Causes and Prevention of Problems in Large-Scale Prestressed Concrete Construction

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As each new construction technology is applied extensively, we gain the confidence and capability to exploit its potential to ever more sophisticated structures. We also build up a storehouse of problems: difficulties and local failures. In most or even all cases, we eventually repair and reconstruct the facility; thereby successfully completing the project, although not without worry, delay, and cost.

We normally treat these two types of experiences quite differently. The first, the successes and the possibilities, we boast of; proud of our achievement, and eager to enthuse and to generate new and bolder projects.

The second type, the problems, we hide, dismissing unpleasant events, blaming others, getting caught up in the maelstrom of legal involvements, and, above all, trying to avoid on-going liability.

Difficulties which are not openly and frankly faced seem destined to repeat themselves on future projects.

Thus has run the story of prestressing in general and post-tensioning in particular: great successes, exciting potential, yet beset with numerous cases of local failures and difficulties, which were generally resolved and repaired to enable successful completion of the

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project. The learning experience gained at great cost can only be transferred to our widely spread profession if we are willing to record and study problems and their solutions and to present them frankly and objectively, so that all may benefit from the dispersed experience.

In this paper, examples will be drawn from several recent bridges, including the I-205 Columbia River Bridge, nuclear reactor containment vessels, and concrete sea structures, especially the Ninian Central Platform. In all cases, the problems and local failures were overcome, and the projects were successfully completed. The cooperation of the contractors and the engineers in carrying out the repairs and reconstruction promptly and effectively was instrumental in preventing their development into more serious problems.

It must be emphasized here that the opinions presented in this paper are solely those of the author and not of the designer, contractor, or owner. Further, descriptions and assignment of causes have purposely been simplified, since in practice several causes and effects are often overlapped.

Synopsis

The application of prestressing to large and complex concrete structures such as long-span bridges, nuclear containments, and offshore platforms has enormously enlarged our experience in practical construction. Major projects, although completed successfully, have frequently encountered problems in the application of the prestressing. Many of these problems repeat themselves in projects of different types, locations, and systems. It is believed that a frank and open discussion of these projects will benefit both the designer and constructor.

The specific problems described in this paper were all corrected during construction, the projects have been successfully completed, and in-service performance has been extremely satisfactory. The repair methods utilized included epoxy injection, stitch bolting, supplemental post-tensioning and reconstruction of the damaged area.

On future projects, appropriate specifications, design details and construction quality control can prevent their recurrence. Behind the immediate causes lie more fundamental causes, inherent in our present contracting procedures, which deserve reconsideration.

PROBLEMS AND SOLUTIONS

The main and anchor arm spans of the I-205 Columbia River Bridge (Fig. 1) are among the longest in North America, 480-600-480 ft (146-183-146 m). The contractor elected to construct the spans by the cast-in-place segmental method. He used two pairs of overhead form travelers to form and support segments until they were temporarily post-tensioned back to the previously completed sections.

The cycle time for partially collapsing the form, moving the traveler ahead, anchoring the traveler down to inserts in the deck, expanding inner forms, placing ducts and reinforcing steel, closing outer forms, concreting, curing, stressing the segments longitudinally, and partially stressing the transverse tendons in the deck ran about 7 days; although at later stages, with a shallower section, fewer ducts and more experience, the cycles were shortened to 4 days.

Concrete achieved a strength of 2500 psi (17.2 MPa) in 1 day, sufficient for 50

Fig. 1. Main span of I-205 Columbia River Bridge.

percent of the permanent stressing, and 3500 psi (24.1 MPa) in 2 days, sufficient for full stressing. Results of field cured concrete cylinder breaks were confirmed by the Swiss hammer test.

Vertical and horizontal alignment were maintained within relatively close tolerances. The profile was adjusted for camber and allowances were made for creep. Closure pours at midspan were made after a delay of several months, so as to enable creep deformation to occur.

The extended arms of each cantilever were strutted and tied (including diagonal ties), so as to prevent relative movement while the concrete of the closure was placed. Ten percent of the lower slab continuity tendons were stressed after final set.

Concreting was carried out in the evening, so cantilevers were tending to rise. Before full sun the next morning, the struts at the bottom were cut and 30 percent more of the lower continuity tendons were stressed.

Freezing of Water in Ducts

In the contractor's value engineering redesign, the shear near the piers was to be resisted by vertical prestressing, installed in U ducts. The first winter, only a few months after the main span construction had commenced, a period of heavy rains was followed by a sudden freeze. Some of the ducts had not been tightly sealed at deck level; as a result the ducts filled up with water which subsequently froze, causing multiple through-web cracks in the webs and, more seriously, laminar cracking in the bottom flange at the bottom of the U's [Figs. 2 and 8(1)].

Repairs to the webs were carried out by epoxy injection. Since there was ample reinforcement, both horizontal and vertical, in the webs, no problems were encountered in achieving full restoration (Fig. 3).

In the bottom slab, adjacent to the web, the 12 in. (305 mm) thick flange had laminated horizontally, usually

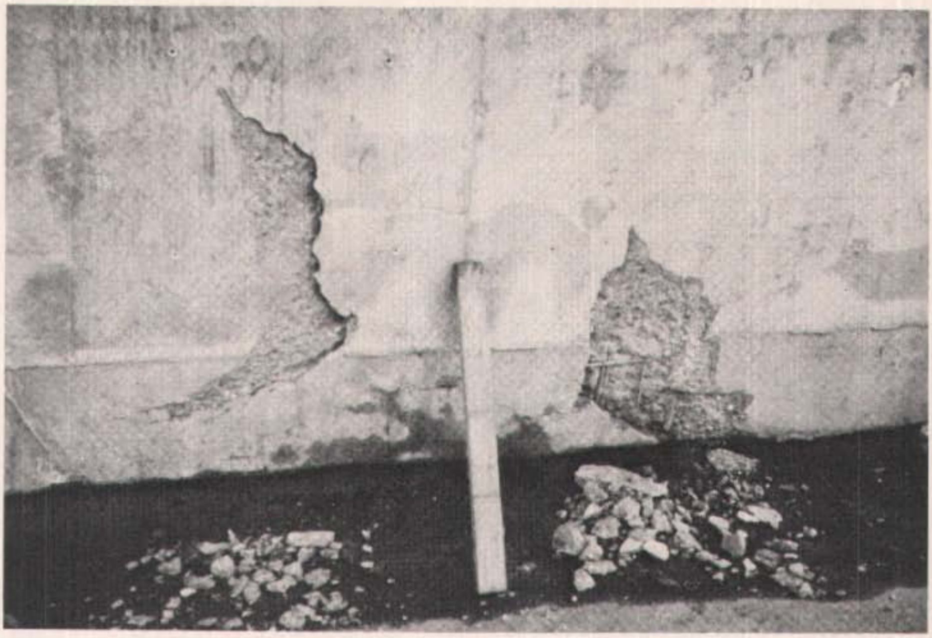


Fig. 2. Water freezing in ducts.

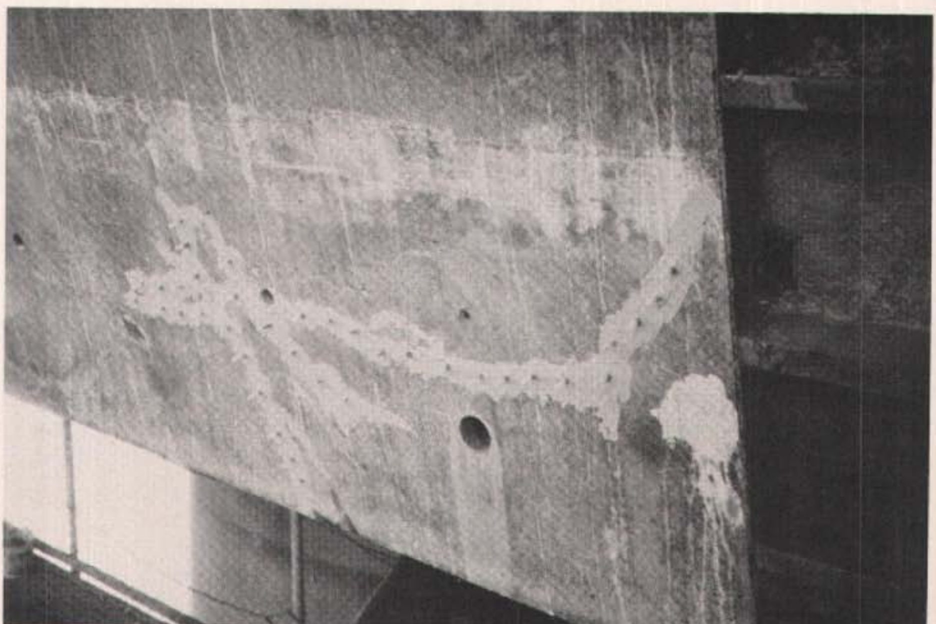


Fig. 3. Epoxy injection of web cracks.

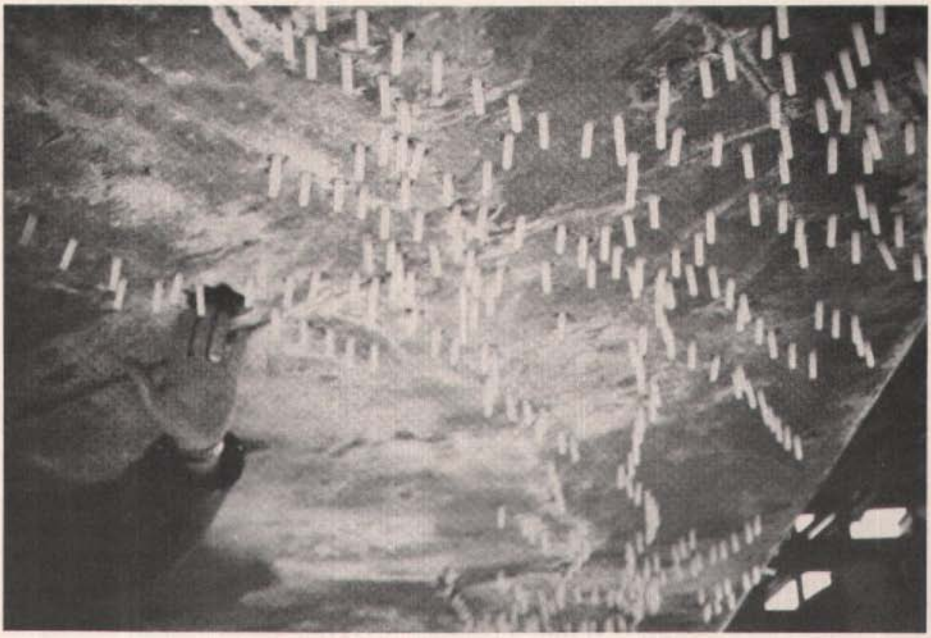


Fig. 4. Epoxy injection of bottom flange.

about 4 in. (102 mm) above the soffit. The extent of the area affected was determined by hammer sound, corroborated by the drilling of cores. These cracks were in-plane laminations, not crossed by steel.

Some temporary stitch bolting was installed by drilling through the bottom slab. Epoxy injection was then carried out in small stages, working inward from the edges, and using low pressure so as to avoid hydraulic fracturing and crack propagation (Fig. 4). Test cores and hammer testing indicated full homogeneity of the restored section.

Laminar Spalling in Deck

Minor laminar spalling was visually noticed in the underside of the top flange (deck slab) during and after post-tensioning of the negative moment tendons. Acoustic checking (hammer) revealed other and much larger areas of delamination.

The causes appeared to be several and were interactive.

1. The deck slab itself was effectively divided into upper and lower portions by the very closely spaced longitudinal ducts in the deck areas adjacent to the pier. Although ducts were designed to be spaced apart, in many areas, especially where they were horizontally curved, they touched. The area of concrete on horizontal planes near midsection was less than half the deck area.

2. The close spacing of ducts, combined with transverse ducts, and longitudinal and transverse reinforcing steel in the top and bottom, made it very difficult to effectively place and consolidate concrete around and under the ducts and reinforcement. Apparently concrete did not always work its way up around the ducts [Figs. 5, 8(2)].

3. Concreting practices were not conducive to full concrete integrity. Concrete crews often stood or walked in the fresh concrete, which in turn pushed the ducts and reinforcing bars down, only to spring back up as the

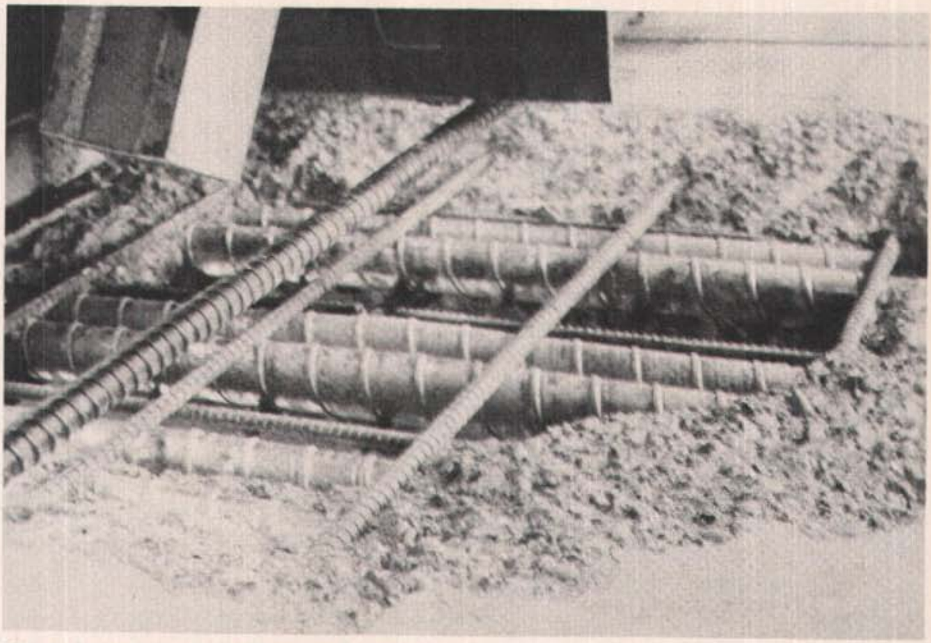


Fig. 5. Longitudinal ducts spaced too closely.

men moved. This may have caused a laminar separation in the concrete. Vibration apparently did not always extend deeply enough to knit top and bottom layers of concrete together and to force concrete up between closely-spaced ducts.

4. Bleed water collected under the flat transverse ducts and also in the recess or V formed by two touching longitudinal ducts.

5. Longitudinal ducts were held in exact position at the leading edge form stop. In between they were deflected downward by the weight of the concrete and men walking in the fresh concrete. This meant that there was a small but significant upstanding peak in the duct profile at each construction joint. When the tendons were later stressed, a downward component of the tendon force was imparted to the concrete in the lower portion of the slab. It appeared that this was the precipitating cause in many of the cases of laminar cracking in the deck slab [Fig. 8(3)].

6. Early grouting experience showed that considerable grout leakage was occurring. Not only was the grout plugging adjoining ducts before the tendons were placed and stressed, but grout was intruding into laminar cracks, making detection and repair more difficult.

The practice was therefore adopted of stressing two or more closely adjoining tendons before grouting any in that cluster. Unfortunately, in several cases there was a slight horizontal bend where the tendons curved out of the deck slab towards the web. Stressing of the tendon on the outer side of the curve caused the duct to crush into the adjoining duct, distorting it vertically and apparently causing the underside to spall [Figs. 6, 8(4)].

7. The cause of many of the leaks in the ducts, enabling grout to penetrate to fill adjoining ducts and to spread into existing laminar cracks, thus causing hydraulic crack propagation, appears due to inadequate sealing of the splices and to poor consolidation of the con-

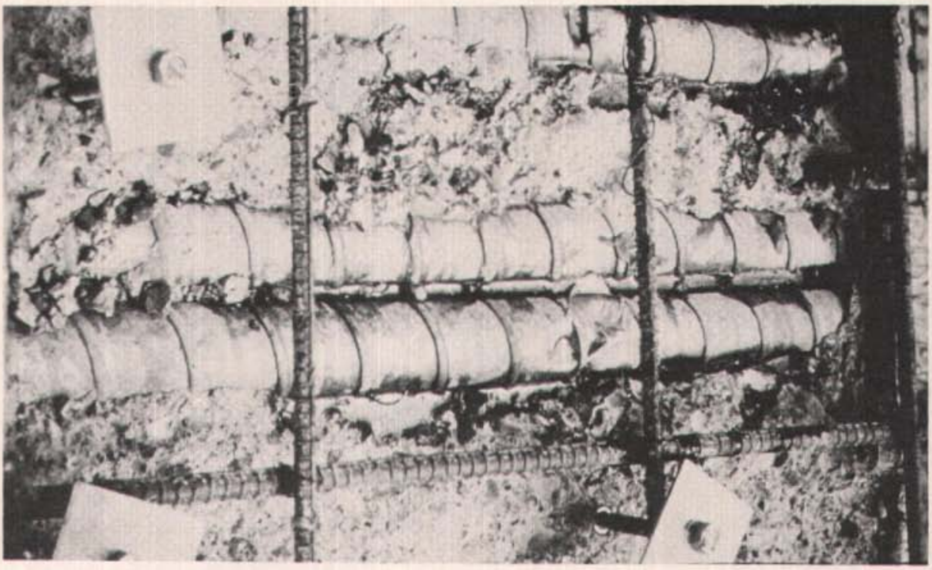


Fig. 6. Crushing of ducts during stressing.

crete around the splices, especially where they almost touched one another.

In many cases the splice sleeve fit very loosely, leaving an annular gap. Even though the joints were sealed with watertight tape, these seals apparently ruptured whenever concrete was not densely compacted around them. In some cases where sealing tape was in direct contact with the sealing tape of an adjoining duct, the grout could break directly through.

This problem of grout leakage at adjoining splices has reportedly occurred in many bridges in Europe. It appears that our present details are not adequate for cases where adjoining or touching ducts must be spliced at the same point.

In many cases, several causes interacted, so that it was difficult to assign the laminar cracking to one single factor. A crack initiated by one cause was propagated by a second cause.

When laminations were found, their extent was carefully determined by hammer testing and coring. Working from scaffolding inside the boxes, the crews drilled in and anchored stitch bolts (headed expansion bolts), usually on 2-ft (0.61 m) centers both ways. Epoxy injection was then carried out using closely-spaced injection points and keeping pressures low, generally at 20 psi (0.138 MPa).

Other methods were tried as a means of detecting laminations. They were generally unsuccessful, due to the false and confused readings imposed by the congestion of ducts and reinforcing steel. The hammer gave quite reliable results, revealing laminations by a dull sound as opposed to a sharp ring from fresh concrete. Coring proved useful in determining the locations of the cracks, i.e., the depth to the crack; and in determining the crack width.

Prevention of these problems should properly start with design: the selection of tendon sizes and duct spacing to maximize the concrete area at mid-depth (i.e., that area capable of transmitting tensile forces through the thickness of the slab). A larger number of strands per tendon or larger wires could well be employed. For example, the 19 x 0.5-in. (12.7 mm) tendons could presumably have been increased to 24 x 0.6 in. (15.4 mm), thus decreasing the number of tendons by 40 percent and increasing the in-plane (horizontal) mid-depth concrete area by 70 percent.

Through-slab stirrups or ties would be useful in preventing the propagation of laminar cracks. On the I-205 Columbia River Bridge, after initial laminar cracking problems were discovered, No. 3 vertical ties were installed on 24-in. (610 mm) centers, between pairs of closely-spaced ducts. While such ties may limit crack spread, they cannot prevent their occurrence.

Tightly-fitting splice sleeves appear to be essential. Commercial ducting is available with closely spaced ribs (threads) that enable the sleeve to be screwed on with a relatively tight fit. Such tightly fitting splice sleeves are now being used on the Statfjord C platform in Norway. Subsequent taping of both ends of the splice sleeve with heavy duty waterproof tape should then suffice to prevent grout leakage even where, due to extraneous reasons, minor voids or cracks exist. At least a 1-in. (25 mm) space between adjacent ducts should be left in order to assure adequate concreting.

Ducts themselves should be of heavy enough gauge to prevent local deformation. When thin ducts deform under bending, the seams open, permitting grout in-leakage. Friction factors are increased by deformations. Splices are rendered more difficult. Angular changes and hence local bursting stresses are more prevalent and serious with thin ducts. Although 0.6 mm (0.023 in.) gauge is widely used, experience indicates a thicker gauge, for example, 1.0 mm (0.039 in.), would be preferable.

To prevent rain water from entering vertical or inclined ducts, with subsequent freezing, covers should be provided. Preferably, these covers should be installed before the duct is installed. Plastic caps, taped on, are recommended.

Drains should be provided in the bottom of the duct profile, since caps are seldom absolutely watertight.

An alternative system is to fill the duct with a mixture of glycol antifreeze and water, so as to lower the freezing point. This was successfully employed on the Statfjord C platform.

Blowing out of ducts, using compressed air, in order to assure they are water-free, is a questionable practice. In the first place, there can be no assurance that all water has been displaced. More serious is the fact that the air, under a pressure of 110 to 115 psi (about 8 MPa) can transfer a high pressure to the water ahead of it or trapped in cracks and voids, thus leading to hydraulic fracturing and crack propagation.

It is significant to note that several of the cases of lamination were noted only during this process of blowing out; this apparently was when minor damage was transformed into serious damage. This same phenomenon has been reported from The Netherlands. The author believes that if the duct has become water-filled for any reason, it is better to use a thixotropic grout to displace it, and to continue grouting long enough after the first grout emerges from the exit vent to ensure complete displacement of the water [e.g., 3 to 4 liters (about 1 gallon) of wasted grout].

During concreting, all workmen (and inspectors) should work from bridges which span across the reinforcing steel and fresh concrete. Great pains should be taken to work the concrete under the reinforcement and to vibrate through all interfaces as concrete layers are placed. Adjoining horizontal ducts should be held apart by spacers in order to allow mortar and concrete to work up through the gap.

It has been found extremely instructive to construct a full scale mockup of a

Fig. 7. Inspecting the laminar cracks in deck.

short section, with all reinforcement and ducts, and to place the concrete; then after curing, to strip the forms and cut the concrete apart with paving breakers. The workmen then can actually see the results of their work and the need for proper placement, while troublesome details of congestion and interference can be corrected by the designer and concrete mixes can be optimized insofar as workability is concerned. Concrete mixes must be designed not only for workability and cohesiveness, i.e., lack of segregation, but also to minimize bleed and air entrapment.

Support of the extended forms in cast-in-place segmental construction requires heavy diagonal as well as vertical ties so as to minimize deflections during concreting. The soffits of slabs need to be adequately stiff to minimize deformation as concrete is placed.

Ducts should be supported during concreting so as to prevent sag. Use of an internal mandrel of relatively stiff pipe is one method that has been found successful in segmental construction. The thicker ducts and careful blocking also help.

When closely-spaced ducts have bends, which can make them susceptible to displacement into one another during stressing, the use of spacers and saddle plates is advised. The tendon on the inside of the curve must be stressed, and grouted before the one on the outside of the bend is stressed. This applies, of course, to vertical as well as horizontal curves.

In one case on the I-205 bridge (Fig. 7) the upper portion of the deck slab laminated and spalled, affecting an area 16 x 32 ft (5 x 10 m). The lamination occurred at the underside of the longitudinal ducts. Investigation revealed that the longitudinal ducts had been deflected downward to accommodate the deflected profile of the transverse ducts; the upward component of the two prestressing systems exceeded the tensile capacity of the remaining concrete areas at that level [Fig. 8(5)]. Spalling occurred during blowing out of water from the last few unstressed ducts. It is believed that this blowing out prop-

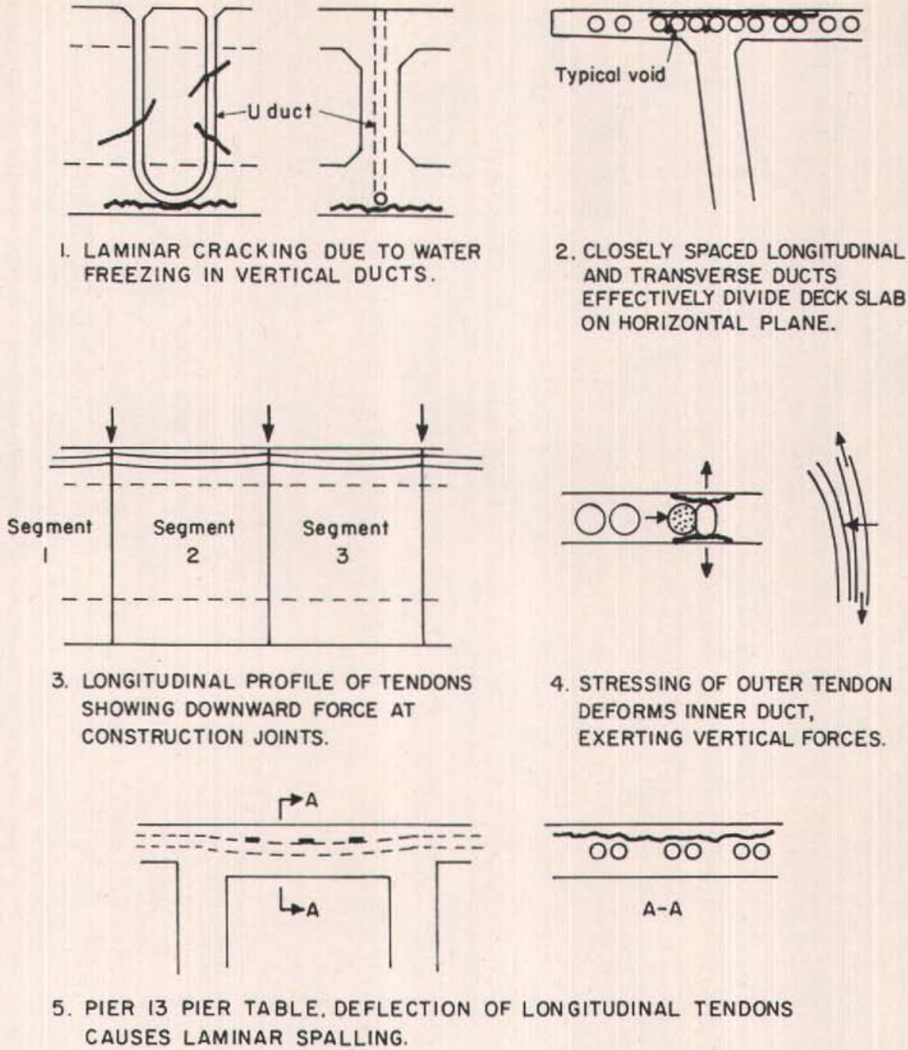


Fig. 8. Schematic representation of cracking (see also Fig. 12).

agated and extended laminar fracturing initiated by the stressing of tendons on the deflected profile.

In this case, the repairs were carried out by removing the damaged concrete, thus allowing the tendons to raise to a level profile; then installing drilled-in anchor bolts and using an epoxy bonding compound as the new concrete was placed, so as to make it act monolithically (Fig. 9).

Slip of Dead End Anchors and Spalling at Anchors

There were several cases in which the H-type dead end anchor failed during stressing, especially where there was an angle change near the anchor. In some cases, the measured elongation was 35 to 50 percent greater than calculated and severe laminar and horizontal cracking occurred at the an-

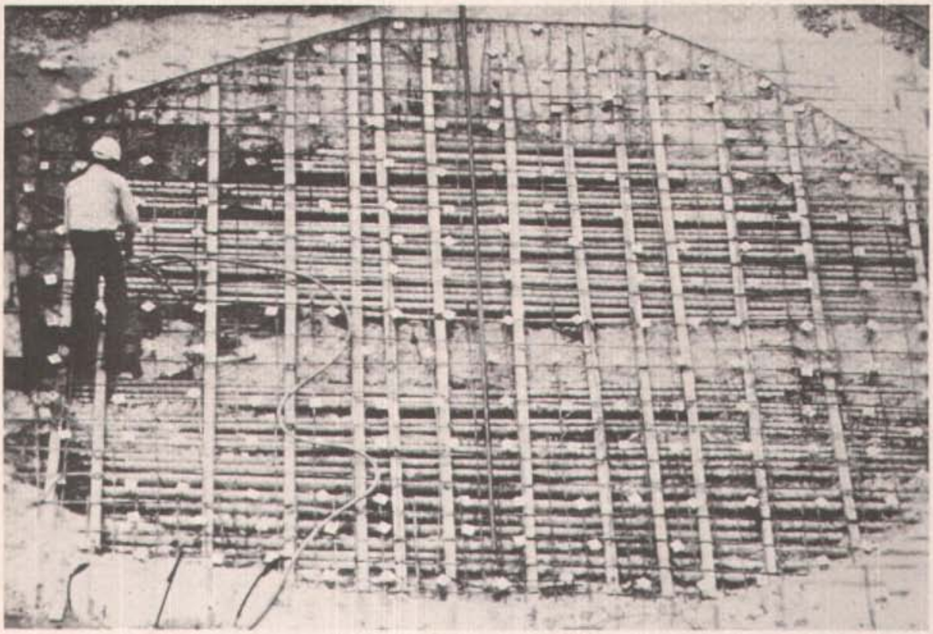


Fig. 9. Repairs to deck after laminar cracking.

chorage itself (Fig. 10). Usually after a slip of several inches, the anchor would re-establish itself mechanically.

Slippage of this type has also occurred with pretensioning, as in railroad tie manufacture. One cause appears to lie with the manufacture of the strands, i.e., a long lay length reduces bond. Another cause lies with the amount and type of lubricant used in drawing the wire. If stress relieving is performed by gas, it tends to burn off contaminants whereas if performed in electric furnaces, it tends to bake the contaminants on as a glaze.

Proper confining steel around the anchorage can help to prevent such cracking and slippage. If the dead end

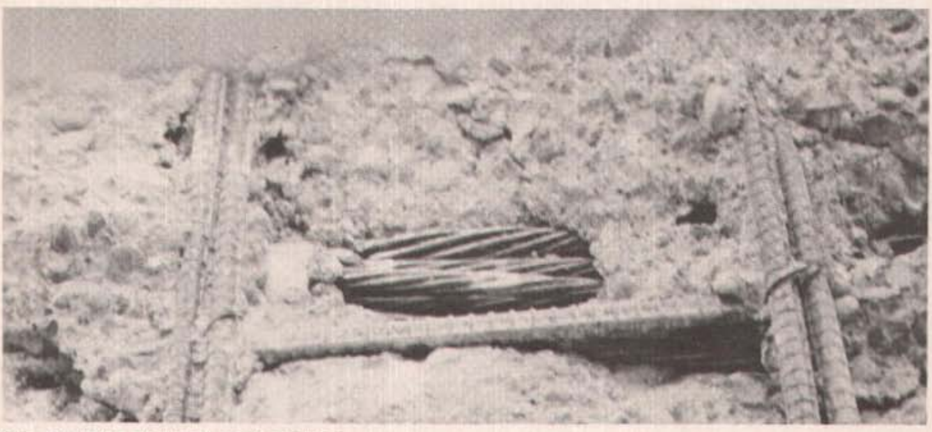


Fig. 10. Slip of dead-end anchor.

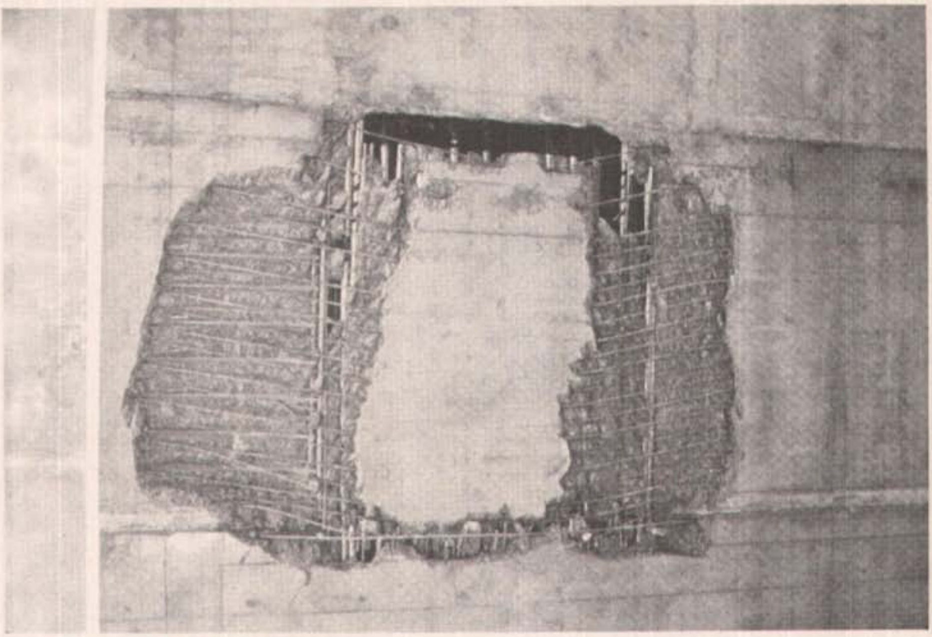


Fig. 11. Shear and tension failure at anchorage.

anchor has been allowed to sag or if there is a sharp bend near the anchor, then there is a radial force developed, which in some cases has led to crack initiation, followed by laminar (in-plane) cracking and intense local spalling. Such spalling near anchorages has led to similar spalling and crack propagation on both the Frigg and Statfjord A platforms. Local confining steel, augmented by full through-wall stirrups at the zone of curvature, is required, since the radii of curvature are usually sharp (intentionally or unintentionally) even if the degree of curvature is small.

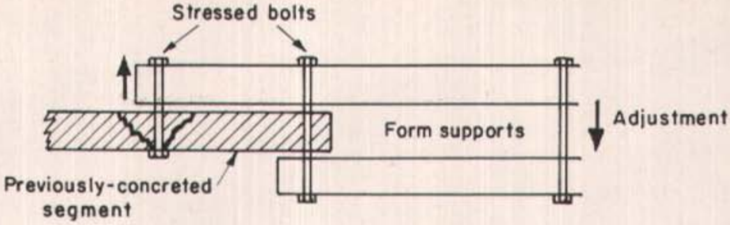
Cracking Behind Anchorages

In several cases, most seriously a bridge in Southern France, the anchorage of large forces in a concentrated zone, e.g., the anchorage of positive moment (continuity) tendons in buttresses in the bottom slab has led to serious transverse cracking behind the anchorages, that is, in the unstressed zone away from the tendon direction. Obviously, the stressed zone tends to shorten although it is restrained partially by the adjoining slab and webs.

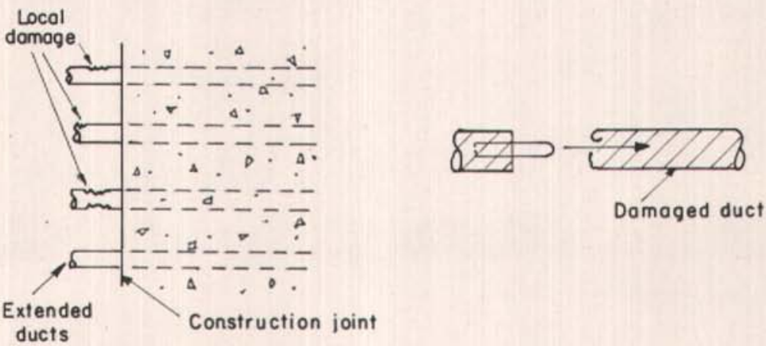
When the force exceeds the shear or tensile strain capacity, cracking occurs, which quickly propagates as the restraining force is relieved. Either a major local failure occurs around the anchor or, more seriously, cracks behind the anchor propagate transversely to the webs and up them, thus reducing the shear capacity. The design of reinforcement in this area must consider the transfer of stress. Sufficient longitudinal steel behind the anchor, and transverse steel abreast the anchor, must be provided to take the initial tensioning forces (Fig. 11).

Grouting of Vertical Ducts

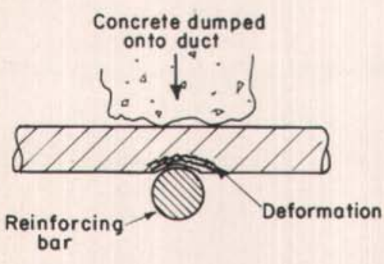
Based on extensive experience with the grouting of vertical tendons in nuclear reactor containment structures and on North Sea offshore platforms, a thixotropic admixture was used in the grout for the vertical ducts in the I-205



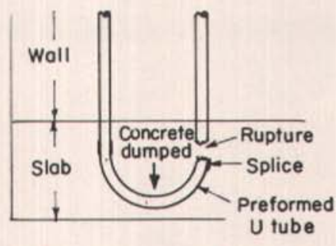
6. ADJUSTMENT OF EXTENDED FORM PLACES HIGH FORCE IN BOLTS AND ON CONCRETE, CAUSING CRACKING. SOME BOLTS ACTUALLY PULLED THROUGH SLAB



7. NINIAN, DAMAGE TO DUCTS AT CONSTRUCTION JOINT LEADS TO JAMMING OF TENDON IN DUCT.



8. DEFORMATION OF DUCTS BY REINFORCING BARS.



9. RUPTURE OF SPLICES OF VERTICAL DUCTS TO U DUCTS.

Fig. 12. Schematic representations of local failures (see also Fig. 8).

Bridge. This greatly reduced the sedimentation and bleed but did not entirely prevent a small void under the anchorage.

Secondary grouting was employed but even then a tiny void was often found below the vent, even though the vent itself was filled with grout. It appears that the joint application of thixotropic admixtures, standpipe, and secondary grouting, as described in the FIP Report on Grouting of Vertical Tendons, published in 1978, is necessary to ensure full grout encasement.

Form Support

As forms are moved forward for construction of a new segment in cantilevered segmental construction of long-span bridges, they are supported at the leading end by the overhanging form traveler and at the rear by anchorage to the previously-constructed segment. They are usually anchored down by prestressing rods to the previously-constructed segment. Any subsequent adjustment of profile places very high stresses in these anchor bars, which in turn, cause high local stresses in the concrete where they are anchored [Fig. 12(6)].

On the I-205 Bridge, in several cases, this led to concrete spalling and cracking and, in extreme cases, to pull-through of the bars.

The details of these supports should perhaps be modified. Perhaps a neoprene pad at the compression edge would permit a small amount of rotation. Alternatively, the leading end must be capable of readjustment of profile without placing additional stresses in the support. If the profile were adjusted before the rearward bars are stressed, then this would also minimize the problem.

The leading ends must also be properly held up to prevent serious deflection as concrete is placed. Obviously concrete should be placed first at the far end, working back to the joint. In one German bridge, it is reported that diagonal rods stretched during concreting of the deck slab, allowing cracks to form along the web joint, thus reducing shear transfer. While epoxy injection proved to be a satisfactory repair, design of the form supports should consider deformation under each stage of concrete placement.

Couplers

The plinth wall of the base raft of another North Sea platform required a large number of vertical prestressing bars, in order to provide proper shear resistance. These bars were placed in two sections, the cast-in-place concrete proceeding vertically in two "segments." The bars were coupled with a standard threaded coupler, carefully screwed onto the first bar for a measured distance.

When the second bar was placed, the coupler was almost inaccessible and hidden by the congestion of forms, reinforcing steel, and ducts. The second bars were screwed into the couplers and the upper segment cast. During stressing, many bars stripped the threads of the couplers. Investigation showed that they had engaged only a portion of the threads.

Tests show that whether vertical or horizontal bars are involved, the act of screwing in the second bar tends to rotate the coupler on the first bar, screwing it further on, instead of engaging the coupler. This is because the friction of the second bar is higher, due to inevitable minor misalignment.

Fig. 13. Ninian central platform.

After trying lock nuts and wire jamming unsuccessfully, the practice was adopted of coating the first bar with epoxy, so that it would bond to the coupler. This worked quite satisfactorily as a job-site remedy. It is believed that future coupler designs could be modified to provide a positive stop at midpoint.

This same problem occurred years earlier with an anchored retaining wall, but the cause at that time was incorrectly assigned to careless workmanship.

The Ninian Central Platform constructed in Scotland, is one of the largest offshore platforms in the North Sea (Fig. 13). It has a circular base 460 ft (140 m) in diameter and the concrete portion is over 500 ft (152 m) high. Its base consists of seven concentric walls (Fig. 14), joined by radial shear walls (Fig. 15). Both horizontal and vertical prestressing was employed. The horizontal tendons radiated out to the exterior walls, and then returned on an adjoining radial, whereas the vertical tendons had a U bend at the base.

Duct Blockages

In the Ninian Central Platform a number of troublesome duct blockages occurred. These prevented insertion of the tendons as planned. The causes were several.

At vertical construction joints, the protruding horizontal ducts formed a very convenient ladder for workmen to climb, resulting in local crushing at the splice [Fig. 12 (7)].

The ends of ducts were often cut by a burning torch, resulting in ragged and burred ends.

A double-female splice sleeve was used, which meant that as strands were pulled and pushed in, their nose often caught on the inner burred and deformed end. Continued efforts to insert the tendons just tore up the thin-walled duct and eventually jammed it tight.

Horizontal bends were made with flexible ducting in order to work around interfering reinforcement. Although the overall radius of the bend was large, local small but sharp bends were made, especially at their juncture with the heavier-walled straight duct runs.

Flexible ducting was deformed and locally crushed as concrete was dumped onto it. The ducts were occasionally forced down onto crossing reinforcing bar supports, denting the duct [Fig. 12(8)].

With vertical U tendons, the dumping of concrete onto the U tore it loose from the vertical duct legs, allowing mortar to fill the U [Fig. 12(9)].

As vertical ducts were extended upward, many were temporarily covered only by rags or left uncovered. As a result, the 165 to 328 ft (50 to 100 m) long vertical ducts were often blocked by such foreign objects as coca cola bottles, screw drivers, milk cartons, gravel, waste concrete, etc. Similar careless blockage has been reported from other North Sea platforms.

Attempts to unplug ducts by high-pressure jets, rotating flexible drills, etc. were only partially successful and never worked at U bends. In some cases, it was necessary to chisel in from the side of the concrete wall to free a plug. In other cases, rock anchors were used as a dead end anchor. In a few cases, the tendon in question had to be abandoned. Fortunately, in most cases, it proved feasible to install larger tendons than planned, i.e., more strands per tendon, in the adjoining ducts.

Preventive steps initiated on this project included the following:

1. Use of heavier-walled duct wherever feasible, preforming bends as necessary.

2. Cutting ends of ducts with a hacksaw, not a burning torch.

3. Changing splice details so that as tendons were entered, the nose always travelled from a male into a female end, so that it could not catch.

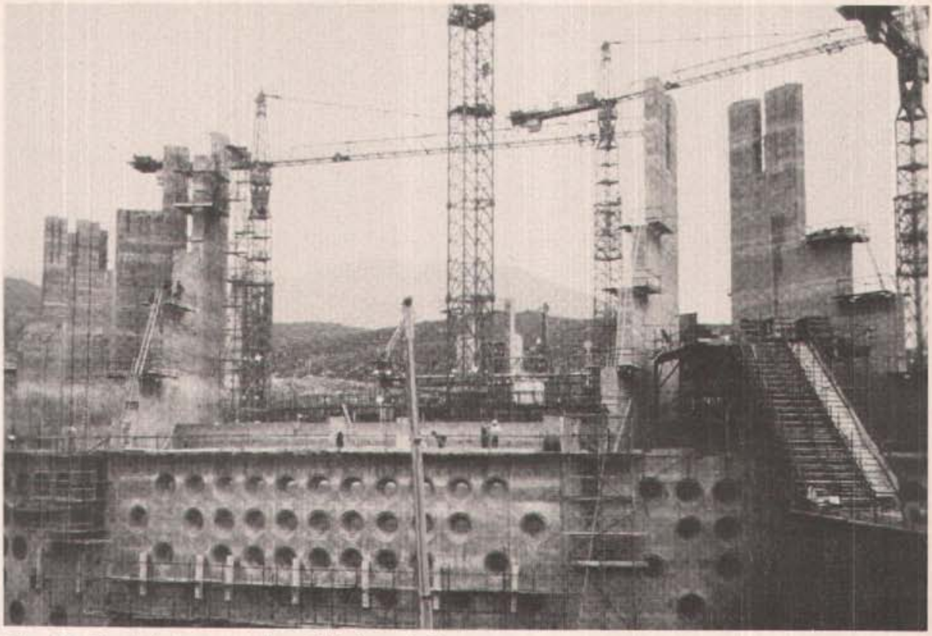


Fig. 14. Radial shear walls of Ninian platform.

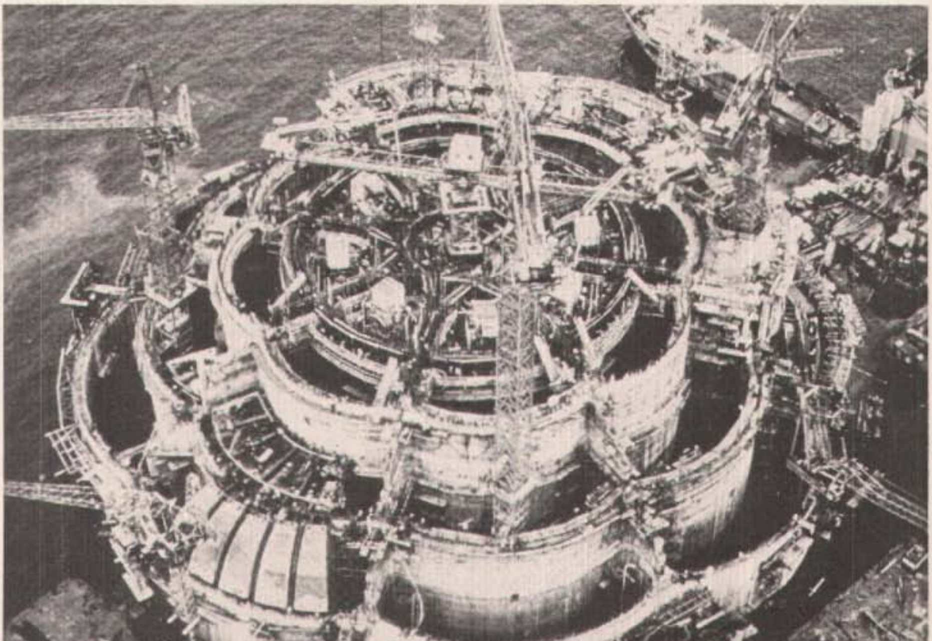


Fig. 15. Concentric walls of Ninian platform.

4. Electrical conduit or thin-walled pipe mandrels were placed inside flexible ducts during concreting, then withdrawn after initial set.

5. Colored plastic caps were taped onto all vertical duct segments before being placed, and were left affixed until the next duct length was installed.

6. Where flexible duct had to be used, it was supported by heavy sheet metal "saddles" wherever it bore on crossing reinforcing.

7. In the case of U ducts, a hose was placed inside during concreting, withdrawn after initial set.

On both the Ninian and the other offshore platforms referred to, full integrity of the structure was accomplished by these repairs and supplemental steps, but the costs in delays and extra work were substantial.

CONCLUSIONS

Problems such as those enumerated above are not unique to the particular projects described, but appear to be endemic. Hence, a full discussion of them seems appropriate in order to eliminate or minimize their occurrence on future projects. The problems can be prevented in most cases by proper detailing, proper specifications and careful quality control in the field. However, since the various post-tensioning systems differ in details and each project has its own special requirements and difficulties, it may not be possible to write general rules that will adequately cover all problem areas. In such cases it is necessary to revert to basic principles, in which the effects of concentrated force, the strains and distortions, bi- and tri-axial effects, crack propagation and its antidote, a crack-arresting mechanism, are all carefully considered. Both designer and constructor need to train themselves to visualize the sequential three-dimensional, multi-material processes involved.

One fact stands out clearly: the cost of unplugging ducts or correcting laminar cracks and other problems far exceeds the cost of doing the job correctly at the start. It is true that heavier walled duct is more expensive to purchase and sometimes more expensive to install, but its total cost is minimal in comparison with the correction of problems that may occur with ducts of minimal thickness.

Repair procedures have been developed and proven in the field which ensure restoration of full structural capacity in most cases of damage. However, their implementation requires extra efforts and costs to both constructor and designer, delays in project completion, and often results in claims.

A major part of the problem appears to be contractual. The present relationships of owners, designer, general construction contractor and specialist subcontractors are of questionable application to highly sophisticated and complex work such as long-span bridges, nuclear reactor containment vessels, and ocean structures. Three or more entities are performing inter-related and overlapping items of work. The designer has selected the prestressing force and location and usually provides some of the details, but he usually does not know which system will be used. The general contractor lets one subcontract to a reinforcing steel subcontractor, and another to a prestressing subcontractor, almost always on the basis of lowest quoted price, often the result of severe competition and extensive haggling. This general contractor usually provides the supports, the forms, places the concrete, vibrates it, and places inserts.

The reinforcing steel subcontractor places reinforcing steel with only superficial appreciation of the importance of accuracy in the detailed reinforcement around anchorages and duct bends. Sometimes he places the ducts as well, with little appreciation of the importance of alignment, grout tightness, etc.

Then the prestressing subcontractor arrives. He furnishes the detailed reinforcement around the anchorages, places ducts, if not already placed, inserts tendons, stresses and grouts. He is dependent on the general contractor for hoisting and for access.

Each of the above is dependent on each other's performance to a high degree, yet each is under a severe constraint as to costs. His profit or loss depends on labor productivity and minimum material costs, and minimal delays.

The owner has a different set and scale of concerns. He is of course interested in a low final cost, but the success or failure or even the selection of concrete instead of steel, seldom if ever depends on such small components as the cost of ducting or grouting procedures, or reinforcing steel details. Quite to the contrary, his risks are those of severe delays, of claims, and in some rare cases, of a less than satisfactory final structure.

It is the author's opinion that the owner's representatives and design engineers do not always recognize the owner's true needs. In their endeavor to secure the lowest possible bid prices, they often permit too wide a range of options and leave too many aspects up to the contractor and his subcontractors. The recent history of post-tensioning, both problems and successes, would seem to indicate that the designer should give full attention to a detailed original design, should write very complete and rigid specifications, should check shop drawings meticulously, and should ensure strict quality control of field operations.

Some contractual mechanism has to be developed to ensure that the general contractor, reinforcing steel subcontractor, and prestressing steel subcontractor are all working for their common good and that of the owner and designer. The failure of any one of them seriously impacts the performance of the others. Is subcontracting of field labor really advisable, or should the three groups all be under the general contractor's control? In some projects, the subcontractor furnishes specialist personnel on a cost-plus-fee basis to the general contractor.

Perhaps consideration needs to be given to the British system of a nominated specialist subcontractor.

Finally, it is a tribute to the contractors and engineers and subcontractors on all the projects referred to above, that despite the many minor but important problems, all were completed as fully satisfactory structures. This illustrates the inherent flexibility and the wide range of solutions available with this elasto-plastic composite material called prestressed concrete.

For other discussions of similar problems and solutions, see References 1, 2, and 3. The first paper discusses examples from France and the Netherlands; the second article, cases from Germany; and the third paper, causes and prevention of problems on the I-205 Columbia River Bridge.

REFERENCES

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2. Leonhardt, F., "Prevention of Damage in Bridges," FIP Proceedings, V. 1, 1982, pp. 58-65.
3. Harwood, Alan C., "I-205 Columbia River Bridge — Design and Construction," PCI JOURNAL, V. 27, No. 2, March-April 1982, pp. 56-77.

NOTE: Discussion of this paper is invited. Please submit your discussion to PCI Headquarters by January 1, 1983.