SURPRISES INSIDE POST-TENSIONING DUCTS AND REMEDIES

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ABSTRACT

The Central Artery/Tunnel Project (CA/T) is the largest infrastructure undertaking ever attempted at a single U.S. location. It involves reconstructing Interstate 93 (I-93) by depressing it into a tunnel and subsequently removing the original elevated highway (known as the Central Artery). The new I-93 will become a tunnel in the vicinity of South Station and emerge from the tunnel near the south bank of the Charles River and rise to cross the river on a cable-stayed bridge. The second phase of the project extends the Massachusetts Turnpike (I-90) through South Boston and under Boston Harbor to Logan International Airport. This paper reports on the post-tensioning tendon investigation, its results, and actions taken to remedy future issues involving voids and the potential corrosion of tendons and their anchorages.

KEYWORDS

Central Artery/Tunnel Project, Tendons, Non-Destructive Techniques, Post-Tensioned Structures, Interchanges, Leonard P. Zakim Bunker Hill Bridge, Investigations, Voids in Ducts, Specifications, Details, Pre-Bagged Grout

INTRODUCTION

Boston's Central Artery, which includes an elevated section of I-93 between Congress Street and Route 1 in Charlestown, is one of the most congested segments of interstate highway in the U.S. With more than 20 on- and off-ramps and no acceleration or deceleration lanes, the Central Artery has an accident rate nearly double the average of the U.S. interstate system. Jammed with more than 190,000 vehicles daily—more than twice its design capacity—the Central Artery experiences 8 hours of congestion each weekday. These conditions were only expected to worsen, to the point that by the year 2010, it was estimated that there would be a 15-hour rush period.

Fig. 1: Central Artery/Tunnel Project area

In an effort to ease congestion, improve driver safety, address issues of environmental quality, and prevent further adverse effects on the regional economy, Massachusetts transportation officials initiated the current CA/T project in August 1986. As planned, major elements of this extensive undertaking include (see Fig. 1):

- A 1.4-mile section connecting I-93 to I-90 via the South Boston Bypass Road
- A 3.9-mile extension of I-90 from the I-90/I-93 Interchange to the airport

- Logan Airport interchange structures
- Replacing a 4.4-mile segment of the Central Artery (I-93) in the heart of Boston
- A 1.7-mile connection of Leverett Circle to I-93 and Route 1
- Eight separate ventilation buildings located around the project area

The crucial links that connect I-90, I-93, Route 1 and 1A are the project's many interchanges. They move traffic between the interstate roadways and the major highways, as well as connect to the local road system. Four of these interchanges utilize post-tensioned structures. Table 1 provides a brief synopsis of pertinent facts regarding these interchanges.

Table 1: Pertinent Interchange Facts

	I-90/I-93 at South Bay (South Bay Interchange)	I-90 at Logan Airport (Logan Airport Interchange)	I-90 with Route 1A in East Boston	I-93/Route 1 in Charlestown (Charles River Interchange)
Permanent Viaducts/Bridges (Area)	1.35 million square feet	258,300 square feet	66,700 square feet	797,600 square feet
Number of Viaducts/Bridges	17 Viaducts/15 Bridges	5 Viaducts/1 Bridge	7 Viaducts/1 Bridge	8 Viaducts/1 Bridge
Temporary Bridges (Area)	96,900 square feet	6,500 square feet	23,700 square feet	107,600 square feet
Number of Temporary Bridges	13	1	1	8
Span Lengths	49 to 250 feet	66 to 197 feet	98 to 197 feet	98 to 197 feet
Superstructure Widths	22 to 96 feet	22 to 66 feet	22 to 85 feet	24 to 94 feet
Curvature of Roadways	120 to 10,500 feet	250 to 590 feet	200 to 2,200 feet	300 to 7,600 feet
Substructure Types	Mostly single column piers; some double column piers; some multi-level straddle bents	Circular concrete columns with conical caps	Circular concrete columns with conical caps; some double column straddle bents	Circular concrete columns with conical caps; some double level straddle bents
Foundation—Types (Diameter)	Steel pipe piles (12 to 16 inch); drilled shafts (3 to 10.5 feet)	Precast, prestressed piles (12 to 16 inch); drilled shafts (6.9 to 7.9 feet)	Drilled shafts (8 to 9 feet)	Drilled shafts (0.7 to 7.9 feet)
Foundation Lengths	82 to 180 feet	160 to 180 feet	130 to 180 feet	43 to 98 feet
Status	I-93 Southbound operational; I-93 Northbound and some ramps are complete with others under construction	All ramps are operational	Under construction	Under construction; Leverett Circle connector to I-93 is operational

POST-TENSIONED STRUCTURES

SOUTH BAY INTERCHANGE

The existing dilapidated interchange at the intersection of I-90, I-93, and Massachusetts Avenue, which has limited capacity, is being replaced in stages and expanded to handle a considerably higher traffic volume (see Fig. 2). The massive, new multi-level interchange will be all-directional and include the I-90 extension to Logan International Airport through the Fort Point Channel and Ted Williams immersed tube tunnels. The entire interchange, split into four construction contracts, was designed in both steel and segmental concrete box girder alternatives for the viaducts and ramps.

Fig. 2: South Bay Interchange under construction

As a result of low bid, most of the interchange connecting I-90 and I-93 is being built using concrete segmental bridge technology (see Fig. 3) and features precast balanced cantilever and span-by-span construction techniques.

Fig. 3: Cantilever erection using gantry

LOGAN AIRPORT INTERCHANGE

This circular interchange with sharply curved elevated structural loops within loops is located at the gateway to Boston from Logan International Airport. The viaducts and ramps are built using cast-in-place post-tensioned concrete construction techniques. Maintaining airport access/egress during construction was a monumental challenge, requiring many of the continuous structures to be built in at least two stages (see Figs. 4 & 5).

Fig. 4: Logan Airport/I-90 Interchange

Fig. 5: Curved interchange ramps

Fig. 6: I-93/Route 1 Interchange

CHARLES RIVER INTERCHANGE

The northern gateway to Boston through Charlestown begins at the I-93/Route 1 Interchange (see Fig. 6), which is intertwined with the state-of-the-art Charles River bridge crossings. I-93 dips into the downtown tunnels after crossing the cable-stayed Leonard P. Zakim Bunker Hill Bridge from the north. This huge complex interchange, connecting Route 1 with I-93 and Leverett Circle, consists of viaducts, ramps, major bridges, and temporary bridges.

Fig. 7: Span-by-span erection at double-deck bents with overhead gantry

Most of the interchange structures north of the Charles River are being built with precast segmental concrete, using span-by-span and balanced cantilever construction techniques (see Fig. 7). In areas with very tight vertical clearances, voided slab construction is used.

LEONARD P. ZAKIM BUNKER HILL BRIDGE

A hybrid cable-stayed bridge, part of the interchange complex, crosses the Charles River just west of the existing I-93 truss bridge adjacent to the lock and dam system. The Leonard P. Zakim Bunker Hill Bridge (see Fig. 8) includes cast-in-place post-tensioned concrete back spans and a composite steel main span. In the main span, four lanes run in each direction between inclined cable planes, with two additional ramp lanes cantilevered outside the cable plane on the east side. This two-lane ramp rests on its own supports in the

Fig. 8: Leonard P. Zakim Bunker Hill Bridge

back spans. In addition, the North Tower straddles the Orange Line subway tunnel, whereas the South Tower skirts the Orange Line subway tunnel and straddles a 2.9-foot-diameter water main. The main span deck slab between the cable planes consists of standard weight precast prestressed slabs with infill concrete. The cantilevered ramp area consists of lightweight concrete panels with concrete infills. The stay cables form a Y-shape, radiating out from inverted Y shaped towers in the main span (see Fig. 9) and are in a single plane in the median between the northbound and southbound lanes in the back spans.

Fig. 9: Stay cable layout on the bridge

THE INVESTIGATION

In late 2000, the CA/T project was apprised of serious corrosion issues faced by the Florida Department of Transportation (FDOT) on several bridges, such as the Sunshine Skyway, Mid Bay Bridge, Keys bridges, and others (Fig 10). Issues included lack of complete grouting inside post-tensioned ducts, lack of complete protection at anchorages, leaking expansion joints and their effects on post-tensioned anchorages, and others. The project also learned of investigations being conducted by FDOT and improvements to specifications that were underway. The negative impacts on post-tensioned tendons due to lack of complete grouting and detailing was naturally a source of concern since the CA/T project includes numerous post-tensioned structures.

Fig. 10: Sunshine Skyway Bridge in Florida

It took some effort to convince the many parties involved in this extremely fast-paced project—with major schedule and cost constraints—that lack of complete grouting could create serious problems years in the future. Therefore, to alert project personnel to the potential seriousness of the situation, we relied on the talents of Mr. William Nickas of FDOT as well as an expert from Parsons Brinckerhoff in England with a similar background. This choice was significant because England began experiencing problems involving post-tensioned structures long before their U.S. counterparts.

These experts conducted seminars for project staff to illuminate the scope of the situation. By that time, the project had already built a considerable number of post-tensioned structures and was, needless to say, concerned. With approval from the "Interface Group," which included representatives from the Massachusetts Turnpike Authority, Federal Highway Administration, and Bechtel/Parsons Brinckerhoff management staff, a two-phase investigative program for post-tensioned ducts was undertaken. Visual inspection formed the backbone of this program due to both the limited and sometimes unreliable nature of non-destructive testing (NDT) methods. Fig. 11 shows the equipment used during visual inspections, while Figs. 12 through 15 show the various observations during the investigation. However, to provide an additional quality check, the visual inspections were accompanied by NDT techniques.

Fig. 11: Visual inspection equipment


Fig. 12: Boroscope examination revealing partially and totally ungrouted tendons


Fig. 13: Water seeping from drilled hole

NDT testing that was performed included echo-impact (EI), ground penetrating radar (GPR), and X-rays. Although each method was somewhat successful under certain circumstances, no one method could be relied upon without accompanying visual observations.

Voids in the draped ducts were more prone to contain bleed water. Approximately 40% of the voids contained several liters of water. This water was tested with litmus paper which indicted the PH level to be 14, which is the same alkalinity as the water tested from a standard bleed test. After the water was drained, visual inspection using a boroscope revealed that the strands were exposed but did not show any visible evidence of corrosion. The voids in the horizontal ducts were usually dry and shallow so that the strands were not exposed. However, some larger voids were identified that did have exposed strands and did contain water.

Fig. 14: Slight onset of strand corrosion

NON-DESTRUCTIVE INVESTIGATION METHODS

Part of the program to inspect the post-tensioned ducts involved developing methods to locate the ducts and determine if and where voids existed. The two basic types of structures that needed to be inspected were precast segmental elements with ducts located in the top and bottom flanges, and cast-in-place post-tensioned structures with ducts located in the vertical webs. Both metal and plastic ducts were incorporated into the structures.

Fig. 15: Water entering ungrouted temporary tendons, creating freeze-thaw damage inside the top slab of segment

Locating Ducts

The majority of ducts were located by drilling through the concrete at locations identified in the shop drawings. If the ducts were not found on the first attempt, a pattern of drill holes was made until the ducts were located.

Meanwhile, GPR was selected as the most likely form of NDT to locate the ducts. Several companies had demonstrated their capabilities using GPR—with marginal results. As a test, they were asked to locate the ducts along given sections (see Fig. 16). They were also provided with drawings to assist them. After the areas were inspected and the locations of ducts marked, the concrete was drilled to determine if the ducts were correctly located.

The degree of accuracy achieved using GPR was not any better than by solely using the drawings. In fact, in many cases, it was worse. GPR, which is extremely operator dependent, is also influenced by the amount of reinforcing present in an area as well as the proximity of other ducts. In fact, only one contract has used GPR to locate ducts with a good degree of accuracy. This was on a cast-in-place structure with ducts in vertical web walls. However, this same contract could not locate transverse ducts located in the top flange, which had a denser reinforcing pattern.

Although the theory behind GPR makes it an attractive method for locating ducts which should perform satisfactorily, field experience to date has not proven to be successful for most applications.

Fig. 16: GPR investigation in progress.

IDENTIFYING VOIDS IN DUCTS

Essentially all voids to date have been found by drilling into ducts at locations selected by the engineer (i.e., at high points for draped tendons and near anchorages on straight horizontal tendons). The ducts were selected at random, except when field engineers indicated that there were problems during the grouting operation. On draped tendons located in the web walls of cast-in-place structures, inspection holes were dilled in the ducts at the high points, as this is the most likely location for bleed water and air to accumulate. The anchorages were not accessible, so the end high points were inspected by drilling through the web walls near the end diaphragms. The center high points on drapes were also not accessible as the high point was in the upper deck, so inspection holes were drilled through the web walls on both sides of the high point. Some random inspections were performed at lower elevations which confirmed that problems were restricted to the high points.

Fig. 17: Echo-impact testing underway.

In precast segmental structures, the straight horizontal tendons were all located in the top and bottom flanges. The anchorages were not accessible, so inspection holes were drilled down through the flanges to access the ducts. The majority of the holes were drilled in close proximity to the anchorages, but numerous inspections were also done at the mid-points. Inspections verified that voids can form anywhere along the duct and were not limited to just the anchorage area. This made finding voids in a horizontal duct much more difficult due to their random nature. Two forms of NDT were used to assist in locating voids: EI and X-ray.

EI was investigated by having a company test ducts that had previously been inspected by drilling holes into them. The test was "blind" in that the testing firm was not informed of the condition of any of the ducts (see Fig. 17). Results, however, have been mixed.

EI offers some positive features, such as: Small and portable testing equipment that can be taken into confined spaces; collected data can be interpreted in real time; testing can be conducted quickly, allowing for long lengths of duct to be inspected; and the cost, relative to other testing methods, is quite low.

On the negative side, however, the limitations of the procedure need to be considered. The ducts must be accurately laid out so that the testing equipment can be located directly over the duct, which brings us back to finding an accurate method of locating ducts. EI will also not work on nonmetallic ducts; anomalies in the concrete will affect the test; the methodology cannot distinguish between small and large voids; and orientation in relation to voids affects the accuracy of the test.

The trials indicated that EI produced promising results on precast segments where the ducts were located in the top and bottom flanges, but was not very accurate when detecting voids in vertical web walls. The probable reason for this discrepancy is that in the top and bottom flanges voids present a larger surface area for the sound waves to travel through and therefore result in longer delay times, thereby indicating the presence of a void. In vertical web walls, however, the side profile of the void presents a much smaller target. This technology looks promising, but at present has not demonstrated its reliability.

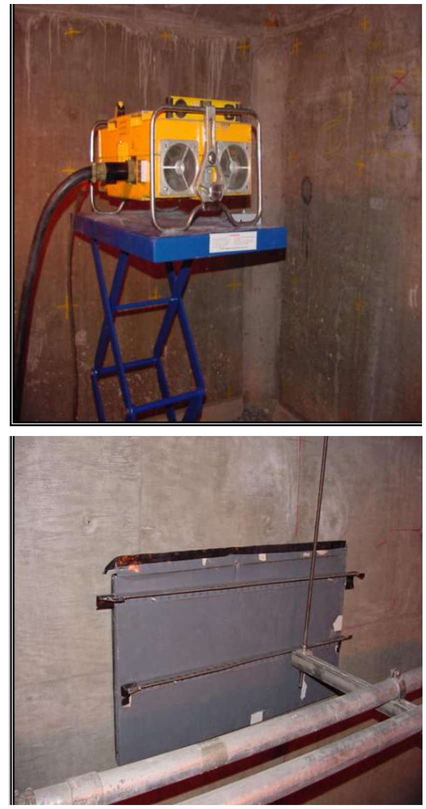

Fig. 18: X-ray equipment set-up

X-rays were also used as part of a blind test on ducts in the web walls of a cast-in-place structure (see Fig. 18). These were draped metal ducts, 4 inches in diameter that were centered in a web wall that was approximately 12 inches thick. Sixteen separate test areas were examined. The results of this testing method were found to be very accurate for all areas being evaluated. X-rays were able to show the degree of voiding and water in the ducts (see Fig. 19). The system has other positive features in that the equipment used is relatively portable; the film can be developed on site and used to guide any necessary excavations; interpretation of the film is not as complicated as some other testing methods; and the testing time is faster than drilling.

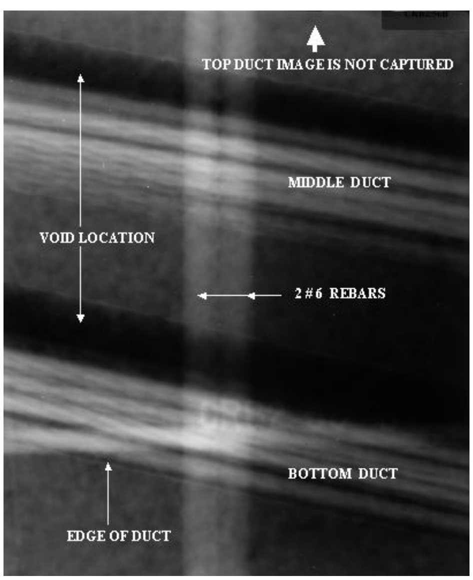

Fig. 19: X-ray image showing voiding

However, X-ray limitations involve factors such as: Safety issues that limit other activities in the area;

both sides of the ducts need to be accessible; and the X-ray source and film must be aligned with each other. The cost of this process is also relatively expensive, but in some applications is still cost effective. At this time, its effectiveness on ducts located in top and bottom flanges is unknown.

VISUAL OBSERVATION RESULTS

The first phase of the investigation was accomplished during May-June 2001 and involved checking 180 locations in five contracts selected at random or where construction personnel suspected inadequate grouting. During the second phase of the investigation, conducted during October-November 2001, an additional 154 locations were examined. In total, the effort covered seven contracts and 334 locations. Of course, this is a small amount compared to some 20,548 tendons in the entire project. However, this sampling largely achieved the goals of determining if grout voids existed and if corrosion had started in the strands. Table 2 provides a summary of the investigation.

Contract	# Tendons	# Insp.	% Insp.	%Voids
C09A4	4208	55	1.31	7
C09C1	2462	69	2.80	57
C019D1	745	79	10.60	37
C019B1	8032	30	0.37	13
C07D2	4207	72	1.71	14
C07D1	734	23	3.13	13
C019E7	160	6	3.75	0
Total	20548	334	1.63	27

Table 2: Phase I & II Investigation Summary

IMPLEMENTED IMPROVEMENTS

The project developed standard specifications for both segmental concrete and cast-in-place concrete structures in 1995-1996. The specifications, which were reviewed by many experts both inside and outside the project, were based on current thinking at the time and were considered state-of-the-art.

The grout mix design used on all the structures was in accordance with Post-Tensioning Institute guidelines in effect at the time. The grout had a water cement ratio of less than 0.45, consisted of Type II cement, and included a minimal amount of expansion agent. The batched grout needed an efflux time between 11 and 30 seconds as verified by a standard low cone test and needed to develop a compressive strength of 4,000 psi at 28 days. The only grout bleed test required by the project was that the grout should have no visible bleed water for 15 minutes using the standard bleed test per ASTM C940. All the contractors submitted independent laboratory test data to verify that their grout mix design met this criteria. However in early 2000, when issues related to grouting inside post-tensioned ducts, and the resulting corrosion of strands, became known, action had to be taken. This included:

- Updating specifications to reflect lessons learned to date
- Specifying the use of "pre-bagged" grout
- Testing and approving pre-bagged grout suppliers (presently, we have three suppliers approved for use and a fourth has passed the necessary tests)
- Expanding grout testing requirements

- Dealing with cold weather grouting issues (see below for further explanation)
- Specifying grouting injection and recharge requirements for continuous parabolic tendons
- Improving anchorage protection at expansion joints
- Entrusting post-tension system suppliers with responsibilities for grouting in addition to post-tensioning operations
- Introducing an updated "Tendon Grouting Inspection Checklist" for field inspectors (see Fig. 20)

Massachusetts Turnpike Authority
Central Artery/Tunnel

TENDON GROUTING INSPECTION CHECKLIST

DATE: ________ CONTRACT NO.: ________

STRUCTURE: ________ LOCATION: ________

WEATHER: ________ TEMPERATURE: ________

MIX ID NO./SUBMITTAL NO.: ________ TEST CUBES MOLDED: YES ☐ NO ☐

PRE-GROUTING	CONTRACTOR	OWNER
1. Mix design A. Prepackaged B. Field mix		
2. Check materials (Concrete Si02 <20%) verified shelf life and temperatures		
3. Ducts are clean and dry (blow air through until no evidence of moisture)		
4. Check ducts for leakage and possible crossover		
5. All grout openings and vents shall be open at the start of grouting		
6. Check mixer and pump (screens and pressure gauges should be in place)		
7. Flushing equipment should be available (should not be the same pump as for grouting)		
8. Concrete and air temperature should be 35 degrees or higher		
9. Verify actual material quantities based on environmental conditions (measuring devices)		
10. Initial testing of grout (flow cone and bleed)		

GROUTING										
Duct ID										
Length										
EFFLUX TIME										
Time In										
Time Out										
TIME										
Start										
Finish										
Pressure										

POST GROUTING ACTIVITIES	CONTRACTOR	OWNER
1. Check for complete grouting at anchorages and vents		
2. Within 14 days exposed end anchorages to be cleaned & coated with bonding compound		
3. Place concrete over anchorages		
4. Apply two coats of mineral stabilized coal tar base emulsion		

Signatures:

Installer: (Acceptance) ________ Date ________

Contractor: (Acceptance) ________ Date ________

B/PB-MTA (Acceptance) ________ Date ________

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Fig. 20: Tendon grouting inspection checklist

Controls on cold weather grouting were improved. A significant factor being that most manufacturers require an ambient temperature of 40 degrees Fahrenheit for proper grout curing. However, since project schedules required grouting to be performed throughout the winter months, it was necessary to meet both sets of requirements. To accomplish this, the concrete and ducts were heated to 40 degrees Fahrenheit using electric heaters inside the structures and insulation blankets placed on the top surfaces. Bags of grout were also stored inside and mixers were pre-warmed by running hot water through them. The grout was batched using warm water at approximately 60 degrees Fahrenheit. The heat was maintained until the grout obtained 800 psi compressive strength.

Our experience with pre-bagged grout is as follows:

- Far superior to previous grouts in the industry
- Provides good results with the ducts fully grouted
- Water is pushed out ahead of the grout during grouting operations
- Temperature sensitive in that both the ambient temperature and the temperature of the grout materials effect the amount of required water to obtain a specific flow rate (requiring frequent adjustments in the batching quantities)

- Need to adjust water content at the start of every grouting operation
- Grout bags need to be better waterproofed, as moisture getting into the bags causes clumping (resulting in many bags being rejected)
- Pumping pressure should average 60-70 psi (not less than 30 psi)

It is not adequate merely to improve the specifications. Many of the details connected with post-tensioning tendons and their anchorages also needed improvement. In this regard, while also considering the project's already advanced state, we took the following steps:

- Added additional vent ports in bottom slabs
- Required all post-tensioning anchors to have permanent caps irrespective of concrete pour backs
- Enhanced grout inspection access (see Fig. 21)
- Improved waterproofing details to protect anchorages at expansion joints (see Fig. 22)
- Investigated improved methods to connect post-tensioned ducts at segment joints
- Enhanced sequencing details for grouting parabolic tendons (see Fig. 23)
- Dedicated grout inspection team

CONCLUSION

The issue of grouting for post-tensioning tendons, whether internal or external, is of paramount importance. Previously in the U.S., we were of the opinion that strand corrosion due to inadequate grouting and poor detailing was not an issue as in some other countries. Unfortunately, our complacency has been shattered and we now realize that we're facing the same challenges as engineers in other parts of the world. Accordingly, we

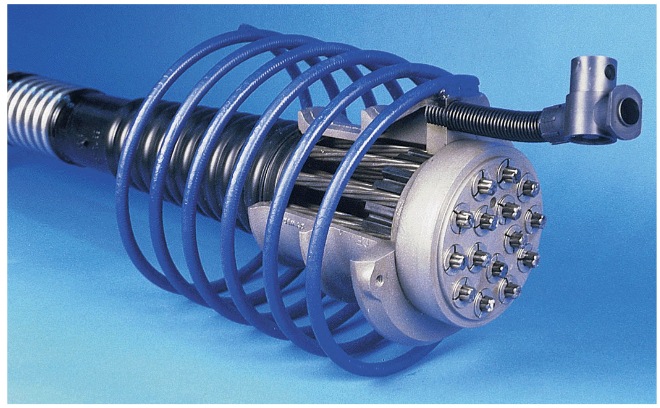

Fig. 21: Improved grout inspection access

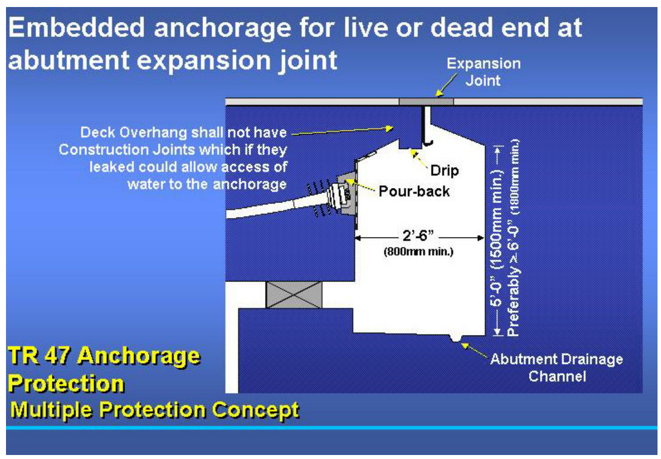

Fig. 22: Protection of anchorage at expansion joint

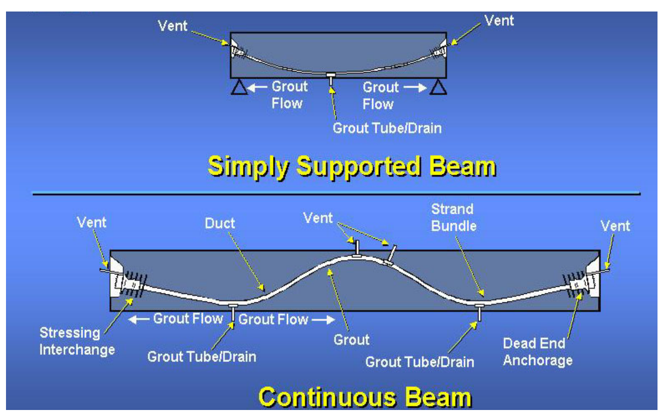

Fig. 23: Parabolic tendon grouting

need to strive continuously to improve our specifications and details. But there are many areas that still need improvement. To build ever more durable concrete structures, we should all work to improve the inter-related fields of post-tensioning and inspection techniques.