate one-sixth scale model of the three-span continuous bridge was built and tested at the Civil Engineering Structures Research Laboratory of The University of Texas' Balcones Research Center in order to verify design criteria and to check construction techniques.<sup>22</sup> During the cantilever construction of the model and under service loading after completion, the experimental results showed good agreement with theory. However, during the actual construction of the prototype a problem not encountered in the main model study did arise. When stressing the negative moment tendons, a crack appeared in the web of numerous segments along the path of one of the tendons. Subsequent model tests revealed that these cracks did not reduce the ultimate load-carrying capacity of the bridge, but their unexpected formation caused ample concern and delay while the causes and effects were evaluated.

#### 5.2 Object and Scope

The object of the exploratory anchorage investigation was to isolate the cause or causes contributing to the premature cracking along the tendon path which occurred while stressing the negative moment tendons, and to recommend measures to prevent reoccurrence. Particular attention was given to effects of anchorage geometry, concrete strength, and to the alteration of local reinforcement. Additional tests investigated the effect of longer spiral reinforcement, vertical prestressing, tendon curvature, and percentage of web steel. In addition to field measurements on the prototype, fifteen specimens were tested to observe basic behavioral

characteristics. These specimens were designed to be one-sixth scale models similar to the web section of the prototype bridge and were patterned after the one-sixth scale model.<sup>22</sup>

### 5.3 Problem Background

When the cracks along the tendon path occurred in the prototype a comparison between details of the model and details of the prototype revealed two basic differences which could have caused the problem. First, during the fabrication of a few of the prototype box units, web and top slab reinforcing bars were cut with a torch to allow for insertion of anchorages and placement of tendon ducts. In some instances these bars were apparently never replaced and in other instances the reinforcing may have been only partly replaced. In the model, bars which were cut for anchorage insertion or placement of tendon ducts were always replaced. Second, there was an apparent difference in the anchorage systems. At the time of anchorage selection for construction of the model, the prototype plans allowed the choice of wedge type or bearing type anchorages. For the one-sixth scale model, the bearing type was chosen. During very preliminary model tests investigating construction details, a tendency for splitting along the tendon was observed, which was restricted by use of a spiral. All anchorages used in the main model<sup>22</sup> were bearing type with adequate spiral reinforcement. A cone type anchorage was later chosen for the prototype by the contractor. Anchorage details developed by the contractor and his agents called for substantially less spiral and local reinforcement than used in the main model tests.

After the cracking was noticed in the prototype, an accelerated study on another one-sixth scale model was conducted using one-sixth scale cone type anchorages to determine if these web cracks would be reproduced in the model, and if so, to determine the effect such cracks would have on performance of the bridge.<sup>22</sup> These supplemental tests did produce cracks along the tendon path which were similar to the prototype cracks. Fortunately, the tests revealed that the cracks did not reduce load-carrying capacity. The prototype was subsequently completed and is performing satisfactorily. The Texas Highway Department required the contractor to

"vee" and seal the cracks with epoxy to protect the units from the corrosive environment of the Texas coastline.

Thus three basic differences existed between the prototype details and the model details:

- (1) In some prototype box units, web and top slab reinforcing bars were cut for placement of hardware, but the sections which had been removed were never replaced or only partly replaced. In the model, such bars were always replaced.
- (2) The prototype used a cone type anchor whereas the model used a bearing type anchor.
- (3) The spiral reinforcement used in the model was relatively much longer and heavier than the spiral reinforcement used in the prototype.

Since during the construction of the 84 box units of the model bridge no cracking along the tendon path occurred, the subsequent cracking in the prototype was unexpected. This study clarified the reasons for the cracking and the relative contribution of various factors.

#### 5.4 Anchorage Problems

In the construction of cast-in-place post-tensioned box girders, it is feasible to use enlarged anchorage zones at the ends of the spans or at interior diaphragms to contain the bursting stresses which generally occur at the point of application of the post-tensioning forces and to reinforce the end zone to control spalling stresses set up on the end faces of the member. In construction of post-tensioned "I" beam type girders, cast as a monolithic unit, it has been the general practice to provide a thickened end block with substantial additional reinforcement to accommodate these anchorage zone stresses. However, in segmentally constructed box girders it is not economically practical to provide massive thickness to control anchorage zone stresses. In fact, the entire length of the structure becomes the anchorage zone for an appreciable number of tendons and overconservative anchorage zone design could greatly penalize the economics of construction.

A number of problems have occurred in such post-tensioned applications in both the bridge and the building field which indicate that the

design procedures and design criteria for post-tensioned anchorage zone bursting stresses need further examination and refinement. Substantial cracking along the tendon path was experienced in the prototype precast segmental bridge and in a cast-in-place box girder bridge reported by Dilger and Ghali.<sup>14</sup> In both of these bridges the cable profiles required significant curvature, inclination, and eccentricity in and near the anchorage zones. Similar cracking has been reported in post-tensioned slab structures and other thin web applications. A number of cases have been reported of cracking at web-flange junctures in I and T type cross sections.

The cracking which has occurred in these anchorage regions has generally not opened further under actions of live or dead loads on the member and has not appreciably reduced the member strength. However harmless these cracks might appear, they provide a path for penetration of moisture and salts and thus present potential corrosion and frost damage threats. The formation of these cracks negates one of the major factors leading to the choice of prestressed concrete--minimization of service load cracking.

Major and contradictory changes have taken place in the AASHTO, ACI, and PCI design specifications in recent years, based more on the results of field experience and proprietary data than on published analysis or test procedures. These changes have led to a series of recommendations which, while vague, are both conservative and workable for many applications where massive end blocks can be used with relatively straight or gently curving tendons in cast-in-place post-tensioned construction. However, they are not necessarily appropriate for many new developments in post-tensioned applications. There is an immediate need for a comprehensive study to update design criteria for post-tensioned anchorage zones in order to consider a wide range of emerging applications and to provide more specific guidance to bridge designers and builders regarding fundamental criteria for anchorage systems so that they can better assess the adequacy of proposed anchorage procedures and tendon layouts for a given project. The limited exploratory anchorage study reported herein reinforces that need.

Current design specifications for post-tensioning anchorage zones are based on the concept of a well-defined "end block" and a high degree of reliance on the supplier of the anchorage to provide the necessary details. The 1973 AASHTO Specifications,<sup>2</sup> Sec. 1.6.15--Anchorage Zones, provides that "For beams with post-tensioning tendons, end blocks shall be used to distribute the concentrated prestressing forces at the anchorage." The specific provisions seem to apply primarily to an "I" section and not a thin webbed box girder section, since they indicate "Preferably they shall be as wide as the narrower flange of the beam." It specifies that:

In post-tensioned members a closely spaced grid of both vertical and horizontal bars shall be placed near the face of the end block to resist bursting stresses. Amounts of steel in the end grid should follow recommendations of the supplier of the anchorage. Where such recommendations are not available, the grid shall consist of at least #3 bars on 3 in. centers in each direction, placed not more than 1/2 in. from the inside face of the anchor bearing plate.

Closely spaced reinforcement shall be placed both vertically and horizontally throughout the length of the end block in accordance with accepted methods of end block stress analysis.

In a similar fashion, the current ACI 318-71 Building Code,<sup>4</sup> Sec. 18.11, specifies that end blocks shall be provided when required, and that reinforcement shall be provided when required in the anchorage zone to resist bursting, horizontal splitting, and spalling forces induced by the tendon anchorages. The Code Commentary indicates a guideline for determining permissible direct bearing stresses under the anchor, but provides no information as to the procedures that might be used for determining proper reinforcement for bursting, spalling, section change, or tendon curvature stresses.

In both of these key design specifications, and in general practice, heavy reliance is placed on the supplier of the anchorage system to provide reasonable amounts of containment reinforcement to resist bursting stresses. In the 1973 edition of the AASHTO Specifications, the allowable anchorage bearing stress was set at 3000 psi but not to exceed  $0.9f'_{ci}$ , irrespective of concrete strength, degree of confinement, type of anchorage device, or spacing of anchorages. For many cases this represents a reduction from the values allowed under the AASHO 1969 edition and the recommendations of

the current ACI Building Code. It certainly is a conservative interim step, reflecting difficulties which have occurred in a number of applications, but deals with only a portion of the overall problem.

An excellent summary of the state-of-the-art in 1966 was presented by Hawkins.<sup>19</sup> A number of authors have presented design procedures for anchorage zone stresses, based on both elasticity and photoelastic analyses and on test results. Among the more comprehensive are the recommendations of Guyon<sup>16</sup> and of Leonhardt.<sup>29</sup> The fundamental design procedures suggested by Guyon, and in particular the "symmetrical prism" method for dealing with multiple tendons anchored in the same end block were generally confirmed in a comprehensive test program by Zielinski and Rowe.<sup>42,43</sup> The Rowe and Zielinski papers contained an excellent summary on both analytical and experimental work carried out prior to the early 1960s. Unfortunately, most of the analytical procedures and all of the tests were aimed at conditions representative of anchorage zones at the ends of girders with tendons usually horizontal or at a very small angle of inclination. Often only a concentrically reinforced prism was tested. The research results of Zielinski and Rowe were translated into a design recommendation by Rhodes and Turner,<sup>38</sup> who primarily dealt with the proper design of an "end block" and in particular the local area around the anchorage. They did mention that other tensile zones can occur at the junction of the web and the flange away from the specific anchorage assembly, but this factor does not occur in the main body of their design method. Some supplementary reinforcement in the immediate vicinity of the anchorages for the Intracoastal Waterway segmental box girder bridge was provided, apparently based on the general procedures outlined by Rhodes and Turner. Unfortunately, the "recommendations of the supplier of the anchorage" were inadequate to resist stresses in the overall anchorage zone and unexpected and regrettable cracking occurred in the webs of the box girders during erection.

This exploratory series of tests was utilized to investigate the specific factors contributing to this cracking and to determine possible ways to control or eliminate such distress. These tests indicated that the cracking which occurred in the prototype could be reproduced in accurate models. The tests showed that in lower strength concretes

(4000-4500 psi) the anchorage zone barely developed the full ultimate strength of the tendons and that several positive procedures may be used to minimize cracking. Out of these preliminary tests and the concurrent search of the literature has grown the realization that the present state-of-the-art for determining and controlling anchorage zone tensile stresses in this type of post-tensioned application is quite deficient. Very little guidance is available for the designer in either initially designing anchorage zone reinforcement, or in checking shop drawings submitted by the anchorage supplier. The recently issued PCI Post-Tensioning Manual<sup>34</sup> gives little guidance to the designer, other than a recommended formula for computation of allowable bearing stress and a footnote calling attention to the previously cited References 16 and 29. The recommendation for allowable bearing stress given in the PCI Manual can be substantially above the 3000 psi allowed in the current AASHTO Specifications. The origin of this expression is not well-documented. Hawkins<sup>19</sup> traces a similar expression to tests of column base plates.

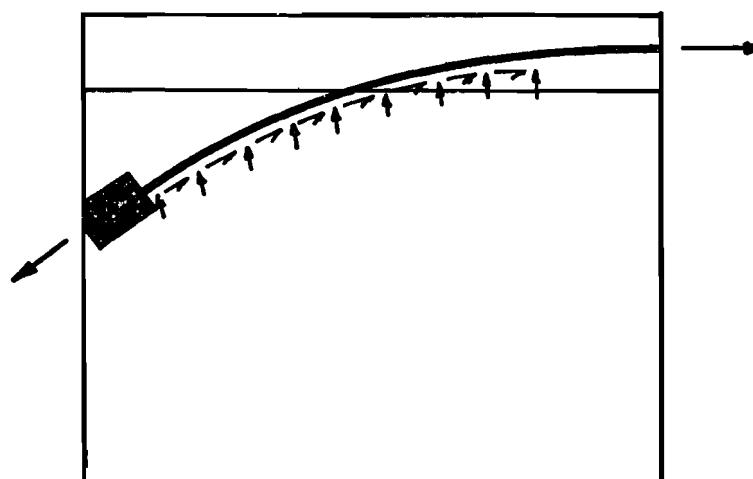
In the early stages of development of pretensioned concrete members, considerable research and development efforts were devoted to a study of required anchorage zones and end blocks for pretensioned members. With growth of experience and increased knowledge of fundamental principles, satisfactory designs for anchorage systems in pretensioned members were developed. However, that phenomenon is basically one of load transfer distributed over a length of the member with a large multiple of small tendons which again tend to be reasonably horizontal in the end zones. The problem in post-tensioned anchorages is fundamentally different.

A few studies have considered details which are more representative of typical anchorage problems in post-tensioned thin section applications. Leonhardt<sup>29</sup> has considered both eccentric prestress forces and prestress forces at a small inclination from the horizontal axis ( $0^{\circ}$  to  $6^{\circ}$ ). Christodoulides<sup>12</sup> studied the end block of a gantry beam with a number of inclined tendons and stated that the maximum tensile stresses were considerably greater than those calculated using the Guyon approach. A number of tests were carried out by Sargious<sup>39</sup> using both photoelastic and reinforced concrete end block tests for girders with multiple tendons and with

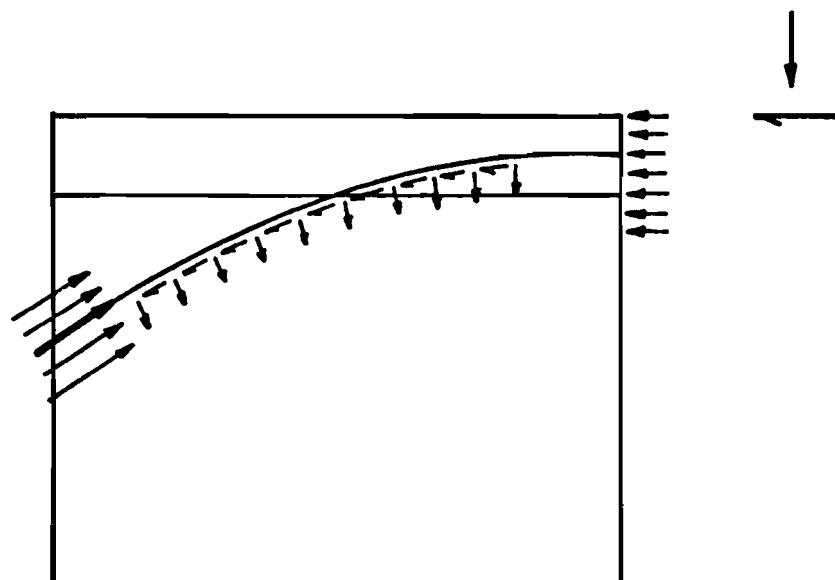
tendon inclinations up to 6° from the horizontal axis. His experimental results indicated substantial transverse tensile stresses occurring away from the loaded face and effectively on the line of action of each force. Again, the measured tensile stresses were substantially above the predictions of the Guyon theory and were shown to be a function of eccentricity as well as force magnitude. Gergely<sup>15</sup> and Lenschow and Sozen<sup>28</sup> have suggested an elemental beam equilibrium analysis which has great potential for design of anchorage zone crack control reinforcement. It deals primarily with straight tendon paths.

In a typical post-tensioned anchorage zone in a segmentally constructed box girder, a complex set of combined stresses is set up, as shown in Figs. 5.1 and 5.2. The tendons are usually both curved and inclined to the horizontal (sometimes at a relatively large angle) at the point of anchorage. Current developments in European practice have introduced substantial horizontal curvature in this zone in addition to vertical curvature. Many anchorages have a tapered or wedge shape. Thus, in addition to the normal anchorage stresses cited in the literature for a compressive point load applied to a solid mass, there are additional effects due to the curvature of the cables, the eccentricity and angle of inclination of the point of application of the force, and the shape of the anchor. Virtually all analytical and most of the test results to date have looked at the problem of an essentially linear horizontal force or combination of forces applied to a rectangular cross section.

In addition to the proper sizing of the bearing plates, it is necessary to provide additional active or passive reinforcement along the beginning of the tendon path to control the tensile forces. Active reinforcement can be in the form of vertical prestressing, while passive reinforcement can be in the form of concentric spirals or conventional web reinforcement. Design criteria for both the provision of proper reinforcement close to the anchorage devices and reinforcement for the entire analogous "end block" in thin sections are definitely needed. This exploratory series was intended to indicate effective interim procedures until a comprehensive design procedure could be developed.



(a) TENDON



(b) CONCRETE

Fig. 51. Post-tension forces

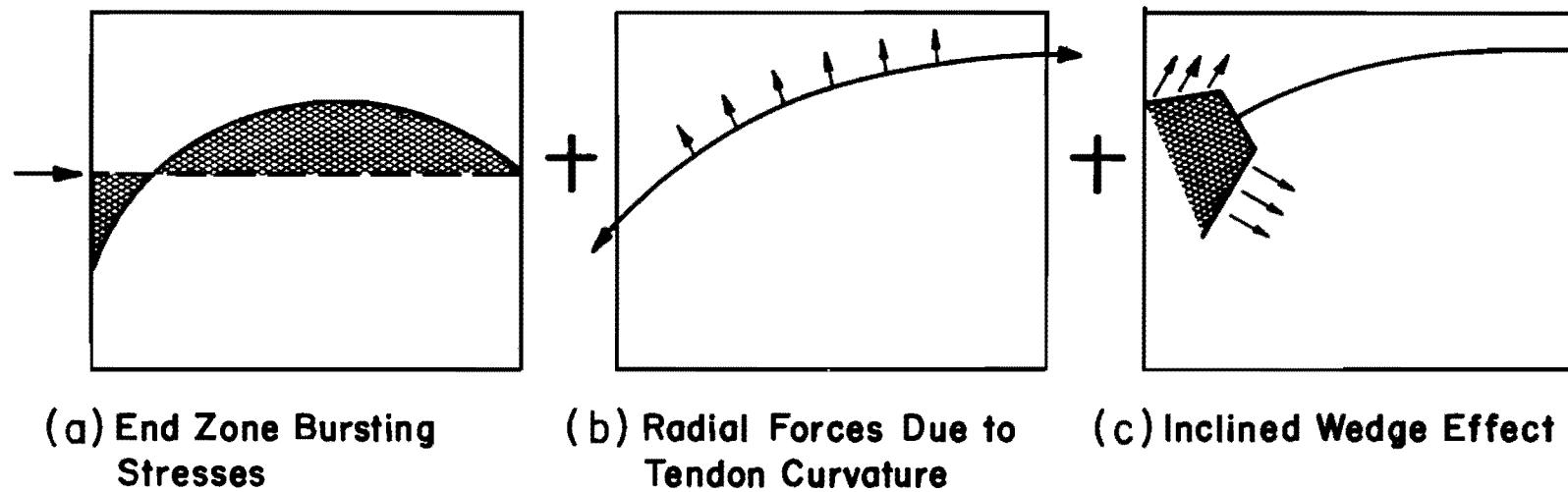


Fig. 5.2. Combination of end zone, radial, and inclined wedge effect stresses

### 5.5 Organization of Investigation

The anchorage study was part of the much larger research program which began at The University of Texas at Austin in 1968. Since this study will frequently refer to other parts of the much larger overall project, a chronological sequence of events may be beneficial. The cooperative research project was started in 1968. Extensive analytical studies resulted in a preliminary design for a typical water crossing which was finalized for a specific application in Corpus Christi by the Texas Highway Department. The bridge consisted of 84 box units using epoxy joints. In order to ascertain the accuracy of the design criteria, analytical methods, construction techniques, and shear performance of the epoxy joints, an accurate one-sixth scale model of the three-span continuous bridge was built and tested by Kashima. The construction and testing of the model is detailed in Ref. 22. Fabrication of the one-sixth scale model units began in the Fall of 1970, with construction of the model bridge beginning approximately a year later. The model tests ended in the Summer of 1972, when the bridge model was loaded to failure. Construction of the prototype bridge began in the Fall of 1972. The box units had been cast several months prior. Construction was slowed when cracks were found along the tendon paths near the anchors in numerous segments. Field measurements were made which verified the nature of cracking. The discovery of these cracks led to an accelerated model study in February 1973. A model of a portion of the bridge was used to determine the effect such cracks would have on anchorage and shear strength. These tests were conducted by erecting three interacting box units which had been constructed using a model cone-type anchorage very similar in detail to the one used in the prototype. In addition, web and top slab reinforcing bars were cut and the sections which were removed were never replaced or only partly replaced, as had occurred during the fabrication of some of the prototype box units. The results of these tests showed the same cracking pattern as the prototype units. Ultimate strength tests showed no significant decrease in anchorage or shear strength.<sup>22</sup> Construction of the prototype resumed and it was completed in June 1973.

The additional web model test program reported in this study began in September 1973. A total of fifteen specimens was tested. These specimens were divided into three groups:

- (1) Series 1 specimens were single web models which were fabricated using the general details of the accelerated one-sixth scale box section model study. These tests reproduced cracks similar to the prototype cracks just as the box units of the accelerated study had. An evaluation of the results of the Series 1 tests indicated that removing or omitting some of the web and top slab reinforcing steel, once believed to be a primary cause of premature cracking, had little effect on the cracking load.
- (2) Series 2 specimens investigated the effects of anchorage geometry. The results of these tests showed a significant lowering of the cracking load for specimens with conical-type anchorages.
- (3) Series 3 specimens investigated alternate anchorage zone details such as local spiral reinforcement, percentage of web steel and vertical post-tensioning of the web. Substantial increases in cracking loads were achieved.

#### 5.6 Prototype Anchorage Zone Measurements

During the cantilever construction of the prototype structure in Corpus Christi, a series of cracks was noted while stressing the negative moment tendons. As shown in Fig. 5.3, the cracks formed in the web and generally coincided with the upper tendon path when double tendons were present in a web. The cracking was not generally visible over most of the anchorage assembly, but could be seen, although narrower in width, in the several inches immediately ahead of the anchorage where some supplementary reinforcement had been provided. The cracking generally was lost from sight as it neared the web-top flange juncture. The level of loading at which the cracking formed was somewhat erratic, but generally occurred near the end of stressing single tendon webs or during stressing of the second tendon in double tendon webs. In most cases, cracking was visible on both the interior and exterior surfaces of the webs, although the higher cover on the exterior surface seemed to reduce exterior cracking.

In order to quantify and document the problem, as well as to assist in assessment of the potential seriousness of the cracking, a series of field measurements was made. Initially a segment which had been



Fig. 5.3(a). Typical web crack in prototype  
(cracking accentuated by black  
marking pen in the field)

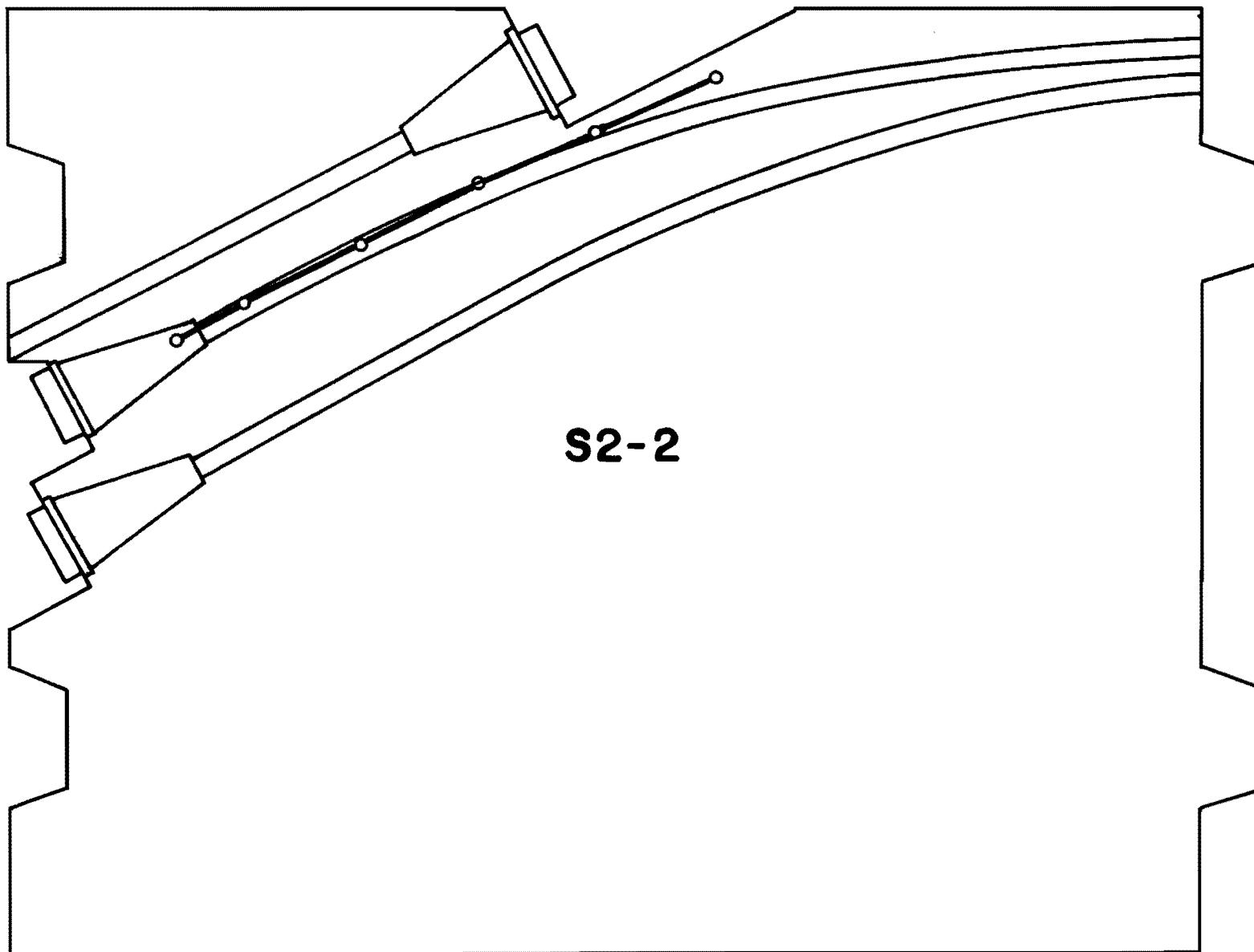
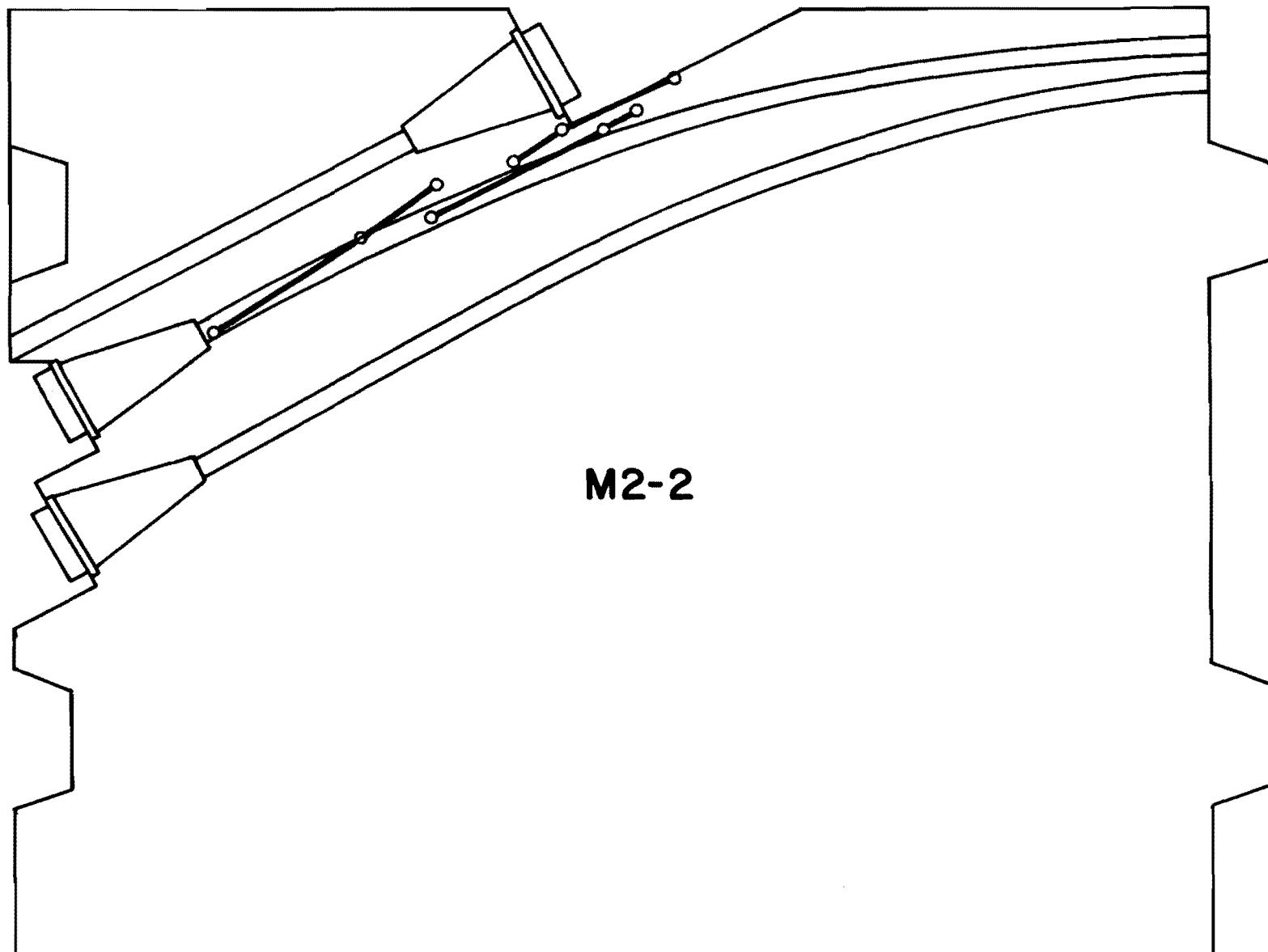


Fig. 5.3(b). Typical web crack pattern in prototype--continuous curve



125

Fig. 5.3(c). Typical web crack pattern in prototype--discontinuous crack

rejected for use in the bridge because of excessive honeycombing was instrumented to document the level of strains at which cracking occurred and to determine the feasibility of field measurements. A simple end bearing anchor device was developed for the far end of a tendon and the tendon was then stressed in the single segment unit in the casting yard. Cracking identical to that which had occurred in the cantilevered units was noted. This simple test, accomplished in less than 48 hrs. including development of an anchor plate, made everyone conscious of the desirability of running such a proof test before going into full production in the casting phase. Had such a test been run immediately after the initial units developed specified strength for post-tensioning operations (usually a few days), anchorage zone alterations could easily have been made to the units.

Since all 84 segments were cast when the cracking was discovered, a more extensive program of field measurements was undertaken to assess the potential seriousness of the cracking. Four segments were selected for instrumentation. Strain gages were applied, as shown in Fig. 5.4, to the segment concrete surface, measuring strain normal to the tendon path and to selected web reinforcement directly along the tendon path.

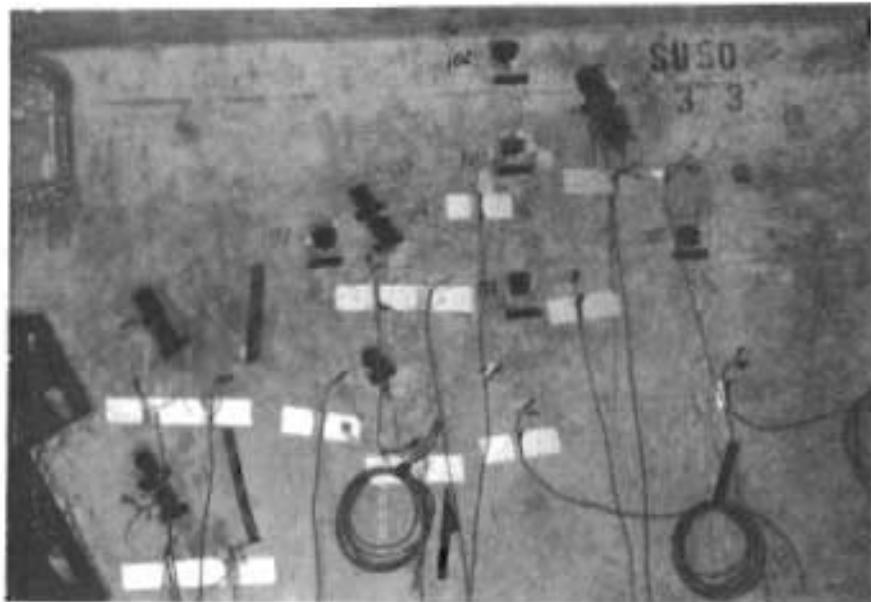


Fig. 5.4. Prototype segment with strain gages attached along tendon paths

Typical locations of concrete and stirrup steel strain gages are shown in Figs. 5.5 and 5.6. All strain gages were mounted on the segments while on barges at the construction site. Concrete was removed to expose the stirrups, deformations were ground off, surfaces prepared, and gages mounted and waterproofed.

Portable battery-operated strain indicators were connected to the gages and measurements were taken during the complete stressing sequence. The particular segments chosen for instrumentation were the M3 and S3 units in quadrant 3. Each web had two tendons. Each tendon consisted of twelve 1/2 in. diameter 270k strands. Maximum stressing load for each tendon was 396k. The lower tendons were stressed to 396k and then the upper tendons were stressed. Although previous field experience had indicated a number of webs first cracked during stressing of the first tendon at less than 400k, in three of the four webs observed cracking occurred along the upper tendon path during stressing of the second (upper) tendon. In one web cracking occurred along the lower tendon path and occurred subsequent to stressing of the lower tendon but before stressing of the upper tendon had begun. All cracking was sudden with a distinct noise of cracking and little change in appearance under subsequent stressing.

Typical concrete strain measurements during stressing are shown in Fig. 5.7. Detailed locations of these gages were shown in Fig. 5.5. The two gages located approximately 29 in. along the tendon (No. 126 and 127) show distinctly different strains. Gage 126, which was slightly above the tendon axis, was continually in compression and based on an assumed  $E_c = 4.8 \times 10^6$  psi for  $f'_c = 7000$  psi concrete, indicated a compressive stress normal to the tendon axis of 1200 psi at the time of visible cracking. The maximum stress at completion of stressing for this same gage would correspond to 1850 psi compression. However, the generally similarly located Gage 127, which was on the tendon axis and very close to the cracking, indicated tensile stresses throughout the stressing. A maximum tensile strain of 78 micro in./in. was noted immediately prior to crack formation. Assuming the same elastic modulus, this would correspond to a tensile stress of 375 psi, or about  $4.5 \sqrt{f'_c}$ . Since this is an arbitrary location along the tendon path and not necessarily the point of

#4 NORTH  
M3-3

128

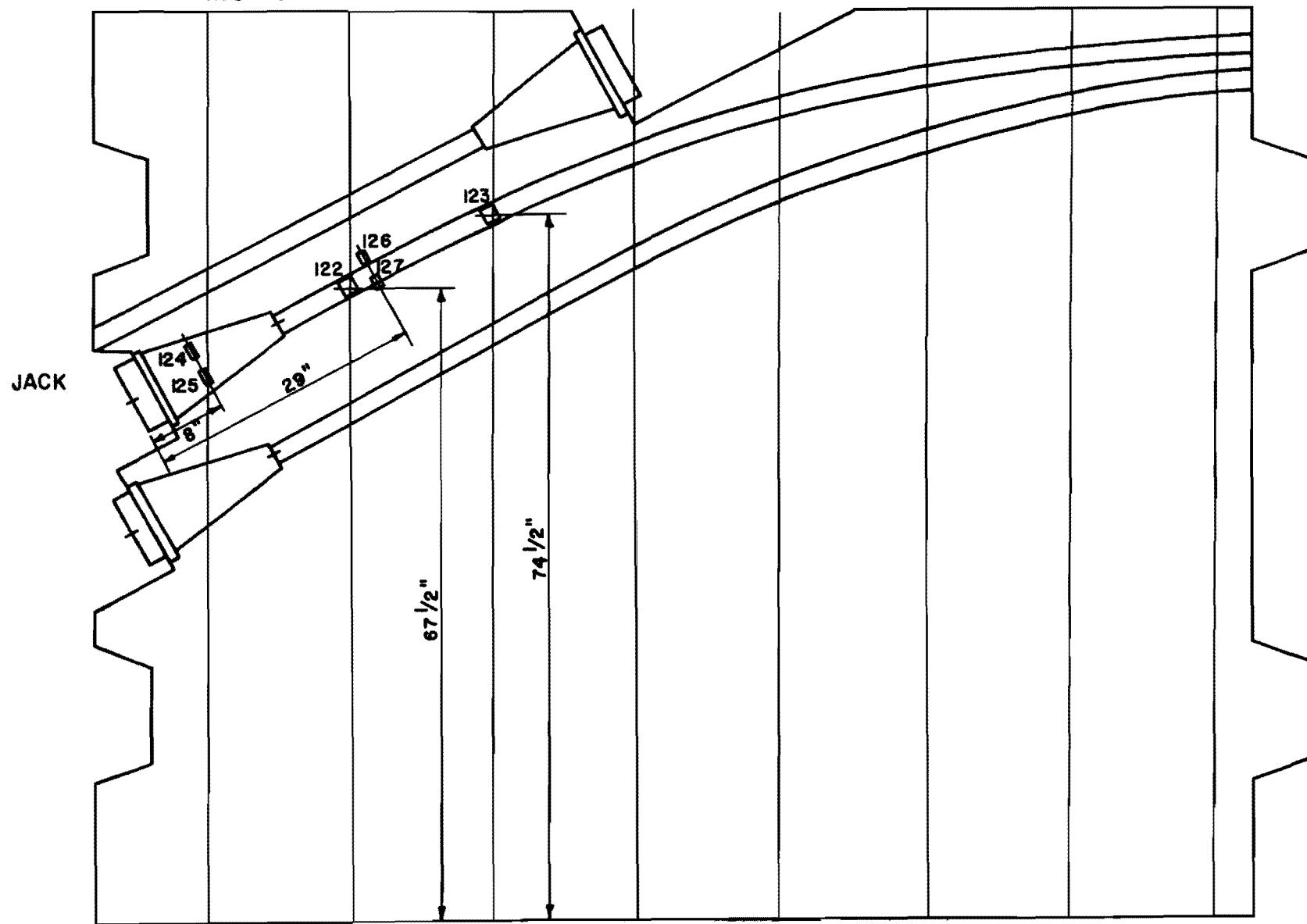


Fig. 5.5. Typical location of strain gages on stirrups and on concrete in prototype instrumentation

□ Concrete Gauge  
□ Stirrup Gauge

#3 SOUTH  
M3 - 3

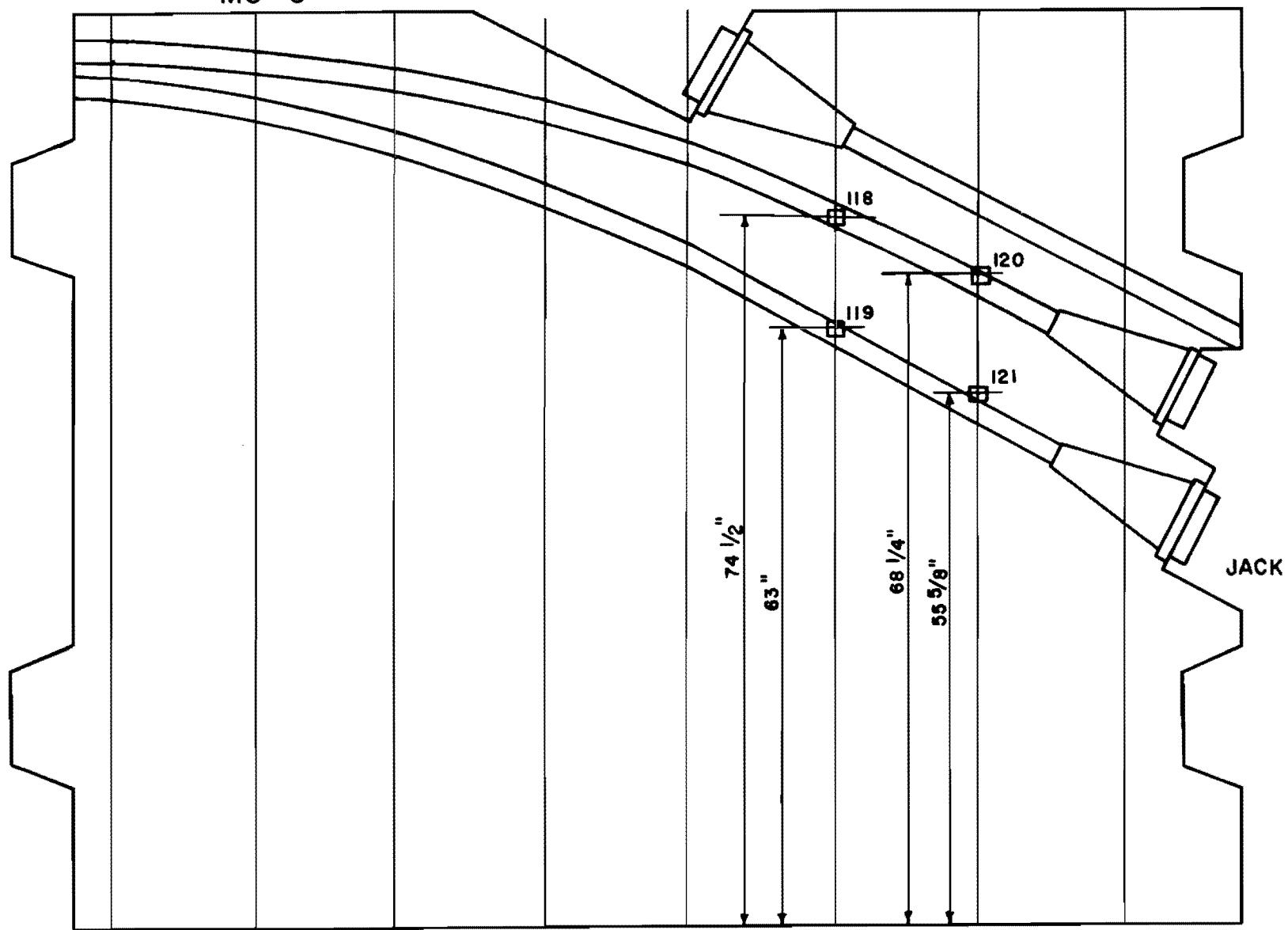


Fig. 5.6. Typical location of strain gages on stirrups in prototype instrumentation

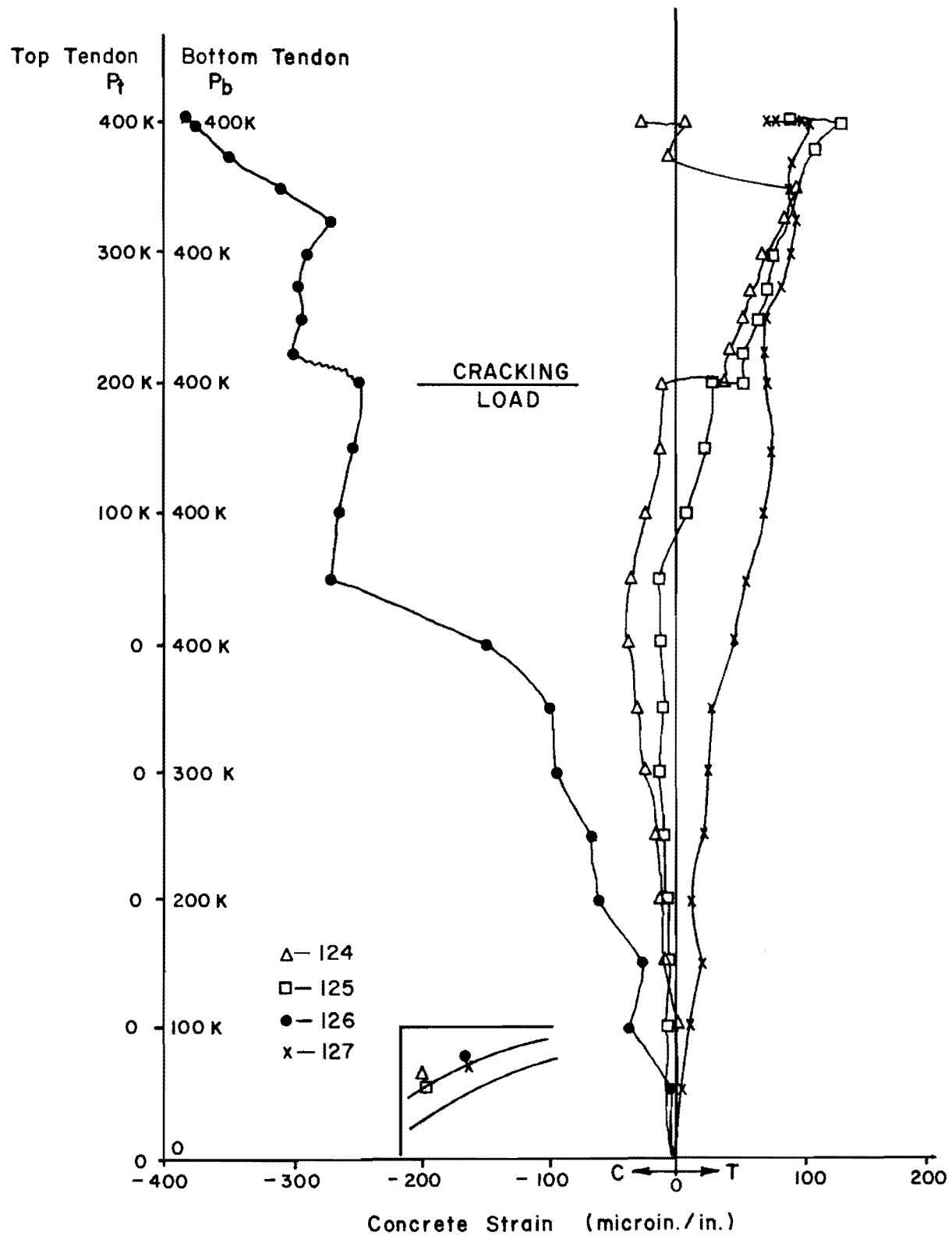


Fig. 5.7. Typical prototype concrete strains  
North web, mainspan unit

maximum tension, this value is simply an indication of general magnitudes of tensile stress in the zone. This value is less than the generally assumed values for concrete tensile strength. Interestingly, both Gages 124 and 125 indicate low compression strains during stressing of the lower tendon and then tensile strain tendencies during the stressing of the upper tendon. Pronounced tensile strains accompanied cracking, suggesting that the supplementary reinforcement in the immediate vicinity of the anchorage was effective in controlling cracking.

The occurrence and effect of cracking is distinctly more pronounced in the web reinforcement strain gage readings. Figure 5.8 indicates the measured strains in the third stirrup from the end of the unit, as measured in the immediate vicinity of the crack. All stirrups had extremely low strains prior to cracking (25 micro in./in. compression to 45 micro in./in. tension). However, when cracking occurred, the Grade 60 web reinforcement immediately was stressed between 11 and 26 ksi in various webs. When the crack formed during stressing of the lower tendon, the gage near the crack (Loc. 1) showed relatively small change (1.5 ksi) during stressing of the upper tendon. However, when cracking occurred during stressing of the second tendon, the gages on web reinforcement immediately adjacent to the crack showed substantial stress increase. At completion of stressing the strains measured in these gages indicated web reinforcement stresses of 25 to 34 ksi. These stresses were observed to decrease slightly as additional units were added, since the nature of shear stresses was to counteract the splitting stresses.

The strain measurements on the web reinforcement also indicated that the high strains were extremely local. For instance, Gage 118 and Gage 119 were mounted on the same stirrup and were approximately 12 in. apart. Gage 118 was in the immediate vicinity of the crack and indicates 29 ksi tensile stress at completion of stressing. Its immediate neighbor, Gage 119, indicates only 1.5 ksi tension at completion of stressing. This indicated the cracking effect to be very localized and most of the web reinforcement strength remained fully available to resist flexural and shear stresses.

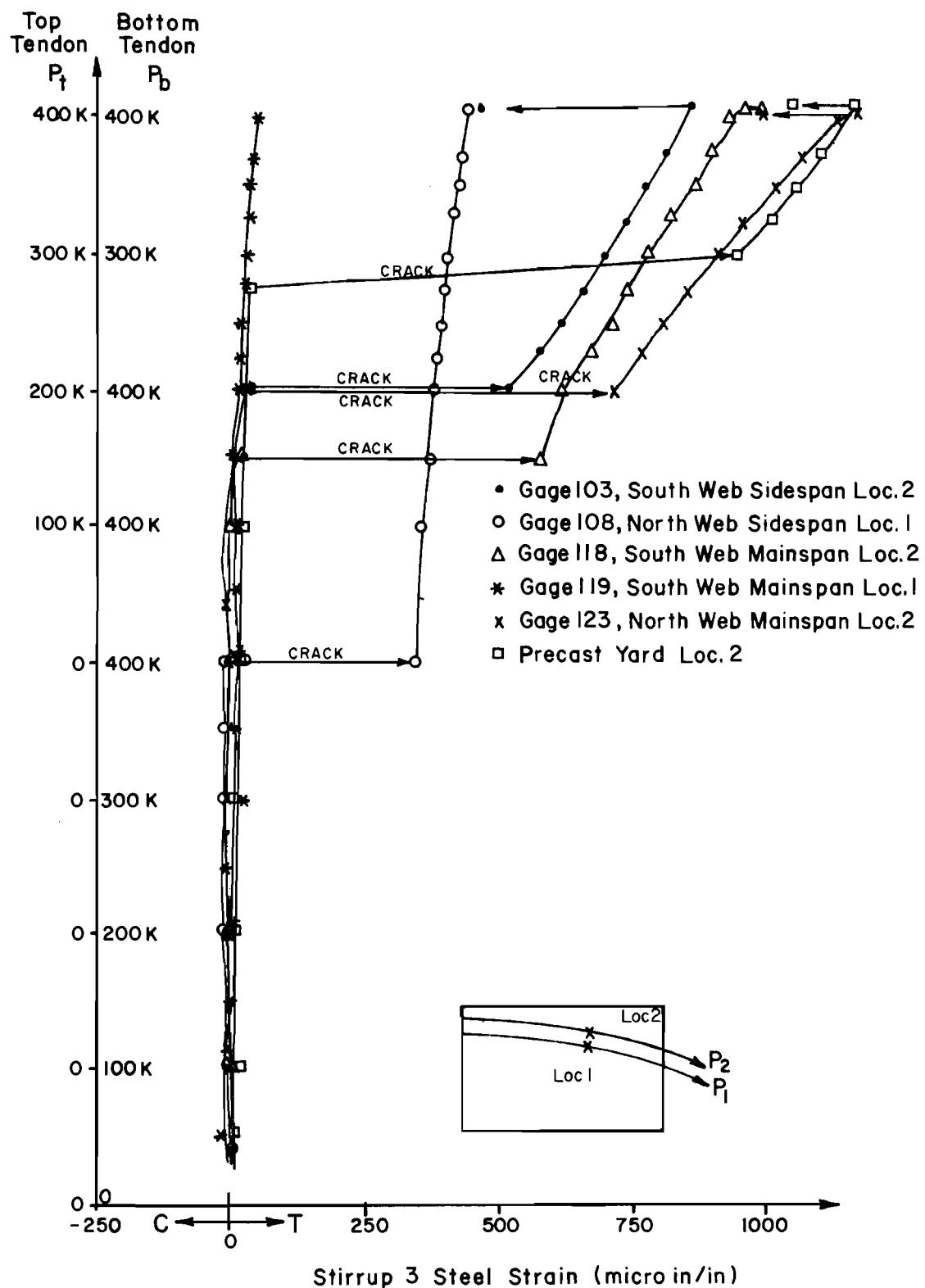


Fig. 5.8. Typical prototype stirrup strains

A crude set of calculations was made for a typical double 400k tendon web to indicate the expected level of tension stress when calculated by conventional methods of analysis. Using the equivalent prism method suggested by Guyon, the maximum bursting tensile stresses in the end zone due to the eccentric, inclined tendon forces [see Fig. 5.2(a)] would be about 340 psi. This should be less than the tensile strength of the concrete. However, this does not include any effect of radial forces due to tendon curvature [see Fig. 5.2(b)] or inclined wedge effect [see Fig. 5.2(c)]. Assuming an average tensile force distribution across the web width, the radial force due to tendon curvature in the upper tendon is approximately 240 psi. Thus, for a zone of 2 ft. to 4 ft. from the anchor, a minimum tensile stress of  $340 + 240 = 580$  psi would exist with no special consideration of the wedge effect of the conical anchor. It is probable that the actual average tensile stress exceeded 600 psi and that substantially higher local stresses existed. The section was thus vulnerable to splitting in the absence of sufficient supplementary splitting reinforcement which could contain such splitting internally.

### 5.7 Accelerated Box Girder Model Test

While construction was halted to evaluate the effect of the web cracking, three one-sixth scale model segments were constructed using the general model materials and procedures described in Report 121-5.<sup>22</sup> The principal changes from those procedures were: (1) the web reinforcement and top slab bars in the vicinity of the anchorages were purposely misfabricated in some webs to match a few isolated reports of construction errors, (2) conical anchors matching the geometrical dimensions of those used in the prototype were used, and (3) supplementary spirals used in the main model series were omitted.

Upon completion of three segments, they were erected in a cantilever fashion from a special reaction frame, as shown in Fig. 5.9(a). At the time of test,  $f'_c$  was 5.44 ksi. Web cracking identical to that noted in the prototype occurred in two of the three segments. This cracking appeared to be independent of the minor fabrication errors. The lack of substantial spirals was significant. It was concluded that the conical

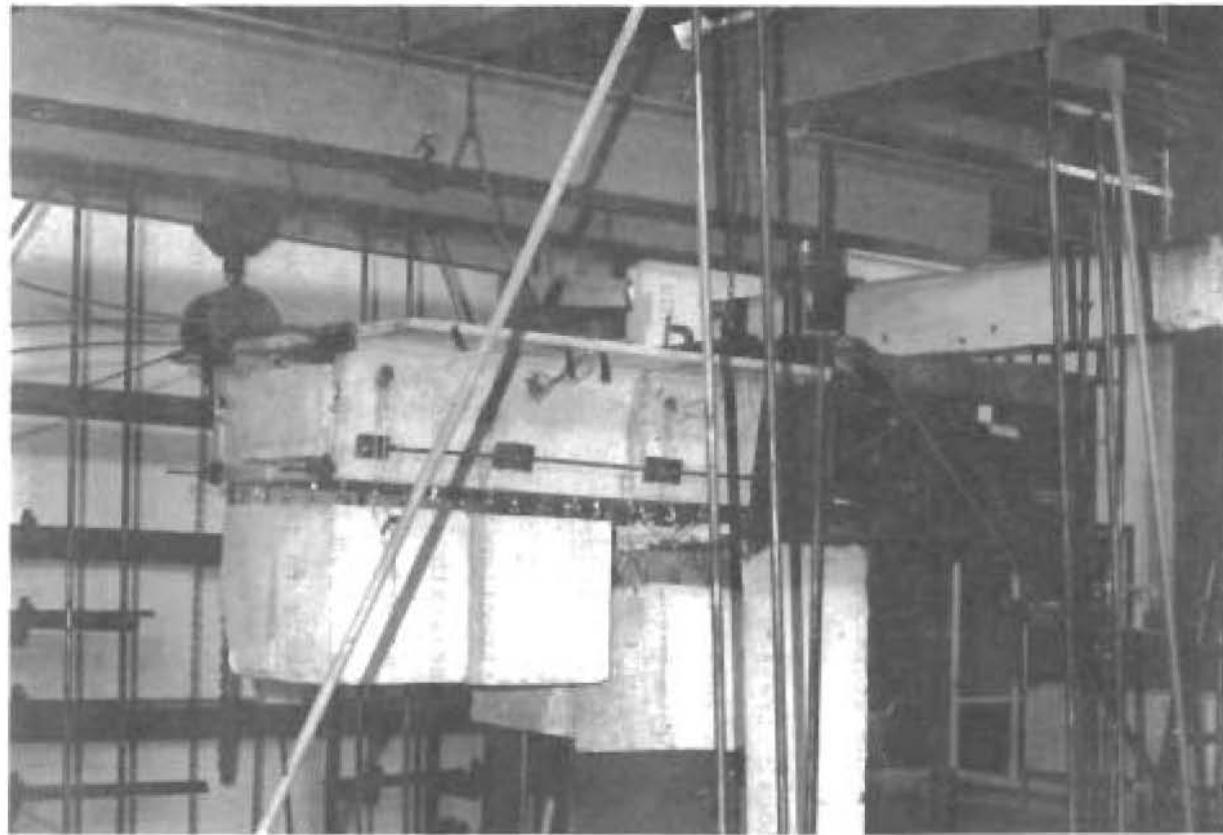


Fig. 5.9(a). Three segment model with conical anchors

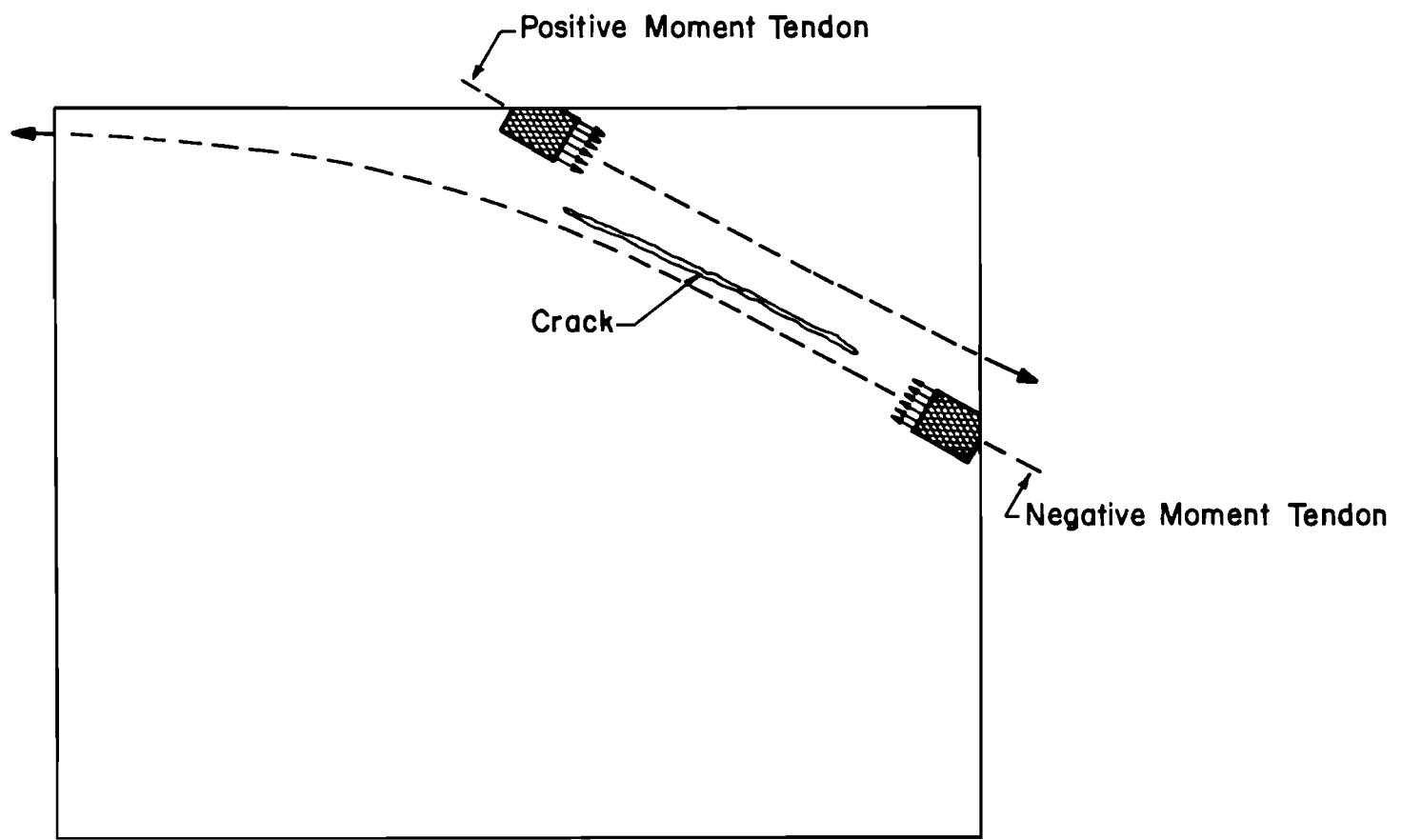


Fig. 5.9(b). Tendency for corner splitting off above negative moment tendon

anchorage and lack of spiral were the principal factors causing the premature cracking in the prototypes, as compared to the main model tests.

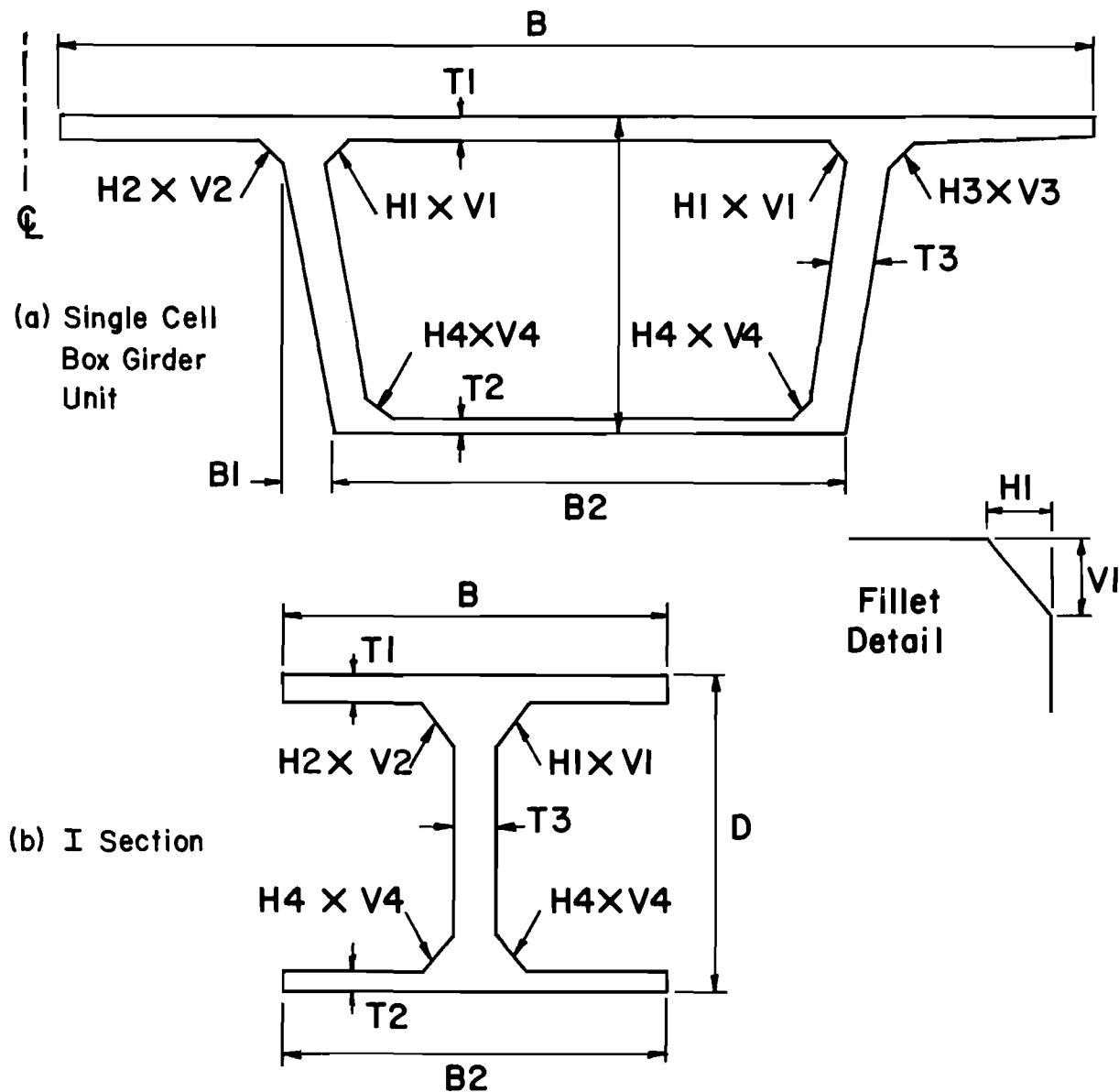
The completed cantilever unit then had positive moment tendons inserted and stressed to determine if the subsequent stressing of the positive moment tendons might result in either local distress in the vicinity of the positive tendon anchorages [see Fig. 5.9(b)] or might aggravate the web cracking. Web strain measurements indicated the cracks closed somewhat under the action of the positive moment cable stressing and no harmful effect was found. Superimposed loads were then applied and the reduced model taken to failure. No deleterious effect on shear strengths was found.<sup>22</sup> Based on the prototype measurements and this test, construction was allowed to be resumed.

## 5.8 Single Web Model Anchorage Investigation

Complete details for all specimens in this program were reported by Cooper.<sup>13</sup> This section summarizes the general procedures and most important results. All details and procedures used in construction were similar to those reported in Report 121-5 unless otherwise indicated.

### 5.8.1 Test Specimens

5.8.1.1 Specimen Geometry. A transverse cross section of a typical single cell box unit used in the prototype and in the main model study is shown in Fig. 5.10. Since the premature cracking along the tendon path experienced in the prototype was confined to the web, fabrication of an entire single cell box unit was considered unnecessary. Instead, a modified I section was chosen to conserve material and to permit easier visual inspection during testing [see Fig. 5.10(b)]. Critical web dimensions, i.e., thickness and depth, of the I section are identical to the typical single cell box unit. Thus the one-sixth scale factor remained unchanged. The flange width, B, was chosen to approximate flange action. Tests revealed the I section developed first cracking at relatively the same magnitude of load as the supplementary one-sixth scale models and the prototype. The cracking patterns were also similar.



	$B$	$B_1$	$B_2$	$T_1$	$T_2$	$T_3$	$D$	$H_1$	$V_1$	$H_2$	$V_2$	$H_3$	$V_3$	$H_4$	$V_4$
Prototype	336"	14"	156"	7"	6"	12"	96"	8"	6"	8"	6"	8"	6"	4"	4"
1/6 Scale Model	56"	2.3"	26"	1.17"	1"	2"	16"	1.33"	1"	1.33"	1"	1.33"	1"	0.67"	0.67"
I Section	20"	0"	20"	1.17"	1"	2"	16"	1"	1.38"	1"	1.38"	1"	1.38"	1"	1.38"

Fig. 5.10. Cross section of prototype, 1/6 scale model and I section

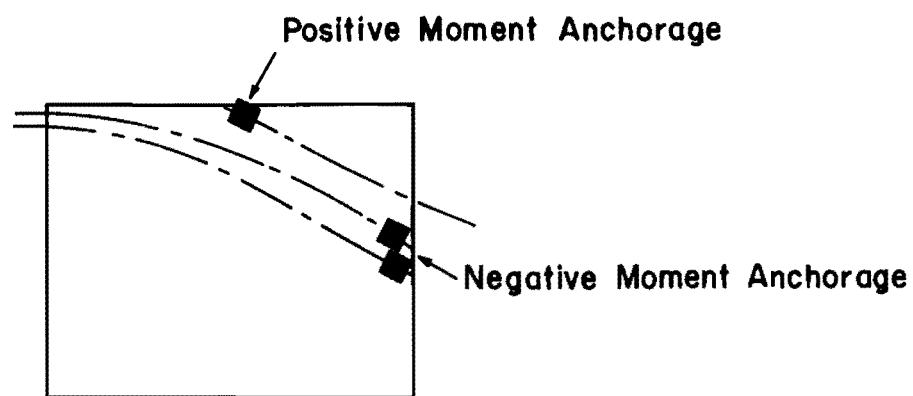
5.8.1.2 Specimen Reinforcement. Web reinforcement for this study was identical with the web reinforcement used in the fabrication of the one-sixth scale typical single cell box unit.<sup>22</sup> Flange reinforcement differed only in flange width, B. Reinforcement yield points varied from 71 to 79 ksi.

5.8.1.3 Tendon Profiles. Segments near the piers are unique in that one positive moment tendon and two negative moment tendons make large curvatures and must be anchored in that segment, as shown in Fig. 5.11(a) (Tendon Pattern A). Segments near midspan typically have only a single negative moment tendon which must be anchored, as shown in Fig. 5.11(b) (Tendon Pattern B).

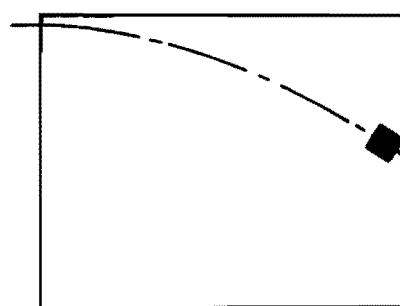
In this study, five specimens were post-tensioned using tendon pattern A, and ten specimens were post-tensioned using tendon pattern B. A profile of the negative moment tendons is shown in Fig. 5.12(a). Note that tendon pattern B had tendon No. 1 only. The profile of the positive moment tendon used in tendon pattern A is shown in Fig. 5.12(b). The positive moment tendon was never stressed, but was placed to simulate exact detail of the prototype.

Each specimen was post-tensioned using 3/8 in. diameter, 250 ksi strand. The average ultimate strength,  $f'_s$ , of this strand was 259 ksi ( $F = 22.0$  kips). Although this size strand is substantially larger than that required to exactly model the prototype tendon force, it was chosen so that forces above the design load of 11 kips per tendon could be achieved ( $0.8f'_{pu}$  for a 12 strand, 1/2 in. diameter, 270 ksi tendon using a scale factor of  $1/6 \times 1/6$  for tendon area). Thus, cracking patterns could be observed to a load approximately twice the maximum tendon design load or until the specimen failed.

5.8.1.4 Anchorages and Anchorage Reinforcement. Two different types of anchorages were used in this study--cone type and bearing type. The conical anchorage was a crude one-sixth scale model which simulated the external dimensions of the Prescon/CCL Strand System W conical anchorage used in the prototype bridge [see Fig. 5.13(a)]. Two sizes of bearing type anchorages were used. The smaller of the bearing anchorages, designated

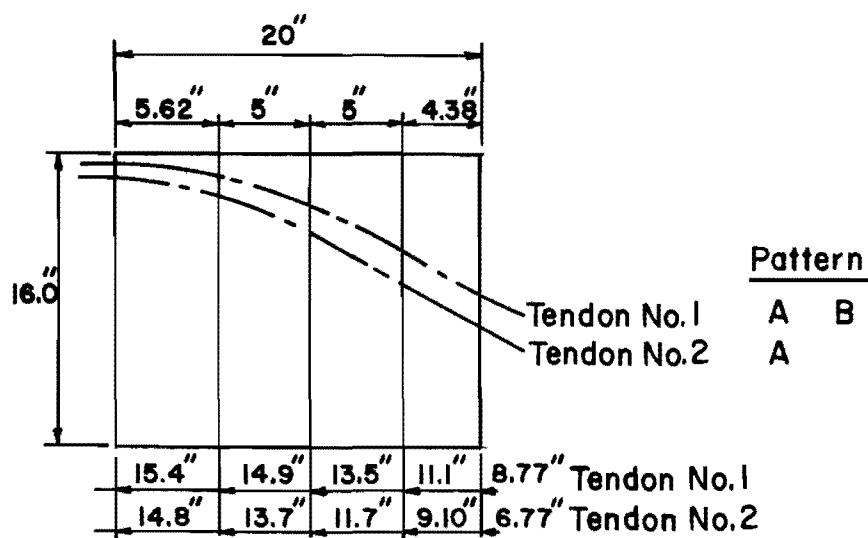


(a) Tendon Pattern A



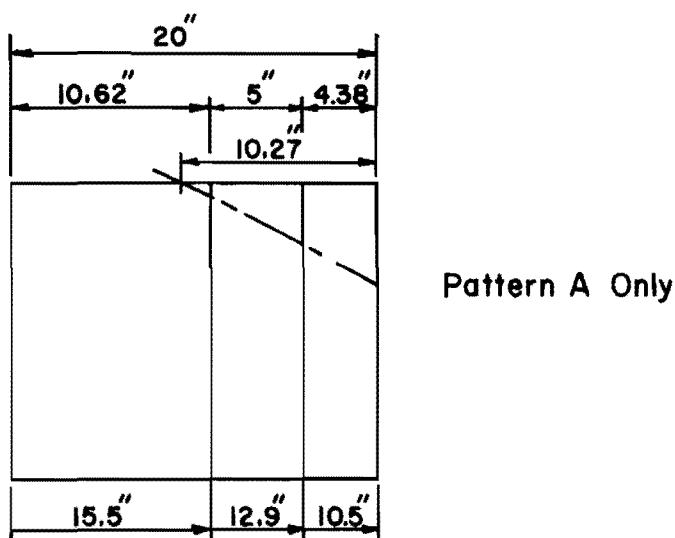
(b) Tendon Pattern B

Fig. 5.11. Tendon duct profiles



(a) Negative Moment Tendons

General Note: Dimension at Bottom indicates distance from Bottom of Precast unit to  $\frac{Q}{4}$  of tendon.



(b) Positive Moment Tendon

Fig. 5.12. Details of tendon profiles

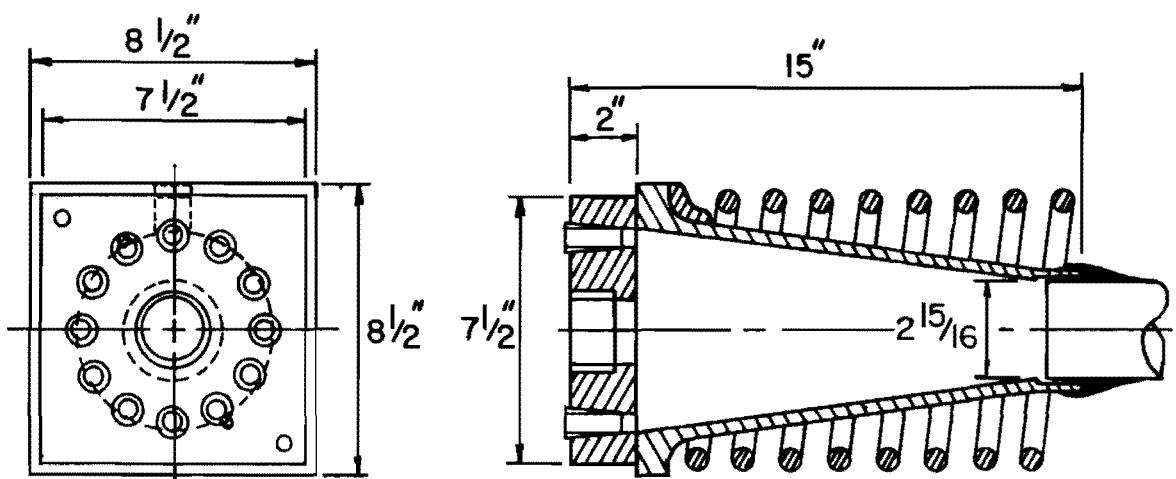
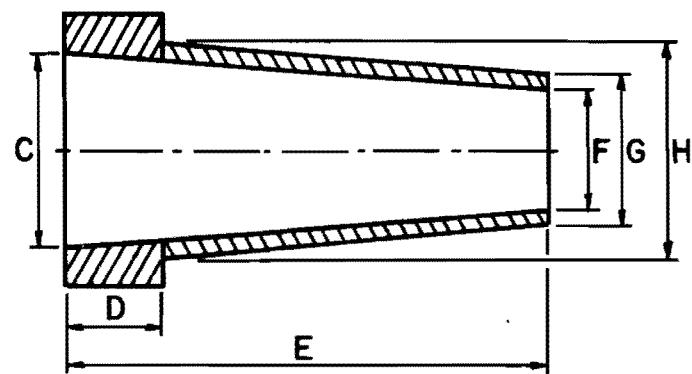
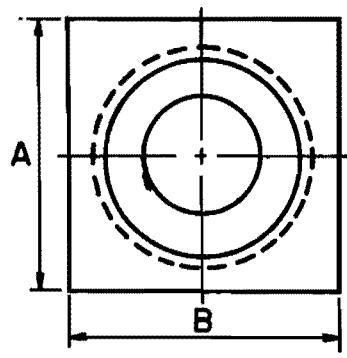
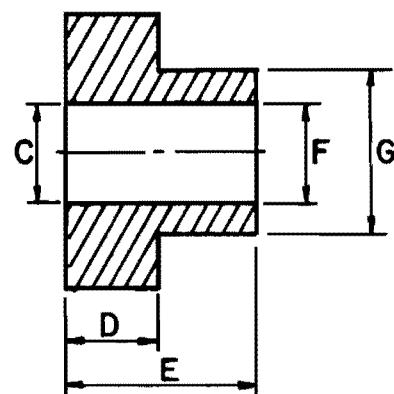
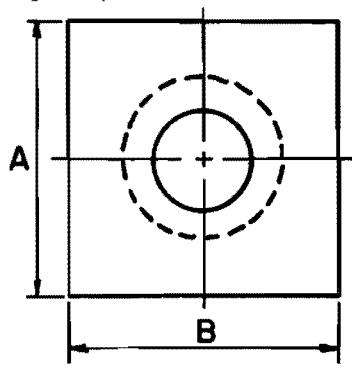
**a) Prototype Anchorage**

Fig. 5.13. Details of prototype and model anchorages

## b) Cone Type



## c) Bearing Type



Anchorage Type	A	B	C	D	E	F	G	H
Cone	1.42"	1.42"	0.95"	0.50"	2.5"	0.55"	0.80"	1.10"
SB	1.42"	1.42"	0.50"	0.50"	1.0"	0.50"	0.85"	—
LB	2.20"	1.42"	0.50"	0.50"	1.0"	0.50"	0.85"	—

Fig. 5.13 (Continued)

SB, has a bearing area equal to the projected area of the conical type anchorage. The larger of the bearing type anchorages, designated LB, has a bearing area approximately 1-1/2 times larger. Figure 5.13(b) and (c) show the anchorages used in this study.

The LB anchorage has the same bearing area as anchorages used in the one-sixth scale model bridge study. At the time of anchorage selection for the model study, the prototype plans allowed the choice of wedge type or bearing type anchorages. For the one-sixth scale model, the bearing type was chosen. The dimensions of the LB anchorage were sized to meet the requirements of then current ACI<sup>3</sup> and AASHO<sup>1</sup> codes. The ACI 318-63 allowable is:

$$f_{cp} = 0.6 f'_{ci} \frac{A'_b}{A_b} \quad (26-1)$$

where

$f'_{ci}$  = compressive strength of concrete at time of initial prestress

$A'_b$  = maximum area of the portion of the anchorage surface that is geometrically similar to and concentric with the area of the anchor plate of the post-tensioning steel

$A_b$  = bearing area of anchor plate of post-tensioning steel

Since the edge of the anchorage was flush with the interior edge of the web of the specimen:

$$\frac{A'_b}{A_b} = 1.0$$

and with

$$f'_{ci} = 6.5 \text{ ksi}$$

then

$$f_{cp} = 3.9 \text{ ksi}$$

Allowing approximately 10 percent increase in required anchorage area due to the presence of the hole, the required area becomes

$$A = \frac{11 \text{ kips}}{(3.9 \frac{\text{kips}}{\text{in.}^2})(0.90)} = 3.13 \text{ in.}^2$$

The dimensions chosen for the LB anchorage were 1.42 in. x 2.20 in. ( $A = 3.12 \text{ in.}^2$ ).

The SB anchorage was sized to give the same projected area as the cone type anchorage which modeled the prototype anchorage external dimensions. The projected area for these anchorages was  $A = 1.42 \text{ in.} \times 1.42 \text{ in.} = 2.02 \text{ in.}^2$  or only 65 percent of the area required for the bearing type anchor to meet the ACI and AASHO requirements. The vendor for the prototype anchorage claimed that the provision of the spiral around the trumpet [see Fig. 5.13(a)] had produced test results which indicated the anchorage would transfer the full tendon load without damage.

Each specimen also had supplementary reinforcement to match the details used in the prototype, as shown in Fig. 5.14. This detail was constant throughout this study for all anchorages, with the exception of Set 7, Specimen 1, which had a different spiral detail.

5.8.1.5 Concrete. Two microconcretes were used in this study. Both utilized reduced aggregate having a similar gradation to ordinary concrete aggregate. Both microconcretes were designed to have similar properties ( $E_c$  and  $f'_c$ ) as prototype concretes. Mix design K was intended to give  $f'_c$  of 6000 psi, and mix design G was intended to give  $f'_c$  of 5000 psi.

5.8.1.6 Fabrication of Specimens. Fabrication of specimens followed a uniform procedure similar to the main model study. Some modifications and additions such as rerouting bars around anchorages, ducts, and shear keys had to be made to each of the basic cages before casting. Spiral reinforcing and U-shaped ties were added at the anchorages, and tendon ducts were placed into proper position, as shown in Fig. 5.15(a).

To minimize the possibility of honeycombing, plexiglass forms were used to permit visual inspection during casting [see Fig. 5.15(b)]. Two separate forms were used. These forms are identical with the exception of the right-hand - left-hand nature of the anchorage unit (see Fig. 5.16). Only Set 8, Specimen 1; Set 9, Specimen 1; and Set 10, Specimen 1 used the right-hand form. Figure 5.16 indicates that the anchorage unit is offset to one side of the web. This detail also occurred in the prototype. The purpose for the offset is to permit easier removal of the forms and to prevent spalling around the anchorage during the removal of the forms.

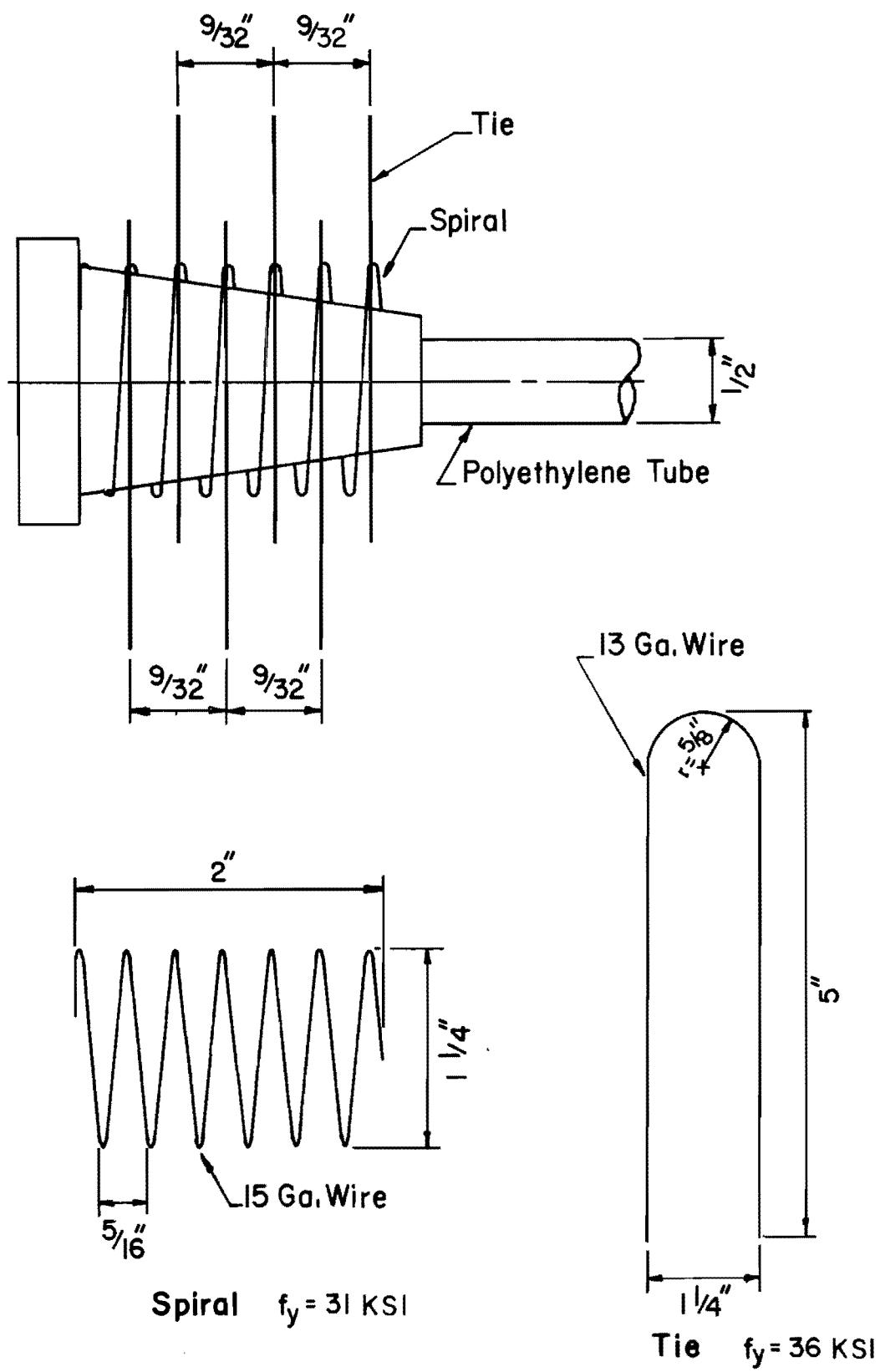
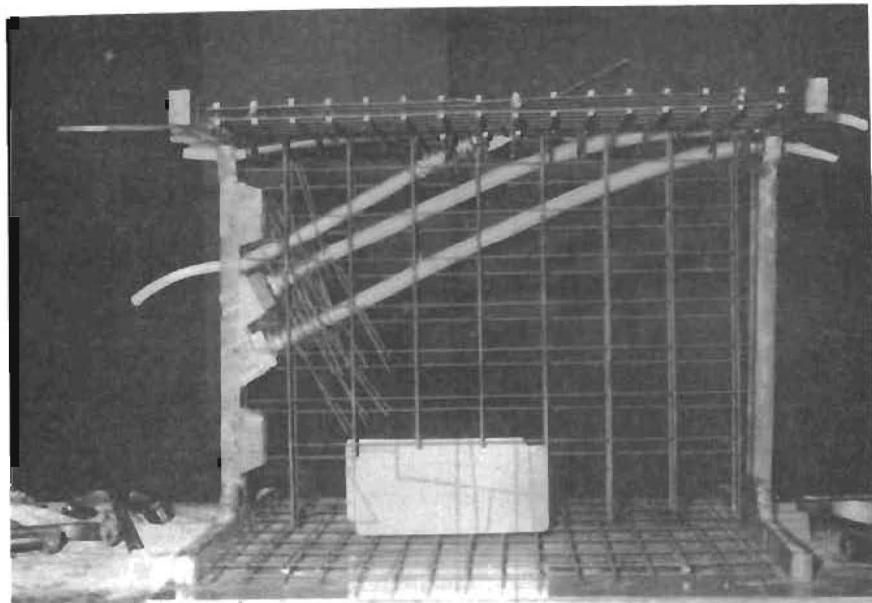
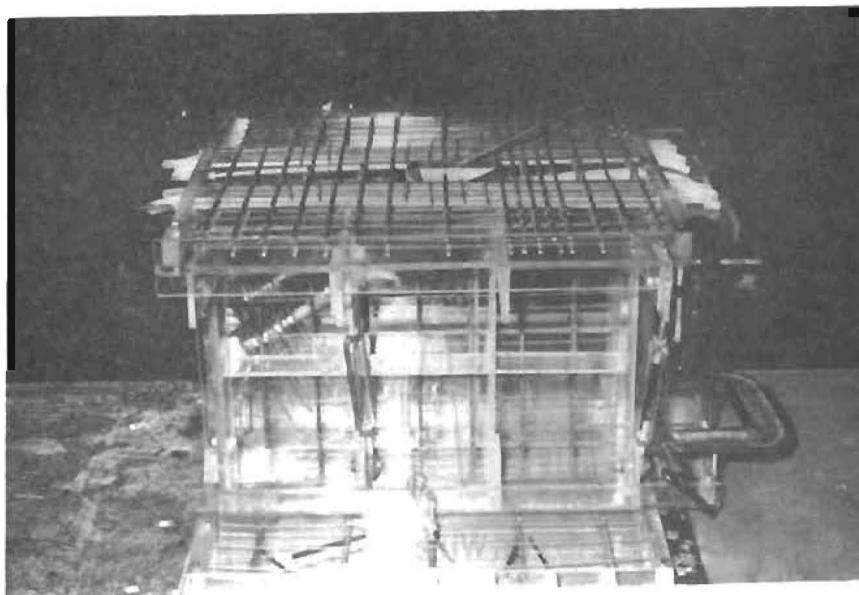


Fig. 5.14. Details of anchorage reinforcement



(a) Cage reinforcement before placement of forms



(b) General view of assembled form

Fig. 5.15. View of cage reinforcement and assembled forms

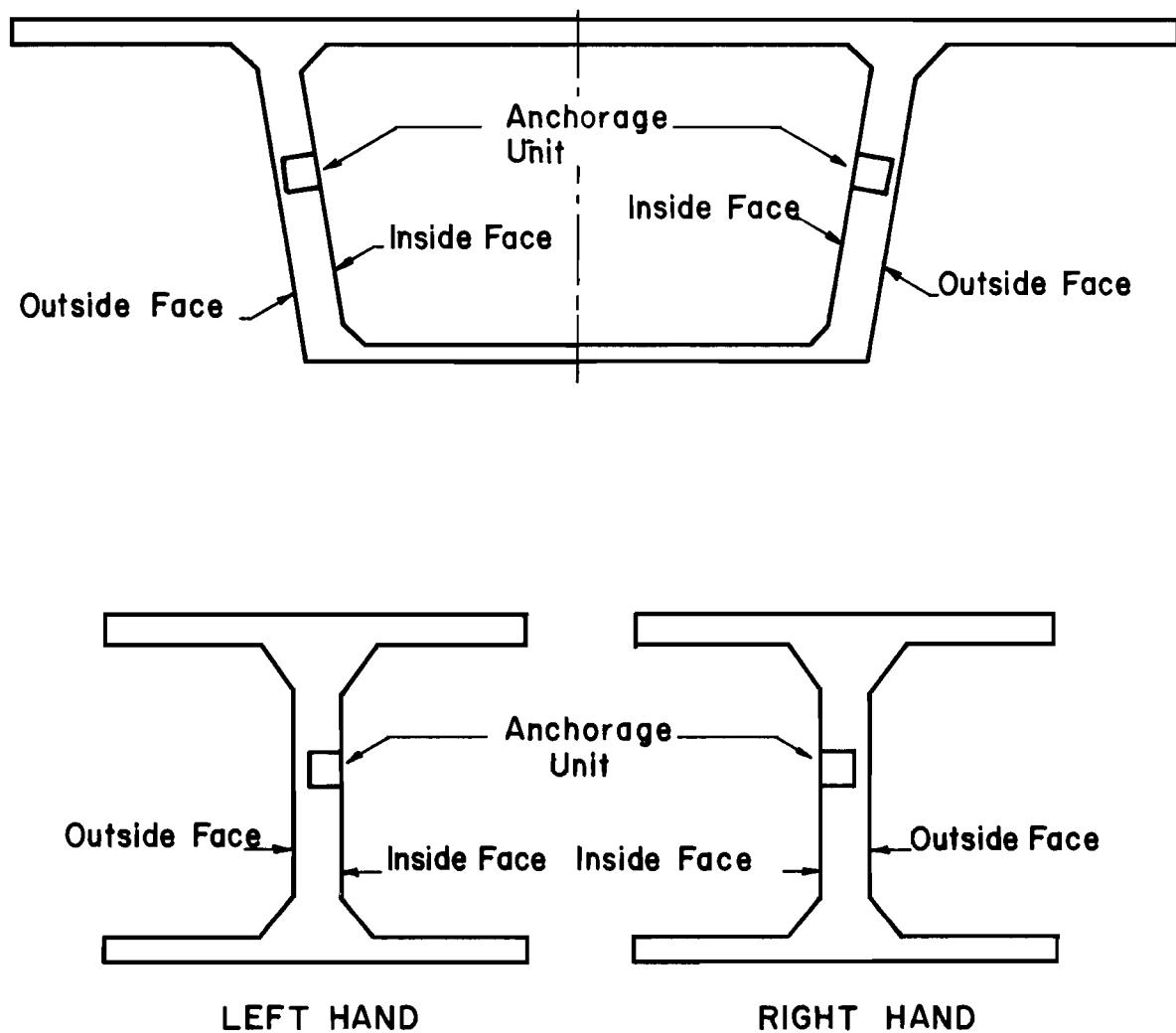


Fig. 5.16. Right-hand - left-hand nature of anchorage unit

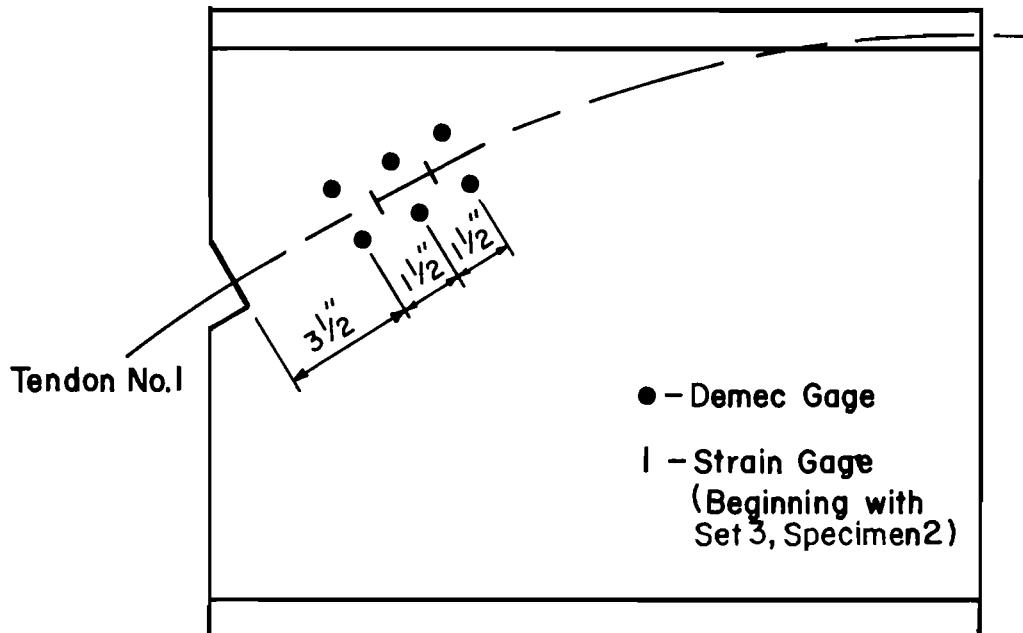
5.8.1.7 Specimen Instrumentation. Instrumentation for this study consisted of a mechanical extensometer (Demec) with appropriate gage points (Demec gages) and paper straining gages placed on the smooth surface of the concrete along the tendon path. The primary purpose of this instrumentation was to detect first cracking. The gage length of the Demec gages was 2 in. and the gage length of the paper strain gages was 0.64 in. Because the gage length of the Demec gages was approximately three times longer than the gage length of the strain gages, the Demec gages proved more useful as indicators of first cracking since first cracks which would tend to miss the shorter paper strain gages would still pass through the longer Demec gage length.

Experience gained during the construction phase of the prototype bridge indicated that the first cracks appeared on the inside face of the web. Therefore, the Demec gages were always placed on the inside face of the web. Paper strain gages were placed on the outside face of the web, and beginning with Set 3, Specimen 2, additional strain gages were placed on the inside face of the web. Figure 5.17 shows the position of the Demec gages and paper strain gages. For specimens with two negative moment tendons, tendon pattern A, the Demec gages were always placed along the top tendon.

Figure 5.18 shows a typical plot of a Demec gage and a corresponding strain gage on the opposite face of the web. It can be seen that the first crack is detected by the Demec gage and that the crack is not detected by the paper strain gage until the crack has propagated through the web at a higher load. From Fig. 5.18, it is evident that the Demec gages are more useful as indicators of first cracking. Strain gages placed on the inside face of the web showed better agreement with the Demec gages; however, all plots of load versus strain will use strain readings taken from the Demec gages.

5.8.2 Loading System. The loading frame used in this study is shown in Figs. 5.9 and 5.19. The frame supported the segment and provided a means for minimizing seating losses by permitting use of longer tendons than the length of one segment. Load was applied by a 30 ton hydraulic ram and the force was controlled by a load cell. As checks, the hydraulic

## (a) Demec Gage Locations



## (b) Strain Gage Location

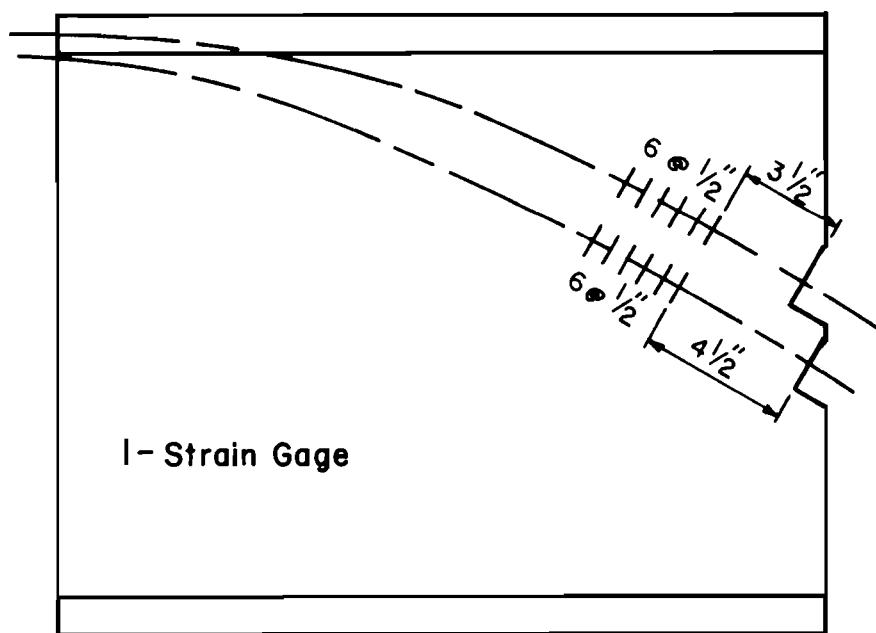


Fig. 5.17. Location of specimen instrumentation

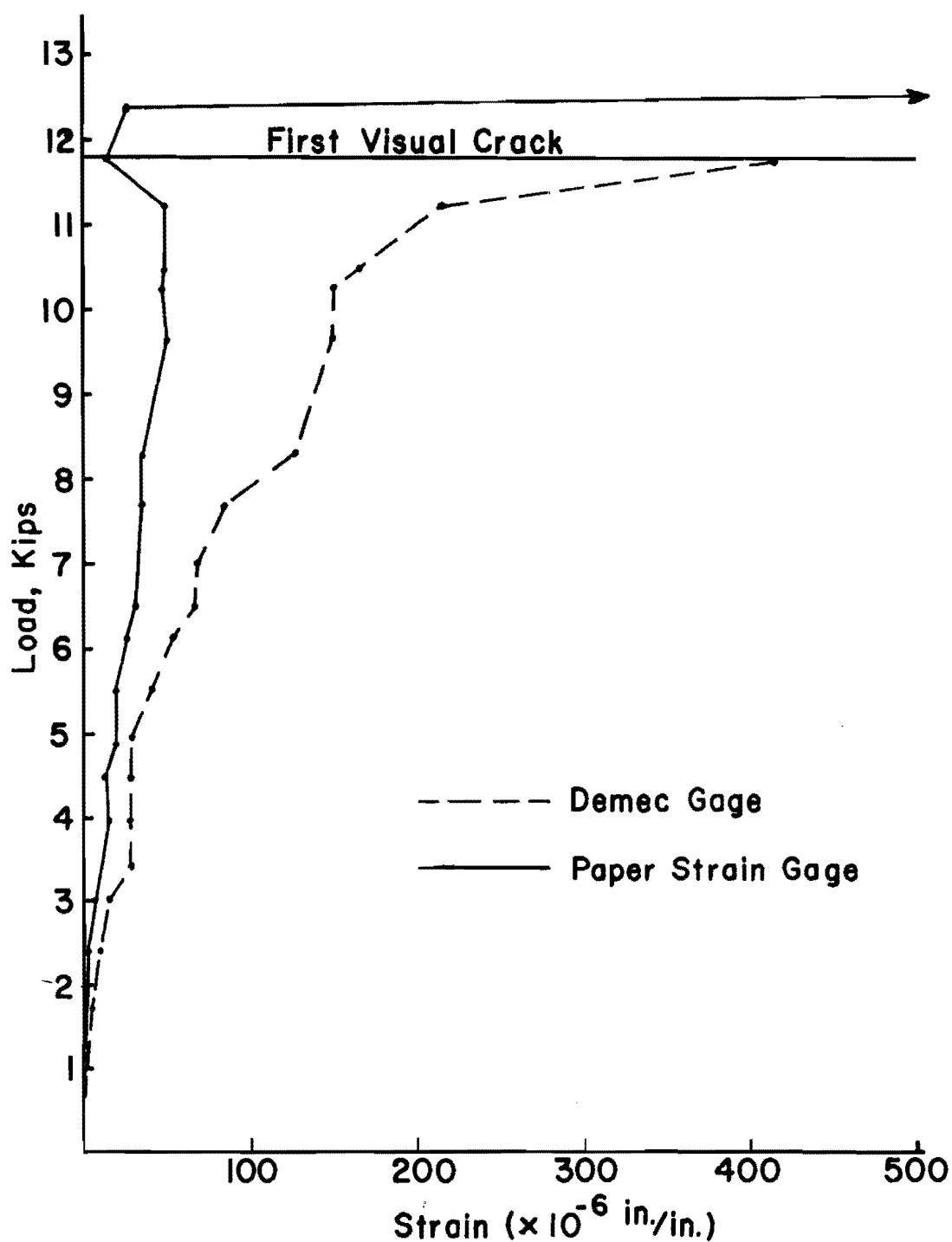


Fig. 5.18. Comparison of Demec gage and strain gage

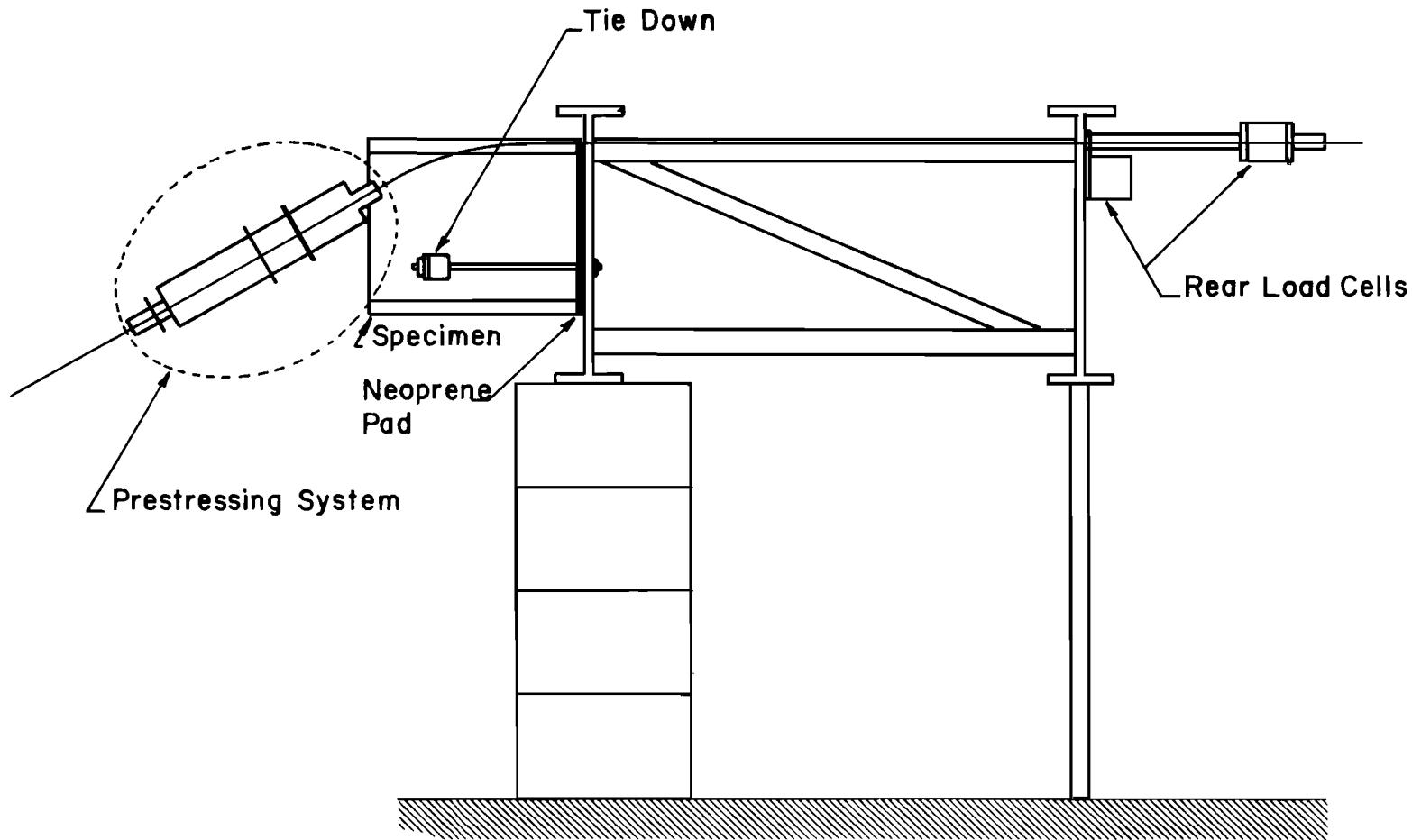


Fig. 5.19. Loading frame

pressure was monitored with a pressure transducer and the force at the rear of the frame was also measured using a load cell.

#### 5.8.3 Test Procedure

5.8.3.1 Single Tendon Specimens (Tendon Pattern B). Load was typically applied in 1 kip increments. The design load was 11 kips per strand. When approximately 2 kips had been applied, a 350 lb. block of concrete was set on top of the specimen in an attempt to duplicate the dead load requirements to satisfy similitude. At every load stage all instrumentation data were recorded and cracks, if any, were marked. Each load stage took approximately 3 minutes. Loading was stopped when the force in the strand approached 20 kips.

5.8.3.2 Double Tendon Specimens (Tendon Pattern A). When testing double tendon specimens the bottom tendon was stressed first. This was the same stressing sequence used in the construction of the prototype and the one-sixth scale model. The design load was 11 kips per tendon or a total of 22 kips. Load was applied in the same manner as for a single tendon specimen up to a level of 11 to 13 kips. At this level of load the lower tendons were anchored. Generally, no more than 15-20 percent loss occurred during seating. The top tendon was then stressed to the same level as the bottom tendon. When the forces in the two strands were approximately equal, a ram on the bottom tendon in the rear of the test frame was engaged and the test proceeded with both tendons being stressed simultaneously. The test was terminated when the force in each strand approached 20 kips, or when failure occurred.

5.8.4 Test Results. This report will briefly describe each specimen and summarize major results such as cracking load, crack development and other aspects of behavior. The magnitude of load,  $P$ , is in kips. When the load is on only the bottom tendon of a pair, it will be called  $P_b$ . When the load is on the top tendon in addition to the bottom tendon, the load in the top tendon will be called  $P_t$  and the total load on the tendon group will be termed  $P_G$ . The load will be followed by a symbol giving the ratio of load with respect to the design load,  $P_D$ , based on  $0.8f_{pu}$ . For individual tendon anchorages,  $P_{D_b}$  or  $P_{D_t}$  is 11 kips. For the double

anchorage groups,  $P_{DG}$  is 22 kips. First cracking as used in this report will designate when the first crack is noticed by the naked eye.

For some cases basic test data will be presented in graphical form as a plot of load applied to the anchorage versus strain recorded from the Demec gages up to a level corresponding to the appearance of the first crack. Data beyond this level are not presented, since after first cracking strain values became extremely large and go off scale. In addition, some Demec gages exhibited erratic behavior after the appearance of the first crack. Figure 5.20 shows a typical plot of load versus strain which indicates erratic behavior after cracking. Figure 5.21(a) indicates position of Demec gages and cracks numbered in typical order of their appearance. From Fig. 5.20, the occurrence of the first crack is evident from the large increase in strain. Figure 5.21(b), (c), and (d) represent the formation of cracking and ensuing erratic behavior of the Demec gages.

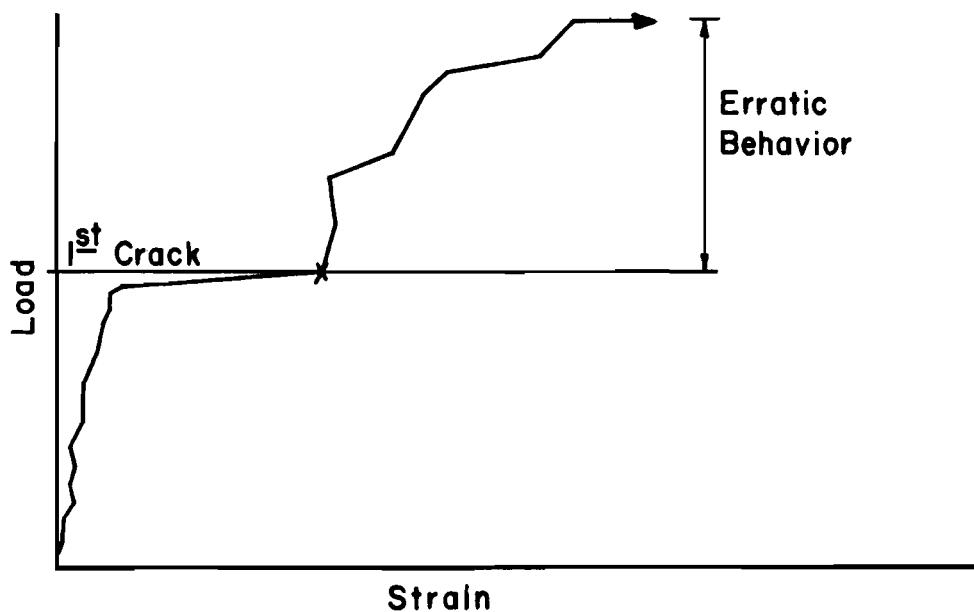
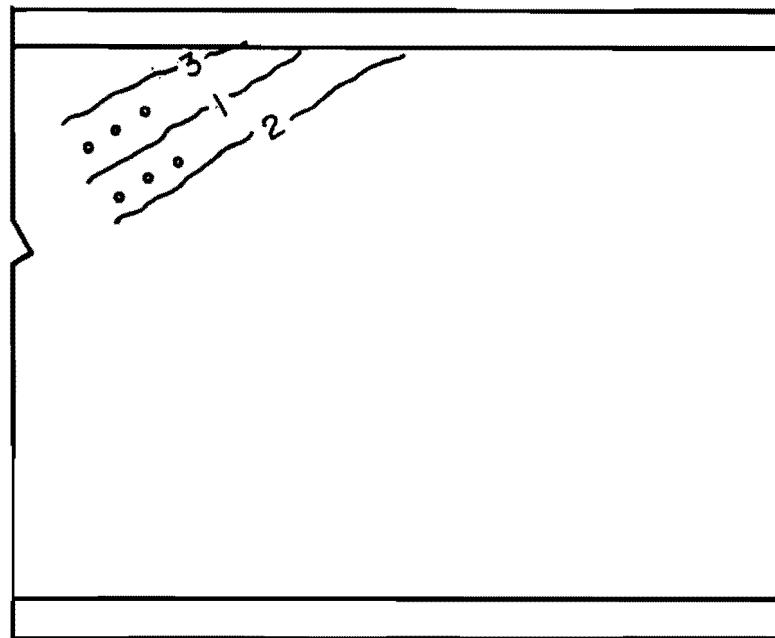
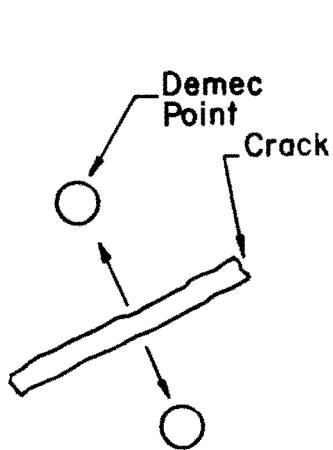


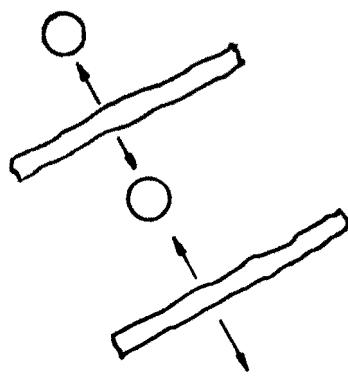
Fig. 5.20. Erratic behavior of typical Demec gage after first cracking



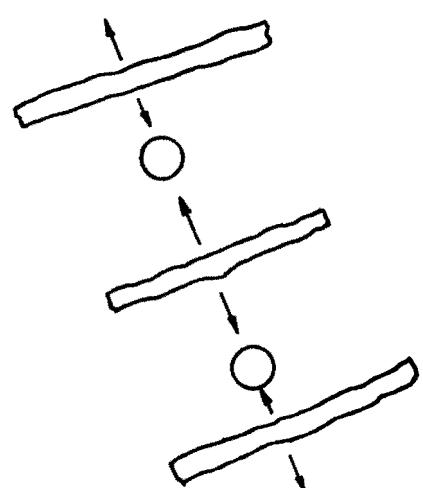
(a) Typical cracking sequence showing relative Demec gage locations.



(b) Initial Crack



(c) Second Crack



(d) Demec Points Floating

Fig. 5.21. Explanation of erratic behavior

When the first crack appears, the deformation tends to separate the Demec gage [see Fig. 5.21(b)]. Upon formation of a second and third crack, the deformations, indicated by arrows, will tend to oppose each other and the Demec gages tend to "float" between the cracks [see Fig. 5.21(c) and (d)]. Thus, strain readings taken after first cracking often fluctuate and provide no meaningful information. Reference 13 contains full descriptions and photos of each specimen. Where photographs are given, they are views of the inside face. All cracks have been accentuated by a black marking pen.

#### 5.8.5 Series 1 Specimens

5.8.5.1 General. At the beginning of this exploratory study, establishment of a credible model was imperative. Thus, the basic purpose of the Series 1 specimens was to establish credibility between the laboratory model and the actual prototype, and to obtain results which could later be used for comparison with results of other anchorage zone details.

During the fabrication of the prototype box units, a series of unfortunate errors occurred which were documented in random photographs. In some units, web and top mat reinforcing bars were cut to place anchorages and in some instances these bars were never replaced. This omission of steel often occurred in units which had two negative moment tendons (tendon pattern A).

To establish the credibility of the model I section, six specimens were fabricated and tested. Bars were cut and omitted to ensure exact correspondence with prototype and supplemental model units. Four of the specimens had tendon pattern A. Of the four, two had bars cut which were not replaced; the other two had bars cut with partial replacement. The remaining two specimens had tendon pattern B, and partial bar replacement was observed.

All Series 1 specimens used cone type anchorages and anchorage zone details like those of the prototype.

5.8.5.2 Set 1, Specimen 1. This specimen had tendon pattern A. Several cut bars were not replaced and stirrup hooks were shortened. Mix design K was used and  $f'_c$  was 6.3 ksi. The typical loading sequence as

described in Sec. 5.8.3.2 was waived for Set 1, Specimen 1, only and instead the bottom tendon was loaded in considerable excess of its design load,  $P_{D_b}$  of 11 kips, up to a level of  $P_b = 17.1$  kips. This was done in order to study the cracking pattern and extent of distress which would be experienced with load applied only to the bottom tendon.

The appearance of the first visual crack occurred on the inside face of the web along the top tendon. The load was 8.8 kips ( $0.8 P_{D_b}$  or  $0.4 P_{D_G}$ ). The plot of load versus strain is shown in Fig. 5.22. The crack began approximately 4 in. from the top anchorage and extended 3 in. At 13.4 kips ( $1.22 P_{D_b}$  or  $P_{D_G}$ ), this first crack had extended 1 in. in each direction. Also, at this level a crack appeared on the outside face of the web running along the top tendon. At 15.9 kips ( $1.44 P_{D_b}$  or  $0.72 P_{D_G}$ ), a crack along the bottom tendon appeared on both sides of the web running parallel to the crack along the top tendon. The crack was 11 in. long and began 3-1/2 in. from the bottom anchorage plate. After achieving 17.1 kips ( $1.56 P_{D_b}$  or  $0.78 P_{D_G}$ ), the load was released. The following day the test was resumed with both tendons being loaded simultaneously in an attempt to determine if the specimen would fail under a high level of load being applied at the anchorages. The highest level of load achieved was approximately 40 kips,  $1.82 P_{D_G}$ . The test was terminated at this point because the load in each strand was approaching the ultimate strength of the strand. Although the specimen did not fail, the cracks were extremely wide, especially in the region of greatest curvature of the tendon, i.e., at midlength of the specimen in the fillet region. Cracks were much narrower as they approached the vicinity of the anchorage. Figure 5.23(a) shows Set 1, Specimen 1, after completion of the test.

5.8.5.3 Set 1, Specimen 2. This specimen had tendon pattern A. Three web bars were cut and not replaced. Several top bars were altered and not replaced. Mix design K was used and  $f'_c$  was 4.43 ksi.

While stressing the bottom tendon the first visual crack appeared along the top tendon at a load of 7.0 kips ( $0.64 P_{D_b}$  or  $0.32 P_{D_G}$ ). At 13.1 kips ( $1.20 P_{D_b}$  or  $0.60 P_{D_G}$ ), a crack appeared on the outside face of the web along the top tendon. A crack opened along the bottom tendon on

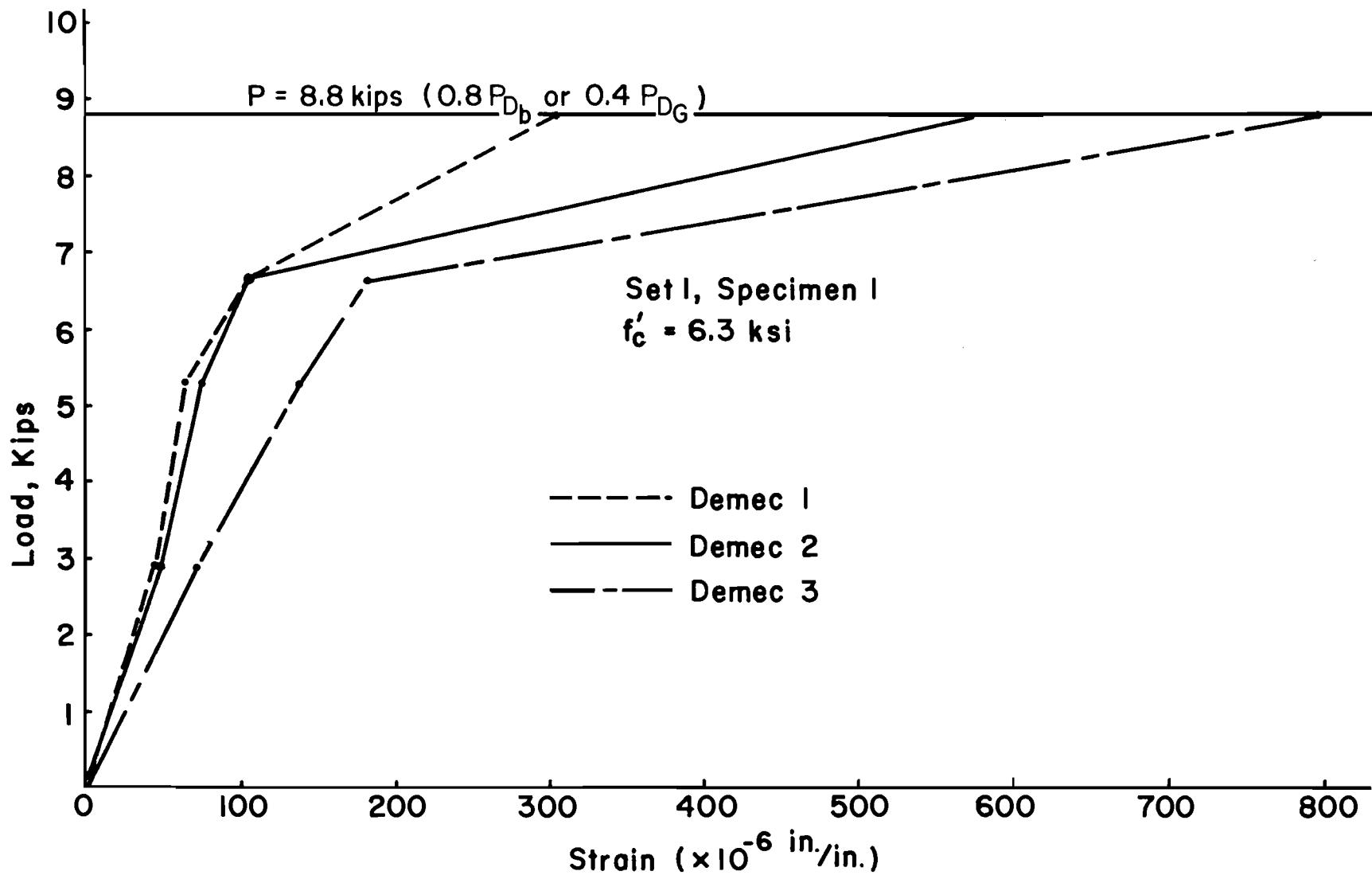


Fig. 5.20. Plot of load versus strain for Set 1, Specimen 1

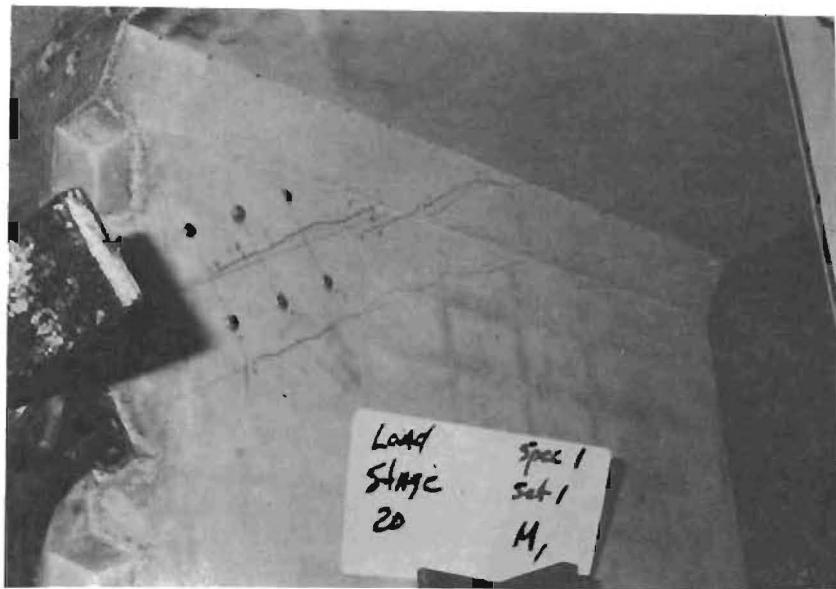


Fig. 5.23(a) Set 1, Specimen 1, after completion of the test

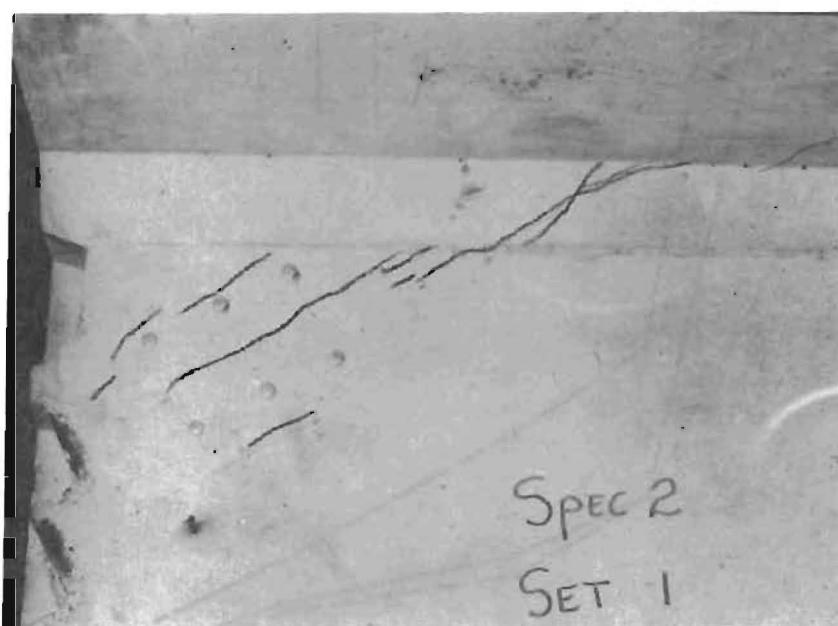


Fig. 5.23(b) Set 1, Specimen 2, after completion of the test

the inside face of the web at 30.8 kips,  $1.4 P_{D_b}^{D_G}$  (17.8 kips bottom tendon,  $1.62 P_{D_b}$ , plus 13 kips top tendon,  $1.18 P_{D_t}^{D_G}$ ). At this same level, a crack also appeared above the top tendon on the inside face of the web. The highest load level attained was 35 kips,  $1.59 P_{D_b}^{D_G}$  (20 kips bottom tendon,  $1.82 P_{D_b}$ , plus 15 kips top tendon,  $1.36 P_{D_t}^{D_G}$ ). Although the specimen did not actually fail at this load, it appeared to be on the verge of failure. Cracks were extremely wide in the area of greatest curvature of the tendons. Nearer the anchorages the cracks were much narrower. Figure 5.23(b) shows Set 1, Specimen 2, after completion of the test.

5.8.5.4 Set 1, Specimen 3. This specimen had tendon pattern B. No web bars were altered and only two of the altered top bars were not replaced. Mix design K was used and  $f'_c$  was 5.73 ksi.

The first visual crack in the test of this specimen occurred in a manner unlike any other specimen in this research program. Load was applied to a level of 3.6 kips,  $0.33 P_{D_t}^{D_t}$ , using the loading sequence as described in Sec. 2.4.2. However, at this level of load an equipment malfunction occurred which required that the test be postponed until the following day. Before releasing the load, a careful inspection of the specimen was conducted which revealed the specimen was uncracked. Following this inspection the load was then released and another inspection followed. This inspection revealed a small crack on the inside face of the web (along the tendon path). The crack began 4-1/4 in. from the anchorage plate and was 1-1/2 in. long.

The following day the test was resumed. The crack which had formed on the previous day propagated even at relatively low levels of load. At 11.8 kips,  $1.07 P_{D_t}^{D_t}$ , a crack appeared on the outside face of the web along the tendon path. Loading was terminated at 18.4 kips,  $1.67 P_{D_t}^{D_t}$ . The cracks on both sides of the web had extended to the anchorage plate and were extremely wide. In addition, spalling was evident immediately in front of the anchorage on the inside face of the web.

5.8.5.5 Set 2, Specimen 1. The details of this specimen are precisely like the details of Set 1, Specimen 1, except it used mix design G and  $f'_c$  was 4.89 ksi.

The appearance of the first visual crack occurred simultaneously on both sides of the web. The crack was approximately 5 in. long and began 6 in. from the anchorage plate extending up into the fillet in the region where the tendons have greatest curvature. The crack appeared between the two tendons at a load of 8.8 kips ( $0.8 P_{D_b}$  or  $0.4 P_{D_G}$ ). In subsequent load stages this crack propagated along the bottom tendon path towards the anchorage plate. At 15.7 kips a large crack was observed along the top tendon on both sides of the web. Upon reaching 33.2 kips,  $1.51 P_{D_G}$ , a sudden explosive failure of the concrete occurred on the inside face of the web, as shown in Fig. 5.24.

5.8.5.6 Set 2, Specimen 2. The details of this specimen are precisely like the details of Set 1, Specimen 2, except it was cast using mix design G and  $f'_c$  was 4.88 ksi.

Due to an inadvertant loading error, the cracking load for Set 2, Specimen 2, was not determined. However, the crack pattern for this specimen is similar to the other specimens in this series. The first visual crack appeared on the inside of the web along the bottom tendon path.



Fig. 5.24. Set 2, Specimen 1, after explosive failure

The crack began 4 in. from the anchorage plate and extended 2 in. At 15.4 kips a crack formed 1-1/2 in. above the top tendon on the inside face of the web. Upon reaching 33 kips,  $1.5 P_{D_t}^G$  (18.2 kips bottom tendon,  $1.66 P_{D_b}$ , plus 14.8 kips top tendon,  $1.35 P_{D_t}^G$ ), a sudden explosive failure of the concrete occurred on the inside face of the web.

5.8.5.7 Set 2, Specimen 3. The details of this specimen are precisely like the details of Set 1, Specimen 3, except it was cast using mix design G and  $f'_c$  was 4.87 ksi.

The first visual crack appeared at 9.5 kips,  $0.86 P_{D_t}^G$ , running along the tendon path on the inside face of the web. A crack on the outside face of the web opened at 12 kips,  $1.09 P_{D_t}^G$ . Loading was terminated at 18.8 kips,  $1.71 P_{D_t}^G$ . No failure occurred.

#### 5.8.6 Series 2 Specimens

5.8.6.1 General. The purpose of the Series 2 specimens was to examine the effect of anchorage geometry on cracking load. In this series five specimens were tested. All specimens used bearing type anchorages. Each specimen also had normal reinforcement details, i.e., all bars which were cut for anchorage insertion or duct placement were replaced.

5.8.6.2 Set 3, Specimen 1. This specimen had tendon pattern B. The anchorage type was SB. It was cast using mix design K and  $f'_c$  was 7.48 ksi.

The first visual crack appeared at 12.6 kips,  $1.15 P_{D_t}^G$ , running along the tendon path on the inside face of the web. A crack on the outside face of the web opened at 17.8 kips,  $1.62 P_{D_t}^G$ , along the tendon path in the vicinity of the fillet. Loading was terminated at 19.9 kips,  $1.81 P_{D_t}^G$ . No failure occurred. The cracks were extremely small, but again were wider near the vicinity of maximum curvature of the tendon. Upon releasing the load cracks were barely visible to the naked eye.

5.8.6.3 Set 3, Specimen 2. This specimen had tendon pattern A. The anchorage type was SB with the exception of the positive moment anchorage which was conical. By not altering the geometry of the positive moment anchorage, the results obtained can be directly compared with

corresponding Series 1 specimens without introducing an additional variable. Mix design K was used and  $f'_c$  was 6.62 ksi.

At 17.2 kips,  $0.78 P_{DG}$  (9.8 kips bottom tendon,  $0.89 P_{Db}$ , plus 7.4 kips top tendon,  $0.67 P_{Dt}$ ), the first visual crack appeared on the inside face of the web running along the bottom tendon. The crack began 4-3/4 in. from the anchorage plate and extended 3 in. A 5 in. crack opened along the top tendon on both faces of the web at 24.5 kips,  $1.11 P_{DG}$ . The highest load level was 44 kips,  $2.0 P_{DG}$ . At this load slight spalling was beginning to occur immediately in front of the bottom anchorage. However, no failure occurred.

5.8.6.4 Set 4, Specimen 1. This specimen had tendon pattern B. The anchorage type was SB. Mix design G was used and  $f'_c$  was 5.55 ksi.

The first visual crack appeared at 12.1 kips,  $1.1 P_{Dt}$ , running along the tendon path on the inside face of the web. The crack began 5 in. from the anchorage and extended 2-1/2 in. At 13.5 kips,  $1.23 P_{Dt}$ , a crack along the tendon path opened on the outside face of the web. Loading was terminated at 18.5 kips,  $1.68 P_{Dt}$ . At this level the crack on the inside face of the web had extended to within 1/2 in. of the anchorage plate and was 9 in. long. The crack on the outside face of the web remained confined to the fillet region and was 3 in. long. The cracks were very small and could not be seen with the naked eye after the load had been released.

5.8.6.5 Set 5, Specimen 1. This specimen had tendon pattern B. The anchorage type was LB. It was cast using mix design K and  $f'_c$  was 6.98 ksi.

At 11.3 kips,  $1.03 P_{Dt}$ , the first visual crack appeared on the inside face of the web running along the tendon path. The crack began 3 in. from the anchorage plate and extended 6 in. A 2 in. crack opened on the outside face of the web in the vicinity of the fillet at 12.4 kips,  $1.13 P_{Dt}$ . Loading was terminated at 20.2 kips,  $1.84 P_{Dt}$ . Cracks on both sides of the web had extended within 1/2 in. of the anchorage plate and were very narrow.

5.8.6.6 Set 6, Specimen 1. This specimen had tendon pattern B. The anchorage type was LB. Mix design G was used and  $f'_c$  was 5.47 ksi. At a load of 11.8 kips,  $1.07 P_{D_t}$ , a very narrow crack appeared on the inside face of the web running along the tendon path. The crack began 4 in. from the anchorage plate and extended 2 in. A narrow crack on the outside face of the web opened at 12.8 kips,  $1.16 P_{D_t}$ , in the vicinity of the fillet. Loading was terminated at 20 kips,  $1.82 P_{D_t}$ . The cracks on each side of the web had extended to within 2-1/2 in. of the anchorage plate.

#### 5.8.7 Series 3 Specimens

5.8.7.1 General. The purpose of the Series 3 specimens was to examine several different variables and the effect each had on cracking load. In this series four specimens were tested. All specimens used cone type anchorages. Each specimen also had normal reinforcement details.

5.8.7.2 Set 7, Specimen 1. This specimen had tendon pattern B. While it used a cone type anchorage, its spiral reinforcement differed from that shown in Fig. 5.14. Details of the spiral used in this specimen are shown in Fig. 5.25. This spiral detail is the same as that used in the main model tests.<sup>22</sup> The six U-shaped ties were also placed around the anchorage as in the other specimens. It was cast using mix design K and  $f'_c$  was 5.6 ksi. Figure 5.26(a) shows this specimen before casting and gives an overall view of the long spiral reinforcement.

At a load of 14.8 kips,  $1.35 P_{D_t}$ , an extremely small crack appeared on the inside face of the web running along the tendon path. The plot of load versus strain is shown in Fig. 5.27. The crack began 7 in. from the anchorage plate and extended 2 in. up to the fillet. Loading was terminated at 18.6 kips,  $1.69 P_{D_t}$ . At this level no crack had formed on the outside face of the web. The crack on the inside face of the web had extended to the anchorage plate. The crack was extremely small and could not be seen when the load was released. Figure 5.26(b) shows Set 7, Specimen 1, after completion of the test. (In a prototype this spiral would be approximately equivalent to a 1/2 in. diameter at 2-1/4 in. pitch.)

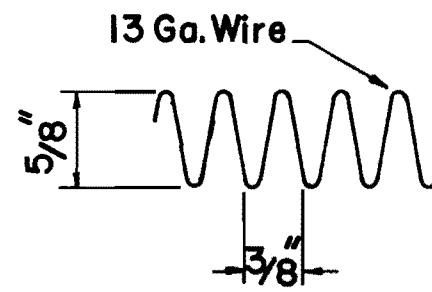
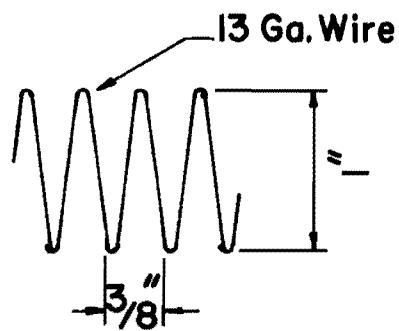
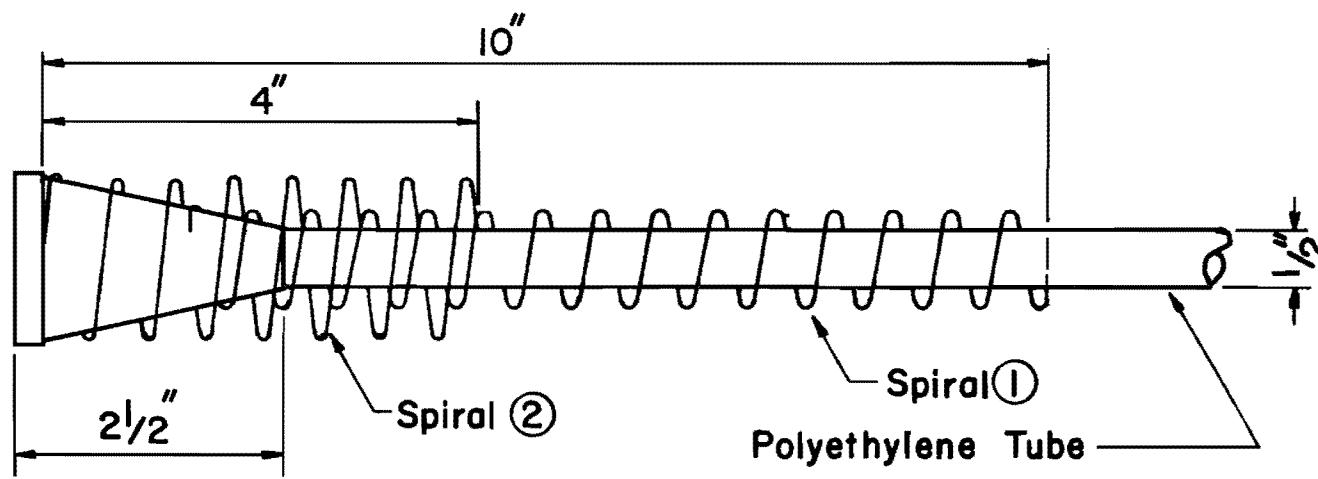


Fig. 5.24. Details of anchorage and spiral reinforcement for Set 7, Specimen 1

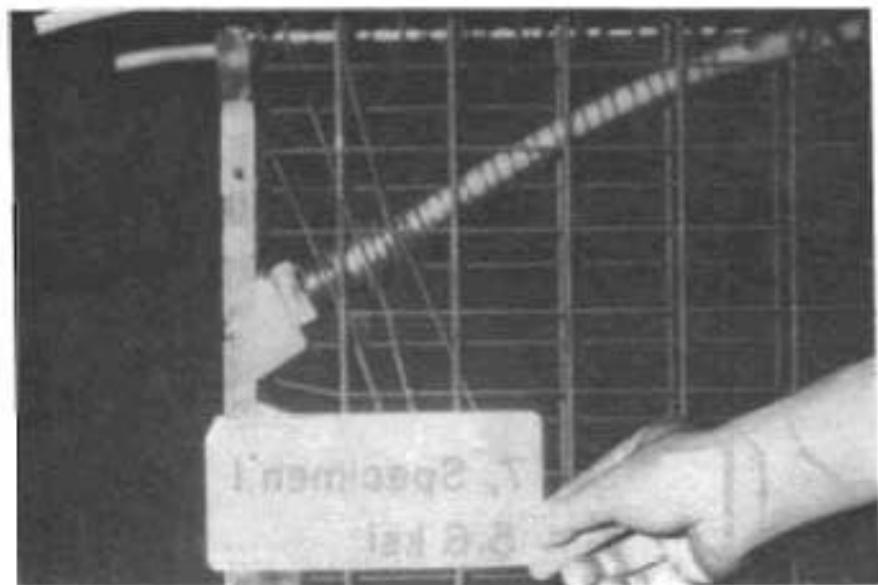


Fig. 5.26(a) Set 7, Specimen 1 before casting showing long spiral reinforcement

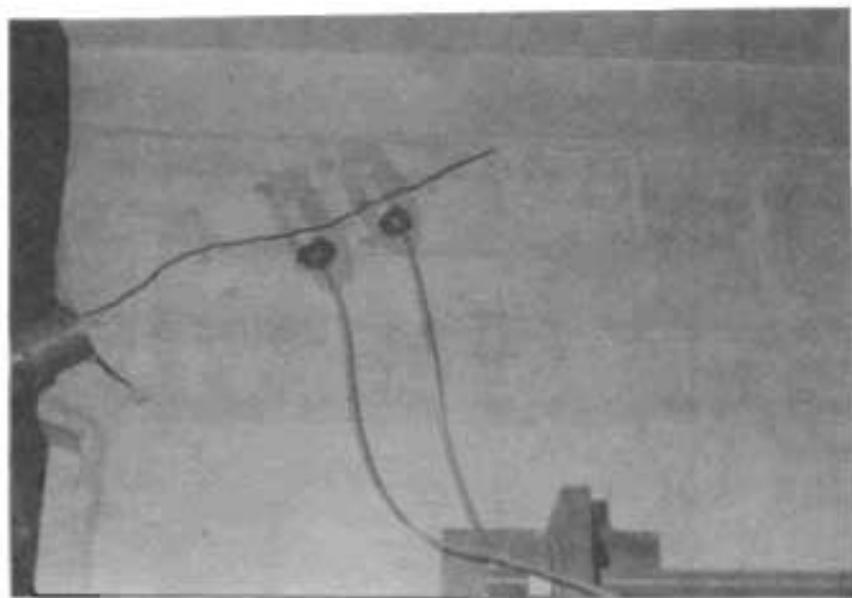


Fig. 5.26(b) Set 7, Specimen 1 after completion of the test

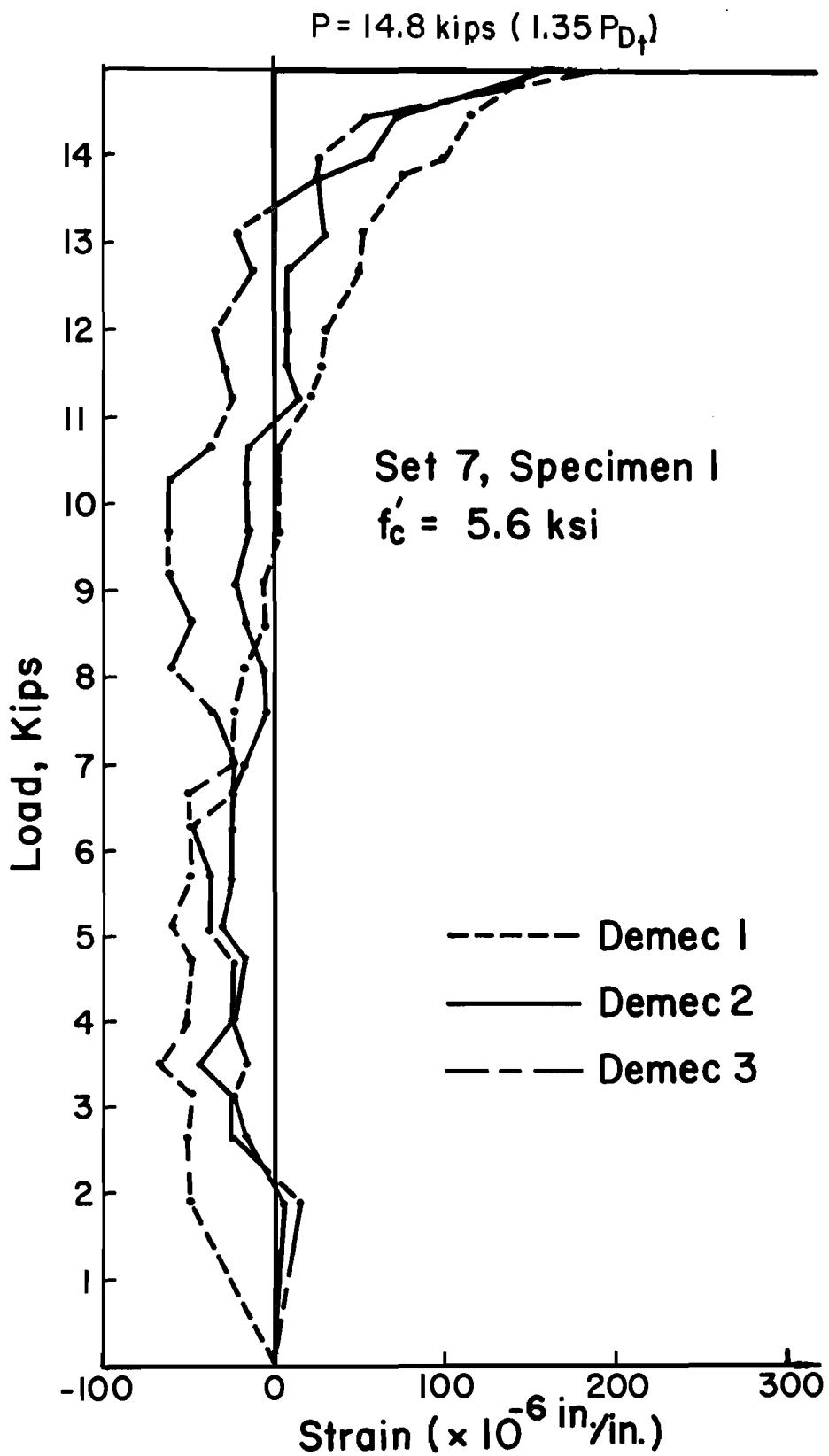


Fig. 5.27. Plot of load versus strain of Set 7, Specimen 1

5.8.7.3 Set 8, Specimen 1. This specimen had tendon pattern B. A cone type anchorage was used with standard anchorage zone details. To investigate the effect of percentage of web reinforcement, the number of stirrups in this specimen was doubled. All stirrups were spaced evenly throughout the length of the specimen. Mix design K was used and  $f'_c$  was 7.07 ksi.

The first visual crack opened at 6 kips,  $0.55 P_{D_t}$ , on the outside face of the web running along the tendon path. The crack began 6 in. from the anchorage plate and extended 1-1/2 in. along the tendon path. At 10 kips,  $0.91 P_{D_t}$ , a very narrow crack opened on the outside face of the web along the tendon path. Loading was terminated at 19 kips,  $1.73 P_{D_t}$ , and cracks could not be seen after the load had been removed. Cracks on both sides of the web extended to the anchorage plate.

5.8.7.4 Set 9, Specimen 1. This specimen had tendon pattern B. A cone type anchorage was used with standard anchorage zone details. It was cast using mix design K and  $f'_c$  was 7.7 ksi.

The web of this specimen was vertically post-tensioned to 300 psi compression using 7 pairs of plastic encased 3/32 in. 250 ksi stainless steel aircraft cable spaced at 2-1/2 in. (see Fig. 5.28). Figure 5.29 shows an overall view of the stressing mechanism during the actual vertical post-tensioning operation. Prior to the actual vertical post-tensioning operation, each pair of strands was prestretched to minimize friction loss and to determine what magnitude of friction loss would remain. The completion of the operation to determine friction loss was followed by the vertical post-tensioning procedure. Load was applied to a pair of strands until the load cell indicated the desired load had been reached. At this point the load was transferred. Losses due to seating were minimized because load was applied by pulling directly against the chuck body.

During the loading sequence, no cracks were noticed on either side of the web along the tendon path. The plot of load versus strain is shown in Fig. 5.31. The highest load attained was 19.6 kips,  $1.78 P_{D_t}$ , and at this load a small 1/2 in. crack appeared on the inside face of the web directly in front of the anchorage plate, making approximately a

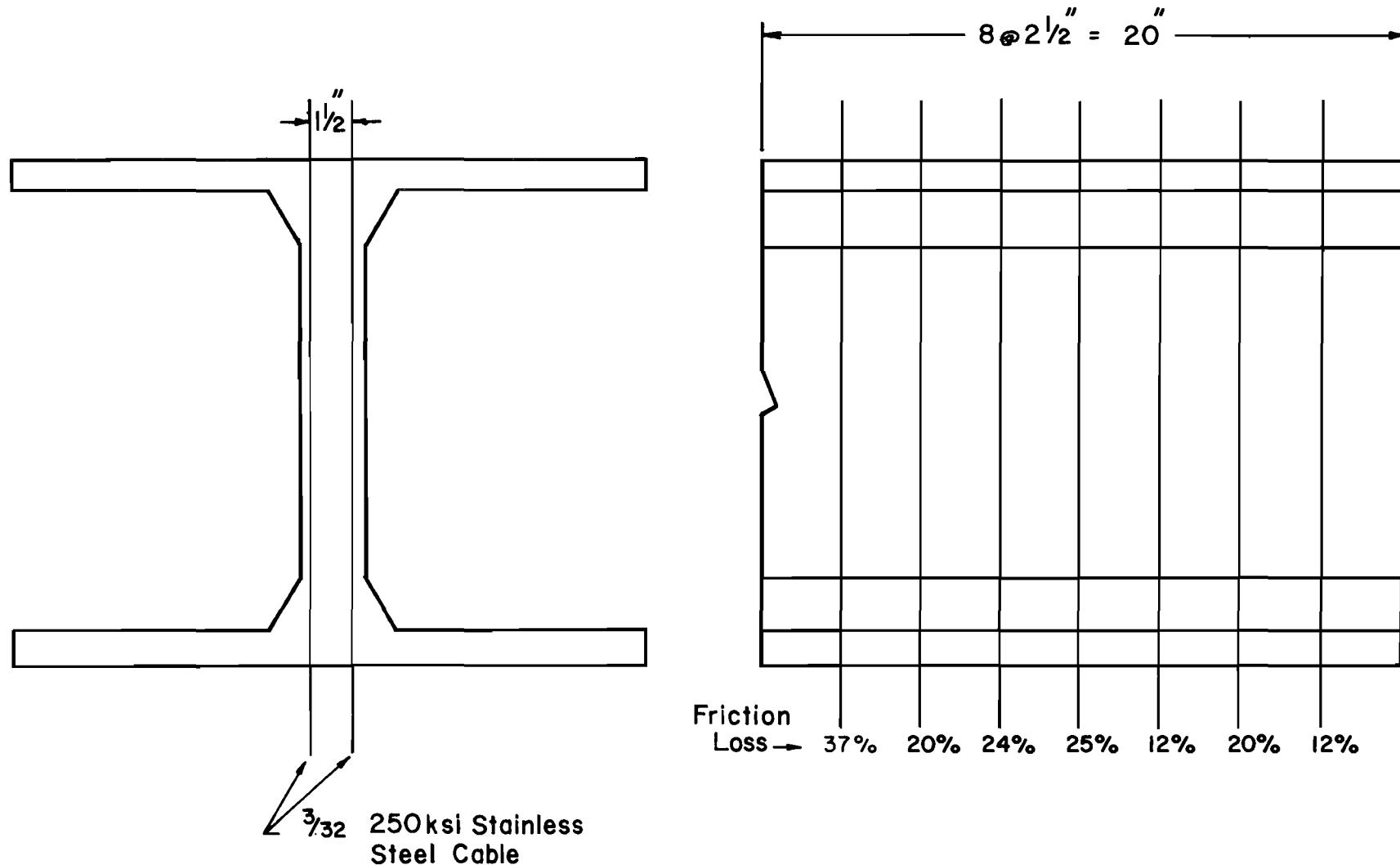


Fig. 5.28. Location of vertical post-tensioning strands

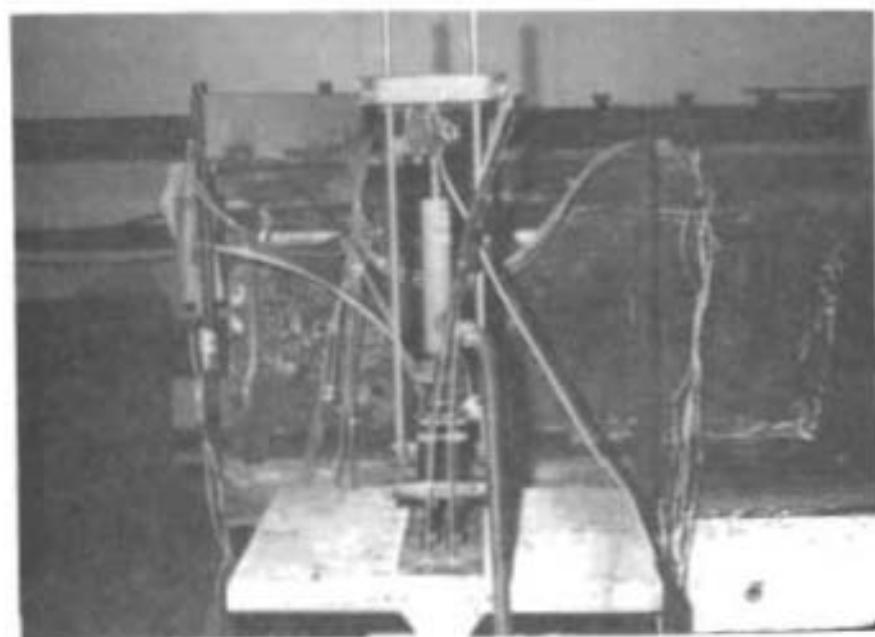


Fig. 5.29. Overall view of the stressing mechanism used to vertically post-tension the web



Fig. 5.30. Set 9, Specimen 1 after completion of the test

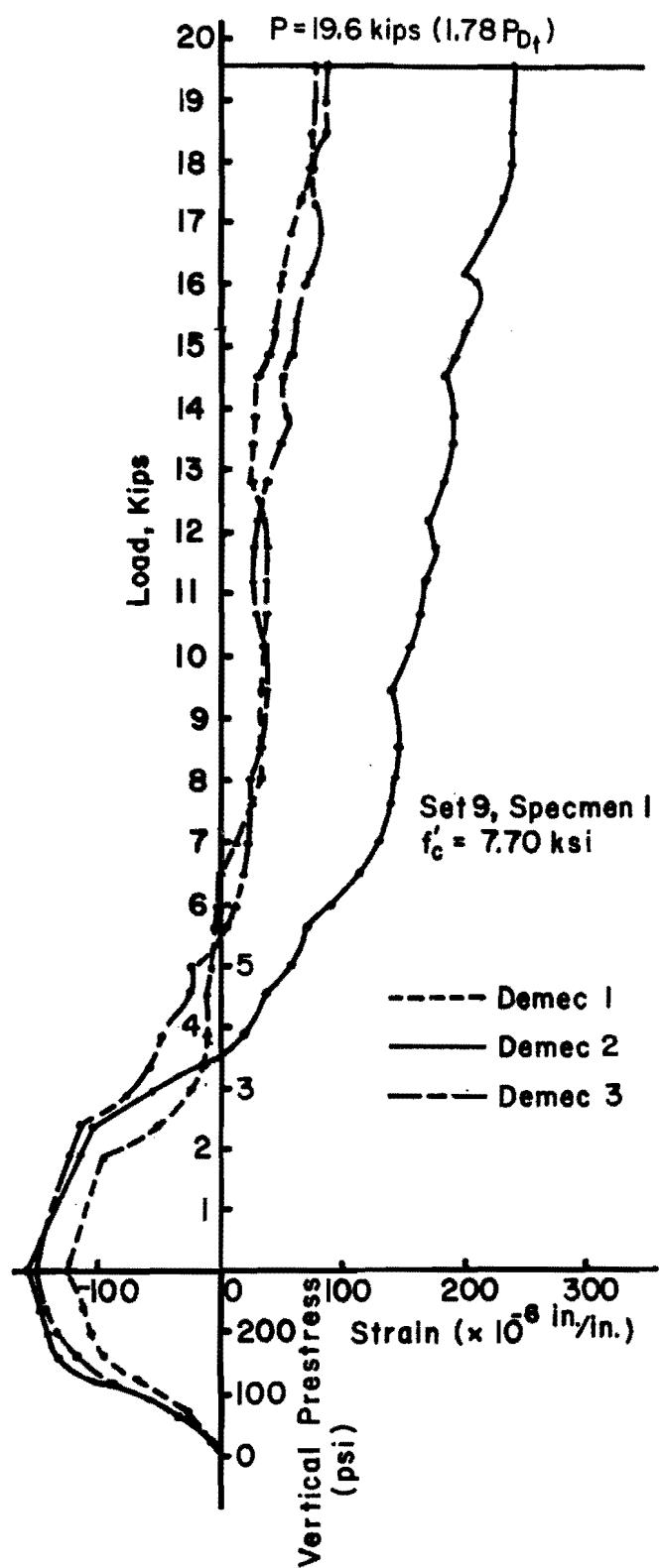


Fig. 5.31. Plot of load versus strain for Set 9, Specimen 1

$30^{\circ}$  angle with the tendon path. Figure 5.30 shows Set 9, Specimen 1, after completion of the test.

5.8.7.5 Set 10, Specimen 1. This specimen had a single tendon placed horizontally through the web at midheight (see Fig. 5.32). A cone type anchorage was used with normal anchorage zone details. Mix design K was used and  $f'_c$  was 6.83 ksi.

During the loading sequence no cracks were noticed on either side of the web along the tendon path. The plot of load versus strain is shown in Fig. 5.34. At  $16.0 \text{ kips}, 1.45 P_{D_t}$ , a crack formed on the inside face of the web directly in front of the anchorage plate making approximately a  $60^{\circ}$  angle with the tendon path. Loading was terminated at  $19.7 \text{ kips}, 1.79 P_{D_t}$ . Figure 5.33 shows Set 10, Specimen 1, after completion of the test.

5.8.8 Data Summary. Principal results for each specimen are summarized in Table 5.1 along with a brief description of the special features of each specimen.

5.8.9 Comparison of Model and Prototype. The Series 1 specimens were constructed with the fabrication errors believed to have occurred in a few prototype units. The primary purpose of the Series 1 tests was to determine if the reduced flange single web model (model I) would produce a cracking pattern similar to that of the prototype and the supplemental one-sixth scale model tests. If reasonable correlation with the prototype was found, then these results could later be used as a base of reference for comparison of changes made in the Series 2 and Series 3 specimens.

Construction personnel reported that the earliest that prototype web cracks tended to appear when stressing the first of a pair of tendons, or when stressing a single tendon was at a stressing jack pressure of about 5600 to 6000 psi. For a 12 strand 1/2 in. diameter 270 ksi tendon in the prototype, this level is a tendon force of about 280 to 300k. On a one-sixth scale model, this would be equivalent to  $290 \times (1/6)^2$  or 8.0 kips ( $0.75 P_{D_b}$ ,  $0.75 P_{D_t}$ , or  $0.38 P_{D_G}$ ). In the prototype sections which were instrumented, cracking loads varied from 396k to 670k, based

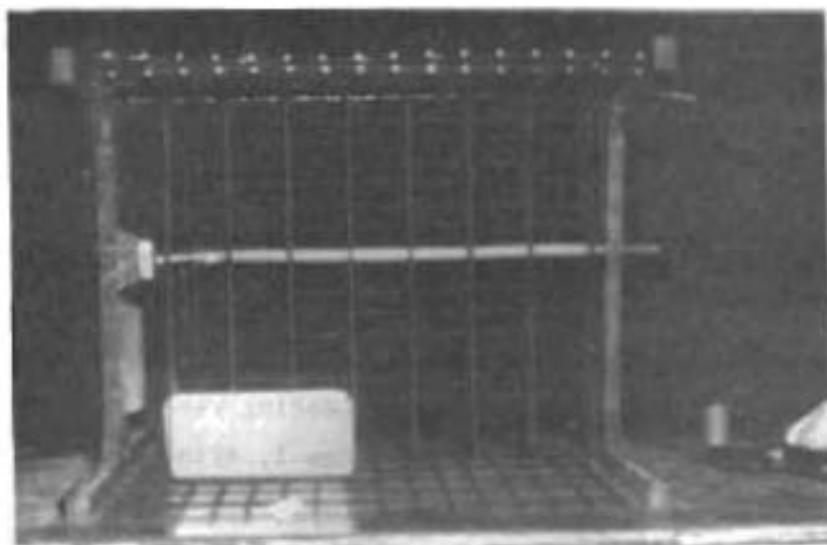


Fig. 5.32. Set 10, Specimen 1 before casting showing horizontal tendon

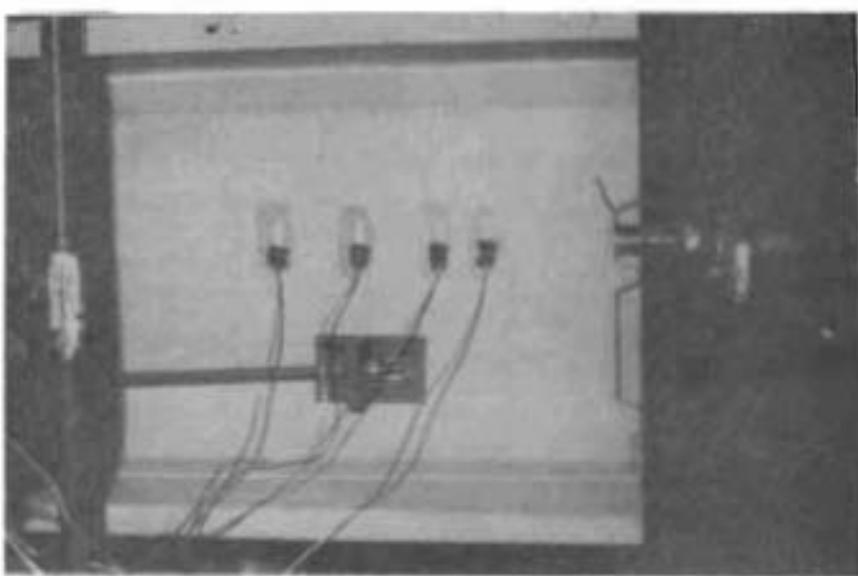


Fig. 5.33. Set 10, Specimen 1 after completion of the test

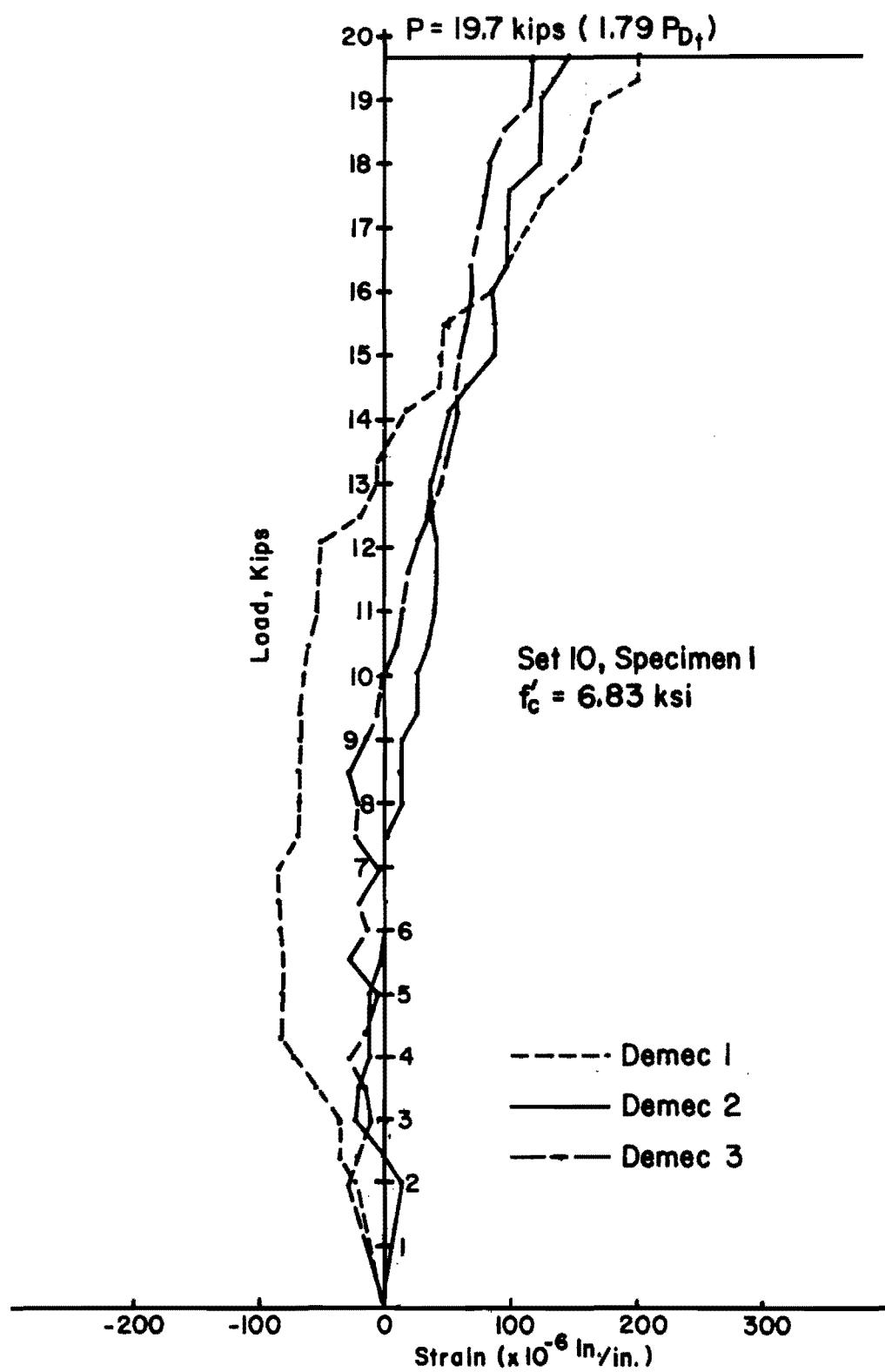


Fig. 5.34. Plot of load versus strain for Set 10, Specimen 1

TABLE 5.1. SUMMARY OF TEST RESULTS

Specimen Designation	Anchorage Type	Tendon Pattern	Special Feature	$f'_c$ ksi	Cracking Load kips, P	$\frac{P}{P_{D_b}}$	$\frac{P}{P_{D_t}}$	$\frac{P}{P_{D_G}}$	Ultimate Load kips, $P_{max}$	$\frac{P_{max}}{P_D}$
Set 1, Spec. 1	Cone	A	Altered Reinf.	6.30	8.8	0.80		0.40	40.0 **	1.82
Set 1, Spec. 2	Cone	A	Altered Reinf.	4.43	7.0	0.64		0.32	35.0 **	1.59
Set 1, Spec. 3	Cone	B	Altered Reinf.	5.73					18.4 **	1.67
Set 2, Spec. 1	Cone	A	Altered Reinf.	4.89	8.8	0.80		0.40	33.2	1.51
Set 2, Spec. 2	Cone	A	Altered Reinf.	4.88					33.0	1.50
Set 2, Spec. 3	Cone	B	Altered Reinf.	4.87	9.5		0.86		18.8 **	1.71
Set 3, Spec. 1	SB	B	Bearing Anchor	7.48	12.6		1.15		19.9 **	1.81
Set 3, Spec. 2	SB	A	Bearing Anchor	6.62	17.2	0.89	0.67	0.78	44.0 **	2.00
Set 4, Spec. 1	SB	B	Bearing Anchor	5.55	12.1		1.10		18.5 **	1.68
Set 5, Spec. 1	LB	B	Bearing Anchor	6.98	11.3		1.03		20.2 **	1.84
Set 6, Spec. 1	LB	B	Bearing Anchor	5.47	11.8		1.07		20.0 **	1.82
Set 7, Spec. 1	Cone	B	Long Spiral	5.60	14.8		1.35		18.6 **	1.69
Set 8, Spec. 1	Cone	B	Double Stirrups	7.07	6.0		0.55		19.0 **	1.73
Set 9, Spec. 1	Cone	B	Vertically Prestressed	7.70	19.6 *		1.78		19.6 **	1.78
Set 10, Spec 1	Cone	B	Horizontal Tendon	6.83	16.0 *		1.45		19.7 **	1.79

\* Crack did not form along tendon path.

\*\* Loading discontinued before complete failure.

on the group load or from 275k to 396k for individual tendons. Cracking loads in the prototype varied and the values above are an approximate lower bound, but the actual scatter was large.

The three box units for the supplemental test were cast using mix design G. The average concrete compressive strength at time of test was 5.44 ksi. Since these tests were conducted in a relatively short period of time and dead load compensating blocks obscured visibility of web interiors, an extensive study of the cracking load was not feasible. The assessment of the possible effects of such cracks on shear strength was more important. Therefore, cracking loads were precisely determined on only one box unit (both webs). This box unit had tendon pattern A. Both webs of this specimen cracked along the top tendon path when the bottom tendon had been stressed to slightly above 9 kips ( $0.8 P_{D_b}$  or  $0.4 P_{D_G}$ ). Comparison of the cracking patterns between model and prototype showed marked similarity and the magnitude of load at first cracking in the model was in general agreement with observed cracking loads in several of the prototype units. To more exactly simulate the prototype construction technique, the dead load blocks were in place when the other two box units were stressed and these dead load blocks did not permit a visual inspection of the inside face of the web. Although the webs of the second and third boxes definitely cracked, the load at first (visual) cracking was not determined.

Figure 5.35 shows a comparison of the cracking loads of the prototype, supplemental box girder models and the specimens of Series 1. From Fig. 5.35 it can be seen that there is very good agreement between the prototype and model results. Actual concrete strengths of the prototype concrete were quite high. The average compressive strength at 14 days was reported as 7500 psi. The data in Fig. 5.35 are not corrected for the higher actual strengths of the prototype. This would tend to explain the somewhat higher prototype values.

A photograph of a typical crack pattern found in the prototype is shown in Fig. 5.36(a). Shown in Fig. 5.36(b) is a typical crack pattern from the supplemental one-sixth scale box girder model which was sawed in

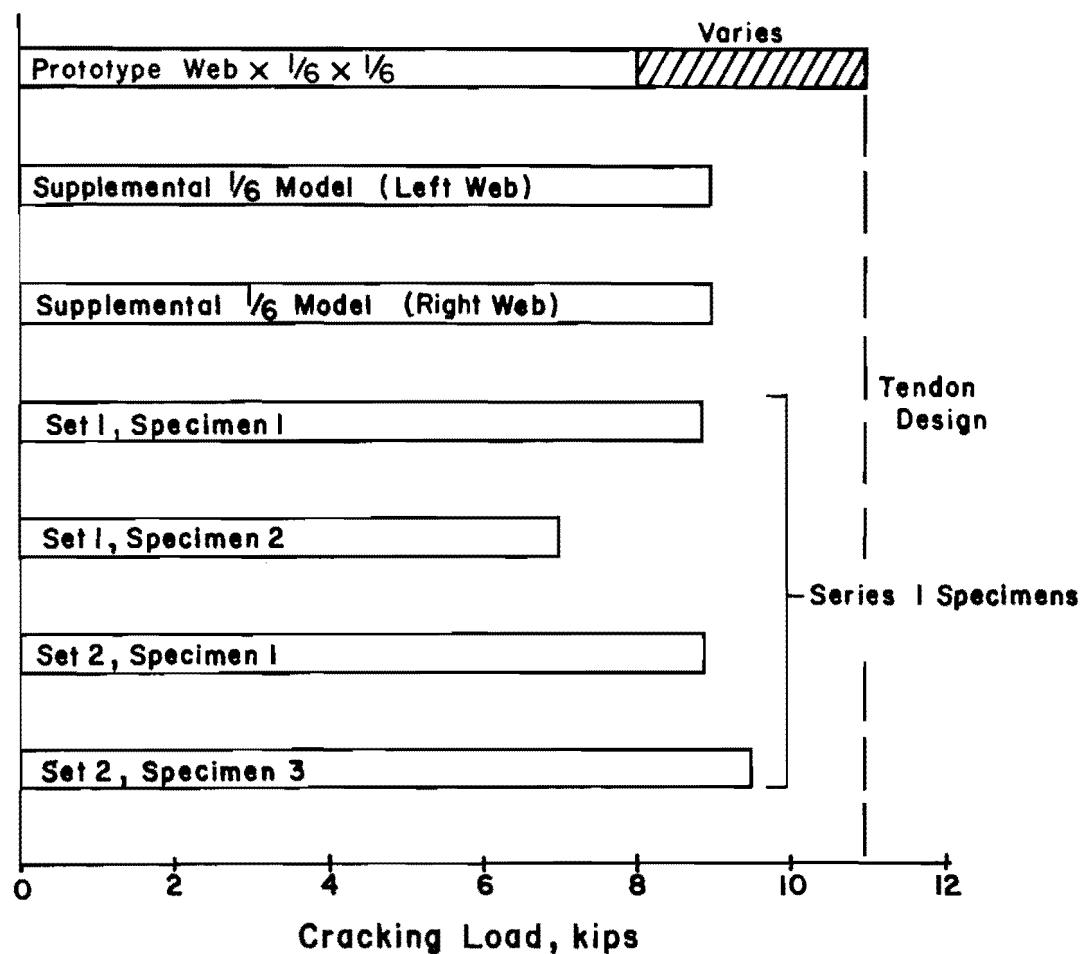
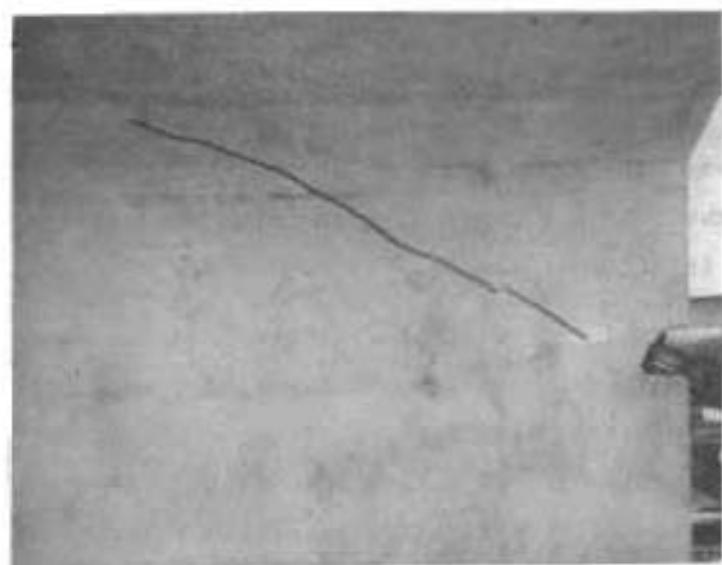


Fig. 5.35. Comparison of cracking loads between prototype and models



(a) Prototype

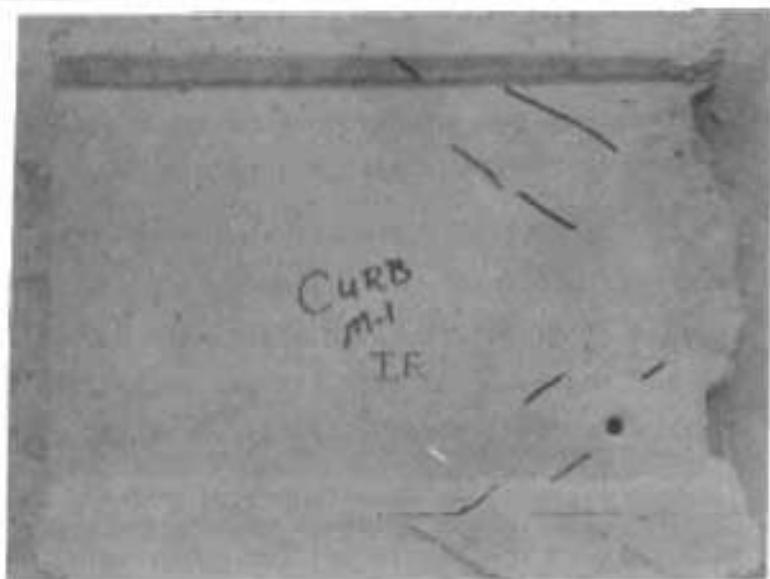
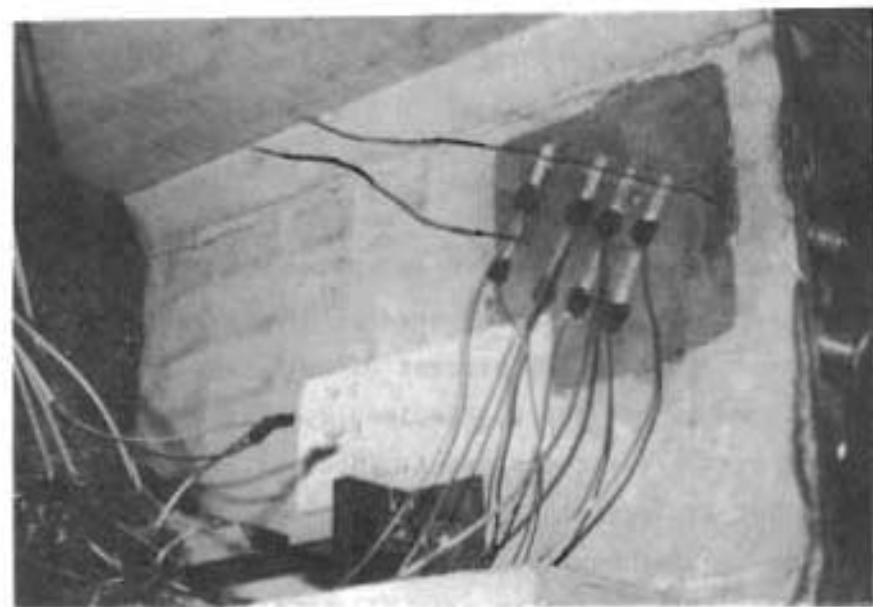
(b) Supplemental  
1/6 scale  
model(c) Model I  
section

Fig. 5.36. Typical crack patterns of prototype and models

half for the photograph. Figure 5.36(c) shows the cracking pattern in a typical Model I specimen. The extent of cracking is greater in both model specimens because they were post-tensioned or test loaded far above design levels. However, the cracks along the tendon path are quite evident and quite similar in all specimens.

Due to the generally good agreement of cracking loads between the prototype, supplemental one-sixth scale model tests, and the model I section tests, and due to the strong similarities of the cracking patterns between both models and the prototype, it can be assumed that the behavior of the model I section is reasonably representative of the behavior of the prototype webs.

**5.8.10 Influence of Concrete Strength.** Two concrete mix designs were used in this study. Mix design K was designed to give  $f'_c$  of 6 ksi and mix design G was designed to give  $f'_c$  of 5 ksi.

Naturally, there was some scatter in concrete strength from different specimens cast using the same mix design. Mix design K had an average  $f'_c$  of 6.47 ksi and a standard deviation of 0.99 ksi. Mix design G had an average  $f'_c$  of 5.13 ksi and a standard deviation of 0.35 ksi. The average concrete compressive strength for each specimen is given in Table 5.1.

Figure 5.37 shows the effect of concrete strength on cracking load. Each pair of specimens used for comparison in Fig. 5.37 is identical with the exception of concrete strength. The influence of concrete strength on cracking load is small. However, this small influence is not surprising since cracking loads are usually dependent on the tensile strength of concrete which is generally assumed to vary as the square root of the compressive strength.

The influence of concrete strength on ultimate anchorage force could not be determined from this limited test program because the tendon load was limited to approximately 200 percent of anchorage design value,  $P_D$ . Only Set 2, Specimen 1, and Set 2, Specimen 2, actually experienced a complete failure of the concrete containment. However, these two specimens can be compared with Set 1, Specimen 1, and Set 1, Specimen 2, which have matching details. Table 5.2 shows this comparison. Although the ultimate load of Set 1, Specimen 1, could not be reached, it did carry 20 percent

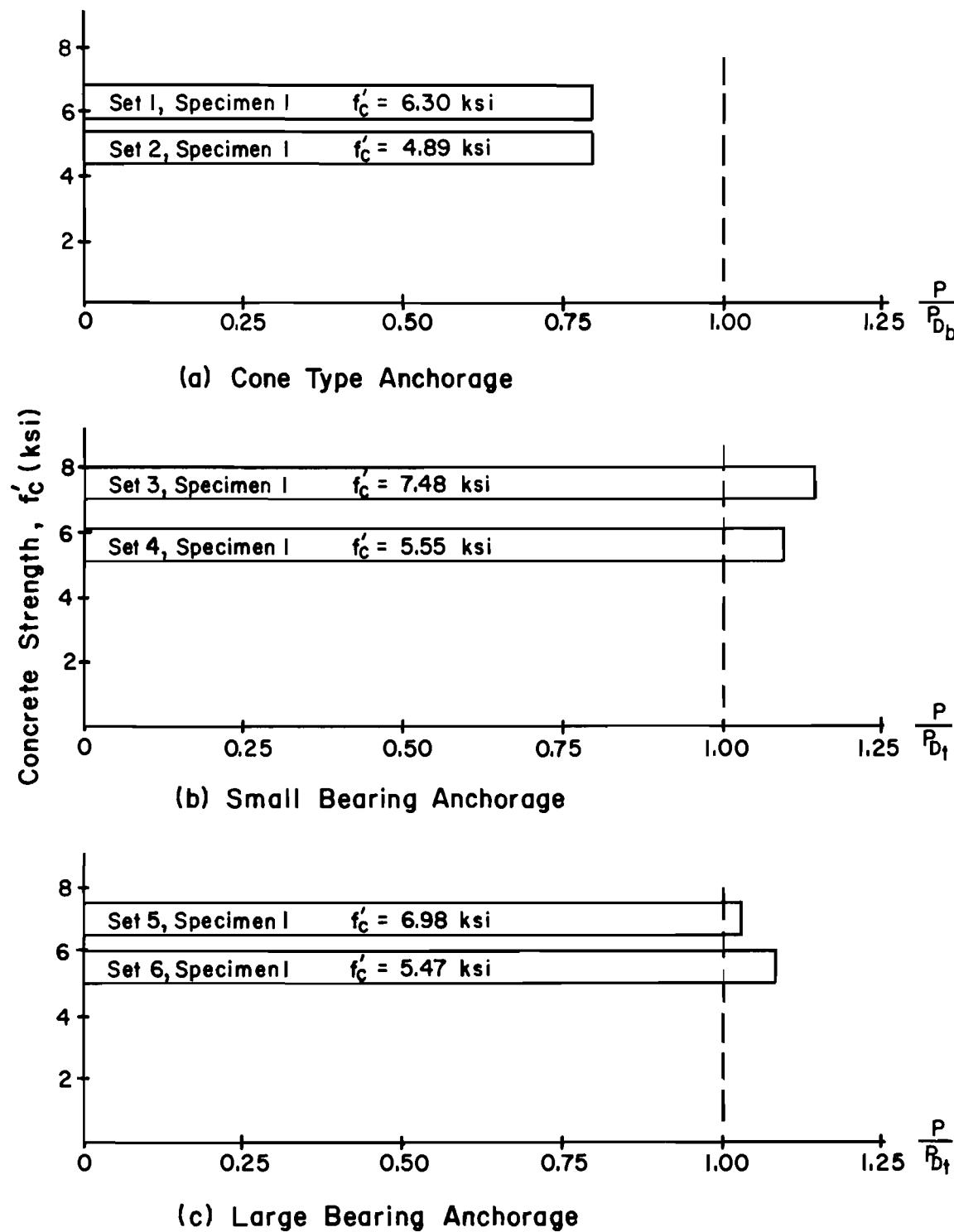


Fig. 5.37. Influence of concrete strength on cracking load

TABLE 5.2. INFLUENCE OF CONCRETE STRENGTH ON ULTIMATE LOAD

Specimen	$f'_c$ ksi	$P_{max}$ kips	$\frac{P_{max}}{P_{DG}}$	Remarks
Set 1, Spec. 1	6.3	40.0	1.82	No failure
Set 2, Spec. 1	4.9	33.2	1.51	Concrete failure
Set 1, Spec. 2	4.4	35.0	1.59	No failure
Set 2, Spec. 2	4.9	33.0	1.50	Concrete failure

more tendon force than Set 2, Specimen 1, which did reach ultimate. Comparison of Set 1, Specimen 2, and Set 2, Specimen 2, is less conclusive. Set 1, Specimen 2, had not failed when loading was terminated at 35.0 kips,  $1.59 P_{DG}$ , but it appeared to be on the verge of failure. Even though Set 2, Specimen 2, had 11 percent higher concrete strength, it failed at 33.0 kips,  $1.5 P_{DG}$ .

Nothing very conclusive can be drawn from the small number of comparable specimens. Greater stiffness and higher ultimate loads should be expected as concrete strength increases. The test results for one set indicate that strength of anchorage assemblies showed more gain than would be indicated by the ratio of the square roots of  $f'_c$ . The other set shows a loss. An upper limit on the ultimate load of a specimen is reached when the strands develop their ultimate strength. A design value substantially greater than tendon ultimate is probably meaningless except as an indicator of reliability for the anchor system.

5.8.11 Effect of Altering Reinforcing Bars. The alteration of reinforcement cages to place anchorage units without adequate replacement was initially suspected as being a primary cause of prototype premature cracking along the tendon path. However, as shown in Fig. 5.38, the effect of even drastic alteration of this reinforcement on the cracking load is small. Set 1, Specimen 1, and Set 2, Specimen 1, had more reinforcement removed than Set 1, Specimen 2, yet they both cracked at a higher load. Set 2, Specimen 3, which had only minimal steel removed, cracked at only 8 percent higher load than Set 1, Specimen 1, and Set 2, Specimen 1.

## KEY

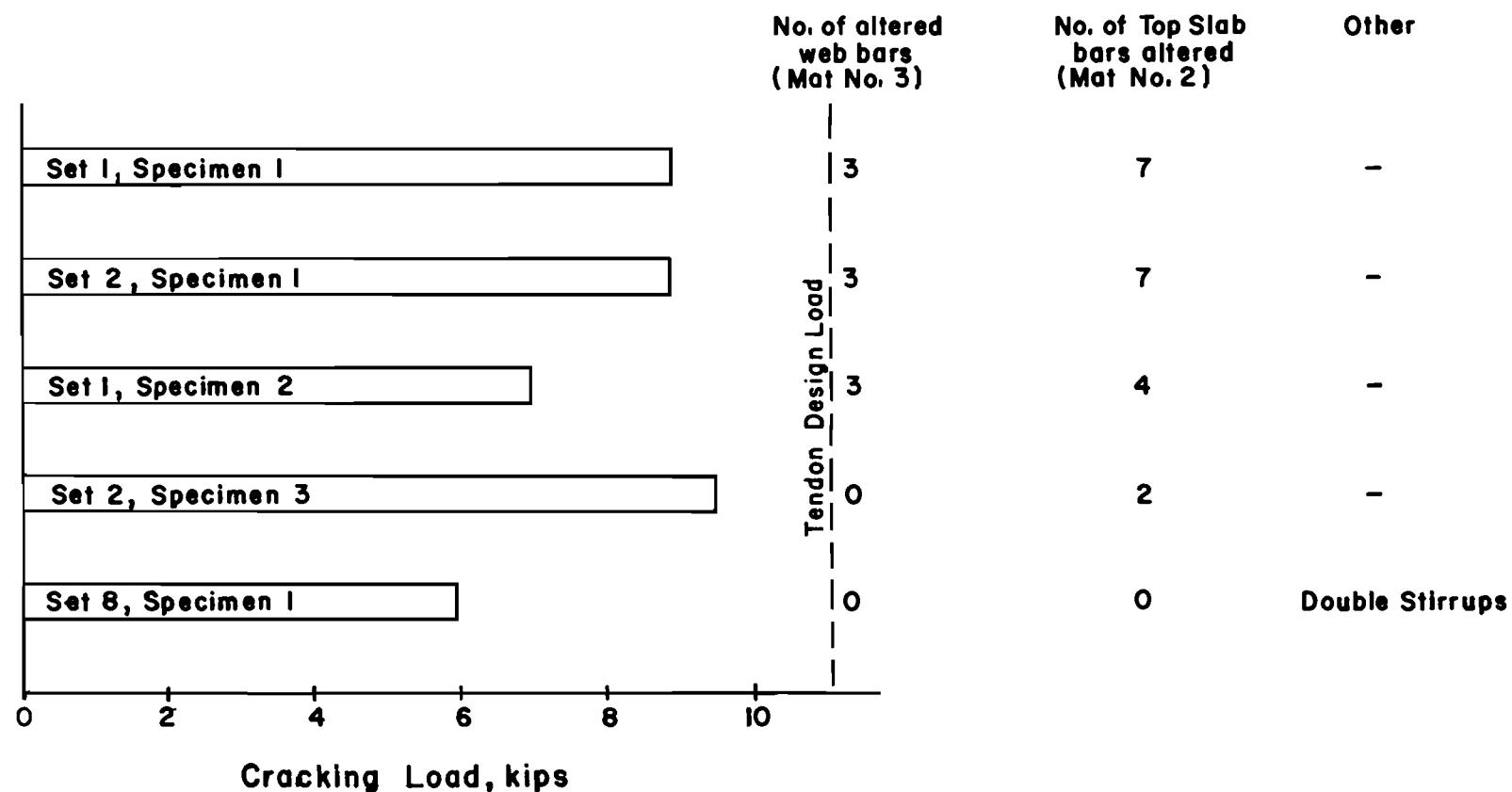


Fig. 5.38. Influence of altering reinforcement.

In order to get a general indication of the influence of percentage of web steel, Set 8, Specimen 1, is also included in Fig. 5.38. The number of stirrups for this specimen was approximately doubled and all bars altered for anchorage insertion or duct placement were replaced. As Fig. 5.38 indicates, the effect of doubling the number of stirrups did not improve the cracking load. It may have even harmed it. However, visual observations made during the test did indicate that the cracks which occurred in Set 8, Specimen 1, were much narrower than in the other specimens shown. The web reinforcement becomes effective only after initial cracking has taken place and works to control crack sizes. It was concluded that the faulty alteration of the reinforcement was not the main cause of crack formation. A number of units, in both prototype and models, with completely adequate reinforcement as detailed, displayed definite cracking. All models of Series 2 and Series 3 were fabricated as detailed with no omitted reinforcement.

5.8.12 Influence of Anchorage Type. Figure 5.39 indicates the significance of anchorage type on cracking load for specimens with tendon pattern A. Set 2, Specimen 2, is not included since its cracking load could not be determined. Figure 5.40 reflects the significance of anchorage type for specimens with tendon pattern B. Set 1, Specimen 3, is not included due to the unusual circumstances leading to the formation of the first crack, as previously described. All specimens used to investigate the influence of anchorage type had identical anchorage zone reinforcement details.

Figure 5.39 shows that the specimen which had the SB type anchorage did not crack until a tendon force of nearly twice the applied tendon force of the specimens with cone type anchorage was applied. However, even with this increase, the cracking load is still below  $P_{DG}$ .

Figure 5.40, for single tendon specimens, indicates that all specimens which were equipped with either small or large bearing type anchorages were able to achieve the design prestress force before cracking. The cone type anchorage in the same type specimen cracked at approximately 85 percent of tendon design load.

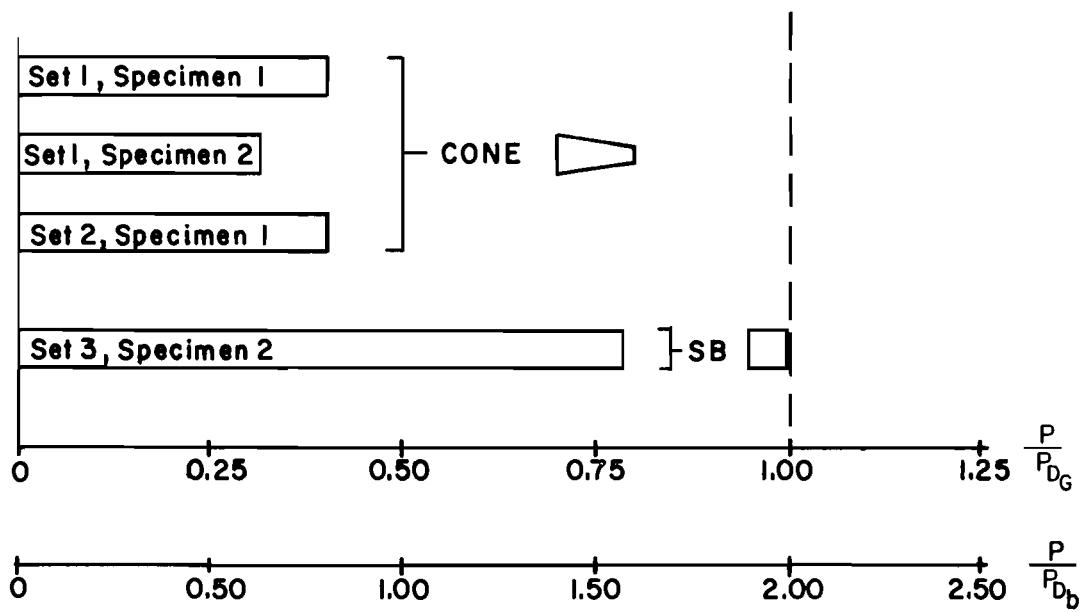


Fig. 5.39. Influence of anchorage type on cracking load for specimens with tendon pattern A

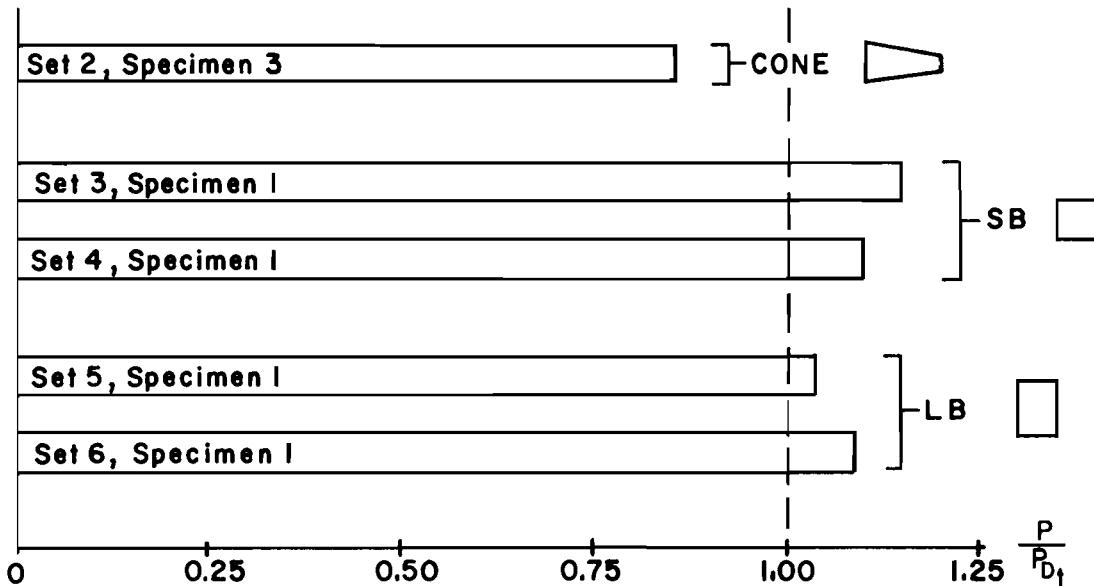


Fig. 5.40. Influence of anchorage type on cracking load for specimens with tendon pattern B

From the small number of tests conducted it is unclear why the specimens with LB type anchorages cracked at slightly lower loads than the specimens with SB type anchorages. The only basic difference between the SB anchorage and LB anchorage other than area was that the LB anchorage was rectangular whereas the SB anchorage was square. The LB type anchorage should produce a smaller bearing stress than the SB type anchorage for a constant force, but as Fig. 5.40 indicates, the SB anchorage specimens cracked at higher loads although the difference is relatively small (less than 12 percent). A similar result was published in Ref. 6 by Ban, Muguruma, and Ogaki. They reported that the initial cracking load of the anchorage end of a Lee-McCall type post-tensioned beam remains approximately constant, regardless of surface area of the anchorage plate. They further reported that when the surface area of the anchorage plate becomes larger that a slight increase in the ultimate load may result. In light of this small difference, a specimen with tendon pattern A using the LB type anchorage was considered unnecessary.

From this limited test program any judgment concerning the effectiveness of the SB anchorage as compared to the LB anchorage would be premature. It is important to note that both sizes of bearing type anchorages allowed all tendon pattern B specimens to reach their design prestress force. This trend for higher load capacity was also evident with the tendon pattern A specimen although this specimen cracked before the design prestress force was reached.

Similar results concerning increased tensile stresses resulting from cone type anchorage systems have also been reported by Zielinski and Rowe in Ref. 42. According to Zielinski and Rowe, the method of anchoring the tendons (either through cone action or bearing plate) did not significantly affect the pattern of distribution of the transverse stresses or the ultimate load. Although in this research program specimens with cone type anchorages tended to show a lower ultimate load than specimens with bearing type anchorages, the first conclusion reached by Zielinski and Rowe concerning distribution of transverse stresses appears to be also reflected in the results of this study. Although no direct measurements of transverse stresses were attempted, specimens with

cone type and bearing type anchorages alike experienced first cracking within a narrow range of distance from the anchorage plate, indicating the maximum tensile stresses to be in approximately the same location along the tendon path. In addition, Zielinski and Rowe reported that the method of anchoring tendons does affect the magnitude of the transverse stresses, conical action increasing the stresses and decreasing the cracking load between 5 and 15 percent. They did not, however, limit the range but stated that the effect of increased tensile stresses is dependent on the anchoring technique employed in each system and that each new type of anchorage will require investigation. The largest difference in cracking load between cone type and bearing type specimens (tendon pattern B) in this research program was approximately 25 percent. Thus, the conclusion reached by Zielinski and Rowe seems to be consistent with findings of this report.

#### 5.8.13 Discussion of Special Modifications--Series 3 Specimens

5.8.13.1 General. The purpose of Series 3 specimens was to spot-check several different variables and determine the approximate effect each had on cracking load and anchorage performance. Since this research was primarily intended to be an exploratory investigation, an extensive test program, although desirable, was not feasible at this time. Subsequently, a total of four specimens with tendon pattern B was planned so that the cause of premature cracking could be further defined and so that general recommendations for improving the anchorage zone details could be made. The effect of Set 8, Specimen 1, with doubled web reinforcement, has previously been discussed.

5.8.13.2 Influence of Spiral Reinforcement. Premature cracking along the tendon path was not experienced in the 84 segments of the one-sixth scale model test conducted by Kashima.<sup>22</sup> When comparing the anchorage and anchorage zone details used in the one-sixth scale bridge model to those used in the prototype bridge, two basic differences can be found. First, the anchorage used in the one-sixth scale model was a bearing type with a projected area meeting the requirements of the then current ACI and AASHO codes, whereas a cone type anchorage with significantly less

projected area was used in the prototype bridge. The significance of anchorage type was discussed in the previous section. Second, the extensive spiral reinforcement used in the one-sixth scale complete model bridge study (Fig. 5.25) differed greatly from the short spiral reinforcement used in the prototype bridge (Fig. 5.14).

To further examine the effect of spiral reinforcement and to check the behavior reported by Kashima and Breen,<sup>22</sup> Set 7, Specimen 1, was fabricated using the spiral detail employed in the one-sixth scale model bridge study combined with the conical type anchor.

From Fig. 5.41 it can be seen that Set 7, Specimen 1, was able to carry over 1-1/2 times the applied prestress force of Set 2, Specimen 3. These specimens have cone type anchorages and are directly comparable since they differ only in spiral reinforcement. The specimen with the long spiral did not crack on the outside face of the web and the crack on the inside face of the web was very narrow. The specimen with the short spiral experienced wide cracks on both sides of the web. The specimen with the long spiral did not crack until substantially above the tendon design force.

A specimen using a bearing type anchorage and having the spiral detail shown in Fig. 5.25 was not fabricated since the 84 segments of the one-six scale model study which had these details experienced no cracking along the tendon paths.

This test reemphasizes the desirability of adequate spiral reinforcement throughout a substantial length of the anchorage zone in thin web applications.

5.8.13.3 Evaluation of Vertical Post-Tensioning. The technique of vertically post-tensioning webs to resist tensile stresses created in anchorage zones has been previously suggested by Guyon.<sup>17</sup> Guyon terms this type of reinforcement as "active" reinforcement. The advantage of this type of system is that tensile stresses associated with anchorage zones are immediately resisted upon application of a post-tensioning force by an existing compressive stress. This type of "active" reinforcement is inherently more effective in resisting tensile stresses than conventional "passive" reinforcement, such as web stirrups, which have little

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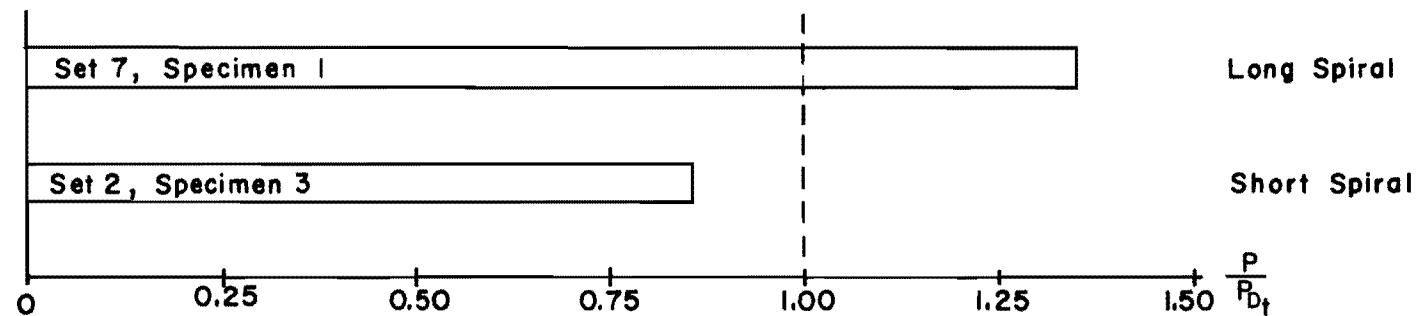


Fig. 5.41. Influence of spiral reinforcement

effect on the load at which cracks form, but instead act to control crack width after its formation.

From Fig. 5.42 it can be seen that Set 9, Specimen 1, which was vertically post-tensioned, was able to carry more than twice the applied anchorage force of Set 2, Specimen 3. The standard prototype detail, as represented by specimens with doubled web reinforcement and the long spiral, are also shown for purposes of comparison. Again, assuming that the effect of concrete strength on cracking load is relatively small, this increase can be primarily attributed to vertically post-tensioning the web to 300 psi. Table 5.1 indicates that Set 9, Specimen 1, with vertical post-tensioning, achieved the highest load before cracking of any specimen reported. It should also be noted that when loading was terminated at 19.6 kips, or  $1.78 P_{D_t}$ , this specimen had not cracked along the tendon path. At this load a small 1/2 in. long crack appeared directly in front of the anchorage plate, making a  $30^{\circ}$  angle with the tendon path. As mentioned previously, this type of crack was common with many of the specimens at loads approaching 20 kips.

One advantage other than preventing cracks along the tendon path is not immediately obvious from this study. The strands which pass through the web may also be designed to carry part or all of the shear stresses associated with service loads carried by a bridge. In addition, the strands need not be vertical, but could be placed at a  $45^{\circ}$  angle through the web to more efficiently carry the shear. Thus, the need for conventional web bars can be minimized. Post-tensioning the web can be accomplished at the precast plant, thus limiting the amount of field installation and ensuring greater quality control.

This test indicates the feasibility and desirability of vertically post-tensioning thin web sections.

**5.8.13.4 Influence of Tendon Curvature.** A preliminary survey of available literature on anchorage zone stresses in post-tensioned concrete members reveals that the bulk of research has been conducted on bearing type anchorages which transfer prestress force to an end block. Typically, the axis of the member coincides with the axis of the anchorage. Very little information is available where the axis of the anchorage makes a

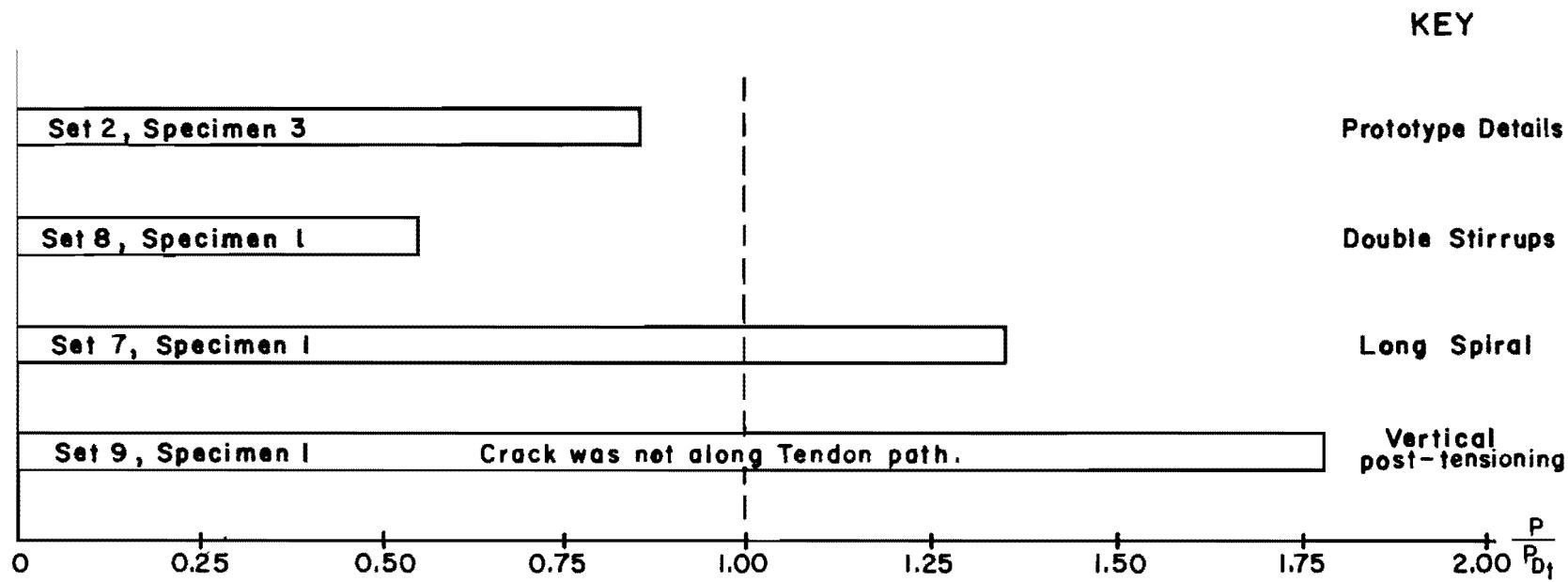


Fig. 5.42. Influence of vertical post-tensioning of the web

large angle with the axis of the member as in the segments used in the prototype. In this study (and in the prototype), the axis of the anchorage makes a  $28^{\circ}$  angle with the axis of the member. Thus, in a distance equal to approximately 1/2 the length of the segment, the tendons must change from a horizontal position to an axis oriented  $28^{\circ}$  with the axis of the member (see Fig. 5.12).

The purpose of Set 10, Specimen 1, was to investigate the influence of tendon curvature. This was accomplished by stressing a horizontal tendon, thus removing the radial stresses due to tendon curvature (see Fig. 5.2). Set 10, Specimen 1, which had a tendon placed horizontally through the web, is directly comparable with Set 2, Specimen 3, which had tendon pattern B. From Fig. 5.43 it can be seen that the specimen with the horizontal tendon was able to carry before cracking approximately 1.7 times the anchorage force of a specimen with an inclined and curving tendon. It should also be noted that the first crack was not along the tendon path, but formed in front of the anchorage plate, making approximately a  $60^{\circ}$  angle with the tendon path. This type of crack was common with many of the specimens in this research at higher loads approaching 20 kips per anchorage. When loading was terminated at  $1.79 P_{D_t}$ , no crack along the tendon path had occurred.

The results of this test indicate the need for testing anchorages using in-situ conditions. An anchorage embedded in an end block and tested as shown in Fig. 5.44 would neglect potentially important radial stresses due to tendon curvature if in fact such stresses would be present in the field. The method of testing anchorages shown in Fig. 5.44 is common in the literature.

When planning a post-tensioned thin-webbed box girder bridge such as the one built in Corpus Christi, a preliminary test program should be instigated unless previous detailed experience with the anchorage and its details is available. Typically, in American practice, the design of such a bridge does not include the detailed design of the anchorage system. Generally, the anchorage system will be supplied by one of the many companies engaged in this specialty. By running these preliminary tests, a check on the adequacy of anchorage zone details can be made and potential

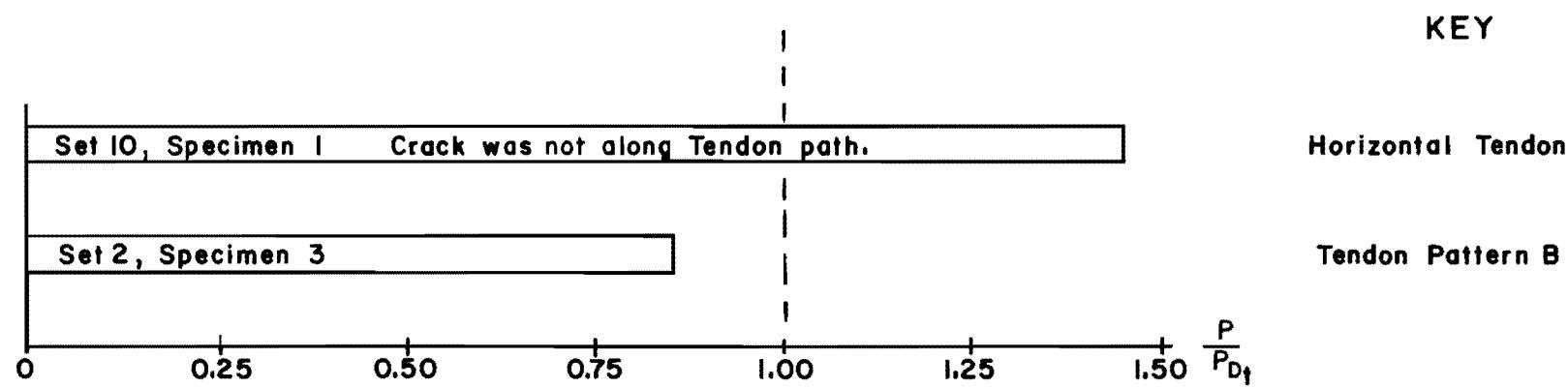


Fig. 5.43. Influence of tendon curvature

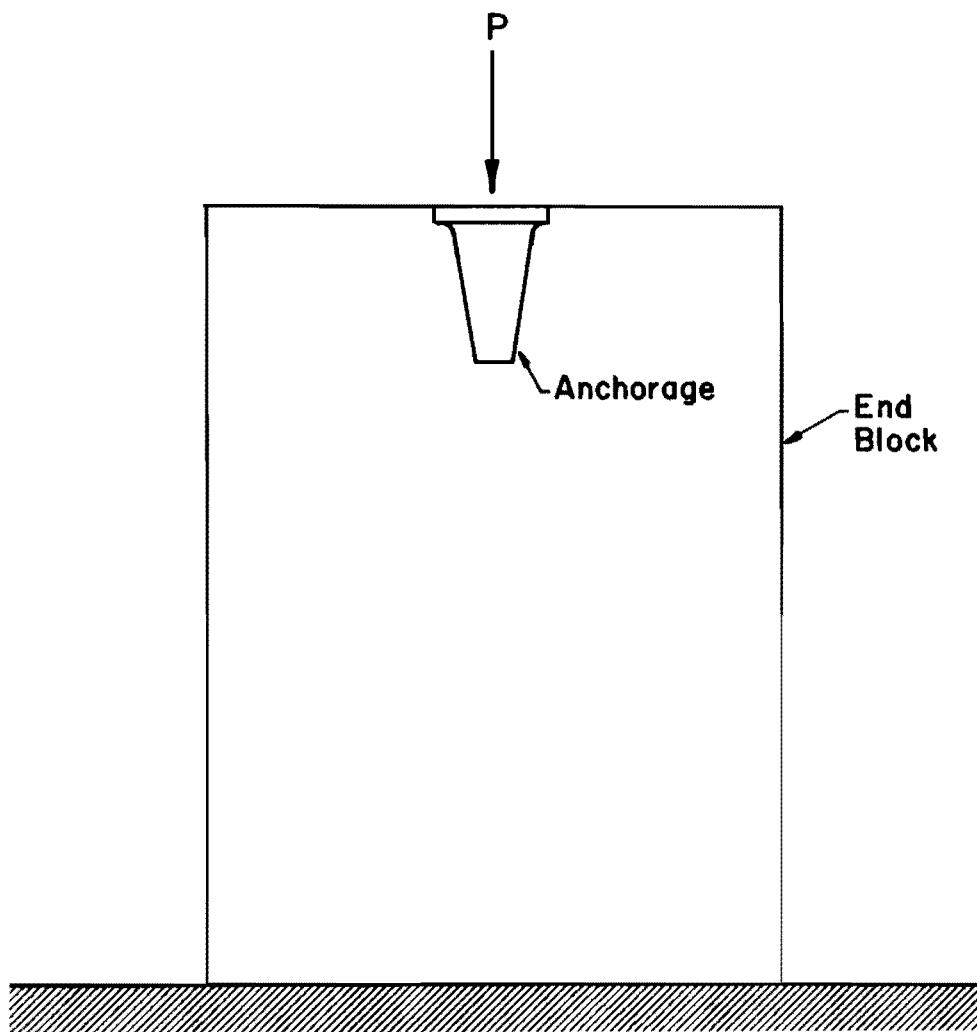


Fig. 5.44. Frequently used testing arrangement

problems can be solved in the early stages before construction begins. Hopefully, this can be a nondestructive test on one of the first units cast. Any expense incurred in performing these preliminary tests would be small when compared to the expense of repairing a large number of box units after the construction has begun.

5.8.14 Anchorage Ultimate Capacity. In the discussion of test results, attention has focused on the cracking load and factors affecting initial cracking. In all tests, the anchorage behavior in terms of ultimate capacity was satisfactory. In an actual structure, the principal function of the anchorage is to develop the tendon strength. The tendon design load,  $P_D$ , represented 80 percent of tendon ultimate strength ( $0.8f_{pu}$ ). Thus, any test which developed  $1.25 P_D$  would fully develop the tendon ultimate strength. Table 5.1 shows that the minimum value of the anchorage design load developed was  $1.50 P_D$ . Most testing was discontinued at about  $1.75 - 1.80 P_D$  without having reached the full anchorage capacity. Thus, the anchorages were fully adequate in terms of ultimate capacity.

## 5.9 Conclusions

Based on the results presented, the following conclusions can be made. Since this study is an exploratory investigation, and the number of test specimens was few, these conclusions should be considered as qualitative.

- (1) The cracking behavior of the one-sixth scale model box girder and model I section is representative of the behavior of the prototype web.
- (2) The web cracking did not endanger the ultimate strength of the webs, except as a possible path for corrosion effects.
- (3) Premature cracking along the tendon path appears to have been caused by a combination of three factors: conical anchorage, inadequate anchorage zone reinforcement, and radial stresses due to tendon curvature. Since the tendon curvature was fixed by the overall bridge design, only anchorage geometry and anchorage zone

reinforcement can be considered variable parameters. In other designs, reduction of tendon curvature would lessen splitting tendencies.

- (4) The load at first cracking appears to be only slightly affected by appreciable differences in the concrete compressive strength.
- (5) Failure to replace all web and top slab reinforcement after anchorage installation and duct placement was not a significant factor leading to the premature cracking along the tendon path. However, removal of reinforcement to allow placement of hardware should be avoided to ensure structural integrity. Reinforcement which must be cut should always be replaced.
- (6) All anchorages tested were fully adequate in terms of ultimate capacity with the minimum test result developing  $1.50 P_D$  (which is  $1.2 f_{pu}$ ).
- (7) Specimens using bearing type anchorages exhibited much better overall performance than similar specimens with cone type anchorages. Specimens having tendon pattern B and using bearing anchorages were able to achieve the design prestress force without cracks forming along the tendon path. A specimen having tendon pattern A and using bearing anchorages was not able to reach design prestress force before cracking occurred, although the cracking load was nearly twice that of similar specimens with conical type anchorages.
- (8) Doubling the percentage of web reinforcement (passive reinforcement) did not increase the cracking load, although it was effective in controlling crack width.
- (9) Substantially longer spiral reinforcement (approximately half the length of the segment) was very effective in delaying first cracking.
- (10) Increasing the surface area of the bearing anchorage 50 percent did not appreciably increase the cracking load.

- (11) Vertically post-tensioning the web (active reinforcement) promises to be a highly effective means of resisting cracking along the tendon path.

#### 5.10 Recommendations

Based on the experience gained during the construction of the prototype and in the model studies, the following recommendations are made:

- (1) There is a definite need for development of design procedures for anchorage zones in thin web box girders where inclination or curvature of the tendon path exists.
- (2) Until completion of such a study, as an interim design measure webs should be vertically post-tensioned or tendons should be encased in adequate local spirals for approximately half the segment length or to the point of tangency.
- (3) The severity of the problem could be reduced by using more gradual transition curves or less inclination in tendons.
- (4) There is a definite need for development of a clearer set of analysis criteria to check the tendon anchorage details proposed by tendon suppliers. It is desirable that the design engineer take a more active role in specifying or checking anchorage zone details.
- (5) The anchorage problem in segmental construction can be somewhat alleviated by increasing web thickness in the area of anchorage zones, by using external anchorages (outside the web) where feasible and by anchoring some tendons in the top or lower flanges, thus minimizing web congestion.
- (6) Unless detailed experience is available with a tendon anchorage device and segment details, a pilot test should be run on the first units cast to ensure adequacy of proposed details. This type of test can be performed at a very nominal cost in the plant using the stressing equipment and a reaction bearing plate at the far end of the segment.

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## C H A P T E R    6

### CONCLUSIONS AND RECOMMENDATIONS

#### 6.1 Conclusions

Methods for obtaining efficient and economical construction of long span bridges built with the use of segmentally precast box girders have been presented. Emphasis has been placed on reducing potential difficulties, describing problems and solutions which were experienced in construction of the model and prototype bridge at Corpus Christi, Texas. Many bridges have been erected similarly in other countries; a considerable potential may be realized with the use of these bridges in the United States. There are two general types of segmental bridges: those erected on falsework and those erected by the cantilever method. Many applications exist for both types, with economics determining the best one for each application. In general, only those bridges which can be erected on relatively inexpensive support systems will be designed for erection on falsework; erection by the cantilever method will be most common in inaccessible areas, water crossings, and locations where traffic patterns must not be interrupted.

The industrialization of this type of construction requires complete coordination; procedures must be established to ensure that all persons concerned with any particular bridge are aware of their effect on the entire project. Coordination must begin with the initial site selection and continue through the design stage, production stage, and erection stage. Preliminary planning is very important in the attainment of maximum efficiency. Seemingly insignificant items may be of primary concern at some particular phase of the construction. The importance of standardization should be reiterated. Mass production techniques require a maximum of standardization. Depending on local conditions, every effort should be made toward the use of common details in all segments. This means economy

through reduced materials, labor, and construction time. The use of standardized cross sections seems to offer the most benefits. Other items include the tendons, tendon layouts, anchorages, reinforcement, and shear keys. Typically, bridges with span lengths more than 300 ft. will be haunched, thereby precluding standard cross sections. However, proper planning can eliminate the need for expensive forming techniques.

The casting procedures can be generally the same whether the bridges are to be erected on falsework or in cantilever. Anchorage details are usually different between the two types. Casting segments end-to-end is the only practical means of achieving the accuracy which is required in the segments. Errors in the casting can cause serious alignment problems during the erection stage. However, any errors made in casting of one segment can usually be corrected with the succeeding unit. Forming procedures were presented to aid in the planning stage of construction. Forms should be adaptable to suit various configurations in the tendon patterns, shear keys, reinforcement, and other details which may be present. Care must be exercised to ensure that there are positive means of providing alignment in all of the ducts as required; misalignment can cause serious and costly delays. Internal stiffening of ducts is particularly recommended. In all bridges there will be certain segments which require special attention. These should be kept to a practical minimum. Other segments may be most economically produced in assembly-line fashion.

Erection of the segments on falsework can be an economical method of assembly, especially if the supports can be rather rapidly positioned. The use of falsework will depend largely on the available equipment, as well as the local terrain. Erection by the cantilever method is more complicated, but can offer significant advantages. With the use of epoxied joints, erection rates are independent of the joint curing time. Pier segments are fastened to the piers in some manner to prevent rotation during the placement of other segments. Segments may be positioned by any of several methods. They are usually held in position by other equipment until stressing of the tendons is completed. The closure joints have to be cast in place because the tolerances would otherwise be too critical. A means must be provided to ensure that the joint remains unstressed while

the concrete cures. The completion of the structure involves procedures common to many other bridges in which the bearings are adjusted to a prescribed reaction.

Numerous minor problems which can occur in segmental cantilever construction, such as duct blockage, excessive friction, grouting cross-over, and section spalling, are described. Suggested solutions and precautionary procedures are given. Improvements in precasting procedures are suggested.

The most serious problem which occurred in the construction of the Corpus Christi bridge was the formation of web cracks in the vicinity of the anchorages along the tendon ducts. The exploratory test program indicated that these cracks were caused by combined tensile stresses due to the conical type of anchor used, the normal bursting stress in front of a concentrated load, and the radial stresses due to tendon curvature in the vicinity of the anchor. Tests indicated that the supplementary reinforcement provided was insufficient to prevent formation of surface cracks but that the web reinforcement was adequate to carry structural forces and control crack size. Provision of active reinforcement in the nature of spirals around the anchors and ducts as used in the model construction or by use of vertical post-tensioning would have prevented formation of surface cracking.

## 6.2 Recommendations

The Corpus Christi bridge is an excellent example for demonstrating the industrialization possible with segmentally precast bridges within traditional construction relationships in the United States. Close liaison was maintained throughout the duration of the project. Design changes were made to provide for easier construction and erection of the bridge. Training seminars were held to make all those concerned more familiar with the construction techniques. Many of the casting and erection problems were solved and demonstrated with the model. Information developed from this project should be widely disseminated to assist designers and constructors of similar structures.

When several segmental bridge projects have been completed, a study committee should be established, perhaps on a national basis, to review specific procedures used in bridges built both in the United States and in other countries. Evaluations of the most successful methods should be provided in the form of guidelines for designers and contractors, with specific recommendations for the various details which might be common to all types of segmental bridges. Cost studies should also be included. They could be analyzed to show the effect of changes in design or in construction procedures. Such a move towards standardization should not be started prematurely, as the constructors of North America have had little opportunity to innovate in this type construction. Relatively more attention should be paid to pier design and to partial and full falsework methods of construction.

### 6.3 Detailed Recommendations

Because of the large number of specific detailed recommendations presented in this report, they are summarized at the end of each chapter and not repeated herein. Most of these are embodied in the special project specifications of Appendix B2.

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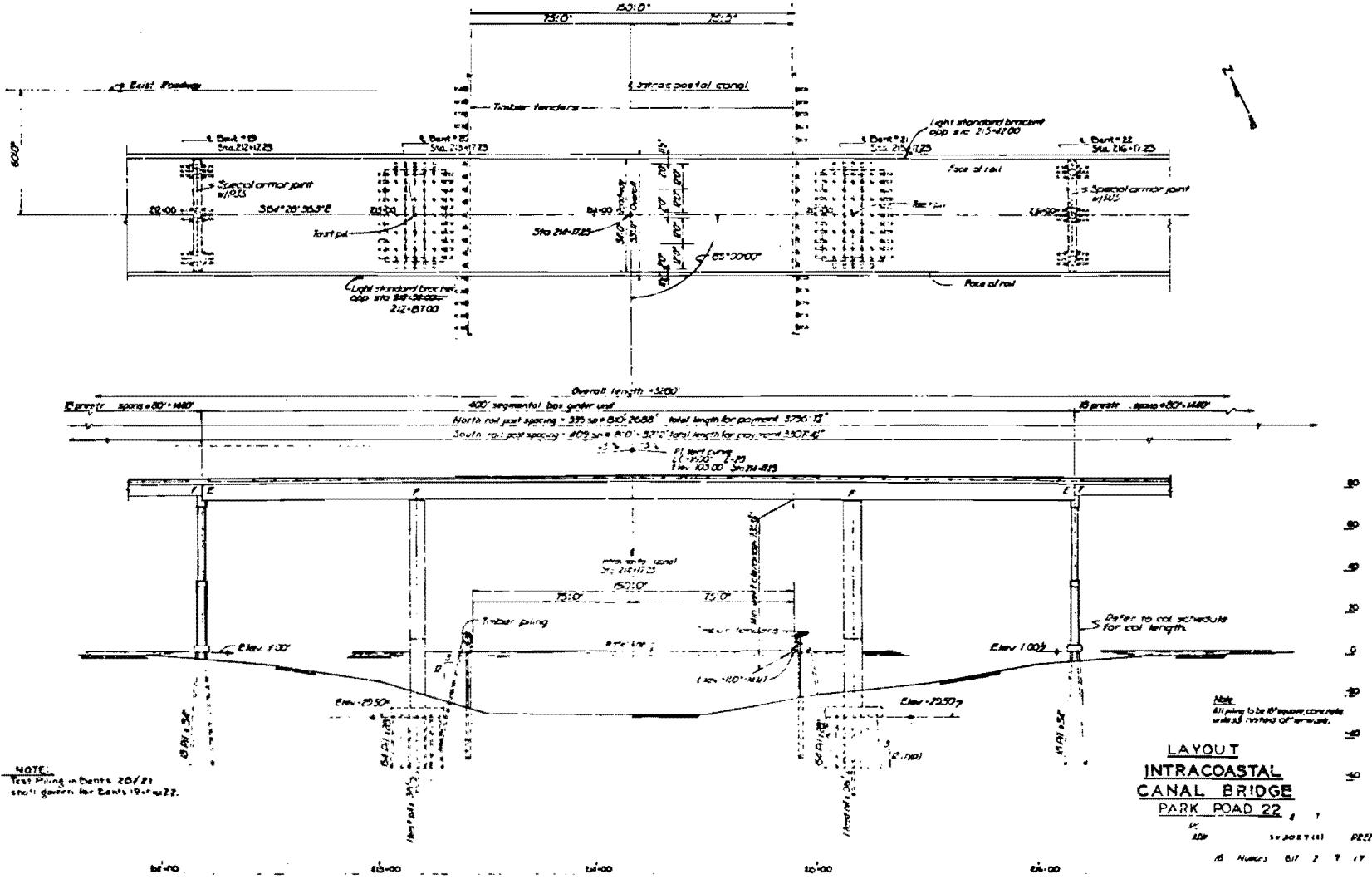
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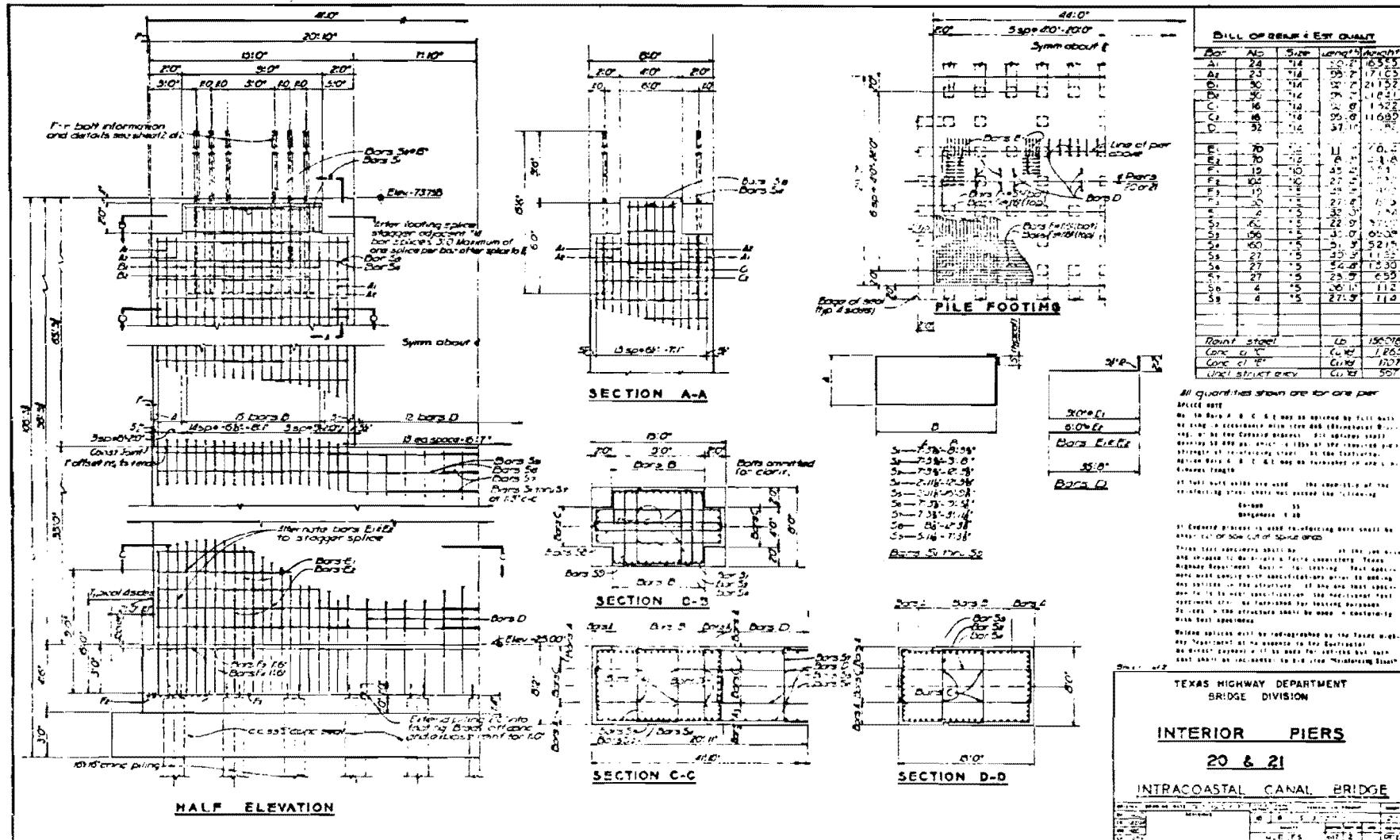
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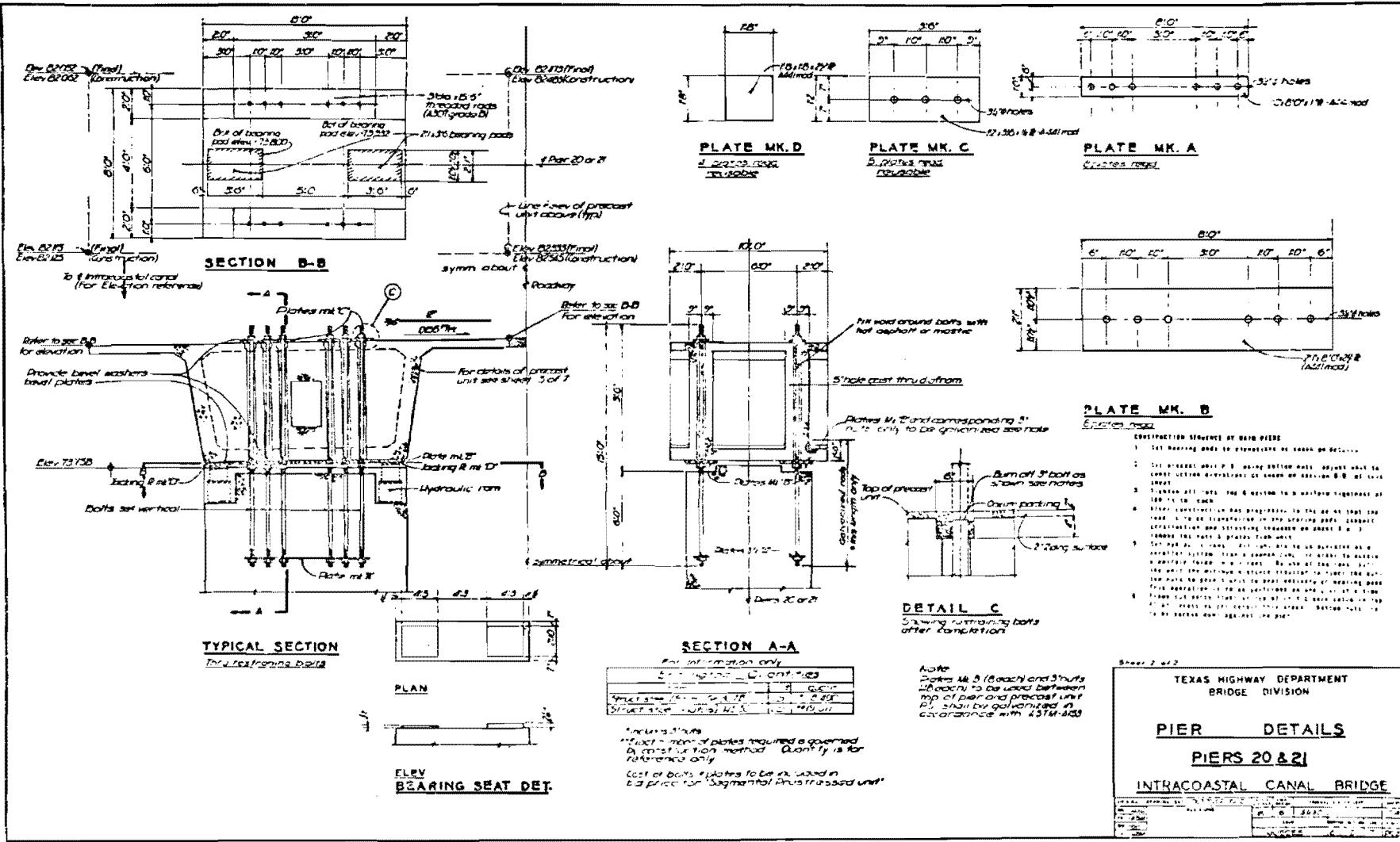
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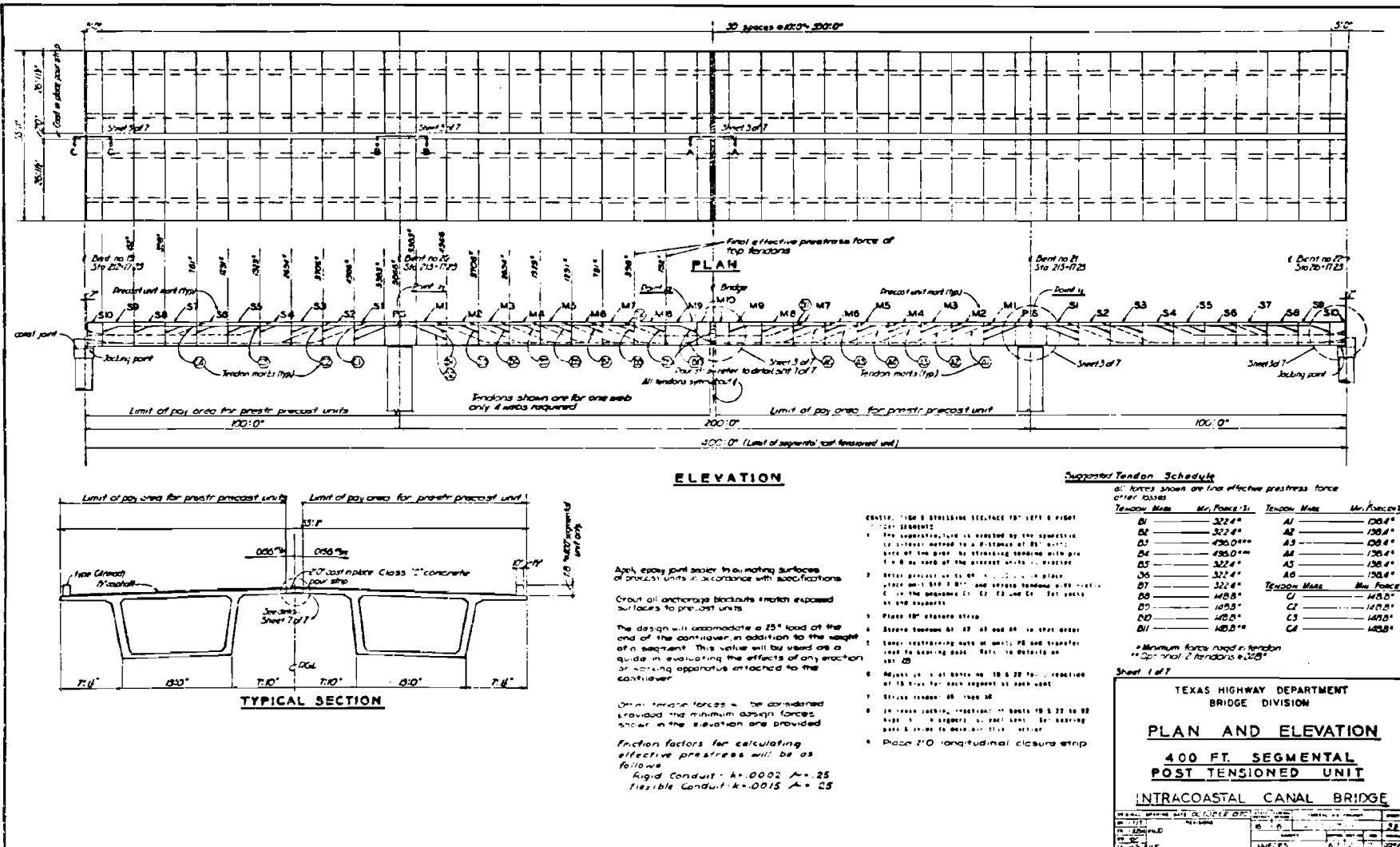
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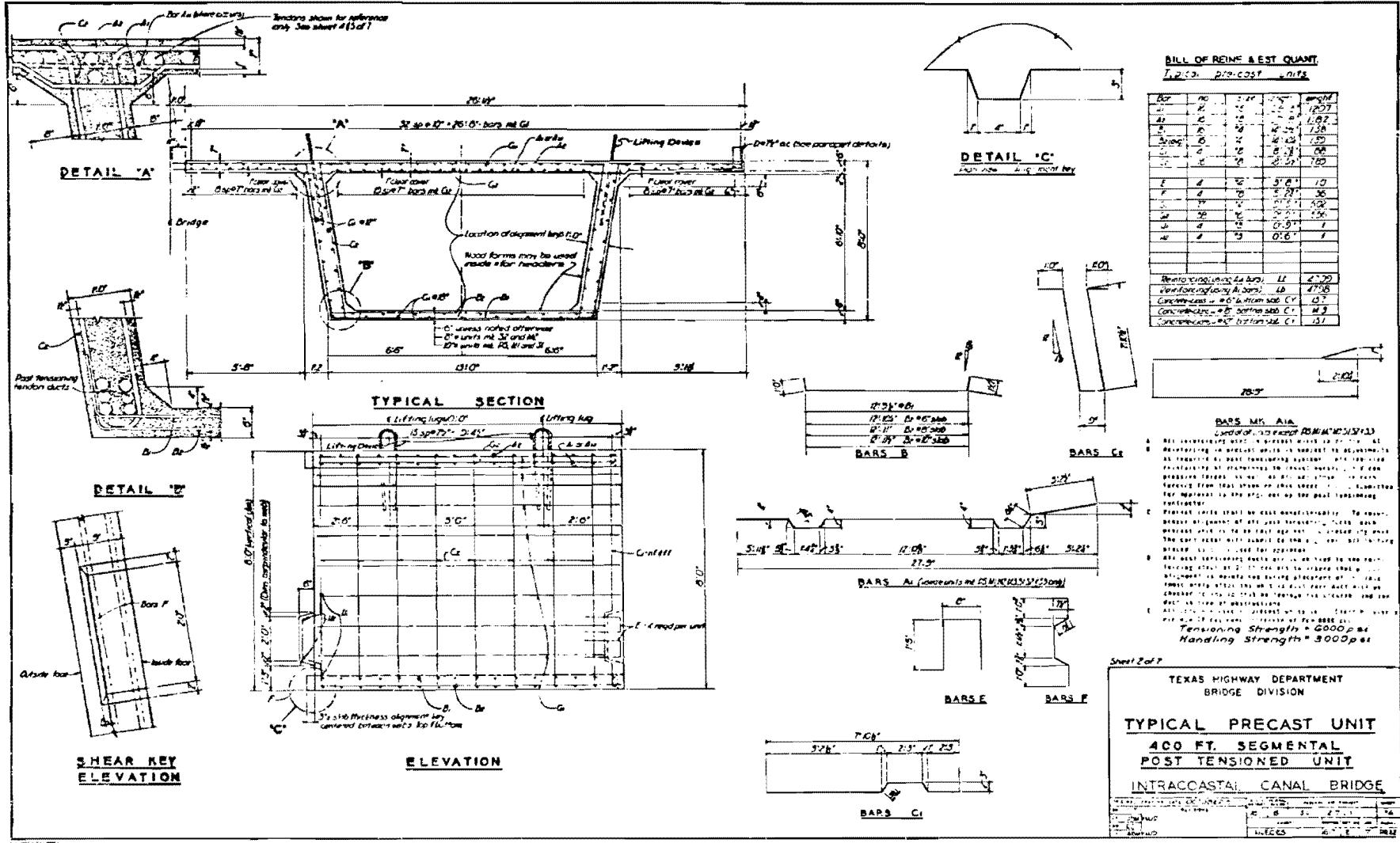
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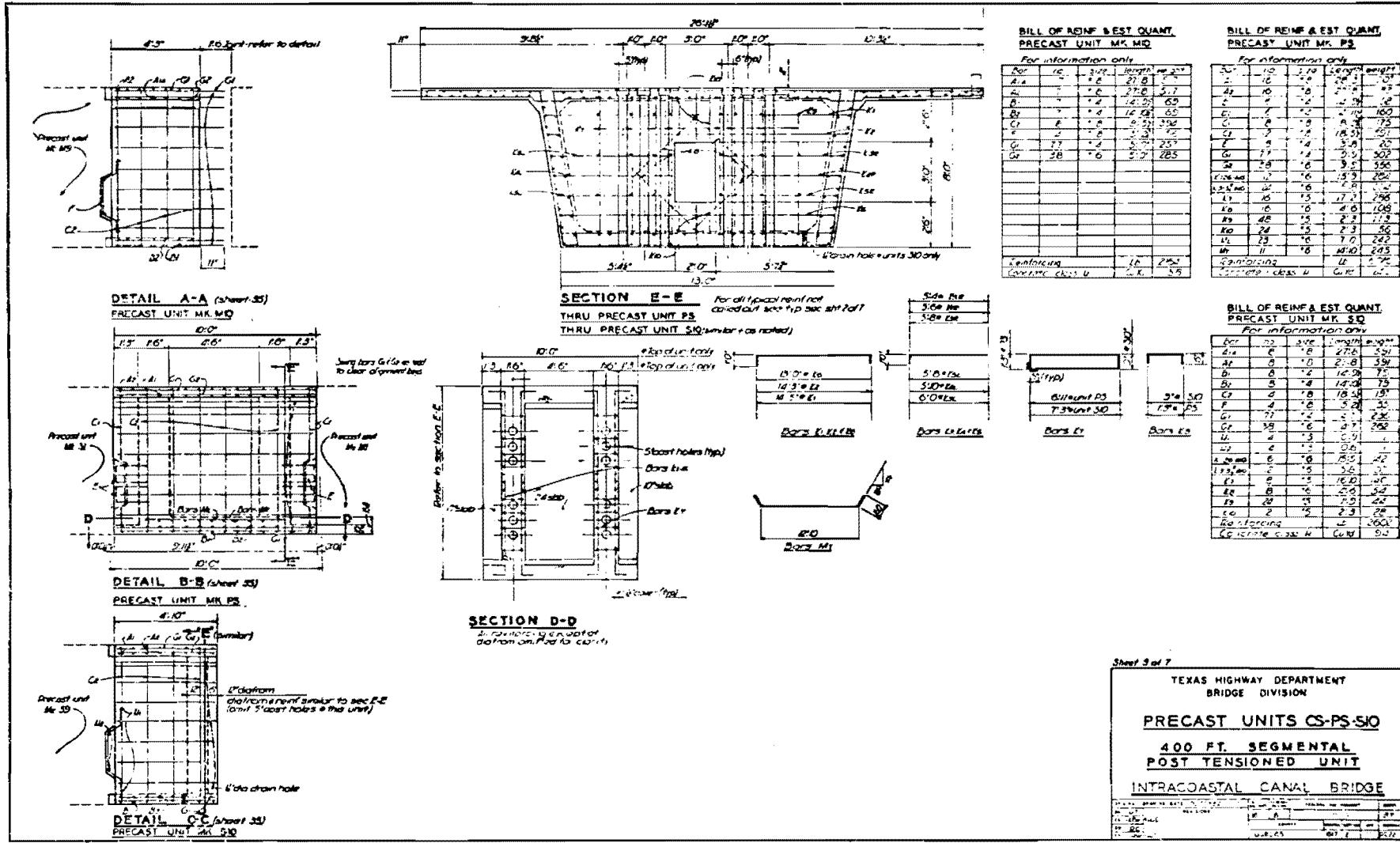


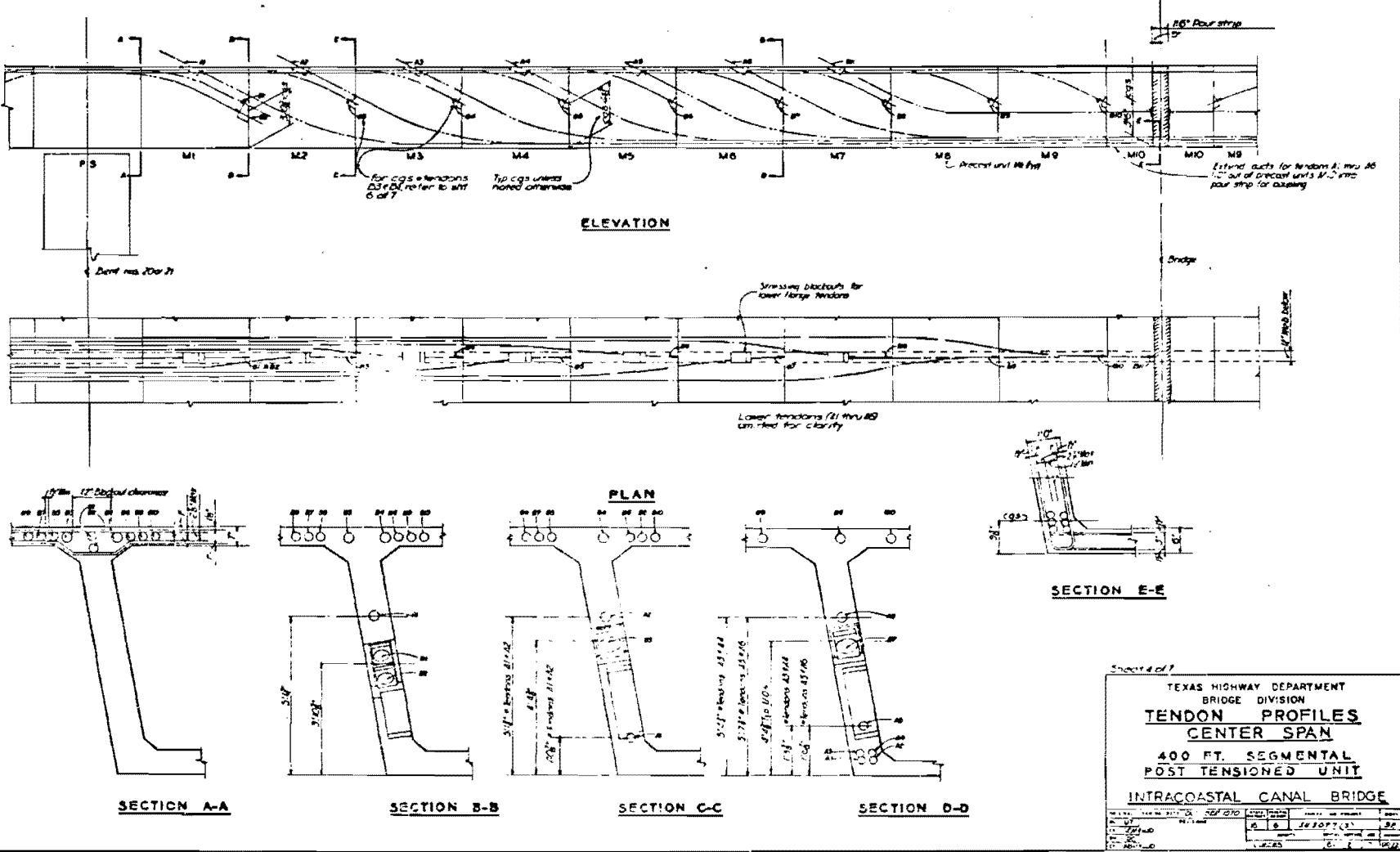


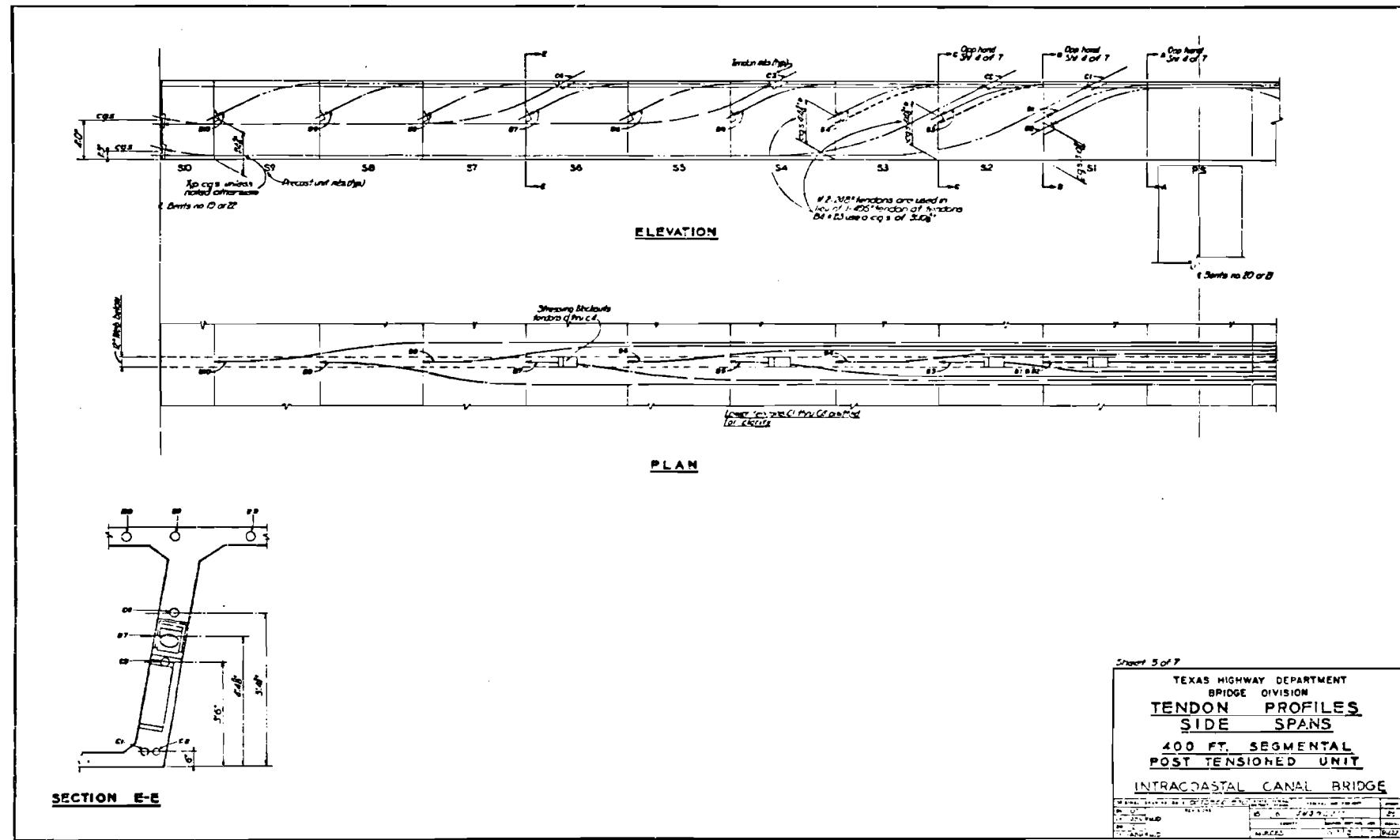


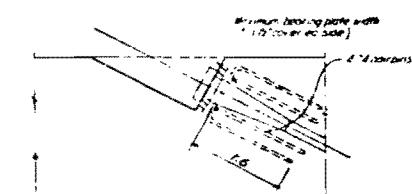
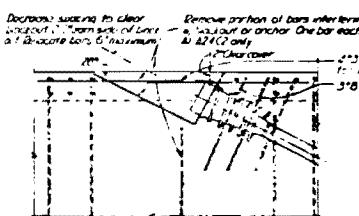
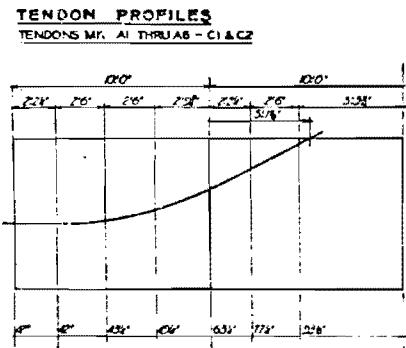
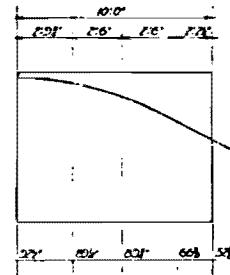
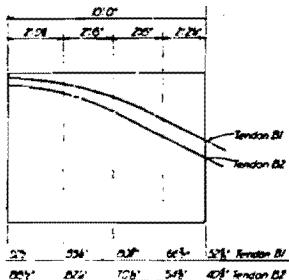
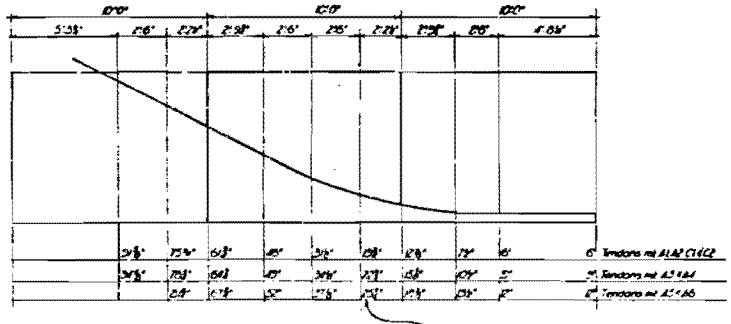




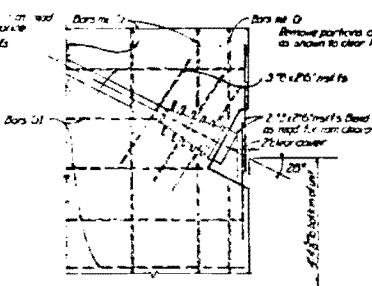






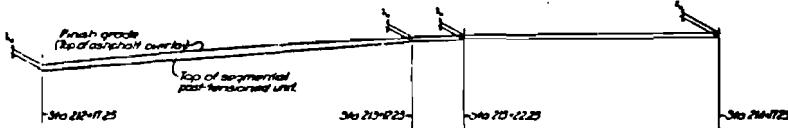


TENDON ANCHORAGE  
BEARING PLATE TYPE ANCHOR  
TYPICAL DETAIL



Note:  
Tendon anchorage details are not intended to limit types of strand systems used.

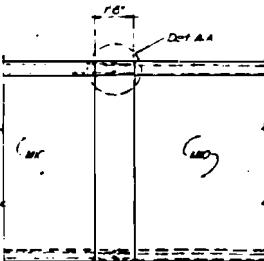
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<b>TENDON PROFILES AND DETAILS</b>																																									
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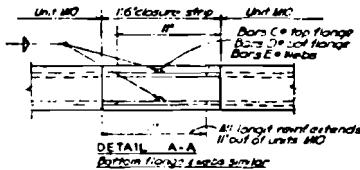
## BLOCKING DIAGRAM



SECTION THRU LONGITUDINAL JOINT



ELEV. TRANSVERSE JOINT



43  
Total of 114 cu ft and 227 cu yd reinforcing  
for the various joints are to be included in bid  
price for segmental prestressed unit

<sup>a</sup>Includes 6 v.0 tops.

TEXAS HIGHWAY DEPARTMENT  
BRIDGE DIVISION

**BLOCKING & DETAILS**

**400 FT. SEGMENTAL  
 POST TENSIONED UNIT**

**INTRACOASTAL CANAL BRIDGE**

A P P E N D I X      B1

TEXAS HIGHWAY DEPARTMENT SPECIAL SPECIFICATIONS

CORPUS CHRISTI BRIDGE

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-- CTR Library Digitization Team

## TEXAS HIGHWAY DEPARTMENT

## SPECIAL PROVISION

TO

## ITEM 425

## PRESTRESSED CONCRETE STRUCTURES

For this project, Item 425, "Prestressed Concrete Structures", is hereby amended with respect to the clauses cited below.

Articles 425.1 through 425.9 are hereby voided and replaced by the following:

425.1. Description. This item shall govern for the construction, pre-stressing and erection of precast prestressed concrete members, in accordance with the plans, with approved shop drawings and these specifications.

425.2. General. The method of construction and of prestressing shall comply with the requirement of the plans and in accordance with approved shop drawings. Prior to beginning the casting of members, the Contractor shall give the Engineer ample notice as to the location of the casting site and the date on which work will begin.

An inspection laboratory shall be furnished in accordance with Item 498, "Plant Inspection Laboratory (Equipped)".

Shop Plans showing the following information shall be submitted for approval. The number of copies required, and routing, shall be as shown in Table 1. When fabrication is to be at more than one plant, two additional copies of drawings shall be submitted for each additional casting location.

A. Erection Plans. An erection plan shall be submitted for approval showing information for field erection (location, type member, erection mark of member, bearing pads with marks, etc.). The erection mark system employed shall not conflict with beam designations shown on the contract drawing. On projects requiring numerous types of beams of various lengths and strand patterns, the erection mark system shall indicate the Structure Number, Superstructure Unit Number and Beam Number.

B. Fabrication Details. Complete information necessary for fabrication shall be submitted for approval (member lengths, type, skew angle, dimensions for diafram holes, bearing pad data, bevels, erection devices, details of reinforcement, inserts to be used in forming, etc.). On projects requiring numerous types of beams of various lengths and strand patterns, an Index sheet showing all beam and concrete data (cast length, pay length, concrete strengths, strand data, special casting devices, etc.) shall be furnished.

C. Facility Plans. Central Plant. Prior to stressing of members, drawings showing the complete facilities to be used for fabrication shall be submitted.

These drawings shall include a master plan of the bed layout, assembly and subassembly units, details of pulling head, anchor plates, anchor rods, pulling rods, posts, jacks, hold-down devices, dead and live end abutments complete with dimensions, thicknesses of plates, size and spacing of bolts, and types of materials used.

This data shall be sufficiently complete to permit accurate analysis of stresses in the stressing equipment and anchorage. Detailed drawings shall

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be prepared on sheets 22 x 36 inches and submitted for approval of the system prior to casting of any unit.

At any time changes are made in the facilities, details of these changes shall be submitted.

The design of the bed and facilities, with plans and specifications, shall be done by a Professional Engineer registered in the State of Texas. The Contractor (Fabricator) shall furnish a certificate bearing his signature, or that of a responsible Officer of the company, certifying that the bed and hardware have been constructed, fabricated and installed in accordance with the above plans and specifications.

The Contractor (Fabricator) shall specify the maximum loading for which the bed is to be used. Prior to approval of that loading, the facilities shall be proofloaded to a minimum ten percent overload for eight hours. Additional proof loads shall be performed every 12 months at a ten percent overload for four hours, if deemed necessary by the Engineer. Minor changes in facilities will not require proofloading but will require submission of the details of changes accompanied with design calculations.

**D. Facility Plans. Job Site Fabrication.** Prior to stressing of members, drawings showing complete facilities to be used shall be submitted in accordance with all requirements specified in Section 425.2.C. above.

**E. Prestressing Details.** Complete prestressing details shall be submitted showing details of the member, forms, devices for holding prestress steel in place, method and details of draping strand, anchorage details, method and details of prestressing the steel, elongations, jack pressures, and all other features of proposed prestressing. Calculations shall be included to justify the system and method of prestressing to be used.

**F. Corrected "As-Built" Shop Plans.** At the completion of the job, corrected "As-Built" shop plans shall be submitted to be incorporated as a part of the final plans of the project. The "As-Built" plans shall reflect the designs used and their location in the structure.

**G. Methods of Handling and Transporting.** Details of handling and transporting need not be submitted for approval except that inserts used for pick-up shall be shown on shop drawings.

(See Explanation Below)	A	B	C	D	E	F	Disposition
TYPE OF MEMBER	Erection Plans Fabrication Details	Facility Plans Job Site	Facility Plans Central Plant*	Prestressing Details	Corrected "As-Built" Shop Plans		Submit to Bridge Division Direct to Materials and Tests Division
1. Pretensioned Members	7	6	4	3	6	1	A, B, D, E, F C
2. Post-tensioned Members	7	6	4	3	6	1	A, B, D, E, F C
3. a. Pretensioned Piling			4	3	4		D, E <sup>+</sup> B, C, D, E, F C, E <sup>++</sup>
b. Post-tensioned Piling		6	4	4	6	1	

TABLE 1  
REQUIRED SHOP PLANS

(Numbers indicate the number of prints to be submitted.)

\*For plant operation, submission need be made only once initially and thereafter for jobs only if type of product, method or bed facilities are modified or altered.

+Job Site Casting.

++Central Plant Casting.

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425.3. "Materials". Materials for concrete and water for curing shall be in accordance with Item 421, "Concrete for Structures", and/or Item 423, "Light-weight Concrete for Structures". Materials for prestressing shall be in accordance with Item 426, "Prestressing". Materials for reinforcing steel (non-prestressed) shall be in accordance with Item 440, "Reinforcing Steel". Structural steel bearing plates, fittings, etc., shall be in accordance with Item 441, "Steel Structures", and Item 442, "Metal for Structures". Bearing pads shall be in accordance with Item 435, "Elastomeric Materials", and with Special Specifications contained in the contract. Special Provisions contained in the contract to any of the above items shall govern.

425.4. "Construction Methods". Prestressing shall be in accordance with Item 426, "Prestressing".

Reinforcing steel shall be fabricated and placed in accordance with the plans and as required herein.

The construction of forms and the placing, curing and finishing of concrete shall be in accordance with the provisions contained herein and notes shown on the plans.

A. Forms. All side and bottom forms for precast prestressed concrete construction shall be constructed of steel, unless otherwise noted on the plans. End headers and inside forms may be of other material as approved on the shop drawings.

Forms shall be of sufficient thickness, with adequate external bracing and stiffeners, and shall be sufficiently anchored to withstand the forces due to placement and vibration of concrete. Internal bracing and holding devices in forms will not be permitted if such would remain in the finished prestressed member. Joints shall be maintained reasonably mortar tight.

The grade and alignment of forms shall be checked each time they are set and shall be maintained during the casting of concrete.

Metal forms shall be reasonably free from rust, grease or other foreign materials. All forms shall be cleaned thoroughly prior to each casting operation.

Wood forms, when permitted, shall conform to requirements of Item 420.9.

The soffit for casting members shall be constructed and maintained to provide not more than 1/4-inch variation in any 50-foot length of the bed from the theoretical plane of the bottom of the member.

Forms for internal voids in members shall be anchored securely to prevent movement or misalignment during the placing of concrete. For forming internal voids with a mandrel, special attention shall be given to maintaining the correct position and alignment of the mandrel throughout the casting operation.

The facing of all forms shall be treated with form oil or other bond breaking coating prior to placing of concrete. The oil or other materials used for this purpose shall be of a consistency and composition to facilitate form removal. Materials which appreciably darken or react with the concrete will not be permitted.

All forms shall be constructed to facilitate removal without damage to the concrete. At the Contractor's option, forms for piling may be constructed with a 1/8-inch draft to permit ease of removal.

B. Placing Concrete. All concrete shall be placed during daylight hours unless the fabrication plant or site is provided with an approved lighting system.

The method of concrete placement shall avoid segregation of the aggregate or displacement of the reinforcing steel and/or prestressed tendons. Concrete shall be deposited as near as possible in its final position in the forms. Depositing large quantities of concrete at one location in the forms and running or working it along the forms will not be permitted.

Special attention shall be directed toward working the coarse aggregate back from the face of the concrete and to forcing the concrete under and around the prestressed tendons and reinforcing steel.

Concrete may be placed in one lift. At the Contractor's option, concrete may be placed in beams and girders in multiple continuous horizontal layers. In such instances, the thickness of the first layer shall be slightly above the juncture of the bottom flange and web. Not more than one hour shall elapse between the placing of the successive layers. Vibration of subsequent layers of concrete shall extend into the previously placed layers as specified in 425.4.C. Vibration.

When casting concrete piling or concrete slab units, the concrete shall be placed in one continuous horizontal layer.

Concrete shall not be placed at outdoor casting beds during inclement weather or when impending weather conditions may result in rainfall or low temperature during the casting operation which might impair the quality of the finished member. In case rainfall should occur after placing operations are underway, the Contractor shall provide adequate covering to protect exposed concrete. The completion of a member being cast will be permitted, provided adequate provisions are made to prevent damage to the concrete.

1. Placing Concrete in Cold Weather. When members are produced in a fabricating plant which has adequate provisions to protect the concrete when placed and which has approved steam curing facilities, concrete may be placed under any low temperature conditions provided:

a. The temperature of the concrete is not less than 50 F nor more than 90 F when placed in the forms.

b. The framework and covering are placed and heat is provided for the concrete and forms within one hour after the concrete is placed. This shall not be construed to be one hour after the last concrete is placed but that no concrete shall remain unprotected and unheated for longer than one hour.

c. Steam heat shall keep the air surrounding the concrete between 50 F and 90 F for a minimum of three hours prior to beginning the temperature rise which is required for steam curing.

d. The temperature of the concrete shall not be less than 50 F at any time after all materials are added and mixing commences.

For central fabrication plants and job site casting operations which do not provide facilities necessary to accomplish the above provisions, concrete may be placed when the atmospheric temperature is 35 F or greater. The temperature of the concrete at the time of placement shall not be less than 50 F nor more than 90 F. The concrete shall not be placed in contact with

any material having a temperature less than 32 F or any material coated with frost.

Aggregates shall be free from ice, frost and frozen lumps. When required, in order to produce the minimum temperature specified above, the aggregate and/or the water shall be heated uniformly in accordance with the following:

Water shall be heated to a temperature not to exceed 180 F and/or the aggregate shall be heated to a temperature not to exceed 150 F. The equipment furnished shall be capable of heating the aggregate uniformly to eliminate overheated areas in the stock pile which might cause flash set of the cement. The temperature of the mixture of the aggregates and water shall be between 50 F and 90 F before introduction of the cement.

Protection shall be provided to maintain the temperature of the concrete at all surfaces above 50 F for the required total curing time as specified in 425.4.E. Protection shall consist of providing additional covering and, if necessary, supplementing such covering with artificial heating. When impending weather conditions indicate the possibility of the need for such temperature protection, all necessary heating equipment and covering material shall be on hand ready for use before permission is granted by the Engineer to begin placement of concrete.

2. Placing Concrete in Hot Weather. When concrete is to be placed during hot weather, the concrete shall be placed without the addition of more water to the concrete than required by the design (slump and consistency), and the concrete shall be finished properly without adding water on the surface. Control of the initial set of the concrete and lengthening the time for finishing operations, under adverse wind, humidity and hot weather conditions, may be accomplished with the use of an approved cement dispersing agent (retarder) in accordance with Item 421.7.

The maximum time interval between the addition of mixing water and/or cement to the batch, and the placing of concrete in the forms shall not exceed the following:

AIR OR CONCRETE TEMPERATURE <u>(WHICHEVER IS HIGHER)</u>	MAXIMUM TIME (ADDITION OF WATER OR CEMENT TO PLACING IN FORMS)
NON-AGITATED CONCRETE	
Over 80 F	15 Minutes
50 F to 79 F	30 Minutes
AGITATED CONCRETE	
90 F or Above	45 Minutes
75 F to 89 F	60 Minutes
50 F to 74 F	90 Minutes

The use of an approved cement dispersing agent (retarder) in the concrete will permit the extension of each of the above temperature time maximum by 30 minutes, except that for non-agitated concrete, the maximum time shall not exceed 30 minutes.

Under conditions of extreme temperature, wind or humidity, when the specified temperature-time maximums are excessive, the Engineer may require the use of an approved cement dispersing agent (retarder), or may suspend concrete placing operations, if quality concrete is not being placed.

The values which govern for minimum concrete strengths during different phases of construction shall be as shown on approved shop drawings.

For Class H, Y and Z Concrete, the control of the concrete shall be by compressive tests of cylinders. An adequate number of cylinders will be made for each pertinent strength test required. For determining "Release Strength" of members, a test shall be defined as the average of the breaking strength of two cylinders.

All test specimens, beams or cylinders representing tests for removal of forms and/or falsework and for "Release Strength" shall be cured under the same conditions, be subjected to the same curing materials and to the same weather conditions as the concrete represented.

"Design Strength" cylinders for acceptance of members shall be cured with the member which the cylinders represent until release of stress or until partial tensioning strength is obtained. These cylinders shall then be cured for the remainder of the test period in accordance with Test Method Tex-704-I in a curing tank as described in Item 498.

C. Vibration of Concrete. All concrete shall be compacted and the mortar flushed to the surface of the forms by continuous working with approved high frequency mechanical vibrators, operating at a minimum of 7,000 impulses per minute. Use of external vibrators in conjunction with internal vibrators will be permitted when the forms are of steel.

At least one stand-by vibrator shall be provided for emergency use to avoid delays in vibrating due to breakdowns.

The vibrators shall be inserted systematically into the concrete immediately after deposit, thoroughly consolidating and working the concrete around the reinforcement, and into the corners and angles of the forms until it has been reduced to a plastic mass. When the concrete is placed in more than one layer, the vibrator shall be operated so that it will penetrate the previously placed layer of concrete. The vibration shall be of sufficient duration to accomplish thorough compaction and complete embedment of the reinforcing steel and prestressed tendons, but not so excessive as to result in segregation. Vibration shall be supplemented by hand spading, if necessary, to insure the flushing of mortar to the surface of all forms.

D. Finishing of Concrete. Top surfaces of prestressed concrete members against which cast-in-place concrete will be placed later shall be screeded or rough floated to bring grout to the surface and cover all aggregate. At the approximate time of initial set, the surface shall be roughened by brushing, brooming or other approved methods. Sound concrete shall not be removed or aggregate loosened. Fresh concrete shall be removed from exposed reinforcing steel.

The top surfaces of beams upon which panels are to be placed shall be finished smooth from the reinforcing bar out to the outside edges. The center portion of these beams shall be roughened.

Top surfaces of members which are to be the riding surface shall be given the

same finish required if the concrete were cast-in-place. Roadway surfaces which are to be given a wearing course, shall be screeded and given a wood float finish.

Erection holes (lifting eyes, form anchors, etc.) in exterior beams shall be filled with mortar and made flush with the surrounding surface. Holes in interior beams need not be filled unless steel is exposed. Erection or fabrication holes in the bottom of all beams shall be filled with non-stain, non-shrink mortar and made flush with the surrounding surface.

Form marks in excess of that permitted in section 425.7. and all fins and rough edges along chamfer lines shall be removed in an acceptable manner.

Unless otherwise shown on plans, strands shall be removed flush with the end of the member. The ends of the strands and a minimum of 1 inch around each strand shall be cleaned and coated with approximately 10 mils of an acceptable, commercial grade epoxy.

After slab placement, the outside and bottom surfaces of exterior beams shall be given the same type surface finish specified for the structure.

**E. Curing of Concrete.** Careful attention shall be given to the proper curing of concrete. The Contractor shall inform the Engineer regarding the methods and procedures proposed for curing; shall provide the proper equipment and necessary materials; and, shall have approval of the Engineer of such methods, equipment and materials prior to placing concrete.

Inadequate curing facilities or lack of attention to the proper curing of concrete shall be cause for the Engineer to stop all construction until approved curing is provided. Inadequate curing may be cause for rejection of the member.

Side forms may be removed at the discretion of the Contractor at any time after the concrete has reached sufficient strength to prevent physical damage to the member. Weight supporting forms shall remain in place until the concrete has reached the handling strength shown on the plans. Removal of the forms shall be done in such a manner that curing is not interrupted on any member by more than 30 minutes. Cracks caused by form restriction may be cause for rejection.

Curing shall be commenced prior to the formation of surface shrinkage cracks but in no case delayed longer than one hour after the concrete has been placed in the forms.

An approved water or membrane cure (when permitted) shall be used as an interim measure prior to elevated temperature or other methods of curing.

Concrete shall be cured continuously, except as provided for form removal, until the concrete strength as indicated by compressive test of cylinders cured with the members, has reached the release strength or handling strength designated on the plans or shop drawings. Concrete piling shall be steam or water cured for an additional three curing days. Other members shall be covered to prevent rapid drying for a period of 72 hours after release of tension. All members shall be protected from freezing during the above period.

A period not to exceed four hours will be permitted for removal to a storage area prior to resuming the balance of curing and protection required.

A curing day is defined as a calendar day when the temperature, taken in the shade away from artificial heat, is above 50 F for at least 19 hours, or for colder days, if satisfactory provisions are made to maintain the temperature at all surfaces of the concrete above 50 F for the entire 24 hours.

All concrete shall be steam or water cured, except that membrane curing may be used as interim curing on the top surface of concrete piling. Only Type 1 membrane curing compound will be permitted for interim curing.

1. Water Curing. All exposed surfaces of the concrete shall be kept wet continuously for the required curing time. The water used for curing shall meet the requirements for concrete mixing water as specified in Item 421, "Concrete for Structures". Sea water will not be permitted. Water which stains or leaves an unsightly residue shall not be used.

Water curing will be permitted as follows:

a. Wet Mat Method. For water curing by the wet mat method, cotton mats, polyethylene sheeting, or polyethylene burlap blankets may be used.

The mats, sheets, or blankets shall not be placed in contact with the pre-stressed concrete member until such time that damage will not occur to the surfaces.

The mats, sheets, or blankets shall be adequately anchored and weighted to provide continuous contact with all concrete surfaces. Any concrete surfaces which cannot be cured by contact shall be enclosed by mats, adequately anchored, so that outside air cannot enter the enclosure. Sufficient moisture shall be provided inside the enclosure to keep all of the surfaces of the concrete wet for the required curing time.

b. Water Spray Method. For water curing by the water spray method, overlapping sprays or sprinklers shall be used so that all concrete surfaces are kept wet continuously.

2. Elevated Temperature Curing. Curing by elevated temperature will be permitted as follows:

a. Steam Curing. (Steam curing is defined as use of steam above 90 F for curing.) When steam curing of concrete is provided, the temperature inside the curing jacket at the surface of the concrete shall not exceed 165 F for more than one hour during the entire steam curing period. Concrete exposed to temperatures exceeding 180 F will not be accepted.

Sufficient moisture shall be provided inside the curing jacket so that all surfaces of the concrete are wet.

An unobstructed air space of not less than six inches shall be provided between all surfaces of the concrete and the curing jacket. Steam outlets shall be positioned so that live steam is not applied directly on the concrete, reinforcing steel or tendons.

The location of steam lines, location of control points for discharge of steam into the curing jacket, and the number and type of openings for steam distribution within the curing jacket shall be arranged in such manner that temperature variation between any points in the enclosure shall not exceed 20 F.

Steam curing shall not commence until the concrete has been in place a minimum of three hours.

During the application of steam, the temperature inside the curing jacket should be raised uniformly at a rate not to exceed 40 F per hour.

Temperature decrease at the end of the curing operation shall not exceed the same rate.

When elevated temperature curing is used, the release of stress shall be done prior to reaching 90 F during temperature reduction. Members shall remain protected until the differential between the temperature inside the curing jacket and the outside air is not more than 25 F.

b. Alternate Methods. Other methods of elevated temperature curing may be permitted by the Engineer provided that temperature maximums, rate of temperature variation, humidity control, etc., are in accordance with the requirements for steam curing. Permission shall be obtained from the Engineer, in writing, for any alternate method.

425.5. "Handling, Hauling and Erection". The Contractor (Fabricator) shall be responsible for proper handling, lifting, storing, hauling and erection of all members so that they may be placed in the structure without damage.

Beams or girders shall be maintained in an upright position at all times and shall be picked up and supported near the end of the member in such a way to prevent torsion. Members may be lifted with the lifting devices as approved on the shop plans or by other methods approved by the Engineer in writing.

Piling shall be handled in accordance with THD Bulletin C-15.

No member shall be moved from the casting yard until all requirements for tensioning, curing and strength have been attained.

All concrete beams or girders, placed over a traveled roadway or railroad, shall be securely tied and/or braced to prevent overturning until diaframes capable of providing lateral stability are permanently in place.

425.6. "Defects and Breakage". If any prestressing tendon or portion thereof is broken prior to placing concrete in the member, it shall be replaced with a satisfactory unit properly prestressed. The breaking of one wire of a seven wire strand in a unit during concrete placing operations will be subject to a structural review prior to acceptance.

Fine hair cracks or checks on the surface of the member which, as determined by the Engineer, do not extend to the plane of the nearest reinforcement will not be cause for rejection unless they are numerous and extensive. Diagonal cracks on vertical surfaces, which indicate damage from torsion, will be subject to a structural review prior to acceptance. Vertical and horizontal cracks, which are one-sixteenth inch or less in width and which tend to close upon release of stress, are acceptable. Cracks in excess of this are subject to review prior to acceptance.

Cracks which extend into the plane of the reinforcing steel and/or prestressed tendons, but are acceptable otherwise, shall be repaired by sealing with a latex-base, adhesive grout or with epoxy.

Small areas of honeycomb which are purely surface in nature may be repaired. Honeycomb extending to the plane of the prestressed strands will be tentatively rejected, but will be subject to structural review. Before repairing honeycombed areas, all loose material shall be removed. Surfaces which are repaired shall be given a first rubbing which shall extend over a sufficient area to blend into the surrounding unfinished surface. This will not be construed to require the rubbing of large adjacent areas to gain uniformity of color and

texture. No extra compensation will be allowed for the extra work or materials involved in repairing or replacing defective concrete.

**425.7. "Workmanship and Tolerance".**

**A. Prestressed Beams, Girders and Box-Type Beams.**

1. Variation from shop plan lengths: Plus or minus one inch.

2. Variation from plan height:

a. Box-Type Beams, plus or minus one-fourth inch.

b. Others, plus or minus one-half inch.

3. Maximum sweep (upon release of stress):

a. Box-Type Beams, three-fourths inch total sweep.

b. Others, one-fourth inch per 10 ft of length.

Members which later develop more than the above sweep will be accepted if they can be delivered, erected, aligned and held until after diaframes are placed, without visible damage to the beam.

4. Out of square (vertical or horizontal) or deviation from plan skew angle:

One-eighth inch per foot of dimension.

5. Bearings:

Out of perpendicular with vertical axis measured on beam end at centerline:

One-sixteenth inch maximum.

6. Form Fit-up:

Where sections of forms are to be butt-jointed, an offset of 1/16 inch for flat surfaces and 1/8 inch for corners and bends will be permitted. Offsets between adjacent end header sections shall not exceed 1/4 inch.

B. Piling. Tolerances for piling shall be as specified in Item 409, "Concrete Piling".

C. Reinforcing steel shall not project above the top of the member more than 1/2 inch or less than 3/4 inch from plan dimension. In the plane of the steel parallel to the nearest surface of concrete, bars shall not vary from plan placement by 1/4 inch, or 1/12 of the spacing between bars, whichever is greater. In the plane of the steel perpendicular to the nearest surface of concrete, bars shall not vary from plan placement by more than 1/4 inch.

Variations greater than specified above will be subject to structural review.

**425.8. "Measurement".** Precast, prestressed concrete beams or girders of the type specified, cast and stressed as required by the plans, will be measured by the linear foot, as established on approved shop drawings. Other precast, prestressed concrete members of the size and type specified, cast and stressed as

required by the plans, will be measured by the linear foot, by each, or by the square foot, as the case may be, as established on approved shop drawings, or as shown on the plans.

Precast, prestressed concrete spans of the size and type specified, cast and stressed as required by the plans, will be measured as each prestressed span complete in place.

Cast-in-place structures (or structures where the Contractor has the option of casting-in-place) will be measured in accordance with the provisions of Item 426, "Prestressing".

425.9. Payment. Precast, prestressed concrete beams or girders will be paid for at the unit price bid per linear foot for "Prestressed Concrete Beams" of the type specified.

Precast, prestressed concrete spans will be paid for at the unit price bid for each "Prestressed Concrete Span".

Other prestressed concrete members will be paid for at the unit price bid per linear foot for "Prestressed Concrete Member" (specify name and type), or at the unit price bid for each "Prestressed Concrete Member" (specify name and type), or at the unit price bid per square foot for "Prestressed Concrete Member" (specify name and type), as the case may be.

A partial allowance will be made for materials and for precast or prestressed concrete members (cast, but not erected) in accordance with the provisions of Item 9.6, Partial Payment.

The above prices shall be full compensation for constructing the members, furnishing and tensioning prestressed steel, conduit, when required, furnishing and placing reinforcing steel, bearing plates, bearing pads, all bars, anchorage plates and appurtenances which become an integral part of the structure; for grouting of holes; for any necessary repair and for any special treatment of end anchorages and shoes as indicated on plans; and, for furnishing all materials, tools, equipment, labor and incidentals necessary to fabricate, transport and erect the members in the structure as indicated on the plans.

## TEXAS HIGHWAY DEPARTMENT

## SPECIAL PROVISION

TO

ITEM 426

## PRESTRESSING

For this project, Item 426 "Prestressing" is hereby amended with respect to the clauses cited below and no other clauses or requirements of this item are waived or changed hereby:

Article 426.2 through 426.5 are hereby voided and replaced by the following:

426.2. Materials and Equipment.

(1) Concrete. All concrete shall conform to the provisions of Item 421, "Concrete for Structures" and/or Item 423, "Lightweight Concrete for Structures". Cast-in-place concrete shall further conform to Item 420, "Concrete Structures".

The class of concrete shall be as designated on the plans.

(2) Grout. The recommended composition of grout for post-tensioning tendons shall be:

94 Lbs	Portland Cement - Type II
1 Lb.	Admixture - Interplast C as manufactured by
	Sika Chemical Company or Intrusion Aid as
	manufactured by Concrete Chemicals Company.
5 $\frac{1}{2}$ Gals. Max.	Water
50 Lbs. Max. of Fly ash with a maximum carbon content of 8% may be	
	added to the above mixture if desired.

Other mixtures of equal or better strength, workability and freedom from corrosive elements may be approved by the Engineer.

(3) Reinforcing Steel. Reinforcing steel, not prestressed, shall conform to Item 440, "Reinforcing Steel".

(4) Structural Steel. Structural steel bearing plates, fittings, etc., shall conform to Item 441, "Steel Structures", and Item 442, "Metal for Structures".

## (5) Prestressing Steel.

(a) Post-Tension Method. Alternate types of prestressing tendons and systems may be used as provided on the plans. End anchorages, tendon couplers or connections shall develop at least 95 percent of the required ultimate strength of the tendon based on the gross area and the required minimum unit strength. Certification of the above by the manufacturer will be required. Friction type anchorage will not be permitted when coated tendons are used.

Anchorage which depend upon both friction and mechanical interlocking such as serrated wedges and threaded nuts will be permitted for coated tendons subject to the following tests to be performed at the manufacturer's expense by a commercial laboratory approved by the Engineer:

Static Test. Anchorage shall develop at least 95 per cent of the required breaking strength of the tendon with a minimum elongation of 3 percent.

Dynamic Test. Anchorage shall withstand 500,000 cycles from 60 to 70 percent of the required breaking strength of the tendon without failure or slippage. Tendon and anchorage shall be lubricated for this test.

Bearing and shim plates shall be structural steel in accordance with the provisions of Item 441, "Steel Structures", and Item 442, "Metal for Structures".

As used herein, "tendon" shall be defined as any single prestressing element used to apply prestressed forces to the member. For post-tensioned construction this shall be each group of wires, each group of seven wire strands, each large diameter strand, or each bar having common end anchorage.

All tendons shall be identified by heat number, or reel in the case of seven wire strand, and tagged for identification purposes. Anchorage assemblies shall be identified in a like manner. The Contractor shall furnish, free of additional charge, one representative specimen of each size of tendon from each 10 tons of each heat. For seven wire strand one specimen shall be furnished from each reel up to a lot of three reels or from each third reel for lots of four or more reels. These specimens are to be obtained at the casting yard or at the tendon fabrication plant. Each specimen shall be 4 feet in length.

When required by the Engineer, the Contractor shall furnish, free of charge, two specimen of each size of prestressing unit of the selected type, with end fittings attached, for tensile tests to be performed by the Engineer. These specimen shall be 5 feet in clear length, measured between ends of fittings. If the results of the test indicate the necessity of check tests, additional specimen shall be furnished without additional cost.

For prestressing systems previously tested and approved on Department projects complete tendon samples need not be furnished, provided there is no change whatsoever in the material, design, or details previously approved. Shop drawings shall contain an identification of the project on which approval was obtained, otherwise sampling will be necessary.

When testing as a complete unit is not required, the tendon shall be tested and specimen submitted as specified above.

For prefabricated tendons, the Contractor shall give the Engineer at least 10 days notice before commencing the installation of end fittings or the heading of wires. The Engineer will inspect all end fittings, installations, and wire headings while such fabrication is in progress at the plant and will arrange for all required testing of the material to be shipped to the site.

Tendons shall be grouted or coated as shown on the plans. Tendons and conduit for grouted tendons and all anchorage devices shall be free of

lubricant, oil, loose rust and other deleterious material at the time of placing in the member. The Engineer may prohibit the use of components which show excessive signs of oxidation or weathering.

1. Steel Wire. Parallel steel wire shall conform to ASTM Designation: A 421. Type BA shall be used for "Button" anchorages and Type WA shall be used for "Wedge" anchorages. Samples for testing shall be as specified in Section 426.2 (5)(a).

2. Manufactured Steel Twisted Wire Strand (Large Diameter Strand). Unless otherwise specified on the plans, strand shall not be galvanized. The minimum ultimate strength shall be 220,000 pounds per square inch based on the gross section of the tendon.

The minimum ultimate elongation over a gage length of 10 inches shall be 3 percent. After tendons are fabricated, such shall be prestretched at sufficient stress and for sufficient length of time to equalize the stress in all wires and to reduce creep to a minimum.

3. Prestressing Bars. The minimum ultimate tensile strength based on the gross section of the bar shall be 145,000 pounds per square inch. The minimum ultimate elongation in 20 bar diameters shall be 4 percent. Each bar shall be proof loaded by the manufacturer to 90 percent of its required ultimate strength. If splices are allowed, test reports showing complete chemical analysis of the bar and of the splicing device shall be submitted for approval prior to their use. If splices are used, tensile specimen submitted shall develop 95 percent of the required ultimate when tested with a 2 degree cold bend in the bar at the root of the thread or at the end of the splicing device.

4. Manufactured Steel Seven Wire Strand. Seven wire strand used for post-tensioned tendons shall conform to the requirements of Article 426.2 (5)(b)2.

Wires, strands or bars with greater ultimate strength but otherwise produced and tested in accordance with A.S.T.M. A421, A.S.T.M. 416 and the requirements of this specification will be permitted provided the physical properties as outlined in the applicable specification are shown on the shop drawings.

(b) For Pretension Method. As used herein, "tendon" shall be defined as "any single prestressing element used to apply prestressed forces to the concrete". For pretensioned construction, this shall be each strand of straight wire.

1. Steel Wire. The requirements for steel wire shall be the same as the requirements for steel wire for the post-tension method as specified above. Samples for testing shall be as specified in Section 426.2.(5)(a).

2. Manufactured Steel Seven Wire Strand. Seven wire strand shall conform to the requirements of ASTM Designation: A 416, and as outlined for higher strength strand in Section 426.2.(5)(a)4, above.

The Contractor shall furnish, free of charge, a representative sample of each size of wire or strand from each reel up to a lot of three reels. In lots of four reels or more, one sample shall be furnished from each third reel or fraction thereof. Each sample shall be 4 feet in length.

3. Prestressing Bars. Bars will not be allowed in the pretension method.

(6) Bearing Pads. Bearing pads shall conform to the requirements of Item 435, "Elastomeric Materials".

(7) Conduit. The conduit to enclose grouted, post-tensioned tendons shall be mortar tight, and may be either rigid, galvanized, ferrous metal, with a smooth inner wall, capable of being curved to the proper configuration, or may be flexible, interlocking, galvanized ferrous metal. Couplers for either type shall also provide a mortar type connection. Rigid conduit may be fabricated with either welded or interlocking seams. Galvanizing of welded seams for the rigid conduit or for conduit couplings will not be required. Either type of conduit shall be capable of withstanding all forces, due to construction operations, without damage.

For all conduit, the inside area shall be at least two times, but not more than two and one-half times the area of the enclosed prestressing steel. It shall be adaptable to enlarging or flaring near the ends as required to accommodate the necessary anchoring system. The conduit shall be equipped with fittings for injection of grout and ports for venting.

(8) Equipment. When required to post-tension members or units, the Contractor shall furnish a compression testing machine, meeting the requirements of Section 498.4(1) of Item 498. In lieu of the above, and with written permission of the Engineer, the Contractor may provide the facilities of an acceptable Commercial Laboratory for the testing of cylinders for "Tensioning Strength", "Partial Tensioning Strength" or "Handling Strength".

#### 426.3. "Construction Methods".

(1) General. The following paragraphs are primarily intended to apply to the prestressing of concrete structures. However, the methods of tensioning and other requirements where applicable shall apply to the prestressing of any type of structure or member, subject to special requirements on the plans.

Calculations of anticipated losses, spacings, clearances, etc., shall be in accordance with the latest AASHO Tentative or Standard Specification for Highway Bridges, unless otherwise shown on the plans.

Prior to stressing, the Contractor shall furnish the Engineer certified copies of load calibration curves on all jacks and gauge systems to be used in the work. Stressing systems shall be recalibrated when required by the Engineer.

(2) Post-Tensioning. When grouted tendons are to be deflected, the metal conduit shall be set low enough to offset the eccentric position of the tendons or bars in the conduit after stressing. Unless otherwise specified, the location of the prestressing units with respect to the member being prestressed as shown on the plans shall be construed to be the final location after stressing. The Prestressing Details to be furnished by the Contractor shall show the offsets from the bottom of slab or beam to the bottom of conduit taking into account the position of the prestressing steel within the conduit. The conduit shall be supported at intervals of not more than 5 feet unless otherwise shown in the plans and shall be securely fastened to prevent displacement during placement of concrete. The Contractor shall submit for approval details of his proposed method of supporting the conduit. Unless otherwise shown on the plans the allowable tolerance for vertical positioning of the tendon will be  $\pm 1/4"$ .

"Grouted tendons shall be equipped with fittings for injection of grout and ports to vent entrapped air. Grout ports will be required at the far end of the tendon and at the high points of the tendon profile when there is more than 6" variation in the vertical position of the conduit. Tendons with less than 6" vertical variation shall have grout ports at the far end and at intervals between the far end and injection end not to exceed 100'. Grout ports shall consist of 1/2 inch minimum diameter galvanized metal pipe with caps or other arrangements having a minimum port diameter of 1/2 inch as approved by the Engineer. The tops of the grout ports shall be set approximately 1/2 inch below the finished surface of the concrete. Recesses caused by the grout ports in the concrete surface shall be filled with mortar and finished as directed upon completion of grouting.

If end anchorages permit, tendons may be inserted in the member after placing of the concrete. In all cases, the metal conduit previously described shall be used. If tendons are to be inserted later, a stiffening bar may be required inside the conduit during placement of concrete. Positive means of holding the conduit in its correct position shall be provided in all cases and shall be indicated on the working drawings submitted to the Engineer for approval.

After the concrete has reached a compressive strength equal to the "Partial Tensioning Strength" specified on the plans, as indicated by compressive cylinder tests, the members may be partially prestressed for the purpose of removal of members from the casting bed to a curing area, provided satisfactory provision is made to bring all stressing units (including units stressed for handling) up to full required stress at the time of final stressing. The units to be stressed and the amount and pattern of the stressing for the partial prestressing shall be submitted with the prestressing details and approved by the Engineer.

The Contractor shall furnish hydraulic jacks for stressing the steel, of a type suitable to the purpose and equipped with gauges graduated to read directly to one percent of the total load applied, and calibrated to measure accurately the stress induced in the steel. For post-tensioning, the jacks shall have a stroke of adequate length so that the stressing, including temporary overstress, can be done in one movement. They shall be equipped with proper ports or windows for adequate visual examination and measurement of tendon movement. They shall also be capable of slow release of stress to allow relaxation from overstress to the proper seating force.

In all methods of tensioning, the stress induced in the prestressed steel shall be measured by elongation of the steel and checked by gauge pressure. The results shall be within 5 percent of each other. Suitable means shall be provided for measuring the elongation of the steel to the nearest one-sixteenth of an inch. In the event of apparent discrepancies of more than 5 percent between stresses indicated by elongation and gauge pressure, the entire operation shall be checked carefully and the source of error determined and corrected before proceeding further. When both ends of tendons are being pulled simultaneously the final elongation at either end shall be within 5 percent of the calculated elongation for that particular end. If one or both ends differ more than the 5 percent mentioned in the preceding, the tensioning operation shall be partially or totally repeated until the required tolerances are obtained.

The temporary overload force, when required and the post-tensioning force as shown on the plans may be applied when the concrete has reached a compressive strength equal to the "Tensioning Strength". These forces may be applied to one or both ends of the tendon as necessary to reduce friction. The temporary overload force shall be held for a period of 2 minutes before reducing to the post-tensioning force required. The sequence of post-tensioning the units shall be such as to prevent overstressing the member in vertical or lateral bending at anytime, and shall be so indicated on the prestressing details submitted to the Engineer for approval.

A test for "Partial Tensioning Strength" and "Tensioning Strength" shall be defined as the average breaking strength of two cylinders.

The following is the general tensioning procedure. The Prestressing Details to be submitted by the Contractor shall reflect the same general tensioning procedure modified for each particular installation:

- a. The tendons will be tensioned in the sequence designated in the Prestressing Details.
- b. Initial tensioning to take the slack out of the tendons will be at ten percent of the maximum tensioning load unless otherwise shown on the approved Prestressing Details.
- c. After the initial tensioning, the tendons shall be reference marked to determine elongation.
- d. After tensioning to overcome friction the tension should be reduced to that required to set the anchorage. With friction type anchorages, it will be necessary to check the required elongation after the tendons have been released in order to ensure that the required elongation is obtained after the anchors have set. If the resulting elongation after anchorage set is outside the tolerances required, the entire tensioning operation shall be repeated until the necessary requirements are met.
- e. After stressing and anchoring all tendons and upon the Engineer's approval, projecting tendons shall be trimmed as shown in the approved Prestressing Details.
- f. For grouted tendons the conduits shall be grouted within 48 hours after the completion of the tensioning operation unless delayed by permission of the Engineer. Grout must be pumped toward an open vent. Grout shall be pumped continuously under moderate pressure at one end of the conduit until all entrapped air is forced out the open vents downstream from the grout pump. The open vents shall be closed as soon as grout issues in a steady stream. After all grout ports have been closed, the pressure shall be increased to a minimum of 75 psi and held at this pressure for approximately 15 seconds. The grouting entrance port is then closed.

Unless otherwise shown on the plans, lightweight concrete prestressed units shall be stressed with a temporary force equal to 125 percent of the final prestress force and maintained for a period of 45 days. The temporary force shall be applied when the concrete has aged a minimum of 10 days, and when the test cylinders indicate the "Tensioning Strength" has been attained. At the end of the period specified, all the individual post-tensioning units shall be restressed to correct final prestress force.

- (3) Pretensioning. All tendons to be prestressed in a group shall be brought to a uniform initial tension of 1000 pounds (plus or minus 50 pounds) per tendon prior to being given their full pretensioning. Initial tension greater than that specified herein may be used when designated and approved on the shop drawings. The uniform tension shall be measured by some suitable means such as a dynamometer so that its amount can be used as a check against elongation computed and measured.

After this initial stressing, the group shall be stressed to a total tension as required by the plans by means of hydraulic jacks equipped with gauges

graduated to read directly to 1 percent of the total load to be applied, and calibrated to measure accurately the stress induced in the steel. The induced stress shall be measured by elongation of the tendons and checked by gauge pressure. The results shall be within 5 percent of each other.

Means shall be provided for measuring the elongation to an accuracy of one-sixteenth of an inch. In the event of apparent discrepancies of more than 5 percent between stresses indicated by gauge pressure and elongation, the entire operation shall be checked carefully and the source of error determined and corrected before proceeding further.

Where a combination of straight and deflected tendons is used, the stress indicated by total elongation shall not vary by more than 5 percent from that indicated by gauge pressure. Measurements on individual deflected tendons to establish differential stresses at different points in the beam shall be averaged at a cross section of the beam and the averages shall be within 5 percent of the computed elongation. No individual tendon shall vary from the computed elongation by more than 10 percent at any measured cross section.

Independent references shall be established adjacent to each anchorage to indicate any yielding or slippage that may occur between the time of initial stressing and final release of the tendons.

With the tendons stressed to full tension as prescribed above and all other reinforcing in place, the concrete shall be cast to lengths necessary to provide the lengths required by the plans, after shrinkage and elastic shortening has occurred.

After strength requirements are attained, the tension in the tendons shall be gradually and simultaneously released and the tendons cut off as required, using a sequence to minimize shock and reduce premature tendon breakage.

When elevated temperature curing is used, the release of stress shall be prior to the beginning of temperature reduction. Members shall remain protected until there is a differential of temperature inside the curing jacket and air temperature of not more than 25 F.

At the ends of members, the tendon ends and a minimum area of one inch around each tendon shall be coated with approximately a 10 Mil coating of a commercial grade of epoxy.

When draped tendons are used, positive external hold downs may be required to offset the vertical forces in the beam at the time of stress release.

(4) Combined Pretensioning and Post-Tensioning. Where the plans call for a combination of pretensioning and post-tensioning, all of the requirements of both the pretensioning and post-tensioning shall apply, in this order, and the requirements shall overlap as necessary to fulfill the intent of this specification.

426.4. "Measurement".

(1) All precast concrete members (except piling) either pretensioned, post-tensioned, or combined pre- and post-tensioned, will be measured and paid for as specified in Item 425, "Prestressed Concrete Structures". Prestressed piling will be measured and paid for as specified in Item 409, "Concrete Piling", or Item 412, "Prestressed Concrete Piling".

(2) Cast-in-place prestressed concrete structures and units will be measured as follows:

(a) Concrete, non-prestressed reinforcing steel, and structural steel (except bearing and anchorage devices integrally a part of the post-tensioning system) will be measured by the cubic yard or by the pound in accordance with the applicable specifications for these items. Grout and ducts for post-tensioning will not be measured but will be considered subsidiary to this item.

(b) The prestressing steel required and the work involved in prestressing of cast-in-place structures will be measured by the product of the minimum design prestress force and the horizontal length over which the prestressing is applied, expressed in thousands of kip-feet (MKF). For slab units this will be calculated as the product of the minimum design prestress force per foot and the area out-to-out of the slab unit. Unless otherwise shown on the plans the length of prestressing or the area in the case of a slab will be taken as the overall dimensions of the unit, no deduction being made for the clearance distance between the ends of tendons and the edges of the unit.

426.5. "Payment". Payment for the work and all materials for prestressing cast-in-place units, measured as specified above will be made at the unit price bid per thousand kip feet of Prestressing (Grouted) or Prestressing (Coated).

The amount to be paid for will be that quantity shown on the contract plans except as modified by the following:

Either party may request an adjustment of quantities shown on the contract plans (by each separate bid item), if the quantities, calculated as outlined in Article 426.4, "Measurement", vary from those shown on the contract plans by more than the following:

- (a). Over 5000 MKF ----- one half of 1 percent
- (b). 1000 MKF through 5000 MKF ----- 1 percent
- (c). Less than 1000 MKF ----- 1½ percent

The party to the contract which requests an adjustment shall present to the other, two copies of the calculations showing the revised quantities for the portion or portions of the structure in question. These revised quantities when proven correct, together with all other quantities under the same bid item, shall constitute the final quantity for which payment will be made.

The above payment shall be full compensation for furnishing all prestressing steel, all materials, fabrication, transportation, erection, prestressing, and for furnishing all metal encasing ducts, grout fittings, and anchorages, bearing plates, and all tools, labor, equipment and incidentals necessary to complete the work.

## TEXAS HIGHWAY DEPARTMENT

## SPECIAL SPECIFICATION

## ITEM 2131

## EPOXY BONDING AGENT

1. DESCRIPTION. This item shall govern for the furnishing and application of epoxy material for use in the joint between precast concrete units, as required by the plans.

2. MATERIALS. The epoxy material shall be of two components, a resin and a hardener (1 to 1 ratio), meeting the following requirements:

- a. Pot Life Min. 90 minutes at 68 F (ASTM D1338)
- b. Compressive Strength 6,000 p.s.i. min.
- c. Tensile Strength (Direct or Bending) 2,000 p.s.i. min.
- d. Specific Gravity 70 to 120 lbs./cu.ft.
- e. Viscosity at 68 F 10,000 to 50,000 cps
- f. Coefficient of Thermal Expansion Within 10% of that for concrete

The material shall have a rate of absorption, rate of shrinkage, chemical resistance and weather resistance compatible with concrete and a consistency such that it will not flow appreciably when applied to a vertical concrete surface. The color shall be concrete gray.

The Contractor shall furnish the Engineer a sample of the material for testing, and a certification from a reputable laboratory indicating that the material complies with the above requirements.

The sample of the material submitted will be tested additionally for the following:

- a. Ability to join test specimen under the following conditions:

Temperature Range	50 F to 100 F
Surface Conditions	Dry to Moist
(Moist is defined as 'one hour drying after complete saturation'.)	

b. The joint material shall be able to develop 95 percent of the flexural tensile strength and 70 percent of the shear strength of a monolithic test specimen.

The test specimen shall be made of concrete having a minimum compressive strength of 6,000 p.s.i. The specimen will be tested with both dry and moist surface conditions.

3. CONSTRUCTION METHODS. Surfaces to which the epoxy material is to be applied shall be free from all oil, laitance or any other material that would prevent the material from bonding to the concrete surface. All laitance shall be removed by sanding or by washing and wire brushing.

Mixing of the resin and hardener components shall be in accordance with the manufacturer's instruction. Use of a proper sized mechanical mixer will be required.

The epoxy material shall be applied to all surfaces to be joined within the first half of the pot life as shown on the containers.

The coating shall be smooth and uniform and shall cover the entire surfaces to be joined with a maximum thickness of 1/16 inch. The units shall be joined within 45 minutes after application of the epoxy material.

No jointing operations shall be performed when the ambient temperature is below 50 F or above 100 F. When the temperature is above 85 F the epoxy coated surfaces shall be shaded from direct sunlight.

If the jointing is not completed within 45 minutes after application of the epoxy material the operation shall be stopped and the epoxy material shall be completely removed from the surfaces. Fresh material shall be applied to the surfaces before resuming jointing operations.

4. MEASUREMENT AND PAYMENT. No direct measurement or payment will be made for the materials, work to be done or equipment to be furnished under this item, but it shall be considered subsidiary to the particular items required by the plans and the contract.

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TEXAS HIGHWAY DEPARTMENT SPECIAL SPECIFICATIONS  
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## TEXAS HIGHWAY DEPARTMENT

## SPECIAL SPECIFICATION

## ITEM 4122

## PRECAST CONCRETE SEGMENTAL STRUCTURES

4122.1. Description. This item shall govern for the furnishing, in place, of precast concrete segmental structures properly stressed in accordance with the plans, approved working drawings and with these specifications.

4122.2. General. The Contractor has the option of furnishing any type of post-tensioning system meeting the requirements of the plans and specifications for temporary and permanent stressing that has been used successfully in a similar configuration. The system selected must be capable of providing the magnitude and distribution of prestressing force and ultimate strength required by the plans without exceeding allowable temporary stresses. Unless otherwise shown on the plans all design procedures, coefficients and allowable stresses as well as tendon spacing and clearances shall be in accordance with the latest Standard AASHTO Specifications for Highway Bridges.

Prior to casting any permanent segments, the stressing system shall be successfully demonstrated on a segment designated on the plans. The segment shall conform to the size and configuration required by the plans, including post-tensioning anchorage pockets, reinforcing steel, concrete and conduits (curvature and spacing). The tendons designated on the plans for this test shall be stressed to the forces shown. No additional payment will be made for this test, however it is the intent that the segment be used in the structure unless damaged by this test.

Prior to the beginning of fabrication, the Contractor shall give the Engineer notice as to the location of the casting site, the date on which the work will begin, shall submit to the Engineer a proposed sequence of erection by quadrant, section, span or unit number, and a casting schedule.

An Inspection Laboratory shall be furnished in accordance with the Item, "Plant Inspection Laboratory (Equipped)".

The design of casting beds for the segments, including plans and specifications, shall be prepared by a Professional Engineer registered in the State of Texas, and shall bear his seal. The Contractor (Fabricator) shall furnish a certificate bearing his signature, or that of a responsible Officer of the Company, that the bed and hardware have been constructed in accordance with the above plans and specifications.

Casting bed and forms shall be structurally adequate to support the segments without settlement or distortion. The casting bed shall be designed for a method and the hardware needed to adjust and maintain grade and alignment. Details for hardware and adjustment procedure shall be included in the above plans and specifications for the casting bed.

Grading of the soffit form and the top portion of each segment shall take into consideration the relative position of the member in the structure.

A unit is defined as a central segment and all segments that cantilever from it. After the first segment of each unit is cast, all succeeding segments shall be cast against previously cast segments to insure complete bearing and proper alignment on all mating surfaces.

**4122.3. Materials.** Materials required for use under this Item shall conform to the following:

<b>Concrete</b>	Item 421 and/or 423
<b>Reinforcing Steel (non-prestressed)</b>	Item 440
<b>Structural Steel</b>	Items 441, 442
<b>Elastomeric Materials</b>	Item 434 and/or 435
<b>Expansion Joints</b>	Item 420 and/or 436
<b>Prestressing Hardware</b>	Proprietary
<b>Epoxy Bonding Agent</b>	Item 4123

Concrete shall be of the class and strength shown on the plans. Design slump shall be 4 inches with a maximum of 5 inches. The maximum water/cement ratio shall be 5.0. Type I cement will be permitted.

Reinforcing steel shall be of the grade shown on the plans or as approved on the fabrication drawings.

Bearings, anchor bolts and anchor rods shall be the type shown on the plans or as approved on the fabrication drawings.

Prestressing steel shall conform to one of the following types:

Steel wire conforming to ASTM Designation: A421.

Seven wire strand conforming to ASTM Designation: A416

Tendon is defined as "any single unit used to apply prestressing forces to the member". For post-tensioned units, this shall be each group of wires or each group of strands having common end anchorage. Post-tensioned tendons shall be grouted or coated, as required by the plans.

When temporary external tendons are required by the plans, the tendons and anchors shall be in a protective enclosure capable of protecting the tendons from damage by erection equipment and containing a strand or tendon that breaks or otherwise releases tension rapidly during or after tensioning and anchorage.

Protective enclosure proposals shall be submitted to the Engineer for approval.

Tendon couplers shall be used only at locations specifically shown on the plans or approved by the Engineer.

End anchorage and tendon couplers for grouted tendons shall develop a minimum of 95 percent of the required ultimate strength of the tendon with a minimum elongation of 2 percent when tested in the unbonded condition. End anchorages and tendon couplers for coated ungrouted tendons shall develop at least 100 percent of the required ultimate strength of the tendon, with a minimum elongation of 2 percent, and in addition shall withstand 500,000 cycles from 60 to 70 percent of the required ultimate strength of the tendon without failure or slippage. Testing of complete tendons for compliance with the requirements of this paragraph shall be at the Contractor's expense and the results certified in writing to the Engineer by an acceptable laboratory.

The Contractor shall further furnish for testing, one specimen of each size of prestressing tendon, including couplings, of the selected type, with end fittings attached, for ultimate strength tests only. These specimen shall be 5 feet in clear length, measured between ends of fittings. If the results of the test indicate the necessity of check tests, additional specimen shall be furnished without cost. For prestressing systems previously tested and approved on Department projects having the same tendon

configuration, complete tendon samples need not be furnished, provided there is no change in the material, design, or details previously approved. Shop drawings or prestressing details shall identify the project on which approval was obtained, otherwise sampling will be necessary.

Material for coating non-bonded (ungrouted) tendons shall be a nonvolatile, low frictional mineral oil base grease, with a rust preventing additive having a relatively uniform viscosity under temperature range of 20F to 120F. The tendons shall be sheathed with a waterproof material capable of maintaining the tendon tightly bundled and containing the lubricant.

The conduit to enclose grouted, post-tensioned tendons shall be mortar tight, and of galvanized, ferrous metal, and may be either rigid with a smooth inner wall, capable of being curved to the proper configuration, or may be a flexible, interlocking type. Couplers for either type shall also provide a mortar tight connection. Rigid conduit may be fabricated with either welded or interlocking seams. Galvanizing of welded seams for the rigid conduit or for conduit couplers will not be required. During placing and finishing of concrete in a segment, for either type of conduit, inflatable hoses capable of exerting sufficient pressure on the inside walls shall be placed internally in all conduits and shall extend a minimum of two feet into the conduit in the previously cast segment. Either type of conduit shall be capable of withstanding all forces, due to construction operations without damage. Other types of conduit and/or internal protection systems will be considered subject to the approval of the Engineer.

For all conduit, the inside area shall be at least two times the area of the enclosed prestressing steel, unless the conduit size is shown on the plans. It shall be adaptable to enlarging or flaring near the ends as required to accommodate the necessary anchoring system. The conduit shall be equipped with fittings for injection of grout and ports for venting.

Grout to be used in tendons shall be proportioned to obtain acceptable strength and expansion requirements as proven by acceptable tests. Approximate proportions will be as follows:

94 Lbs.	Cement (Type I)
1/2 Lb.	Expanding Admixture
4 1/2 Gal. Max.	Water

50 Lbs. maximum of fly ash with a maximum carbon content

of 8 percent may be added to the above mixture if desired. Other pozzolans will not be permitted. Other mixtures of equal or better strength, workability and freedom from corrosive elements may be used with approval of the Engineer.

All tendons shall be identified by heat number, or reel number in the case of seven wire strand, and tagged for identification. Anchorage assemblies shall be identified in a like manner. The Contractor shall furnish specimen for test purposes in accordance with Test Method Tex-710-I.

For prefabricated tendons, the Contractor shall notify the Materials and Tests Engineer at least 10 days prior to the installation of end fittings or the heading of wires in order that sampling and testing may be arranged.

All prestressing steel shall be protected against physical damage and corrosion from the time of manufacture until grouting and/or encasing in concrete.

Rust on prestressing steel which can be removed by light rubbing is acceptable. Streaks or spots, which may remain after rust removal, are acceptable if no pitting is present. Tight mill scale is acceptable but loose mill scale shall be removed.

Prefabricated post-tensioning elements shall be protected from moisture by taping or wrapping the ends and all openings in the conduit or by other acceptable means.

The Contractor shall provide equipment to be used for uniform separation of match cast segments without damage. The method as well as details of the equipment to be used for separating match cast segments shall be included in the shop plans.

The Contractor shall furnish suitable hydraulic jacks for stressing the tendons. These jacks shall be equipped with gages graduated to read directly to one percent of the total load applied, and calibrated to measure accurately the stress induced in the tendons.

For post-tensioning, the jacks shall have a stroke of adequate length so that the stressing, including temporary overstress, can be done in one movement. They shall be equipped with proper ports or windows for adequate visual examination and measurement of tendon movement. They shall also be capable of slow release of stress to allow relaxation from overstress to the proper seating force.

The Contractor shall furnish a grout pump of sufficient capacity to properly place grout in the quantities and pressures required.

The end anchorage system must permit tendons to be inserted in the member after erection of segments.

The sequence of post-tensioning shall prevent overstressing the member in vertical or lateral bending at anytime, and shall be indicated on the prestressing details. Balanced and symmetrical post-tensioning shall be required during the entire stressing operation.

The bond breaking material shall be used to break the bond of concrete between the face of previously cast segments and a newly cast segment, as well as the end headers when required. The bond breaker shall consist of flax soap and talc, or other material approved by the Engineer. A demonstration shall be performed on large specimen, prior to the casting of segments, to prove the adequacy of the material. The material shall not be injurious to the concrete and shall permit removal of a segment without pull-outs caused by adhesion of the concrete.

The epoxy bonding material shall be used on each matching face of the segments for the purpose of bonding them into their final position in the structure and shall conform to the Item, "Epoxy Bonding Agent".

**4122.4. Working Drawings and/or Shop Plans.** Working drawings and/or shop plans shall be submitted for approval to the Bridge Engineer, Texas Highway Department, 11th and Brazos Streets, Austin, Texas, 78701.

Shop plans shall be prepared on sheets 22x36 inches. The margin on the left shall be one and one half of an inch wide and all others one half of an inch. Each sheet shall have a title block in the lower right-hand corner. The title block shall include the sheet index data shown on the lower right-hand corner of the project plans, sheet numbering for the shop drawings, name of the structure or stream and name of the fabricator and contractor.

Shop plans for segments shall be submitted for approval in accordance with these specifications and shall consist of Forming Details, Falsework Details, Erection Plan (including sequence), Index Sheet, Bearing Sheets, Fabrication Sheets, Prestressing Details (including temporary tendons to overcome tensile forces at the bottom of segments), and Design Calculations. Design calculations may be on standard letter size sheets. Six sets of all required drawings and calculations shall be submitted.

For precast segments in addition to forming and falsework plans required by the Item, "Concrete Structures", the Contractor shall submit prestressing details containing all necessary information for construction.

Sufficient design calculations to support the system and method of stressing, the required jacking force for each tendon and type of conduit furnished, the seating losses for the system and the supplementary reinforcement to resist tendon bursting, splitting and spalling, will be required.

After design approval, drawings showing details of type, size and number of units per tendon, forces applied per tendon, end anchorage systems, tendon profile, grouting and venting ports, marking used to identify unlike tendons and their location, total elongation and 90 percent elongation (known as measurable elongation), temporary overstress, seating losses and other information necessary to properly complete the work, will be required.

A layout showing the pattern to be followed in stressing tendons to properly stress the unit, and a step-by-step stressing sequence shall be submitted. Provisions shall be included for each operation, including tendon slack removal, overstressing, seating of anchorages, elongation measurement, shim insertion, wedge seating and seating losses.

Fabrication details showing each different type of segment will be required showing location and placement of all reinforcing steel as well as the conduits, grout ports, etc. These shall be submitted on one sheet so that fabrication and inspection can be properly coordinated. Conduit sizes and locations shall also be submitted with the post-tensioning details. Details of the method of support for, and location of the conduit, with dimensions provided to properly locate the conduit so that the center of gravity of the enclosed tendon will be at its proper location, will be shown. The details shall show the offsets from the bottom of conduit taking into account the position of the prestressing tendon within the conduit.

4122.5. Fabrication. Reinforcing steel shall be fabricated and placed in accordance with the plans and as required herein. No reinforcing steel shall be cut and removed to permit proper alignment of stressing conduits. Any bar that cannot be fabricated to clear the conduits shall be replaced by additional bars with adequate lap lengths and shall be submitted to the Engineer for approval.

All segments shall be marked on the inside with a unique identification at the time of form removal. This identification shall be used to identify each segment on shop plans, post-tensioning details and calculations and any other document pertaining to the fabrication and erection of precast concrete segments.

Positive means of holding the conduit in its correct position shall be provided in all cases and shall be indicated on the working drawings submitted for approval. The conduit shall be supported at intervals of not more than 5 feet, or as shown on the plans, and shall be securely fastened to prevent movement during placement of concrete.

The construction of forms and the placing, curing and finishing of concrete shall be in accordance with the provisions contained herein and requirements of the plans.

Side forms, bottom forms and end headers for precast concrete segments shall be constructed of steel, unless otherwise noted on the plans. Inside forms may be of other materials as approved by the Engineer on the shop drawings.

Forms shall be of sufficient thickness, with adequate external bracing and stiffeners, and shall be sufficiently anchored to withstand the forces due to placement and vibration of concrete. Internal bracing and holding devices in forms will not be permitted to remain in the concrete. Joints shall be maintained reasonably mortar tight.

The grade and alignment of forms shall be checked each time they are set and shall be maintained during the casting of concrete. Slab finish grade will be checked after the concrete is in place.

Metal forms shall be reasonably free from rust, grease or other foreign materials. All forms shall be cleaned thoroughly prior to each casting operation. End headers must be maintained to provide a smooth casting surface.

Wood forms, when permitted, shall conform to requirements of Article 420.9., of the Item, "Concrete Structures". Wood forms may be used on the cast-in-place longitudinal and transverse closure strips.

The soffit for casting members shall be constructed and maintained to provide not more than one fourth of an inch variation from the theoretical plane of the bottom of the member in any 50 foot section of the bed.

The faces of all forms, other than end headers, shall be properly cleaned and treated with form oil or other bond breaking coating prior to placing concrete. The oil or other materials used for this purpose shall be of a consistency and composition to facilitate form removal. Materials which appreciably stain or react with the concrete will not be permitted. All forms shall be constructed to facilitate removal without damage to the concrete.

All concrete shall be placed during daylight hours unless the fabrication plant or site is provided with an approved lighting system.

The method of concrete placement shall avoid segregation of the aggregate or displacement of the reinforcing steel, prestressing steel or conduit. Concrete shall be deposited as near as possible in its final position in the forms. Depositing large quantities of concrete at one location in the forms and running or working it along the forms will not be permitted. The sequence of depositing concrete in the forms shall be shown on the shop plans.

Special attention shall be directed toward working the coarse aggregate back from the face of the concrete and to forcing the concrete under and around the reinforcing steel, prestressing steel and conduit.

Not more than one hour shall elapse between the placing of the successive layers. Vibration of subsequent layers of concrete shall extend into the previously placed layers as specified herein.

When casting the slab of a segment, the concrete shall be placed in one continuous horizontal layer.

Concrete shall not be placed at outdoor casting beds during inclement weather or when weather conditions may result in

rainfall or low temperature during the casting operation which might impair the quality of the finished member. In case rainfall should occur after placing operations are underway, the Contractor shall provide adequate covering to protect exposed concrete. The completion of a member being cast will be permitted, provided adequate provisions are made to prevent damage to the concrete.

When members are produced at a site which has adequate provisions for the protection of concrete placing operations and which has approved elevated temperature curing facilities, concrete may be placed under any low temperature conditions provided that the temperature of the concrete is not less than 50F nor more than 85F when placed in the forms; the framework and covering are in place and heat is provided for the concrete and forms within one hour after the concrete is placed (this shall not be construed to be one hour after the last concrete is placed, but that no concrete shall remain unprotected and unheated for longer than one hour); the air surrounding the concrete shall be kept between 50F and 85F for a minimum of 3 hours prior to beginning the temperature rise which is required for elevated temperature curing; and the temperature of the concrete shall not be less than 50F at any time after all materials are added and mixing commences.

When members are produced at a site which does not provide facilities necessary to accomplish the above provisions, concrete may be placed when the atmospheric temperature is 35F or greater. The temperature of the concrete at the time of placement shall not be less than 50F nor more than 85F. The concrete shall not be placed in contact with any material having a temperature less than 32F or any material coated with frost.

Aggregates shall be free from ice, frost and frozen lumps. When required, in order to produce the minimum temperature specified above, the aggregate and/or the water shall be heated uniformly in accordance with the following:

Water shall be heated to a temperature not to exceed 180F and/or the aggregate shall be heated to a temperature not to exceed 150F. The equipment furnished shall be capable of heating the aggregate uniformly to eliminate overheated areas in the stock pile which might cause flash set of the cement. The temperature of the mixture of the aggregate and water shall be between 50F and 85F before introduction of the cement.

Protection shall be provided to maintain the temperature of the concrete at all surfaces above 50F for the required total curing time as specified herein. Protection shall consist of providing additional covering and, if necessary, supplementing such covering with artificial heating. When weather conditions indicate the possibility of the need for such temperature protection, all necessary heating equipment and covering material shall be on hand ready for use before permission is granted by the Engineer to begin placement of concrete.

When concrete is to be placed during hot weather, the concrete shall be placed without the addition of more water to the concrete than required by the design (slump and consistency), and it shall be finished properly without adding water to the surface. Control of the initial set of the concrete and lengthening the time for finishing operations, under adverse wind, humidity and hot weather conditions, may be accomplished with the use of an approved retarder in accordance with Article 421.7. The maximum time interval between the addition of mixing water and/or cement to the batch, and the placing of concrete in the forms shall not exceed the following:

<u>AIR OR CONCRETE TEMPERATURE (WHICHEVER IS HIGHER)</u>	<u>MIXTURE TIME (ADDITION OF WATER OR CEMENT TO PLACING IN FORMS)</u>
NON-AGITATED CONCRETE	
Over 80F	15 Minutes
50F to 79F	30 Minutes
AGITATED CONCRETE	
90F or Above	45 Minutes
75F to 89F	60 Minutes
50F to 74F	90 Minutes

The use of an approved retarder in the concrete will permit the extension of each of the above temperature-time maximum by 30 minutes, except that for non-agitated concrete, the maximum time shall not exceed 30 minutes. Under conditions of extreme temperature, wind or low humidity, when the specified temperature-time maximums are excessive, the Engineer may require the use of an approved retarder, or may suspend concrete placing operations.

The values which govern for minimum concrete strengths during different phases of construction shall be as shown on approved shop drawings.

For Class H, Y and Z Concrete, the control of the concrete shall be by compressive tests of cylinders. An adequate number of cylinders will be made for each pertinent strength test required. Tests for determining "Form Removal" and/or "Handling Strength" of members, shall be in accordance with Test Method Tex-704-I.

All test cylinders representing tests for removal of forms and/or falsework and for "Release Strength" shall be cured under the same conditions, be subjected to the same curing materials and to the same weather conditions as the concrete represented.

"Design Strength" cylinders for acceptance of members shall be cured with the member which the cylinders represent until handling strength is obtained. These cylinders shall then be cured for the remainder of the test period in accordance with Test Method Tex-704-I.

All concrete shall be compacted and the mortar flushed to the surface of the forms by continuous working with approved high frequency mechanical vibrators, operating at a minimum of 7,000 impulses per minute. Use of external vibrators in conjunction with internal vibrators will be required for wall sections of segments.

At least one stand-by vibrator in working condition shall be provided for emergency use in case of malfunction.

The vibrators shall be inserted systematically into the concrete immediately after deposit, thoroughly consolidating and working the concrete around the conduits, reinforcement, and into the corners and angles of the forms until it has been reduced to a plastic mass. When the concrete is placed in more than one layer, the vibrator shall be operated so that it will penetrate the previously placed layer of concrete. The vibration shall be of sufficient duration to accomplish thorough compaction and complete embedment of the reinforcing steel and conduits for prestressed tendons, but not so excessive as to result in segregation. Vibration shall be supplemented by hand spading, if necessary, to insure the flushing of mortar to the surface of all forms.

Top surfaces of segmental shall be screeded and/or floated to bring grout to the surface and cover all aggregate. Fresh concrete shall be removed from exposed reinforcing steel.

Roadway surfaces of segments which are to be given an additional wearing course shall be screeded and given a wood float or light broom finish.

Form marks in excess of those permitted in Article 4122.8., and all fins and rough edges along chamfer lines shall be removed in an acceptable manner.

Careful attention shall be given to the proper curing of concrete. The Contractor shall inform the Engineer regarding the methods and procedures proposed for curing; shall provide the proper equipment and necessary materials; and, shall have approval of the Engineer of such methods, equipment and materials prior to placing concrete.

Inadequate curing facilities or lack of attention to the proper curing of concrete shall be cause for the Engineer to stop all construction until approved curing is provided. Inadequate curing may be cause for rejection of the member.

Weight supporting forms shall remain in place until the concrete has reached the "Handling Strength" shown on the plans. Removal of the forms shall be done in such a manner that curing is not interrupted on any portion of a segment by more than 30 minutes.

Curing shall be commenced prior to the formation of surface shrinkage cracks but in no case delayed longer than one hour after the concrete has been placed in the forms.

An approved Type I membrane curing compound may be used on the roadway surface on an interim basis prior to elevated temperature or other methods of curing, and will be required on the inside, side walls and bottom of overhang.

Concrete shall be cured continuously, except during form removal, until the concrete strength as indicated by compressive test of cylinders cured with the members, has reached the "Handling Strength" designated on the plans or shop drawings. The top roadway surface of segments shall be steam or water cured a total of 8 curing days and all surfaces shall be protected from freezing during this period.

Steam and/or water curing shall be in accordance with the Item, "Concrete Structures".

Segments shall not be moved from the casting yard until all requirements for curing and strength requirements have been attained.

4122.6. Construction Methods. The Contractor (Fabricator) shall be responsible for proper handling, lifting, storing, transporting and erection of all segments so that they may be placed in the structure without damage.

Segments shall be maintained in an upright position at all times and shall be lifted and/or moved in a manner to prevent torsion and other undue stress. Members shall be lifted or hoisted with lifting devices approved on the shop plans or by other methods approved by the Engineer in writing.

A full scale test of the lifting and temporary holding hardware shall be performed to demonstrate the adequacy of this equipment prior to beginning any erection of the segments.

Prior to stressing, the Contractor shall recalibrate all jacks and gage systems to be used in the work and shall furnish the Engineer certified copies of load calibration curves. Stressing equipment shall be recalibrated when required by the Engineer. Jacks and gages will not be interchanged without recalibration or proof loading using load cells or other methods approved by the Engineer.

For stressing, suitable means shall be provided for measuring the elongation of the steel to the nearest one sixteenth of an inch. In the event of apparent discrepancies of more than 5 percent between stresses indicated by elongation and actual stressing force, the entire operation shall be checked carefully and the source of error determined and corrected before proceeding further.

The entire tendon shall be pulled through the conduit by an acceptable method approved by the Engineer.

The temporary overload force, when required, and the post-tensioning force may be applied when the concrete in the segments has reached a compressive strength equal to the "Tensioning Strength". These forces may be applied to one or both ends of the tendon, unless otherwise shown on the plans. Similar tendons in each wall or side shall be stressed simultaneously.

The tendons will be tensioned in the sequence designated in the Prestressing Details.

Initial tensioning to take the slack out of tendons will be 10 percent of the maximum tensioning load unless otherwise shown on the approved Prestressing Details.

After the initial tensioning, the tendons shall be reference marked to determine elongation.

After tensioning to overcome friction, the tension shall be reduced to that required to set the anchorage.

Loss of elongation due to anchor set shall be checked for agreement with the anticipated value used in the stress calculations. Excessive anchor set shall be deducted from the measured elongation and corrected if necessary to maintain 5 percent agreement between elongation and stressing force.

After stressing and anchoring all tendons and upon the Engineer's approval, projecting tendons shall be trimmed as shown in the approved Prestressing Details.

Grouting of required conduits shall be within 48 hours after the completion of each tensioning operation. Grout must be pumped toward an open vent and shall be pumped continuously under pressure at one end of the conduit until all entrapped air is forced out the open vents downstream from the grout pump. The open vents shall be closed as soon as grout issues in a steady stream. After all grout ports have been closed, the pressure shall be increased to a minimum of 75 psi and held at this pressure for approximately 15 seconds. The grouting entrance port shall then be closed.

Grout ports will be required at the far end of the tendon, and at the high points of the tendon profile when there is more than a 6 inch variation in the vertical position of the conduit. Tendons with less than 6 inch vertical variation shall have grout ports at the far end and at intervals between the far end and injection end not to exceed 200 feet. Grout ports shall consist of one half of an inch minimum diameter galvanized metal pipe, with caps or other arrangements having a minimum port diameter of one half of an inch, and as approved by the Engineer. The tops of the grout ports shall be set approximately one half of an inch below the finished surface of the concrete. Erection of additional segments will not be permitted for 20 hours after any grouting operation.

Recesses caused by the end anchorage pockets and grout ports in the concrete surface shall be filled with concrete or mortar and finished as shown on the plans or as approved on shop drawings, upon completion of grouting.

Where the plans call for a combination of pretensioning and post-tensioning, all of the requirements of both the pretensioning and post-tensioning shall apply, in this order, and the requirements shall overlap as necessary to fulfill the intent of this specification.

Surfaces to which the epoxy material is to be applied shall be free from all oil, laitance, form release agent, or any other material that would prevent the material from bonding to the concrete surface. All laitance and other contaminants shall be removed by water blasting, light sandblasting, by steam cleaning, wire brushing or grinding. Surfaces contaminated with oils shall be cleaned with an approved solvent such as acetone. Wet surfaces shall be dried with a heater or gas flame before applying epoxy.

Mixing of the resin and hardener components shall be in accordance with the manufacturer's instructions. Mixing shall continue until the epoxy is thoroughly mixed and of uniform color. Use of a proper sized mechanical mixer operating at no more than 600 RPM will be required. Contents of damaged or previously opened containers shall not be used. Mixing shall not start until the segment is prepared for erection.

The epoxy bonding agent shall be applied to all surfaces to be joined within the first half of the pot life, as shown on the containers.

The coating shall be smooth and uniform and shall entirely cover both surfaces to be joined with a maximum thickness of one sixteenth of an inch. Epoxy shall be thinned out within three eighths of an inch of prestressing ducts to minimize flow into the ducts. The segments shall be joined within 45 minutes after application of the epoxy bonding agent and a compressive force shall be applied to all of the matching faces within the contact time of the epoxy bonding agent, either by the holding hardware or by temporary or permanent post-tensioning.

The joint shall be checked immediately after erection to ascertain uniform bearing and fit of the matching faces. Excess epoxy squeezed from the joint shall be removed where accessible. All tendon ducts shall be swabbed immediately after stressing while the epoxy is still plastic in order to remove or smooth out any epoxy in the conduit and to seal any pockets or air bubble holes that have formed at the joint.

No jointing operations shall be performed when the ambient temperature is below 50F or above 100F. When the temperature is above 85F the epoxy coated surfaces shall be shaded from direct sunlight.

If the jointing is not completed within 45 minutes after application of the epoxy material the operation shall be terminated and the epoxy bonding agent completely removed from the surfaces. The surfaces must be re-prepared and fresh epoxy applied to the surfaces before resuming jointing operations.

When friction must be reduced on post-tensioning tendons, water soluble oil may be used subject to the approval of the Engineer. This oil shall be applied uniformly to the tendons as they enter the conduit. The oil shall be flushed from the duct as soon as possible after stressing is completed by use of water under pressure. These ducts shall be flushed again just prior to the grouting operations.

**4122.7. Defects and Breakage.** Failure of individual wires in a seven-wire strand, or of wires in a parallel wire tendon is acceptable provided the total area of wire failure is not more than 2 percent of the total cross-sectional area of tendons in any member. Failure of an entire strand will be subject to structural review.

Fine hair cracks or checks on the surface of the member which, as determined by the Engineer, do not extend to the plane of the nearest reinforcement will be acceptable unless they are numerous and extensive. Diagonal cracks which indicate damage from torsion, longitudinal cracks that follow stressing tendons or any cracks which extend into the plane of the reinforcing steel and/or pre-stressed tendons will be subject to a structural review prior to acceptance. If found acceptable, the cracks shall be repaired by 'veeing' out one fourth of an inch deep and wide and sealing with epoxy.

Minor breakage, spalling or honeycomb (not over one inch deep) shall be repaired as directed by the Engineer. Major breakage or honeycomb in excess of that specified herein will be subject to structural review. If found to be satisfactory these areas will be repaired as directed by the Engineer. Breakage, spalling or honeycomb on any mating surface found to be acceptable, shall be repaired and the concrete cured, prior to casting the mating segment if such segment has not yet been cast.

**4122.8. Workmanship and Tolerance.** In the plane of the steel parallel to the nearest surface of concrete, bars shall not vary from plan placement by more than one fourth of an inch, or one

twelfth of the spacing between bars, whichever is greater. In the plane of the steel perpendicular to the nearest surface of concrete, bars shall not vary from plan placement by more than one fourth of an inch.

Tolerance for vertical and positioning of conduit or tendons will be plus or minus one fourth of an inch.

Where sections of forms are to be joined, an offset of one sixteenth of an inch for flat surfaces and one eighth of an inch for corners and bends will be permitted. Offsets between adjacent matching faces of segments shall not exceed one fourth of an inch.

Variations greater than specified above shall be corrected to within these tolerances or be subject to structural review.

**4122.9. Measurement.** Precast concrete segmental structures complete in place will be measured by the square foot of roadway surface area. The roadway surface area will be determined by the product of the upper horizontal surface out-to-out width normal to the centerline and the overall centerline length of segments, using the dimensions shown on the plans.

**4122.10. Payment.** Payment for precast concrete segmental structures will be made at the contract unit price bid per square foot measured as prescribed above. The above payment shall be full compensation for casting segments, for furnishing and placing all reinforcing steel, conduits, and prestressing steel, for all testing, for transportation and erection, for furnishing and applying epoxy bonding agent, for prestressing, grouting, grout fittings, anchorages, bearings, bearing plates, all concrete, all special handling, erection and assembly equipment or jigs, all tools, labor, equipment and incidentals necessary to complete the work.

A P P E N D I X    C

SUGGESTED INSPECTION PROCEDURES FOR  
PRECAST SEGMENTAL UNITS

Prepared by

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-- CTR Library Digitization Team

## FABRICATION AND INSPECTION

"One test is worth a thousand expert opinions."

Our Division's motto fit this first inspection of precast segmental units to a T--a Texas T. The challenge of being first was offset by the privilege of working with Dr. John Breen and his outstanding research team.

At the time of letting of this project, all bridges of this type had been fabricated outside the United States and inspection criteria were not available. We were able to observe some of the fabrication of the 1/6th scale units at The University of Texas Balcones Research Center. As time marched on we found ourselves facing many of the same problems on the prototypes that Dr. Breen and his staff faced during the forming and casting operations of the models. Many problems such as bond break agents, admixtures, form control, etc., were averted due to model construction.

Quality control of segmental unit fabrication cannot be oversold. The parent casting method used helped to assure positive fit for erection and stressing operations that followed. Please consider that fabrication errors in the precast yard could become all too evident when cantilevered out from the supporting pier. Inspection forces should check and recheck critical items such as tendon ducts paths, unit mating faces, grade alignment of units, concrete consolidation around anchor systems, etc., etc.

Alert inspection should play an important role in assuring the precast segmental units fit in place and withstand design forces.

Many of the lessons learned during this project were related to the Bridge Design Section. The following review aided in the preparation of Special Specification Item 4122 ("Precast Concrete Segmental Structures") for a subsequent project and is a good summary.

I. Plant Facilities

Bed and forms must be structurally adequate to support segmental units. Approval of bed should include method and hardware needed to adjust grade and alignment.

II. Shop Drawings and/or Contract Drawings

- A. To aid in inspection of rebar spacing, details should be shown on the same sheet with the conduit path spacing (one unit - one sheet).
- B. Particular attention should be given to jacking and anchor pockets with respect to rebar spacing. Spacing should be shown between bars, spacing from edges of pockets and/or units to bars, and vertical dimensions from floor or deck to edge of pockets.
- C. Conflicts in bar spacing with other bars, conduit, and interference with jacking and anchor pockets, shear keys, etc., should be held to a minimum.
- D. Design rebar (larger and fewer bars, etc.) where possible to allow for maximum concrete flow in critical areas such as around jacking and anchor pockets.
- E. Drawings should show much more detail as to placement of conduit paths, especially more vertical and horizontal dimensions either by plan views or cross section views or both to provide for a smooth curve of conduit.
- F. Get away from "typical" details and show actual rebar placement, conduit paths, jacking and anchor pockets, etc., for each separate unit.

- III.
- A. Conduit - Give some consideration as to alternate material for conduit such as smooth steel pipe with welded seam, plastic pipe, etc.
  - B. Headers - Material used for headers should be maintained to provide a smooth casting surface.

- C. Bond Break Material - Bond break materials should be subjected to in-plant tests to determine the acceptability of the material. Should be tested on large sections to more represent actual conditions.
- D. Grout Port Vents - More rigid materials should be used in fittings and hardware for grout port vents.
- E. Concrete - Should give some consideration to specifying that a retarding admixture be required in all concrete used in segmental units.
- F. Consider surface area waiver for Type II cement for segmental units similar to statement for Prestressed Concrete Piling, Ref. Item 421.
- G. Suggest a requirement to recess around conduit at joint for rubber gasket seal.

#### IV. Fabrication

- A. Shear Keys
  - 1. Consider alternate design of shear keys such as rounded surfaces to aid in bond break and help eliminate breakage, i.e., get away from square corners.
  - 2. Consider reducing width of shear keys in the web. This would allow a thicker wall between the shear key and outside of unit.
  - 3. Consider reducing size of rebar in shear keys.
  - 4. If horizontal alignment keys are necessary they should be redesigned to allow for proper reinforcement.
- B. Tolerances
  - 1. Consider developing tolerances for various conduit locations. Can the same tolerances be used for both vertical and horizontal? Can entry through unit tolerances be the same?
  - 2. Present rebar spacing tolerances should be adequate as long as required concrete cover can be maintained. Can cover of

concrete be less than 1 in.? If so, specify in which areas. Careful consideration should be given to requiring a minimum 2 in. concrete cover for exposed surfaces in structural members exposed to salt water.

3. Develop dimensional tolerances for the unit. Example:  
Length, width, depth, thickness of bottom slab, top slab, webs, etc.
  4. Develop grade tolerances both longitudinally and transversely for the unit.
  5. Develop vertical batter and/or skew tolerances for the unit headers.
  6. Develop placement tolerances for jacking pockets, anchor pockets, and shear keys.

Placing of Concrete - Suggest that sequence of casting be shown on shop drawings along with method of consolidating concrete.  
Example: Sequence casting: (1) Web, (2) Unit Floor, (3) Deck

Method of Consolidation: (1) External and Internal Vibration,  
(2) External Vibration only,  
(3) Internal Vibration only, etc.

Method of consolidation and equipment used shall be approved by the Engineer.

Finish of Concrete - Specify concrete finish required for top of deck.

Curing of Concrete - Specific curing should be outlined for segmental units as to curing days, etc. Curing method and equipment shall be subject to approval of Engineer.

Separation of Parent Cast Units - Require method and equipment to be used for separation of parent cast units be submitted and approved by the Engineer.

All segmental units shall be properly identified before removing from casting bed.

The following check list was of assistance to on-site inspectors in highlighting some of the special areas requiring checking.

#### Inspection Procedures

- A. Dimension check of deck, wall, soffit, and headers
  - 1. Tightness of fit of wall form to soffit to prevent grout leakage
  - 2. Fit of headers to steel deck section--otherwise offset may break off during unit separation, disturbing mating faces
- B. All jacking anchorages checked for dimension and slope--inside throat of anchorage through which strand will pass must be free from burrs, etc.
- C. Conduits
  - 1. Size--must guide strand
  - 2. Condition--broken or damaged will allow grout to enter and block
- D. All jacking pockets for positive and negative tendons are formed by plywood or steel block-out
  - 1. Proper shape, slope, and size for correct angle of jacking
  - 2. Proper cover for wall reinforcement
- E. Special inspection should be given anchorage items: check for damage, broken welds, etc.

#### Inspection Procedures--Rebar

Reinforcing steel in segmental units performs basically the same task as in pretensioned concrete, that of containing the prestressing forces and reinforcing the concrete.

- A. Grade--40, 60, etc. These members require Grade 60 for all reinforcement with the exception of the parapet steel.
- B. Size and Shape--Coverage or difficulty in getting bars into plan position cannot govern size or shape; any changes might affect the forces required to contain prestress forces. Even minor changes of rebar in stress areas should be submitted to the design section of D-5 for approval.
- C. Bending Tolerance
  - 1. Deck Steel--slight errors in bending radius may cause the lower layer of top slab steel to interfere with the conduit paths at the haunch (deck to web).
- D. End Trimming--Rebar which requires end trimming must be inspected closely to prevent the bar ends from falling between conduit and thereby preventing proper concrete consolidation. This is very important in midspan members where the positive moment tendons are grouped in the bottom of the web section.

- E. Floor and Diaphragms--Rebar placement in these areas are of a routine nature and require normal inspection.
- F. Tendon Ducts - Tendon duct positioning takes place in top deck assembly after the bottom or lower mat of deck bar is in place. The paths that will remain in a deck position in a particular member are laid inbetween the deck mats. Paths dropping into the web are the last to be positioned prior to placing interior forms.

#### Inspection Procedures for Tendon Ducts

Since frictional problems can cause many problems in stressing, the path of the conduit must maintain its integrity. Thin wall conduits must not be allowed to lay directly on transverse rebar. The concrete weight can deform the conduit at this point and result in a hump effect in the floor of the conduit.

- A. Pier Section--Duct vertical and horizontal measurements are set in this starting member.
- B. Typical Unit Duct Paths
  1. Duct must stay surrounded by steel
  2. Path should be smooth
  3. Rebar must be routed around ducts which are in the drop position
  4. Turn down area to anchorage must be checked carefully
  5. Positive and negative tendon ducts must cross without interference
  6. Positive deck anchors must be set with proper slope to deck surface
  7. Parent casting allows ducts to extend into new member prior to assembly of cage. This ensures exact positioning--rubber hoses should be used to stiffen ducts
  8. Final inspection after headers in place
    - a. Height of soffit
    - b. Horizontal alignment
  9. After casting should be inspected for rupture or breakage

#### Inspection for Completed Units

- A. Mating Faces--inspected for damage
- B. Any repairs to unit must not interfere with mating surfaces
- C. Concrete around all jacking anchors must be checked for soundness
- D. All ducts must be inspected to assure the open path for tendons; a plug should be passed through all ducts

- E. The surface of all mating faces must be inspected for bond break agent removal. If the surface is not cleaned, a proper bond by the epoxy joint material cannot be attained. The best method of removal is to use a light sandblast
- F. Upon final inspection, the units must be given permanent identification for erection and stamped for shipment

This outline of suggestions for future design and inspection of segmental bridge units was compiled after the project had been completed. It does not include all of the inspection procedures used, but was intended to reflect on the major areas of concern from the "as built" viewpoint. The Materials and Tests Division of the Texas Highway Department is pleased to share our hindsight of inspection processes. Whatever this hindsight is worth, it does prove - "the thing was done once"!