Design Provisions for a Replaceable Segmental Bridge Deck

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A major precast concrete segmental bridge located in Glenwood Canyon, Colorado, was mandated by the Federal Highway Administration and, the Colorado Department of Highways to have the capability of full depth deck replacement. A simple procedure for replacement of the top slab of the box girder was developed and integrated into the final design of the structure. Although erected by the balanced cantilever method, no compromises were required of the design and replacement capability was satisfied with little initial cost impact to the construction of the viaduct. The deck replacement procedure, the special design features and the analysis method and results are presented. It is concluded that full depth deck replacement capability is economically feasible for a precast segmental bridge, ensuring the owner of a durable concrete structure for many years of service.

The Hanging Lake Viaduct in Glenwood Canyon, Colorado, is part of the last link to complete the I-70 route through Western Colorado. The project site is located within an extremely steep and narrow portion of the environmentally sensitive canyon. There is no reasonable alternate route capable of handling the traffic from I-70 at either end of the canyon and, therefore, maintenance of traffic during construction is of equal importance as preserving the environment. Thus, the design requirements of the viaduct included that the superstructure be completely erected from the bridge itself and prohibited the use of falsework or temporary bents during construction in areas where traffic or the environment would be affected.

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Based on those construction limitations, the Bidding Documents were prepared for both a precast, concrete segmental box girder alternate and a steel box girder alternate. In August of 1989, a joint venture of Flatiron Structures Company and The Prescon Corporation submitted a low bid of $34,091,925 for the concrete alternate and construction of the project began in the Fall of 1989.

The project consists of approximately 8,400 lineal feet of bridge with a typical span length of 200 feet and twin 300 foot spans crossing the Colorado River. To meet the construction requirements described above, the structure was designed to be built using the balanced cantilever method. The segments are trucked over the completed portion of the bridge and then placed in cantilever on either side of the pier using a piece of erection equipment called a launching gantry (see Figure 1). The gantry is self-launching and can be advanced from the previous cantilever to the following pier without the aid of additional ground or structure based equipment.

Prior to work beginning on the conceptual design of the project, the Federal Highway Administration (FHWA) and the structure's owner, the Colorado Department of Highways (CDOH), mandated that the concrete alternate be designed and constructed so as to allow for complete replacement of the entire deck.

It is worth noting, however, that while deterioration of reinforced concrete decks has been a serious problem for both concrete and steel bridges, concrete segmental bridges have not experienced the same problem. A recent survey, by the author's firm of existing segmental bridges, both in the United States and Europe, showed that major bridge deck deterioration is virtually non-existant, with some of the structures 25 years old. This is particularly true for structures built using precast segments with decks prestressed in the transverse direction. One particular example is the segmental bridge in Maine, crossing the

Sheepscot River at Wiscasset. Recent inspection by Maine DOT, both interior and exterior, revealed no deterioration in the deck of the seven year old structure. Other than 5,000 psi concrete and transverse post-tensioning of the deck, the only additional protection provided for the severe weather conditions was a 1-1/4" latex modified wearing surface. The main reasons that this and other segmental bridge decks are so durable are that the segments are typically cast with high strength concrete under controlled, factory conditions and are prestressed in both the transverse and longitudinal directions. This results in high quality decks which are designed to be essentially free of cracks under typical service conditions.

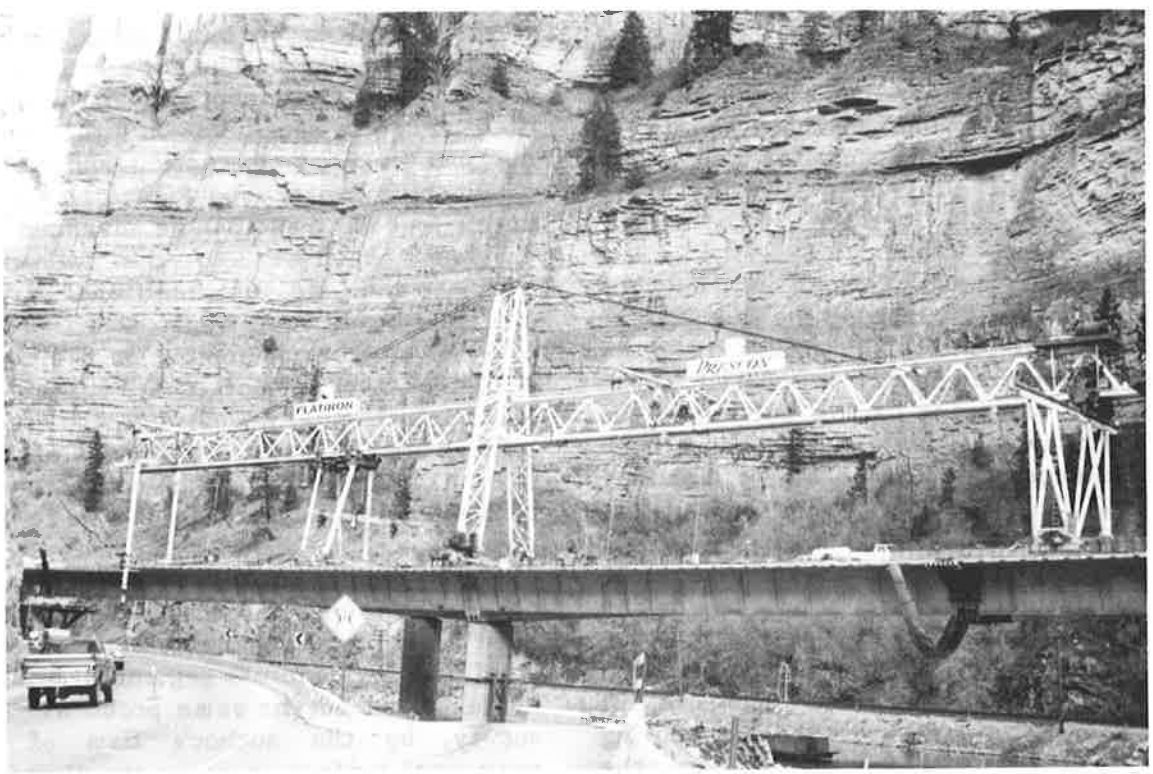


FIGURE 1 Overhead launching gantry.

Further evidence of the performance of this deck system can be found in the research by Poston, et al (1), who studied deck deterioration under harsh evnironmental conditions. The testing consisted of subjecting both prestressed and conventionally reinforced concrete slabs to an aggressive deicing salt exposure. Another variable used in the study was crack width and three cases were studied for both slab types - uncracked, .002" and .015". The results showed that chloride penetration occurred primarily at crack locations, and although the prestressing greatly reduced the penetration at crack locations in which the crack width was limited to .002", the chloride concentration was above the chloride-corrosion threshold at the reinforcement level. The uncracked sections, both prestressed and non-prestressed, had chloride values below the widely accepted chloride-corrison threshold. Thus, the researchers concluded that the primary benefit and goal of prestressing is to eliminate and to control greatly cracking so as to restrict the chloride and oxygen penetration into the concrete.

Therefore, the probability of deterioration of a segmental bridge deck to the point of complete replacement is highly unlikely. With proper inspection and maintenance, two repair options exist which require little, if any, special design considerations. The first is a concrete or asphalt wearing surface placed above a waterproof membrane adhered to the bridge deck. This protects the structural concrete deck and can be easily replaced as necessary. Second, if the deterioration extends into the concrete deck, the top 3 to 4, inches including the top mat of reinforcing steel, may be replaced and leaving the remainder of the deck intact.

However, should deterioration of the deck become so great, or damage occur to the deck by some other means, the consideration of complete deck replacement by the bridge designer provides the owner with the security of knowing that the option will always exist in the future. Based on that philosophy, the primary objectives of the design requirement were as follows:

1. Develop conceptual deck replacement scheme and associated details.
2. Incorporate into the final design of the structure the necessary details and structural capacities to allow for the deck replacement scheme.
3. Complete a structural analysis of a typical span through all stages of the deck replacement operation to verify the feasibility of the replacement scheme.

Each of the three objectives are discussed in more detail below.

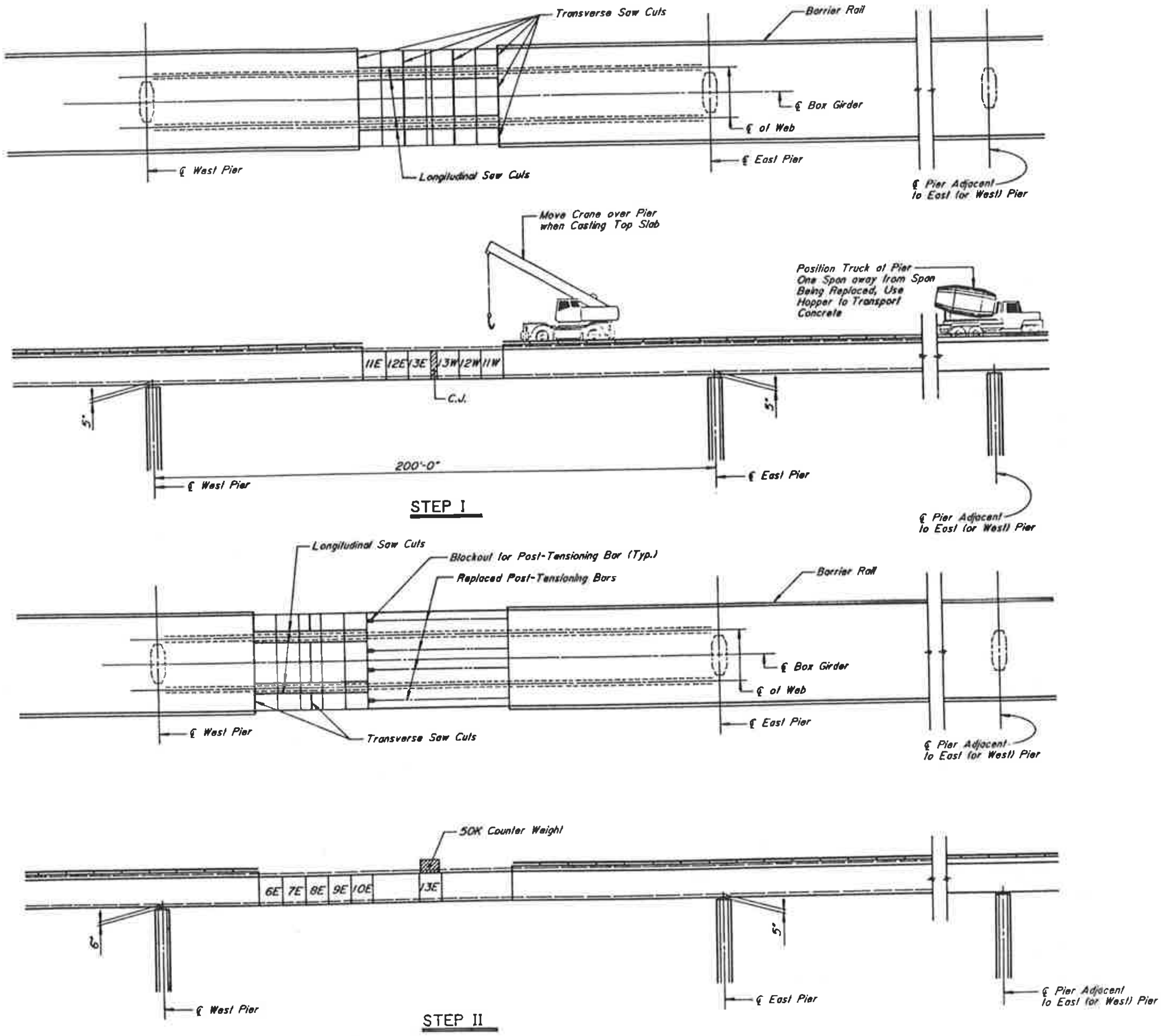
Deck Replacement Scheme

In developing a conceptual scheme for the complete deck replacement of a prestressed concrete segmental box girder, several important factors were kept in mind:

- As previously discussed, the procedure may never by necessary; therefore, the initial cost to the owner for the required details provided in the design of the structure should be kept to a minimum. The initial serviceability and economy of the structure should not be sacrificed to decrease the actual replacement costs.
- The restrictions due to the sensitive environment required during construction will also apply during the deck replacement. Therefore, no intermediate temporary bents or falsework will be allowed.
- The scheme should be as simple as possible and minimize the requirement for special construction equipment.

For this project there was no economic analysis required by the owner to determine if the deck replacement capability would be a final design requirement; this was previously decided by the owner and the deck replaceability was not an option to be studied. Therefore, no life cycle cost analyses of the structure including initial costs and deck replacement costs were done. However, the special design provisions proved to have minimal negative impact on the cost of initial construction and in fact may have had a slight positive impact. Also, although no actual cost estimates and comparisons were made, the cost of the developed deck replacement procedure should not be much different from that for a typical reinforced concrete deck on steel or concrete girders.

The scheme which was developed for the deck replacement of an entire span is illustrated fully in Figures 2 and 3 and the



REPLACEMENT SEQUENCE

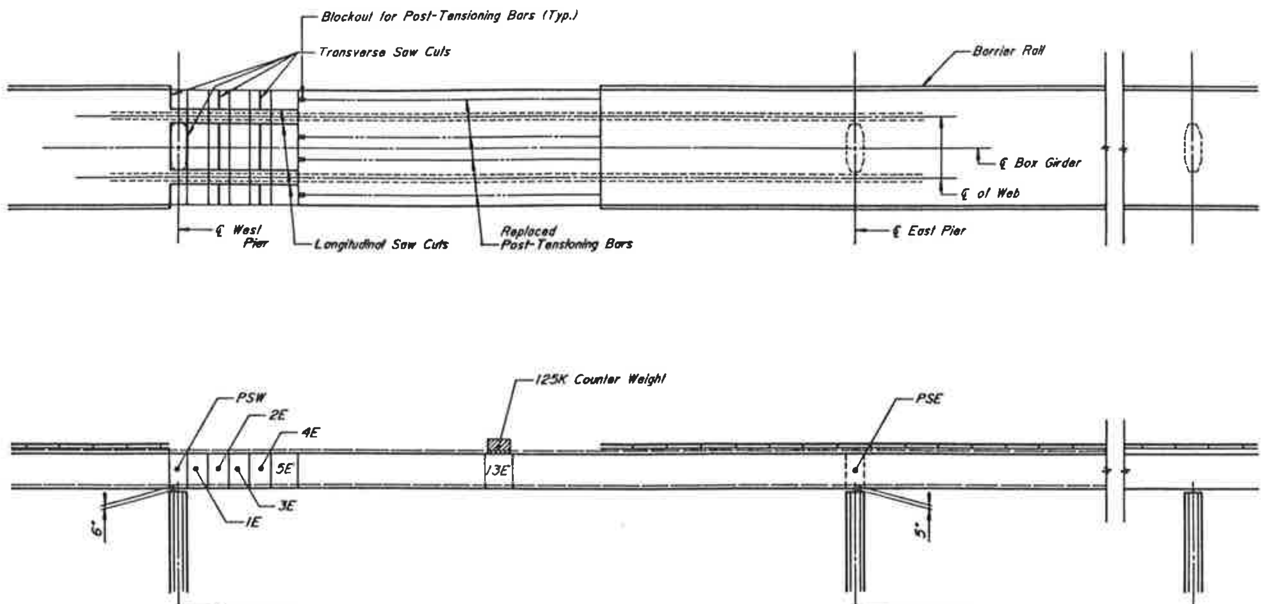
STEP I

1. Jack up deck 5" at piers adjacent to span being replaced.
2. Remove barriers from Segments 11E to 11W.
3. Cut post-tensioning bars in top slab at each transverse saw cut.
4. Make longitudinal and transverse saw cuts in deck and remove top slab.
5. Jack hammer area above and beside head of web to leave a rough surface.
6. Replace reinforcing, transverse post-tensioning and longitudinal post-tensioning bars.
7. Clean exposed concrete surfaces of all loose material.
8. Cast top slab in Segments 11E to 11W.
9. Stress transverse tendons in top slab.
10. Stress 4 longitudinal post-tensioning bars in top slab.
    NOTE: Blockout may be left in slab to accommodate stressing jack for bars.

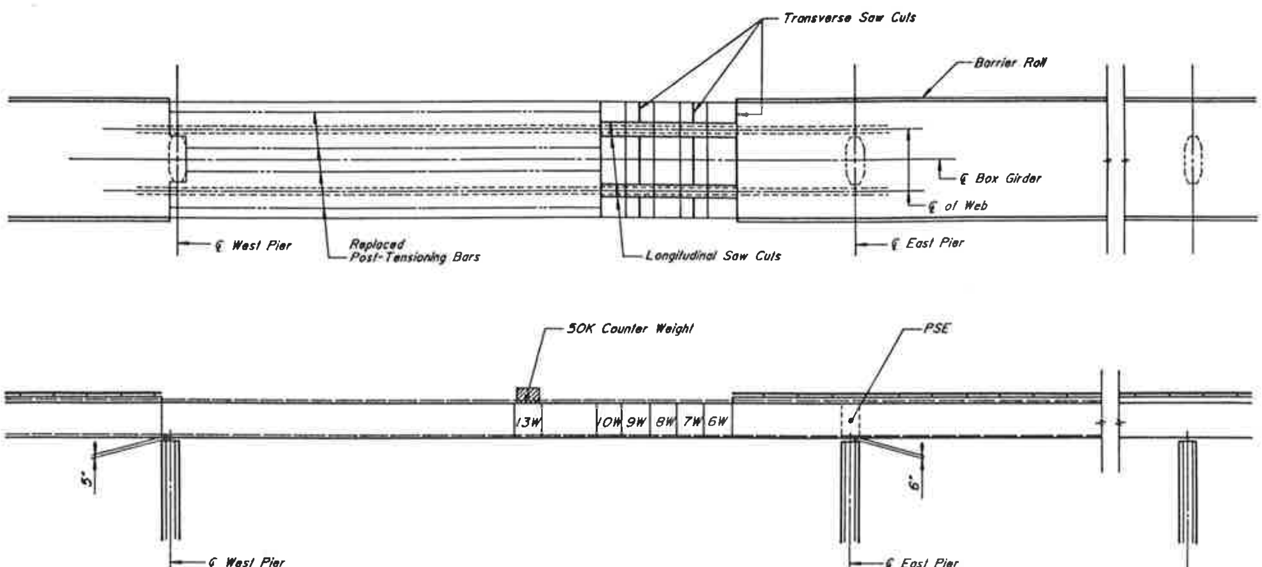
STEP II

1. Place 50^k counterweight over webs of segment 13E.
2. Jack up an additional 1" at west pier (to 6" total).
3. Remove barriers from Segments 6E to 10E.
4. Replace top slab in Segments 6E to 10E (Steps I.3 to I.8).
5. Couple post-tensioning bars to those in previously replaced portion and stress.

FIGURE 2 Deck replacement procedures, Steps I and II.



STEP III



STEP IV

STEP III

1. Place an additional 75^k counterweight over webs of Segment 13E. (To 125^k total).
2. Remove barriers from Segments PSW, IE to 5E.
3. Replace top slab in segments PSW, IE to 5E (Steps I.3 to I.9).
   NOTE: Replace only top 6" of deck between webs over the pier segment diaphragm.
4. Couple post-tensioning bars to those in previously replaced portion and stress.

STEP IV

1. Lower deck 1" at west pier and raise deck an additional 1" at east pier (to 6" total).
2. Remove 125^k counterweight from segment 13E and place 50^k counterweight over webs of segment 13W.
3. Remove barriers from segments 6W to 10W.
4. Replace top slab in segments 6W to 10W (Steps I.3 to I.9).
5. Couple post-tensioning bars to those in previously replaced section and stress.

STEP V

1. Replace top slab in Segments PSE, 1W to 5W, using same procedure as Step III.

STEP VI

1. Lower deck to grade at east and west piers.
2. Cast barriers and wearing surface.
3. Stress the contingency tendons in the replacement span.
4. Return bridge to service.

FIGURE 3 Deck replacement procedure Steps III through VI.

important features are discussed below.

As stated earlier, the deck in a typical segmental bridge is under compression in both the longitudinal and transverse directions under service conditions. Therefore, prior to removing the deck, this compression must be removed. Transversally, this compression is provided only by the transverse post-tensioning and can be eliminated simply by making a series of saw cuts in the longitudinal direction through the concrete and the post-tensioning tendons. Longitudinally, the compression is due to a combination of post-tensioning and structure dead load. The most straight forward method of removing the compression is by raising the structure vertically at the piers adjacent to span under consideration. This induces a constant negative moment in the span and thus a constant decompression of the top slab. The structure is raised enough to nearly provide a state of zero stress down to the level of concrete to be removed. For the typical 200 foot span, this was found to be 5 inches. Once the compression has been removed, the concrete can then also be safely cut in the transverse direction in the area being replaced. The 6 inches of concrete over the webs is removed down to the level of the cantilever tendons by using a jackhammer.

The section of the span replaced at one time was limited for two reasons. First, it allows for replacement of a limited section of the top slab if the damage is local. Second, it provides more stability by limiting the length of span having the decreased torsional stiffness. Once the concrete has been removed in the section under repair, the new reinforcement and transverse post-tensioning is placed. Figure 4 depicts in detail the portions of the deck which are replaced and the overall cross-section remaining once the replaceable deck sections have been removed. As discussed more fully in the next section, no main longitudinal post-tensioning is located within the replaceable deck area; however, there are longitudinal post-tensioning bars which are replaced and connected to the existing bars by using standard coupling devices. Once the concrete has been cast and reaches the necessary strength, the new post-tensioning is stressed and work may proceed to the next section if necessary, or the span is lowered if no additional deck needs replacing.

Special Design Provisions

Once the scheme has been developed for the deck replacement procedure, provisions had to be made in the design of the structure to make the scheme feasible. By choosing jacking of the span to induce a constant negative moment to remove top slab compression, the final stress state of the top slab should be as nearly uniform as possible along the entire span. The typical final top slab compression for a bridge built in balanced cantilever is a minimum at the pier, somewhat higher at midspan and is the largest near the quarter-point of the span. This "bubble" is due to the cantilever post-tensioning being governed by the stresses at the pier during cantilever erection, resulting in an excess amount of post-tensioning near the cantilever midpoint. A stress distribution of this shape would result in either the zero stress point being too low near the piers or excess compression remaining near the quarter-point of the span when applying the constant negative moment by raising the span.

To solve this problem, the longitudinal post-tensioning layout was optimized to produce as nearly a uniform compression state as possible, while still meeting the allowable stress requirements during construction and service. The result was a layout shown in Figure 5, using both typical cantilever tendons located in the top slab and external draped tendons anchoring in blocks above the bottom slab on either side of the pier. The addition of the draped tendons, as seen in Figure 6, provides the negative moment capacity at the piers, while reducing the excess top slab compression away from the pier.

The external draped tendons solved other problems as well. First, the drape of the tendon provides additional shear resistance near the pier. This is important when replacing the deck near the pier when the section properties are reduced. Second, it helped reduce the problem of where to locate the cantilever tendons such that they are not within the replaceable top slab. The draped tendons reduced the number of typical cantilever tendons required and enabled all of them to be placed above the web as shown in Figure 4. The typical 200 foot span required only 6 19-0.6 inch diameter strand cantilever tendons per web, although their efficiency was reduced by their lower placement in the top slab. FHWA and CDOH agreed to limiting the replaceable concrete to a depth of

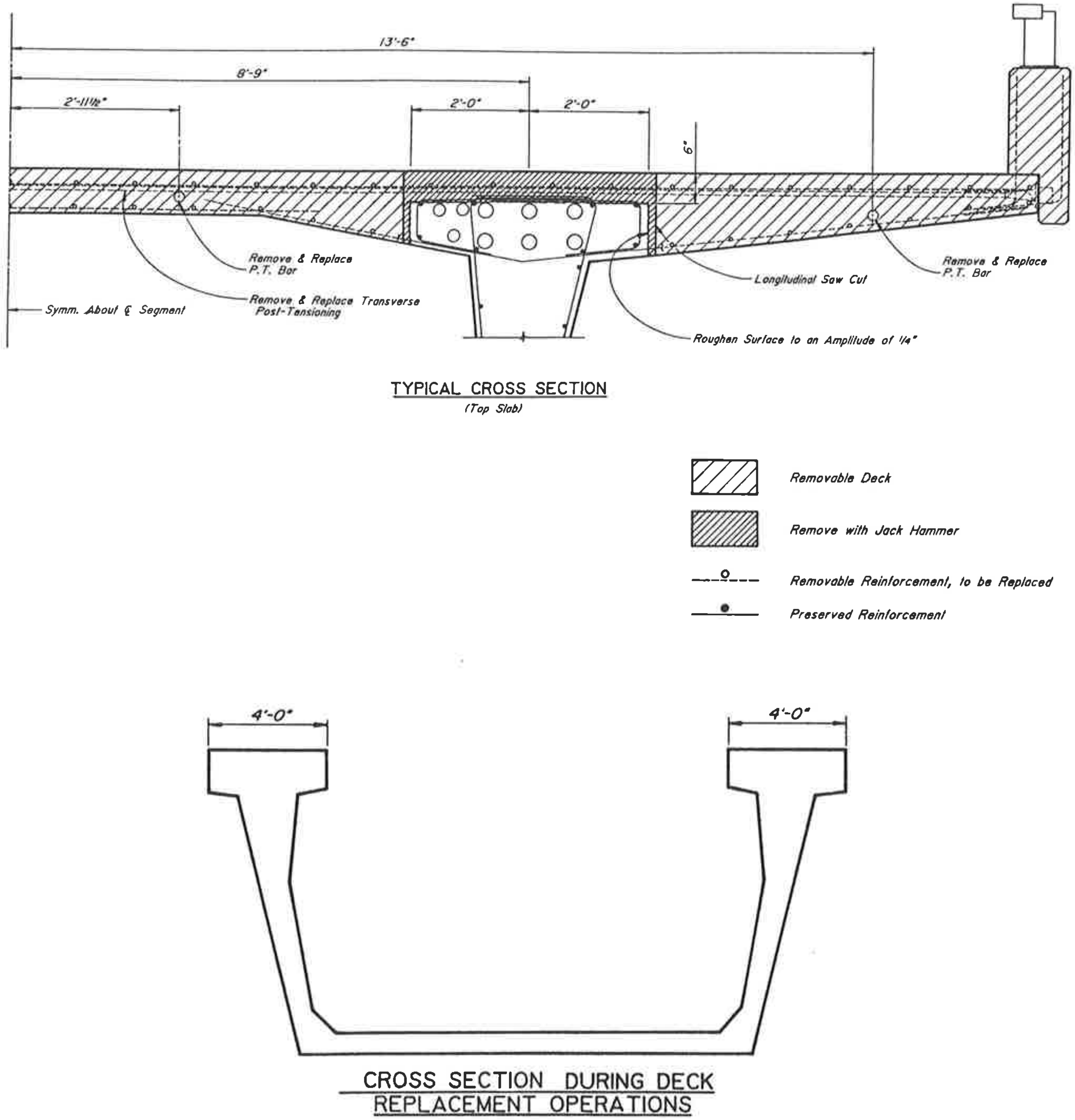


FIGURE 4 Typical cross-sections during replacement.

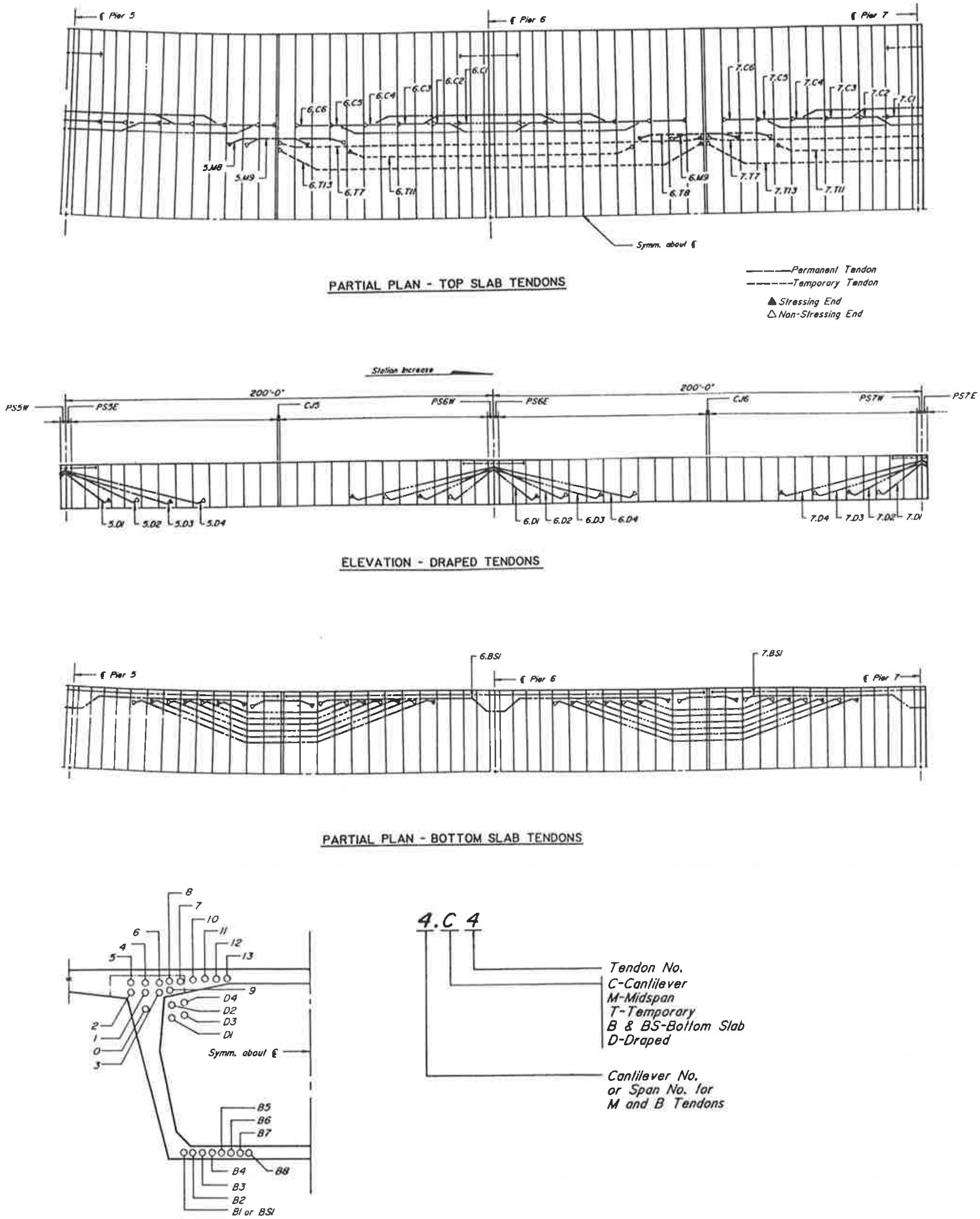


FIGURE 5 Typical post-tensioning layout.

approximately 5-1/2 inches in this area. The three tendons shown in Figure 4 outside of the web reinforcement are top slab continuity tendons located at midspan.

The area containing the 6 cantilever tendons was specially reinforced for the deck replacement condition. The hammerhead left at the top of web also provides additional transverse stiffness to the top of web when the top slab is removed. The area above the web is typically where the cantilever tendon achorages are located. Since the tendons are located in this area, the anchorages were forced to be lowered into the web resulting in a necessary thickening of the top portion of the web. This is basically the only change to the box girder cross-section necessitated by the deck replacement scheme. A typical full box girder cross-section is shown in Figure 7 just after casting. The duct locations in the top slab outside of the hammerhead are temporary tendons used only during construction and are not a part of the permanent post-tensioning.

Figure 8 shows a plan view of a typical pier cap and the placement of the jacks for lifting the span. The pier cap and the superstructure were specially reinforced for the relocated bearing reactions while jacking. However, the jack locations shown are also used for lifting the bridge approximately 1/2 inch to replace the pot bearing assemblies, if necessary.

Deck Replacement Analysis

The structural changes occurring within the bridge during the deck replacement procedure are complex due to the change in stiffness, the change in dead load, the change in the internal post-tensioning moments and the resulting change in moments due to redistribution. The analysis required to accurately and completely include all of the above behavior was beyond the scope of the work requested by the owner. Therefore, the analysis completed at this time was preliminary in nature and intended only to verify that the scheme was feasible and accounted for in the structure design. Should deck replacement ever be required, a more complete and exact analysis would be necessary at that time. Therefore, the level and methods of analysis used were determined by the following primary objectives:

- The analysis and information shown in the contract documents would be only for a typical, interior 200 foot span.
- The bridge must be stable during all phases of replacement.
- The superstructure stresses must be within allowable limits during deck replacement and upon return to service.
- Local stresses in the transition area at each end of the section to be replaced must not be critical.

The global longitudinal analysis was completed by using the same two-dimensional, plane frame analysis computer program used to design the superstructure. The program is capable of time-dependent analysis of prestressed concrete; however, that feature was not used for the deck replacement analysis. It was assumed that all time-dependent behavior of the initial structure would be complete at the time of replacement and that the new concrete deck is not stressed high enough to cause a significant redistribution of moments after replacement. This assumption and the effects of shrinkage of the new deck should be investigated further in the analysis done at the time of replacement.

The prediction of forces during and after deck replacement was made using the following assumptions:

- The effect of all external loads, including jacking loads, counterweighting, removal of and replacement of top slab, and top slab post-tensioning bars, were evaluated by calculating the change in moments and shears by superposition, using the stiffness of the bridge at the time the load is applied. This stiffness changes as various portions of the top slab are removed and replaced.
- The redistribution of internal loads was calculated by assuming the change in forces to be the difference between the forces in the bridge with the different stiffnesses, both subjected to the same loading. To accurately predict the redistribution, an iterative program would be needed. This procedure would be similar to that used in non-linear finite element codes used to predict redistribution due to material yielding.

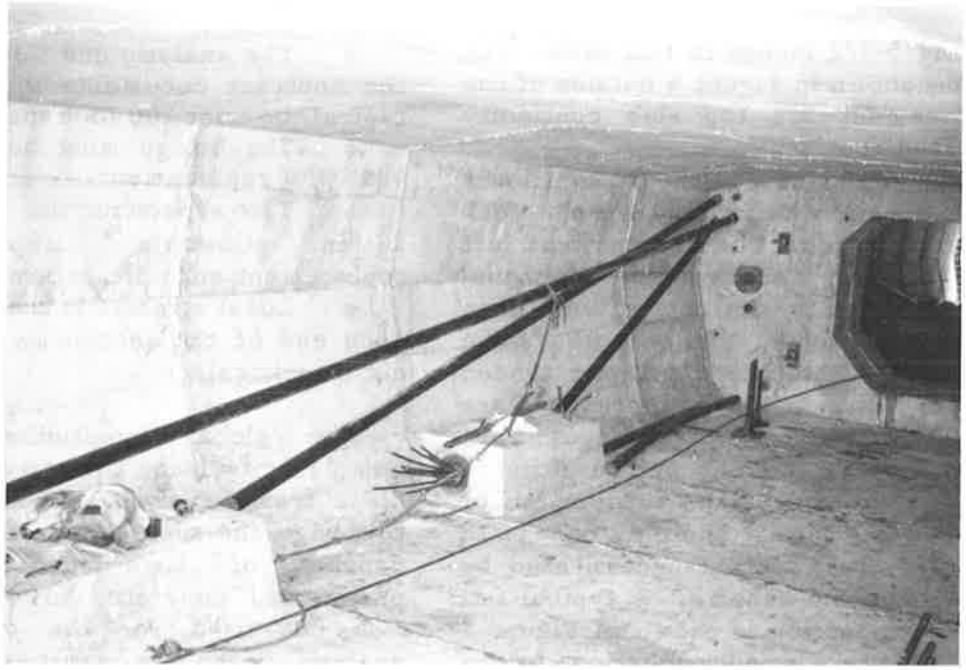


FIGURE 6 Draped external post-tensioning tendons.

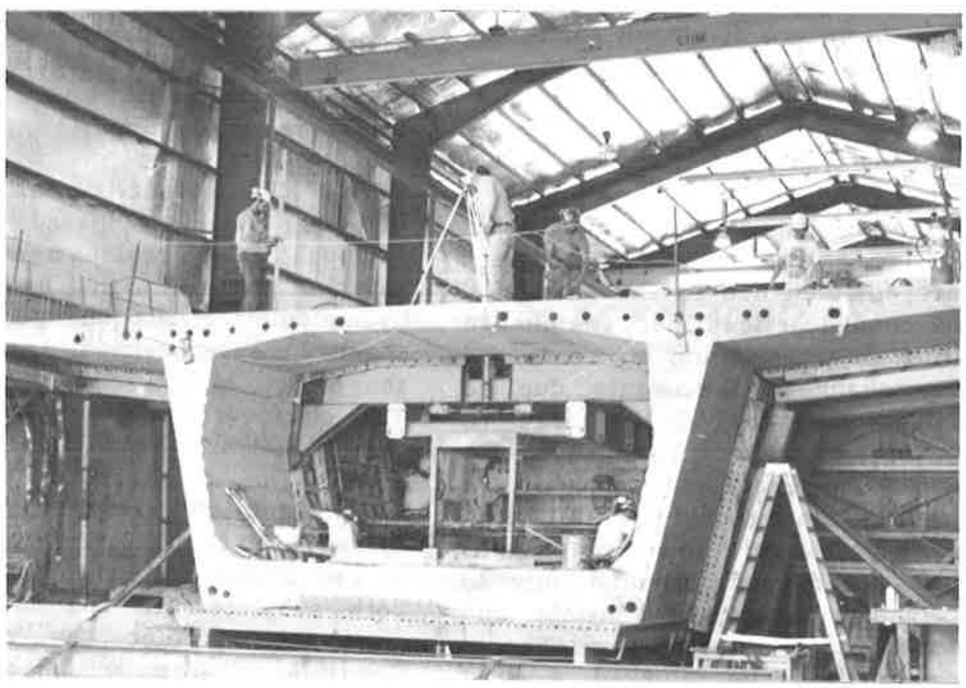
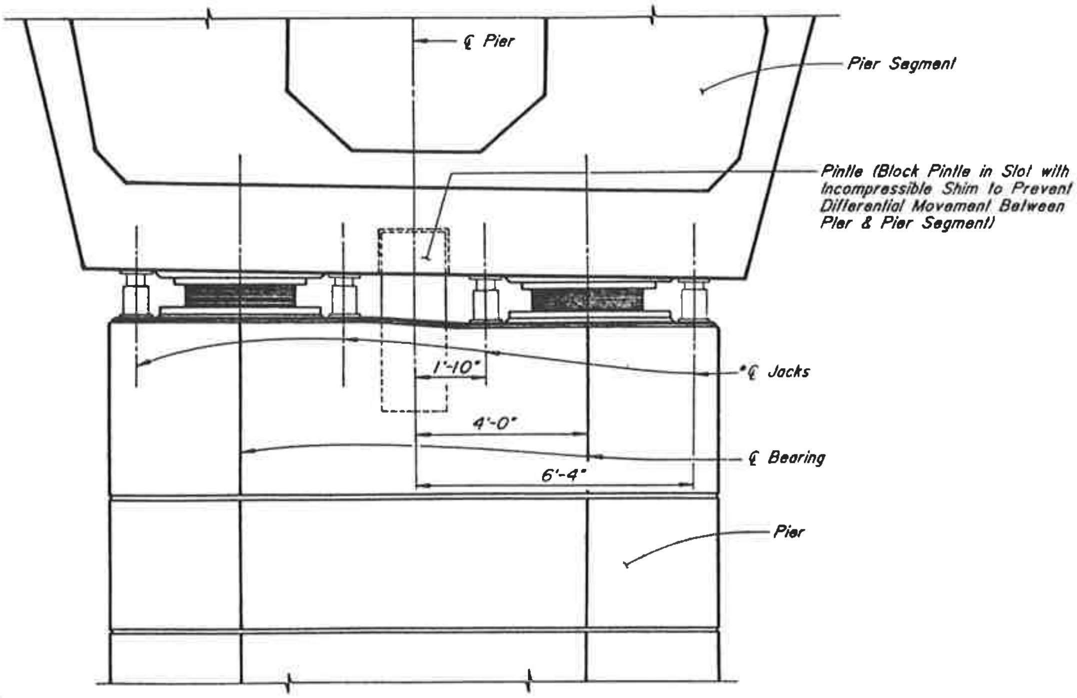
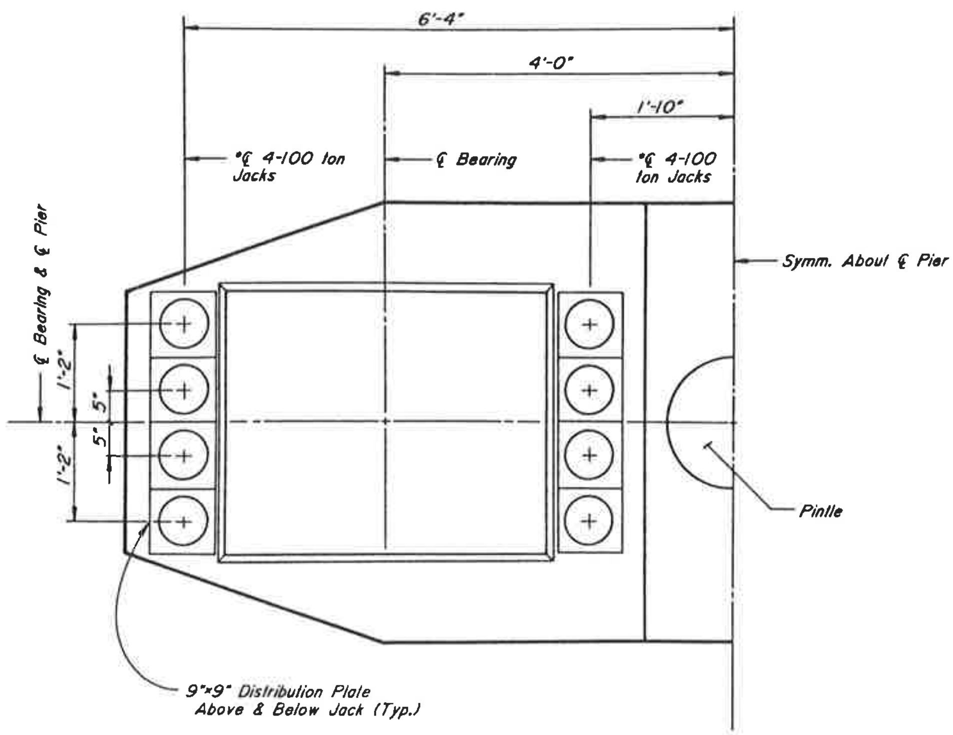


FIGURE 7 Typical box girder section



TRANSVERSE ELEVATION



HALF PLAN

FIGURE 8 Jack placement for lifting structure.

To calculate the completed end result of replacing the entire span in sections, the individual changes in forces for each step of each section are simply summed. This results in new moments and shears at each segment joint in span. The stresses must also be calculated in a similar manner by using the flexural stress equation Mc/I at each step of the process. As an example of the process involved, the upper portion of Figure 9 shows the moment redistribution in the structure due only to a reduction in stiffness at the center of a span with the deck removed. The increase in negative movement at the piers is due primarily to the span acting more like two cantilevers from the midspan flexural stiffness reduction. During the phase of replacing the deck near the pier the opposite effect is seen, as the span acts more as though it is simply supported at the pier and an increase in positive moment near midspan occurs. The increase in positive moment at midspan for the case of the deck removed at midspan, is due to the change in location of the neutral axis and thus a change in the post-tensioning moment. The lower portion of Figure 9 illustrates the superposition principle used to calculate the changes in stresses. Note the discontinuity which occurs at the bottom of top slab after it has been replaced.

The above step-wise superposition process must be done for each change in stiffness and load for each section replaced. The final result is then the summation of all the steps. The results of the analysis are shown in Figure 10. The structure begins with the top slab stresses in the segments at Day 5000 (time assumed at which long-term effects are finished), which are approximately 75 ksf across the span. The structure is raised 5 inches at each pier and the stresses reduce to approximately 35 ksf. The next three graphs show the stresses after removing the post-tensioning bars (c), then the slab itself from the center section (d), and then with the slab replacd in the center section (e). Finally the last (f) graph shows the final stresses in the top and bottom slabs after all sections in the span have been replaced. Note the elimination of the "spikes" in the top slab stresses due to the cantilever post-tensioning anchorages. The average stress in the top slab is approximately the same as it was prior to the replacement. However, there is a net redistribution of the positive moment in the span after all sections of the deck have replaced in the span (Figure 10(g)).

This causes a slight decrease in the bottom slab compression and may require stressing of the contingency tendons within the span. The necessity of stressing these tendons would depend on the stress results obtained in the more accurate analysis done at the time of replacement.

The shear stresses were also checked by using the analysis described above. The final results indicated the following:

- Shear stresses are not critical during the replacement, although no large equipment is allowed on the span during replacement.
- The service shear stresses increase by approximately 5% due to redistribution.
- The service shear capacity increases by 15% due to the redistribution of axial load to the unreplaced concrete (which reduces the principal tensile stress).

A three-dimensional finite element analysis was used to check local stresses a the transition areas and to evaluate torsional stability. The accuracy of the finite element model was checked by comparing the global longitudinal moments with those predicted by the two-dimensional analysis. The correlation between the two analyses was good.

The local stresses at the transition areas were not found to be critical. Stress concentrations were found to be higher at post-tensioning anchorages than at the transition section.

An estimation of the torsional redistribution in horizontally curved spans was made using a procedure analagous to that used in the longitudinal bending analysis. It is found that an increase in torsion in the section with the deck removed can occur. This is counter-intuitive, since the torsional stiffness in these sections decreased dramatically. The reasons for this redistribution are:

1. Torsion alone, does redistribute such that the section with the deck removed carries less torque.
2. However, redistribution of the longitudinal bending moments can increase the net torque due to the component of bending moment "transferred" (due to the curvature) to a torsional moment.

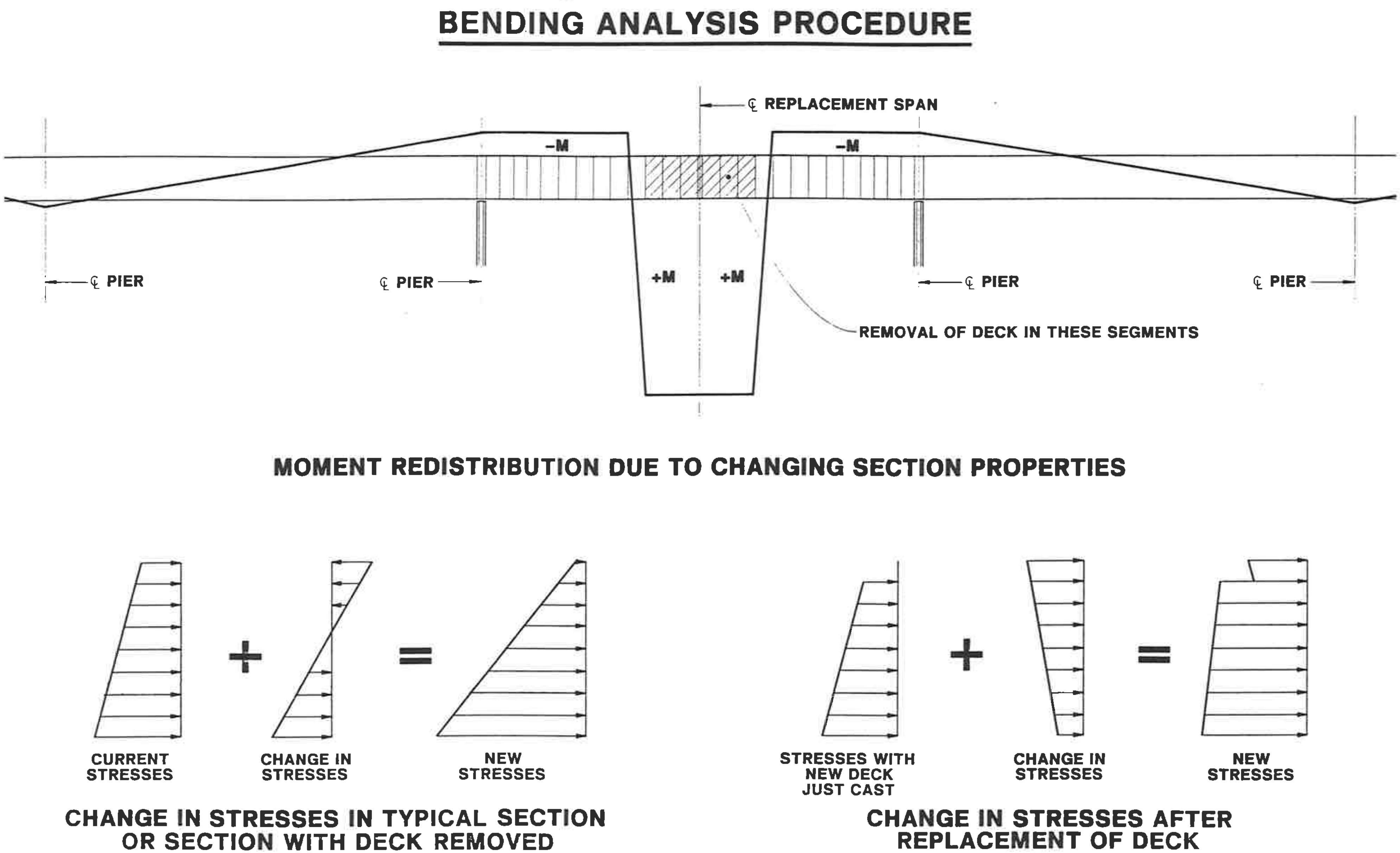


FIGURE 9 Longitudinal bending analysis procedure.

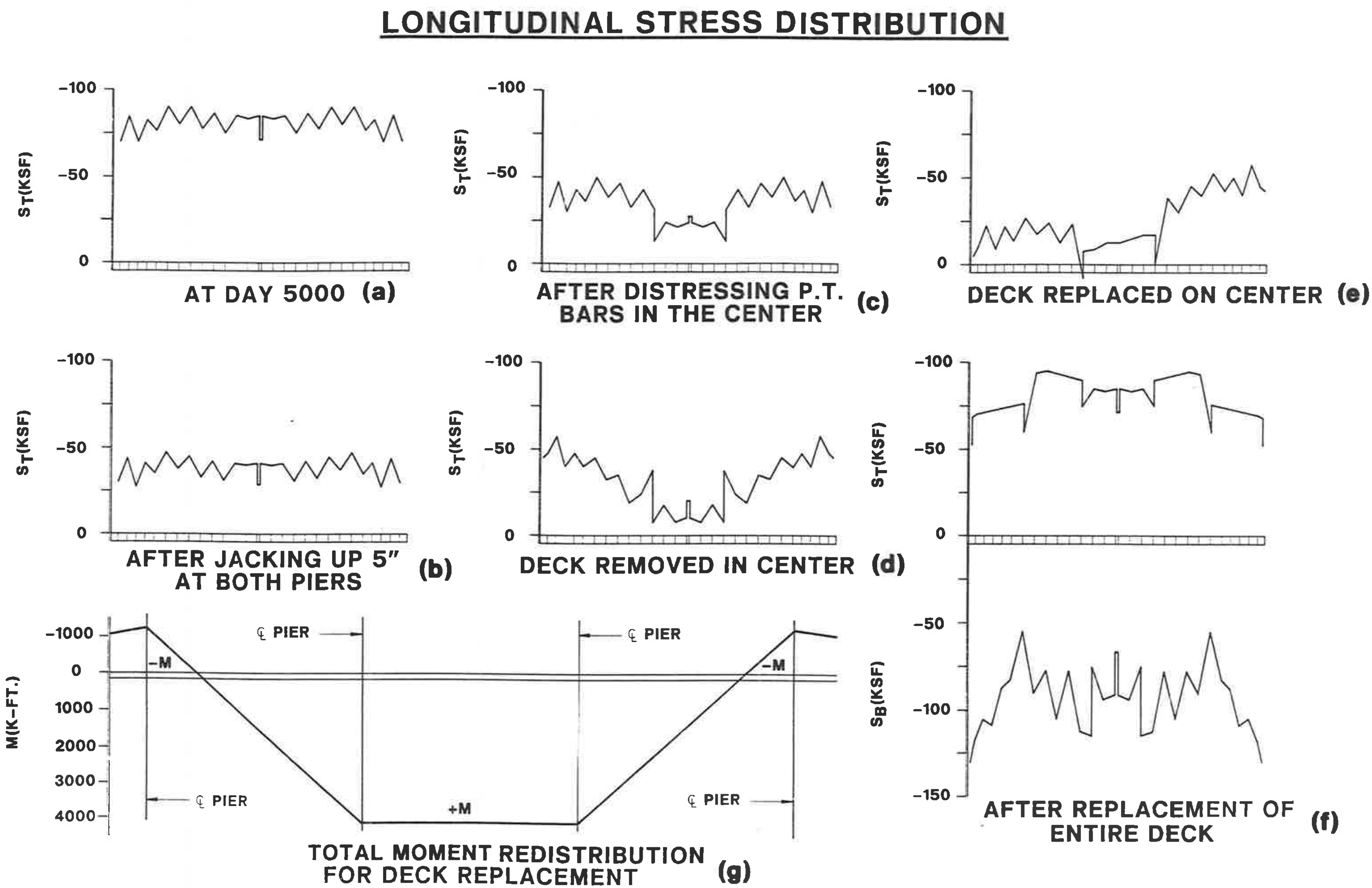


FIGURE 10 Longitudinal bending analysis results.

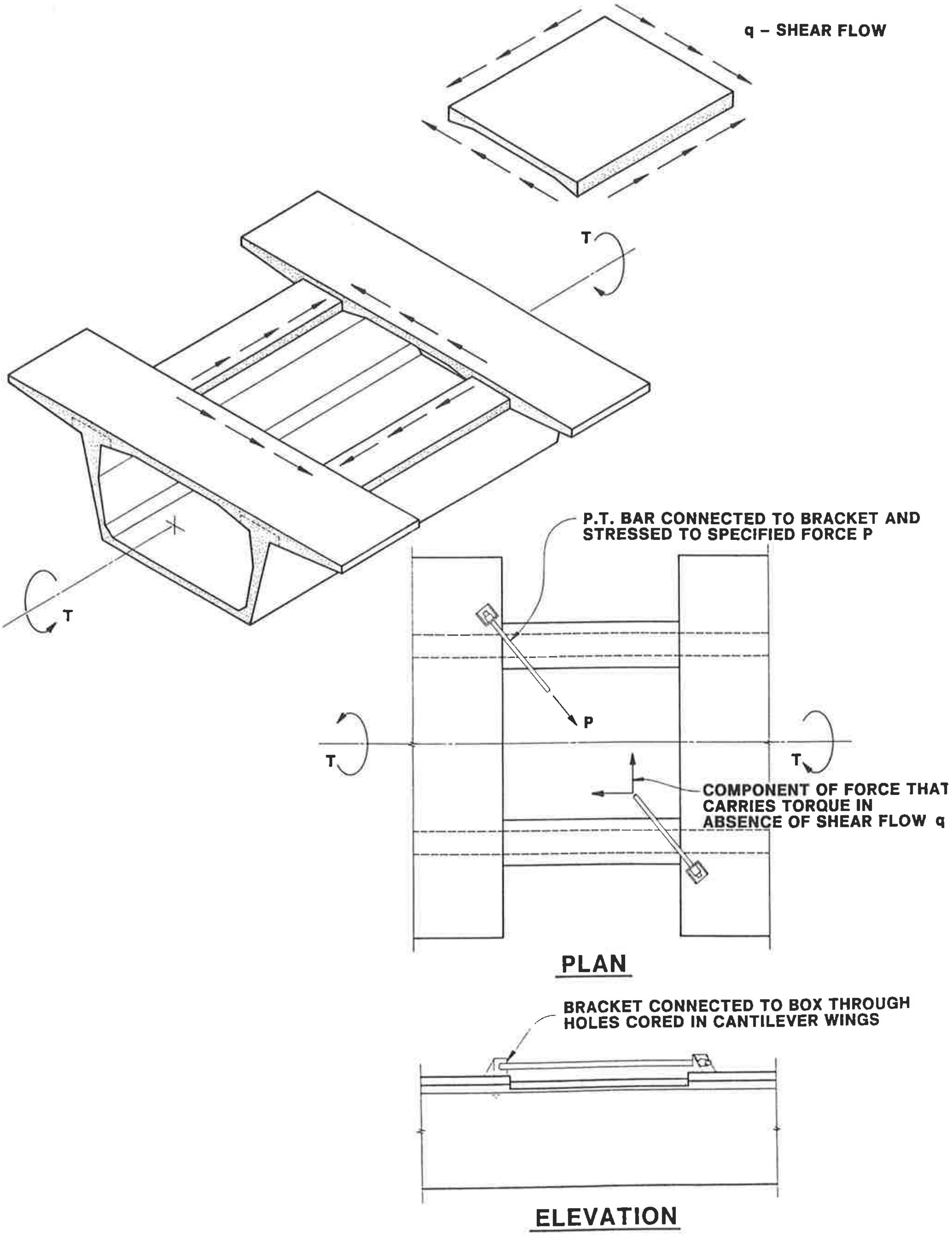


FIGURE 11 Temporary tension tie.

To minimize the effects of the torsional redistribution in the span, the following recommendations were made:

- For straight spans, proceed with the deck replacement procedure as presented.
- For curved spans with radii greater than 1900 ft, limit deck replacement/removal to two segments (16 ft.) at a time. This allows for the development of a diagonal compression strut in the webs.
- For curved spans with radii less than 1990 ft, the stability of each individual span should be checked. A "tension tie" to prevent torsional distress could be used in high curvature spans.

A schematic of the tension tie is illustrated in Figure 11. As the top slab of the box is removed, the section is not able to transfer the torsion through shear flow in the top slab. Therefore, the tension tie, which is simply a post-tensioning bar temporarily anchored to the top slab, is used to carry that shear flow across the gap and warping while the top slab is removed. Again, this simply allows for a simple solution to the problem if it occurs. The final analysis may show that the tie is not needed, or another solution may be developed.

CONCLUSIONS

Although it has been shown that deck deterioration has not yet been a problem in prestressed concrete segmental decks, a simple and feasible deck replacement procedure has been developed for bridges built in balanced cantilever. The procedure can be performed using typical construction practices such as jacking the structure at the piers, concrete sawing and jackhammering. Perhaps the most important feature is that there is relatively little additional cost for necessary details in the initial construction, while still providing the owner with the possibility of complete deck replacement in the future. In summary, it has been shown by this project that prestressed concrete segmental bridges cannot only provide economical solutions to today's bridge requirements, but also to those of the future.

REFERENCES

1. R.W. Poston, R.L. Carrasquillo and J.E. Breen, "durability of Post-Tensioned Bridge Decks", ACI Materials Journal, July-August 1987, pages 315-326.