Reconstruction-West Approaches San Francisco-Oakland Bay Bridge

by **Norman C. Raab***

Introduction

Since the advent of the automobile and its increasing popularity for individual transportation, many problems have been created for those in the field of highway construction. These problems are more intensified in the urban areas where thousands of workers use this means of transportation between home and business. There appears to be no immediate solution except the continued construction of additional highway facilities. Until ways and means are provided for new construction the existing facilities must be improved upon and enlarged to keep pace with the increasing traffic volume.

A Difficult Problem

One of the most challenging problems arose recently during the reconstruction of the San Francisco-Oakland Bay Bridge. The reconstruction was done to enable the bridge to handle its ever increasing automobile traffic. Due to the geographical location of the City of San Francisco its main ingress, except from the south, is over two large bridges. One of these transbay bridges is a structure six miles in length connecting the City of San Francisco with that of Oakland across the Bay. The westerly half of the crossing is composed of twin suspension bridges, with main spans of 2,310 feet, between San Francisco and Yerba Buena Island. Its easterly half consists of a 1,400 foot cantilever span followed by a series of long steel truss spans from the Island to Oakland. From the time the Bridge opened in 1937, traffic has increased from a daily average of 25,000 vehicles to over 100,000 today. Although studies have indicated that a completely new and additional crossing is necessary to handle the increasing volume, high costs and local disagreement as to its location has prevented its construction. Of necessity, therefore, every means of increasing the capacity of the existing structure has been of paramount importance.

As originally constructed, the upper deck of the Bridge carries six, 9 ft 8 in. wide, lanes of automobile traffic only; three in each direction. The lower deck had three, 10 ft 4 in. wide, lanes for heavier commercial vehicles on the north side of the Bridge and two tracks for interurban trains on the south. Patronage on the trains had steadily declined from a maximum of 26 million in 1945 to 5 million in 1957 and, as a result, in 1958 permission was granted by State authorities to abandon the electric railway lines and inaugurate motor coach service.

Studies were made of the ways and means of converting the lower deck, with its approaches, for the exclusive use of vehicular traffic. The report of the California State Department of Public Works outlined a plan of reconstruction and means of financing the project over a four year period. The scheme was adopted, plans prepared and construction started during the latter part of 1958.

The reconstruction called for five, 11 ft. 7 in. wide. lanes westbound

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on the upper deck to San Francisco and the same number eastbound on the lower deck to Oakland and the East Bay. The work was programmed to start from the west end of the crossing and to proceed eastward since routing of the transbay buses from the city streets into the idled train terminal directly from the Bridge was of immediate importance.

The first contracts required that all removed trackage be transported over the abandoned bridge railway and stockpiled in the East Bay Storage Yard. This eliminated the additional traffic on the lower deck lanes resulting from construction equipment. Subsequent contracts to date covered the paving of the Terminal Loop, the reconstruction of the west approaches, and the repaving of the lower deck of the west bay spans from San Francisco to Yerba Buena Island. Contracts covering work on the Island and for the repaving of the east bay spans are now in progress.

Prestressing Existing Structure

One of the major keys to the success of the project centered upon the "Reconstruction of the West Approaches". Originally the lower deck with its 31 foot, three lane roadway, had but one connection to the city streets of San Francisco. There are now six roadways leading to the lower level, including connections to the Bayshore Freeway, one to the Terminal bus loop, and four to the surrounding streets. The number of lanes required for each of these ramps was determined from traffic counts. To accomplish these changes, three additional lanes plus three side ramps were brought in under the upper deck at the west end. The construction of these roadways required the removal of some 30 columns supporting the existing upper deck. About half of these columns, which previously had separated the railway half of the lower deck from the truck traffic, were along the centerline of the new roadway. The remainder were exterior columns which had to be moved to gain access for the new ramps from the sides. In most cases this meant the center of the three supporting columns of a bent had to be removed, and the existing floor beam strengthened to span the greater distance between the outside columns. See figure No. 1. The upper deck floor framing consisted of reinforced concrete "Tee" beam construction with longitudinal girders and transverse floor beams. Loads from 250 to 300 tons which were supported by the center columns would have to be carried by the floor beams on increased span lengths now measuring 65 to 85 feet. The original pattern of supporting columns was highly irregular, particularly in the area where the railway tracks emerged from the structure and proceeded to the Transbay Transit Terminal in San Francisco. This made exact determination of column loads rather difficult resulting in one of the requirements for the new supporting system; namely, that of possible adjustment under actual load.

Of the available methods of strengthening the bents for support of the upper deck at locations where columns were removed, the following were considered:

1. The addition of built up steel girders.

2. The addition of prestressed concrete beam sections.

3. Prestressing the existing structure.

The first two methods would require extensive revisions to the ex-

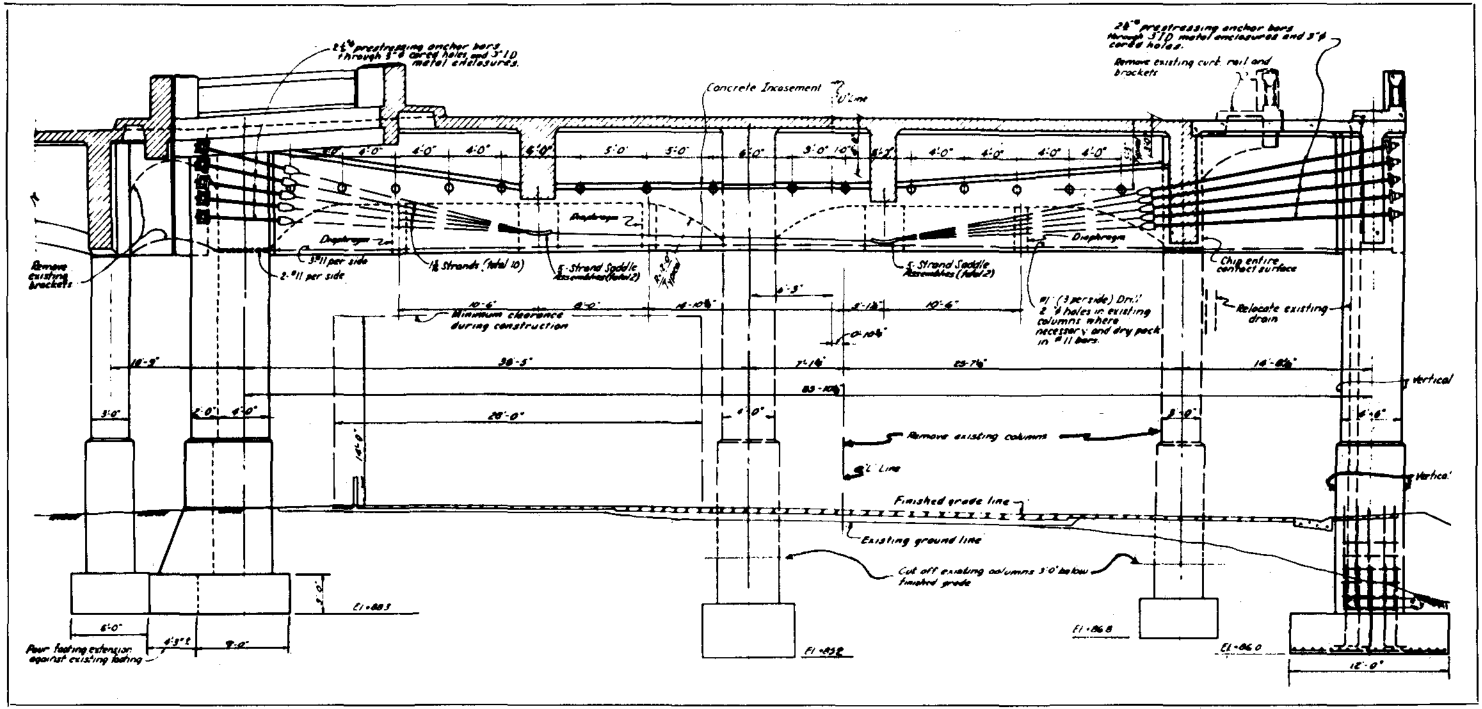


Fig. 1—Elevation of Bent 43 Indicating Columns to be Removed and Application of Prestressing System.

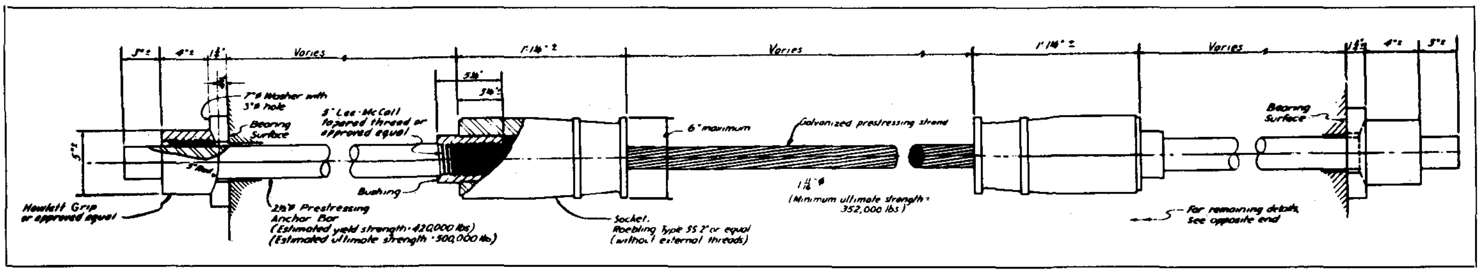


Fig. 2—Prestressing Assembly Combining Strands and Bars.

isting construction resulting in an unsightly massive heterogeneous structure and not subject to a determinant analysis.

It, therefore, became apparent that short of extensive remodeling, the best prospect for a solution with the least inconvenience to the existing heavy traffic was the third method noted above, that of "prestressing the existing structure".

A Unique Solution

The removal of the center columns required a tensioning device which would support the transverse beams near mid-span and which could be anchored to the backs of the two outside columns.

This led to a new prestressed concrete concept used for the first time in American construction, a post-tensioned tendon composed in part of high tensile steel anchor bars and the remainder of prestressing strand.

During the solution of this problem, it was necessary to determine that the existing structure had the proper cross sectional area in the transverse floor beams at the bents to take the prestressing force which amounted in some cases to three million pounds. Fortunately, due to architectural treatment as well as structural reasons the original floor beams at the columns were unusually heavy and, coupled with the excellent quality of the 24 year old concrete, proved to have sufficient strength for prestressing.

Due to the varying cross section of the existing floor beams and the different modulus of elasticity between the new and old concrete, a method of checking the theoretical stresses was used that is of interest. See figure No. 3.

In order to analyze the frame, the resultant of the compressive force on the concrete, the "C" Line, was used to determine the deflected shape of the members. The resulting moment corrections were then found and the final shear and flexural stresses computed accordingly.

The prestressing tendons had to be so arranged to properly distribute this large force that is jacked into the beam. It also had to be of such size that the columns and girders were not perforated with numerous small cored holes detrimental to the strength of the carrying members.

This led to the choice of the large combination tendons, see figure No. 2, which consisted of the following:

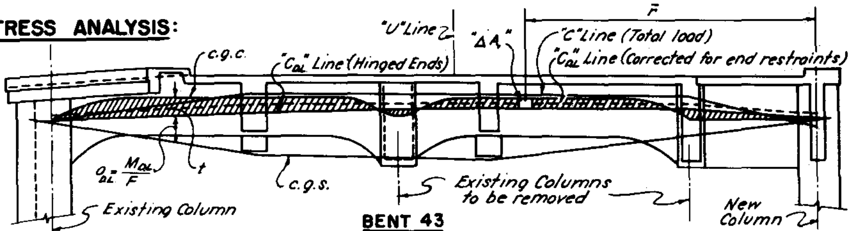
1. High strength prestressing strands.
2. A steel socket on each end of the strand assembly.
3. An internal and external threaded bushing for the socket.
4. Two high strength bars.
5. End plate and washer.
6. Howlett grip nuts.

Wire strands were used for the center portion of the assembly as they were flexible and would conform to the radii of the steel strand shoes located in the vicinity of the removed column.

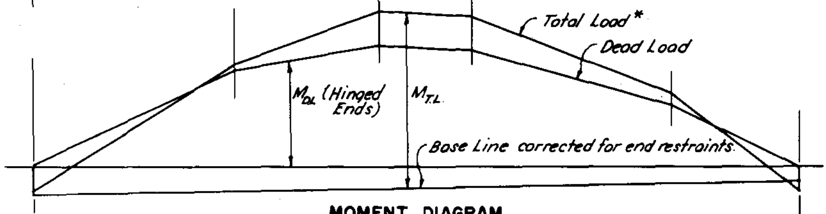
The manufacturer of the strands were reluctant to guarantee the length, workmanship and strength of the assembly unless the strand was prestretched, cut and babbitted into the socket at the factory. If a full length strand were used, a large hole would be required through the concrete to pass the socket. This led to the selection of a strand, socket and bar assembly.

The open ends of the sockets were machine threaded at the factory for the accommodation of the standard medium strength jacking bars. However, a high strength bar would reduce the diameter of the hole to be

STRESS ANALYSIS:

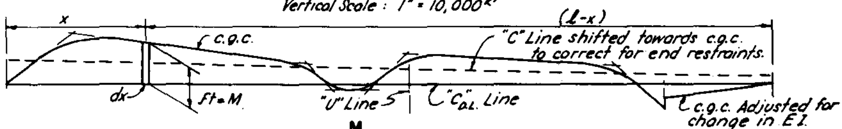


BENT 43



MOMENT DIAGRAM

Vertical Scale : 1" = 10,000k'



DEAD LOAD $\frac{M}{EI}$ DIAGRAM

Vertical Scale : 1" = 4'

END ROTATIONS: $\theta_L = \int_0^{\ell} \frac{(\ell - x)(Ft)\,dx}{EI\ell}$ $\qquad \theta_R = \int_0^{\ell} \frac{x(Ft)\,dx}{EI\ell}$

graphically $\theta_L = \frac{F}{EI\ell} \sum \Delta A_1 \bar{r}_{Rt}$ $\qquad \theta_R = \frac{F}{EI\ell} \sum \Delta A_1 \bar{r}_{Lt}$

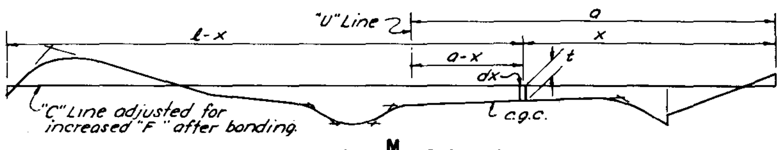
FIXED END MOMENTS: $M_{DL} = \frac{4EI\theta}{L_{col}}$; then distributed and Base Line of above moment diagram corrected accordingly.

* Note: Other moments, such as liveload, temperature, seismic, etc., found by conventional methods.

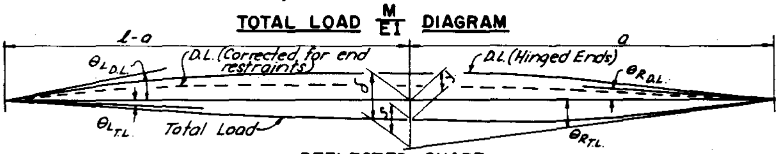
DEFLECTION "y" at any point "a":

$$\text{"}y\text{"} = \delta - s = a\int_0^{\ell} \frac{(\ell - x)Ft\,dx}{EI\ell} - \int_0^{a} \frac{(a-x)Ft\,dx}{EI}$$

graphically $y = \frac{aF}{EI\ell} \sum \Delta A_n \bar{r}_{Lt} - \frac{F}{EI} \sum \Delta A_n (a - x)$



TOTAL LOAD $\frac{M}{EI}$ DIAGRAM



DEFLECTED SHAPE

STRESSES IN CONCRETE: $f_c = \frac{F}{A} \pm \frac{Ftc}{I}$

Fig. 3—Determination of Stresses and Deflections.

cored through the concrete and was better adapted to the particular jacking device and end anchorage developed for this case. It was therefore decided to manufacture a bushing with external machine threads that conformed to those of the socket and tapered internal pipe threads to fit those on the bar. By using a tapered thread the full area of the high strength bar was utilized. See figure 4.

The anchorage washer had a concave machined surface surrounding the bar hole which fitted the machined convex end of the anchor nut. This acted as a universal joint which adjusted to the small inaccuracies in the concrete drilling and the setting of the bearing plates. See figure 5.

The next step involved the anchoring of the high strength bars. Due to varying lengths of tendons a method that could take adjustment was called for. The previously tested and tried system for prestressing bars up to 1¼ inch diameter using an anchorage device called a "Howlett Grip", was investigated. This device consists of a serrated bushing fitting over and forced onto the prestressing bar by an external nut with wedged threads. As this method appeared promising, a prototype was made and tensioned in a three million pound universal testing machine at the University of California. The resulting grip, four inches in length was found capable of developing over 500,000 pounds ultimate load and satisfied the requirements of the assembly.

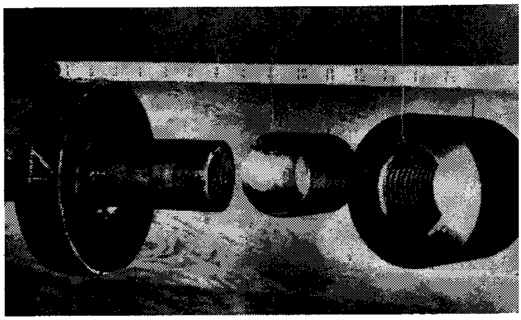


Fig. 5—Anchorage Assembly, Showing Concaved Machined Washer.

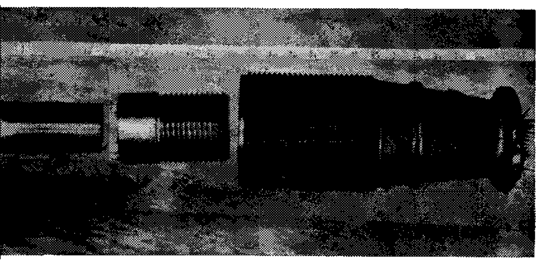


Fig. 4—Strand Socket, and Bushing and Anchor Bar Assembly.

Another question was the choice of steel for the anchor bars. In order to minimize the size of hole to be drilled through the existing concrete and reduce the amount of creep in the steel the following characteristics were required: A high tensile alloy steel bar of tempered martensitic microstructure, with a yield strength at .2 percent offset of at least 90,000 psi, and an ultimate strength of over 105,000 psi. A minimum Charpy Impact value, based on the Keyhole notch test at 0°F., of 15-foot pounds was also specified. The bars used were 2½ inches in diameter, of heat treated steel. (AISI) Designation: 4140, quenched and tempered. The actual working stress in the bars was under 60,000 psi. All bars were proof loaded to 90,000 psi before use.

The strands used were $1\frac{11}{16}$ inches and $1\frac{13}{16}$ inches in diameter, socketed at both ends. Due to the different prestressing forces the number of assemblies varied from a minimum of six to as many as twelve. Here again, tests were made of complete assemblies identical to those planned for use in the structure. Four tensile tests were made and in each case failure occurred first in the strand. This load at failure was 389,000 pounds or more for the $1^{11}/_{16}$ inch diametered strands and 433,000 pounds or more for the $1^{13}/_{16}$ inch diametered strands; well over the minimum ultimate loads specified.

The next question pertained to the path of the prestressing tendons. Since the superstructure loads produced a maximum stress in the area of the removed column it was apparent that a deflected path leading under the column stubs was required. An inverted saddle with a radius of curvature of 3 ft. 6 in. was devised under which the strands would pass. To reduce friction between the strand and saddle and to prevent line bearing of the strands, sheet lead was used as a separator.

Since the strand pattern flared out at the supporting columns it was necessary to core the holes to various vertical angles. To allow some coring tolerance and for slight variations in setting of bearing plates, spherical washers were designed to work as a ball and socket arrangement with the anchorage nut. These provided for a two degree variation in horizontal or vertical alignment from the theoretical.

Due to addition extensions of the concrete caps, in some cases, and to the multiple units involved in the

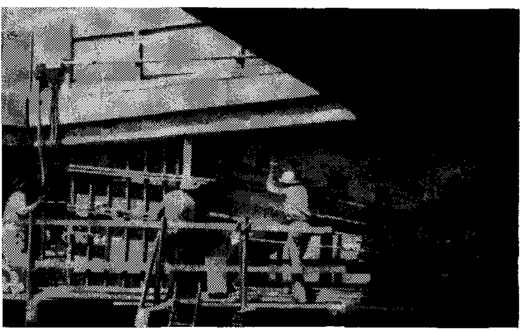


Fig. 6—Positioning Prestressing Assemblies on Bent 43.

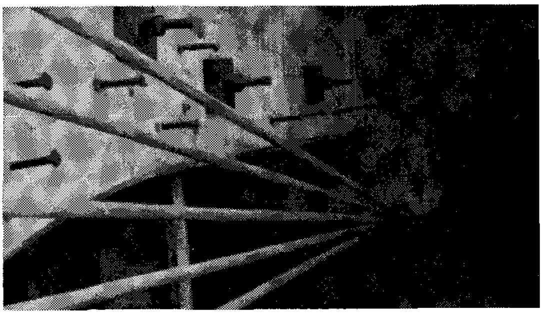


Fig. 7—Strands and Saddle at Bent 43. Note Encasement Steel Extending from the Beam.

prestressing tendons, the problem of shrinkage and creep in the concrete had to be considered. Creep, mechanical take-up and seating losses in the steel anchor bars and prestressing strands was also a consideration. As a value of these losses was difficult to predetermine, retensioning of the tendons was specified one week after the initial tensioning. Dynamometers of two hundred ton capacity were coupled to each jack for a check on the accuracy of the tensioning load. The tendons were jacked in symmetrical pairs and from both ends simultaneously. Measurements of the elongation of the tendons as well as the elastic shortening of the concrete were also made as an additional check on the prestressing force being transmitted to the structure.

This again proved the advantage of the anchorage device chosen, as it could take up the losses of various amounts by merely advancing the nut on the sleeve. In the actual construction this amounted to about a 15 percent loss in stress or approximately a $1/4$-inch adjustment at each nut at the time of retensioning.

It would be well to mention here that the adjustment capabilities of the grip would also have answered the problem stated earlier; that of adjusting the prestress value to

Fig. 8—Looking East from Bent 51 Before Column Removal.

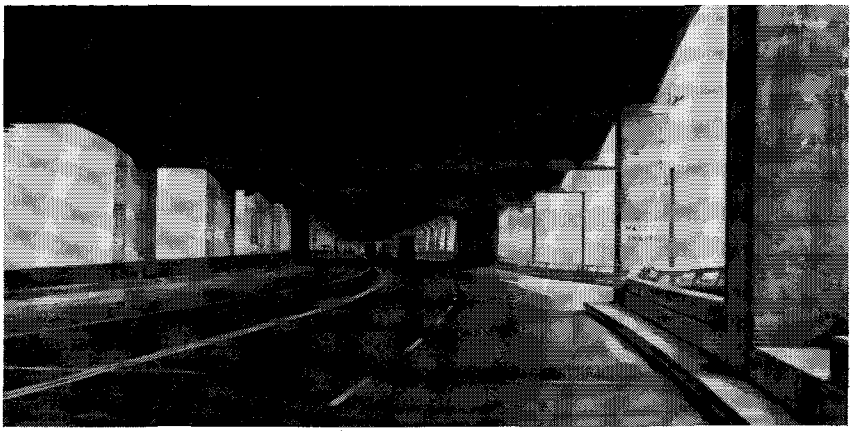


Fig. 9—Looking East from Bent 51 After Column Removal by Prestressing.

Fig. 10—Upper Deck Upon Completion Looking Toward Bay Bridge Suspension Spans.

match the actual column loads found in the field. At the time of cutting the columns free after prestressing, the resulting structure reacted as expected with an upward camber of ⅛ to ⅜-inch. This cambering checked the computed values. However, in the event they had not, at this point a third tensioning could have been employed to obtain the proper results. To determine this movement precise levels were run, before and after tensioning, over bench marks placed in the upper deck and from which periodic measurements are to be taken in the future.

These measurements also covered a side issue often overlooked while concentrating on analysis and other related structural problems, that of the additional insurance premiums required on structures of this type to cover possible costs of any failure during major remodeling. As these premiums are high it is necessary that the brokers be furnished with information of this type to assure them that the safety of the structure has been maintained.

The final question pertained to the bearing plates for the prestressing tendons. In certain cases since the tendons passed on the outside of the transverse floor beams they also passed outside the surface of the supporting columns, or they passed through the longitudinal girders supporting the upper deck. In these cases it was necessary to relieve this additional shearing force on the longitudinal girders and/or to put the load back onto the existing columns. To place the prestressing force back into the transverse floor beams a built-up bearing plate forming a yoke around the outside face of the columns was provided and, thus, the last major problem had been solved.

After final tensioning was completed, the holes for the anchor bars through the columns were grouted and the complete anchorage assemblies and tendons encased with lightweight concrete for protection and to add to the ultimate load carrying capacity of the structure.

Conclusion

The reconstructed west end of the San Francisco-Oakland Bay Bridge, consisting of one mile of concrete approaches, cost $3,000,000 and required 18 months to complete. The high cost and time element involved in this instance was due primarily to the hand tailored multistage construction requirements. Prestressing these parts of the structure, where needed, permitted work to be accomplished under abnormally heavy traffic conditions and with little inconvenience to the travelling public. It is doubtful if the motorists using the upper level were conscious of what was going on underneath their riding surface.

Progress in the understanding of prestressing techniques and the cooperation of the related industries in the development of parts and materials needed in its application continues to open new fields to the structural engineer. Ten years ago a solution of this type would not have been possible while today it is a relatively simple procedure.