



Durability of post-tensioning tendons

Workshop 15-16 November 2001
Ghent (Belgium)



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Durability of post-tensioning tendons

Technical report

**Proceedings of a workshop held at
Ghent University on 15-16 November 2001**

edited by

Luc Taerwe

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Any publication not having met the above requirements will be clearly identified as preliminary draft. This Bulletin N° 15 is published as a <i>fib</i> Technical Report collecting the contributions to a workshop jointly co-sponsored by <i>fib</i> and IABSE. Invited participants present their individual views. Although not yet discussed in any of the association's working bodies, its highly topical contents is believed to be of general interest to <i>fib</i>'s members and to document a starting point for future work in this field. Therefore, the Council of <i>fib</i> agreed to publish these papers in <i>fib</i>'s series of Bulletin.	

Cover picture: Ghent - venue of the workshop

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First published 2001 by the International Federation for Structural Concrete (*fib*)

Post address: Case Postale 88, CH-1015 Lausanne, Switzerland

Street address: Federal Institute of Technology Lausanne - EPFL, Département Génie Civil

Tel (+41.21) 693 2747, Fax (+41.21) 693 5884, E-mail fib@epfl.ch, web <http://fib.epfl.ch>

ISSN 1562-3610

ISBN 2-88394-055-X

Printed by Sprint-Digital-Druck Stuttgart

PREFACE

In the past, problems have arisen regarding durability of post-tensioning tendons, which in some countries have led to fairly restrictive regulations. As a result, on the one hand improvements to execution procedures have been developed, and on the other hand new and improved prestressing systems have been proposed.

These problems have been discussed in IABSE Working Commission 3 "Concrete Structures" and *fib* Commission 9 "Reinforcing and prestressing materials and systems". It was decided to organise a workshop in order to review the different aspects of the problems encountered and to discuss the available solutions. Key-note speakers from various countries were invited to contribute and their presentations are published in this bulletin under the following themes: inventory and condition, investigation and repair, technical progress, strategies for improvement.

The workshop took place on 15-16 November 2001 at Ghent University at the occasion of the 75th anniversary of the Magnel Laboratory for Concrete Research. It was co-sponsored by *fib* (fédération internationale du béton - international federation for structural concrete) and IABSE (International Association for Bridge and Structural Engineering).

We express our thanks to the experts and speakers for sharing their experiences with colleagues thus allowing further progress in this particular field of prestressed concrete construction.

Luc TAERWE
Editor and Chairman of SC
Director Magnel Laboratory

Joost WALRAVEN
President of *fib*

Manabu ITO
President of IABSE

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Durability of post-tensioned tendons on road bridges in the UK

R. WOODWARD

Dr., TRL Ltd., Crowthorn, UK

1. Introduction

Post-tensioned concrete was first introduced into the UK during the middle of the last century and has been used for the construction of all forms of building and civil engineering structures including nuclear pressure vessels and bridges. This paper will focus on the durability of post-tensioned tendons in concrete highway bridges. There are over 100,000 highway bridges in the UK of which it is estimated that there are 3,000 post-tensioned concrete bridges. About a third of these structures are on the Trunk Road Network and the remainder are on local roads.

The first post-tensioned bridge built in the UK was Nunns Bridge in Lincolnshire that was completed in 1947 with a span of 22.5m (Sriskandan 1989). Over the next five decades post-tensioned concrete became the preferred method of construction for bridges in the 30m to 50m span range. Its use has brought many benefits to the economy and construction of large bridges, particularly the extended range of possibilities open to designers, the use of various combinations of precast and in-situ concrete and the use of cantilevered methods of construction.

There has always been concern about the inspection of tendons in post-tensioned concrete bridges mainly because of the difficulty of detecting and quantifying any deterioration. Bridges are particularly vulnerable as they are exposed to the environment and are subjected to the widespread use of de-icing salts during winter. In addition many engineers have expressed concerns about the effectiveness of the methods used to protect the tendons against corrosion, particularly the efficacy of grouting techniques for completely filling ducts with grout.

The following sections describe the performance of post-tensioning tendons in internal grouted ducts in bridges in the UK and the reasons that lead to the (then) Department of Transport's moratorium on this type of construction in 1992. This is followed by a description of the programme of Special Inspections that were subsequently undertaken and a preliminary summary of the findings. Finally a brief review is given of the durability of tendons in externally post-tensioned bridges.

2 Performance of Post-Tensioned Bridges 1947 to 1992

During the 1950s and early 1960s post-tensioned concrete bridges gave satisfactory performance. However the collapse of two footbridges during the 1960's gave an indication of the problems that could arise. Both were of segmental post-tensioned construction and both collapsed without warning under self-weight (Storror 1993). Another structure that illustrated the type of problems that could occur was a small footbridge in Lincolnshire. It had been built in 1958 and when it became redundant and was demolished seven years later, the

tendons were found to be severely corroded. No detailed investigations were carried out but the corrosion was attributed to difficulties incurred during grouting.

The first problems with a major structure were found during the early 1970s when longitudinal cracks were observed on some of the I-beams of a beam and slab bridge built in 1961 (Storror 1993). They followed the profile of the tendons and holes drilled into the ducts disclosed the presence of voids and water. The cracks were attributed to expansion of the water on freezing. There was only slight corrosion on the tendons and duct sheaths.

Holes were also drilled into the ducts in a neighbouring bridge that was not showing any external evidence of trouble. This investigation revealed that 62% of the ducts contained voids that were continuous along the whole of their length and two ducts were empty. Some lined ducts also contained water and moist grout. Unlined ducts were dryer and no water was found. Wires in ducts containing grout showed some surface corrosion but were generally in good condition. In the two ungrouted ducts the wires varied from near perfect to extensive surface corrosion. Wire fractures were found at an air water interface in a partially grouted duct and only 600mm from the fracture the wires were in good condition.

Subsequent investigations were undertaken on twelve bridges built between 1958 and 1977 (Woodward 1981). They differed in respect of type of bridge, prestressing system, designers and contractors and none of the bridges examined showed any external indication of trouble. However voids were found in over 50% of the ducts examined and were present in all but one of the structures. The size of the voids varied considerably and they were often of sufficient size to expose the tendons. The distribution of voids was uneven. In continuous structures they were usually concentrated at high points over the piers. In general ducts were dry and there was usually a thin film of cement paste over the wires. There was no evidence of serious corrosion with the exception of slight pitting on a strand in a virtually empty duct and it was concluded that the presence of some grout in a duct provided an environment in which corrosion did not take place. The results were passed to the Standing Committee on Structural Safety, which concluded at the time, that in structures containing a large number of tendons, "the risk of sufficient tendons failing by corrosion at any time to cause sudden collapse is considered to be small" (Standing Committee on Structural Safety 1979).

During the 1980s problems were found in an increasing number of post-tensioned bridges (Storror 1993). In 1980 leakage through the joints, and between precast and in-situ beams was observed under a ten span post-tensioned bridge built in 1960. Some of the stalactites were rust coloured and further investigation revealed loose rusted wires near anchorages where they were opened up at road level. The following year problems with a large segmental bridge were found during a principal inspection when lumps of concrete were found to have spalled from the lower flanges of the main span. Further investigations revealed serious corrosion of the tendons, which was caused by road salt percolating through the deck waterproofing.

The most serious event occurred in December 1985 when a single span segmental post-tensioned bridge collapsed without warning (Woodward & Williams 1988). Fortunately there was no traffic on the bridge at the time. The structure consisted of precast units post-tensioned together both longitudinally and transversely, and chlorides had penetrated to the joints and corroded the tendons. There had been no evidence of distress prior to collapse.

A sample of nine segmental bridges was subsequently examined and voids were found in seven of the structures and severely corroded tendons were found at two locations in one of the bridges. It was concluded that despite widespread deficiencies in grouting, the tendons were generally in good condition.

Despite the findings of the survey the number of structures with problems continued to increase and during the late 1980s and early 1990s about a dozen post-tensioned structures were found with serious corrosion problems and they required either major refurbishment or

replacement. These included discovery of corrosion of the pressing tendons in Folly New Bridge Bladon in 1988, the M1 Blackburn Road Bridge in 1990 and Botley Road Flyover in 1992, all of which have been replaced.

3 Moratorium and Programme of Special Inspections

Many of the defects described above were found by accident rather than as part of the normal inspection process. At Botley Road Bridge the corroded tendons were discovered when the waterproofing was being replaced and the concrete cover over the ducts was inadvertently removed. In another case an investigation of an apparently minor problem revealed serious corrosion of the tendons that would otherwise have gone unnoticed.

In September 1992 the (then) Department of Transport (DoT) announced that it would not be commissioning any new post-tensioned concrete bridges with internal grouted tendons until design and construction standards could be reviewed (Department of Transport 1992). The reason for the moratorium was concern over the vulnerability of prestressing systems to corrosion resulting from the penetration of water and de-icing salts. At the same time the DoT announced that all existing Trunk Road bridges of this type would be systematically inspected over a period of five years. These Special Inspections commenced, effectively, in the spring of 1993.

Following the announcement, the Department of Transport's Highways Agency developed a number of Standards and Advice Notes to assist Maintaining Agents manage the inspection programme. The areas addressed were as follows (Design Manual for Roads and Bridges, 1993):

- Prioritisation for inspection (BD 54/93)
- Planning and execution of the inspections (BA 50/93).

Standard BD 54/93 specified a method for assigning priorities to bridges for the programme of Special Inspections. This was based on three factors relating to the bridge - its age, structural form and presence of vulnerable details - and on three factors relating to the site - the traffic on and under the bridge and the importance of the route. Five Priority Ratings were defined, numbered 1 to 5, 1 being the highest. The intention was to inspect the higher priority bridges first, to establish the condition of the most vulnerable and important bridges at the beginning of the programme. This prioritisation approach provided a useful guide for planning purposes but was not followed slavishly because restrictions on access to the highway and other practical considerations may take precedence.

Advice Note BA50/93 was issued at the start of the programme of Special Inspections. It set the framework for the inspection and provided advice on techniques, and is described in more detail in the following section.

4 Methodology for the Special Inspection Programme

The Special Inspections were carried out in three phases: Phase 1, a desk study; Phase 2, a preliminary inspection; and Phase 3, the site investigation itself. The site investigation concentrated on inspecting tendons at critical sections which were defined as those at risk from water ingress, where yield points may form in a collapse mechanism, or both.

The desk study was undertaken to obtain detailed information on the structure and identify details that might provide access for deleterious materials into the duct. These included:

- joints (construction joints in in-situ concrete and joints between in-situ and precast concrete and between precast elements.)
- absence of a deck slab
- detailing at anchorages
- duct linings
- deck waterproofing
- drainage systems.

All of these details affect the protection given to the tendons. Other factors that affect the durability of the structure include the use of sea-dredged aggregates or admixtures containing chlorides, location of the structure, design and traffic loading.

The location of the structure was a major factor. For example, structures in a marine environment or on routes where de-icing salts are used during winter are more at risk than inland structures in milder parts of the country.

The detailed design of structures was also considered. For example, the class of structure, ie whether cracks or tension were allowed under full design loading. If cracking was allowed, there would be a reduction in the degree of protection provided to the tendons. Traffic loading may accelerate this, particularly if it caused cracks to open and close, as it would make it easier for deleterious materials to gain access to the ducts. Such problems would be more acute on structures used by overloaded vehicles. However, in the UK bridges are designed as class 1 (ie no tension) so the risks of cracks opening under load are small.

Finally the results of previous inspections were examined to determine whether there was any evidence of distress. The reports were reviewed for evidence of water leakage through the structure, ingress of chlorides, rust staining or cracking, particularly along the line of the ducts.

Phase 2 was a preliminary inspection that was undertaken to confirm the findings of the Phase 1 study. Areas showing signs of deterioration were identified and an appropriate investigation planned to determine the causes and consequences of the deterioration.

Phase 3 was a more detailed inspection that concentrated on the critical sections identified during the Phase 2 study. This almost always included an intrusive examination of the tendons, as the only reliable method of checking their actual condition was to expose them and inspect directly. This was done either by drilling or coring into ducts but needed to be done selectively as it damaged the structure, there was a risk of damage to the tendons and it disturbed the environment inside the ducts. Non-destructive testing was mainly limited to locating tendons prior to an intrusive investigation. It was little used for fault detection.

When entry had been gained to a duct, the inspection was confined to the hole itself or a short distance to each side. Occasionally when there was a significant void, a flexible videoscope was used to increase the length of tendon inspected. Samples of any water or other contaminants in the duct were taken for analysis where present. Duct leakage and void volumes were also measured on some occasions.

Inspection of anchorages was sometimes undertaken but this often involved the removal of a large amount of unstressed concrete to examine the exposed anchorage surface. Drilling-out the grout hole in the casing sometimes allowed an internal inspection behind the anchor plate.

5 Findings from the Special Inspections

TRL was commissioned by the Highways Agency to collate the reports from the Special Inspection Programme and record the findings on a database under three main headings - voids, tendon corrosion and anchorage faults (Cullington et al 1996). In the analysis that is described below, two classifications have been used: one for severity and another for incidence. In assigning the severity classification it has been necessary to interpret the descriptions given in inspection reports and attempt to achieve overall consistency.

Severity is related to the size of void or extent of corrosion. Void sizes were classified as:

- small - tendon covered in grout
- medium - tendon partially exposed
- large - tendon virtually wholly exposed but covered in grout wash and
- ungrouted - no evidence of grouting.

The extent of corrosion was classified as:

- minor - surface corrosion, possibly present at construction in some cases
- moderate - pitting without significant section loss
- heavy - significant section loss, fracture in strands or large-diameter wires
- severe - fracture of at least half of tendon.

Incidence was defined in four categories:

- widespread - meaning occurring at half of the critical points inspected or more
- common - meaning occurring in less than half down to one quarter of points
- occasional - meaning less than one quarter of points and
- isolated - meaning only one or two points in a small inspection or up to 5% in a large inspection.

When this work ended, there were 447 bridges on the database and full entries for 281 of these. An analysis of the data showed that overall, the condition of the bridges was surprisingly good considering the problems that preceded the Special Inspection Programme and that led to its implementation.

Based on the sample of 447 bridges, voids were found in over 50% of the bridges inspected (Figure 1). Only those that were sufficiently widespread to constitute an unbonded system or contain water, chlorides or both were considered to present a threat to the long-term durability of the structure. This risk is related to how well sealed voids are from the atmosphere ie where there is leakage this may provide a path for water and chlorides to enter the ducts and corrode the tendons. However leakage rates were not systematically recorded and even where data is available it is not currently possible to quantify the corrosion risk. About 25% of bridges contained large voids or ungrouted tendons. Of these, about 14% had a high incidence of occurrence (3% of the total sample). About 36% of bridges contained voids large enough to allow the tendons to be at least partially exposed. These observations relate to the condition at the inspected position and not the extent or volume of the voids. They also denote the worst case found on a bridge.

Based on year of construction, the earlier bridges in the sample were more inclined to have larger voids. In the 1950s and 60s, over 50% of the bridges had small voids at worst (the remainder having larger voids). In the 1970s, the figure was over 70%. For the 1980s and 1990s, there were relatively few bridges in the sample, but of these very few had significant voids detected. The late 1960s and early 1970s were the most productive years to date in the UK for the construction of post-tensioned bridges.

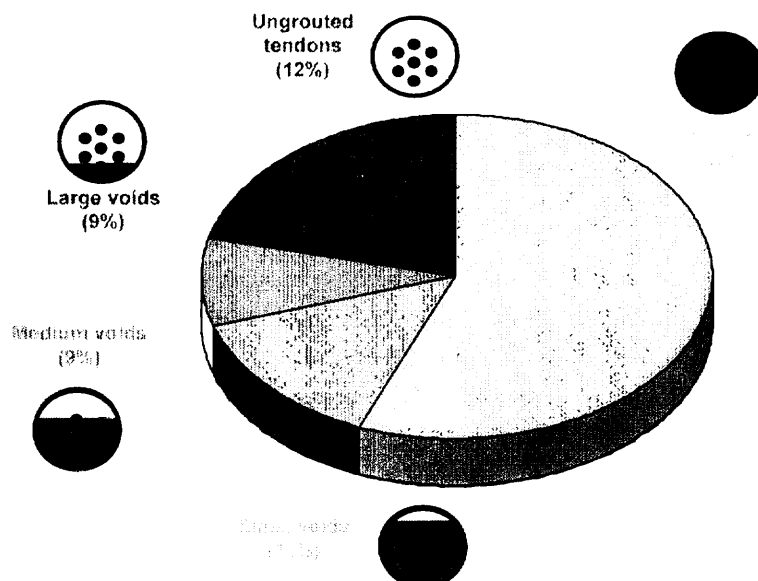


Figure 1 Void sizes

Compared with voids, the tendon corrosion was much less severe and more isolated in occurrence, 90% of bridges containing minor corrosion at worst (often just surface discoloration) (Figure 2). Sometimes this was widespread but more substantial corrosion (ie worse than minor) was generally isolated. As expected, corrosion was more prevalent in older bridges, the 1950s being the worst with approximately one-quarter of the sample containing substantial corrosion. Very little substantial corrosion was found in bridges from the late 1970s onwards.

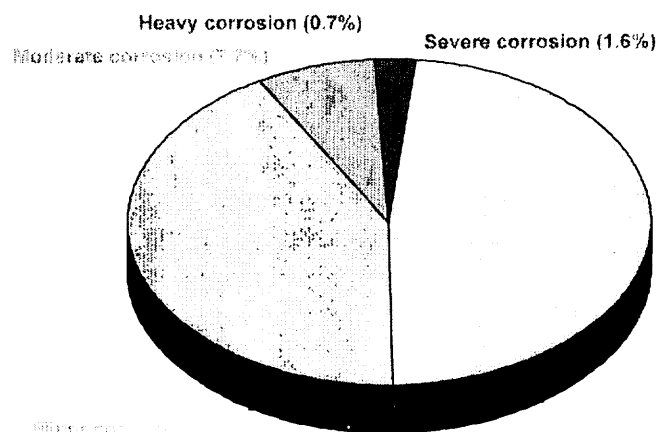


Figure 2 Tendon corrosion

The observations relating to year of construction must be treated with caution as they are based on small samples in each five-year period adopted in the analysis.

More detailed information was extracted from a smaller sample of 281 bridges. This included data on anchorage inspections and observations made during the inspections.

Based on this smaller sample, anchorage inspections were carried out on about 63% of the bridges inspected. Of the anchorages inspected, 75% contained minor corrosion at worst, 19% contained moderate corrosion and 6% contained heavy or severe corrosion (Figure 3).

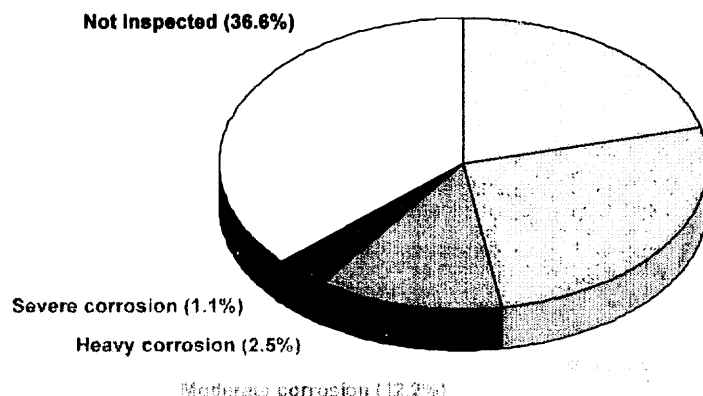


Figure 3 Anchorage Corrosion

The presence of water, sometimes also with chlorides in the grout, was reported in 13% of the 281 bridges. Many of these were connected with corrosion classified as moderate or more. Most of the 13% had voids present in the ducts, a factor that arguably assists in water movement and suggests the potential for future deterioration.

In terms of overall condition, 3% of bridges were classified as having significant defects and a further 4% as needing attention. Over 50% were classified as being in good condition with the balance having minor problems. For the future, the important factors are likely to be the rate of deterioration of the post-tensioning systems and the frequency of inspection necessary to ensure continuing bridge safety.

6 Externally Post-Tensioned Tendons

The vast majority of post-tensioned concrete bridges in the UK contain internally grouted tendons although the moratorium on this form of construction did lead to renewed interest in the construction of bridges with external unbonded tendons. In addition there are a small number of structures with either internal unbonded tendons and or tendons which were initially placed external to the section but were subsequently covered with concrete. A list of these bridges is given in Tables 1, 2 and 3.

Eight externally post-tensioned bridges were built before 1980. Wire failures have been found in two of them and both used PVC sleeving over the strands (Porter 1985, Brooman & Robson 1996). The failures were due to brittle fracture but the exact cause is unknown. All but one of the other structures had strands housed in polypropylene sleeves and these appeared to have performed well. The corrosion protection systems that have been used on more recent structures all appear to be performing well although it is too early to comment on their long-term performance. The only problem that has been noted is leakage of wax from wax filled ducts. Problems with structures with external bonded tendons have occurred where chlorides have gained access into the concrete box section.

Table 1
Externally post-tensioned bridges in the UK

Bridge	Date Opened	Corrosion Protection
Braidley Road	1970	Wires coated with Bridon's Metal Coat A - an aluminium resin – and each strand in a PVC sleeve.
Canning Town	1972	Strands individually sheathed in grease filled PVC sleeves, grouted at anchorages.
A3/A31	1976	Each strand in grease filled PVC sheath, grouted at anchorages.
Robsart Bridge	1977	Each strand in a grease filled polypropylene sheath, grouted at anchorages.
Botley Interchange		
Exminster Viaduct		Each strand in a grease filled polypropylene sheath.
Exe Viaduct		
River Camel	1993	HDPE ducts filled with grout, wax at anchorages.
Basingstoke Canal Aqueduct	1995	Individually sheathed monostrands in wax filled HDPE ducts.
Batheaston - Avon Viaducts	1996	HDPE ducts filled with grout, wax at anchorages.
Second Severn Crossing	1996	HDPE ducts filled with wax.
A13 Flyover	2000	HDPE ducts filled with grout.

Table 2
Post-tensioned bridges in the UK with internal unbonded tendons

Bridge	Date Opened	Corrosion Protection
River Ouse	1975	Thermofit polythene sheathing filled with grease.
Batheaston - Bathford Bridge	1996	HDPE ducts filled with wax.
River Clwyd Viaduct	1996	50mm dia smooth HDPE ducts filled with petroleum wax

Table 3
Post-tensioned bridges in the UK with external bonded tendons

Bridge	Date Built	Corrosion Protection
Wentbridge	1961	Fine concrete placed around strands
Hammersmith Flyover	1962	In-situ concrete
Bluther Burn Bridge	1962	In-situ concrete
Greyfriars Bridge	1966	In-situ concrete protected by a layer of bitumen.
Calder Bridge	1968	Fine concrete placed around strands
Five Ways Interchange	1979	Tendons encased in concrete

7 The Future

The programme of Special Inspections has given a good overview of the condition of the stock of post-tensioned bridges with internally grouted tendons. In many bridges the post-tensioning system is in good condition, with no corrosion. For others containing significant faults - singly or in combination - such as large or interconnected voids, leakage, water, chlorides and corrosion a management strategy is required. Depending on the severity of the faults, this seems likely to include monitoring and further inspections in the future - particularly for structures that are expected to remain in service for 120 years. One important issue that has to be addressed is the need and practicality of regrouting. In the longer term some structures may be suitable for supplementary post-tensioning.

The moratorium has given time for the industry to re-consider design details and measures to protect the post-tensioning systems of new bridges. These were described in Concrete Society Technical Report TR47 which was published in 1996 (Concrete Society 1996). This was followed by the lifting of the moratorium for cast in-situ grouted duct post-tensioned concrete construction although the moratorium on post-tensioned concrete pre-cast segmental construction remains in place (Chief Highway Engineer's Memorandum 1996). Evidence of faults found during the inspection programme confirms that this has been worthwhile and the programme has laid the basis for future bridge management. New procedures to improve grouting will be helpful, as will additional protection systems. Measures to permit inspection and monitoring should not be overlooked.

8 Conclusions

Corrosion of post-tensioned tendons in internally grouted ducts has occurred in a small number of post-tensioned concrete highway bridges in the UK. However, the majority of structures have a good record of durability. Even though voids are common and many ducts contain exposed tendons, very few tendons are showing serious signs of corrosion after several decades in service.

Isolated voids in themselves are not a major problem. Where the ducts are well sealed from the atmosphere, the presence of even a thin layer of grout over the surface of the wires has provided effective corrosion protection. What is not known is for how long this layer of grout will continue to protect the tendons.

Serious corrosion problems can occur where tendons are inadequately protected whether due to poor detailing or poor workmanship exacerbated by inadequate grouting. Corrosion is caused by the ingress of chlorides and other deleterious materials through vulnerable areas such as anchorages, joints or cracks. The difficulty of detecting corrosion and the absence of visual evidence of deterioration in most cases means that visual inspection alone is not sufficient to determine the condition of these structures.

There are only a small number of bridges with unbonded external post-tensioned tendons. Brittle fractures occurred in two of the earlier structures within a short period of construction. No other problems concerning the durability of the tendons have been reported with this type of construction.

Further durability problems may occur with internal grouted tendons in the future as the corrosion protection provided to-date begins to break down and moisture and chlorides gain access over time. Structures identified during the Special Inspection Programme as being most at risk will require careful management to ensure that any deterioration that occurs is detected at an early stage.

9 Acknowledgements

The work described in this paper was carried out in the Infrastructure Division of TRL Ltd and was funded by the Highways Agency. The author would like to acknowledge the contribution of David Cullington and Valerie Stephens who developed the methodology for collating the results from the Special Inspections.

10 References

Brooman H and A Robson (1996). *A3/A31 Flyover - Case history of an externally post-tensioned bridge*. International Conference on Bridge Management. University of Surrey. April.

Chief Highway Engineer's Memorandum (1996). *Post-Tensioned Grouted Duct Construction*. CHE Memorandum 40/96.

Concrete Society (1996). *Durable Bonded Post-Tensioned Concrete Bridges*. Technical Report 47. The Concrete Society. Crowthorne. UK.

Cullington D W, Hill M E and R J Woodward (1996). *Special inspections on post-tensioned bridges in England: Report on progress*. FIP Symposium. Symposium Papers Vol 1 pp 482-491. London 25-27 September.

Department of Transport (1992). *Standards for post-tensioned prestressed bridges to be reviewed*. Press Notice No. 260. 25 September.

Design Manual for Roads and Bridges (1993). Vol 3 Section 1: Part 2. BD 54/93. *Post-tensioned concrete bridges. Prioritisation of Special Inspections*. Vol 3 Section 1: Part 3. BA 50/93. *Post-tensioned concrete bridges. Planning, organisation and methods for carrying out Special Inspections*. Department of Transport. London

Porter M G (1985). *Repair of post-tensioned concrete structures*. Concrete Bridges Investigation, maintenance and repair. Proceedings of a one-day symposium, London 25 September.

Sriskandan K (1989). *Prestressed concrete road bridges in Great Britain: a historical survey*. Proc Instn Civ Engrs, Part 1, 86, Apr, 269-303.

Standing Committee on Structural Safety (1979). *Safety of post-tensioned concrete bridges*. Third Report of the Committee for the period ending 31 March 1979. London pp15 -17.

Storrar D B (1993). *The management of post-tensioned grouted-duct bridges owned by the Department of Transport*. Proceedings of fifth international conference on structural faults and repair, Edinburgh, 29 June 1993. Editor M C Forde. Engineering Technics Press, Edinburgh.

Woodward R J (1981). *Conditions within ducts in post-tensioned prestressed concrete bridges*. TRRL Laboratory Report, LR 980. TRL Crowthorne, Berkshire.

Woodward R J and F Williams (1988). *Collapse of Ynys-y-Gwas bridge, West Glamorgan*. Proc Instn Civ Engrs, Part 1, Vol 84. Aug 1988. pp635-669.

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Problems, Solutions and Developments at Post-Tensioning Tendons from the German Point of View

D. Jungwirth

Prof. Dr.-Ing., DYWIDAG/DSI, München (Germany)

Abstract

Tendons with and without bond have a high level of development. Their durability, however, is considerably influenced by the quality of execution, as for example the grouting for sufficient corrosion protection. In Germany two trends developed:

Beside the quality assurance of the components and of the injection grout, in national and international approvals is striven to set high demands also to the prestressing contractors and to install an effective internal and external quality control with check lists.

A second trend in Germany was to counteract the bad quality of execution by controllable, exchangeable and strengthenable PT-tendons. At present this resulted in a preference of the external tendons. Beside the advantages very soon also the disadvantages of these tendons at planning/dimensioning and execution were realised.

In connection with the a.m. quality assurance system, the mixed prestressed concrete structures become preferred, which consist of a reasonable combination of external and internal tendons.

A new development is presented: controllable, exchangeable internal tendons without bond which remedies all the notable disadvantages of external tendons and preserves their advantages.

1 Introduction

The following tables 1 to 5 show the history of the internal and external post-tensioning [1 and 2], its strong and weak points, its synergies in case of advantageous combination of internal and external PT-tendons (internally with and without bond).

Within the last years big controversial discussions developed [3 to 8]:

- Webs must be free of PT-tendons – instead of making them wider
- The injection technique is so bad that a factory made corrosion protection system with wax/grease and PE-sheathing is preferred – instead of improving the injection technique itself
- PT-tendons must be controllable, restressable and exchangeable - mixed application is better!
- Prestressing steel is failing due to crack corrosion; correspondingly increased amount ordinary steel helps to get "crack before rupture" – this only should refer to quenched and tempered steels which should be forbidden.

- Not yet solved is the price decay (Fig. 1) of the offers, also in bridge construction, so that all involved persons complain insufficient quality as per a study of BAST (Federal German Institute of Road Research) is shown (Fig. 2). Other concepts of awarding are required, such as total price consideration and total responsibilities from the turning the first rod to the demolition of the bridge. Only progressive technical standards and web-connected total quality management systems are not enough.

In the meantime these construction methods are established in modern technical standards (section 2). Nowadays the qualified civil engineer chooses out of a multitude of possibilities with the desired target of minimising the total costs for planning, erection, use until demolition.

- 1886 first patents
- 1927 first prestressed concrete bridge (Alsleben)
- 1943 Prestressed Concrete Regulations (Prof. Rüsch)
- Construction Boom after the Second World War
- Problems – Back Kicks [2] - Improvements of Regulations
 - like coupler joints
 - sensitive, quenched and tempered steel
 - problems with grouting
- nowadays 1 million tons of prestressing steel are installed worldwide per year

Table 1: History of internal post-tensioning

- 1936 first externally PT-bridge over the railway station in Aue (Design by Dischinger)
- since 1960 bridges are built abroad with segmental construction method with external tendons
- since 1980 external tendons are applied for bridge strengthening
- since 1990 bridges with box section are post-tensioned externally in order to improve the quality in Germany
- 1998 in Germany the Guide Line for Concrete Bridges with external post-tensioning was introduced [10, 11]
- 1999 the "Recommendation for Post-Tensioned Segmental Bridges with External Tendons" [12] replaces the conservative DIN 4227, part 2.

Table 2: History of external post-tensioning

Design experience necessary,

- Tendons following well the moments distribution line
- High requirements to the quality, especially during grouting
- Concentration of the vent hoses at the road surface is disadvantageous
- Low production costs
- System reserves available
- Expensive maintenance

Table 3: Situation of tendons with bonded post-tensioning

ADVANTAGES

- Webs free of tendons, low weight, high quality, no vent hose
- Tendons with high-level corrosion protection even in cracked area "Structural Concrete" mean higher tensile strength in the concrete resp. broader cracks are allowed [10]
- Controllable
- Restressable, strengthenable
- Exchangeable (Single strand)
- Economic construction system in segmental construction
- Clear friction relations, also at the deviators
- In Germany higher tension 0,7 fpt is allowed due to better corrosion prot.
- Low fatigue stress resulting from traffic
- Installation possibly independent from atmospheric influences

DISADVANTAGES

- Can be damaged, exposed to atmospheric influences
- Protection against fire and resistance to vandalism to be clarified
- Reserve is missing due to bond, full loading action on the anchorage
- Yield point almost not exploited, therefore high quantities and costs for ordinary reinforcement (however good crack distribution)
- Small inner lever arm of inner forces
- Expensive anchorages and deviators
- Problems with place in case of large spans
- Additional stresses in the structure at transfer of forces
- Prestressing steel does not contribute to crack distribution
- Critical assembling procedure

Table 4: External Post-tensioning

- Different loading cases in construction phase and reconstruction phase are easier to manage with internal PT-tendons,
- Residual load bearing capacity in case of failure of external tendons (for ex.: fire)
- Possibility of a subsequent strengthening with external tendons

Table 5: Advantageous combination of internal and external tendons (mixed construction)

2 Technical Standards

The technical standards were updated and adjusted to the Eurocodes. The national structure of standards in steel and concrete construction is shown in fig. 3 which after an introduction time of 2 years replaces the former DIN 1045 and 4227. In the bridge construction the BMVBW (German Federal Ministry for Transport and Construction) combines the different new Standards with Supplementary Technical Specifications (ZTV-K) in 4 documents "load assumptions, concrete bridges, steel bridges and composite bridges". The admissible stress in PT-steel in bridge construction due to total losses after creep and shrinkage amounts to $0,65 f_{pt}$ (formerly $0,55 f_{pt}$) without bond to $0,7 f_{pt}$.

Beside the DIBt (German Approval Authority) completed its approval guidelines for external tendons [13]. A European approval guideline exists as a draft [14], defining necessary requirements to the executing parties besides already known requirements to the material (is still being discussed controversially).

3 Design criteria and examples for external PT-tendons and tendons with and without bond

3.1 Prestressing Steel

Due to delayed steel ruptures of quenched and tempered steel "Sigma" and "Neptun" the reputation of the prestressed concrete construction was damaged. Older structures had to be checked [15]. The overreaction of the authorities also affected new constructions, by "Crack before rupture" and increasing the required amount of reinforcing steel. Today it is known that in bridge construction sufficient minimum ordinary reinforcement rather robustness reinforcement always exists and the nowadays approved prestressing steels (see as well prEN 10 138) do not react oversensitive to stress corrosion.

3.2 Anchorages

National and international regulations as well as the optimizing of the multifunctional anchorage parts led to an approach of the different tensioning systems, see VSL, Freyssinet, SUSPA, DSI etc. They are available for all kinds of concrete such as LC, NC or HPC.

The competition only concerns the search for the cheapest supplier and the better installing possibilities. The quality of the components is sufficient.

In case of external post-tensioning the transfer of prestressing force is mostly performed via brackets/disks. Additional stresses as shown in Fig. 4 must be proven. Presently investigations are made at TU Aachen, Karlsruhe and Leipzig [5]. It is necessary to take care for requirements of concreting and tensioning. Even a prestressing possibility in the free length is given by special intermediate anchors. A possibility of removal must be feasible and proven (Section 4).

3.3 Free Length

Regardless whether bar, strand or wire, the active corrosion protection with cement grout acc. to EN DIN 445-447 at the PT-tendons with bond is the most durable one, provided that the quality of execution is okay. For that purpose presently specially continuous quality systems are introduced (Section 5).

It is furthermore possible to improve the protection using thickwalled ribbed PE-PP sheathings [16]. The friction behaviour is thus getting more favourable. Electrically isolated tendons are the most recent improvement.

In case of external PT-tendons the factory-made corrosion protection which is independent from atmospheric influences [17], mostly with corrosion protection compound is most common. Requirements to flowability, de-oiling, water absorption, saponification, micro-biological resistance, coefficient of expansion are not yet definitely defined. The BAM in Berlin is working on that target.

The requirements to PP/PE-tubes are neither clarified completely.

Further investigations are made in this connection for example at TU Aachen.

Special attention must be paid to the influence of temperature.

In principle the system of the free length should be robust, UV-resistant and show up a certain resistance to fire and vandalism, which favours the cement injection.

The limitation of external tendons [10] up to a length of 200 m and 3 MN capacity must still be reconsidered.

3.4 Deviators

In the region of deviators of internal PT tendons the friction corrosion affects especially the fatigue strength, even more under corrosion. Corresponding fatigue proofs must be carried out. PP/PE-sheathing behaves more favourably than metal sheathings.

In case of external PT-tendons the new approval guide line [13] requires for the saddle construction a stressing and a long time test (Fig. 5), possibly with an unintentional angle deviation of 3°. Clear frictional relations (internal and external friction) must be proven. Possibly a robustness test may become necessary, realised by drawing the external tendon over a sharp edge with a bend of 2°. When exchanging single strands, even under transversal pressure, the danger of wearing the strand through PE must be proven.

3.5 Coupler joints

The a.m. tests must be carried out in the same way for coupler joints. BMVBW only allows a 70 % coupling in one section.

3.6 Example [18]

Besides the strands 0,62" St 1770, also wires Ø 7 St 1470/1670 and bars max. Ø 36 St 1080/1230 are available on the market. The biggest approved bundle tendon is 27 x 0,62".

Of special interest are the different external PT-tendons

- With double extruded PE-coating, embedded in corrosion protective compound in a band of 2 or 4 strands (CMM-VT, CMM-Dywidag, VBF), Fig. 6
- PE-sheathing, grease, prestressing wire (SUSPA)
- PE-sheathing with or without cement grout, monostrands (DYWIDAG, BBV) Fig. 7
- PE-sheathing with spacer, cement grout and bare strand (DYWIDAG), Fig. 8

In order to reduce the disadvantages of external tendons, more and more clients have tendons without bond placed in the concrete cross section in order to be able to carry out partial tensioning immediately after partial hardening of concrete. In tests the exchange is tested successfully.

In transverse direction monostrands are applied; they are also exchangeable due to special anchorages. So far it was not possible to accept the free tendon position of these monostrands just as applied in Austria.

The strands can be exchanged for example by galvanized strands, using king wire couplers. Care must be taken to the reduction of the strength of the concrete compressive strut considering the soft PE-sheathing.

4 First application of new post-tensioning systems for concrete bridges with external tendons

Since the end of 1999 the BMVBW requires for Tenders of externally post-tensioned concrete bridges the application of approved and proven tendons. Testing sites are made available where it is possible to install, stress, inject, to tension and to remove new tendons. Lately bbv and DYWIDAG tested their new, approved, external tendons 16 x 0,62" at the site "Einzinger Bach". Tendon length is 145 m with 5 deviators.

It was interesting to learn that also cement grouted bundles of strands can be retensioned over 10 % without problems. It resulted a crack distance in the mortar of about 4 cm with crack widths of < 0,03 mm. Releasing for exchange was neither critical. Between the deviators, the strands were cut individually with the "Flex" one after the other and care was taken that the bond of these strands was disturbed over large lengths (self-disturbed). Due to this the bundle was released mildly and smoothly.

5 Improvements of the Grouting Technology

Grouting technologies with cementitious mortar are known and can be controlled [19]. The quality of execution, however, is not always sufficient. The construction product injection mortar is regulated in the "Building Rules List A" with the Technical Regulations DIN EN 445 to 447 with supplements and the coincidence proof ÜZ (Certificate on the base of recognised Testing Authority). In the MÜTVO (Decree about Control of Activities with Construction Products and Methods) the "Länder" layed down that the injection of cement mortar into ducts must be supervised by qualified Supervision Authorities acc. to § 24c, para 1, No. 5 MBO.

Therefore, the Construction Supervising Authority and Companies of Prestressed Concrete Structures worked out a continuous Quality Assurance System, starting with internal control and including external control at site. The checking lists extend from planning, work preparation, suitability tests, quality control including 10 % of the externally controlled injection procedures at site. An improvement of the injection quality may be expected on the base of this highly efficient quality assurance system.

6 Costs

It is quite understandable that external tendons (see table 6) –even in case of using large quantities- must be more expensive than conventional tendons with bond. The price is about 50 to 100 % higher. Also the smaller inner lever arms result in an increase of masses, however, the higher admissible stress (70 % instead of 55 % GUTS in Germany) yields a certain reduction. Only crossbeams for tendon anchorage and deviator saddles raise the price of the structure between 2 % and 4 % in each case. The a.m. BMVBW-decree [10] makes possible a further steel saving due to higher admissible tensile stresses in the concrete during construction and the final phase. Thinner webs reduce the concrete masses and the weight.

Despite that, however, externally prestressed structures (except segmental structures) will be more expensive between 5 % and 20 % than conventional prestressed concrete structures.

The client expects savings from the yearly maintenance, costs of which amount to yearly 1 to 2 % with respect to the value of a new structure as well as a higher flexibility in case of possible necessary strengthening (Life-Cycle-Cost-Analysis; Total Cost Consideration, from the start until the end).

Strand tendon with bond	100 %
Type W (monostrand, mortar, PE (Fig. 8)	220 %
CMM band or similar (Fig. 6)	200 %
Stainless steel plated strand	180 %
Type MC (strand, mortar, PE, Fig. 7)	150 %
Strand/wire, corrosion prot. compound PE	140 %
Strand bundle without bond in cross-section	120 %
Monostrand	130 %

Table 6: Cost comparison (L = 45 m, two deviators)

7 Execution

Now our attention must face the construction procedure where the external tendons can be assembled only after the demoulding procedure, contrary to the already installed tendons with bond in the cross-section (Table 7) and so the former are on the "critical path". Therefore, the partial post-tensioning to avoid cracks can only be applied late (advantage for the mixed construction). The required place for the stressing jacks must be taken into account, possibly with curved stressing chairs or even with stressable intermediate anchorage.

Internal post-tensioning	External post-tensioning
Formwork	Formwork
Reinforcing	Reinforcing
Install sheathing/duct	-
Concreting, hardening	Concreting, hardening (tensioning against shrinking stresses) partial demoulding (critical path)
	Assembly of external tendons (critical path)
Partial post-tensioning, final tensioning	Partial post-tensioning, final tensioning
Dismantling	Dismantling

Table 7: Different Construction Process for internal and external post-tensioning

Due to the weight, the transportation and the assembly advantageous pre-fabricated tendons are limited in length and size (cranes!). With respect to large spans the space inside the box-section is small for external tendons. It is recommended to reconsider construction and removal phases by using tendons without bond.

Further advantages and disadvantages as well as relevant criteria were already mentioned at different places of this paper.

Not yet finally clarified is the exactitude of the position of the external tendon for example at the deviator as well as the kind of measuring process (laser, wire). 3° of inaccuracy still is not sufficient [6].

8 Forecast

Therefore, I have to point out that it does not exist any remedy or generally correct solution for each individual case. The creative civil engineer is demanded to find the correct, reasonable (in price) and durable solution with external or/and internal post-tensioning with and without bond.

The requirements to external tendons and constructive solutions for the pattern of tendons are continuously reconsidered and improved. The target must be a further improvement of the durability of prestressed concrete construction as well the flexibility for strengthening of structures. A reasonable combination of external and internal tendons will be successful. A high quality of execution by specialised companies is required. For this stands CEP (Convention Européenne pour la Précontrainte), an organization of firms selling post-tensioning systems. As already mentioned in the beginning of this paper, this presupposes, that the necessary price is paid by the client.

9 Literature

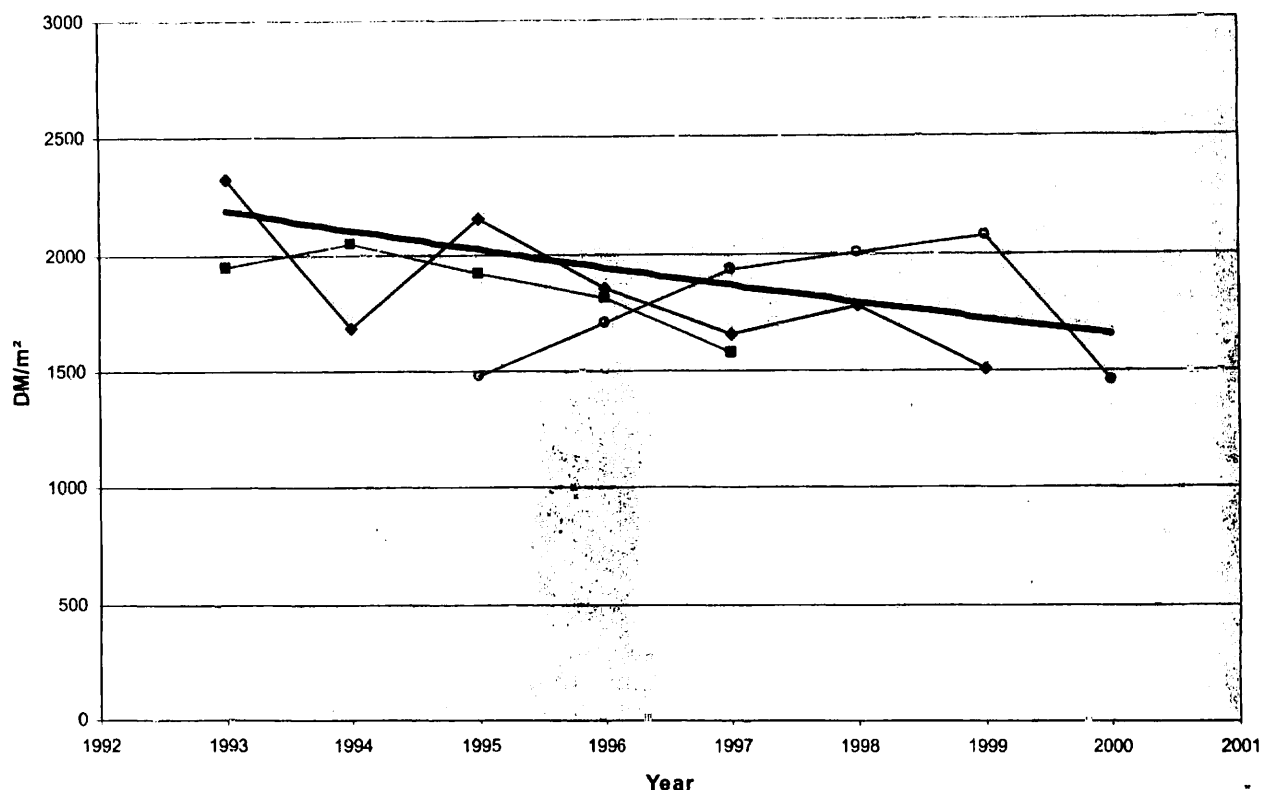
- [1] Current questions concerning prestressed concrete; post-tensioning systems from the beginning to the worldwide application of today; Österreichischer Betonverein, issue 31/Dec. 1997
- [2] Jungwirth, D.: Prestressed Concrete Construction – Yesterday, today, tomorrow, prospects to the future, Betontag 1999; Beton- und Stahlbetonbau 95, 2000, issue 1
- [3] Eibl, J.: The external post-tensioning in Germany – Development and Prospects. Workshop External post-tensioning and segmental construction, University Karlsruhe 1998
- [4] External or internal post-tensioning? Workshop "Deutscher Betontag, Berlin 1999, Jungwirth: Beton- und Stahlbetonbau 94, 1999, issue 8
- [5] Exchange of experience on prestressed concrete bridges with external tendons; BMVBW – Standfuß, Oct. 2000: Bonn
- [6] Baumann, Th.: Prestressing of Bridges, Application of internal and external tendons; Beton- und Stahlbetonbau 95, 2000, issue 11
- [7] Research, Development and Application: Concrete Construction 1999 to 2001; Zilch; Bauingenieur, University Munich
- [8] External post-tensioning, "Further Training", issue 45, Sept. 2000, Austria; Vereinigung für Beton- und Bautechnik
- [9] Iványi, G., Buschmeyer, W., Winter, H-G.: Quality in Concrete Construction as well as Discussion Meeting BMVBW; "Bautechnik" 78 (2001) issue 1
- [10] BMVBW-decree, July 1999: Guideline for concrete bridges with external tendons, Dortmund Verkehrsblattverlag, No. 17/1999
- [11] Standfuß, F., Abel, M. and Haveresch, K.: Explanations to Guideline for concrete bridges with external tendons; Beton- und Stahlbetonbau 93, 1998, page 264 - 272
- [12] Recommendations for segmental precast bridges with external tendons, 1998 Deutscher Betonverein Berlin
- [13] Guide Line for Aptitude Test of post-tensioning systems for external post-tensioning. Information of DIBt, 5/99
- [14] ETAG, Jan./Aug. 2001: Guideline for European Technical Approval of Post-Tensioning Kits (Post-Tensioning Systems) for Prestressing of Structures

- [15] Wölfel, E.: Single prestressed concrete elements possibly jeopardised by delayed tensile steel ruptures ; Beton- und Stahlbetonbau 87, 1992, issue 6
- [16] fib: Corrugated plastic ducts for internal bonded post-tensioning; Technical Report, Bulletin 7, 2000
- [17] fib: Factory applied corrosion protection of prestressing steel; State of art report, Bulletin, 11, 2001
- [18] Different general approvals by Construction Authority DIBt; bbv, DYWIDAG/DSI, SUSPA, VT-CMM and VBF Ratingen
- [19] Engelke P., Jungwirth, D., Manns M.: Regarding Inj. Technology at Tendons > 1500 kN, Informations of DIBt, 1979, No. 6

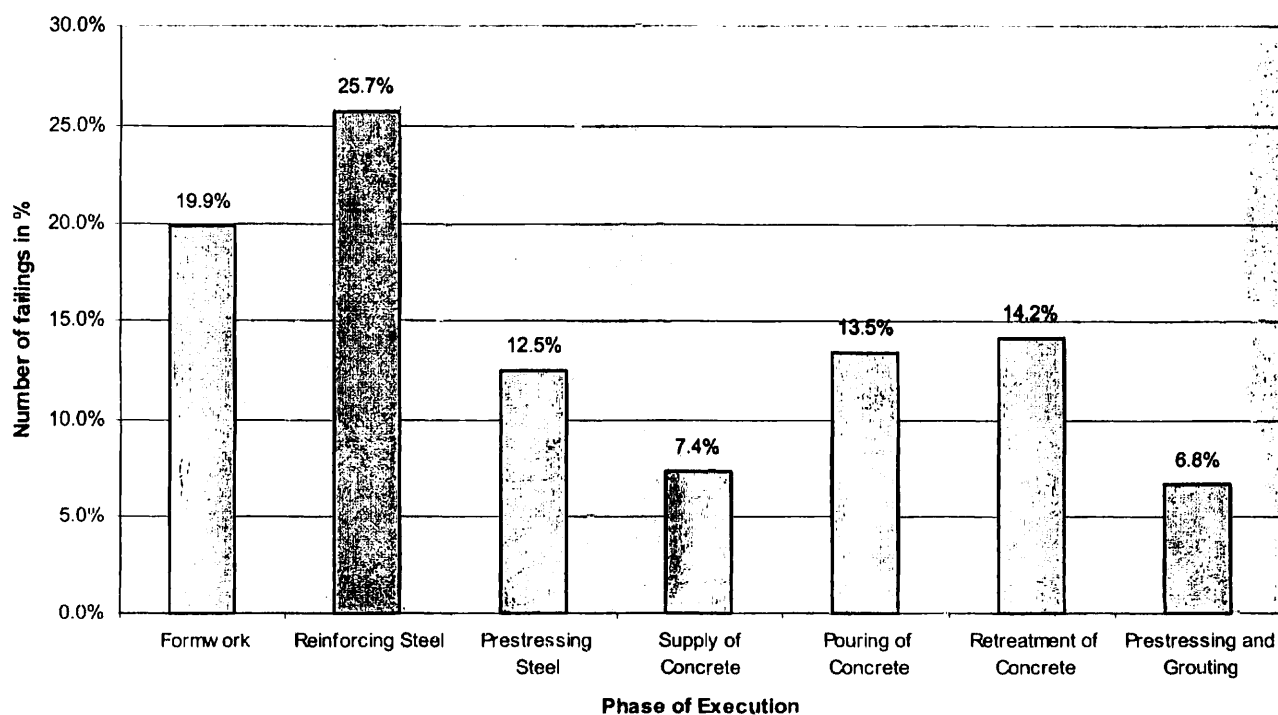
Figures:

- 1. Price decay at Bridge Construction
- 2. Distribution of failings to the execution phases acc. to BASTS [9]
- 3. New German Standards Family
- 4. Load transfer via prisms and/or console/disk
- 5. Saddle test [13]
- 6. Double extruded strand systems
- 7. DYWIDAG MC
- 8. DYWIDAG W

Fig. 1 Price Decay at Concrete Bridges in Germany



**Fig. 2 Distribution of the failings with respect to the execution phases acc. to BAST
Federal German Institute of Road Research [9]**



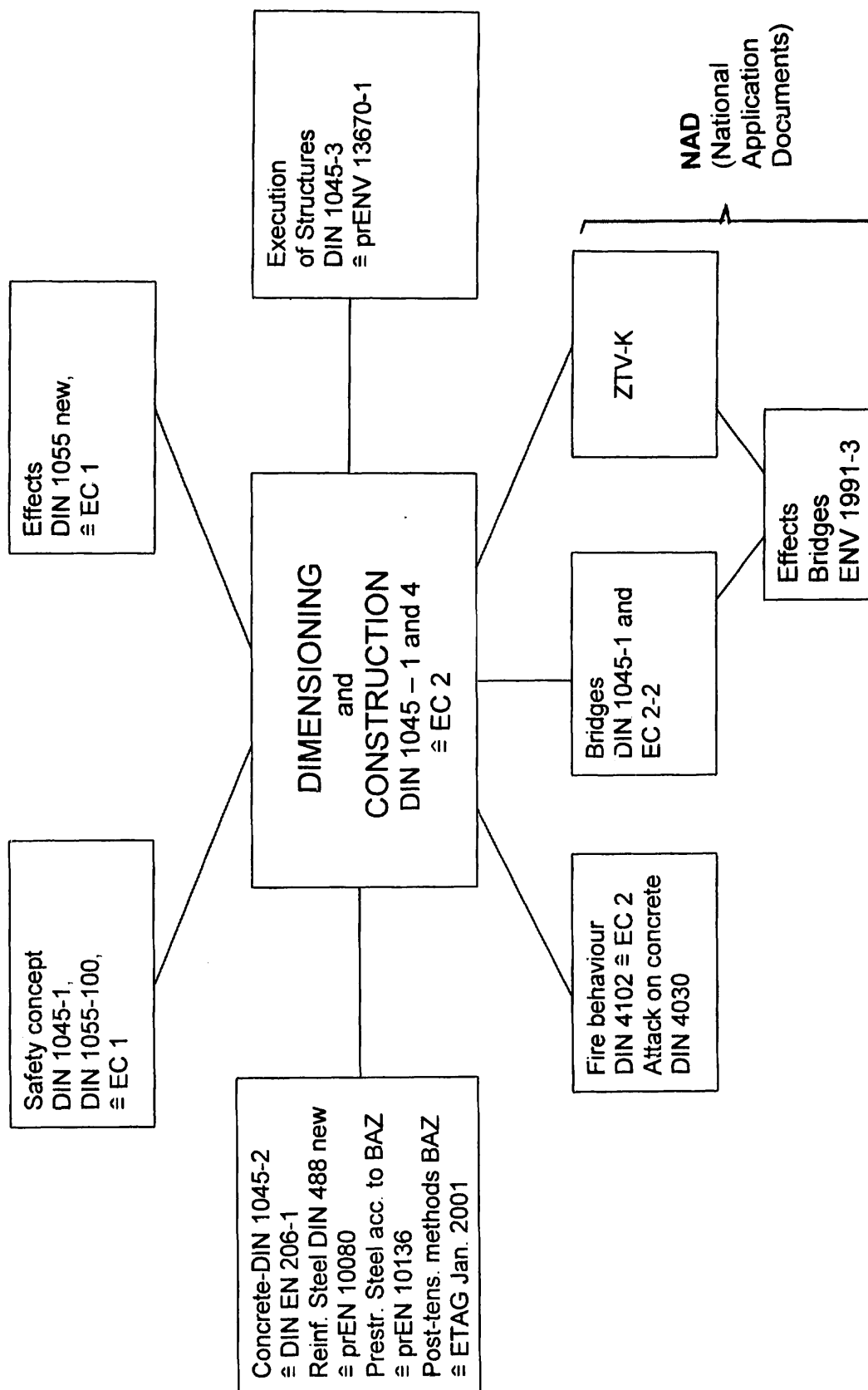


Fig. 3: New German Code Family

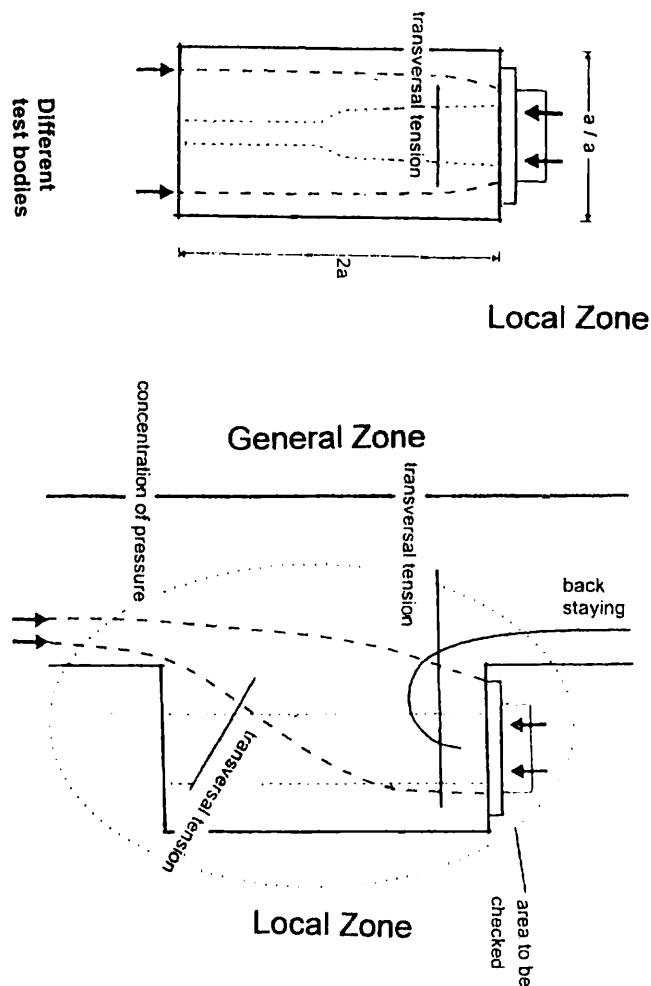
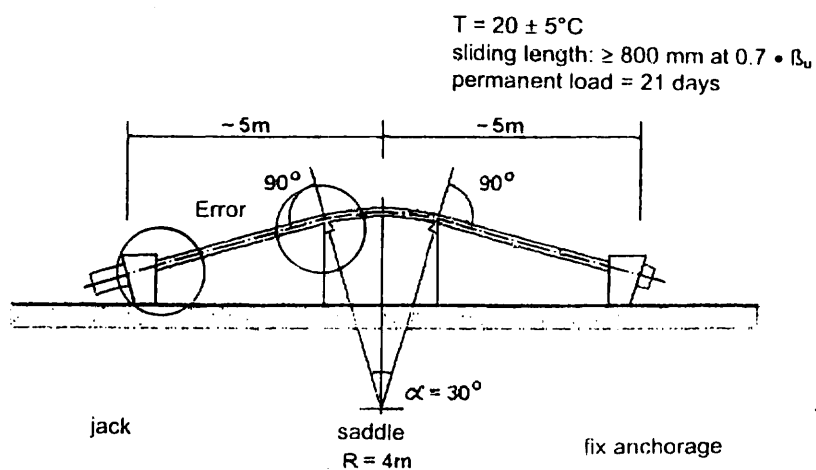


FIG. 4



Dimensions of the test equipment

FIG. 5

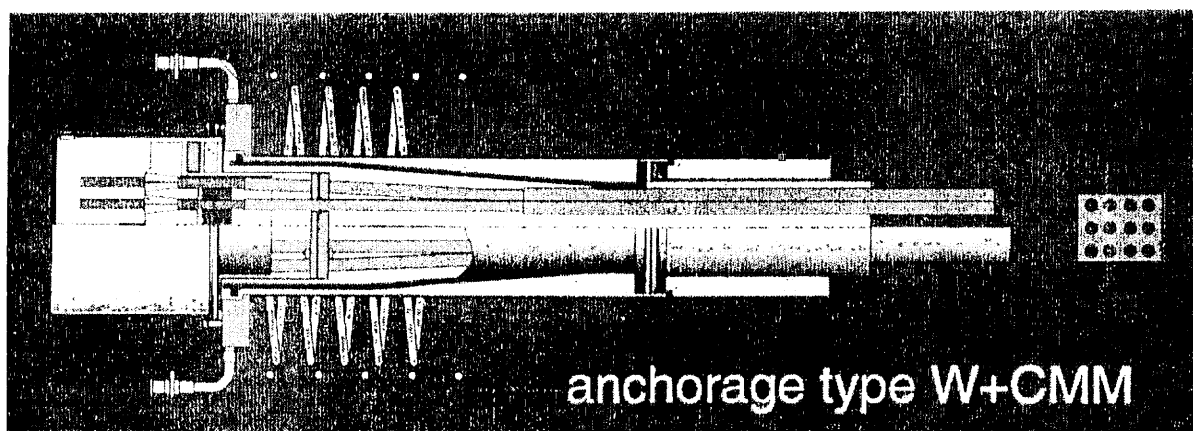


Fig. 6 Double extruded strand systems

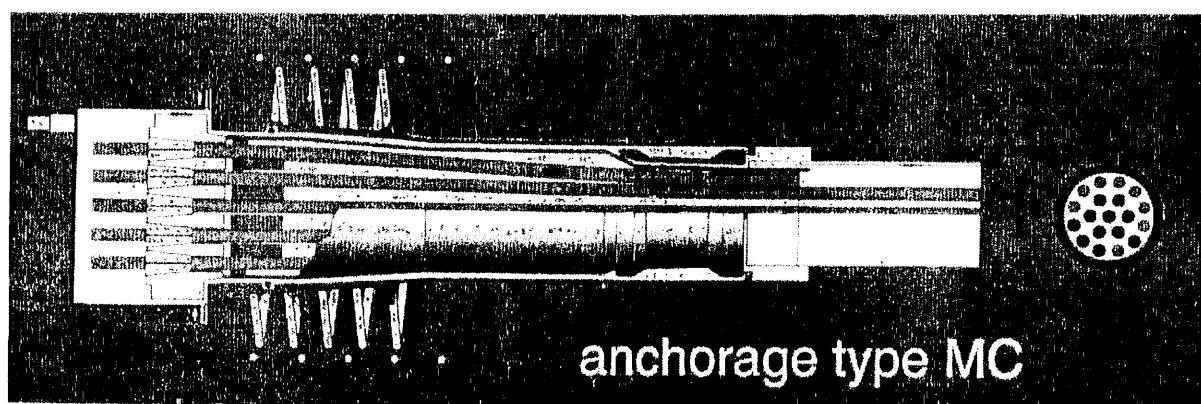


Fig. 7 DYWIDAG MC

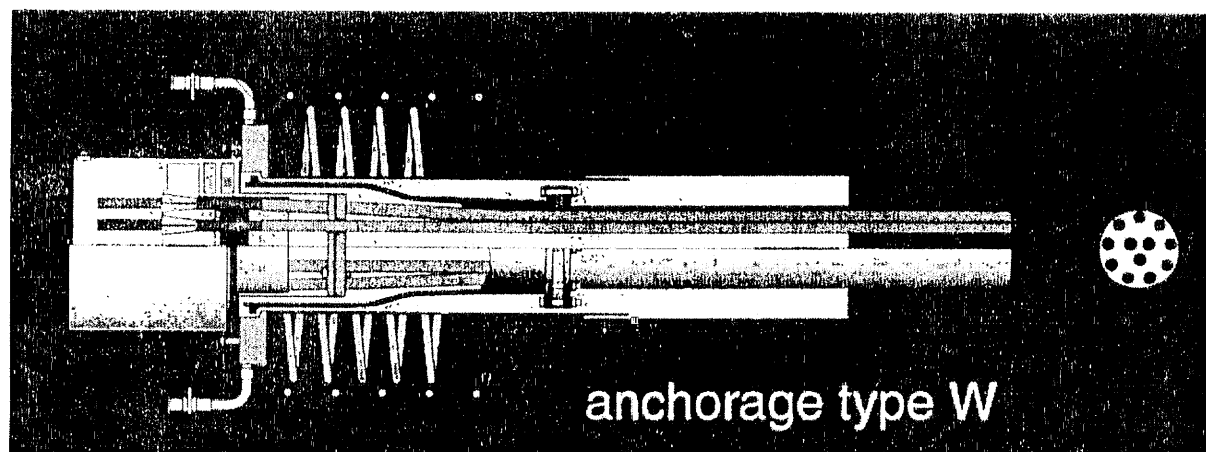


Fig. 8 DYWIDAG W

Status of durability of post-tensioned tendons in France

B. GODART

Laboratoire Central des Ponts et Chaussées (LCPC)
Division Fonctionnement et Durabilité des Ouvrages d'Art
Paris, France.

1 The beginnings of prestress in France

The oldest structure prestressed by post-tensioning in France is the experimental arch of the bridge of Veudre on Allier river. This arch built by Eugene Freyssinet in 1908 and preserved until our days in the property of a private individual at Moulins, has a span of 50 m, a height of 2 m and is supported on abutments linked by a tie prestressed by means of several hundreds of post-tensioned wires which introduced a compressive force ranging between 25 and 30 MN. Openings of windows operated in 1993 made it possible to note the good behaviour of the tendons which were simply laid out in grooves filled with sand and sealed by a hydraulic mortar. It shall be noticed that the whole tie had remained hidden in the ground during several decades without any waterproofing on the surface of the tie.

If Freyssinet built other bridges before 1928, it is however this date which marks the official beginnings of the prestress in France with the patent taken out by Freyssinet on October 2 and November 19, 1928 [Grattasat (1978)]. But it is only after 1939 that the real industrial beginnings of prestress may be witnessed with the supply of high-tensile steel by the steel industry, and during this year 1939, with the development by Freyssinet of anchoring by conical friction and of the prestressing jack with double effect. Unfortunately, because of the second world war, the development of prestress was going to reach a plateau, and one notes only the construction of three bridges prestressed by post-tension according to the Freyssinet process: the bridge of Luzancy on the Marne river (a portal leg bridge with a 54 m span built using small prefabricated segments assembled by filled-in joints and three-dimensional internal prestressing), where the prestress of this bridge appears in good condition and where the thrust at the base of the legs was adjusted in 1997, and the two slab bridges of Longroy and Elbeuf sur Andelle.

This period was dominated by Freyssinet which took as much care in the structural design as in the choice of materials, and which took care personally of the great quality of the construction of its bridges. He was thus aware that the prestressed concrete was a demanding technique as regards quality, and the evolutions which followed this period, evolutions dependent on the dissemination of this technique, even on its popularization, were going to point it out.

2 Prestressed concrete bridges of the period 1946-1960: the rise of the VIPP

2.1 Corrosion defects in VIPP

It is only at the end of the second world war that the large prestressed concrete bridges developed, in a non-regulated context, the euphoria of the reconstruction, and during a period marked by cement and steel shortage. The first builders oriented themselves towards the prefabrication of beams, and this gave place to the construction of simply supported spans, and then to viaducts with simply supported spans made up of beams prestressed by post-tension, more known under the name of VIPP (Viaducs à travées Indépendantes à Poutres Précontraintes).

It is primarily this generation of bridges which concentrates the durability defects of prestress in France. The enthusiasm which prevailed when the first generation of prestressed structures was being built was reflected in a total confidence in full prestressing (i.e. lack of cracking) and by a belief that concrete under compression is watertight [HA, SETRA,TRL, LCPC (1999)]. This is why a number of details appeared such as:

- lack of waterproofing (deck waterproofing only became mandatory after 1966) (Fig. 1);
- lack of sealing behind tendon anchorages;
- piecemeal construction resulting in a large number of unprepared construction joints which could give rise to cracking due to restrained shrinkage;
- use of unprotected transverse tendons in grooves in the deck;
- use of sheaths made from bitumen-coated Kraft paper wrapped around the tendons which made grouting impossible (metal sheaths were not used until the late 1950s);
- large numbers of tendons per span in older structures with deck anchorages (e.g. typically up to 15 out a total of 20 to 25) increasing the number of points of possible water ingress;
- use of lead lined steel ducts between 1950 and 1975 to reduce friction sometimes caused tendon corrosion as a result of bimetallic action between the ducts and the steel;-
- use between 1950 and 1970 of prestressing steel which was susceptible to stress corrosion, which gave rise to the possibility of brittle fracture;
- lack of provision for drainage. (It is often beams located under drainage channels which are most affected by corrosion (Fig. 2);
- leaking expansion joints (Fig. 3);
- insufficient concrete cover to the reinforcement, resulting in corrosion of the reinforcement and spalling of the concrete giving easier access to the prestressing tendons for aggressive agents.

Design defects are therefore mainly linked to construction defects or unsuitable techniques, or to the use of low durability materials. But the construction defects have also a great part in the development of the corrosion of tendons. The two principal construction defects responsible for corrosion are poor waterproofing and incomplete grouting of the prestressing ducts (Fig. 4).

It is common to find thin waterproofing layers which do not extend beneath the footways. It is also common to find partly empty or sand-filled ducts in structures built before 1960 when grouts used to contain sand. Grouting techniques have also been poorly applied, and evidence of blockages has frequently been found.

In addition to these defects there is poor sealing of end anchorages, deck anchorages and transverse anchorages.

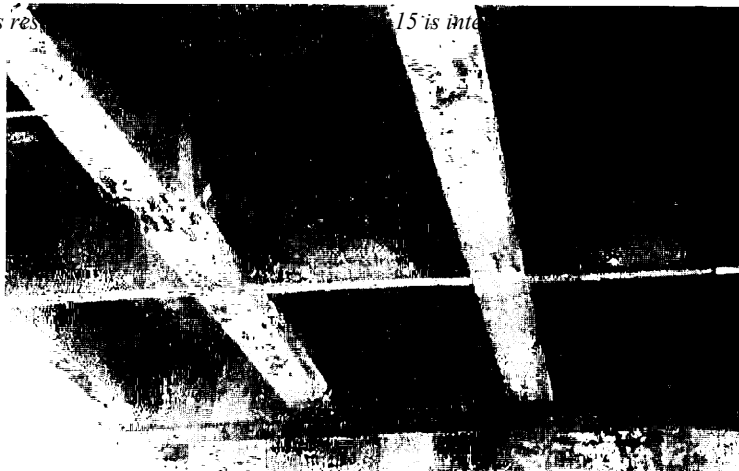


Fig. 1: Damages at toes of VIPP beams due to poor waterproofing



Fig. 2: Condition of a transversal tendon located in the slab below the drainage channel.



Fig. 3: Disorders due to leaking expansion joints.



Fig. 4: Corroded and broken wires in soffit of VIPP type bridge.

2.2 General remarks on corrosion

Corrosion of prestressing reinforcement can be divided into three main types:

- electrolytic corrosion (rust, small pits etc)
- stress corrosion
- hydrogen embrittlement corrosion.

The third form of corrosion has never been found in any of the surveys on French bridges. Likewise no fatigue fractures have so far been discovered in existing structures. Brittle fractures attributed to stress corrosion cracking have been found in some cases. After an investigation by J P Persy it appears that this only affects quenched hot rolled wires containing more than 0.1% of Cu. This type of prestressing steel was only used from 1950 to 1965.

Conventional corrosion is by far the most widespread form. Most of corrosion defects is caused by water which seeps through cracks, leaking seals or zones of porous concrete, and flows through the network of ducts which have been grouted to a greater or lesser extent. Experience shows that problems due to corrosion are distributed randomly, in accordance with the random nature of the movement of water within a structure. In general, relatively few chlorides have been found in steel corrosion products in France.

Experience shows that neither the steel sheath, nor even a well compacted grout can form a sufficiently tight barrier against the percolation of an aggressive water through the more or less porous concrete of a beam. This was particularly well highlighted during the expertise of a viaduct made up of simply supported box girders of good quality, located in a port, whose bottom of the box girders was immersed in sea water only when very strong tides occurred, and where the corrosion of tendons at mid-span was such that the bridge had to be rebuilt. This was also observed on another viaduct whose prestressed piers bathed permanently in sea water.

Although defective grouting of the ducts is a primary condition for the development of corrosion, this condition on its own is not sufficient to trigger the reaction. It has already been found that ungrouted tendons in dry structures located in regions with a continental climate have not corroded at all.

In addition to the question of the physico-chemical condition of tendons conservation, another significant question is to know the mechanical state of conservation of these tendons, and in particular to know if the tension in a healthy wire could be maintained during time. A partial response to this interrogation is provided to us by B. Tonnoir of the Laboratoire Régional des Ponts et Chaussées in Lille : since the 1980s, this one indeed carried out many tension measurements on tendons of VIPP using his crossbow, and he evaluates that the current residual tension of wire of diameter 5 and 7 mm used in the 1950s remains inside an interval ranging from 80 to 90 kg/mm², for an initial tension of the wires equal to 110 or 115 kg/mm² and a final tension estimated at the time of construction to be equal to 85 kg/mm².

2.3 Conclusions on the risks presented by VIPP

A recent investigation [Trouillet (2000)] shows that, over a total of 720 VIPP built before 1966, about fifteen were already demolished because of corrosion of their tendons. Very fortunately, until now, we did not have to deplore any sudden collapse of VIPP spans, thanks in particular to a policy of monitoring and inspection which made it possible to detect sufficiently early the bridges presenting some risks.

And yet, the simply supported spans prestressed by post-tension are among the structures most difficult to monitor. Indeed, VIPP, and especially those of the first generation, are

structures which comprise a small percentage of passive steel and which resists only thanks to the integrity of their prestressing. Two principal types of rupture are to be feared: a rupture by bending and a rupture by shearing force.

As regards bending, the first cracks appear when the deficit of prestressing is already very significant. If we consider only one beam, it is necessary to have lost 40 to 50 % of the longitudinal prestressing to see a bending crack appearing under the current live loads. When these live loads are removed, the residual prestress closes again the crack and this one becomes almost undetectable with the eye. One cannot thus base oneself on the absence of bending cracks to deduce that the bridge is healthy.

If only one beam is strongly damaged, and if the transverse prestressing is in good condition, the other beams must be able to take again the loads supported by the failing beam. If all the other beams are strongly damaged, then the rupture of a beam involves the total ruin of the bridge. Between these two extreme cases a panoply of intermediate cases may exist which are a function of the degree of conservation of the prestress of the slab, of the crossbeams, and of each beam. But generally, in these cases, it seems that a mechanism of ductile ruin corresponding to a progressive damage of the structure may develop.

As regards shearing action, the situation is notably different, because, in the zones close to beam ends, the risks of brittle fracture can exist. Indeed, under the current cases of loading, the resistance of the cross sections to the shearing force in these zones can be ensured thanks to only the concrete strength, and without the help of the prestressing tendons which can be strongly corroded. It is enough whereas an exceptional loading case occurs so that a sudden failure of the beam cross section occurs, if the residual section of the tendons is insufficient to support the efforts released during the cracking of the concrete.

3 From 1960 to 1980 : the rise of the box girder bridges with internal prestressing

If the first prestressed concrete bridges built by balanced cantilevering appeared in Germany (Balduinstein and Neckarrems bridges built in 1950-1951 by Dr. Finsterwalder, then Worms bridge on the Rhine and Coblenz bridge on the Moselle river in 1952-1953, [1]), those bridges developed in France from 1955 with the Chazey bridge on the Ain river (3 spans of 41- 57 and 41 m long), then with the Beaucaire bridge (5 spans of approximately 86 m long) built in 1957-1958, and finally the Savines bridge carried out in 1958-1960 (11 spans of 77 m and 2 sidespan of 38,5 m long) accompanied by the Riou-Bourdou bridge. These four bridges were built by the Grands Travaux de Marseille Company, on the basis of a prestressing by tendons, and a casting in place of the segments.

These four bridges, which comprised an hinge without any thrust at the key of each span, are to our knowledge the only ones which were built in France on the basis of this static diagram of behaviour. Indeed, from 1962, the bridge decks built by the balanced cantilever method were made continuous with the removal of the articulation. It is also from this date that prefabrication was used in construction by balanced cantilevering, with the assembly of segments through matched and glued joints, the first bridge being the Choisy-le-Roi bridge on the Seine.

In 1970, the first serious alarm of pathology was given with the Chazey bridge which presented then, in a side span, several bending cracks whose opening exceeded the millimetre. After expertise, it appeared that the principal cause of these disorders came from a significant corrosion of the tendons. This bridge was demolished in 1972, then rebuilt.

Apart from the case of Chazey bridge, the lesson drawn from the pathology of the first generation of box-girder bridges (until 1975) shows that these bridges do not suffer from

corrosion of tendons; even if many gammagraphies or radioscopies carried out on these bridges teach us that grouting defects exist [Guinez (1997)], the corrosion of tendons was almost never revealed by the few openings of windows that were made; this is probably explained by the good condition of the waterproofing membrane which covers these bridges, as well as by a favourable design of box-girder bridges in view of tendons protection.

On the other hand, most box girder bridges built before 1975 by balanced cantilevering have had insufficient bending strength. This was indicated by cracks opening between the segments in the zones of contraflexure. One could have feared problems of fatigue in the tendons which crossed these open joints, in particular in the case of the bridges with prefabricated segments, therefore without reinforcements to be able to take a part of the fatigue efforts. Among the hundreds of windows which can have been made by the LRPC (Laboratoire Régional des Ponts et Chaussées) to measure tendon overstresses straight in the cracks, no window could highlight a failure of a prestressing wire by fatigue. Nevertheless, a majority of these bridges was strengthened by external prestressing to avoid the appearance of fatigue and of potential tendon corrosion.

4 After 1980 : the rise of external prestressing

Except four box girder bridges built with external prestressing at the beginning of the 1950s, external prestressing had been primarily used until 1980 for the strengthening of prestressed structures. From this date, this one experienced a spectacular development for the construction of new bridges, in particular under the impulse of Michel Virlogeux who worked with SETRA (Service d'Etudes Techniques des Routes et Autoroutes).

The interest presented by external prestressing was to facilitate the concreting of walls, to reduce the structures weight, to limit the prestress losses due to friction, and especially to be able to replace the tendons in the event of problem. It thus aimed at improving the quality and the durability of the bridges, and to facilitate their monitoring.

In 1990, the SETRA published a guideline [SETRA, (1990)] which made the synthesis of the knowledge accumulated during ten years; this document proposed recommendations for the design of the bridges with external prestressing, the construction detailing to facilitate the replacement of the tendons, and the regulations on their deviation and their grouting.

Before tackling the few recent problems encountered with external prestressing, first let us look into the case of the first four bridges built at the beginning of the years 1950: the Vaux sur Seine bridge (1951), the Port à Binson bridge (1953), the Villeneuve Saint Georges bridge (1953) and the Can Bia bridge (1953) [Chatelain (1984)].

4.1 The case of the four bridges with external prestressing of the 1950s.

4.1.1 The Vaux sur Seine bridge

The Vaux sur Seine bridge, build by the Coignet company, is a continuous bridge composed of three spans with respective length 15, 26 and 15 m, and a variable height (1,3 m on pier and 0,6 m at the key). It consists of two boxes whose webs and lower slab have a thickness of 0,10 m. For each box, the external prestressing consists of a belt of 4 tendons, each tendon having 30 wires with a diameter 5 mm, with a tensile strength ranging between 165 and 180 kg/mm², and with a final tension equal to 85 kg/mm² after relaxation. The tendons are anchored at their ends in massive crossbeams, and the tension is obtained by spacing of the bundles taken two to two; then a concrete block replaces the spacing jacks in the final stage. To prevent the corrosion of tendons which are left in the free air, they were

coated by a grease with a strong consistency, to the image of what was done at that time for the cables of the suspension bridges.

Thirty years after, a detailed inspection made it possible to note the absence of cracking in the concrete, but on the other hand the rupture by corrosion of some tendon wires inside the downstream box girder, at a place where the permanent ventilation of the interior of the box girder had been practically suppressed by birds nests installed in the ventilation openings. The corrosion created by condensations on the lower parts of the tendons being likely to worsen, it was decided to carry out an additional prestressing by external tendons. One of these tendons broke besides recently by corrosion, following a defect of protection at the anchorage.

4.1.2 The Port à Binson bridge

The Port à Binson bridge located close to Dormans, comprises 2 simply supported spans with a length of 40,5 m each, and consists of two side by side box girders having a constant height equal to 2,3 m. The thicknesses of the webs, of the lower slab and upper slab are respectively 12, 15 and 25 cm. In each box girder, prestressing is made of 2 x 4 tendons of 49 wires 7 mm in diameter, anchored to the ends of the beam. As for the preceding bridge, the Coignet company put in tension the tendons by spacing of the 2 bundles. When the scaffolding was removed, measurements of the tendons tension were carried out by LCPC with the help of the vibration method.

About thirty years later, the interior of the boxes was perfectly dry, well ventilated, and the tendons coated with a black product still bright are perfectly healthy; the structure does not present a crack.

4.1.3 The Villeneuve Saint-Georges bridge

The Villeneuve Saint-Georges bridge, built by the company Fourre et Rhodes, crosses the Seine in the south-east of Paris. It comprises 3 spans with length 41, 78 and 41 m, and is of the cantilever type with a simply supported central part of 39 m long. It has a width of 14 m, and consists of 3 box girders with variable height (6,91 m on pier and 2,36 m at the key).

External prestressing consists of a total of 102 helicoid tendons made of 193 wires with a diameter 4,1 mm, with a tensile strength ranging between 160 and 180 kg/mm² and tensioned to 63 kg/mm². The tendons are deviated using reinforced concrete rocker bearing provided with cast steel hinges in order to have a practically uniform distribution of the compressive stresses in the concrete under dead loads, and a reserve of compression of 5 kg/cm² under the most unfavourable live loads. The technology of anchoring of the tendons is similar to that used for the suspension bridges: cast steel sockets filled with an alloy of lead, antimony and tin; on these sockets 4 threaded rods are fixed which are used for the prestressing by jacks. They are just protected by grease.

In 1978/79, an abnormal vibration of certain tendons was observed during the passage of heavy vehicles, and some wire were broken. A favourable atmosphere for corrosion was prevailing within the box girders because the holes for inspection existing in the upper slab under the roadway were not tight. In 1980, a doubtful tendon is replaced, and all tendons are re-tensioned to their initial tension. In addition, the inspection holes are closed and new side accesses are bored in the webs of the boxes. A grease is applied to re-protect the whole tendons.

4.1.4 The Can Bia bridge

The Can Bia bridge is a communal bridge which crosses the Tech river in the department of the Pyrénées Orientales. Built by the company Sainrapt and Brice, it comprises only one span of 61 m long and has a constant height of 2,36 m supporting a roadway of 3 m and two pavements of 0,70 m width. The lower slab and web thickness is 12 cm, while that of the upper slab is 36 cm.

External prestressing is ensured by 58 tendons of 12 wires with a 7 mm diameter (STUP process). It introduces a uniform compression of 90 kg/cm² in the central cross section. Nine transverse diaphragms are used as deviators and some tendons are anchored in the upper slab. All the tendons are simply coated with a bitumen paint.

In 1960, a bridge inspection makes it possible to discover that about thirty wires were broken. In 1980, 56 broken wires were counted, that is to say 8 % of existing wires, and a cracking developing in the transverse diaphragms and at the ends of the box girder was noticed. The tendons present at the same time traditional corrosion by dissolution and stress corrosion; this last point was explained by the sensitivity of the wires to stress corrosion because they were rolled and quenched. The most damaged tendons were those anchored in the upper slab, because the absence of a waterproofing layer made it possible for water to infiltrate by upper sealings. Finally the bitumen paint was only partial and did not surround each wire.

This bridge was definitively closed to traffic in the year 1984, and then demolished.

4.1.5 Conclusions on these four cases

In spite of the strongly innovating character of these bridges at that time, three bridges out of the four are always in service. The possibility of readjusting the prestress if necessary (except Can Bia) and the possibility of replacing the tendons in the event of damages (case of Villeneuve Saint-Georges) are significant advantages with respect to the durability of the structures.

If the case of the stress corrosion is put aside, the durability of tendons external to the concrete and well coated with grease is ensured provided that these tendons are in well ventilated box girders. When they are in a wet atmosphere, generally due to bad constructive details (lack of ventilation, absence of waterproofing, non tight inspection access...), wire failures caused by corrosion occur and the durability of the tendons is lowered.

4.2 Three recent cases of failure of external prestressing tendons

Let us look now into three cases of tendon failures which occurred recently in box girder bridges; it is chronologically the bridge on the Durance, the Saint-Cloud viaduct and the Rivière d'Abord bridge.

4.2.1 The bridge over the Durance

The bridge over the Durance is a prestressed concrete bridge built by incremental launching and brought into service in 1986, and which allows A 51 to cross the Durance river. It has an overall length of 288,60 m and comprises 6 spans with the following lengths : 41 m – 4x 51 m – 41 m. It is composed of two parallel box girders with a total height of 2,57 m and a width of 5,50 m. The width of the upper slab of the box girder is 10,95 m.

The prestress, entirely external to the concrete, is made up for each box of 32 tendons 19T15 (19 strands with a 15 mm diameter). Those are strands located inside a HDPE (High Density Polyethylene) duct grouted with cement.

In 1994, a team of the LRPC of Aix en Provence discovered the rupture of a tendon having an approximate length of 250 m, inside the downstream box girder; this one was broken right in front of its anchoring within the crossbeam on which it is anchored, and the energy released by the tendon projected it towards its opposite end (Fig. 5). The expertise conducted by the LRPC emphasized the following points:

- the rupture of the tendon is in the form of a bevel, which corresponds to a failure of the various strands in the same horizontal plane (Fig. 6);
- the sheath was empty of grout and partially filled with water over a 2,50 m length in front of the anchoring plate;
- among 109 analysed wires, 21 wires presented a strong corrosion by dissolution and 88 wires presented a striction and had thus broken during the final rupture;
- the pH measured on the water collected in the anchoring caps was high, which leaves us to think that a corrosion in a hyperbasic medium had been able to occur;
- the 33 gammagraphies carried out on the broken tendon confirmed an absence of voids over the current length of the sheath, but without being able to radiograph the anchoring zone.
- the RIMT (Reflectometric Impulse Measurement Technology) carried out on an experimental basis was not conclusive, the zones announced like defective proving to be in good condition after opening of windows.

An opening of the 64 anchoring caps of the downstream box girder was then operated. The observations were as follows:

- 41% of the caps presented traces of oxidation
- 12% of the caps presented traces of corrosion
- 31% of the heads were in a satisfactory condition
- 16% of the heads were very corroded
- 5% of the heads presented flows of oil or a oil-water mixture.

A borescope with a 3,8 mm diameter could be introduced through small holes existing on the anchoring heads, when those were free of grout; in six cases, one could observe a significant absence of grout, traces of oxidation on some wires, and even a reduction of the cross section of a tendon.

The broken cable was replaced in 1996, and the bridge was put under high monitoring. After constitution of a repair document, a replacement of all the tendons of the downstream box girder was carried out during the year 2000. This was done by introducing a provisional prestressing to compensate the actual weight of the deck, by cutting each tendon with a blowtorch through holes carried out in the walls of the box girder by water demolition, by removing the tendons through pieces of 2 to 3 meters long, by tensioning the new tendons injected with wax and by disassembling the provisional prestressing.

During the removal of tendons, it was noticed in the trumpets a whitish paste sometimes accompanied by moisture, and corroded wires. The last part of the tendon located between the last deviator and the anchor block undergoes a buckling which creates whippings and large lateral deformations. In the zones where the sheath bursts, the strands become deformed laterally and form a ball.



Fig. 5: Large lateral deformations of the broken tendon near a crossbeam.



Fig. 6: Rupture of the tendon in the form of a bevel near its anchoring head.



Fig. 7: Extremity of the tendon ejected against the abutment wall.

4.2.2 The Saint-Cloud viaduct

The Saint-Cloud viaduct allows motorway A 13 to cross the Seine at the exit of Paris. Brought into service in 1974, it has a curved alignment, a length of approximately 1 km and comprises spans from 67 to 102 m length. It has a width of 20,40 m, a constant height, and consists of a multicellular box girder with four webs, and its prestressing is completely internal to the concrete. Joints between prefabricated segments being opened a short time after construction, it was strengthened in 1979 by an additional external prestressing which is deviated in a vertical plane.

In 1998, one of the additional tendons which were in the Northern side cell and which had a 300 m length broke in its middle, in a section located between two deviators and near the lower slab, close to a hole of re-grouting. The tendon consisted of one 12T15 with strands having a 15,2 mm diameter and was injected by a cement grout with admixture. The energy brutally released during the rupture made the tendon buckle at its ends and made one of the anchoring heads move back from approximately 1 meter, whereas this head was non dismountable. The autopsy of the broken end of the tendon showed that there remained a pocket of grout having the consistency of a wet sandy paste without any coherence whose pH lays between 12 and 14. The expertises carried out by the LCPC showed that the prestressing wires were sensitive to the stress corrosion, and that a majority of wires presented this type of cracking.

4.2.3 The Rivière d'Abord bridge

The Rivière d'Abord bridge is a box girder bridge which is on the island of La Réunion. It was brought into service in 1991. The bridge comprises three spans build by balanced cantilevering, and has a total length of 150 m. This bridge comprises a dismountable external prestressing which is placed in metal tubes curved inside the crossbeams and deviators.

The rupture of a 19T15 tendon occurred at an anchoring located in the upper part of a segment on pier. If this anchorage did not move, the anchoring head at the other end run out approximately 3 meters and collided with the abutment wall of the tensioning room (Fig. 7). The tendon buckled in the side span, but also in the central span. The autopsy showed the presence of much whitish paste on the surface of the tendon and in the indicted anchorage; conglomerates of healthy grout and whitish paste were also observed in some zones of the tendon. Much corrosion and some broken wires were detected (Fig. 8); a priori, water was not observed, but the whitish products were sometimes wet (Fig. 9). The rupture process of the tendon is similar to that of the bridge on the Durance: loss of section per dissolution, retaking of the efforts by healthy wires, then sudden failure of healthy wires accompanied by striction.

5 The problem arising from the cement grouting

As we saw with the first generation of VIPP, at that time the cement grouting were far from being mastered. But the experience helping, the companies improved their products and their grouting methods in order to solve the many existing problems. From 1970, the gammagraphy investigation of tendons, by ensuring a control of the results of grouting, made it possible to visualize the filling defects and to improve the technique.

The provisional directive LCPC-SETRA of March 1973 on the grouting of the sheaths of prestressed concrete bridges [SETRA (1973)], which was accompanied by the development of regional formulations of grouts and the implementation of a control policy of the ducts filling



Fig. 8: Corrosion and broken wires in the failure zone.



Fig. 9: Whitish products covering the strands.

by gammagraphy made it possible to solve the most significant problems of grouting which had been discovered between 1970 and 1973. From 1978 [SETRA (1978)], the change introduced into the standardization of cements led to a progressive neglect of the regional formulas of grout, and starting from the beginning of the 1980s controls of duct filling by gammagraphy strongly decreased until not being carried out any more but in an episodic way, insofar as one estimated to have a good control of the various grouting operations.

During these 20 last years, an evolution was made with the use of increasingly fluid grouts to facilitate the injection of the prestress ducts. From the years 1980, the search for a greater fluidity was accompanied with a generalised use of superplasticizers. This evolution in the formulation of the grouts and in the grouting methods was imposed by itself, however without ensuring regularly the effective filling of the ducts.

It was when gammagraphy campaign on the external prestressing tendons of a box girder bridge under construction in 1994 was carried out, that anomalies were detected at the upper point of the sheaths which had been grouted with a cement mixed with admixture. Observations carried out after opening of some sheaths highlighted a lack of filling and the presence of a product having the consistency of a wet and soft paste which hardened thereafter with the air. The controls carried out in 1995 on other bridges under construction made it possible to note that this phenomenon also existed in current bridges like some slab bridges and that it was more widespread than one could think at first.

These anomalies could be reproduced during tests of injection of cement grout with admixture carried out in laboratory in tilted transparent tubes. When certain conditions are met, the use of a superplasticizer facilitates the migration of mineral species which are formed during the set of the grout. There is a separation by density between the cement suspension in the course of hydration and these lighter mineral species (in particular ettringite and portlandite). A whitish product is thus created whose migration which occurs all along the tube is accompanied by an ascent of air bubbles. At the end of the test, in the higher part of the tube, a layer of whitish paste surmounted by a more or less yellowish liquid is produced, the liquid being itself surmounted by a space filled with air. Mineralogical analyses carried out at LCPC on the various products show that [Deloye (1996)]:

- the whitish paste which hardens quickly in contact with the air, is primarily made up of ettringite (40 %), portlandite (20 %) and calcite (20 %), the latter coming mainly from the carbonation of the portlandite. The remainder of the paste presents an enrichment in admixtures and sulphates.
- the yellowish liquid presents a composition close to the interstitial solution of a cement paste with a very strong alkalinity (pH of 13,8).

The analyses carried out tend to show that it is a phenomenon of exsudation combined with a settlement of the grout, and that this phenomenon is amplified by the presence of a superplasticizer; this admixture can present in some cases an incompatibility with the cement. The inadequacy of the cement-admixture couple then causes an instability of the grout during the dormant phase of the set.

The coexistence of an air pocket and a liquid phase within a prestress sheath can pose in the long run a problem of durability of prestressing. Indeed, the water-air interface in a confined space implies that the air is always saturated with water (100 % of relative humidity). Under the effect of variations in temperature, a pure water condensation can then occur, either on the interior wall of the sheath, or on the parts of the prestressing steels exposed to this air saturated with water. The conditions are then met so that a corrosion of steels can develop. Moreover, the extremely alkaline pH of the liquid can make fear an alkaline corrosion of tendons, as shown by the Pourbaix diagram.

Thus in the absence of a resumption of the grouting, and even with a grout with weak exsudation (1 %), the injection of very long tendons (200 to 300 m) can create air pockets of which the cumulated length reached 2 to 3 m.

This is why a test on tilted tube of 5 m length was developed in order to check the stability of the formulas of grout under conditions representative of the field reality. This test, which was presented in an information note of the SETRA [Chaussin & al. (1996)], was studied and validated within the framework of an experimental campaign launched by the LCPC [Le Roy & al. (2000)]. The study of the grouting of full scale translucent sheaths made it possible to observe that the bleeding order of the high points depends on the geometry of the sheath, that the mode of filling of the sheath is a function of the grout thixotropy, that the slope of the sheaths and the presence of the strands (by effect of filtration) amplify the volumes of air and water appearing in the higher parts; this research allows also to check the possibility of using a simplified test on tube of 1 m length.

Moreover a recent circular of the Ministry of Equipment [MELTT (2001)] fixed the provisions to be taken for the bridges to build, with the objective to reduce releasable energy in once during a rupture of tendon. It recommends to separate the elementary strands of long tendons, or to partition the tendons in elements being able to behave independently from each other.

6 Injection of ducts with soft products

The injections by soft products cover the injections with grease and oil wax. The first injections of external tendons with grease did not give completely satisfactory results, mainly because grease proved to be unstable in the short term and that non tight ducts caused leakage of the oily part of the grease (example of the strengthening of the Bayonne bridge by additional prestressing). That is why the soft injections are currently carried out with wax which is a more stable product in the time and which did not give place, until now, to a new calling into question of the durability of the protection, according to the few sheaths or anchoring caps openings which have been made.

7 Some rare problems related to the sheaths

Those problems are overall very rare, and this fact is explained by the use, right from the start of the technique of external prestressing, of sheaths made of High Density Polyethylene (HDPE), to the exclusion of lower density polyethylene and of Polyvinyl Chloride (PVC) which is a more fragile material, even breakable with time.

Two cases of sheath cracking can however be noticed, the cause being probably an inadequate formulation of the HDPE. The first one concerns grease sheathed-monostrands which passed on a saddle located on top of a pylon of a cable stayed bridge and which were exposed to the weather; the sheaths cracked on their upper part. The second case relates to the ducts used to protect the additional prestressing of the ONIA bridge in Toulouse; the sheaths filled with cement grout saw their cracking starting from the deviators and propagating slowly during years which followed the strengthening.

8 State of the park of prestressed concrete bridges

After having examined in detail the problems of durability encountered with the post-tensioning tendons, it is advisable to put in perspective these problems in the more global context of management and evaluation of the bridge condition, in order to better measure the relative weight of these problems.

The French road network counts more than 230 000 bridges of span greater than 2 m, including approximately 22 000 bridge managed by the State. Our analysis will be carried out on the stock of bridges located on the national road network, because it is on this stock that we have the best statistical elements relating to the condition of bridges, thanks to IQOA operation (Image of the Quality of Bridges) [Robichon & al. (1995)].

The network managed by the State comprises approximately 4000 concrete bridges prestressed by post-tensioning or by pre-tensioning, but those bridges represent nevertheless an area of 3 737 500 m², that is to say half of the total surface of the patrimony. The main part of the bridges are prestressed by post-tension, and the following table extracted from [HA, SETRA, TRL, LCPC (1999)] gives the distribution by type of bridge:

Type of prestress	Type of bridge	Percentage by number	Percentage by area
Post-tensioned	Slab bridges	70	51
	VIPP (*)	4	6
	Beam and slab bridges	8	10
	Box girder bridges	7	26
	Other	3	3
Pretensioned	PRAD(##)	8	4
	TOTAL	100	100

(*) VIPP : Viaducts with simply supported spans made of post-tensioned beams

(##) PRAD : Bridges with pretensioned beams

Table 1 : Distribution by structural type of the 4000 prestressed concrete bridges on the national road network.

Slab bridges are thus by far the most common (more than 2/3 of the total number), but they represent only half of total surface. The box girder bridges are few (7 %) but represent a quarter of the total surface.

If we examine the condition of the prestressed concrete bridges on the basis of the five IQOA condition classes pointed out hereafter:

- Class 1: bridge in apparently good condition
- Class 2: bridge presenting minor defects
- Class 2E bridge presenting minor defects and requiring an urgent specialized maintenance to
- prevent the rapid development of disorders in the structure

- Class 3: bridge presenting major disorders
 - Class 3U: bridge presenting major disorders and requiring an urgent repair
- the principal results obtained at the end of the evaluation of 1996 are as follows:
- the condition of the prestressed concrete bridges is similar to that of the whole stock of bridges (see table 2); indeed the distributions in classes are almost identical;

IQOA Class	1	2	2E	3	3U
Prestressed concrete bridges	8	35	35	16	6
All bridges	11	37	32	15	5

Table 2 : Comparison of the distribution of bridge condition (percentage by surface area)

Type of prestress	Type of bridge	IQOA Class				
		1	2	2E	3	3U
Post-tensioned	Slab bridges	8	44	37	9	1
	VIPP (*)	4	33	39	19	4
	Beam and slab bridges	3	25	35	34	3
	Box girder bridges	7	24	34	18	16
	Other	4	51	19	26	0
Pretensioned	PRAD(#)	8	62	24	7	0

Table 3 : Analysis by structural type of the condition of prestressed concrete bridges (percentage given by surface area)

- the box-girder bridges present the poorest condition, with more than one third of their surface classified into 3 and 3U (see table 3);
- the slab bridges prestressed by post-tension and the bridges with beams prestressed by pre-tensioning present few problems of pathology (less than 10 % of their surface classified into 3 and 3U);
- the problems of prestressing durability met in the VIPP are classified in the average of the concerns (23 % of their surface classified into 3 and 3U).

9 General conclusion

To our knowledge, prestress is the only "material" in civil engineering (and very likely in our environment) which works to approximately 80 % of its failure strength, and this permanently. It is therefore a sensitive "material" which shall be protected as well as possible if one wants to have a certain durability. Pathology teaches us that the prestressing tendons

are more vulnerable in the case of post-tensioning than in the case of pre-tensioning, and I think that one does not grant enough care to the grouting which is the key operation to ensure the durability of prestressing. But, it is particularly difficult to be ensured of the perfect filling of the ducts...

The integrity of the duct is not the best guarantee against corrosion. Indeed, if the integrity of the duct prevents the water and external agents to penetrate inside, it prevents also water from evaporating or the harmful liquid substances to dry... Pathology teaches us that, 9 years after, water can remain present in a HDPE duct, and that 19 years later a wet and soft paste can remain in permanent contact with the tendons in a plastic duct...

Within the framework of the debate which turns around the use of internal or external tendons, we think that each process has its advantages and disadvantages. For external prestressing, in addition to the fact that it is more sensitive to fire and more easily attackable by the vandals, when a section of tendon is broken, the prestressing force disappears over the overall length of the tendon; on the other hand, it is theoretically replaceable and more easy to investigate. Internal prestressing is much more difficult to investigate and impossible to replace, but it has the advantage, thanks to the possible re-anchoring of the strands, to lose only locally the prestress force in the event of the failure of a section.

Moreover, the attention must be drawn to external prestressing tendons grouted with cement which release their energy in the event of a sudden rupture. The buckling and whipping generated by this rupture create risks for the staff likely to move inside box girder bridges, and also for the other tendons. In addition to the great care which should be applied to the manufacture and to the grouting of stable products for the new bridges, for the existing one, the managers ask for assessing the risks by detecting the wire failures (with the acoustic monitoring for example...) and for ensuring safety of the staff (by carrying out research on anti-whipping devices,...).

Lastly, a global analysis of the condition of bridges located on the national road network shows that the condition of the prestressed concrete bridges is similar to that of the whole stock of bridges of the network, and that the durability of post-tensioning tendons is a problem which is classified in the average of the concerns of the managers. However, it remains that prestressing tendons are sensitive products which require a detailed attention as regards quality of implementation.

The author thanks Messrs Jean-Claude Bastet and Gilles Lemestre for the information provided on some bridges.

REFERENCES

SETRA (1973): Directive provisoire sur les injections des gaines des ouvrages en béton précontraint. SETRA, LCPC, mars 1973.

SETRA (1978): Injection des gaines de précontrainte – Ponts construits par encorbellements et autres ponts similaires. Note d'information du SETRA, 21 décembre 1978.

GRATTESAT G. (1978), Conception des ponts. Editions Eyrolles, 291 pages.

CHATELAIN J. (1984), Bilan de santé des anciens ouvrages à précontrainte extérieure. – Formation continue de l'Ecole Nationale des Ponts et Chaussées, 5 au 7 juin 1984.

SETRA (1990) (Service d'Etudes Techniques des Routes et Autoroutes) : Précontrainte extérieure., février 1990, 119 pages.

ROBICHON Y., BINET C., GODART B.,(1995) « IQOA » : Evaluation of bridge condition for improved maintenance policy. IABSE Symposium « Extended the lifespan of structures », San Francisco, 23-25 août 1995, IABSE report, Vol 73/1, pp 407-412.

DELOYE F.X., GODART B.(1996) , Coulis de précontrainte adjuvants (suite...). Bulletin de liaison Ouvrages d'Art du SETRA n° 23, mars 1996, pp 49-50.

CHAUSSIN R., GODART B., LEFEVRE J. (1996), Coulis pour injection de conduits de précontrainte. Note d'information du SETRA, juillet 1996.

GUINEZ R., SCORPION (1997): Bilan de dix ans d'utilisation. Bulletin des Laboratoires des Ponts et Chaussées n° 207, Janv-Fev 1997, pp 79-90.

HA, SETRA, TRL, LCPC (1999): Post-tensioned concrete bridges (Ponts en béton précontraint par post-tension). Joint publication of Highways Agency, Service d'Etudes Techniques des Routes et Autoroutes, Transport Research Laboratory, Laboratoire Central des Ponts et Chaussées, Thomas Telford, 164 pages, ISBN: 0 7277 2760 5. (French and english)

TROUILLET P. (2000) Communication interne, juillet 2000, Mission d'Inspection Spécialisée Ouvrages d'Art, Secteur Autoroutes Concédées.

LE ROY R., WILLAERT M., ROUANET D., MIRMAND H. (2000), Identification des paramètres favorisant la formation de poches d'air et d'eau dans les coulis de ciment pour gaines de précontrainte. Bulletin des Laboratoires des Ponts et Chaussées, n°229, Nov-Dec 2000, pp 53-70.

MELTT (2001): Circulaire n°2001-16 du 28 février 2001 relative à la conception de la précontrainte extérieure au béton. Bulletin officiel du Ministère de l'Equipeement, du Logement des Transports et du Tourisme du 25 mars 2001.

Status of the durability of post-tensioning tendons in the United States

C.L. FREYERMUTH
American Segmental Bridge Institute

1 Introduction

1.1 Early post-tensioned construction

The advent of post-tensioned construction in the U.S. is usually associated with construction of the Walnut Lane Bridge in Philadelphia in 1949-1950. This cast-in-place I girder bridge (prestressed with the Magnel system) was replaced in the mid 1980's due to detailing and grouting problems which led to significant corrosion damage to the post-tensioning tendons.

Three segmental I-girder bridges were constructed in western New York State, about 43 miles from Buffalo in 1950 and 1951. Due to this area's short construction season, there was a need to use the country's construction crew in winter, as well as the need to speed up construction. These considerations led Jean M. Muller, at that time a representative of Freyssinet International, to the development of the concept of precasting segments the concrete girders in the maintenance yard for early spring erection. The segments were match cast. These bridges became the first segmental bridges designed and built in the U.S., and may have been the first precast segmental bridges built in the world.

The first of the New York bridges to be built was the Griffith Road Crossing of East Koy Creek. The bridge has a single 50ft. span with a 24ft. roadway. An inspection in 1980 showed that the bridge was in "new condition" and in the county highway superintendent could find no record of any maintenance expense. A 1990 visit to the East Koy Creek Bridge showed the gravel road had been paved with asphalt, but the bridge remained in excellent condition. This bridge is located in one of the harshest weather environments in the U.S.

Unbonded monostrand tendons were first utilized by T.Y. Lin in construction of school roofs in Nevada in the mid 1950's. As in the case of unbonded tendons used until the early 1970's, the tendons were greased and spirally wrapped with two layers of heavy kraft paper.

1.2 Early specifications

The first specifications for prestressed concrete in the U.S. were the “Tentative Recommendations for Prestressed Concrete” by ACI- ASCE Joint Committee 323 published in January, 1958. These tentative recommendations stated that steel for bonded tendons should be kept clean and dry, and that foreign matter, grease, oil, paint, and loose rust should be removed before steel is placed in ducts. A light coat of rust was permitted provided loose rust has been removed and the surface of the steel was not pitted. For unbonded tendons, the tentative recommendations stated that wrapping’s and coating’s on non-galvanized steel should protect the steel from corrosion during shipment, storage, construction, and after the steel is in place. Further, the tentative recommendations stated that the method of protection should be specified or approved by the engineer. In retrospect, it is apparent that these corrosion preventive recommendations were grossly inadequate, but they were representative of the state-of-the-art at that time. Much of the early corrosion damage in post-tensioned structures in the early years of practice in the U.S. is traceable to inadequate provisions for corrosion protection of tendons exposed to aggressive environments.

The “Tentative Specification for Post-Tensioning Materials” published in the January-February, 1971 edition of the PCI Journal, required that “Unbonded Tendons shall have the prestressing steel permanently protected against corrosion by a properly applied coating of galvanizing, epoxy, grease, wax, plastic, bituminous or other material.” The specification further stated that, “the sheathing shall prevent the intrusion of cement paste and the escape of coating material. The sheathing may be a continuous tube or spirally wrapped”. By 1975 various types and thicknesses of plastic sheathing were exclusively used for unbonded tendons. However, it must be conceded that the improvement of corrosion preventive requirements between the 1959 tentative recommendations and the 1971 tentative specifications was very limited with regard to both bonded and unbonded tendons.

1.3 Development of the U.S. post-tensioning market

From the beginnings discussed above, the use of post-tensioning grew slowly in the U.S. to the 1999 totals shown in *Table 1.1*. Bridges and earthwork utilizing bonded tendons comprise 22 percent of the total, the remainder is largely made up of unbonded tendon applications. However, VStructural LLC, the U.S. licensee of VSL International has been using grouted multi-strand tendons in flat plastic ducts in building construction in the U.S. for more than ten years as an alternative to unbonded tendons. In grossly approximate dollar terms, the 1999 U.S. post-tensioning market was about 350 million dollars, representing about 4 billion dollars in construction cost.

Table 1.1 1999 U.S. post-tensioning steel tonnage

		Percent %
Buildings	40,720	32
Bridges	20,697	16
Earthwork	8,256	6
Slab-on-ground	56,306	44
Miscellaneous	2,422	2
Total	128,401	100

2 Durability of tendons in U.S. bridges

2.1 National bridge inventory data

The U.S. Federal Highway Administration (FHWA) publication, "Public Roads" for July-August, 1997 contained an article by FHWA engineers on "Nondestructive Evaluation for Bridge Management in the Next Century" that included National Bridge Inventory (NBI) statistics on bridge performance. NBI statistics are based on biennial inspections by personnel of State Departments of Transportation. During these inspections, components of the bridge are given a numeric condition rating or code from 0 to 9 in accordance with *Table 2.1*.

Table 2.1 National bridge inventory condition ratings for concrete bridges

Code	Description
N	Not applicable
9	Excellent condition
8	Very good condition
7	Good condition - some minor problems
6	Satisfactory condition - structural elements show some minor deterioration
5	Fair condition - all primary structural elements are sound but may have minor section loss, cracking, or spalling
4	Poor condition - advanced section loss, deterioration, or spalling
3	Serious condition - loss of section, deterioration, or spalling. Have seriously affected primary structural components. Local failures are possible. Shear cracks in concrete may be present.
2	Critical condition - advanced deteriorating or primary structural elements. Shear cracks in concrete may be present. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.

1	"Imminent" failure condition - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	Failed condition - out of service - beyond corrective action

The 1997 "Public Roads" article referenced above indicates that "Steel Bridges represent about 40 percent of the overall bridge total, but include about 60 percent of the structurally deficient bridges. Reinforced concrete bridges fare well in the comparison of proportions (18 percent of the structurally deficient bridges), and only 3 percent of the prestressed concrete bridges are structurally deficient. Timber bridges, while representing only 9 percent of the entire number of structures, represent 20 percent of the structurally deficient bridges."

It is clear from the NBI data that durability performance of prestressed concrete bridges is substantially better than any other material available for bridge construction. This fact provides a context for evaluation of tendon durability problems. Although tendon problems do occur in the U.S., the number of these problems is relatively small, and the related costs are only a fraction of the costs routinely accepted for repair or replacement of bridges using other materials. There have been a number of failures of steel bridges in the U.S. in recent years under traffic, some involving loss of life, which have not led any U.S. transportation authority to issue a ban on the use of steel bridge construction.

2.2 Overview of tendon corrosion problems

The major portion of the bridge tendon corrosion problems that have been observed in the U.S. have been identified with the following:

- 1) An aggressive environment (northeast U.S. and Florida)
- 2) Areas with a low volume of post-tensioned construction
- 3) Contractors with no experience or expertise in post-tensioned construction
- 4) Grossly inadequate construction supervision
- 5) Design details without adequate provision for corrosion protection of tendons
- 6) Failures to respond to or correct construction problems

Notwithstanding the above, most states in the U.S. have not recorded any significant tendon corrosion problems. California has more than 3,800 post-tensioned bridges, some dating to the late 1950's. Not all of these bridges are in benign environments. The deck area of these bridges exceeds 78 million square feet. To date, there have been no reported tendon corrosion problems in California. The bridges are post-tensioned and grouted by highly experienced crews, and construction is always and without exception inspected by knowledgeable state construction personnel. Georgia, Oregon, Texas, Washington and Virginia, all of which have made significant use of post-tensioned construction have not had any significant tendon corrosion problems.

2.3 Durability performance of tendons in segmental and cable-stayed bridges

The second edition of the American Segmental Bridge Institute "Survey of the Durability of Segmental Concrete Bridges" based on a tabulation of the latest National Bridge Inventory data found that "segmental bridges are performing well with time." No structurally deficient segmental bridges were found in the 2000 ASBI durability survey, and 99 percent had superstructure ratings of "satisfactory" (NBI numerical rating of 6) or better. Although most U.S. segmental bridges are less than 25 years old, bridges constructed with other materials are found to have significant percentages of structural deficiencies in less than 25 years.

The first precast segmental box girder bridge in the U.S., the John F. Kennedy Memorial Causeway, built in 1971 over salt water on the Texas coast, had a 1999 NBI superstructure condition rating of 6 (satisfactory). The Pine Valley Creek Bridge, built in the coastal mountains near San Diego, California in 1974 was the first U.S. cast-in-place balanced cantilever bridge. The 1999 NBI superstructure condition rating of the Pine Valley Creek Bridge was 8 (very good condition).

2.4 Recent tendon durability problems in U.S. segmental bridges

In 1999, one of the external tendons in the Niles Channel Bridge in Florida was found to have failed due to corrosion. The Niles Channel Bridge was one of the early (1983) low-level span-by-span precast segmental bridges built in Florida. The Niles Channel Bridge contains a total of 234 external tendons. The failed tendon was at an expansion joint.

In 2000, eleven tendons out of a total of 846 were replaced in the Mid-Bay Bridge in Florida, constructed in 1993. Ten of the eleven tendons which were replaced, were located at expansion joints. From 1978 to 1997, eight low-level span-by-span bridges were constructed over sea water in Florida with a total length of 19.35 miles. These bridges contain about 4600 external tendons. The replacement rate to date is 12/4600, or 0.26 percent. In addition to tendon replacement, inspections of Florida bridges have revealed a large number of bleed water voids at anchorages resulting from accumulation of bleed water. Voids exposing prestressing steel are being repaired by vacuum grouting.

In a large Florida interstate segmental interchange, a number of partially grouted tendons and ungrouted tendons were found. These problems appear to be the result of construction negligence.

In 2000, several corroded vertical tendons were discovered in segmental piers of the Sunshine Skyway Bridge, which was completed in 1986. An extensive investigation of these problems indicates that the corrosion resulted from seawater in ducts (the caisson flooded during construction), permeable concrete anchorage protection at the top of piers, and splitting of polyethylene ducts.

It appears that the tendon corrosion problems in Florida segmental bridges resulted from:

- 1) Voids associated with accumulation of bleed water at tendons anchorages
- 2) Recharge of ungrouted tendon anchorages with salt water or surface drainage during construction

- 3) Leakage through end anchorage protection details
- 4) Quality of the grout installation and grout material
- 5) Splitting of polyethylene ducts
- 6) Deficiencies in implementation and inspection of grouting procedures

The Florida specification for polyethylene ducts was revised in 1996, and the U.S. specification for polyethylene ducts was revised in 1999.

Recently, bleed water voids have been found at anchorages of the transverse floor beam post-tensioning in the Sidney Lanier cable-stayed bridge in Georgia. Grouting deficiencies have also been found in 2001 in Boston Central Artery bridges in both segmental and cast-in-place post-tensioned bridges. No corroded tendons were found in the Sidney Lanier Bridge, or the Boston Central Artery Bridges, which are still under construction.

There have been no reported corrosion problems with stay cables to date in the U.S.

2.5 Recent responses to observed or potential tendon durability problems

The American Segmental Bridge Institute formed a grouting committee in October 2000 that issued on "Interim Statement on Grouting Practices" on December 4, 2000. This was subsequently published in "Bridge Design & Engineering", Issue No. 23, Second quarter 2001, under the title "The good grouting guide."

The American Segmental Bridge Institute initiated a Grouting Certification Training Program with the first training session held August 6-8, 2001. Approximately 140 construction personnel attended the training. The purpose of the training is to provide supervisors and inspectors of grouting operations with the training necessary to understand and successfully implement grouting specifications for post-tensioned structures. Two additional training events are scheduled in 2002.

The Post-Tensioning Institute published new grouting specifications in February 2001. The specifications include requirements of thixotropic (anti-bleed) grouts to be used in aggressive environments.

3 Durability of tendons in U.S. buildings

3.1 Durability performance of unbonded tendons in U.S. buildings

As indicated by the tonnage statistics in *Table 1.1*, tonnage of post-tensioning building construction was twice the tonnage used in bridge construction in 1999. This large usage reflects the economies resulting from the 1/3 reduction in slab or beam depth compared to conventional reinforced concrete, and the relative economy of post-tensioning reinforcement.

There have been few significant durability problems with unbonded tendons in enclosed buildings. There has, however, been some corrosion of tendons anchored at the outside edge of balconies in aggressive environments. All types of structures with unbonded tendons have performed well in seismic events in California and elsewhere. There has been virtually no

hydrogen embrittlement or stress corrosion cracking of unbonded tendons in the U.S. over the past 45 years.

In contrast to enclosed buildings, there has been a significant amount of corrosion of unbonded tendons in parking garages in aggressive environments, as well as some corrosion of unbonded tendons in milder climates. Lack of adequate specifications for corrosion protection of unbonded tendons often led to undesirable practices such as removal of the plastic sheathing from tendons near anchorages. In a four level parking garage in Cleveland built in 1975, 128 out of 2400 tendons failed due to corrosion between 1985 and 1993. The garage was demolished and replaced. In a parking garage near the ocean in San Francisco all of the beam tendons were replaced by external epoxy tendons enclosed in square concrete sheaths, and many of the slab tendons were replaced by epoxy coated strand.

3.2 Industry efforts to improve durability performance of unbonded tendons

Recognizing the need for detailed specifications for fabrication, placement, stressing and finishing of unbonded tendons, the Post-Tensioning Institute (PTI) prepared and published "Specifications for Unbonded Single Strand Tendons" in the March/April 1985 edition of the Journal of the Precast/Prestressed Concrete Institute (PCI). These specifications were subsequently adopted by the ACI Building Code and Model Building Codes.

The 1985 PTI specifications require complete encapsulation of the tendon, including a grease-filled cap over the wedge cavity. It was originally proposed to use epoxy coated anchors, but this detail was deleted due to a threatened lawsuit by a holder of a U.S. Patent for "electrically isolated tendons." Some engineers use electrically isolated tendons anyway, with or without payment of the related royalty.

Encapsulation of tendons is also required at intermediate stressing anchors. The PTI Specification covers all aspects of tendon materials and fabrication in detail, including quality and quantity of grease, and thickness of sheathing. An updated version of the PTI "Specification for Unbonded Single Strand Tendons", published in 2000, includes additional requirements intended to provide improved corrosion protection, such as increased thickness of the plastic sheathing.

PTI also implemented a certification program for plants producing unbonded tendons and published a field procedure manual for placement and stressing of unbonded tendons.

PTI publications on parking structures include zones determined by the aggressiveness of the environment. The more aggressive exposure zones require more restrictive design provision to enhance durability performance. Exposure zones do have some rational appeal. However, considering the relatively nominal premium for electrically isolated unbonded tendons (about 15 percent additional cost for tendons or about 1.25 percent additional cost for a structural frame), and the large costs involved in repairing and replacing tendons when problems occur, it would seem reasonable to require use of electrically isolated unbonded tendons in all cases where the tendons are exposed to the environment. Further, in addition to requirements for tendon fabrication details, it is also essential to provide adequate supervision of construction to ensure that tendons are properly placed and stressed. Supervision and inspection of building construction in the U.S. is often inadequate.

4 Conclusions

In either grouted or unbonded construction, poor durability performance has been associated with poor construction practices, and inadequate materials specifications. While specifications have been improved, the challenge remains to obtain and train qualified construction personnel.

Notwithstanding some grouting related corrosion problems, the durability performance of prestressed and segmental post-tensioned bridges has been superior to all other types of construction. Recent improvements in grouting materials technology (anti-bleed grouts), and training programs for grouting supervisors and inspectors are expected to significantly reduce the incidence of corrosion problems in grouted tendons.

Durability performance of unbonded tendons exposed to aggressive environments was often deficient prior to publication of greatly improved specifications in 1985, and implementation of other industry efforts to correct problems. While it appears that the incidence of unbonded tendon corrosion problems has been significantly reduced by new specifications, a need remains to improve the quality of construction and inspection. In this context PTI has instituted a certification program for field personnel involved in placing and stressing unbonded tendons.

Durability of post-tensioning tendons : Canadian experience

J. HARDER¹ and N.R. WEBSTER²

¹M.Eng., P.Eng., Principal, Read Jones Christoffersen Ltd., Calgary, Alberta, Canada

²M.Eng., P.Eng., President, Read Jones Christoffersen Ltd., Calgary, Alberta, Canada

Abstract:

This paper presents a brief overview of experience gained in Canada over the past two decades, on the subject of durability of primarily unbonded post tensioning systems in buildings. The paper summarizes conditions found in structures that have been investigated, the investigation techniques that are used, the repair techniques used, the code changes that were made to improve durability, and the public education measures that were undertaken. Comments and recommendations are made in the hope that lessons learned over the past 15 or more years can be applied in future to make ongoing improvements to bonded and unbonded systems, and to improve the approach taken when introducing new technology in the future. Structures in "high-risk" categories should be investigated and possibly repaired for safety reasons. Other structures require long-term monitoring. Structures which have been repaired previously should be monitored if new or innovative techniques have been employed. The body of knowledge is constantly growing. One must periodically reassess the risk related to existing structures, and the performance of the repair methods as more information becomes available. During the past 15 years, many improvements have been made to practices and building code provisions for design and construction. Improvements should be made to some systems to make them less susceptible to moisture ingress. Laboratory testing of tendon waterproofness should also be improved to more closely simulate field conditions. The time required to fully identify and address the durability issue was long. Methods must be found to increase the speed with which issues are identified, information is exchanged and practices are improved.

1 Introduction

This paper presents a brief overview of experience gained in Canada over the past two decades, on the subject of durability of post tensioning systems. Most of this experience relates to unbonded monostrand post tensioning systems in buildings, including parking structures, commercial, residential, and institutional buildings.

The use of post-tensioned construction grew rapidly in Canada, especially during the building boom of the late 1970's and early 1980's. In the mid to late 1980's, durability problems became increasingly apparent in some buildings, while many other buildings continued to perform well.

The authors have been involved in assessment of over 300 facilities containing unbonded post-tensioned reinforcement.

This paper briefly summarizes conditions found in structures that have been investigated, and the investigation techniques that were developed to assess those structures. It also presents a brief overview of repair techniques used, the code changes that were made in order to improve durability, and the public education measures that were undertaken.

It is hoped that some of the lessons learned over the past 15 or more years can be applied in future to make ongoing improvements to bonded and unbonded systems, and to improve the approach taken when introducing new technology in the future.

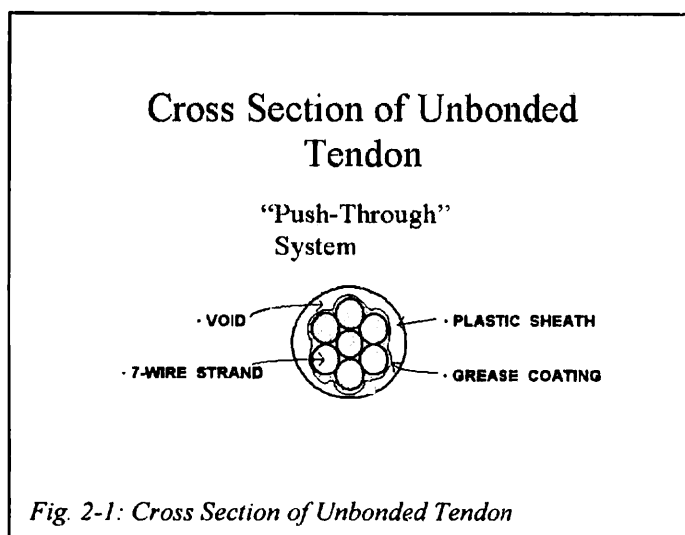
2 Historical background

In the area of building construction, post-tensioning saw isolated use in Canada in the 1950's and increasing use through the 1960's, 70's, 80's and beyond. It is estimated that 7 to 10 million square meters of building structure has been constructed in Canada using post-tensioning.

In the 1960's and early 1970's a wide variety of systems was used in building construction in Canada:

- tendons of wire, bar, or strand
- sheathing of metal (for bonded systems), paper or plastic
- Bonded (least common) and unbonded (most common)

Since the early 1970's, unbonded tendons have typically consisted of 7-wire strands coated with grease and encased in plastic sheathing. Until the mid 1980's both "Push-through" tendons (also called "stuffed" tendons) and "extruded" tendons were used. The former are fabricated by inserting a greased tendon into plastic sheath, leaving an air void inside the duct. (Refer to Figure 2-1) The extruded systems are designed to have no air void in the tendon.



The percentage of buildings constructed using post-tensioning varied considerably throughout the country. Post-tensioning was used more frequently in areas such as Calgary, Vancouver, and Toronto where post-tensioning suppliers established a market presence. In some cities, post tensioning was seldom or never used due to reservations about the concept of unbonded construction, lack of familiarity by designers, and lack of established suppliers.

Grouted post tensioning is common in bridge construction in Canada. It has seen limited use in buildings. Its application in buildings has included single and double-strand tendons in slabs and multi-strand tendons in beams.

Isolated instances of unbonded tendon breakage were first noted in the early to mid 1970's, for example:

- In 1975, two tendons erupted in a University of Alberta parking structure that was approximately five years old.
- In approximately 1975, isolated eruptions of tendons were observed in a parking structure in Ottawa.

The two structures noted above required repair of the entire structure by 1983-1984.

By the mid 1980's, severe deterioration had come to light in several other buildings. Tendon breakage was noted in a wide variety of situations. The following are but a few examples:

- 1984 - Two tendons erupted from the ground floor and parking level respectively in a 12-year old residential complex in Calgary where tendon anchors were situated below grade and accessible to moisture.
- 1985 - A tendon erupted from the ground floor of an office building, also in Calgary. Tendon anchors were at or below grade and allowed entry of water, and probably fertilizer, from adjacent landscaping.
- 1989 - A tendon erupted in a dry service environment in a 12-year old office building in Calgary, apparently due to moisture access into a tendon during construction.
- 1991 - In the ground floor of a 10-year old residential building, breakage was detected during investigation requested by a financial institution. Tendons were of the extruded type. Anchors in exterior basement walls were exposed to moisture.

It was thus apparent that deterioration could occur not only in corrosive environments such as parking structures, but also in non-corrosive environments where moisture had entered the tendons during construction.

Extruded systems at the time did not use waterproof, or "encapsulated" anchors, and the sheathing was easily damaged during construction. It became apparent that the extruded systems were vulnerable to moisture ingress at anchors and at any location where the sheathing was discontinuous.

The majority of structures investigated, and the majority of tendons with problems, were of the “push-through” type. The 1991 item noted above was one of a small number of known problems with extruded systems. However, few such structures have been investigated.

In 1985, improved practices for unbonded tendons were published by the Post-Tensioning Institute [PTI (1985)], and push-through tendons ceased to be used in Canada by the late 1980’s.

By mid 1989, learning from experience gained in Calgary [Webster (1989)], engineers in Vancouver began to specify only “encapsulated” systems with increased sheathing thickness. Engineers also required increased training and care by workers and inspectors. Local suppliers became actively involved in developing improvements to the anchorage hardware to make them more waterproof. These improvements were incorporated into the design standards. [CSA (1994, 1995)]

In Calgary, from 1990 to 1993, many seminars were organized. A considerable effort was put into educating owners on the need to investigate structures with unbonded post-tensioning and educating engineers on how to investigate these types of structures. In Vancouver, Toronto, and other cities, seminars were also organized by local technical societies and by the National Research Council of Canada. There was free exchange of information.

3 Current investigation techniques

3.1 Establish client requirements

Historically, the challenge in evaluating post-tensioned buildings has been the need to gain sufficient information about existing conditions to make informed decisions, while minimizing investigation costs. The clients risk tolerance and business objectives have often influenced the scope of investigation.

Investigations associated with the purchase or financing of buildings are often driven by the need to identify whether there are any significant concerns with regard to a particular facility with a minimum investment in initial investigation costs. Supplementary investigation is initiated if problems are discovered and the client requires a greater level of confidence in predicting future performance or further refinement of costs related to remedial work.

For clients with a limited tolerance for unanticipated expenses, the initial testing program often is designed to assess conditions throughout the structure rather than selective testing in representative areas. This reduces the potential that an area experiencing greater deterioration than the norm would be missed.

With older facilities, some consideration may be given to redevelopment options rather than repair or maintenance, particularly if there is uncertainty in predicting the effectiveness or longevity of repairs. This may influence the scope of testing required.

3.2 Document review

The first phase of a condition review typically includes a review of all available documentation for the particular building, including design drawings, specifications, construction and maintenance records. Efforts are often made to locate shop drawings, as suppliers of post-tensioning systems typically retained their own design engineers and many times, substantial portions of the original design were revised to suit the systems and components supplied by the subcontractor.

3.3 Preliminary design review

Prior to deciding on the scope of testing, a general review of the structural design is completed. This is not intended to be a calculation intensive or thorough review of the structural design. Specified numbers of tendons are used to determine average precompression in the cross section (P/A), and an approximate punching shear calculation for two-way slabs is performed. Drawings are reviewed initially to determine the general arrangement of the post-tensioning system, the type of sheathing, size and grade of strand, locations of live end stressing anchorages, construction and expansion joints. Span to depth ratios for typical elements, the arrangement of spans and their regularity is also reviewed.

3.4 Walkthrough assessment

Prior to commencing any physical investigative work, a walk through of the property is performed to confirm that the building construction is in general conformance with the drawings and specifications. This is also used as an opportunity to confirm the building occupancy is as was anticipated by the original structural designer. As an example, a building may have been designed as an office or mercantile occupancy but now is being used as a storage facility.

Signs of visible distress such as cracking, spalling, deflection, failed waterproofing, leakage, grease staining, or corrosion are documented. In addition, the service conditions to which the structural elements are exposed are noted along with the condition of the protective systems (if any) exposed to view.

Physical constraints affecting access to slab soffit and end anchorages, such as interior finishes or exterior cladding may influence selection of inspection locations. The inaccessibility of critical components can impact upon the effectiveness of the inspection program.

3.5 Sample sizes and partitioning of sample

The following factors are often considered when selecting the total number of tendons that will form the sample group to be exposed:

- Use of the structural element (office building, parkade slab, etc.)
- Exposure conditions (waterproofed, indoors and heated, etc.)
- Location of tendon anchorages (above grade, below grade, behind building envelope)

- Exposure conditions during fabrication and construction
- Evidence of structural distress
- Evidence of water ingress or leakage through the structure

The sample is therefore partitioned to reflect varying exposure conditions within different parts of the structure. For example, tendons in parking decks, exterior plazas, roofs and other areas with a higher risk of moisture access may warrant a higher sample density (approaching 10 to 15% of the tendons within the slab). This would reduce the potential that areas with extensive moisture access or significant strand breakage would be missed during the evaluation. Conversely, evaluation of tendons within areas judged to have a lower risk of past moisture access such as high-rise floor slabs with slab edges protected by the building envelope may be of a lesser sampling density (perhaps only 5%) of the total number of tendons within the areas under review. Typically a minimum of 20 strands are exposed and tested for each exposure condition within a particular building

The following initial sample sizes are commonly suggested for exposure and testing of strands at inspection recesses:

- | | |
|---|---------------|
| – Single level structure under 1000 sq m : | 20 strands |
| – Single or multiple levels, total area under 3000 sq m : | 60-80 strands |
| – High-rise (over 10 floors) : | 100 strands |
| – Multiple tower or larger buildings (over 50000 sq m) | 200 strands |

If there were any critical members whose failure would be catastrophic (e.g. transfer girders) some strands would be inspected in all such members.

A statistical method has been developed to assist in selecting sample size to give the desired confidence level that test results represent overall conditions [Pandey and Nessim (1996)]. If there is inherent excess capacity in the original design such that a high tendon loss can be tolerated, the sample size need not be as large. Conversely, if there is minimal tolerance for tendon loss, or if significant tension deficient strand are identified in the initial testing, additional testing may be required to achieve satisfactory confidence in testing.

3.6 Confirmation of super-imposed loadings and as-built construction

Confirmation of current superimposed dead loads such as partitions, toppings, and landscaping often involves exploratory removal of finishes to confirm wall assemblies, thickness of toppings and landscaping, etc.

Removal of roofing ballast, concrete/asphalt toppings, landscaping or architectural finishes in representative areas is performed to assess the condition of waterproofing.

Confirmation of anchor details at expansion joints or construction pour joints may require removal of coverplates and exploratory concrete removal. Grout in anchor pockets is removed at a few locations to assess grout quality, concrete cover, anchor type and signs of moisture contact.

3.7 Recording of field data

The use of standard data collection forms has helped to simplify data recording and reduce the likelihood of errors or missing information. Appendix A includes examples of forms used for inspection recesses and extracted strand inspections. Test locations and results of testing are recorded schematically on key-plan drawings to identify any moisture access patterns and relate signs of external distress to the condition of the post-tensioning system.

3.8 Strand inspection recesses

Removal of concrete cover (by chipping/jackhammering) and inspection/testing of the exposed strand lengths is the most frequently used preliminary investigation method. An inspection recess length of 250-300mm is typically used to allow for inspection and penetration testing of all 6 perimeter wires of the 7 wire strand.

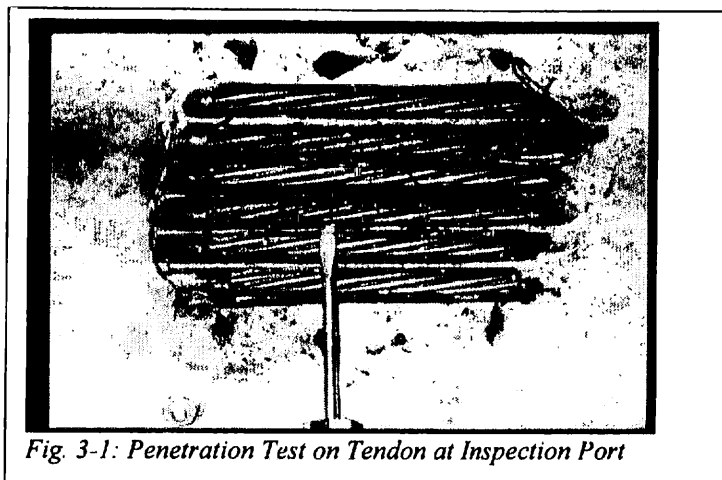
Inspection recesses are normally chipped in the slab soffit at tendon low points where concrete cover is reduced. Recesses are also selected at locations where exposure conditions suggest a potential for concern exists; for example, adjacent to actively leaking cracks, construction joints and expansion joints, or at the first low point in from a slab edge containing unprotected stressing anchorages.

Whenever possible, inspection recesses are chipped on the underside of the structure to avoid the possibility of moisture accessing the strands through the recess.

Inspection of strands is completed immediately after concrete removal to avoid the potential for moisture within the sheathing to drain or evaporate at the recess. Where appropriate, samples of grease and bulk water are retained for further analysis.

3.9 Assessing insitu strand tension

Screwdriver penetration testing is the most commonly used method to assess strand tension at inspection recesses. The method most commonly used in western Canada utilizes a screwdriver and a regular carpenter's hammer (28-32 oz). The screwdriver tip has a standard blade thickness of 0.04 inch, a blade width of 0.25 inch, and a shaft length of 4 inches. The screwdriver tip is placed in the groove between two of the outer wires of the strand. The screwdriver is perpendicular to the strand and struck smartly with the hammer. If the screwdriver tip penetrates between the wires, it indicates that one, or both of the wires touching the screwdriver are not fully stressed. The test is repeated for all 6 outer wires of the strand. If sufficient restraint exists between a strand failure and the inspection recess the penetration test may not identify tension deficiencies in the strand. The test is also subjective as results may vary depending on the operator.



A modification of the screwdriver test is under development at the University of Alberta to overcome these difficulties. It involves mounting the screwdriver tip on a Schmidt hammer. The rebound hammer is a spring-loaded device that delivers a standard impact and measures a “rebound number” which gives an objective measure of resistance to the blow. In trials against the manual screwdriver test, it was found that the rebound hammer based screwdriver test was able to identify more partially tensioned wires. Subtle differences in resistance that go undetected in the manual test are readily detected by differences in the rebound number.

The cut-wire test estimates the force in the strand by measuring the elastic shortening of the strand when it is cut. The separation of strand wires is measured after cutting to assess the net effective tension in the strand.

Mechanical deflectometers have been developed to overcome the subjectivity and uncertainty of the screwdriver test. Commercially available devices are typically used to measure the tension in guy wires and other tension elements. Halsall and Associates have developed a version that is suitable for prestressing tendons. The mechanical deflectometers require a large recess and unobstructed access to about 900 mm of strand that is free to deflect. The device also requires side clearance so that it can grip the strand. The device is reported to have an accuracy of better than 5%.

3.10 Strand extraction and inspection

Removal and full-length inspection of a small number of strands is performed to yield additional information regarding the extent of moisture present in the system and the degree of strand corrosion. Moisture accessing strands at anchorages or at other locations along the strand length may not be detected at inspection recesses, particularly when extruded sheathing has been used. As it is not possible to chip inspection recesses within the highly compressed anchor zone concrete, removal and inspection of strands is the only means to identify if corrosion is occurring at the anchors.

Where strand tension deficiencies have been detected, a sampling of suspect strands is removed to confirm the cause of the tension deficiencies. Strand tension deficiencies have been attributed to:

- Strand breakage due to corrosion.
- Strand broken or cut by workers drilling or coring through member.
- Tendon was not properly stressed initially.

3.11 Metallurgical analysis and testing of strands

Additional testing is often performed on removed strand lengths to identify the deterioration mechanism and gain additional information to help predict the time to failure of corroded strands. Electromagnetic Induction (EMI) and Wet Fluorescent Magnetic Particle inspection (WFMPPI) techniques are performed on strand lengths to detect section loss and stress corrosion cracking of the wires. Sectioning and scanning electron microscopic examination of individual wires at failure surfaces can be used to measure cross-section loss and topology of fracture surfaces for signs of brittle failure mechanisms.

Current practice in Canada is to use seven-wire prestressing strand conforming to ASTM A416 Grade 270 Low-Relaxation Strand. CSA Standard G279 is the reference CSA standard, but most strand in Canada is manufactured to the ASTM specification. Some older structures utilized Grade 270 and Grade 250 Stress-Relieved Strand. If conclusive evidence of the strand type is not found in the project documentation; sections of strand are tension tested to failure to assess ultimate strength and elongation characteristics of un-corroded and corroded samples. The former tests help to identify the grade and type of strand, while the latter give a quantitative measure of the changes in mechanical properties in corroded zones.

3.12 Measurement of humidity within strand sheathing

A proprietary test method has been developed to measure the humidity within the strand sheathing and thereby assess the potential for corrosion of the strands. Injection ports are typically installed at the cable anchors. Dry air is pumped into the sheathing at one port and the exhaust air is measured for relative humidity at the venting port. Increases in humidity in the exhaust air indirectly indicate that there is entrapped water or moist air present within the sheathing that may contribute to further corrosion of the strand. The advantage of this approach in comparison to inspection of strands at recesses is that it yields information regarding moisture along the entire strand length rather than just at the inspection point. This method is potentially applicable to tendons with pushed-through sheaths and heat sealed sheaths. Tendons with extruded sheaths have a tighter fitting sheath that is completely grease filled and will not allow air to flow along the tendon. [NRC (1998b)]

3.13 Analysis of contaminants within strand sheathing

Post-tensioned cables have been found to corrode in the presence of even pure water however, a number of other contaminants have been seen to accelerate the corrosion process dramatically. Bulk water obtained from within the sheathing can be analyzed for the presence of chlorides or fertilizer that may have accessed the sheathing in below grade parking decks or slabs under landscaped areas. Grease samples can be analyzed for the presence of nitrates, valences of sulphur, fungi or

decomposing organics that may be producing hydrogen sulfide. Fourier transform infrared spectroscopy (FTIR) can be used to identify organic species present that may exacerbate strand corrosion rates.

4 Results from investigations

4.1 Limitations of selective sampling

The most commonly used investigation method involves testing of cables at inspection recesses. In numerous investigations where strands have been extracted for full-length inspection, moisture access has been identified at multiple locations along the strand length, or in the immediate vicinity of anchors. Conditions along the strand length have also been observed to vary significantly from conditions at the inspection recess locations. Assessment of post-tensioning systems with extruded sheathing is particularly difficult as moisture accessing the strand at discrete points of sheathing damage or at anchors will not be detected at inspection recesses. Considerable subjective judgment based on past experience is often used when extrapolating testing information obtained at inspection recesses to the overall post-tensioning system condition.

Initial test results obtained will also influence the extent of testing required in individual areas. For example a floor area exhibiting significant strand tension deficiencies in the initial test sample may require expanded testing to confirm that an adequate number of cables are intact and contributing to structural capacity. On this basis investigations are often completed in multiple phases.

Observed conditions reflect the condition at the time of inspection. With the passage of time, actual cable conditions can change. For example, additional moisture may access the strands through failures in waterproofing membranes, or additional strand breakage may occur as a result of continuing corrosion.

4.2 Extent of past moisture access

Evidence of past moisture access to tendons has been seen to vary widely between structures and within individual structures. In general, test results obtained to date suggest the following categories of moisture penetration risk:

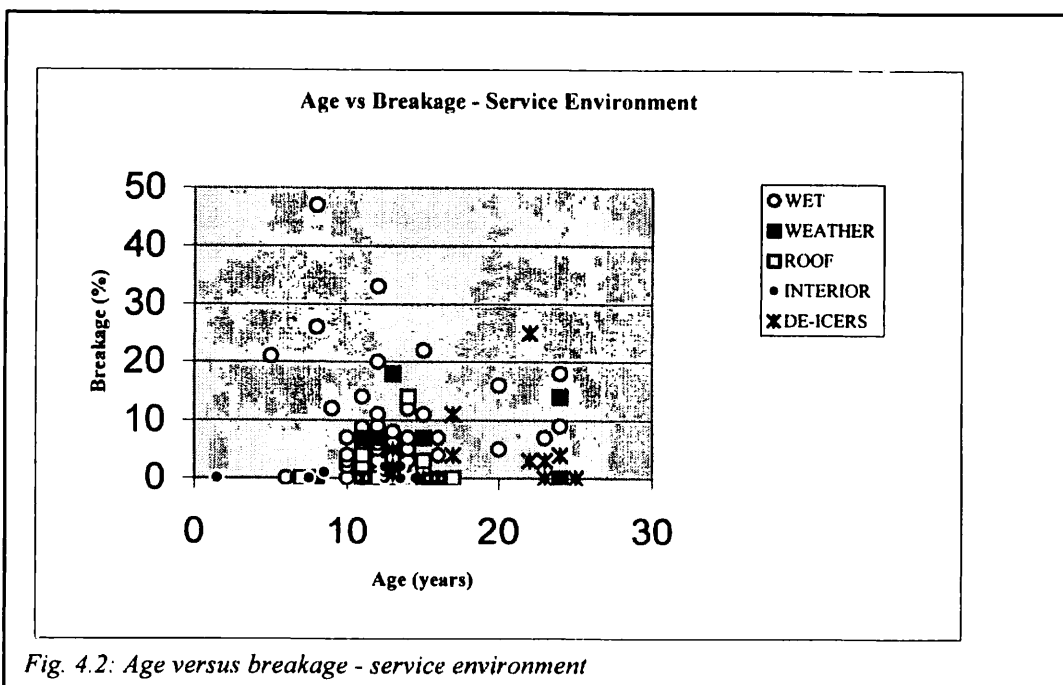
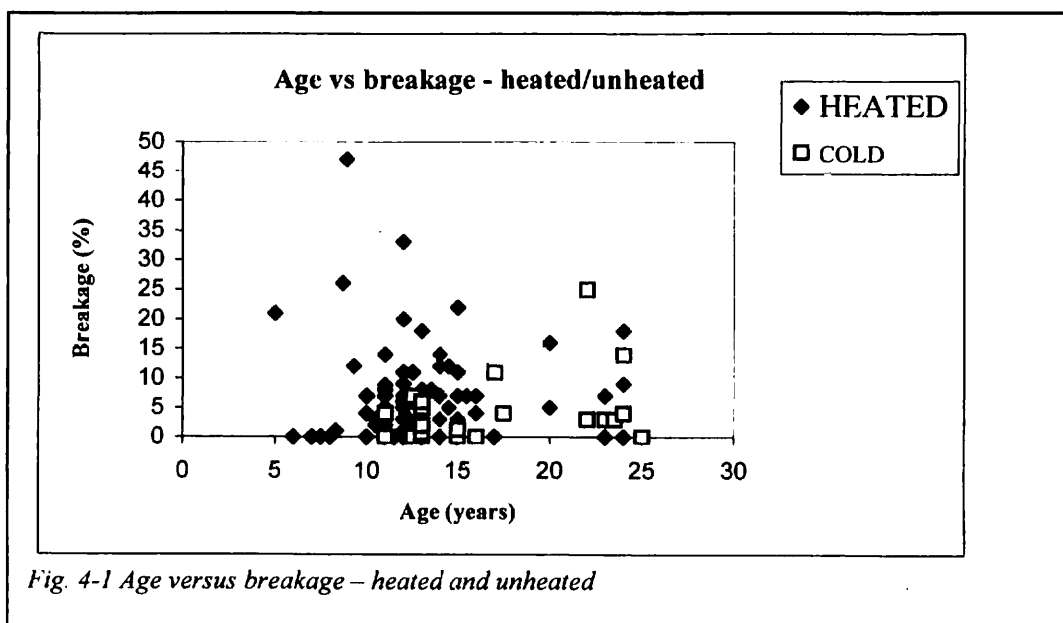
- High Risk: Underground levels where tendon anchors are exposed to ground water infiltration and parking decks without waterproofing.
- Medium Risk: Exposed balcony slabs or plaza levels where rainfall or snowmelt may access anchors at slab edges or penetrate slab cracks.
- Lower Risk: Upper floor interior areas where the slab edges are protected by exterior walls and slabs are in a dry service environment.

A number of investigations have also identified buildings where moisture accessed tendons which were stored outside after fabrication or exposed to rainfall during transportation and construction of the building. In some cases individual floors within highrise towers have been found to contain significant amounts of

moisture within tendons, while adjacent floors have experienced minimal moisture access.

4.3 Timing and extent of strand breakage

Experience to date suggests the major factors influencing the potential for corrosion-induced strand breakage are the amount and extent of moisture access, the ambient temperature of the structure, contaminants within the sheathing and the susceptibility of the strands to develop stress corrosion cracking. Figures 4-1 and 4-2 illustrate the impact of temperature and exposure to moisture in service on strand corrosion rates.

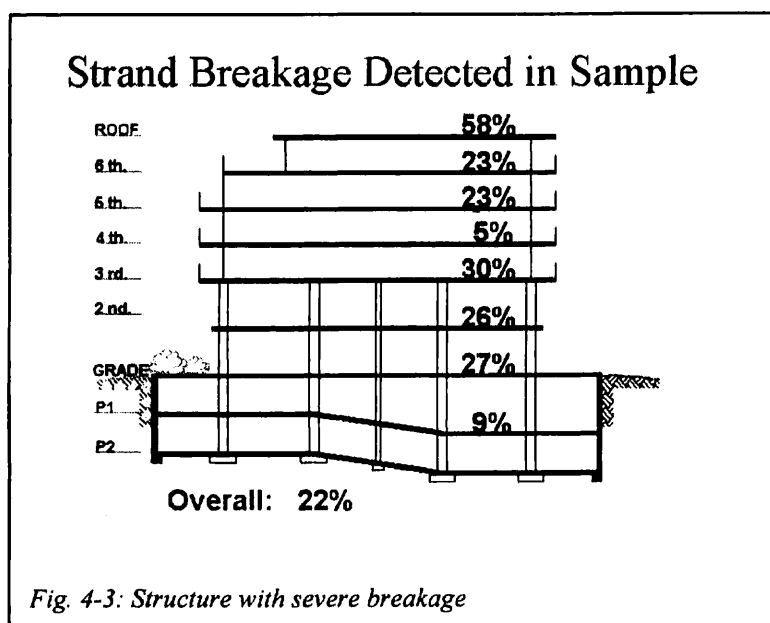


Some structures have experienced widespread strand breakage within 7 years of construction, while strand breakage in others structures is only beginning to occur after 25 years in service.

There is a growing database of information from strand breakage monitoring programs on existing structures, which helps to define trends for particular types of facilities and exposure conditions. Strand breakage has not been found to occur uniformly over time. Typical results suggest corrosion advances within the system to a threshold level, at which time the frequency of strand breakage increases.

4.4 Distribution of strand breakage

The distribution of strand breakage throughout individual structures has been seen to vary significantly, and in some cases counterintuitively to predicted moisture access patterns. Figure 4-3 illustrates strand breakage test results obtained within a 6-storey office/residential complex. In this case the below grade levels experienced lower frequency of strand breakage while above grade floors contained significant amounts of strand breakage.



Constructed details have been seen to influence the distribution of strand breakage. Increased strand breakage has been detected adjacent to unsealed expansion and construction joints, areas of frequent water ponding on slab surfaces, deck cracking, etc.

In a number of structures, extensive strand breakage has been detected during initial investigations, however, no external evidence of cracking or deflection was observed. This suggests that with increasing levels of strand breakage, some structures may not fail in a ductile manner with adequate warning.

4.5 Strand failure mechanisms

Assessment of strand metallurgy and corrosion mechanisms is a relatively new area of study with limited understanding of how specific factors relate to time to failure of strands. In the majority of investigations completed where strands have been extracted for examination, the failure surfaces typically exhibit a brittle fracture appearance, with little loss of cross-sectional area as illustrated in Figure 4-4.

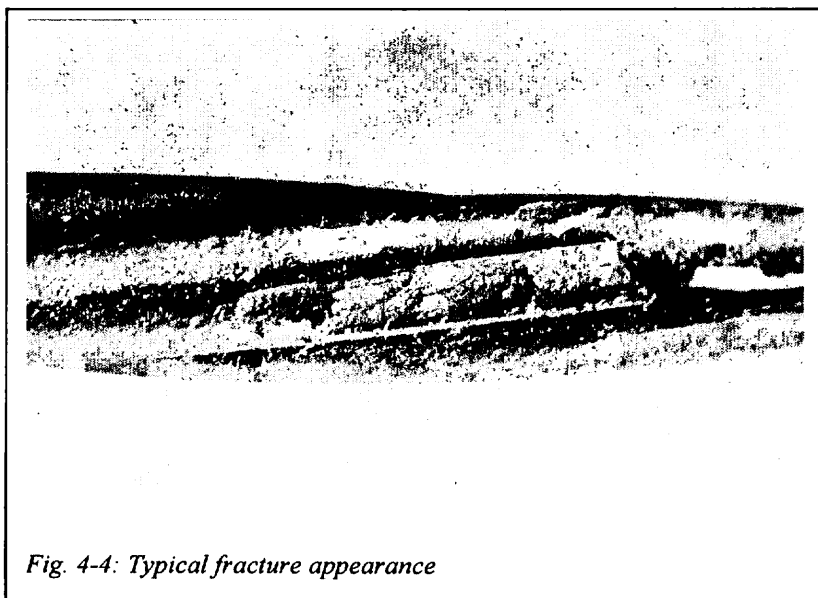


Fig. 4-4: Typical fracture appearance

Information to date suggests that chlorides, nitrates, valences of sulphur, and fungal mechanisms can contribute to accelerated strand failures.

4.6 Risk of strand eruptions

Failures of strands have occurred quite suddenly with a significant amount of energy released. Instances have been recorded where strands have projected from anchors or erupted in loops projecting from the top or underside of the structural slabs. Considering that a significant number of strand failures have been identified by testing, the frequency of strand eruptions to date has been quite limited (less than 1%), however the potential for personal injury or property damage still exists.

Experience to date suggests that strands with at least 19mm concrete cover at high and low points in the strand drape are unlikely to create a loop type eruption in the event of breakage. Eruption potential at anchors has been seen to vary depending on concrete cover, length of the strand, tightness of sheathing, and the location of the strand failure.

4.7 Strand breakage monitoring

When testing has identified past or present moisture penetration into the P/T system, continued monitoring of cable tension is typically recommended to identify any future strand breakage and implement repairs in a timely fashion. Annual testing of strand tension is recommended initially, with frequency of monitoring dependent upon results of testing. If no change in strand tension is identified within the first 3 years the period between update testing is often extended.

Retesting strand tension at existing inspection recesses is frequently used to identify any changes in the post-tensioning system. However strand breakage rates can vary widely between different structures and within individual structures, and test results obtained at inspection recesses may represent less than 2% of the total number of strands within the structure. If advancing strand breakage is detected in an area, additional strands are often exposed and tested to confirm that an adequate number of intact strands are present within a particular structural bay.

A proprietary monitoring system has been developed which detects the sudden release of energy that is associated with wire failures within strands. A series of sensors distributed throughout the structure transmit information to a computer which analyzes the acoustic profile of the event, and through triangulation identifies the location. The advantage of this system in comparison to testing strand tension at recesses is that it identifies strand breakage throughout the structure rather than only identifying strand breakage within the sample. The increased level of information obtained can lead to less conservative repair recommendations based on limited information. [Elliot, McCarthy (1998)] [NRC (1998a)]

5 Remediation

5.1 Repair Methods

Methods which have been used include the following:

- Structural steel sub-frame.
- Concrete (e.g. shot crete) sub-frame.
- Externally applied steel or advanced composite reinforcing,– top reinforcing can be grooved-in or embedded in a new topping slab, bottom reinforcing can be attached by bolts and/or epoxy and other innovative techniques.
- External post tensioning.
- Demolition and reconstruction.
- “Rethreading” of tendons: removal of strands, cleaning of sheath, and insertion of new strands and new grease. Epoxy strands can be used. The authors have found that the epoxy coating can be damaged during the re-threading and re-stressing process.
- Combinations of the above.

Often the designer must work from first principles and use innovative techniques, because the existing conditions, or the novelty of solutions, place one in an area not addressed by building codes. For example, if the post tensioning system is abandoned, the remaining concrete will sometimes be un-reinforced. In such a case, one may rely on the concrete acting as an unreinforced flexural member, or preferably acting as an un-reinforced arch, spanning a short distance, say less than 10 times the slab thickness.

5.2 Management of damaged structures

As many structures are not in immediate need of repair, many owners wish to delay repairs in which case the question becomes one of managing the situation. Measures which can be taken include:

- Prevention of ingress of additional moisture, de-icer salts, etc. into the tendons.
- Repairing only those areas where safety, or convenience, dictate that repairs be done at a particular time
- Protection against eruption, at anchorages or at locations with deficient concrete cover, in areas where the public could be placed at risk.
- Implementing new techniques to dry out moisture from voids inside the tendons (e.g. Gas Purge technique). This technique does not have a long established track record but is one of the few means available to mitigate future corrosion.

5.3 Performance of repaired structures

As many of the repair methods are innovative, it is important to monitor their performance.

Some of the earlier re-threading operations did not involve replacement of the entire length of the original strands, nor filling of the annular void with grease. In one instance, the authors found breakage of new tendons, as well as some sections of the original tendons which had been left in place. The original repair was done five years after construction. Some breakage of new and old tendons was found approximately 15 years later.

6 Recommendations to avoid technology problems in the future

6.1 Precautions for existing unbonded structures

Structures, or portions of structures, in “high risk” categories (Refer to Section 4.2) should be investigated to establish that they are safe. Other structures may also require investigation depending on the needs of the particular owner, lender or purchaser.

Structures which have been repaired previously in some cases should be monitored, especially if new or innovative techniques have been employed, depending upon the track record or perceived risks inherent in the methods of repair that were used.

The body of knowledge is constantly growing. One must periodically reassess the risk related to existing structures, and the performance of the repair methods used, as more information becomes available from ongoing monitoring and investigations.

6.2 Improvements to new construction

During the past 15 years, many improvements have been made to practices and building code provisions for design and construction. It is important that these improvements be incorporated into new construction. The improvements introduced in Canada include, among other things:

- Increased emphasis on durability in design standards [CSA (1994, 1995)].
- All unbonded systems must have waterproof (encapsulated) tendons without voids, increased sheathing thickness (1.5 mm), multiple lines of defense against moisture and salt ingress, and rejection of tendons if a “trace” of water is found.
- Full time inspection is required during concrete placement.
- Mandatory training is required for workers for post-tensioning work on parking structures. [CSA (1994)]

6.3 Future considerations regarding unbonded construction in Canada

Among the new systems available on the market, some are more vulnerable than others to moisture ingress during storage, transportation and installation. Some systems may tend to have voids within the anchorage systems, which present a risk similar to that which existed in old “push-through” systems. Improvements should be made to such systems to make them less susceptible to damage or moisture ingress during construction.

Laboratory testing of tendon waterproofness should also be improved to more closely simulate field conditions.

Some practices no longer permitted in Canada are still permitted in the U.S. For example, non-waterproof tendons are permitted by ACI for some applications. Re-introduction of these practices must be avoided.

Continuing education of designers and builders will be required to reinforce the reasons for the current precautions and the consequences of not maintaining high standards. (Designers and builders in future will not have had the “benefit” of the painful experiences of the past two decades.)

6.4 Learning from experience with past problems

The problems with unbonded post-tensioning arose because people did not foresee or did not address problems that were inherent in the new products or methods. Some of the lessons learned and experience gained in the course of dealing with durability issues in unbonded post-tensioning can hopefully be applied in the future, not only to durability of post-tensioned structures, but also to instances where new technology is being introduced, or where a problem is beginning to surface.

Design professionals generally should receive improved training, at the university level as well as Continuing Education after university, on the topic of durability.

Design professionals often develop new techniques (or judge new systems developed by others) based on judgment and past experience. New technology, on the other hand, requires a more inquisitive, analytical or “scientific” approach than has been applied in the past. This could be achieved in part through education of design professionals. Increasing regulatory constraints on new methods is another alternative but has the disadvantage of inhibiting new methods and innovation.

For the most part, exchange of experience and information among engineers has been quite open, facilitated by relationships between engineers in competing firms, and by seminars arranged by local, national, and international technical organizations. Confidentiality and legal concerns can and must be overcome in favor of open exchange of information. For example, where confidentiality is a concern, information can often be exchanged openly by avoiding specific reference to a project. It is important that this exchange of information continues, especially in cases where the information has wide implications for safety, improving technology and avoiding unnecessary cost to society.

The time required to fully identify and address the unbonded post-tensioning durability issue was amazingly long, (from first suspicions of a problem, to recognizing the cause and extent of the problem, to finding solutions, to implementing new practices). In the event that problems are discovered in future with a product or technology, one would hope to shorten the time required to recognize the problem and address it.

In retrospect, engineers were constrained in their thinking by virtue of their own past experience. People were fundamentally reluctant or slow to respond to new data that indicated that problems existed. People expected external evidence of problems. Based on experience with parking structures and bridges, people also expected deterioration to require aggressive chemical agents such as road salts. A real paradigm shift was required. The first step in overcoming inertia and solving future issues more quickly is to recognize the inherent human tendency to be slow to respond to new stimuli.

Knowledge of the need for improved standards did not cross the Atlantic quickly. Durability issues were raised and addressed in the late 1970's in Europe. [Etienne et al (1981)] This information began to be disseminated in North America

in the 1980's. [PTI (1985)] Practices did not change in most cases until the late 1980's and even into the 1990's. Methods must be found to increase the rate at which information is exchanged and practices are improved.

Fortunately, commercial interests were not major factors in slowing down progress or improving practices in Canada, once the problems became apparent. Several individuals in the industry realized the need for improvement and participated in code and industry committees to implement changes.

For new construction, increased field inspection, and education at the level of the construction worker have been very helpful. (These developments were initiated in the Vancouver area. At the time, most of the country was in recession and Vancouver was the only region with significant construction activity). The inspectors and workers gained an understanding of the reasons why special care was needed, and the consequences of not applying good practices. At the construction site, the special care identified for post tensioning placed a higher status on this trade on the job site and reduced the likelihood of the post tensioning being compromised by actions of other trades.

In Calgary, the public education process was effective. A large inventory of post-tensioned buildings was built in the boom years of the 1970's and early 1980's. Consequently, the problems were recognized and addressed earlier than in other parts of the country. Education of building owners and stakeholders was broadly implemented by engineers and by owners' organizations. Issues were explained to owners in common-sense terms. There were many opportunities for them to learn about the concerns and gain a broad understanding. Extreme reactions (denial, or alternatively over-reaction) were the exception. The majority of owners realized the need to act responsibly where safety was concerned. Mortgage lenders also began to require assessments of post-tensioned buildings upon renewal of mortgages. Hence a large proportion of the building inventory in Calgary has had some degree of assessment.

Government funding for research activities has been cut back over the past decade. In addition, no single private stakeholder or industry organization could justify an economic return from investing in research on post-tensioning deterioration. Some research has been conducted by government agencies. However, pure research on the topic of post-tensioned construction has been limited

References:

ACI 423 (1998) "Corrosion and repair of unbonded single strand tendons, ACI 423.4R-98." American Concrete Institute, Farmington Hills, MI pp. 1-20.

CSA (1994), "CSA A23.1/A23.2-94 Concrete Materials and Methods of Concrete Construction/Methods of Test for Concrete", Canadian Standards Association, Rexdale, Ontario, Canada

CSA (1994) "CSA A23.3-94 Design of Concrete Structures", Canadian Standards Association, Rexdale, Ontario.

CSA (1995), "CSA Standard S478-95 (R2001) Guideline on Durability in Buildings", Canadian Standards Association (1995), Rexdale, Ontario, Canada

CSA (1994), "CSA Standard S413-94 Parking Structures", Canadian Standards Association (1995), Rexdale, Ontario, Canada

Elliott, J. F. and McCarthy, D. J. (1998), "Continuous Monitoring of Existing Prestressed Concrete Structures: Applications and Results", American Concrete Institute 1998 Spring Convention, Seattle WA.

Etienne, C.F., Binnekamp, D.C., et al, (1981); "Corrosion Protection of Unbonded Tendons", Heron, (Delft), Vol. 26, No. 3, 1981, 74 pp.

Harder, J.A. (1997) "Reliability Based Evaluation and Monitoring of Structures With Unbonded Post-Tensioned Tendons" American Society of Civil Engineers Structures Congress XV Proceedings, pp. 329-333.

Harder, J.A., and Rogowsky, D.M. (1999) "Inspection and Monitoring of Buildings with Unbonded Prestressing." 1999 Structural Faults + Repair Conference Proceedings, Commonwealth Institute, London, UK.

McGuire, G.J. (1989) "Unbonded Post-Tensioned Construction: The Canadian Perspective." Paper presented to ACI Alberta Chapter, November 7 and 8, 30pp.

NRC (1996) "Commentary K Application of NBC Part 4 for the Structural Evaluation and Upgrading of Existing Buildings." *User's Guide – NBC 1995 Structural Commentaries (Part 4)*. Canadian Commission on Building and Fire Codes, National Research Council of Canada, Ottawa, Ontario, pp. 93-102.

NRC (1998a) "Effectiveness of an Acoustic Continuous Monitoring System for Post-Tensioned Buildings – An Evaluation Report", National Research Council Canada Rahman, A.H. and Pernica, G.

NRC (1998b) "Effectiveness of a Corrosion Potential Method for Evaluating Post-Tensioned Tendons – An Evaluation Report", National Research Council Canada Pernica, G. and Rahman, A.H.

Post-Tensioning Institute (1985) Ad-Hoc Committee for Unbonded Single Strand Tendons, "Specification for Unbonded Single Strand Tendons", PCI JOURNAL, Prestressed Concrete Institute, Vol. 30, No. 2, March/April 1985, pp 22 – 39.

Rogowsky, D.M., and Webster, N.R. (1999) "Strength Evaluation of Unbonded Post-Tensioned Structures with Severe Breakage", 1999 Structural Faults + Repair Conference Proceedings, Commonwealth Institute, London, UK.

Webster, N.R. (1989) "Unbonded Monostrand Post-Tensioned Construction – A Question of Durability" Presentation to Canadian Society of Civil Engineers, Vancouver, B.C.

P/T RECESS INVESTIGATION LOG

Page ____ of ____

Project _____

Job No. _____

Inspected By: _____

Date: _____

[illegible]

Grease Condition

G	Apparently good condition
E	Emulsified
R	Runny/oily
D	Dry Strand
M	Cement mortar between strands

Strand Condition

G	Apparently good condition
R	Rust Staining, no pitting
IP	Intermittent pitting, some shiny strand
HP	Heavy pitting, no shiny strand
B	Broken / not stressed

* Indicate number of wires where penetration achieved

P/T STRAND INVESTIGATION LOG		PROJECT _____	PAGE _____
		JOB NO. _____	
		INSPECTED BY _____	
		DATE _____	

STRAND IDENTIFICATION	
RUNNING DIMENSIONS (mm) 	Length = _____
STRAND CONDITION G _____ R _____ IP _____ HP _____	_____ _____ _____ _____
STRAND DISCONTINUITY _____ _____ _____ _____	_____ _____ _____ _____
GREASE CONDITION G _____ E _____ O _____ D _____ M _____	_____ _____ _____ _____ _____

ANCHOR PRESENT _____ ANCHOR CONDITION _____		LOW POINT ALONG CABLE PROFILE	ANCHOR PRESENT _____ ANCHOR CONDITION _____
GREASE CONDITION G Apparently good condition E Emulsified O Runny/oily D Dry strand M Cement mortar in sheath			STRAND CONDITION G Apparently good condition R Rust staining, no pitting IP Intermittent pitting, some shiny strand HP Heavy pitting, no shiny strand
			STRAND DISCONTINUITY C Cut* CT Torch cut* T Cup/Cone Tension failure (ductile)* F Fractured (brittle)*

* Indicate number of wires affected

PROJECT _____

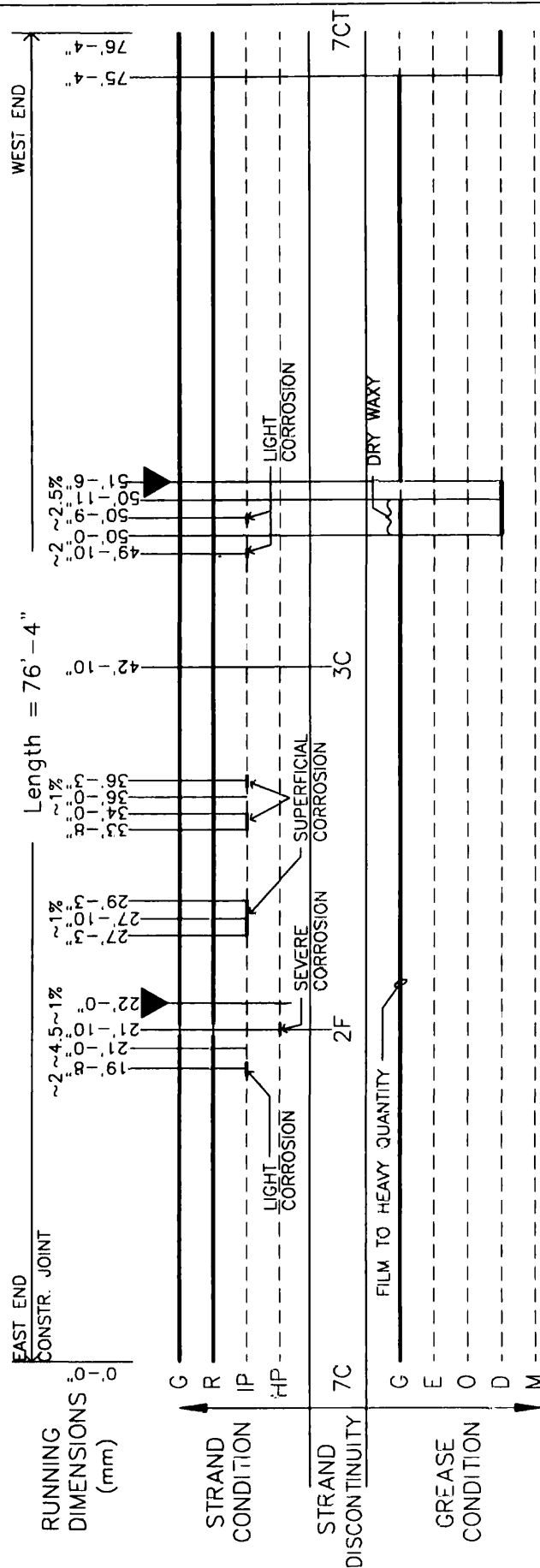
JOB NO. _____

INSPECTED BY _____

DATE _____

STRAND IDENTIFICATION

92S447.1



ANCHOR PRESENT	LOW POINT ALONG CABLE PROFILE	ANCHOR PRESENT
NO		NO
-		-
ANCHOR CONDITION		ANCHOR CONDITION

GREASE CONDITION		STRAND CONDITION		STRAND DISCONTINUITY	
G	Apparently good condition	G	Apparently good condition	C	Cut *
E	Emulsified	R	Rust staining, no pitting	CT	Torch cut *
O	Runny/oily	IP	Intermittent pitting, some shiny strand	T	Cup/Cone Tension failure (ductile) *
D	Dry strand	HP	Heavy pitting, no shiny strand	F	Fractured (brittle) *
M	Cement mortar in sheath				

* Indicate number of wires affected

* Indicate number of wires affected

Present Situation of Durability of Post-Tensioned PC Bridges in Japan

H. MUTSUYOSHI

Professor, Department of Civil and Environmental Engineering,
Saitama University, Saitama, Japan

1 Introduction

The first actual prestressed concrete (hereafter PC) structures constructed in Japan were sleepers for railways in 1948. The first PC bridge, Tyousei Bridge located in Ishikawa Prefecture was built in 1951. With the industrialization, numerous PC bridges have been constructed all over Japan to cope with the demand for better and efficient transportation facilities in the following decades. Fig. 1 shows the numbers and total lengths of highway bridges with spans longer than 15 m in Japan as of April 1, 1997. Percentages of steel, RC and PC bridges are 40%, 20% and 36 % respectively out of 132,000 highway bridges. Currently, 2000 to 2500 bridges are being built a year in Japan. Fig. 2 shows the trend of construction of different kinds of highway bridges with increasing year. Many bridges were built in the boom period of 1965 to 1980 and the number of PC bridges constructed is increasing with year. The reason why the number of PC bridges is increasing is that the initial cost and life cycle cost of PC bridges including repair and maintenance are cheaper than those of steel bridges. Most of such PC bridges have been constructed using post-tensioned steel tendons with grouting in sheaths. With such large stocks of existing PC bridges in running transportation networks, it has become extremely important to assess the structural condition of these structures in terms of their durability.

Deterioration of existing concrete structures has become a serious problem today all over the world. Major forms of deterioration of concrete structures are corrosion of steel bars and prestressing tendons due to chloride attack, alkali aggregate reaction, spalling of concrete due to cold joint, etc. In the case of PC structures, apart from the above types of deterioration, quality of grout in sheaths is a very important factor for life and durability of post-tensioned PC structures. Recently, construction of post-tensioned PC structures with grouting has been forbidden by the Japan Highway Public Corporation, which governs most of the highway structures in Japan because bad quality of grouting after construction has been found in many PC bridges that were constructed in the boom era. Now, almost all new PC structures are being constructed using fully external tendons using transparent sheath with grouting. Fig. 3 shows the influence of various factors that cause deterioration of existing PC bridges. The figure shows the results of an investigation conducted on 120 PC bridges in Japan. It is seen from the figure that the major factor causing deterioration of PC bridges is chloride attack followed by bad grout condition. Major forms of deterioration due to chloride attack are spalling of concrete and corrosion of tendons, whereas bad grouting causes the breaking of prestressing tendons.

This paper describes the present situation of quality of grouting in PC bridges in Japan based on an extensive investigation of many existing PC bridges by x-ray penetration and impact elastic-wave method and of deterioration of PC bridges due to chloride attack.

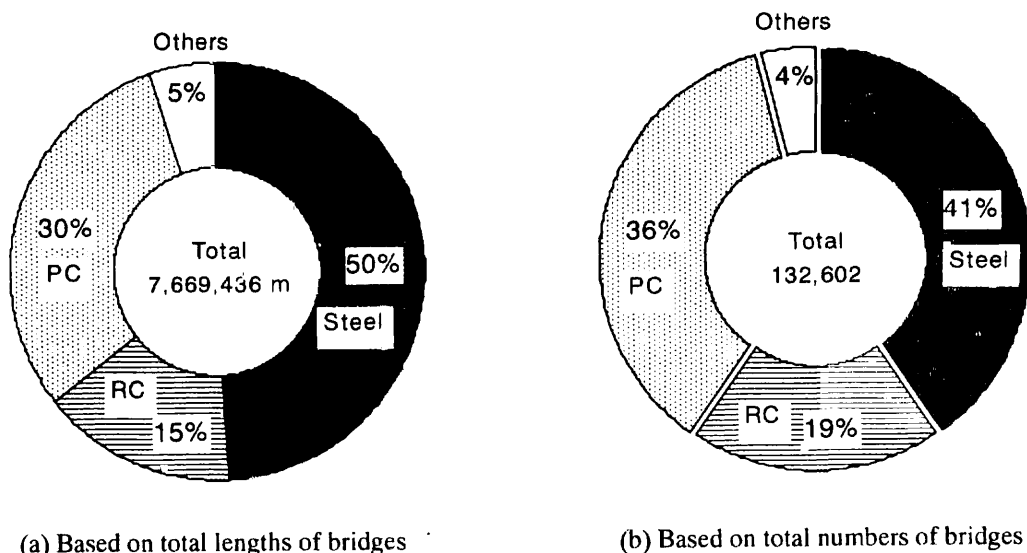


Fig. 1: Total lengths and numbers of highway bridges having length >15 m as of April 1st 1997 in Japan [1]

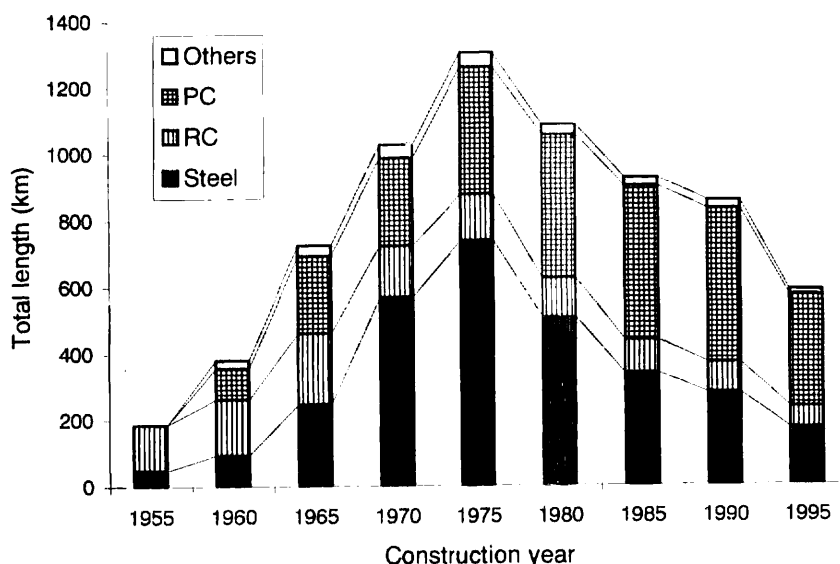


Fig. 2: Trend of construction of different types of bridges in Japan with increasing year [1]

2 Investigation on quality of grout

In order to investigate the quality of grouting in existing post-tensioned PC bridges more than 10 years after the construction, 84 bridges from all over Japan were selected at random. Total numbers of main girders were 340 and numbers of tendons investigated were 620. These bridges have been built between 1962 to 1985. Using nondestructive testing methods such as x-ray and impact elastic-wave methods, the condition of degree of grouting in main tendons in main girders and lateral tendons in lateral girders were investigated. Investigation

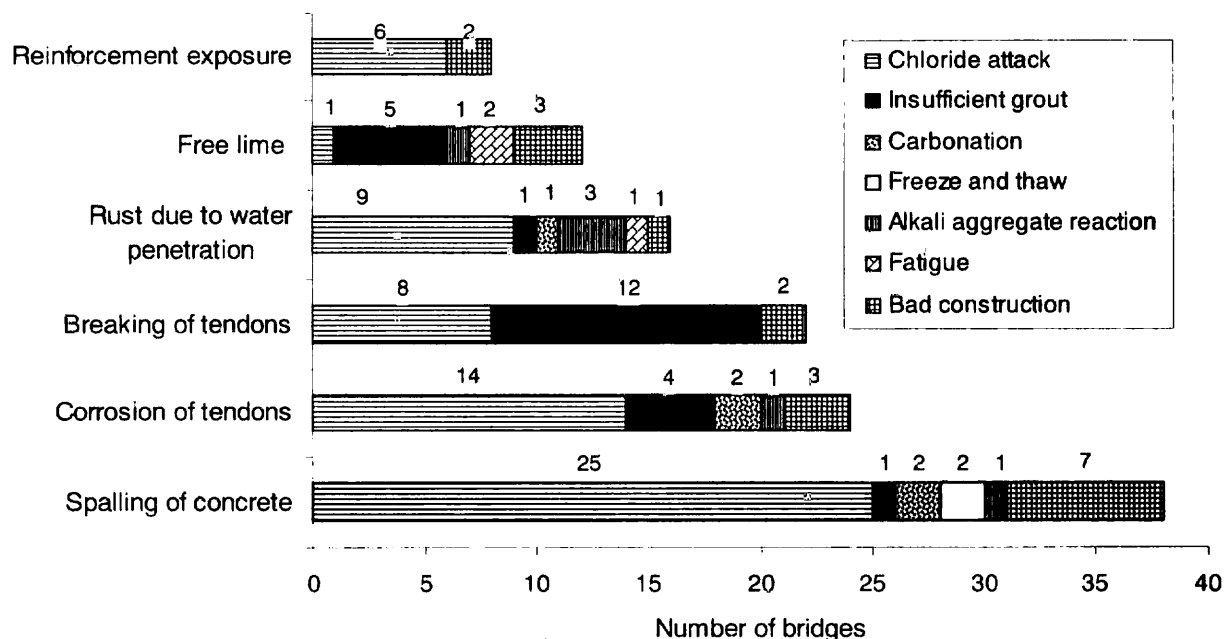


Fig. 3: Influence of various factors on deterioration of PC bridges (120 bridges) [4]

Table 1: Details of the investigated bridges

Post-tensioned girders		Pre-tensioned girders	
Type	Numbers	Type	Numbers
Simple T girder	8	Simple T-girder	20
Continuous T girder	52	Continuous T girder	1
Simple composite girder	38	Connected T girder	2
Continuous composite girder	15		
Connected composite girder	2		

Table 2: Methods of investigation

	Main cables (tendons)	Tendons for lateral girders		Lateral tendons for slab
Method	x-ray penetration	x-ray penetration	Impact elastic-wave	Impact elastic-wave
Quantity	620	325	533	2302

results were defined into five categories, A: Perfect grout, B: Grout with some voids, C: Inadequate grout, D: Imperfect grout and E: No grout. Except the perfect grout condition, other conditions are termed to be bad grouting and appropriate repair and rehabilitation strategies are needed. Table 1 shows the details of investigated bridges and Table 2 shows the methods of investigation of bridges.

Results from the investigation are presented in terms of above-mentioned grout conditions inside the sheaths of PC bridge girders. The results are given in terms of percentages of the observations in each category of tendons. Summary of the results is shown in Fig. 4. It is found that for main cables, almost 69% of total 620 investigated tendons were in perfect grout condition, whereas about 31% tendons showed imperfect grout condition ranging from minor voids to complete loss of grout. The complete loss of grout, i.e. no grout condition was found in 19 tendons out of 620 tendons investigated. In the case of lateral tendons for lateral girders almost half tendons were in imperfect grouting condition. About 6% tendons out of total 325

investigated were found to have complete loss of grout. So it is necessary that such imperfect tendons should be repaired as soon as possible either by new grouting or epoxy injecting. In severe cases, the whole tendon set up should be replaced.

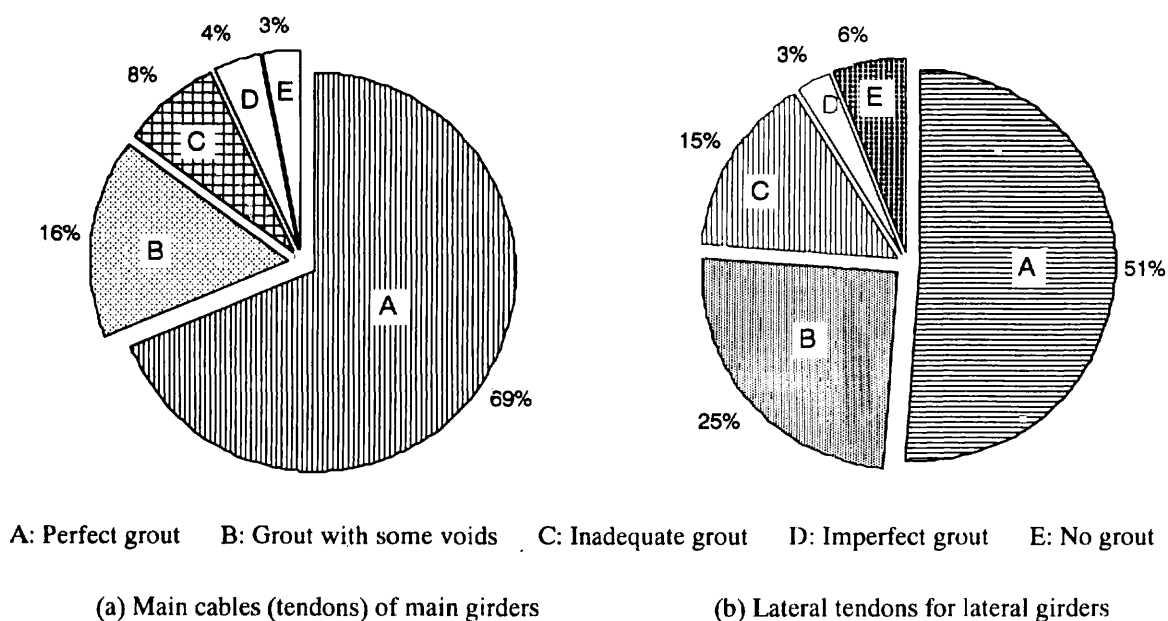


Fig. 4: Quality of grout in existing PC bridges by x-ray penetration method

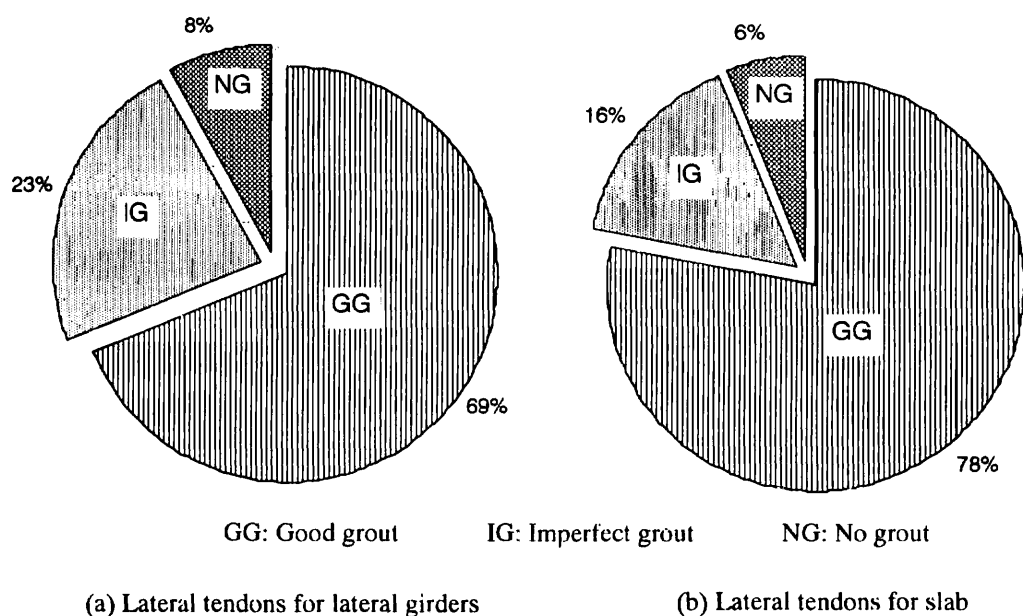


Fig. 5: Quality of grout in existing PC bridges by Impact elastic-wave method

Fig. 5 shows the results by impact elastic-wave method on lateral tendons for lateral girders and lateral tendons for slab. It is observed that relatively high percentage of grout was in good condition for slab. About 22 percent tendons were found to have defective grout either imperfect and complete loss of grout. Complete loss of grout in lateral girders were found to be 44 (8%) out of 533 observations and 135 (6%) out of 2302 observations in deck slabs of investigated bridges. Fig. 6 shows the observation results of a type of tendon. It is seen that the strand type cables showed little larger percentage of better quality of grouting

than the wire type. In strand types, the lowest two quality categories were nil and relatively larger percentages were in higher quality categories. The reason might be due to the fact that in the case of prestressing steel strand, anchors were at the end of the girders and sheath diameters were also larger than 65 mm in all cases.

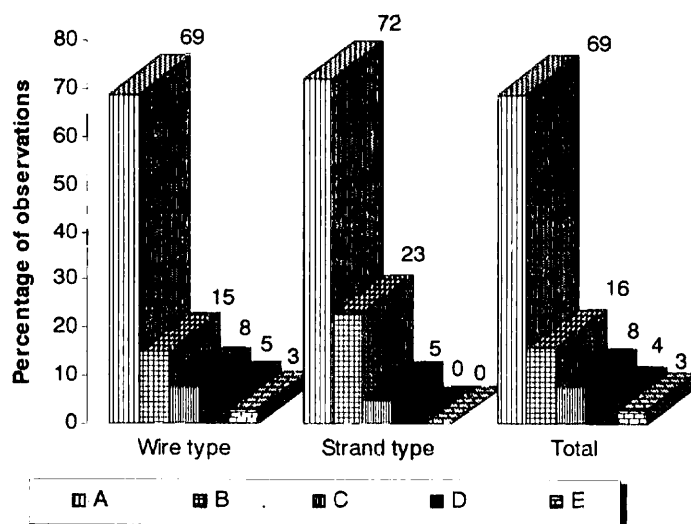


Fig. 6: Influence of type of tendons on quality of grout

Fig. 7 shows the influence of anchorage position of cables on grout condition. It is seen that relatively larger percentages of tendons were found to be in perfect grout condition when anchorages were provided at the top. However, the trend is not clear enough, the rest categories showed better results in the case of an end anchorage. Fig. 8 shows the influence of length of girders on grout condition. It is found that the better grout conditions were found in shorter lengths. For longer lengths, average of 67% of observed cables out of 524 were found to have grouts in perfect condition. For shorter length girders smaller than 15 m, no grout and imperfect grout categories were nil.

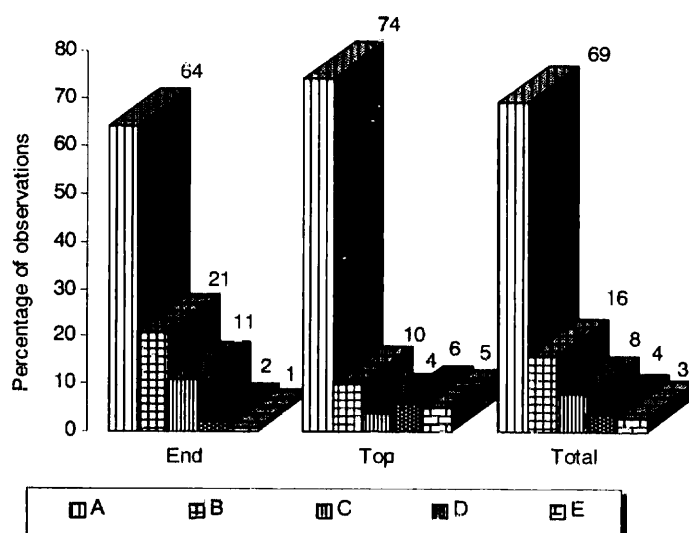


Fig. 7: Influence of anchorage position of cables on quality of grout

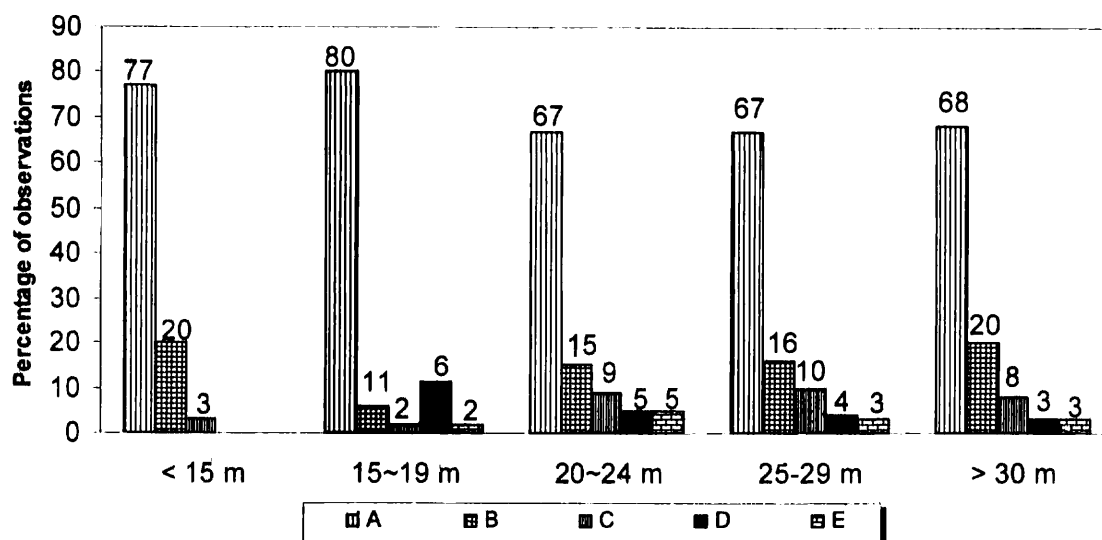


Fig. 8: Influence of length of girders on quality of grout

Table 3 shows the construction year versus observed grout quality of all the investigated bridge girders. Many girders showed 100 % perfect grout condition even after 25 years, but some new girders showed somewhat low percentages of category A. In fact no correlation could be drawn in respect to construction year for the grout quality of these PC bridges. However, the perfect quality condition was varied between 50% to 100 %. Thus almost half observed cases in some particular years had bad or deficient grout conditions. These observations present serious durability problems in such post-tensioned PC bridges in Japan.

Table 3: Construction year versus observed quality of grout

Construction year	Perfect grout (%)	Grout with voids (%)	Inadequate grout (%)	Imperfect grout (%)	No grout (%)	Total (numbers)
1966	81	0	3	16	0	32
1967	74	5	5	11	6	84
1968	55	29	8	3	5	38
1969	77	16	4	0	3	73
1970	100	0	0	0	0	4
1971	72	22	6	0	0	36
1972	62	15	19	2	2	47
1973	51	28	13	2	6	83
1974	75	10	10	0	5	40
1975	78	11	11	0	0	9
1976	67	8	0	25	0	12
1977	60	7	20	7	7	15
1978	50	25	25	0	0	8
1979	56	38	4	0	2	48
1980	70	13	3	13	0	30
1981	88	8	0	4	0	24
1982	88	0	12	0	0	17
1985	100	0	0	0	0	14

Fig. 9 shows the influence of diameter of sheath on grout condition. It is seen that with large diameter sheath, grout condition relatively stays good. For a 65 mm diameter sheath,

about 94% grout remained in category A. For all diameters greater than 57 mm, lower three categories of grout were not observed. This might be due to good filling of grout in large sheath compared to smaller diameter sheaths. Fig. 10 shows the relationship between space ratio of sheath and grout quality. Space ratio of sheath is given by eq.1 as given below.

$$\text{Space ratio} = \left(1 - \frac{\text{Total area of tendons}}{\text{Sectional area of sheath}} \right) \times 100 \quad (\%) \quad (1)$$

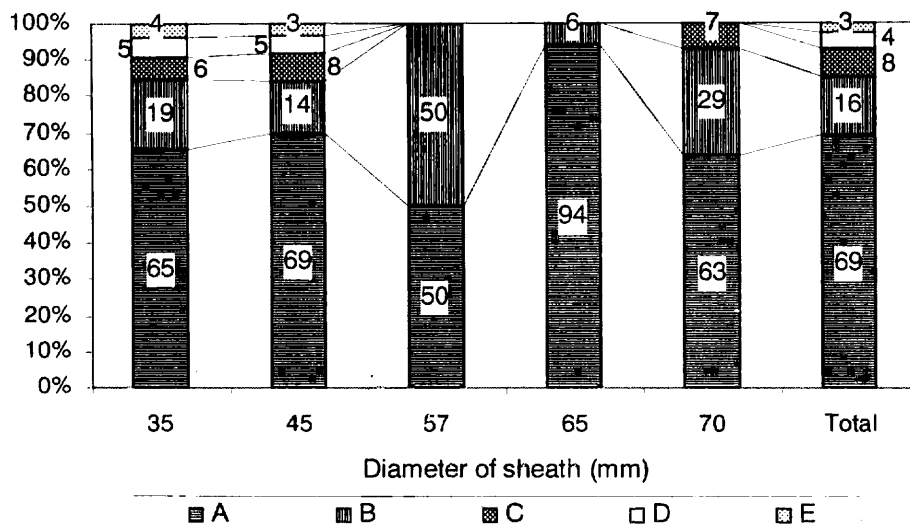


Fig. 9: Influence of diameter of sheath on quality of grout

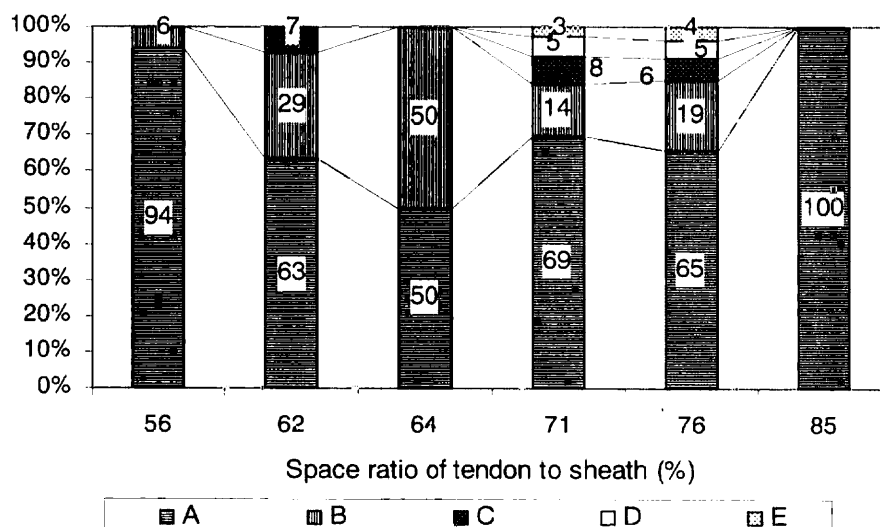


Fig. 10: Relationship of space ratio of sheath and quality of grout

It is seen that the space ratio did not have any significant correlation to the quality of grout of investigated tendons. For highest space ratio of 85%, grout quality was perfect for 100%, at the same time lowest space ratio of 56% also showed 94% perfect grout quality. Fig. 11 shows the influence of relaxation of tendons or loss of prestressing force on quality of grout. Out of 620 tendons investigated, 70 showed smaller prestressing force than the initial values and 11 tendons had no prestressing force in them. From the results, it can be seen that prestress relaxed girders showed somewhat lower percentage of perfect quality grout. However, almost similar results were observed for lower quality of grouts. Fig. 12 shows the relationship between damage of girders and grout conditions. Damage means visible cracks,

scaling, spalling, reinforcement corrosion, tendon corrosion and other visible defect on surface of the girders. It is seen that compared to undamaged girders, damaged girders showed larger percentage of bad quality grout. Out of 542 tendons studied in undamaged condition, 71% grout was in category A, and the rest had some sort of defects. In case of damaged girders, out of 88 tendons investigated, about 60% grout was in perfect condition, i.e., about 10% reduction from the case of girders with no damage. Complete loss of grout was observed in 11% of tendons in the case of girders with damage in lower flange.

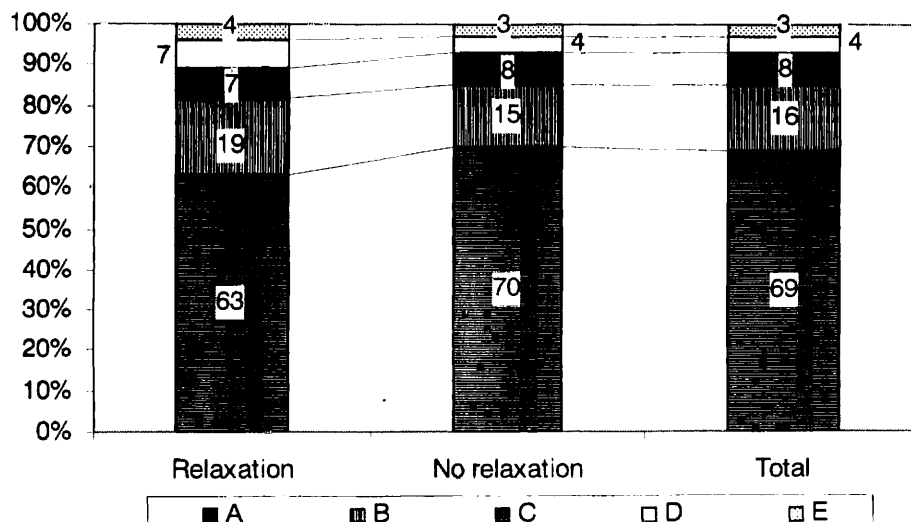


Fig. 11: Influence of relaxation of tendons on quality of grout

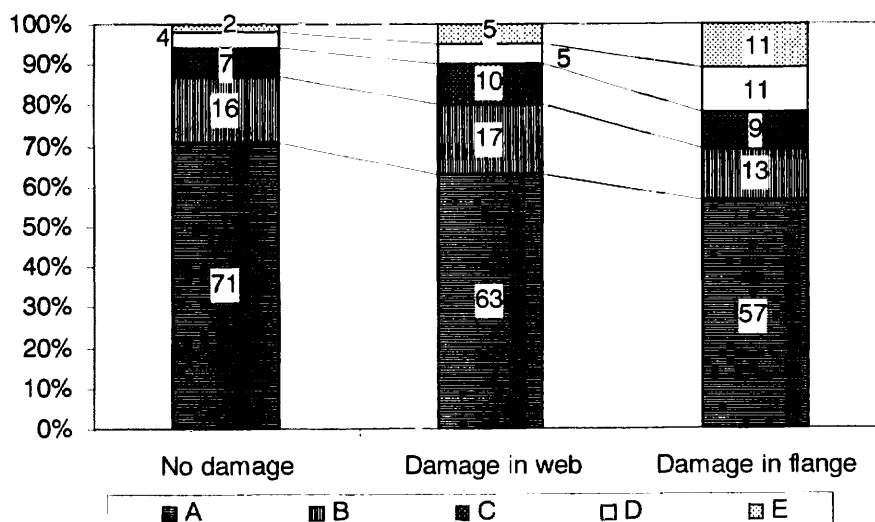


Fig. 12: Relationship between damage of girder and quality of grout

3 Deterioration due to chloride attack

The major factors that influence the deterioration of PC bridges are chloride attack, alkali-aggregate reaction, freezing and thawing action, carbonation of concrete and fatigue due to live loads. Chloride attack has been found to be the severest factor in Japan and many PC bridges have been found deteriorated due to this factor. Thus, it is supposed that chloride attack will be the most important factor in future durability of PC bridges in Japan. Presence

of chloride in concrete is due to various reasons such as, use of sea sand in concrete construction, spindrift with salt in coastal area, and use of deicing agents such as calcium chloride on road surface of PC bridges. This chapter describes an example of a deteriorated PC bridge due to chloride attack and a new idea of design method to prevent chloride attack assuming the life of the bridge is more than 100 years.

Fig. 13 shows the Kuretsubo Bridge located in the coastal area of Japan Sea. This bridge was built in 1965 by post-tensioned PC girders having T-shape. The total length of this bridge is 144 m with 5 spans. Some signs of rust in reinforcements due to spindrift with sea salt were found on the bottom side of the main girders in 1975 (see Fig. 14 and Fig. 15). Subsequently, deteriorated parts of the girders were removed and repaired by resin. In 1992, this bridge was

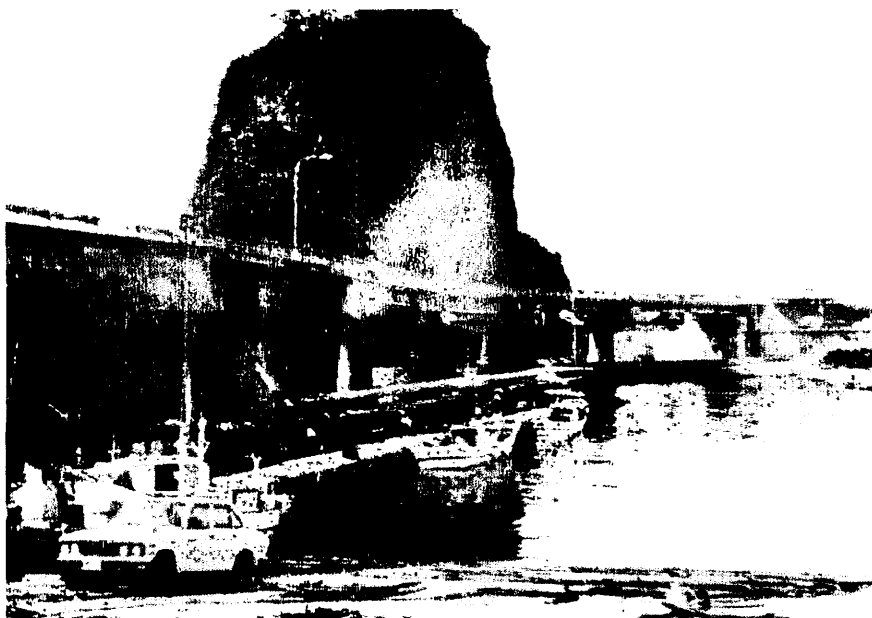


Fig. 13: Kuretsubo Bridge in Japan [1]



Fig. 14: Rust and damage in girders of Kuretsubo Bridge [1]

strengthened by external post-tensioning and additional piers were built between the existing piers to reduce the working stress in the girders (Fig. 16). Fig. 17 shows the breaking of prestressing cables. This old PC bridge was reconstructed recently changing design details such as increase of cover concrete, use of epoxy-coated steel bars and prestressing *tendons* and use of polyethylene sheaths. Learning lessons from terrible deterioration of this particular bridge, many PC bridges were investigated in Japan for contents of chloride ion in concrete, cover of reinforcements and distance of bridge from the nearest coastline.



Fig. 15: Rust and damage in girders of Kuretsubo Bridge [1]



Fig. 16: Construction of additional supports in Kuretsubo Bridge [1]



Fig. 17: Breaking of prestressing tendons in Kuretsubo Bridge [1]

Fig. 18 shows the damage level of PC bridges due to chloride attack with changing years in service of bridges and distance between the locations of bridges and the nearest coastlines. From this figure, it is clear that the number of damaged PC bridges and the level of damage increase when the bridges are located near the coastline. Based on the previous studies [2], the following equations have been proposed and the followings were assumed to calculate the chloride content in concrete.

Water cement ratio: $w/c = 50\%, 43\%, 36\%$ (normal Portland cement)

Design life of PC bridge: 100 years

Initial content of chloride ion: $C_{in} = 0.3 \text{ kg/m}^3$

Limit content of chloride ion for corrosion of steel: $C_{lim} = 1.2 \text{ kg/m}^3$

Equation of diffusion coefficient: $D = (5 \times 10^{-7}) \times e^{-1.6(w/c)}$

Average volume of splashing salt per 1.0 km of bridge is given below for different area A, B and C in $\text{mg/dm}^2/\text{day}$ (mdd), where, $\text{dm}^2 = 100 \text{ cm}^2$

$$\begin{array}{ll} \text{Area A: } C_1 = 0.24 & C_1(+16) = 1.35 \\ \text{Area B: } C_1 = 0.92 & C_1(+16) = 2.51 \\ \text{Area C: } C_1 = 0.11 & C_1(+16) = 0.44 \end{array} \quad (2)$$

Content of chloride ion in the surface of concrete at distance X from coastline is:

$$C_o = 1.2 (C_1 \cdot X^{-0.6})^{0.4} \quad (3)$$

Design content of chloride ion at the position of reinforcement:

$$C_d = (C_o - C_{in}) \left(1 - \text{erf} \left(\frac{0.1c}{2\sqrt{Dt}} \right) \right) \quad (4)$$

The error function erf is given by eq. 5.

$$\text{erf}(s) = \frac{2}{\pi^{1/2}} \int_0^s e^{-\eta^2} d\eta \quad (5)$$

where, σ : standard deviation, C_1 : volume of chloride per km of bridge in mmd, C_d : design content of chloride ion at the position of reinforcement, c : concrete cover, D : diffusion coefficient, t : time, i.e., life of bridge

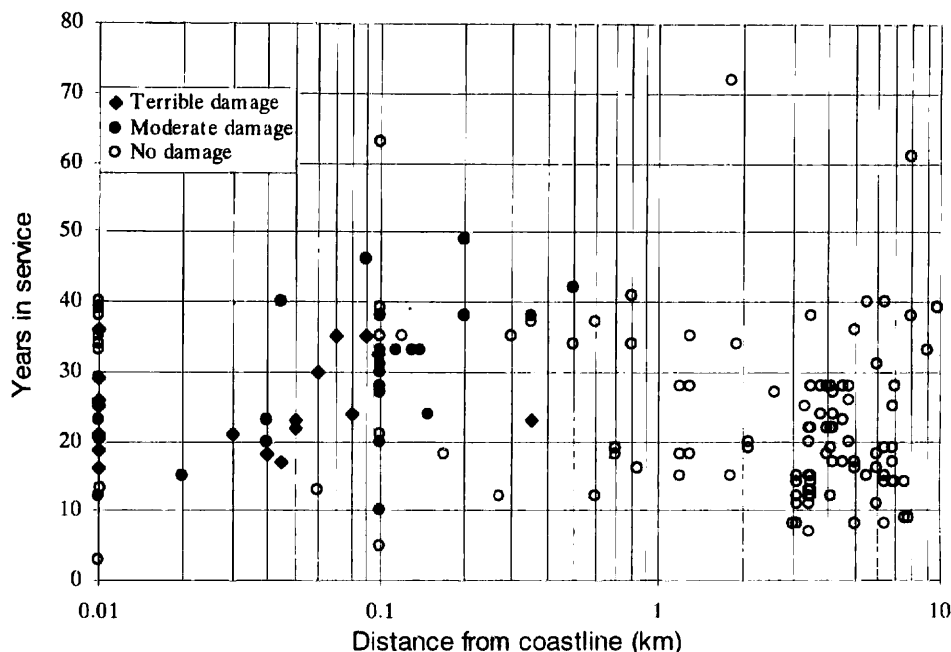


Fig. 18: Damage level of PC bridges with years in service versus distance from coastline

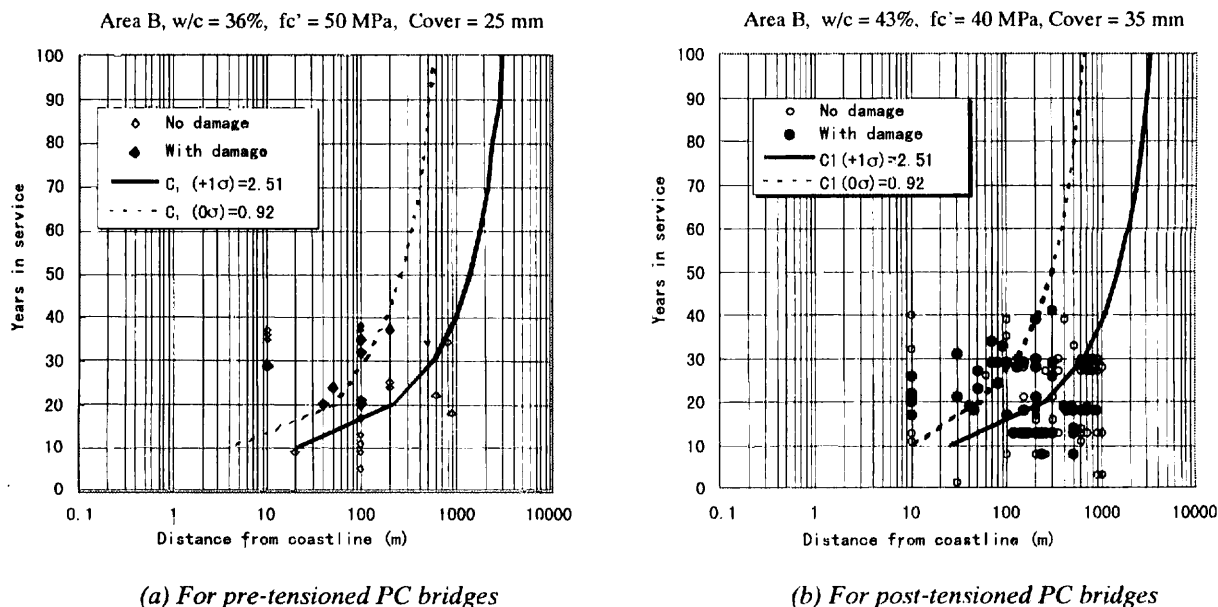


Fig. 19: Relationship between years in service and distance from nearest coastline [4]

Fig. 19 shows the relationship between years in service and the distance from the coastline obtained from analysis and the measured data for area B, which is the severest area in terms of chloride attack. For pre-tensioned PC bridges shown in Fig. 19(a), w/c ratio was 36%, average compressive strength of concrete was 50 MPa and the cover concrete was 25 mm thick. In the case of post-tensioned PC bridges shown in Fig. 19(b), w/c ratio was 43%, average compressive strength of concrete was 40 MPa and the cover concrete was 35 mm thick. The expected line from the analysis agrees approximately well with the measured points. Based on these results, a new design code for highway bridges stipulates required concrete cover as shown in Fig. 20. A stepped value for concrete cover has been proposed depending on the distance of the nearest coastline from the bridge site for pre-tensioned as well as the post-tensioned PC bridges. The figure also shows the present code values, which are relatively lower than the new code values. For pre-tensioned bridges, a maximum value of 5 cm and minimum value of 2.5 cm have been stipulated, whereas a maximum of 7 cm and minimum of 3.5 cm have been suggested for post-tensioned PC bridges.

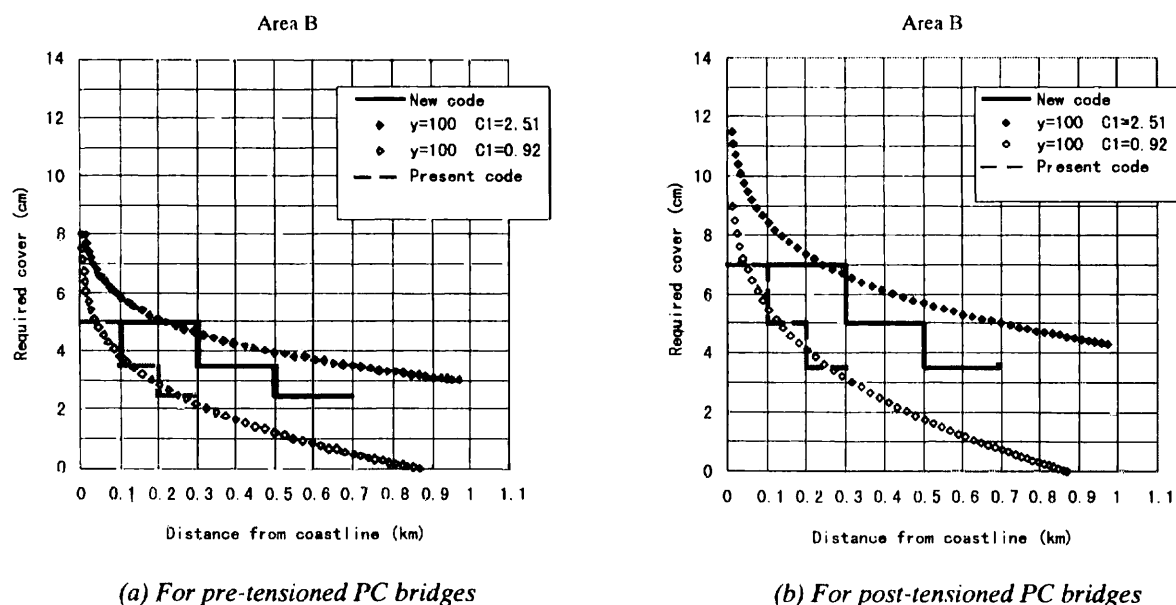


Fig. 20: Relationship between required cover concrete and distance from nearest coastline [4]

4 Conclusions

Present situation of PC bridges in Japan were presented in terms of their durability as influenced by the quality of grout in sheaths and chloride attack. Different degrees of grouting were found, such as perfect condition, grouts with some voids, imperfect grout and complete loss of grout. A number of bridges have been found severely deteriorated due to loss of grout and imperfect grout conditions. Learning from such bridges, the Japan Highway Corporation has forbidden the construction of new post-tensioned PC structures with internal grouting. To improve the durability of new PC bridges, use of external cables with transparent sheaths may be one solution. Moreover, use of no-bleeding type of grout and pre-grouted tendons are also recommended.

Many PC bridges located in coastal areas were found to be deteriorated due to chloride attack. The extent of deterioration is inversely proportional to the distance from the nearest

coastline. The deterioration due to chloride attack can be reduced considerably by increasing the concrete cover, using epoxy-coated bars and prestressing tendons, and using polyethylene sheaths. Based on the investigation results, the new code for highway concrete bridges in Japan stipulates thicker concrete cover for both pre-tensioning and post-tensioning types of PC bridges than the present code.

References

- (1) Investigation on Life Cycle Cost of Concrete Bridges- Investigation of Deterioration and Maintenance Cost of Concrete Bridges, *Technical Memorandum of Public Works Research Institute, No. 3811*, 2001.
- (2) Standard Specification for Design and Construction of Concrete Structures- Construction, *Japan Society of Civil Engineers*, 1999.
- (3) Specifications for Highway Bridges- Part 3, Concrete Bridges, *Japan Road Association*, 1996.
- (4) Specifications for Highway Bridges- Part 3, Concrete Bridges, *Japan Road Association*, 2001(expected).
- (5) Manual for Maintenance of Durable Prestressed Concrete Bridges, *Japan Prestressed Concrete Engineering Association*, 2000.

Corrosion induced failure mechanisms

U. NÜRNBERGER

Professor, Otto-Graf-Institut, Universität Stuttgart, Germany

1 Introduction

In prestressed concrete the purpose of prestressing lays in exerting pressure on the low tensile strength concrete in that areas, where the concrete normally is exposed to tensile stresses and threatened by cracking and failure. Therefore in prestressed concrete structures the high strength prestressing steel performs essential bearing action.

In post-tensioned concrete members high strength steel wires, strands or bars are arranged in ducts. After casting and hardening of concrete the prestressing reinforcement is tensioned and compressive stresses are generated in concrete. After that ducts are grouted with cement mortar, in order to protect the steel against corrosion and to guarantee a permanent load capacity.

During the past 40 years in Germany some serious damages in post-tensioned and pre-tensioned components have been occurred because of stress corrosion cracking of the prestressing steel [1-3]. These failures happened due to onsite conditions and were favoured of the sensitiveness of the used prestressing steel.

Stress corrosion cracking and the subsequent failure of steel and construction may occur

- if the protection is not guaranteed from the beginning as a result of poor workmanship,
- or it is lost because of deterioration of the construction in the course of the time,
- or the prestressing reinforcement is predamaged during handling.

Also an application of unsuitable materials for prestressing steel, injection mortar or concrete can alone or in combination with other factors favour SCC.

2 The basis of stress corrosion cracking

Fractures of prestressing steel as a rule can be referred to hydrogen induced stress corrosion cracking (H-SCC) [4,5]: It may happen during erection of the construction or during the later use. The following conditions are necessary for H-SCC:

- a sensitive material or state,
- a sufficient tension load,
- at least a slight corrosion attack.

During the corrosion process hydrogen atoms have to be set free and get absorbed by the steel. In sensitive steels the hydrogen under the effect of mechanical stresses can create pre-cracks in critical structural areas such as grain boundaries. These cracks may grow and result in material fracture.

Special conditions have to exist to activate the formation of adsorbable hydrogen atoms. To understand the correlations between procedure on site and development of damage, the chemical reactions of corrosion should be considered (Table 1).

anodic iron dissolution ① $\text{Fe} \rightarrow \text{Fe}^{2+} + 2\text{e}^-$
cathodic reactions ② $\frac{1}{2} \text{O}_2 + \text{H}_2\text{O} + 2\text{e}^- \rightarrow 2 \text{OH}^-$
if pH < 7 ②: ③ $\text{H}^+ + \text{e}^- \rightarrow \text{H}_{\text{ad}}$ (hydrogen discharge)
rivalry reaction with regard to ③ ④ $2\text{H}_{\text{ad}} \rightarrow \text{H}_2$ (recombination) is prevented in the presence of promoters
⑤ $2\text{H}_{\text{ad}} + \frac{1}{2} \text{O}_2 \rightarrow \text{H}_2\text{O}$ if oxygen is present or air access

Table 1: Chemical reactions of corrosion

Harmful hydrogen can arise only

- if the steel surface is in the active state or depassivated (this is expressed by reaction 1),
- if the cathodic reaction of corrosion is discharging hydrogen (this is described by reaction 3),
- if the adsorbable atomic hydrogen is not changed into the molecular state (see reaction 4).

Therefore at the surface of corroding steel amount of adsorbable hydrogen atoms rises

- with increasing hydrogen concentration (reaction 3 is accelerated),
- in the presence of so-called promoters (reaction 4 is hindered).

From the practical point of view one can say that hydrogen induced damages are only possible

- in acid mediums
- or in the presence of promoters such as sulphides, thiocyanates and compounds of arsenic or selenium.

In concrete structures the attacking medium is mostly alkaline and acid solutions are limited to exceptions. Nevertheless, in natural environments the pitting induced H-SCC can take place (Fig. 1). Pitting induced H-SCC means crack initiation within a corrosion pit. In the corrosion pits the pH-value falls down because of hydrolysis of the Fe^{2+} -ions. Pittings or spots of local corrosions can be explained by differential aeration or concentration cells. Especially effective is the attack of condensation water or salt enriched watery solutions when erecting the constructions. In prestressed construction carbonation of concrete and mortar as well as chloride contamination are responsible of local corrosion attack.

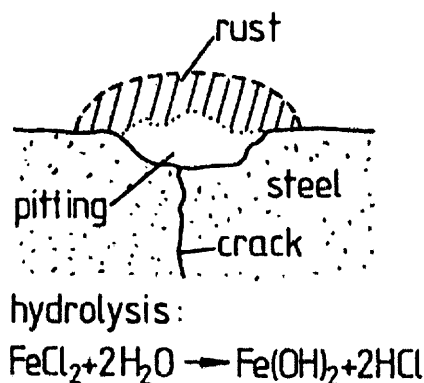
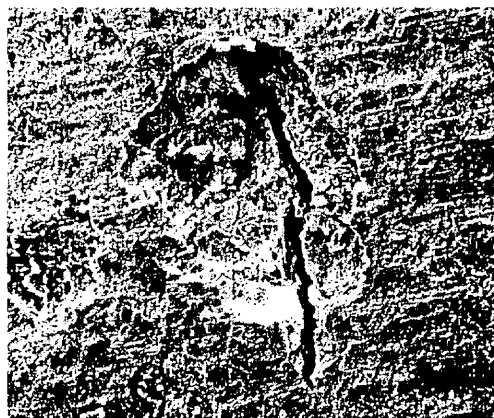


Fig. 1: Pitting induced stress corrosion cracking

3 Prestressing steel

In case of sensitive prestressing steel already minimal contents of hydrogen can lead to irreversible damages. Therefore the steel quality and the susceptibility to hydrogen of the applied steel melt is of enormous importance [6,7]. In Table 2 one can find a survey of the different steel types. World wide dominates the application of cold deformed wires and strands. They yearly production of cold deformed material is 1 million tons. The strength of commonly used cold deformed prestressing material amounts with falling diameter of the wire from 1570 up to 2060 N/mm². In the case of strands the upper strength limit is higher.

type	shape, surface	diameter	anchorage system	strength class European Standard	production (world wide) tons/year
cold deformed ● wire	round - smooth	4 - 12,2 mm	wedge or button heads	1570 - 1860 ¹⁾ (N/mm ²)	1.000.000 (world-wide)
	round - profiled	5 - 5,5 mm			
	round - smooth (7 wires)	9,3 - 15,3 mm		1700 - 2060 ¹⁾ (N/mm ²)	
hot rolled ● bar	round - smooth	26 - 36 mm	thread (ends)	1030 - 1230 (N/mm ²)	50.000 (Germany, UK)
	round - ribbed	26,5 - 36 mm	thread (full length)		
quenched and tempered ● wire	round - smooth	6 - 14 mm	wedge	1570 (N/mm ²)	5.000 (Germany, Japan)
	round - ribbed	5 - 14 mm			
	oval - ribbed	40 - 120 mm ²			

¹⁾ in Germany max. 1770 N/mm²

Table 2: Survey about produced prestressing steel

The larger diameter hot rolled bars with 50.000 tons-a-year production have a considerably lower strength from 1030 to 1230 N/mm². The production of quenched and tempered steel wires is with yearly 5.000 tons significantly lower. Their strength corresponds with those of the larger diameter deformed wires. Hot rolled and quenched and tempered steel are produced only in few countries.

Table 3 summarises the advantages and application of the different steel types. Cold deformed steel may be applied for all types of prestressing and in all types of members. This material is economical to produce and relatively high strength can be reached. The strand has an advantage of easy transport and storage, flexibility, easy installation and good bond behaviour. Nowadays the hot rolled bars and quenched and tempered steel have their special areas of application. Bars, which are tension members with high load-bearing capacity and easy handling, are given the preference if transverse prestressing and earth anchors are required. Ribbed quenched and tempered wire with oval cross section and very good bond behaviour are needed for prefabricated elements and sleeper for the railway.

type	especial advantages	application
cold deformed ● wire	• economically to produce • high strength • low coil diameter, high coil weight (strand) • flexible tension members (strand) • easy to install (strand) • good bond behaviour (strand)	for all types of prestressing and all types of elements
● strand		
hot rolled ● bar	• tension members with high load • simple to anchor • easy handling • effective bond of ribs	transverse prestressing, earth anchors
quenched and tempered ● wire	• very good bond behaviour	prefabricated elements, sleeper (railway)

Table 3: Advantages and application of prestressing steel

4 Reason of damages

In connection with special circumstances all types of prestressing steels may suffer SCC. Making a diagnosis of defective building can help to find solutions to avoid future problems. Reasons of damages of prestressed concrete structures can be classified as [3]:

- insufficient design,
- incorrect handling of building materials,
- incorrect execution of planned design,
- unsuitable mineral building materials,
- unsuitable (sensitive) prestressing steel.

The reason 1 to 4 are responsible for lack or time dependent loss of passivation and a promotion of SCC.

Damages as the consequence of these influences should not be the main topic of this contribution. However, especially those problems caused a great stir throughout the world of prestressed concrete, where the attendant circumstances of design, execution and building materials were not unusual. But the prestressing steel was unsuit to resist the inevitable conditions on construction site and inside the construction. During the last 10 years in Germany some serious damages in pre-tensioned and post-tensioned components have been occurred, which were strongly favoured by the sensitivity of the used prestressing steel. In the following two cases will be discussed and we will end up with conclusions respectively consequences.

Some cases steel fractures in the yet ungrouted ducts of post tensioned structures as well as serious collapses of building components can be attributed to the presence of aggressive water in the ducts which results from bleeding [6,8]. Bleeding is a separation of fresh concrete, where the solid content sink down and the displaced water rises or penetrates in inner hollows. In the bleeding water significantly high contents of sulphates and increased quantities of chlorides may be accumulated (Table 4) by leaching of the construction materials cement, aggregates and water. The high amounts of potassium-sulphate result from the gypsum in the cement. The watery phase of fresh concrete penetrates into the ducts through anchorages, couplings and defects in the sheet and accumulates at the deepest points. Already in the not grouted and not prestressed condition the steel may suffer from strong pitting. If the steel is sensitive to hydrogen pitting induced stress corrosion cracking takes place (Fig. 2).

sulphate	1,90 - 5,20	mg/l
chloride	0,13 - 0,18	mg/l
calcium	0,06 - 0,09	mg/l
sodium	0,18 - 0,37	mg/l
potassium	3,60 - 7,30	mg/l
pH-value	10 - 13	

Table 4: Analysis of bleeding water

In some cases construction elements of buildings failed after more than 25 years as a consequence of SCC [2,3,5,6,8]. For example the beams of the roof of a laboratory had been affected causing the roof collapse (Fig. 3). In an other case the beams over a factory building were affected. It was found, that in both cases the used quenched and tempered steel was very sensitive to hydrogen induced cracking. Investigations showed, that in the time of erecting the building the steel had suffered precorrosion and precracking probably owing to contact with normal influences such as condensing water and bleeding water. A confirmation of this hypothesis gives this X-ray pattern photograph with many cracks in the steel (Fig. 4). And it was concluded from the research, that the precracks, initiated in a very early stage of the building, could grow discontinuously in the grouted state over a very long period of time.

Fig. 5 illustrates the typical intercrystalline fracture of the steel wires with crack propagation pattern on the grain faces. Such a mechanism evidently can occur only on the condition, that the prestressing steel has a very high sensitivity to SCC, that means it react to very small amounts of hydrogen.

The wires of the bundle were broken in one section. The ducts were grouted completely and fractures have occurred within an alkaline and chloride free mortar.

The problem of very high sensitivity of prestressing steel is not restricted to special steel types. A cold deformed wire was involved in another case. Numerous circularly wrapped and prestressed concrete pressure water pipes in the ground exploded after some years in service (Fig. 6). As a consequence an extensive expanse has been flooded. During service life concrete cover had suffered loss of bond and microcracking by sulphate action. Responsible for this effect was the production technology of the pipes, namely the accelerated hydration by heat curing [5]. As a consequence the concrete around the steel, that means the concrete in the contact zone steel / concrete, was carbonated.

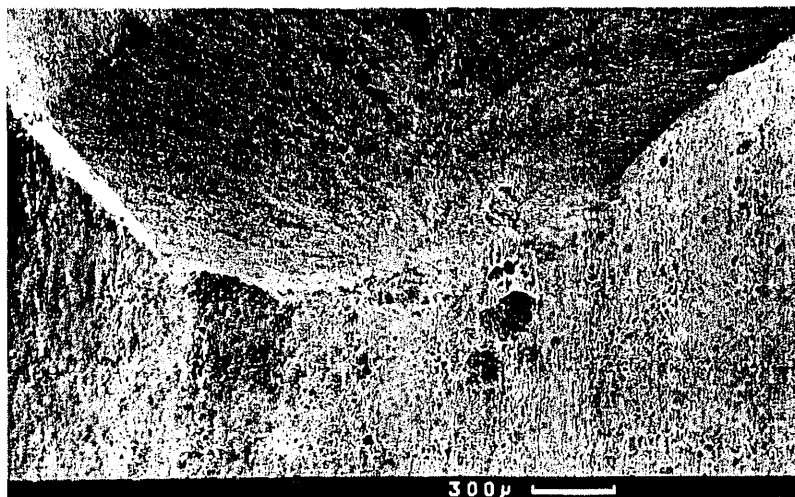


Fig. 2: Fracture starting at a corrosion pit



Fig. 3: Broken beam as a result of stress corrosion cracking

In this case the prestressing steel proved to be extremely sensitive to hydrogen influence. The strength of the steel was higher than 2000 N/mm² and as a result of radial shear stresses firstly longitudinal and later also transverse cracking took place (Fig. 7).

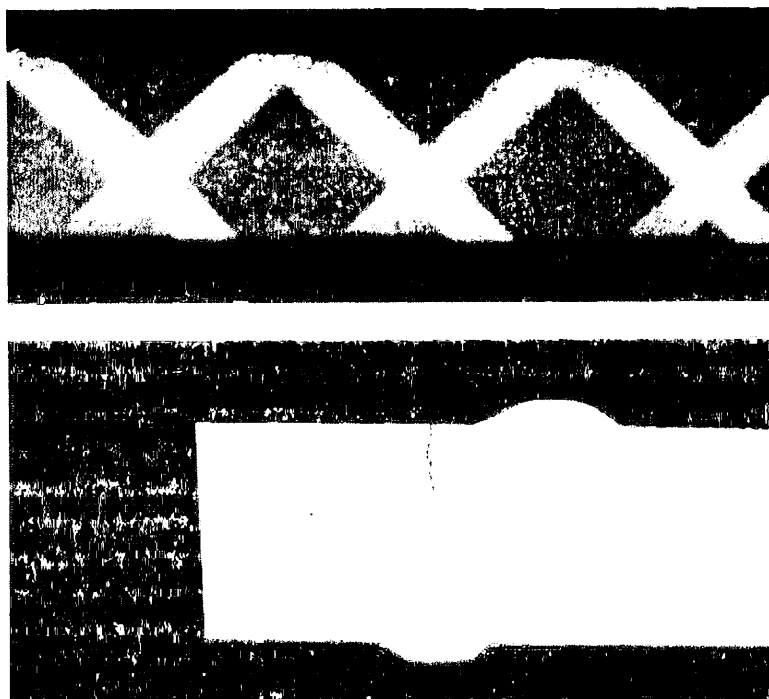


Fig. 4: X-ray pattern photograph of a precracked steel, below: cut of the wire

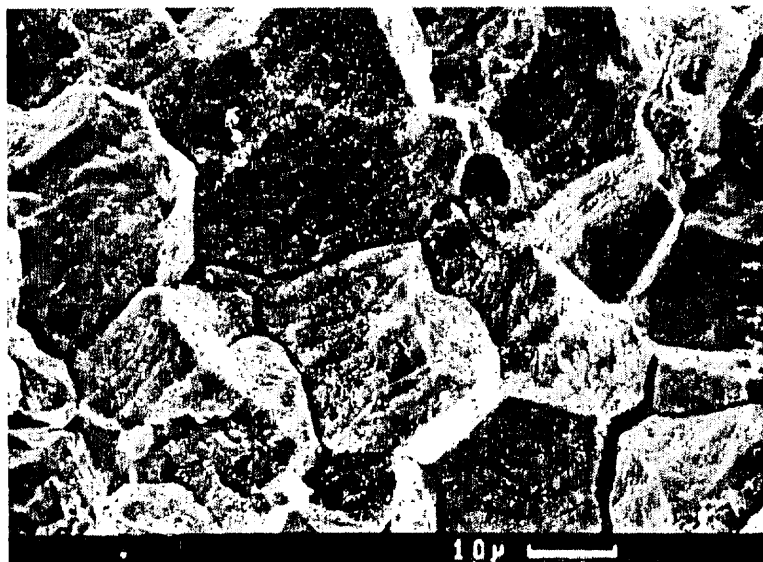


Fig. 5: Intercrystalline fracture with crack propagation pattern on the grain faces

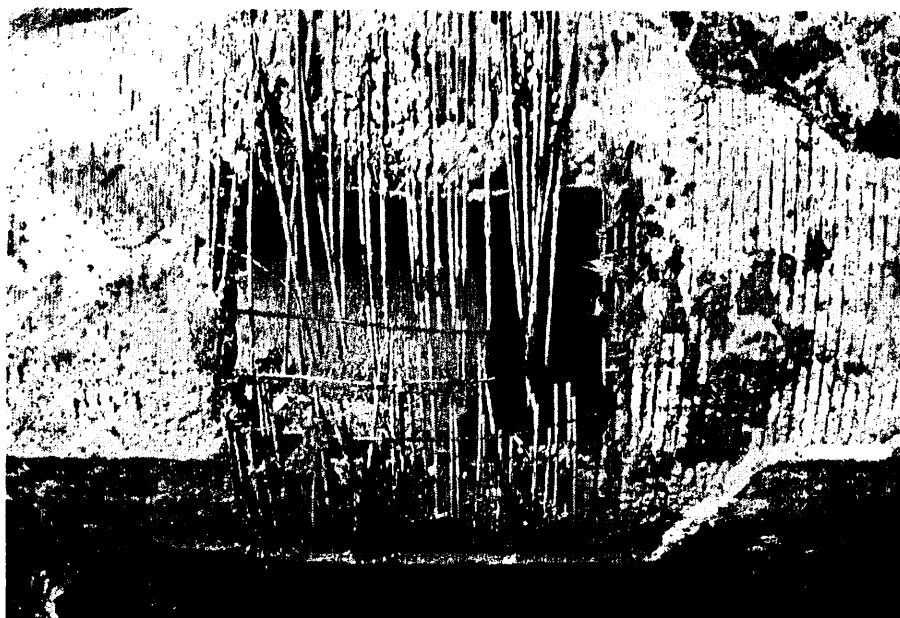


Fig. 6: Stress corrosion cracking of a circularly wrapped pipe



Fig. 7: Cracking of the cold deformed wires

5 Consequences

Table 5 summarises the most important measures introduced in Germany after some periods with increased number of damages related to SCC.

Corrections as a result of insufficient design are the concern of design engineer and of constructional experts. They include improved standards and recommendations for planning and execution of new constructions and for strengthening of older ones. Focus of the last years were measures during erection the building related to

- investigation of concrete technologies, which favour bleeding of fresh concrete. In this connection it was found, that addition of additives to fresh concrete such as retarder and liquefier may increase bleeding [6].
- In other cases it was recommended to flush out the salt rich bleeding water after concreting [4].
- A far reaching decision was the limitation of time between prestressing of steel and injection of ducts in German standard DIN 4227.

All measures had the aim to hinder corrosion and cracking before protecting reinforcement with injection mortar.

reason of damage	consequences
insufficient design	• strengthening of construction • improved standards and recommendations for planning and execution of work • inspection of other constructions of the same kind • demolition of elements or construction
incorrect execution of planned design and incorrect handling of building materials	• see above • training and careful education of personnel • supplementary reinforcement to avoid unannounced failure
unsuitable mineral building materials	• limitation of chlorides in water, cement and aggregates • prohibition of cements, additives and accelerators which favour corrosion and hydrogen evolution
unsuitable (sensitive) prestressing steel	• prohibition of qualities, which suffer SCC under on-site conditions • development of steel types which are more resistant to hydrogen • limitation of maximum strength • long-time SCC-test (2000 h) under practical conditions

Table 5: Reasons of damages of prestressed concrete structures and consequences

Much research was done to develop nondestructive measuring techniques to investigate older prestressed constructions. There exist numerous buildings where formerly produced prestressing steel was used. Nowadays we know because of other damages, that some of the older steel types are very sensitive to H-SCC. Therefore a magnetic methods was developed to assess cracks and fractures before a possible catastrophic collapse [9,10]. The prestressing steel is magnetised straight through the concrete cover by a strong magnet. Deep cracks and fractures are to detect by an alteration of the flow of the magnetic lines (Fig. 8). To examine the magnetic flow, characteristically changed by cracks and fractures, a very sensitive Hall-

analyser is used. It is carried on the concrete surface along the tension members. If a multitude of broken steel wires are detected and the further bearing capacity is not sure a demolition of the construction may be recommended.

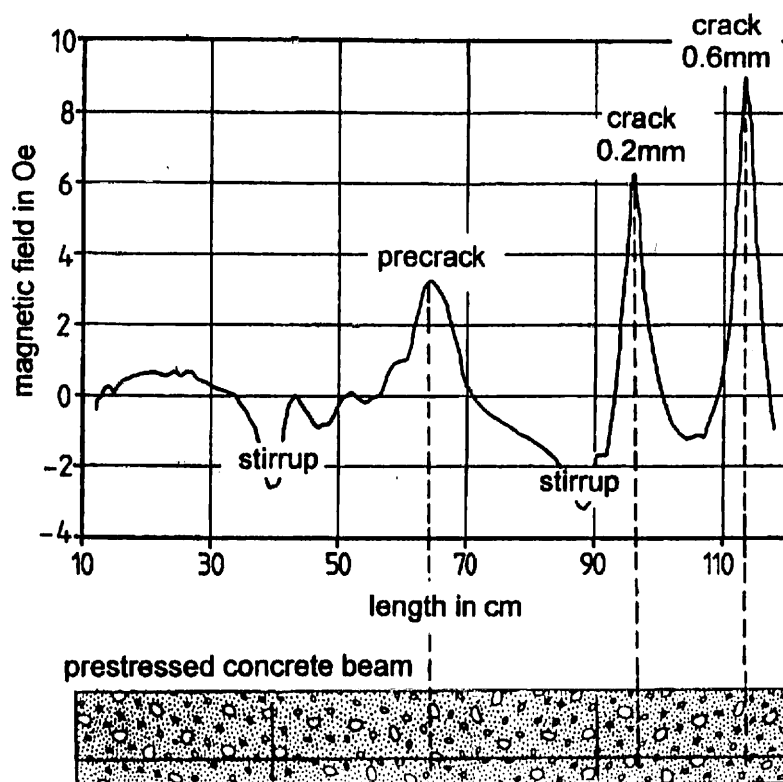


Fig. 8: Magnetic field of a cracked wire embedded in concrete (Sawade)

Further incorrect execution of the planned design and incorrect handling of the building materials may be found. For instance the concrete cover is too low and of no adequate quality or the ducts are not injected well. In such case the improper dealing with the prestressed concrete is responsible for problems.

In Germany the application of a supplementary reinforcement was under discussion [3]. In the case of loss of capacity a failure without warning should be avoided. This measure was very controversial discussed because it represents a turning away from the principle of prestressed concrete and induces economic disadvantage for the German construction industry.

With regard to building materials in the past strict instructions led to an essential reduction of failures [11]. Already 30 years ago the content of chlorides in water, cement and aggregates was limited to a very low level. Cements which not guarantee durable protection of the embedded steel had been forbidden, and a steady control prevents, that unwelcome materials can come on the market. That concerns all types of cements and additives, which are continuously investigated electrochemically with regard to corrosion and hydrogen evolution.

But the most radical change took place on the field of prestressing steel. In Germany a great variety of steel types is in use (section 3). New types find their way into the prestressing technique, other types are excluded because they not proved to be reliable. That means they suffered SCC under onsite conditions [1 - 3]. As well as hot rolled bars, the quenched and tempered and the cold deformed steel had been improved in the last 40 years.

Concerning the failures with quenched and tempered steel a so called old type of steel was involved [12]. In the beginning these material was carbon-silicon-manganese alloyed (Table 6). Because of a non sufficient stress corrosion cracking behaviour, already an attack of condensating water resulted in hydrogen induced problems, the chemical composition was changed in 1965 [13]. The carbon content was lowered and chromium was alloyed. Because of this precaution the full quenching and subsequent tempering was improved and the retained martensite and detrimental residual stresses were reduced. Further the manganese content was decreased and silicon content was increased. As a consequence absorption, solubility and diffusivity of hydrogen was considerably diminished [14].

	old type	new type
carbon	0.65	0.48
silicon	1.19	1.80
manganese	0.88	0.62
phosphorus	0.014	0.012
sulphur	0.020	0.014
chromium	0.04	0.46

Table 6: Analysis of quenched an tempered prestressing steel

As mentioned at the beginning the old type quenched and tempered steel was involved in numerous and serious damages. But nearly no failures happened after application of the new steel type.

In a similar action the sensitive hot rolled bar with bainitic structure 1985 was replaced by a new type pearlitic steel [5].

Based on experiences with damages and on laboratory testing we know, that the susceptibility to H-SCC increases greatly with increasing strength [5,6]. An interpretation of numerous SCC-tests conducted according to the FIP-standard showed, that with an increase of the strength of cold deformed steel from 1700 to 2000 N/mm² the service life drops by a factor of 100. Therefore the upper strength is limited in Germany to about

- 1400 N/mm² for hot rolled steel,
- 1700 N/mm² for quenched and tempered steel,
- 1950 N/mm² for cold deformed steel

and there is no tendency to release this requirement. In the European standard EN 10138 an upward extension of the strength of cold deformed wire and strand was introduced. In the case of strands the strength range was extended up to 2060 N/mm². Essential German objections to this European standard [15] are related to the extension of the strength limit for cold deformed material because a such high strength steel seems not to be sufficiently safe under practical conditions. In Germany such very high strength prestressing wires and strands will not be applied in future.

At long last also the development and application of an improved corrosion testing in Germany is a consequence of damages caused by using unsuitable prestressing steel [4,16,17]. In most cases SCC-tests are carried out according to the so called FIP-standard [18] in a highly concentrated thiocyanate solution. The result of the FIP-testing is a brittle fracture after hydrogen charging and general embrittlement of the whole cross section. The newly used test solution is adopted to practical mediums over a testing time up to 2000 hours. The advantage of this standard testing procedure is, that the mechanism of cracking agrees with that observed

in the practice. I is a pitting induced SCC where the crack initiation is connected with corrosion processes on the steel surface. In Table 7 the conditions of the FIP- and of the German long time test are compared. Besides a mixture of neutral salts the new solution contains also 0,5 g/l thiocyanate as a promotor in order to provide hydrogen in a amount which can occur in defective concrete constructions. The new test is applied in steel production quality control and also in examining causes of damages. The main observation is all steels or steel melts that caused difficulties on practice failed in standard test within 2000 hours. This observation is valid for cold deformed material, quenched and tempered wire and hot rolled bars.

In Fig. 9 the results of tests with cold deformed wires in comparison with those of the FIP-test are shown. The strength of the steel increases from left to right. In the FIP-test lifetime decreases steadily with increasing strength. In the long time test only steel with a very high strength failed. We may have stress corrosion fractures within 2000 hours, if the strength exceeds about 2000 N/mm². Similar results on quenched and tempered steels are published in [7].

standard	concentration	temperature	stress	lifetime (request)
FIP	20 mass.-% NH ₄ SCN	50°C	0,8 R _m	hot-rolled > 30-50 h quench., temp. > 10-15 h cold-deformed > 2-3-h
DIBT	0,5 g/l Cl ⁻ 5 g/l SO ₄ ²⁻ 1 g/l SCN ⁻	50°C	0,8 R _m	> 2000h

Table 7: Parameters and criteria of SCC-test of prestressing steels

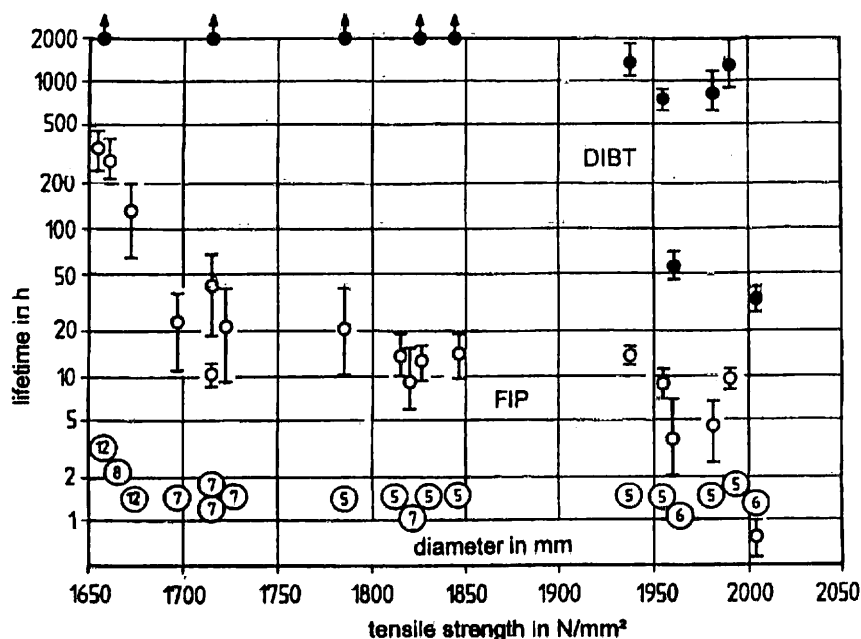


Fig. 9: Results of stress corrosion tests of cold deformed wires after FIP- and (German) DIBT-guidelines

REFERENCES

- [1] U.Nürnberger: Analyse und Auswertung von Schadensfällen bei Spannstählen. Forschung, Straßenbau und Straßenverkehrstechnik 308 (1980) 1 - 195
- [2] J. Mietz, U. Nürnberger, W. Beul: "Untersuchungen an Verkehrsbauten aus Spannbeton zur Abschätzung des Gefährdungspotentials infolge Spannungsrißkorrosion der Spannstähle", Abschlußbericht des BMV Forschungsvorhabens FE 15.209 R91D, BAM Berlin und FMFA Stuttgart 1994
- [3] U. Nürnberger: Studie zu Spannstahlbrüchen. Forschungsauftrag DBV 210 i. A. des Deutschen Betonvereins, Wiesbaden, FMFA BW Stuttgart 1998
- [4] D. Grimme, B. Isecke, U. Nürnberger, E. M. Riecke, G. Uhlig: Spannungsrißkorrosion in Spannbetonbauwerken. Verlag Stahleisen mbH, Düsseldorf 1983
- [5] U. Nürnberger: Korrosion und Korrosionsschutz im Bauwesen. Bauverlag, Wiesbaden 1995
- [6] U. Nürnberger: Einflüsse von Werkstoff und Verarbeitung auf die Spannungsrißkorrosion von Spannstählen. Werkstoffe und Korrosion 48 (1997) 602 - 612
Siehe auch: Vorträge der DBV-Arbeitstagung "Forschung" am 07.11.1996 in Wiesbaden. Deutscher Beton-Verein e. V.
- [7] J. Mietz, K. Pasewald, B. Isecke: Untersuchungen zum wasserstoffinduzierten Sprödbruch vergüteter Spannstähle. Abschlußbericht zum DIBt-Forschungsvorhaben AZ: IV 1-5-651 (1998), s. a. B. Isecke, J. Mietz: 33. DAfStb-Forschungskolloquium, BAM Berlin, 10. und 11.10.1996, Vortragsbeiträge S. 135 - 139
- [8] U. Nürnberger: Special corrosion problems in post-tensioned structures. Otto Graf Journal 3 (1992) 148 - 161
- [9] G. Sawade, K. Menzel: Zerstörungsfreie Feststellung von Spannstahlrissen in vorgespannten Deckenträgern. Werkstoff und Korrosion 40 (1989) 451 - 456
- [10] B. Hillemeier, C. Flohrer, A. Schaab: Die zerstörungsfreie Ortung von Spannstahlbrüchen in Spannbeton-Deckenträgern. Beton und Stahlbetonbau 84 (1989) 268 - 270
- [11] U. Nürnberger, B. Neubert: Entwicklung und Bewertung des Regelwerkes im Spannbetonbau aus korrosionstechnischer Sicht, S. 111 - 124; in "Fortschritte im Konstruktiven Ingenieurbau", Verlag Ernst & Sohn, Berlin 1984
- [12] Technische Mitteilungen Hüttenwerk Krupp - Rheinhausen, 1953
- [13] Fried. Krupp Hüttenwerke AG, Werk Rheinhausen, Beiträge zum FIP-Symposium "Spannstähle", 06. und 07.06.1968, Madrid
- [14] K. Schneider: Einfluß des Silizium- und Mangangehaltes im ölschlußvergüteten Spannstahl St 140/160 auf die Beständigkeit gegenüber Wasserstoffrißkorrosion. Neue Hütte 16 (1971) 529 - 535
- [15] P. Schießl: Europäische Normen für Betonstahl und Spannstahl und europäische Regelungen für Spannverfahren. Betonwerk + Fertigteiltechnik (1993) 64 - 72
- [16] G. Rehm, U. Nürnberger: Neue Methoden zur Beurteilung des Spannungsrißkorrosionsverhaltens von Spannstählen. Betonwerk + Fertigteil-Technik 48 (1982) 287 - 294
- [17] B. Isecke: Kritische Beurteilung neuer Spannstahlentwicklungen. Forschungsbericht i. A. des Deutschen Betonvereins e. V., Berlin 1998
- [18] Fédération Internationale de la Précontrainte: Report on prestressing steel. Stress corrosion cracking resistance test for prestressing tendons, 1980

Non-destructive Evaluation and Monitoring of Post-tensioning Tendons

P. MATT

Civil Engineer, Engineering-Consulting, Ittigen, Switzerland

1. Introduction

Post-tensioning tendons are structural elements essential for the safety, serviceability and durability of prestressed concrete structures. Consequently it would be desirable to reliably assess their behaviour in existing structures. Such checks to detect possible defects or damages such as grout voids or tendon corrosion should preferably be done by non-destructive or at least low-destructive inspection methods and with minimum disturbance to the user.

In a report published in 1988, the author analysed the then available inspection methods and assessed their usefulness and potential [1]. The conclusion was that these methods provide meaningful results limited only to localized areas, if at all. In the meantime, some of these methods were developed further and new ones have appeared.

The aim of this paper is to describe the actual situation with regard to the evaluation and monitoring of post-tensioning tendons from the author's point of view. Furthermore, its content focuses on internal, bonded tendons using prestressing steel such as strands, wires or bars as their tensile members. Up to now, this type of tendon represents the bulk of all applications even if in a number of countries external tendons have become increasingly popular.

2. Inspection and monitoring methods

The non-destructive or low-destructive inspection and monitoring methods discussed below have either the aim to detect existing grout voids, corrosion of the prestressing steel in progress or even ruptured wires, strands or bars in tendons:

2.1 Georadar and covermeter

Experience with practical applications have shown that georadar is only suitable for the location of tendons. This is however often a prerequisite for a detailed tendon inspection. Whereas under favourable condition (no congestion of reinforcement) georadar allows the location of tendons to a depth of up to 300mm, even a powerful covermeter is generally not capable to detect ducts at concrete covers of more than 40 to 50mm and again only if light reinforcement is present [2].

2.2 Potential mapping

Whereas potential mapping (measuring the potential field) is a powerful tool for finding corroded normal reinforcement, in case of tendons it is only successful under very favourable conditions (e.g. small concrete cover to the ducts and light normal reinforcement as they may exist in thin webs of precast beams).

2.3 Impact-Echo Method

Since 1983, the Impact-Echo Method has been under development primarily in the United States. It is stated that it can also be used for detecting grout voids in tendons [3].

In [4], the method was verified in this respect and the findings can be summarized as follows:

"It is possible to use the Impact-Echo Method for checking a tendon for grout voids. It is however a delicate operation requiring experienced personnel. The presence of cracks and other concrete defects as often found in real structures significantly influence the test results and can make the evaluation impossible."

Applications in the United States have shown that under favourable conditions and in accessible areas, the Impact-Echo Method is able to identify grout voids.

2.4 Remanent Magnetism Method

The Remanent Magnetism Method was developed in Germany for detecting fractures in prestressing steel [5]. The magnetizing and recording equipment has to be moved along the tendon path on auxiliary guidance rails and scaffolding fixed to the concrete surface. Thus it allows to localize fractures in the accessible areas. The difficulty is to cope with the disturbing magnetic signals originating from other embedded steel elements such as normal reinforcement, anchorage elements, duct couplers, steel plates, nails etc.

2.5 Radiography

The application of radiography is today limited to special cases. Even in France, where the method had formerly been widely used, it has practically disappeared. Apart from the high cost, another important reason is that most countries have national regulations for the protection of people, animals and the environment when applying radiography. Whereas some of these regulations impose total evacuation of human beings in the neighbourhood (min. distances depend on the intensity of the source; this generally means that all traffic has to be stopped in the area concerned), others ask for traffic suspension only when it cannot flow continuously.

2.6 Reflectometrical Impulse Measurement

Since about 1985, it was tried to use the Time Domain Reflectometry known from applications to coaxial telecommunication cables also for grouted tendons under the acronym RIMT (Reflectometrical Impulse Measurement). The method consists of sending high frequency impulses from an exposed anchorage through the tendon. By evaluating the recorded reflections it was hoped to detect anomalies along the tendon path. In [6], the results of research work done at the Institute of Technology in Zurich is reported. The aim of the project was to understand the fundamentals when applying RIMT to a prestressed concrete structure. The conclusion was that *"the recorded signals do not contain information regarding the condition of the tendon but are artefacts of the measurement procedure. Thus RIMT can definitely not be used as a diagnostic technique for grouted tendons."*

2.7 Ultrasonic Methods

Test have shown that ultrasonic methods (transmission, reflection) for grouted tendons have very limited possibilities. Ultrasonic waves sent from a transmitter sitting on the end of the prestressing steel can detect anomalies only in special cases (e.g. only for smooth bars or wires) and only within a few meters from the tendon anchorage [7].

2.8 Acoustic Monitoring

To detect failures of prestressing steel by acoustic monitoring has been known for many years in fatigue testing of tendons and stay cables. Therefore, acoustic monitoring can also be successfully applied in practice in equivalent situations such as for unbonded tendons and stay cables. Recently, trials have been carried out in Great Britain to assess whether the method can also be used for internal, bonded tendons. It is reported that these trials have been successful [8]. It could be shown that a single wire fracture can be detected above the ambient noise level, distinguished from other acoustic events and even located in position. It is too early to say to what extent and in which situations acoustic monitoring will find its place in practical application. It can however be expected that the method will be restricted to special cases.

It should finally be mentioned that in the technical literature further methods such as **thermography** (infrared-scanner) and **tomography** are described.

In summarizing, it must unfortunately be concluded that none of the inspection and monitoring methods listed in this chapter fulfils the expectations for a complete and meaningful evaluation of post-tensioning tendons in existing structures. Some of them provide at least partial results in accessible areas. It is a fact, that our structures with their traditional bonded tendons have not been designed and constructed for easy inspection.

3. The Engineer's approach

The evaluation of the suitability and limitations of the methods discussed in chapter 2 shows that none of them allow a full assessment of the conditions of a grouted post-tensioning tendon. Some of them however, permit at least a partial evaluation in ideal structural situations.

The one method however which is quite basic and practical, and overall still rates best in terms of information and interpretation, is the careful opening of tendon ducts by drilling into them, and **subsequent visual inspection with an endoscope**. Alternatively the concrete can be carefully removed by chipping thus creating an opening over the length required for visual inspection and possible renewal work. This method can be applied in particular if there is a reasonable doubt about the condition of a tendon at a particular location based on desk studies, on a thorough visual inspection and maybe based on additional non-destructive test methods. Although this method is not non-destructive, the extent of intrusion is quite moderate, and not considered harmful to the structure or tendon, in general.

In [9] and [10] the authors conclude that the inspection engineer when assessing an existing structure should be aware of the possible hazard scenarios for post-tensioning tendons. Figure 1 shows potentially "weak points" in the case of a typical box girder bridge.

The local opening allows the determination of possible defects and deteriorations of a tendon. If large grout voids are found their volume can be measured using the **vacuum technique**. In case of corroded metallic ducts and corroded prestressing steel its cause and possible contamination can be determined by taking samples from the surrounding concrete, the grout as well as from corrosion products on the duct and the prestressing steel.

It is essential that the planning of such openings does not only include the opening itself but equally important, its repair after having carried out the inspection and possibly required rehabilitation work.

In [9] and [10] the authors conclude that the inspection engineer when assessing an existing structure should be aware of the possible hazard scenarios for post-tensioning tendons. Figure 1 shows potentially "weak points" in the case of a typical box girder bridge.

For each type of structure with its particular protection concept, the water, possibly chloride-contaminated can reach the prestressing steel in different ways. When assessing a prestressed concrete bridge, the study of the structural drawings, the construction and maintenance reports and the observations of the owner and his maintenance staff provide information regarding damaging actions and hazard scenarios. The key-question is: Where does (aggressive) water get in contact with the structure and how does it flow off?

In addition, a thorough visual inspection (preferably after rainfall) of the concrete surfaces provides information on damages and their location with regard to the stressed reinforcement such as:

- water flow, wet or moist areas
- discoloration (e.g. rust stains)
- spalling, delamination
- cracks
- honey-combing
- concrete deterioration by freezing and freezing-thawing
- joint leakage

The findings can then be substantiated by in-situ and laboratory investigations. Following these procedures, potential corrosion damages of prestressing steel can be recognized and counter-measures can be taken in good time.

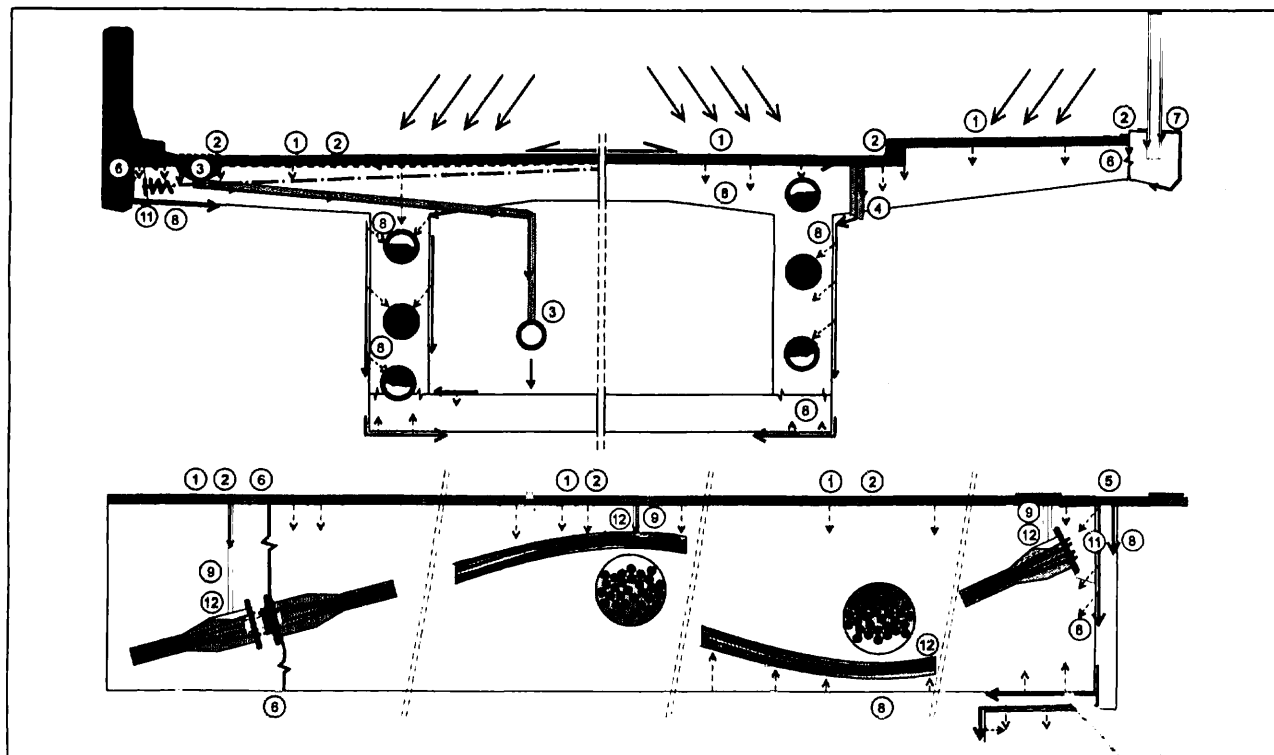


Figure 1: Hazard scenarios for prestressing steel in a typical box girder bridge: Indication of potentially "weak points" where water (possibly contaminated with chlorides) can gain access to the tendons and cause corrosion, [10]

Non-structural elements:

1. Defective wearing course (e.g. cracks)
2. Missing or defective waterproofing membrane incl. edge areas
3. Defective drainage intakes and pipes
4. Wrongly placed outlets for the drainage of wearing course and waterproofing
5. Leaking expansion joints
6. Cracked and leaking construction or element joints
7. Inserts (e.g. for electricity)

Corrosion protection system:

8. Defective concrete cover
9. Partly or fully open grouting in- and outlets (vents)
10. Leaking, damaged metallic ducts mechanically or by corrosion
11. Cracked and porous pocket concrete
12. Grout voids at tendon high points

4. New developments

The evolution of prestressing systems is described in [11]. Some of the ongoing developments include the objective to reliably monitor the condition of tendons. One possibility the author is familiar with, is the use of electrically isolated tendons. Since the early nineties, this new type of tendon has been applied primarily in Switzerland. In these systems, the prestressing steel is entirely isolated not only by a robust plastic duct, but also at the anchorages. Thus it is possible to check the integrity of the chemically stable, diffusion-tight protective envelope by measuring its electrical resistance against the surrounding concrete and the normal reinforcement.

Up to now, about 100 bridges have successfully been constructed using robust plastic ducts of which in over 20 bridges electrically isolated tendons have been installed. More applications are under execution or in planning phase. The electrical resistance is periodically checked, and the results are as

expected. It is important to note, that the protective envelope prevents the ingress of water and harmful substances. The quality of the grout, however must still be perfect.

So far, these applications have been made on the basis of a draft guideline prepared by a working group under the auspices of the Swiss Federal Roads Authority and the Swiss federal Railways. The final German version of this guideline has just been approved. Within the next few months, it will be published under the title: "Measures for providing durability of post-tensioning tendons in bridges". Versions in French and English will subsequently also be prepared and published.

5. Summary and conclusions

The paper provides a survey of the existing possibilities to evaluate and monitor the behaviour of post-tensioning tendons from the author's point of view. It focuses on internal grouted tendons as this type represents at least up to now the bulk of all applications even if in various countries, external tendons have become increasingly popular.

Furthermore it mentions a concrete possibility to improve the durability of post-tensioning tendons permitting at the same time their monitoring during life-time.

The conclusions are as follows:

- The discussed non- or low-destructive inspection and monitoring methods have their limitations ranging from unsuitable to partially suitable. None of them allows a full assessment of the condition of a internal, grouted post-tensioning tendon.
- Up to now, the basic and practical method of carefully opening and visually inspecting a tendon at a location where there is a reasonable doubt still rates best. The visual inspection can be supplemented by tests such as taking samples for laboratory testing investigations. For all these steps the special knowledge of an experienced engineer is required.
- Primarily for prestressed concrete structures in an aggressive environment or for inaccessible structural parts, there is a need for monitoring the tendons. The use of electrically isolated tendons is a concrete possibility which has been proven successful. More development is needed to make other options such as the application of the fibre optic technique or other type of sensors feasible.

References

- [1] P. Matt: "Zerstörungsfreie Prüfung von Spanngliedern in bestehenden Brückenbauten"(Non-destructive testing of post-tensioning tendons in existing bridge structures), Eidg. Departement für Umwelt, Verkehr, Energie und Kommunikation, Bundesamt für Strassen, Bericht Nr. 170, 1988
- [2] X. Dérobert, O.Coffec: "Localisation des armatures des ouvrages d'art en béton armé ou précontraint par les techniques de radar" (Localization of reinforcement in reinforced or prestressed structures using the radar technique), Bulletin des laboratoires des ponts et chaussées, 230, Janvier/Février 2001, pp 57-65
- [3] M. Sansalone, W. Street: "Use of the Impact-Echo Method and Field Instrument for Non-destructive Testing of Concrete Structures", International Symposium ND-Testing in Civil Engineering, Berlin, 1995, Proceedings Vol. 1, pp 495 – 502
- [4] E. Brühwiler et al: "Applicabilité de la méthode Impact-Echo" (Applicability of the Impact-Echo Method), Eidg. Departement für Umwelt, Verkehr, Energie und Kommunikation, Bundesamt für Strassen, Bericht Nr. 550, 2001
- [5] H. Scheel, B. Hillemeier: "The Capacity of the Remanent Magnetism Method to Detect Fractures of Steel in Tendons Embedded in Prestressed Concrete", International Symposium on Non-Destructive Testing in Civil Engineering, Berlin 1995, Proceedings, Vol. 1, pp 211-218

- [6] B. Elsener et al: "Zerstörungsfreie Spannkabelprüfung mit reflektometrischer Impulsmessung" (Non-destructive testing of tendons by the reflectometrical impulse measurement), Eidg. Departement für Umwelt, Verkehr, Energie und Kommunikation, Bundesamt für Strassen, Bericht Nr. 523, 1997
- [7] D. Jungwirth et al: "Dauerhafte Betonbauwerke" (Durable Concrete Structures), Beton-Verlag, Düsseldorf, 1986
- [8] D.W. Cullington et al: "Continuous acoustic monitoring of grouted post-tensioned concrete bridges", NDT & E International, Elsevier Science Ltd., V34 No2, March 2001, pp 95-106
- [9] F. Hunkeler et al: "Korrosionsschäden an Spannstählen in Spanngliedern und vorgespannten Boden- und Felsankern" (Corrosion damages on prestressing steel in post-tensioning tendons and prestressed soil and rock anchors), Eidg. Departement für Umwelt, Verkehr, Energie und Kommunikation, Bundesamt für Strassen, Bericht 534, 1998
- [10] P. Matt et al: "Durability of Prestressed Concrete Bridges in Switzerland", 16th Congress of IABSE, September 2000, Congress Report
- [11] H. R. Ganz: "Evolution of Prestressing Systems", fib/IABSE Workshop on Durability of Post-tensioning Tendons, Proceedings, Ghent, 2001

Maintenance of Prestressed Concrete Bridges

Yuzuru HAMADA

Chief Research Engineer, Research and Development Center,
DPS Bridge Works Co., Ltd., Tokyo, Japan

Yoshinobu ISHIKAWA

Chief Engineer, Structural Design Section,
Oriental Construction Co., Ltd., Tokyo, Japan

Minoru MIZOE

Deputy Manager, Bridge and Structural Engineering Division,
Japan Highway Public Corporation, Tokyo, Japan

Toyo MIYAGAWA

Professor, Department of Civil Engineering,
Kyoto University, Kyoto, Japan

Summary

The Japan Prestressed Concrete Engineering Association (JPCEA) has taken initiatives toward development of durable prestressed concrete structures. The Research Subcommittee on Improvement in Durability of Prestressed Concrete Bridges was set up by JPCEA in 1998. "Manual for Maintenance of Prestressed Concrete Bridges" was established as one of the final output of this subcommittee.

This paper outlines the Manual for Maintenance of Prestressed Concrete Bridges. Some aspects of Non-Destructive Tests and Measures toward the insufficiently grouted PC tendon, which is one of the most important elements of technology on the maintenance of PC bridges, are also reported in this paper.

1 Introduction

Concrete structures, which are appropriately designed and are carefully constructed with appropriate materials, are a form of structures with excellent durability. However, deterioration is progressed as time elapses, when such structures are used for a long period. The 20th century has been an era creating many concrete structures of various forms. In contrast, the 21st century will be an era of mastering the maintenance of such structures. Gone are the days of simple scrap and build.

The Japan Prestressed Concrete Engineering Association (JPCEA) established a subcommittee on "Improvement in Durability of Prestressed Concrete Bridges" to investigate on the enhancement of durability of such structures. The First Subcommittee that functioned during 1994-1996, published the draft "Recommendation on the Design and Construction of PC Bridges with Improved Durability" which provides effective details for the newly built ones [JPCEA (1997)]. Afterwards, several cases of deteriorated concrete structures were reported

that made the durability of concrete structures a serious social problem, drawing the attention of the general public. As for the PC structures, deterioration due to insufficient grouting posed a serious problem, and it was of utmost urgent to regulate the specifications incorporating the latest knowledge on durability of such structures. Considering this need, the Second Subcommittee was formed in 1998, under the chairmanship of Prof. T. Miyagawa. In the first stage of research of this Second Subcommittee, examples of deteriorated PC bridges were collected. In the next stage, repair and strengthening methods used in such bridges were investigated. Moreover, the relation between deterioration cases and methods of repair and strengthening was analyzed, resulting in the publication of the "Manual for Maintenance of Prestressed Concrete Bridges" in 2000 [JPCEA (2000)].

This paper introduces the outline of the "Manual for Maintenance of Prestressed Concrete Bridges". Moreover, non-destructive test (NDT) to examine the degree of grouting within the ducts is discussed, which is one of the important elements of technology on the maintenance of such structures. Further, the maintenance measures of insufficiently grouted prestressed tendons are also discussed.

2 Manual for Maintenance of Prestressed Concrete Bridges

2.1 Background

Considering the importance of maintenance of concrete structures, the Japan Society of Civil Engineers (JSCE) started its research in 1988 in view of preparing a standard on maintenance of concrete structures. Although various investigations had been carried out since then [JSCE (1997), Seki (1998)], preparation of the standard specification for maintenance began from 1999 and the "JSCE Standard Specification for Maintenance of Concrete Structures" was published in January 2001 [JSCE (2001), Miyagawa (2001)]. Thereby, JSCE Standard Specification had the framework to deal with concrete structures in a unified manner, incorporating the aspects of design, construction and maintenance.

On the other hand, the Second Subcommittee was established by the JPCEA in 1998 to study on the improvement of durability in PC bridges. During the period of its activity, the preparation of the JSCE standard specifications commenced, which resulted in specifying the fundamentals for the maintenance of concrete structures. Thereupon, the Second subcommittee decided to prepare a manual for the maintenance of existing PC bridges, to comply with JSCE standard specifications.

2.2 Outline of the Manual

The JSCE standard specification covers the maintenance of all types of concrete structures, including the newly built ones. Further, this specification may be considered to be written for the manual writers. However, site engineers needed a manual to carry out the maintenance of deteriorated PC bridges. As such, it was decided to illustrate in detail on the maintenance of deteriorated PC bridges in the current manual based on the JSCE standard specification. That is, the manual focused on the corrective maintenance (ex post-facto maintenance) measures. Moreover, the JSCE standard specification is a performance-based standard having a hierarchical structure as shown in Fig. 1. As such, the JPCEA manual was also prepared according to performance-based verification. As for the performance, five factors were incorporated namely; safety, serviceability, degree of possible impact on the third-party, aesthetics and durability (performance over time). However, to be easily used by the site engineers, this manual also adopted many prescriptive specifications (conventional

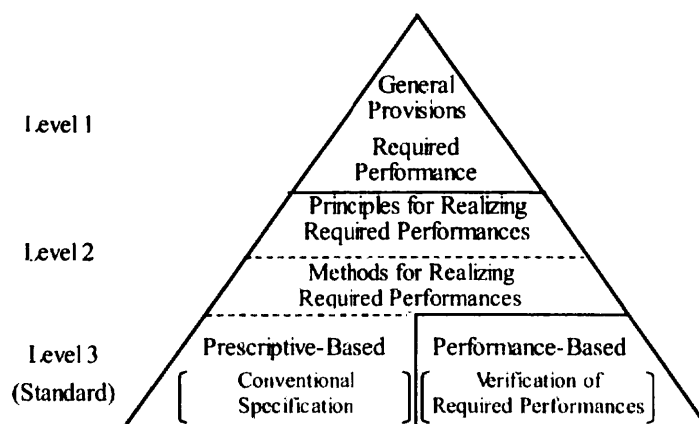


Fig. 1: Hierarchical structure of JSCE standard specification

specification). It is believed that the combination of these two methods would yield the best outcome.

The contents of the manual are shown in Table 1. As shown in Fig. 2, each item is specified according to the maintenance process that would be generally carried out in an existing structure.

2.3 Features of the Manual

Before commencing the preparation of this manual, a survey was conducted based on 164 published literature on the repair and strengthening of existing PC bridges in Japan and overseas. For each of these references, the deterioration phenomena, deterioration factor, deterioration mechanism or selected repair/strengthening methods were sorted which formed the basis of data required for preparation of the manual. The features of this manual are described in the following.

- 1) The deterioration mechanism is discussed on the general aspects such as chloride induced corrosion, carbonation induced corrosion, frost damage and alkali-aggregate reaction. Besides, specific aspects concerning PC bridges such as creep and shrinkage behavior as well as decrease in performance of accessories such as bearing and expansion joint are also discussed.
- 2) The result of the literature survey is shown in Fig. 3. The literature investigation revealed that almost all deterioration cases were caused by chloride induced corrosion. Moreover, it was clear that the cause of rupture of prestressing tendons, which had a significant influence of the performance of PC bridges, was insufficient grouting. Considering this fact,

Table 1: Contents of JPCEA Maintenance Manual

Chapter 1: General
Chapter 2: Inspection
Chapter 3: Determination of Deterioration Mechanisms and Deterioration Prediction
Chapter 4: Assessment and Measures
Chapter 5: Repair and Strengthening Methods
Appendix 1: Analysis of Reference Papers
Appendix 2: Outlines of Repair and Strengthening Methods
Appendix 3: Example of Maintenance of PC Bridges

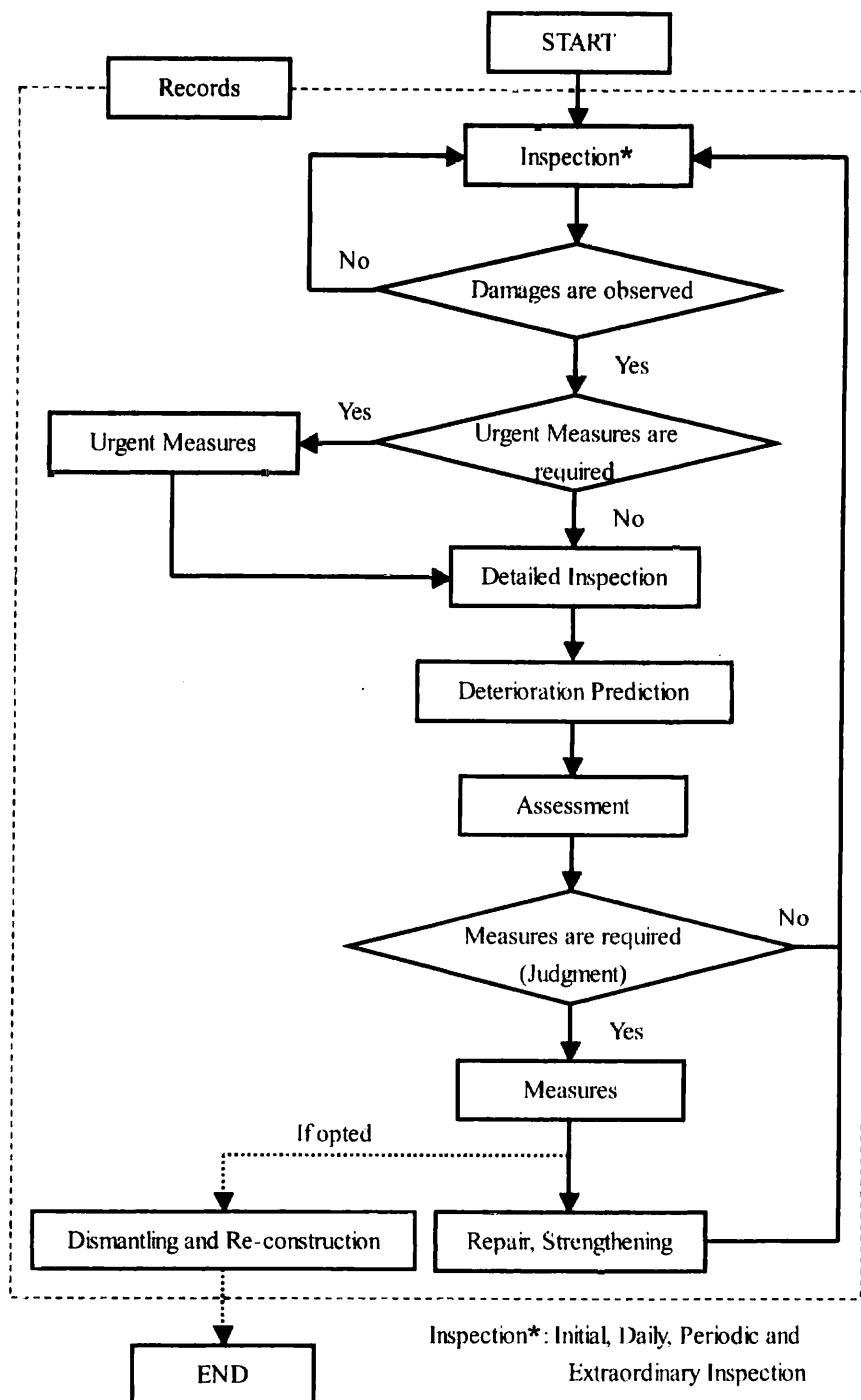


Fig. 2: Flow of maintenance process

insufficient grouting was taken up as one of the deterioration factors of PC bridges in this manual. As for the method of inspection, NDT methods to examine the degree of grouting within the ducts are explained.

- 3) It is decided that the assessment of deteriorated PC bridges and the selection of the measure (such as repair, strengthening, dismantling and re-construction) are based on the initiation limit state of corrosion of prestressing tendons. In the case of chloride induced corrosion, the examination of the initiation of corrosion is carried out by verifying that the chloride content in the concrete layer at the surface of prestressing tendon dose not reach to the limit chloride content of 1.2 kg/m^3 . In the case of carbonation induced corrosion, if the carbonation depth is found to be larger than the concrete cover of prestressing tendon, then

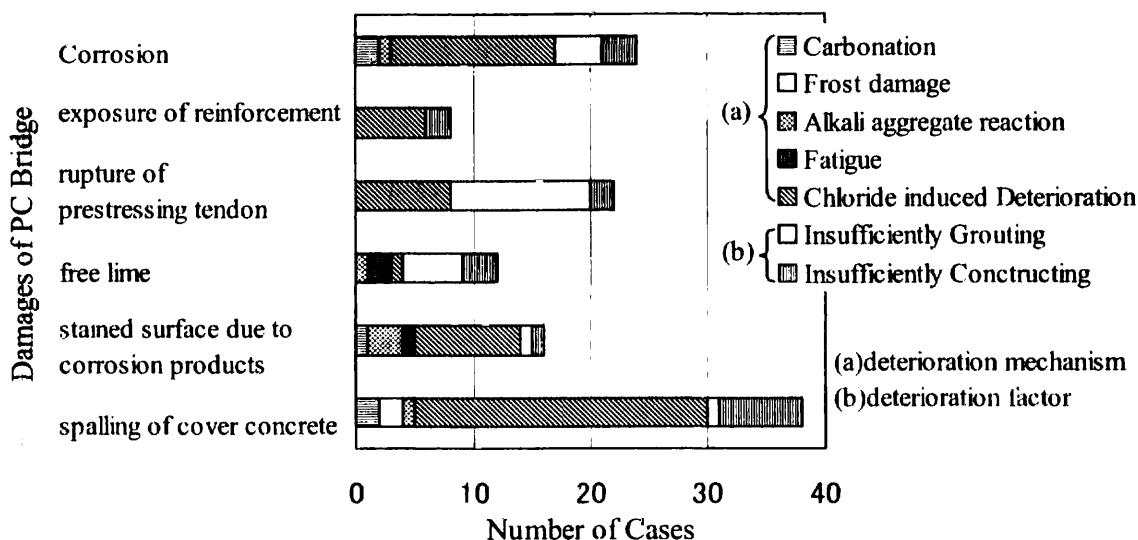


Fig 3: An example of analysis of literature survey

it is judged that corrosion of prestressing tendon has begun.

- 4) As for the repair, methods that can be applied to each deterioration mechanism are summarized. Moreover, relating to the specific characteristics of PC bridges, repair methods such as *patching of large-scale cross-sections* and *cathodic protection method* are discussed. On the other hand, general strengthening methods such as *external cable method* and *steel plate bonding method*, as well as *main girder connection method* that improve the structural function of PC bridges are also explained.
- 5) In Appendix 2 of the manual, 18 kinds of repair and strengthening methods including the future possibilities for PC bridges were outlined, as shown in Table 2. Moreover, an example is illustrated for the selection method for repair and strengthening, based on this manual.

3 Non-Destructive Testing to Examine the Degree of Grouting

3.1 Importance of Non-Destructive Testing

The collapse of Ynys-y-Gwas Bridge in Great Britain in 1985 that was caused by insufficient grouting [Woodward and Williams (1988)] was a shocking incident. Moreover, as mentioned previously, it has been found in Japan too, that serious deterioration has occurred in PC bridges due to insufficient grouting. From such background, it was reconfirmed that grouting is

Table 2: Repair and strengthening method

	Repair Method	Strengthening Method
(1)	Re-Grouting Method	Steel Plate Bonding (Jacketing) Method
(2)	Crack Injection Method	FRP Sheet Bonding (Jacketing) Method
(3)	Fill Method	Top Surface Overlaying Method
(4)	Patching Method	Soffit Surface Overlaying Method
(5)	Surface Treatment Method	External Cable Method
(6)	Cathodic Protection Method	Main Girder Connection Method
(7)	Electrochemical Chloride Removal Method (Desalination)	Support Point Addition Method
(8)	Electrochemical Re-alkalisation Method	Bearing Replacement Method
(9)		Prestressed Concrete Jacketing Method
(10)		Prestress Introduction Method for Wall Piers

Table 3: Different NDT methods and its applicability

	Principle	Test basis		Results of examination	Applicability
		Full length	Local		
Electromagnetic wave method	Electromagnetic wave is sent through concrete and from reflection caused by material with different electro-magnetic property other than concrete, the void location is predicted.		●	- With complicated reinforcement, difficult to get details of deeper locations. - It is difficult to make proper distinction inside the steel sheaths.	Not applicable
Ultrasonic penetration method	Ultrasonic wave is penetrated from concrete surface and the degree of grouting is predicted from change in velocity of received wave.		●	- Member thickness under investigation is limited to about 200 mm. - Difficult to evaluate by observing only amplitude and velocity of propagated waves.	Applicable in some cases
X-ray penetration method	X-ray beam is penetrated from concrete surface and internal condition is predicted from X-ray photograph based on weak and strong properties.		●	- Applicable to members thickness up to 500 mm - Need to protect the radiation zone. - Takes a long time to measure	Applicable
Impact elastic-wave method	AE sensors are fixed at both ends and impact wave is produced by hitting with a hammer at one end, and the degree of grouting is predicted from characteristic of input and output waves.	●		- Applied mostly in slabs and cross-girders - Judged based on total tendon length - Need to expose the anchorage for direct hammering	Applicable

extremely important for PC bridges. Furthermore, considering the maintenance, in order to evaluate the durability of existing PC bridges, it has been thought that it is important to examine the degree of grouting within the ducts. Generally, the degree of grouting is examined by NDT. However, currently there has been no established NDT technique for such examination. The current state in Japan on NDT to examine the degree of grouting is described below.

3.2 Outline of Non-Destructive Testing Method and its Application

The Japan Prestressed Concrete Contractors Association (JPCCA) commenced the research on the NDT to examine the degree of grouting within the ducts in 1996 [Tokura (1999)]. The outline of the different NDT methods investigated and the results of their applicability are given in Table 3. The results revealed that the *X-ray penetration method* and *impact elastic-wave method* were acceptable for the inspection of degree of grouting. These NDT methods have also been recommended as the applicable ones in the manual outlined in the preceding section. The use of *electromagnetic wave method* (*ground penetrating radar method*) were not suitable as the waves could be reflected back due to the presence of steel sheaths, thus making it difficult to make a proper distinction inside the sheaths. Moreover, in the case of *ultrasonic penetration methods*, since the damping of dilatational waves (P-waves) into the concrete is large, member thickness under investigation is limited to about 200 mm. As such, it was judged that the above two methods were not applicable.

In X-ray penetrating method, the rays are generated from the side of concrete surface and photographed after penetrating through the concrete member. The degree of grouting is judged based on the X-ray photograph. When it is sufficiently grouted, the location of sheath is reflected in a whitish color, similar to that of concrete. When insufficiently grouted, the void in the sheath is reflected as black. For member thickness up to 250 mm, X-ray films specified for engineering purpose is used. However, when the member thickness increases beyond this value, an imaging plate is used instead of the film and the image is computer processed to judge the degree of grouting. Generally, the degree of grouting is judged and categorized accordingly as shown in Table 4. On the other hand, by impact elastic-wave method, the AE sensors are attached at both anchorage ends and the one end is hit with a hammer. The degree of grouting is judged from the velocity of elastic wave propagation and the frequency characteristic of an input/output waves. The results of the judgment can be categorized into three types, namely, grouted, insufficiently grouted and not grouted. However, the length of prestressing tendons, tendon layout and thickness of the concrete member influences the judgment. As such, a specialist with sufficient experience should make the judgment.

Table 4: Judgment on the degree of grouting

Judgment	State within the ducts
Grouted	fully injected ducts
Insufficiently Grouted	poorly injected ducts
Not Grouted	completely empty ducts

Table 5: Comparison of the applicability of NDT

	Web	Slab	Transverse Girder	Main Girder
X-ray penetration method	○	△	×	○
Impact elastic-wave method	△	◎	◎	×
Grout trace inspection	○	○	△	○
Bore hole inspection	△	△	△	△

◎ : Most suitable

○ : Suitable

△ : Applicable in some cases

× : Not suitable

In Japan, a review of research on NDT tests to examine the degree of grouting was conducted as early as 1989 [Fuji and Miyagawa (1989)]. However, a systematic research from model tests to application in actual structures is believed to have commenced only after the accident that occurred in 1994, where a prestressing bar protruded from a transverse girder [Kobayashi, et. al. (1995)]. In this research, applicability of NDT methods such as X-ray, infrared thermography and impact elastic-wave were investigated. It was found that the impact elastic-wave method is most convenient to examine the degree of grouting of prestressing bars in slabs and transverse girders where many number of bars are provided. Subsequently, considerable research has progressed on the impact elastic-waves methods. One such finding was that the parameters such as damping of high frequency and change in the peak frequency are sensitive to the degree of grouting [Amasaki et. al (1998, 1999)]. Moreover, it has been found that the grouting condition of fresh cement-paste was related to the damping characteristics, thus applicable even during construction stage for the inspection of the degree of grouting [Tomita, et. al. (2000)]. Besides the above, a new NDT method has been developed recently, adapting the *impact-echo method* [Ohtsu et.al. (2001)]. This method visualizes and evaluates the void in a section using the relation between the theoretical resonance frequency (which is calculate based on the velocity of elastic wave propagation and the distance of the reflection paths), and the actual frequency spectrum measured by impact-echo method. Therefore, this new NDT method is called Stack Imaging of Spectral Amplitudes Based on Impact Echo (SEBIE). SEBIE method makes easier the interpretation of the frequency domain in the elastic wave testing, and it is expected that further research progresses would be made.

3.3 Example of an Application of Non-Destructive Testing

The prestressing steel arranged in a typical PC bridges could be classified into web (for shear), slab, transverse girder and main girder. Since the position and thickness of the member differ from place to place, it is rather difficult to examine the degree of grouting within the ducts using the same inspection method. For this reason, considering the arrangement of these steels, a suitable inspection method needs to be selected. An example of the application of NDT is shown in Table 5. The X-rays penetrating method is applicable to prestressing steel in webs and main girders, which are arranged in comparatively thin webs. This method is not applicable for prestressing steel in slabs, since the photograph is taken from the upper and lower sides, which will misinterpret “insufficient grouting” as “sufficient grouting” as shown in Fig. 4, thus needing care in its application. The impact elastic-wave method is applicable in comparatively

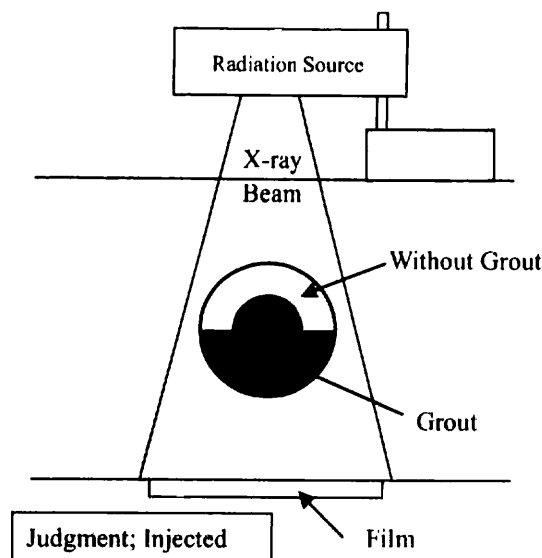


Fig. 4: Misinterpretation of degree of grouting using X-ray penetration method

short prestressing steel arranged in structural members such as slabs or transverse girders. This is especially true in slabs with cantilevered section, since the AE sensor could be attached easily at both ends, which make the impact elastic-wave method as the main inspection method. In case of steel in webs for shear, when large number of such steel is placed at close intervals or when the concrete cover is more than 200 mm, the application of the impact elastic-wave method is difficult.

On the other hand, using a combination of NDT methods is highly recommended. As shown in Table 5, it is proposed that when using X-ray penetrating method or impact elastic-wave method, it should be used in conjunction with *grout trace inspection* or *bore-hole inspection*. Grout trace inspection is a method of checking the degree of grouting indirectly according to the trace of grout poured through the intermediate vent hose or anchorage system. As such, the judgment on the degree of grouting is either “likely to be not grouted” or “possibly grouted”. On the other hand, bore-hole inspection is a way of drilling a hole with a diameter of 20-25mm using a equipment such as hammer drill with automatic shut-off in case of metal contact, and visually estimating the degree of grouting. Generally, the inside of the sheath is photographed using a CCD camera or fiber-scope. This method has a disadvantage of injuring the structure at some specific places. However, unlike the three level judgment as shown in Table 4, in bore-hole inspection, it is possible to get a detailed view of the condition inside the sheaths, such as the condition of grouting as well as any corrosion or rupture of the prestressing tendons. Moreover, the inspection costs are less compared with other NDT tests. The bore-hole inspection is effective to re-examine the degree of grouting within the ducts, when judgment is made as “insufficient grouting” or “no grouting” using X-ray penetrating method or impact elastic-wave method. This method is also suitable to finalize the judgment of an inspection carried out using impact elastic-wave method.

4 Maintenance Measures for PC Tendons with Insufficient Grouting

4.1 Urgent Measures

When damage is found in a PC bridge during inspection, assessment for urgent measures shall be carried out. Urgent assessment has to be performed in a short period of time, since it affects the necessary urgent measures and planning of detailed inspection. However, the degree

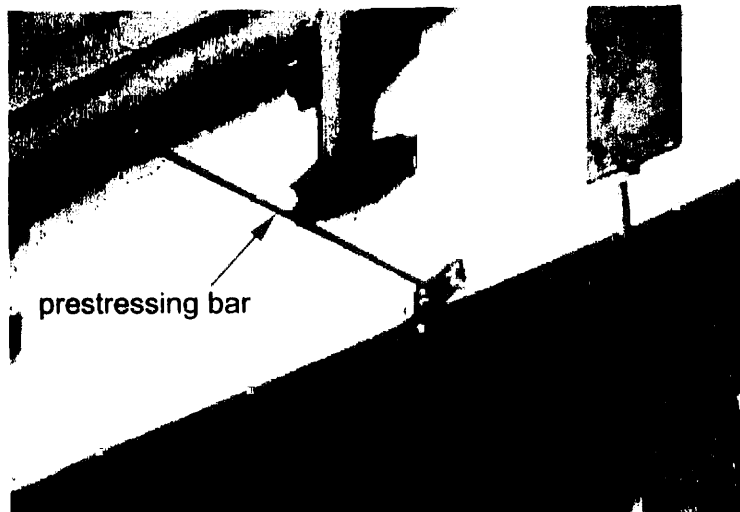


Fig. 5: An example of prestressing bar protruding

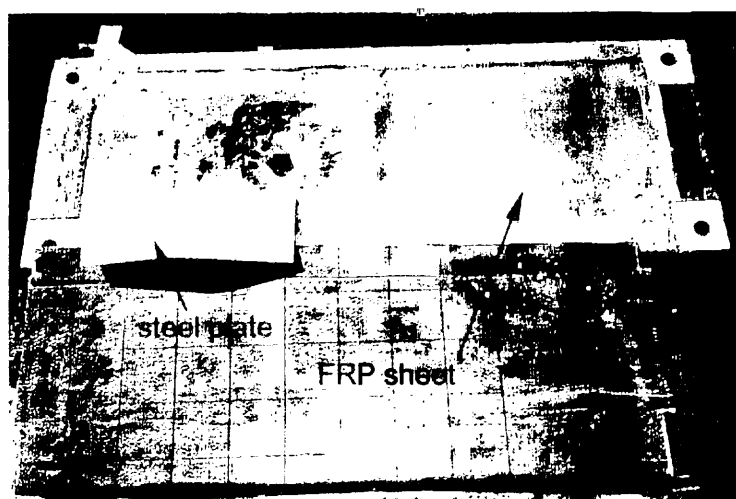


Fig. 6: An example of protrusion prevention method

of grouting in prestressing steel that is not ruptured is not clear unless results of the detailed inspection is known. As such, in urgent assessment, it is good enough to neglect only the ruptured tendons and evaluate the safety of the structure based on the original design plans. If the results of the assessment predict any danger to a by-passer, immediate action is needed such as traffic restriction or traffic interception. Moreover, when it is judged that the advancement of deterioration inflicts damage to a third party, measures need to be taken to prevent accidents such as falling of concrete and protrusion of prestressing bars. Especially, when it is confirmed that the prestressing bar is ruptured as shown in Fig. 5, adoption of protrusion prevention method is recommended. Fig. 6 shows an experimental installation condition of such method. Considerable amount of time and money is necessary for detailed inspection and re-grouting of prestressing bar in slab or transverse girders where many number of bars are placed. As such, it is necessary adopt such methods to prevent the protrusion of prestressing bars until appropriate measures are taken, so that damage to third party is avoided. In the method shown in Fig. 6, steel plate is installed at the location of prestressing bar and FRP sheet is bonded continuously on top of that. The number of layers of FRP sheets and its details are determined by conducting tests.

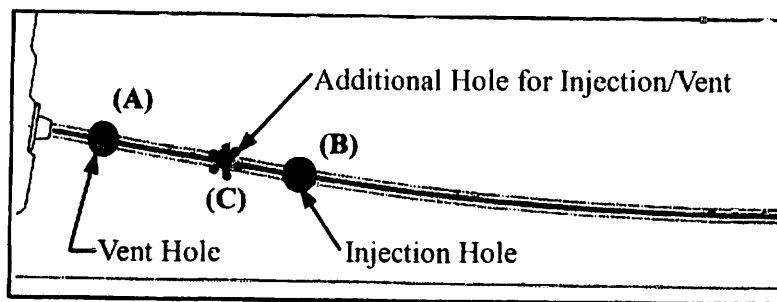


Fig 7: Location of bore-holes for re-grouting

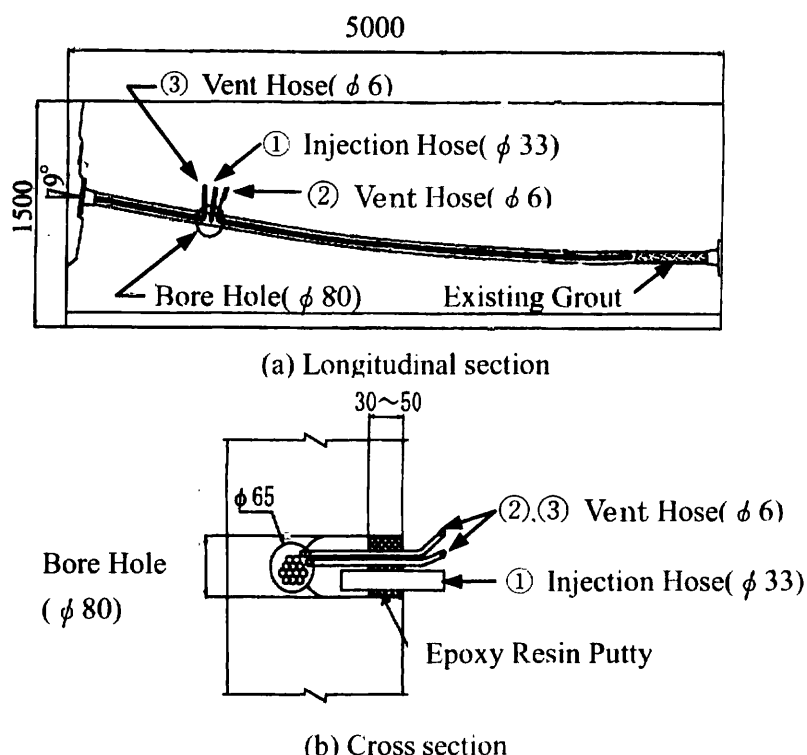


Fig. 8: Newly developed re-grouting technique by RTRI

4.2 Repair

When prestressing steel with insufficient grouting has been discovered, it should be re-grouted to prevent deterioration of the prestressing steel. For re-grouting, generally, holes are drilled at two locations. Before re-grouting, the air is passed through one hole (B) and the checked for its discharge through the second hole (A). When air is not passing, a third hole is drilled in-between (A) and (B) as shown in Fig. 7 and after confirming the passage of air, the re-grouting is done.

According to the above conventional re-grouting method, it is necessary to drill holes at two or more places, which is not an easy task. Considering this, a research group in collaboration with the Railway Technical Research Institute developed an innovative re-grouting method [Tottori et. al. (2001)]. The outline of the newly developed re-grouting technique is illustrated in Fig. 8. When this method is used, it is possible to re-grout with only one hole drilled per repair portion. In the drilled hole, a grout injection hose of 33mm diameter and vent hose of 6mm diameter is arranged. It has been experimentally confirmed that if the tip of vent hose can be placed within 1.0m from the end insufficiently grouted portion, with a grouting pressure of about 1.0 Mpa, it is possible to re-grout it completely. Therefore, this method is effective when

applied to non-grouted regions such as that found in the vicinity of anchorages, produced by bleeding of grout. This method is cheaper than re-grouting using a vacuum pump and the repair work easy. Further examination is necessary on the applicability of this method in actual structures.

4.3 Strengthening

As a result of detailed inspection, when the presence of insufficiently grouted tendons, corroded tendons or ruptured tendons is observed, it is necessary to evaluate the safety of such bridge for assessment. Consequently, when it is judged that the structural capacity is insufficient, strengthening is carried out. Assessment aims at evaluating the necessity for repair and strengthening. It is important to record and keep the results of this assessment for future maintenance planning of the bridge under consideration.

In the evaluating the safety of the bridge, initially, sound prestressing tendons are selected based on the judgment on the degree of grouting. When it is confirmed that the prestressing tendon is not ruptured, the flexural and shear capacity of such structural member is calculated as follows. This assumption is set up so that the results may be obtained on the safer side.

- Prestressing tendons that are judged as “grouted” is considered as tendons with bonding.
- Prestressing tendons that are judged as “not grouted” or “insufficiently grouted” is considered as tendons with bonding if re-grouting is carried out.
- Prestressing tendons that are judged as “not grouted” or “insufficiently grouted” is considered as ruptured tendons if re-grouting is not carried out.

On the other hand, when there is doubt about insufficient grouting, the prestressing tendons are considered as unbonded tendons and the flexural capacity could be calculated by using the appropriate method. In this case, inspection should be carried out at suitable intervals to monitor continuously that the corrosion of tendons has not been initiated. The design value for materials used in the original design may be generally used as the design value for the assessment. However, when the actual strength of concrete is known from drilled concrete core tests, the design value may be defined using this test results. When it is judged from assessment that the capacity is insufficient, strengthening should be carried out using appropriate methods. For PC bridges, the following methods have been used widely for actual repair/strengthening of bridges.

- Crack injection method
- Surface treatment method
- External cable method
- FRP sheet bonding method

4.4 Dismantling and Re-construction

Even after examining the possibilities of strengthening when it is evident that the capacity is not enough, sometime it is not possible to select an appropriate strengthening method to restore the required capacity. Moreover, even if it is possible to select a strengthening method, huge amount of costs may be incurred for the strengthening and subsequent maintenance, resulting in an exorbitant life cycle cost. Sometimes it is practically impossible to re-construct in cases like metropolitan expressways. However, if an alternative route is securable, it is necessary to consider the dismantling and re-construction of the PC bridges evaluating the residual life and the life cycle cost. In Japan, although repair and strengthening were repeatedly carried out on PC bridges that had been damaged by chloride induced corrosion, it has been reported that finally the bridge had to be dismantled.

5 Concluding Remarks – Future of Maintenance

Maintenance of structures should be carried out based on the appropriate design scenario for the individual structures. It is important to perform strategic maintenance of deteriorated structures keeping in mind the dismantling and re-construction, as well as recycling of the structure. However, in the present stage, it is known that strategic maintenance has been only possible using the life cycle cost based on the scenario. From now on, there will be many other considerations that demand further investigation. At the time of publishing of the Maintenance Manual, it is believed that the investigations in connection with maintenance of the PC bridges have progressed a step further. As a result, it can be said that it is necessary to revise and upgrade this Manual reflecting the latest information on maintenance of PC structures.

Reference

- 1) JPCEA: *Recommendation on the Design and Construction of PC Bridges with Improved Durability*, March 1997. (in Japanese)
- 2) JPCEA: *Manual for Maintenance of Prestressed Concrete Bridges*, November 2000. (in Japanese)
- 3) JSCE Subcommittee on Maintenance of Concrete Structures: Recommendation for Maintenance of Concrete Structures -Draft-, *Concrete Library of JSCE*, No.29, pp.1-14, June 1997. (Translation from the Concrete Library No.81 published by JSCE, October 1995)
- 4) Seki, H.: Research Activities Related to Maintenance of Concrete Structures, *Concrete Library of JSCE*, No.32, pp.1-16, December 1998. (Translation from Proceeding of JSCE, No.557/V-34, February 1997)
- 5) JSCE: *Standard Specification for Maintenance of Concrete Structures*, January 2001. (in Japanese)
- 6) Miyagawa, T.: Committee Notes on JSCE Standard Specification for Maintenance of Concrete Structures Standard -Background and Future of Maintenance Specification-, *Concrete Library* No.104, pp.1-7, January 2001. (in Japanese)
- 7) Woodward, R.J. and Williams, F.W.: Collapse of Ynys-y-Gwas Bridge, West Glamorgan, *Proceeding of The Institution for Civil Engineers* Part 1, August 1988.
- 8) Tokura, K.: Durability Improvement of Prestressed Concrete Bridge, *Journal of Prestressed Concrete*, Vol.41, No.6, pp.56-68, November-December 1999. (in Japanese)
- 9) Fujii, M. and Miyagawa, T.: Nondestructive Testing Method for Grouting Condition in Prestressed Concrete Structures, *Proceeding of the JSCE*, No.402/V-10, pp.15-26, February 1989. (in Japanese)
- 10) Kobayashi, K., Miyagawa, T., Hatta, Y. and Kuzume, K.: NDT of Grouting in Transverse Prestressing Steel in T-Beam Bridge, *Proceedings of the Sixth International Conference on Structural Faults and Repair*, Vol.1, pp.73-80, July 1995.
- 11) Nakano, S. and Amasaki, S.: Diagnosing of Grouting in PC Beams by Impact Hammer Method, *Proceeding of JCI*, Vol.20, No.1, pp.287-292, 1998. (in Japanese)
- 12) Saito, H. and Amasaki, S.: Diagnosing of Grouting in PC Beams by Impact Hammer Method, *Proceeding of JCI*, Vol.21, No.2, pp.1267-1272, 1999. (in Japanese)
- 13) Tomita, Y., Iwanami, M. and Otuki, N.: Nondestructive Evaluation of Fresh Grouting Performance in Prestressed Concrete Members by Impact Echo, *Journal of Materials, Concrete Structures and Pavements, JSCE*, No.648/V-47, pp.127-135, 2000. (in Japanese)
- 14) Ohtsu, M., Tokai, M. and Kosaka, K.: Imaging of Defects in Concrete by SIBIE, *Proceedings of the JCI*, Vol.23, No.1, pp.499-504, 2001. (in Japanese)
- 15) Tottori, S., Nitta, K. and Yoshida, K.: Repair for PC Grout Defective Filling, *Railway Technical Research Institute Report*, Vol.15, No.8, pp.7-12, August 2001. (in Japanese)

Repair and strengthening of damaged tendons and structures

M. WICKE

University of Innsbruck, Germany

1 General

During the last two decades numerous damages of post-tensioning tendons were observed. Mainly tendons in superstructures of road bridges were concerned. Poorly executed grouting was the predominant reason for the damages. Consequently water, polluted by chlorides from de-icing agents could contact the prestressing steel and initiate the corrosion process. Two different strategies were developed to solve this problem. An improvement of grouting technology should help to avoid lacunae in ducts. On the other hand the use of unbonded tendons with factory applied corrosion protection was promoted.

Damages in superstructures occurred very often due to a not accurate design philosophy. This holds true especially for central Europe. In DIN 4227-1 and ÖNORM B 4250 the main goal for design of prestressed concrete structures was to avoid cracking under service load conditions. All calculations for Serviceability Limit States run under the assumption of uncracked cross sections. On the action side only loads were taken into consideration and no imposed or hindered deformations. Thus cracks could occur mainly in regions with low stresses due to loads and higher due to imposed deformations. Such cracks occurred unexpectedly and no sufficient normal reinforcement for crack width control was provided. Therefore a lot of problems raised from that cracks and rehabilitation was needed.

Increase of traffic volume and traffic loads requires retrofitting and strengthening of existing superstructures. In some cases the increasing volume of traffic requires the application of additional lanes. The strengthening of the superstructures in longitudinal and transverse direction is a consequence of the widening of the bridge. The increase of axle loads, especially of specific vehicles for heavy loads may also lead strengthening of structures.

For a bridge under consideration specific requirements or combination of requirements have to be considered. Therefore a specific design for each particular structure has to be carried out and no general design rules can be given. For the execution of rehabilitation work well experienced contractors are required. Independent supervision of the work on site is recommended. In the following chapters some examples may demonstrate the problems raised and their solution.

2 Replacing of a corroded concentrated tendon [1], [2], [3]

During a usual bridge inspection executed every 6 years a longitudinal crack in the concrete surrounding the concentrated tendon was observed. The damage was located close to the end of the tendon where it deviates to the reverse direction and is twisted from the normal shape to that at the deviation block (Fig. 1).

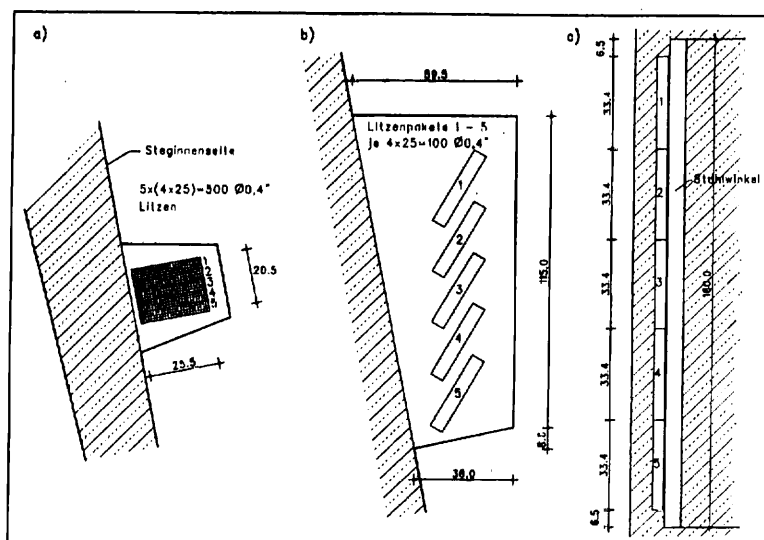


Fig.1: cross section of the concentrated tendon:
a) in the area in front of the damage
b) at the corroded area
c) at the deviation block

The entire length of the crack was about 12 m, with a maximum crack width of 35 mm (Fig. 2). After removing of the covering concrete the full damage could be observed (Fig. 3) and registered (Fig. 4).

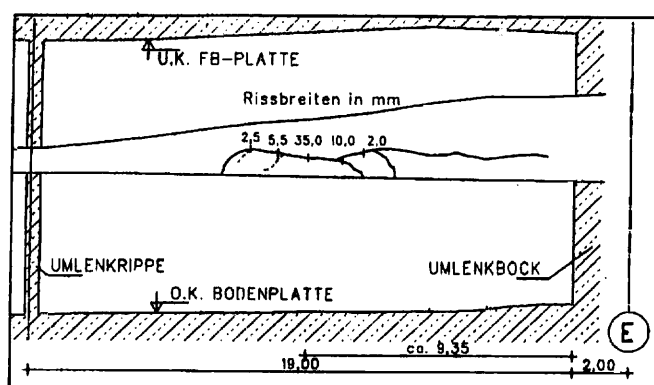


Fig. 2 Crack pattern in the corroded area



Fig. 3 Corroded strands

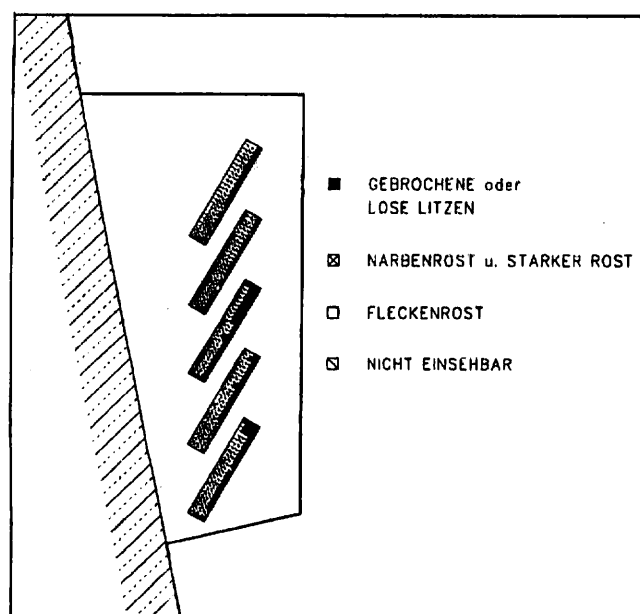


Fig. 4 Damage assessment of the corroded strands

Previously the broken strands in the tendon were fixed by transverse applied prestressing forces (Fig. 5) to prevent a zipper-like propagation of the crack in the concrete cover. The crack was caused due to splitting forces of the re-anchored disrupted strands.

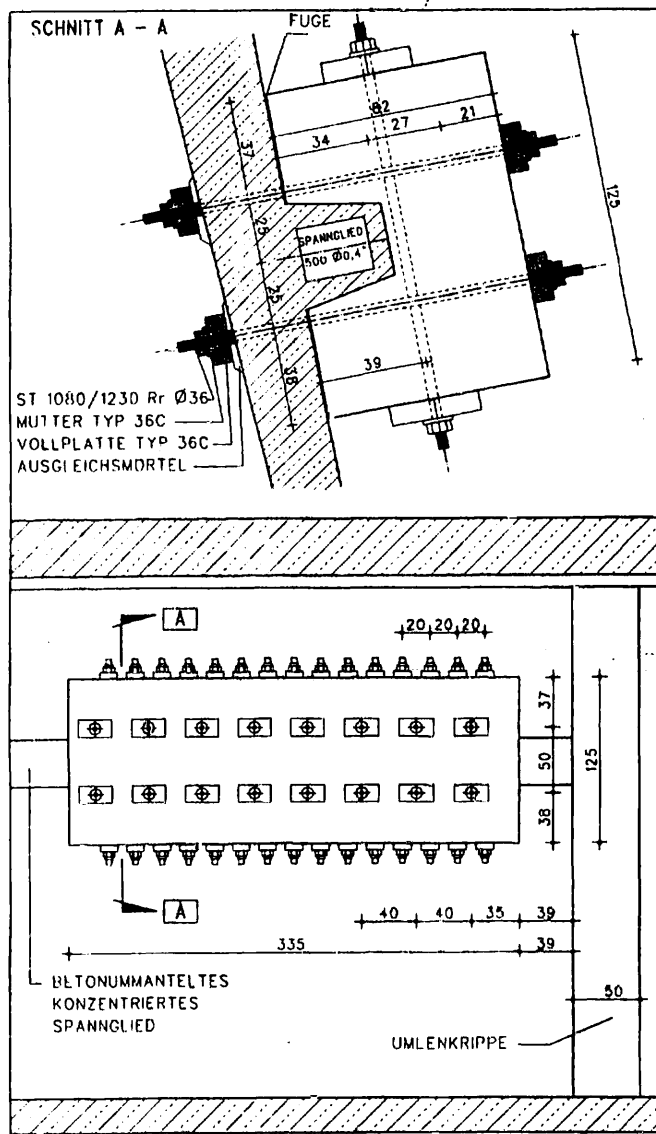


Fig. 5 Additional clamping with prestressed concrete packing ring

After discussing different repair methods very thoroughly it was decided to replace the entire concentrated tendon by a new tendon. After supporting the girder by provisional piers (Fig. 6) the tendon was removed in three steps over the total length of the superstructure (Fig. 7). Correspondingly the forces in the jacks at the intermediate piers were increased.

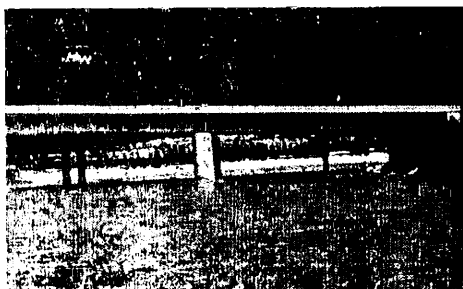


Fig. 6 Provisional Piers



Fig. 7 Removing of the concentrated tendon

The new tendon comprises 6 special rectangular tendons, consisting of 36 strands $\varnothing 15.2 \text{ mm}$ ($= 150 \text{ mm}^2$) each (Fig. 8). The new tendon is not anchored by a deviation block but by usual anchorage devices. Therefore holes were drilled through the existing deviation block (Fig. 9).

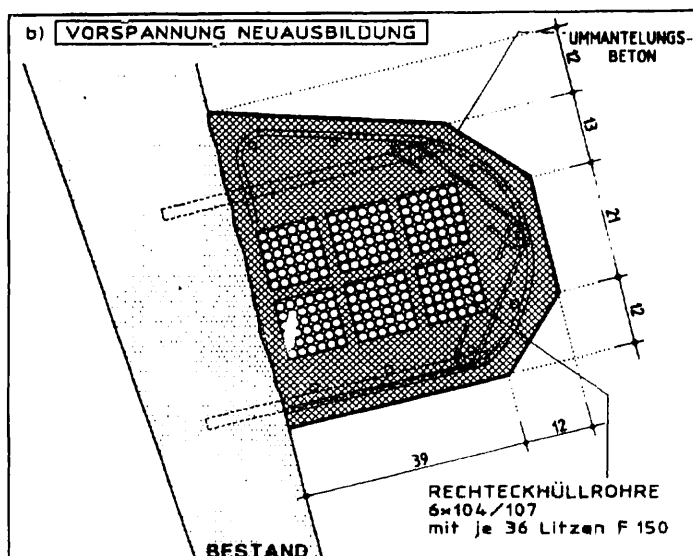


Fig. 8 Cross section of the new tendons

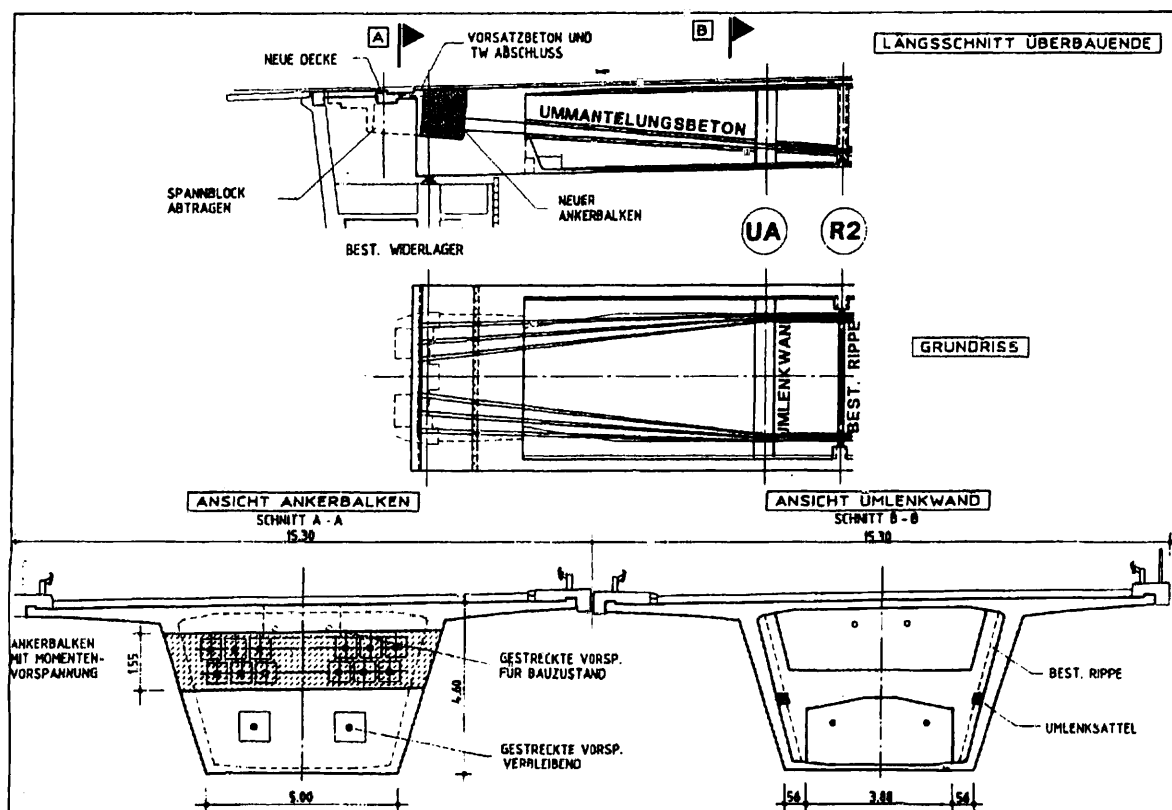


Fig. 9 Layout of the tendons at the support

The strands run from one end of the superstructure to the other. Each layer comprising 6 strands was thread while the rectangular duct was open at the top (Fig. 10). After implementing all 6 layers the duct was closed and grouted. In advance it was successfully tested to grout the special ducts accordingly (Fig. 11).

Numerous original strands of different corrosion degree were tested with respect to residual strength and ductility. The results of this investigation were given in [4].



Fig. 10 Threading of the strands

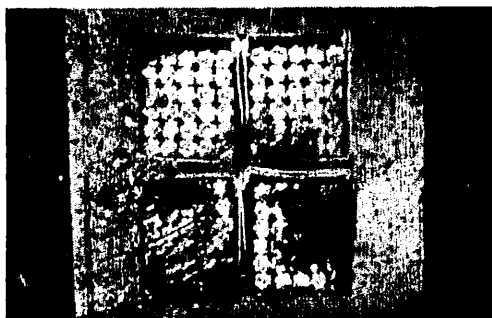


Fig. 11 Cross section of the new tendons after the grouting-test

3 Preventive strengthening and rehabilitation of a box girder bridge [6]

3.1 History of the project and initial situation

Close to the northern end of Lake Attersee near Seewalchen in the Province of Upper Austria, the A1 Motorway crosses the Ager, a river draining the lake, on a viaduct built between 1960 and 1963 according to a design by the Leonhardt-Andrä Engineering Office (Fig. 12). At that time, the bridge constituted a major step in the development of modern bridge-building methods. For the first time, a large superstructure was assembled from transversal, prefabricated elements cast in a stationary on-site factory to a width equal to the cross section [5].

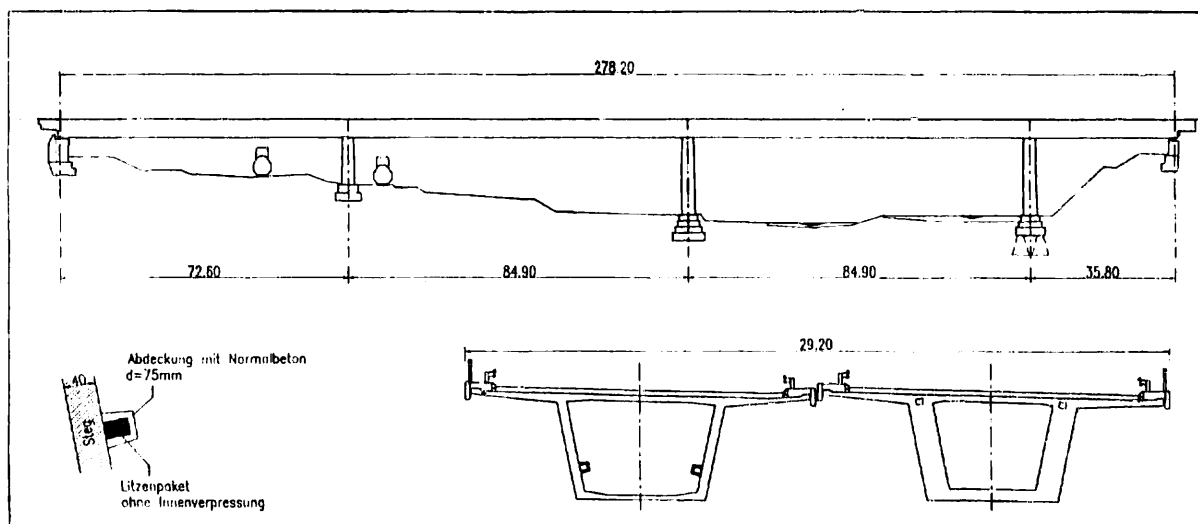


Fig. 12 Existing structure, sideview and cross section.

These elements, prefabricated in a factory in site to a standard length of 8.50 m, were launched onto a continuous falsework at distances of 9.00 m each, the remaining 0.50 m gaps then being closed with in-situ concrete ribs. The addition of in-situ concrete at the bridge ends and special adapted prefabricated elements at the intermediate supports resulted in a continuous superstructure, which was then prestressed by means of a concentrated tendon (Baur-Leonhardt System).

To guide the tendon inside the box girder along the webs, deviators were required, which could be placed in the projecting in-situ concrete ribs between the prefabricated elements. At the bridge ends, the tendon spun from a continuous strand was deviated by 180° via the tensioning blocks. Thus, within the framework of the overall design, the tensioning blocks formed short cantilever beams projecting beyond the end bearings.

Between 1975 and 1976, at a number of tendon high points, the inspection windows dating from the original construction period were opened up for the first time and some of the window fillings were found to be defective and had to be renewed. First signs of moisture entry and incipient corrosion of the prestressing steel were detected. During the 1988 bridge inspection more careful and thorough checks of the tendons were proposed. A report dated May 1989 contains details on the inspection of the tensioning area of the Salzburg-bound lane at the Linz abutment. Although some corroded spots and surfaces were found in the majority of the tendons inspected, pitting or even more severe corrosion was not observed.

During the most recent tendon inspection in 1991, the tendon in the box girder was opened up in eight selected places and visually inspected. It was found that corrosion had occurred in some places. Although even the most seriously corroded areas did not point to a reduction of the static load-bearing function, a loss of ductility and dynamic strength in the places concerned was assumed [4].

The causes of tendon corrosion were found to be manifold:

- Defective sealing of the bridge, which has almost no gradient and had not been provided with a damp-proof drainage system (longitudinal gradient = 0 %, transverse slope 1.5 %).
- According to present-day knowledge, the prestressing steel was insufficiently protected by a single concrete coating.
- Elevated tendon layout above the supports, with the window open during construction and poorly filled afterwards.
- Moisture entry in the tensioning block area, mainly all along the vertical construction joint.

Basically, none of the deficiencies found raised any doubts about the static load-bearing function of the concentrated tendon. However, experience gained during the exchange of tendons at the bridges across the river Inn in Kufstein [7] show that incipient or moderate tendon corrosion cannot be detected at the tendon coating and may therefore remain hidden for a long time.

3.2 Strengthening and rehabilitation programme

Although a preventive exchange of a concentrated tendon involves very high costs, the possibility of a sudden tendon failure, as in the case of Wildbichler Bridge in Kufstein [7], which automatically entails closure of the road to traffic, should be avoided. The principal therefore proposed that the bridge must be strengthened to create a higher load-bearing capacity, which guarantees fulfilment of the standard load-bearing function even in the case of an event comparable to that of Wildbichler Bridge, i.e. the loss of approx. 25 % of the existing prestress over any cross section.

At the same time, the method of strengthening chosen should not make a future tendon exchange impossible, which may become necessary nevertheless, but rather support such a measure in a favourable way.

Additional prestressing appeared to be the most suitable method of preventive strengthening. Given the magnitude of the safety margin to be created, centric prestressing had to be ruled out; the amount of prestressing steel and the increase of centric prestress by approx. 70 % would not have been acceptable. Hence, external, unbonded moment prestressing was chosen, which could easily be accommodated in the box girder.

Moreover, the causes of possible corrosion of the existing tendon had to be effectively counteracted as part of the rehabilitation programme:

- uncovering of the superstructures, corrections and repairs on upper side of superstructure, including the above mentioned tensioning windows;
- micro-cement grouting of the prestressing steel area of the old the tensioning blocks and, in particular, the construction joints at both ends of the superstructure;
- increase of transverse slope to 2.5 %;
- sealing and application of a continuous, high-quality insulating layer;
- repair of outer surfaces of the superstructure ("concrete cosmetics") and improvements in the region of the abutments.

3.3 External prestressing

3.3.1 Design

A trapezoidal tendon layout (longitudinal section) in the main spans, a single deviator in the large end span and no deviation in the small end span corresponded well to the desirable moment diagram (Fig. 13).

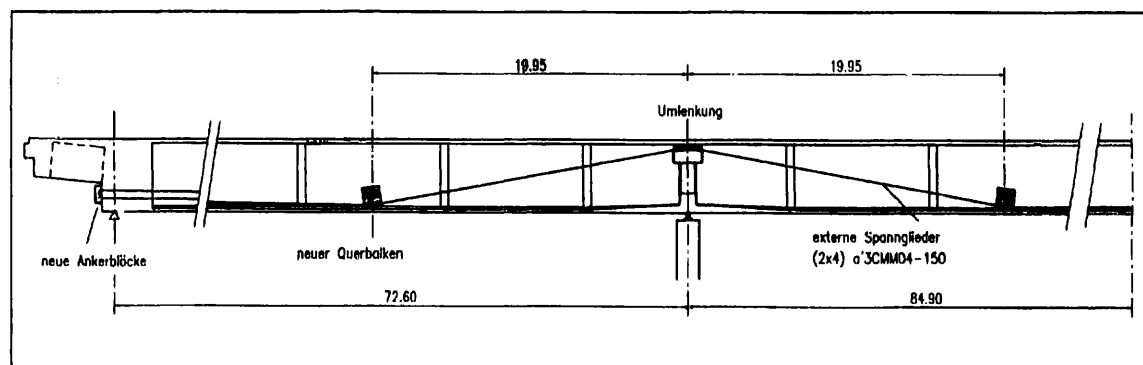


Fig. 13 Layout of additional external tendons

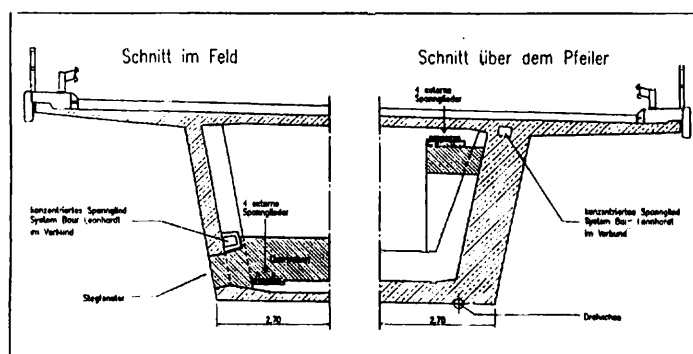


Fig. 14 Deviator in span and at support.

With an inner clearance of the box girder of approx. 3.95 m, approx. 3.50 m were available for eccentric tendon layout (Fig.14). Despite the inclined inner surfaces of the webs, vertical planes were chosen for the tendon layout (Fig.15). This results in a simple geometry, particularly for installation of the deviators, and increases the distance to the web, so that web haunches in the support areas do not cause any problems (Fig.16). For anchorage of the additional prestress at the bridge ends, the very strong cross girders, designed to distribute the tensioning block pressure, were available in the existing structure.

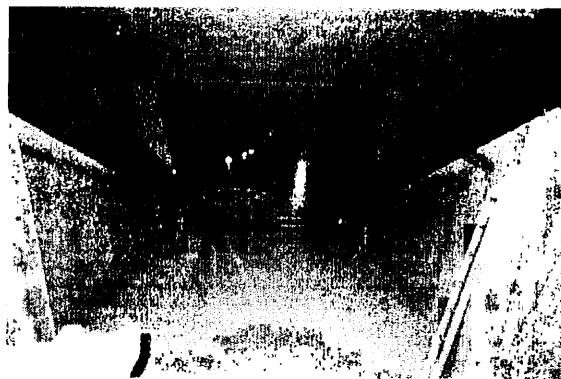


Fig. 15 External tendons running to deviator in span

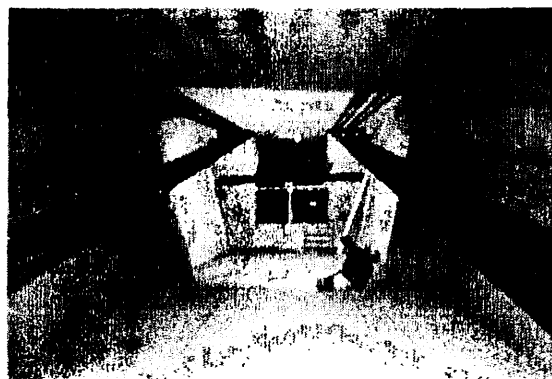


Fig. 16 External tendons close to the deviator at support

There is enough space underneath the tensioning blocks on both bridge ends to accommodate the tensioning anchors for additional prestressing in concrete anchor blocks. As a prerequisite for this type of anchoring, bores had to be sunk into the cross girders in an inclined plane to a very high degree of accuracy to obtain the necessary distribution.

3.3.2 Calculations

To calculate the magnitude of additional prestress required and to establish the service load and the ultimate load in the principal direction of loading bearing, calculations were made on the following assumptions:

- a) Rehabilitated condition (with a slightly heavier bridge deck), existing prestress and traffic load according to present-day standard condition, as the initial situation;
- b) Condition as under a), plus additional prestress;
- c) Condition as under b), but assuming failure of 25 % of the existing prestress;
- d) Condition under rehabilitation: dead loads, existing prestress, removal of pavement, and first stage of additional prestressing.

Calculations according to a) produced an unexpected result, showing that under traffic the superstructure remains fully prestressed, with an adequate load-bearing safety margin under bending moments.

Calculations according to b) showed a considerable safety margin under maximum traffic load: even on the assumption of a temperature gradient of $\Delta T = 10 \text{ K}$, a fully prestressed status was shown to exist in the main direction of load bearing. Under permanent load, the total prestress exceeds the continuous loads.

Calculations according to c) showed similar service stresses as calculations a) and a load-bearing capacity under bending stress well above the requirements of ÖNORM B4250 [8], although the calculations under ultimate load were made without increasing the external tendon forces.

The calculation of service stresses according to d) showed that at any time during removal of the bridge pavement, including pavement removal equipment and loading trucks - a procedure that may again be necessary at some time in the future - the additional prestressing effect must be limited to 40 %, if tensile stresses due to the preponderance of prestress are to be avoided. With the additional prestress being adjustable and/or relievable at any time, this requirement can easily be met also in the future.

According to the results of these calculations, an even smaller amount of prestress would be sufficient for rehabilitation of the bridge. However, this option was not felt to be meaningful, since the additional prestress had to assume a function originally not taken into consideration. In fact, the calculations according to a) had shown clearly that the shear reinforcement in the webs of the box girder was not able to meet the present-day requirements as established by the trussed bridge model. At the time of construction, the usual design method was to cover the main tensile stresses, provided they exceeded certain limits. However, the lack of continuous stirrups of the webs could be largely made up via the additional prestress laid out at a certain inclination near to the support axis.

3.3.3 Tendons

The tendons chosen for additional prestressing were put together from co-extruded, four-strand VT M CMM bands [9]. 3 VT M CMM 04-150 tendons were bundled to form a tendon with a steel cross section of $(3 \times 4 \times 1.50) = 18\text{cm}^2$. Four such tendons were placed for each box girder web. According to the relevant Austrian Standards [10, 11], external tendons may be utilised up to 70 % of their guaranteed tensile strength, which means that a maximum prestressing force of $(2 \times 4) \times 18 \times 177 \times 0.70 = 17,840\text{ kN}$ was available, with a long-term effect - after deduction of relaxation losses, reactivated creep and friction - estimated at 16,600 kN.

The existing, fully effective prestress varies between 55,000 kN and 57,500 kN. Thus, considering a somewhat smaller total eccentricity, the additional prestress corresponds to 25% of the existing prestress, which is the magnitude of prestress to be replaced in the event of failure.

3.3.4 Deviators

Basically, CMM bands, like mono-strands, can be stressed through sliding of the strands in the fixed PE sleeve. If, however, the PE sleeve is neither fixed in concrete nor clamped to the deviators, irregular external sliding may occur on the deviators, if the total force of internal friction in the PE sleeve exceeds the static friction at the deviator. Measurements made at a newly built bridge in Carinthia [12] support the assumption that in the case of external prestressing with CMM bands, the coefficient of internal friction is clearly above the value approved for mono-strands fixed in concrete ($\mu = 0.06$).

However, experience has also shown that special steel and Teflon show a highly favourable sliding behaviour on deviators [7]. In the detailed design, preference was therefore given to deviators with external sliding.

For a package of 3 CMM bands, trough-shaped deviators are required (Fig. 17). To ensure a long useful life and minimum maintenance, the deviators were made of special steel, material No. 1.4571, according to DIN 17440.

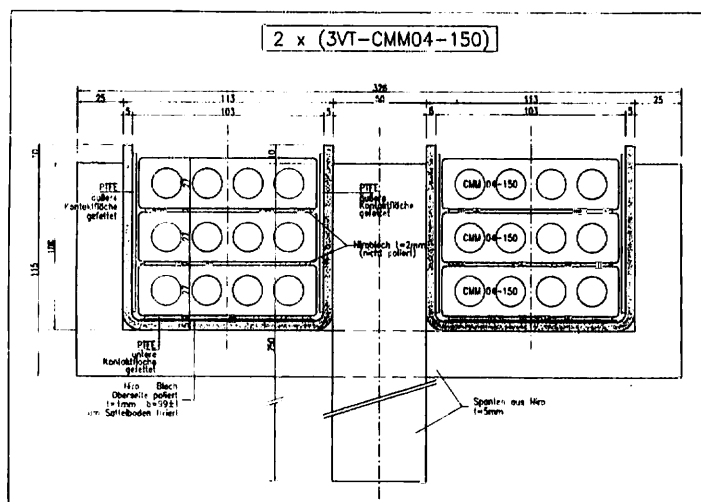


Fig. 17 Twin deviator, cross section.

According to a suggestion made by the supplier (VORSPANN-TECHNIK), the 5 mm sheet troughs were made through folding; the desired deviator radii of $R = 4.50$ m at the supports and $R = 5.00$ m at the span deviations were obtained through rolling. The required narrow tolerances, in particular for the deviator curvature, were accurately met.

The deviators are supplied with a slight overlength on either side to prevent the tendon bundle from resting on the deviator edges in case of slight geometry defects. The deviators were carefully rounded at the intake. Two troughs each were combined into a dimensionally stable unit by means of special-steel plate frames and placed in concrete (Fig. 18). A polished special-steel plate (1 mm thick) is placed on the deviator floor, its sliding partner being silicone-greased Teflon inserts. The three tendon bands resting on the deviator are separated by rough sheet spacers, which help to even out the deviating forces of the individual strands over the band width. Teflon inserts at the sides only serve to prevent undesirable maladjustments.

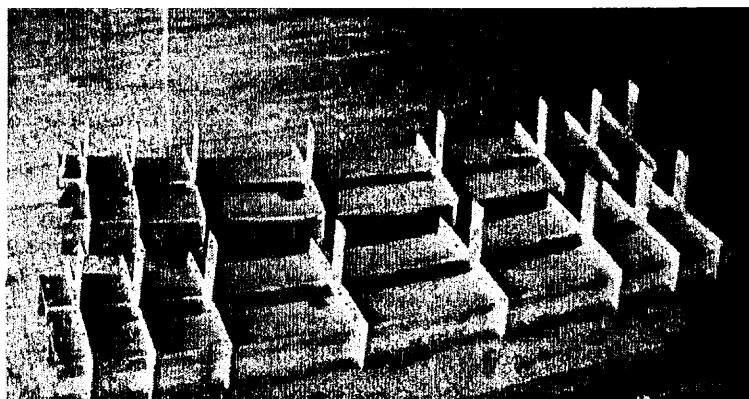


Fig. 18 Deviators before placing.

3.3.5 Deviator elements and anchor blocks

The design of the deviator elements at the supports turned out to be relatively simple. The existing stiffening frame of the box girder cross section was thickened through the addition of toothed parts with bored reinforcements (Fig. 19). A small cross-head supporting the deviator was placed on top longitudinally to the bridge (Fig. 20). The load is transferred directly to the bridge bearing.

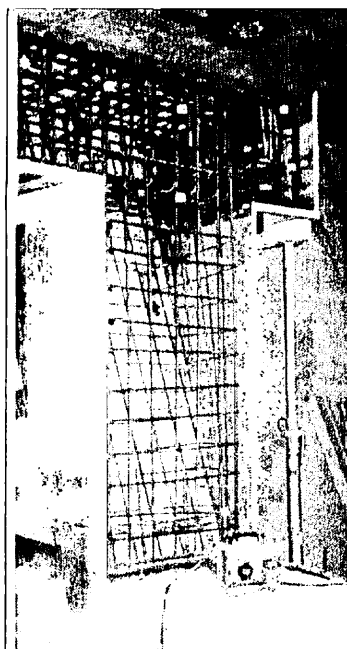


Fig. 19 Reinforcement of deviator at support.

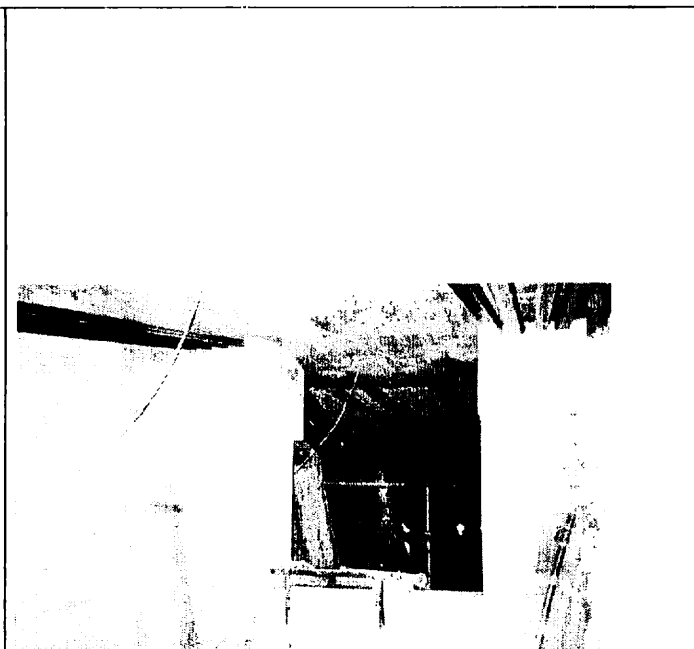


Fig. 20 Deviator at support.

For span deviation, a solution had to be found which permitted a direct transfer of the deviating forces of the tendon into the webs of the box girder. The problem was solved through low-lying web windows for the deviator bar - with the deviators attached to its underside - to connect to. The web windows were cut out from the webs by means of high-pressure water jets, with the reinforcement remaining in place (Fig. 21). This method of force transfer has proved to be ideally suited for this purpose.

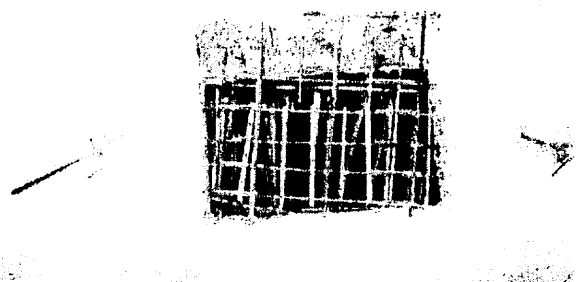


Fig. 21 Opening in web with reinforcement of deviator in span.

The tendons are slid through the above-mentioned, accurately placed bores in the strong cross girders and anchored on the projecting tensioning block. For this purpose, two new anchor blocks were placed to hold the four tendons on each half of the superstructure. Design and manufacture of the blocks, which contained the bursting reinforcement, was relatively simple. However, the further flow of force, i.e. the distribution of the prestressing force to the webs and flanges of the box girder cross section, had to be followed.

This task has to be assumed by the existing end cross girder, in addition to the distribution of forces from the existing tensioning block. It turned out that the cross girders had to be strengthened through the placing and prestressing of vertical rod anchors.

3.3.6 Prestressing work

In accordance with the calculations made, the external tendons were prestressed in two stages: first to 40 %, and then - after a longer period of time - to the full prestress. Given the fact that little practical experience was available with this method of prestressing, and with a view to the deviators with external sliding, a simple measuring programme was incorporated with the stressing procedure. To monitor friction in both prestressing stages, stress was applied unilaterally over the four spans and the force arriving at the other end was measured. Moreover, the tendons were marked at the deviators, so that the sliding displacements on the deviators could be determined after the two stressing stages. The results can be summarised as follows:

- As expected, friction decreases as the prestressing force increases, since the higher deviating pressures on the Teflon sliding layer serve to reduce friction. The measurements agree very well with the assumptions underlying the tensioning sequence. As average values for the entire tendon, the following friction coefficients are obtained:
1st tensioning phase (40 %): $\mu = 0.08$
2nd tensioning phase (100 %): $\mu = 0.06$
- The calculated elongation of the tendons were in agreement with the values obtained in practice.
- The sliding displacement on the deviators in the first tensioning phase differed from the calculated values by approx. 10 %. In the second tensioning phase, the values measured were in better agreement with the theoretical values, which confirms the presence of external sliding.

Because of the favourable friction behaviour, the tendons could be stressed to full length on one side and adjusted to the desired force through stress relief and restressing. Thus, if prestressing jacks are applied on both sides, superstructures of up to 500 m in length and/or more than 180° tendon deviation can be prestressed with unjointed tendons by this method of external prestressing on Teflon deviators. Moreover, the result shows that the concept of external sliding is clearly preferable in both theory and practice.

3.4 Concluding remarks

The principal's pro-active approach, i.e. to strengthen the Ager Bridge through preventive measures rather than wait for failure to occur, is to be welcomed. Strengthening through external prestressing, which could be performed by means of continuous, unjointed tendons on the basis of a suitable deviator design, proved to be adequate both in terms of theoretical calculations and practical construction effort. Turning of the superstructure to increase its transverse slope by means of a novel method proved to be successful and feasible.

The superstructure of the Salzburg-bound lane was rehabilitated in this manner in the course of 1997. The same procedure will be applied to the superstructure of the Vienna-bound lane in 1998.

Use of the innovative method described above was made possible through the open-minded approach of the principal, the Bridge-Building Department of the Provincial Government of Upper Austria. The companies involved, i.e. ERA-BAU AG Salzburg and VORSPANN-TECHNIK Salzburg, contributed a great deal to the success of the project through expedient execution of the contract. The authors of this paper, involved in the project

as design and inspecting engineers, wish to thank all those involved in the construction project for their excellent co-operation.

4 Widening of a bridge deck [13], [14]

The existing structure was built in 1978 with a total breadth of 15.80m. It was designed as a one-lane highway bridge; a second bridge for the lane in the opposite direction was planned for later construction. The bridge consists of a three-span continuous prestressed concrete beam with 36m + 60m + 36m span lengths. A new type of road design, which has a smaller cross-section, enables having a total breadth of 20.00 m for both lanes. This breadth could be realized by extending the existing bridge from 15.50 m to 20.00 m (Fig. 22 and 23).

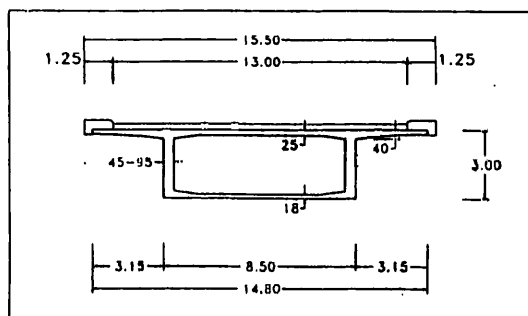


Fig. 22: Existing Cross Section.

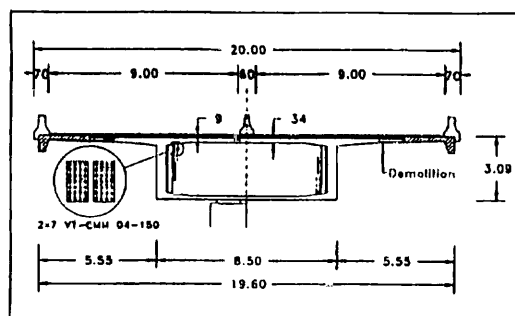


Fig. 23: Widening Cross Section – Longitudinal External Prestressing.

The span length of the cantilever deck slab was increased from 3.15 m to 5.55 m. To control the deflection, the decision was made to use unbonded prestressing with 2 x F 150 monostrands, distance 40 cm, in the deck slab, which runs in the existing part of the slab within a layer of additional concrete 9 cm thick. The amount of prestressing was designed to fulfill for the criterion of cantilever deflection. Under permanent load, no deflections should occur. Under maximum traffic load, the slab will be in a cracked state, where the advantages of using mono-strands lie in the double corrosion protection and avoidance of fretting corrosion.

Strengthening in the longitudinal direction was provided with additional external prestressing CMM-cables (Compact Multi Mono). A set of 4 x 7 VT-CMM 04-150 cables were placed inside the hollow girder, two cables on either side of the web. The cables extend through the entire length of the bridge and are anchored in the end cross-girders. To install the cables, it was necessary to drill holes in the cross-girders at the ends and at the two piers. The total amount of longitudinal prestressing force introduced equalled about 20 MN.

CMM-cables offer several advantages, including good corrosion protection provided by the double PE-sheet and the grease between PE-sheet and the strands, as well as facilitating inspection, following post-tensioning and cable replacement.

Because of the eccentric arrangement of the two piers, there are high torsional moments in the structure. To partially compensate for these torsional moments, external prestressing cables with contrary inclination are placed at the webs (Fig. 24). In this way, deviation forces are produced at the cross-girder of each pier, which in turn produce torsional moment to counteract loading moments.

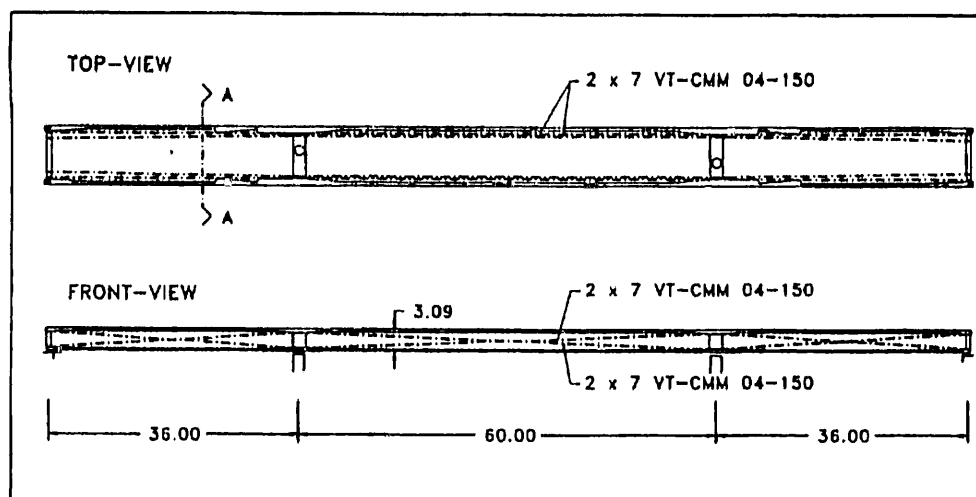


Fig. 24: Longitudinal Additional External Prestressing.

This additional external prestressing allowed limiting the tension stresses in the concrete for servability and obtaining a safe ultimate limit state for the higher loads incurred by widening the bridge.

5 Repair and strengthening of a T-beam bridge [14]

The motorway bridge in this case was built between 1962 and 1964. The superstructure of this prestressed concrete bridge consists of a continuous beam with thirteen spans, the majority of which measure 28 meters in length, with two spanning distances of 41.25 meters (Fig. 25).

A double webbed T-beam system with compression slabs above the supports was selected for the bridge cross-section. The design height of 2.20 meters was coordinated to correspond to the 41-meter spans and was thus more than adequate for the standard spans. As was to be expected, this led to insufficient centric prestressing and high susceptibility to strains. Measurements were carried out according to the load standards valid at the time of construction, whereby full prestressing was foreseen. This degree of prestressing was never attained due to the weight of the pavement layer, and more importantly, because shrinkage and creep losses at the tendons were underestimated, and due to the temperature gradient. Over the years, the bridge showed an increasingly distinct crack pattern, and wide cracks were often evident, particularly at the coupling joints.

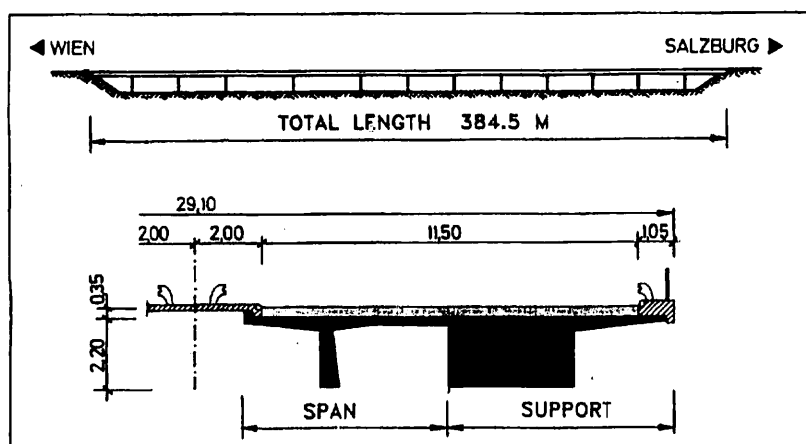


Fig. 25: Side view and Cross-section.

By taking current standards into consideration and applying an intermediate temperature gradient, it was clearly recognizable that the structure suffered from a deficit in prestressing forces. Auxiliary prestressing therefore suggested itself, and subsequent calculations supporting this idea confirmed the suitability of such a measure.

Since the ultimate limit state was sufficient and the structure only required additional prestressed strength under service load conditions, unbonded tendons could be applied, which meant that a very simple engineering solution could be worked out. Much attention was given the question as to whether the profile of the tendons should be matched to the moment line, or whether spanned, axis-parallel tendons should be selected. Ultimately, the decision was made in favor of the straight-line tendon profile, as otherwise friction at the deviation points of the thirteen spans would lead to great losses of prestressing forces in the almost 400-meter-length of tendons. Furthermore, the detailing of straight tendons is simpler, since the saddles must only assume and transfer minimum forces to the supporting structure.

To avoid drilling through numerous cross-beams, tendons were mounted on the outside of the webs. The cross-sectional arrangement of the tendons is shown on Figure 26, and Figures 27 and 28 provide details of the cable cross-section and cable saddles.

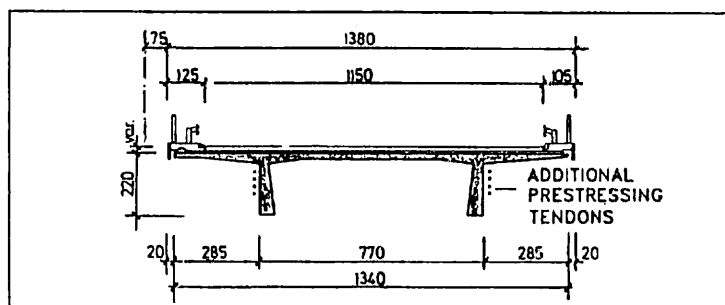


Fig. 26: Cross section with Additional Prestressing

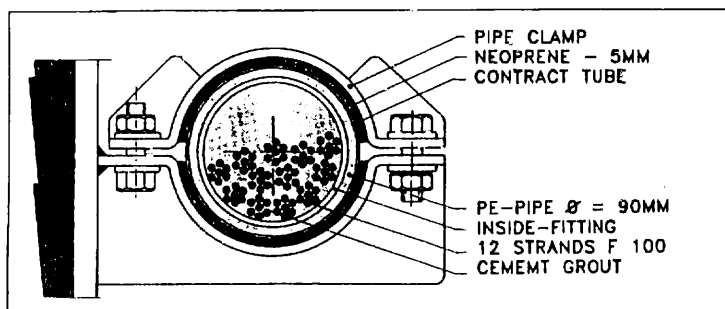


Fig. 27: Cable Cross section; Joint formation and Fastening Clamp

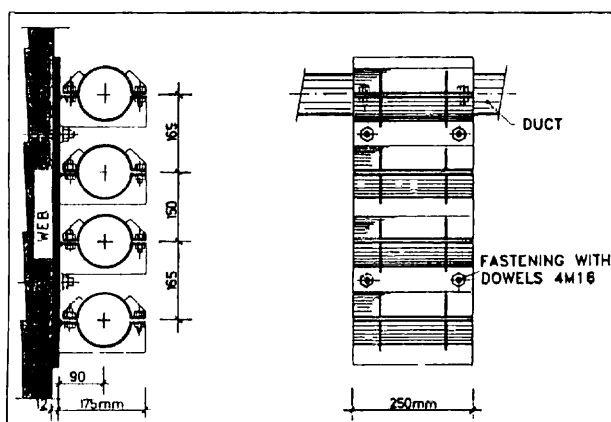


Fig. 28: Standard Cable Saddle

Prestressing forces were transferred to the support structure at the ends of the bridge by embedding the concrete of the anchors in the old, existing concrete. Force transfer was provided with transverse tendons mounted on the specially reinforced end transverse beam. This type of constructional design means that transverse prestressing amounts to approximately half of the longitudinal prestressing (Fig. 29).

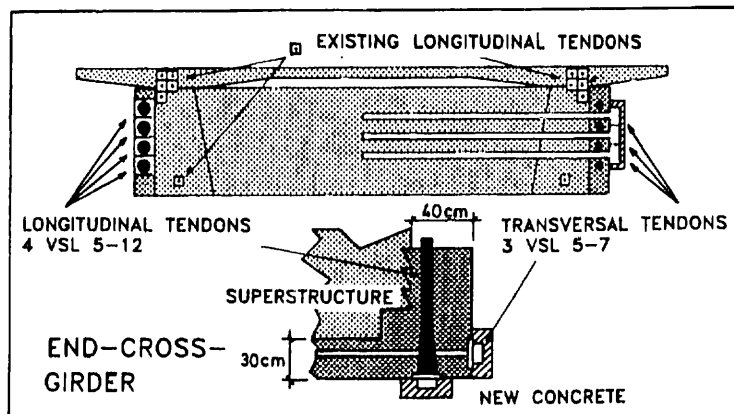


Fig. 29: Anchorage Features

6 Conclusions

In the chapters above specific solutions for particular requirements were given as examples. Of course different solutions were imaginable. Each solution depends on the valuation of the requirements. And it is a compromise between client and designer to find an adequate and economic solution.

From the damages observed in the eighties and nineties we learned our lessons and modified our design criteria, standards and codes of practice. Therefore we may hopefully expect to avoid such damages in future. May be we will have others.

REFERENCES

- [1] Wicke, M.: Ursachen für den Austausch des konzentrierten Spanngliedes der Innbrücken Kufstein. Schriftenreihe des Österreichischen Betonvereins, Heft 26, Seite 9 bis 12, Wien 1996.
- [2] Kirsch, P.: Erneuerung der Längsvorspannung bei den Innbrücken Kufstein. Schriftenreihe des Österreichischen Betonvereins, Heft 26, Seite 58 - 63, Wien 1996.
- [3] Resch, J., Thal, H.: Die neue Vorspannung der Innbrücken Kufstein. Schriftenreihe des Österreichischen Betonvereins, Heft 26, Seite 64 bis 67, Wien 1996.
- [4] Waubke, N., Wicke, M., Paulini, P., Bilewicz, D.: Schnellbeurteilung korrosionsbeanspruchter Spannlitzen. BmfWA Schriftenreihe Straßenforschung, Heft 397, Wien 1994.
- [5] Leonhardt, F.; Baur, W.: Neue Verfahren zur Herstellung weitgespannter, mehrfeldriger Balkenbrücken aus Spannbeton. Beton- u. Stahlbetonbau 57 (1962).
- [6] Wicke, M.; Kirsch, P.: Preventive Strengthening and Rehabilitation of the Ager Bridge. Österreichische Beiträge zum XIII. FIP-Kongress in Amsterdam. Heft 32/Mai 1998 der Schriftenreihe des Österr. Betonvereins, S. 49 bis 55, Wien 1998.
- [7] Wicke, M.; Kirsch, P.; u.a.: Die Wiederherstellung der Innbrücken Kufstein. ÖBV Heft 26/Nov.1996.
- [8] ÖNORM B 4250: Spannbetontragwerke.
- [9] Zulassungsbescheid VT M-CMM Systeme für externe Vorspannung.
- [10] ÖNORM B 4254: Betontragwerke mit Vorspannung ohne Verbund.
- [11] ÖNORM B 4260: Spannbeton - Anforderungen an externe Spannglieder und deren Anwendung.
- [12] Wicke, M.; Fritsche, G.: Reibungsmessungen an Externen Spanngliedern am Bauwerk. Straßenforschung, Heft 464, Wien
- [13] Wicke, M.; Resch, J.; Kirsch, P.: Verbreiterung einer Kastenbrücke unter Einsatz innovativer Techniken (mit Widmung Prof. Bachmann). Bauingenieur 70 (1995), Heft 7/ 8/95, Seite 297 bis 303.
- [14] Wicke, M.; Ostermann, M.: Bridge rehabilitation with external Prestressing. Proceedings of the 3rd International Workshop on Bridge Rehabilitation, Darmstadt, 14. bis 17.Juni 1992; Seite 495

Repair and strengthening of damaged prestressed structures

G. BERTGAGNOLI, V.I. CARBONE, L. GIORDANO, G. MANCINI

Department of Structural Engineering and Geotechnics – Politecnico di Torino

1 Introduction

Post tensioned prestressed r.c. structures extensively built between 1950 and 1985 often show durability problems, especially regarding prestressing tendons. Most of these structures are bridge girders, exposed to severe environment conditions and large amounts of traffic. This question has also a great social account, as it is commonly required to repair and strengthen the structures without stopping the traffic. Operating techniques, often oriented to the addition of extra prestressing tendons, if necessary unbonded, easily let achieve the dual purpose of repairing and strengthening the structure, but need a careful choice of the resisting models, that can sometimes differ from the traditionally coded ones.

Lather on will be dealt some typical examples of prestressed bridge structures repair with the design models adopted for the evaluation of the available resistance after the intervention.

2 Damage causes

Generally the outcoming of severe serviceability problems in postensioned prestressed structures is not due to the only construction methods, but to a concomitance of factors whose main are here listed.

- Not fully satisfactory structure's conceptual design:
 - Road bridges with many isostatic spans, and therefore a high number of joints and structural discontinuities, built in geographic areas where de-icing salts are commonly used for many months each year.
 - Poor attention to the reflux liquids gathering and drainage from the girder.
 - Tendons anchorages localization in percolation hazardous regions, near construction joints, or structure extrados (top slab anchorages).
 - Unfulfilled minimum distance between sheaths and/or ordinary reinforcement, with consequent casting troubles and coming up of nests.
 - Excessively small web thickness compared to sheaths' dimensions.
 - Inadequate sheaths distances from the casting surface.
- Slightly incorrect construction methods:
 - Grouting mortars unfit for composition and water-cement ratio.
 - Wrong grouting method and consequent partially unfilled sheaths.
 - Scarce tendon anchorage protection.

It is therefore clear that the durability of postensioned prestressed structures is not only due to the construction techniques and related materials, but also to the structure's conceptual design and constructive details.

3 Typical repair and strengthening cases

3.1 Rio Berceto viaduct (fig. 1, 2, 3)

This is a motorway bridge made of 8 isostatic 41 m spans built in the early '70 and placed in an Appennini mountain pass zone. During a routine by-bridge inspection was reported a concrete coating expulsion near the mid span of the beams in the inner part of the curve. At more careful inspection, taking away the cover, made it possible to see that 3 of the 6 prestressing tendons of each beam, horizontally equispaced in the lower chord, were completely corroded for a few meters length.

There were several causes of corrosion. The strands were incorrectly placed: the distances between strands and strand and beam surface were too small. The concrete matrix was quite porous. The beams were permanently washed out by rich chloride waters coming from the girder and not efficiently taken away by the drainpipes. A throughout analysis of the beam safety in their originary conditions has shown these lacks with regard to the present code loads:

Slight flexional deficiencies in the middle section

Heavy shear weakness both on the stirrups, for a significant region of the web, and on the lower chord in the end zones.

No use to say that at this level of corrosion, that interested the half of the prestressing tendons, the safety margins were completely missed both in the serviceability and at the ultimate conditions. To face the double lack of reinforcement, in flexion and shear, it was decided to employ external prestressing, using two 12 x 0.6" ϕ deviated tendons in each beam in order to supply to the lack of flexional and shear strenght and two 4 x 0.6" ϕ straight tendons to achieve an extra flexional resistance and increase the strength of the lower chord in the support area.

The external tendons anchorage at the ends of the beam and the deviation of the bigger ones has been achieved adding extremity concrete blocks and intermediate deviators connected to the beams by transversal prestressing realized by means of bars. Intermediate brackets coated with neoprene have been introduced to dump the tendons traffic induced vibrations and to hold them back from second order negative effects induced by the great deflection of the beam expected at the ultimate limit state. External tendons have been shielded with a HDPE sheath filled with mortar, also used to protect the bars.

The action made it possible at the same time to repair the beams and rise their resistance regard to the current variable loads, that are nowadays higher than the law prescribed ones at the time of the construction.

3.2 Via Emilia Crossover (fig.4, 5, 6)

It is a bridge over a major road made of isostatic girders with 15,2 m spans, built in the early '70. In this case the degradation process has begun twenty years after the construction further to the collision of a truck, that was coming along the underlying road carrying a ditcher and reached a height greater than the allowed one. Middle span beams were cast using a pretensioning method and in the impact they were severely damaged as some strands were cut apart and the remaining ones were left uncoated and exposed to the chemical aggression due to traffic pollution. A few years later have come out worrying deterioration outbreaks as

complete corrosion and breaking of every strand in the lower level of the beam.

It was reached the decision of replacing the prestressing related to these strands with external prestressing made of bars on each side of the beams. Placing quite short straight bars in the middle section of the beams was enough as no other lacks came out from the analysis of the bearing capacity of the beams. The anchorage was performed in concrete blocks linked to the web using transversal prestressing whose bars were shielded with a HDPE sheath filled with mortar to achieve a good protection degree.

3.3 Vizzana - Zampogna Viaduct (fig. 7, 8, 9, 10)

This is also an isostatic 24 m long spans bridge dating to the middle '70s in an Appennini crossing motorway. In this case the beams have the feature of being cast in factory using pre tension, but prestressing was completed after the slab casting with a post tensed tendon whose end anchorages were set in the top slab. Practically, in order to put in tension the tendons, was left a socket in the slab that was supposed to be sealed with a second casting after the sheaths grouting.

A recent inspection reported some wide longitudinal cracks along the extreme region of the post tensed tendon in the inner bend beam and somewhere the web explosion for considerable lengths down the tendon way. A more careful survey revealed that the tendons were completely corroded and the sheaths were almost empty. The sealing cast of the socket in the slab was made with bad quality concrete and loosely connected to the rest of the structure by the inlaying reinforcement. The sheaths grouting was also completely forgotten. As the waterproofing sheet suffered from age deterioration, rich chloride water leaked in the sheath through the tendon head. The plies corrosion and subsequent expansion made the web burst as it was too thin to hold such a big sheath and short of transversal reinforcement.

With the choosen solution the post tensed tendon was replaced with exterior prestressing bars connected to the webs as seen before and the damaged webs and slab sockets were locally repaired. The mending was kept to the bare minimum because the work supposed running period is the only next five years as a basic track change is to be completed.

3.4 Barcalesa Viaduct (fig. 11)

This bridge, that is quite similar to the previous ones, needed a strengthening work on the beams because of deficiency of shear resistance in a close section of the span as the new law loads are greater than the design ones. The lack was revealed from the presence of wide and spread shear cracks and was successively checked by a numerical test. It was then introduced a vertical web prestressing by exterior bars fastened in the slab and in the beams lower chord. This external prestressing was later protected with a concrete mix grouting.

This solution presented some modelling troubles overcome as explained in point 4.

4. Resisting models for repair and strengthening

It is useful, first of all, pinpointing that using external tendons to repair and strengthen structures, even if can often be easy and safe, needs to employ non conventional resisting models, as there's no bond between steel and concrete. It is well known in bibliography and in the latest design codes (1) that the increase of the prestressing strength in the external tendons

is, at the ultimate limit state, quite small. Without using more efficient non-linear analysis it can be taken as the 5% of its value after losses.

The ultimate safety coefficient evaluation for longitudinal internal actions is then commonly performed by means of a bending-compression verification; the external tendon is thus considered as a constant acting force, which value is $P_d = 1,05 \gamma_p P_{k\infty}$. Therefore, to satisfy this verification, it is generally necessary a bigger external prestressing than internal one.

In order to verify the ultimate limit state there'll be not neglectable residual compression stresses (1÷2 Mpa) in the serviceability conditions. That has a fair effect on the cracking limit state, that is automatically satisfied, on the fatigue resistance, that is not implied, and on durability, because there're no tension stresses at the edges. Clearly the resisting shear given by the external prestressing shall be evaluated with the same approach, that is taking into account the presence of constant concentrated forces of $1,05 \gamma_p P_{k\infty} \sin \alpha$, where α is the tendon deviation by the horizontal.

The prestressing strength needed to guarantee the connection between the new heads and/or deviators of the external tendons and old concrete can be determined using the shear joints design rules (2). The tangent resistant stress can be written as:

$$\tau_{rd} = \beta f_{ctd} + \mu \sigma_{cd} \leq 0,25 f_{cd}$$

Even if it's possible to roughen the beam surface before casting, clearing away the superficial concrete latex, the following values of β and μ shall be used as a cautionary measure to avoid sensible slips at the prestressing setting up.

$$\beta = 0 \quad ; \quad \mu = 0.6$$

The choice of $\beta = 0$ takes into account the differential shrinkage between old and new concrete and the small eccentricity of the anchorage strength regard to the sliding surface. In conclusion it's suggested for the anchor blocks a $1,67 A_p f_{ptk}$ transversal clamp strength.

The resisting model formulation becomes more difficult when subvertical bars are used to improve the webs shear resistance. In this case the variable inclination stress fields model can also not be useful as there's no bond between steel and concrete and consequently it's impossible to increase the tension in the bars at the ultimate limit state. It's then necessary to use a twodimensional model, where a web region is seen as a wall subjected to membrane actions, whose vertical component comes from the added web prestressing with constant strength $P_d = \gamma_p P_{k\infty}$.

The safety verification can be carried out according to some recent operative proposals (3) (4) (5) (6), that provide almost the same result in the definition of resistant domains v , n_x , n_y .

5. Conclusions

The case histories shown in the previous paragraphs clearly allows to highlight that the pathology affecting posttensed prestressed structures always have heterogeneous causes, not condensable to the only inadequate construction procedure. The repair and strenghtening of those structures can however be carried out easily, even without stopping traffic, using external prestressing tendons, also achieving clear advantages in the possibility of time monitoring the structural safety.

A great deal of attention should be paid to the adoption of design models suitable for the description of the structural behaviour in fields not always covered by traditional ones.

References

1. Eurocode 2: Design of concrete structures – Part 1 – General Rules and rules for bridges, 2nd draft, January 2001
2. C.E.B. – F.I.P. Model code 1990, Thomas Telford Editor, 1993
3. Marti, P., Alvarez, M., Kaufman, W., Sigrist, V., Tension chord model for structural concrete, Structural Engineering International, n° 04/98, 1998
4. Kaufmann, W., Marti, P., Structural Concrete: Cracked membrane model, Journal of Structural Engineering, ASCE, December 1998
5. Carbone, V.I., Giordano, L., Mancini, G., Resisting Model for r.c. and p.c. panels, Accademia delle Scienze di Torino, Atti Scienze Fisiche, n° 134, 2000
6. Carbone, V.I., Giordano, L., Mancini G., Design of r.c. membrane elements, accepted for publication on Fib Journal, December 2001



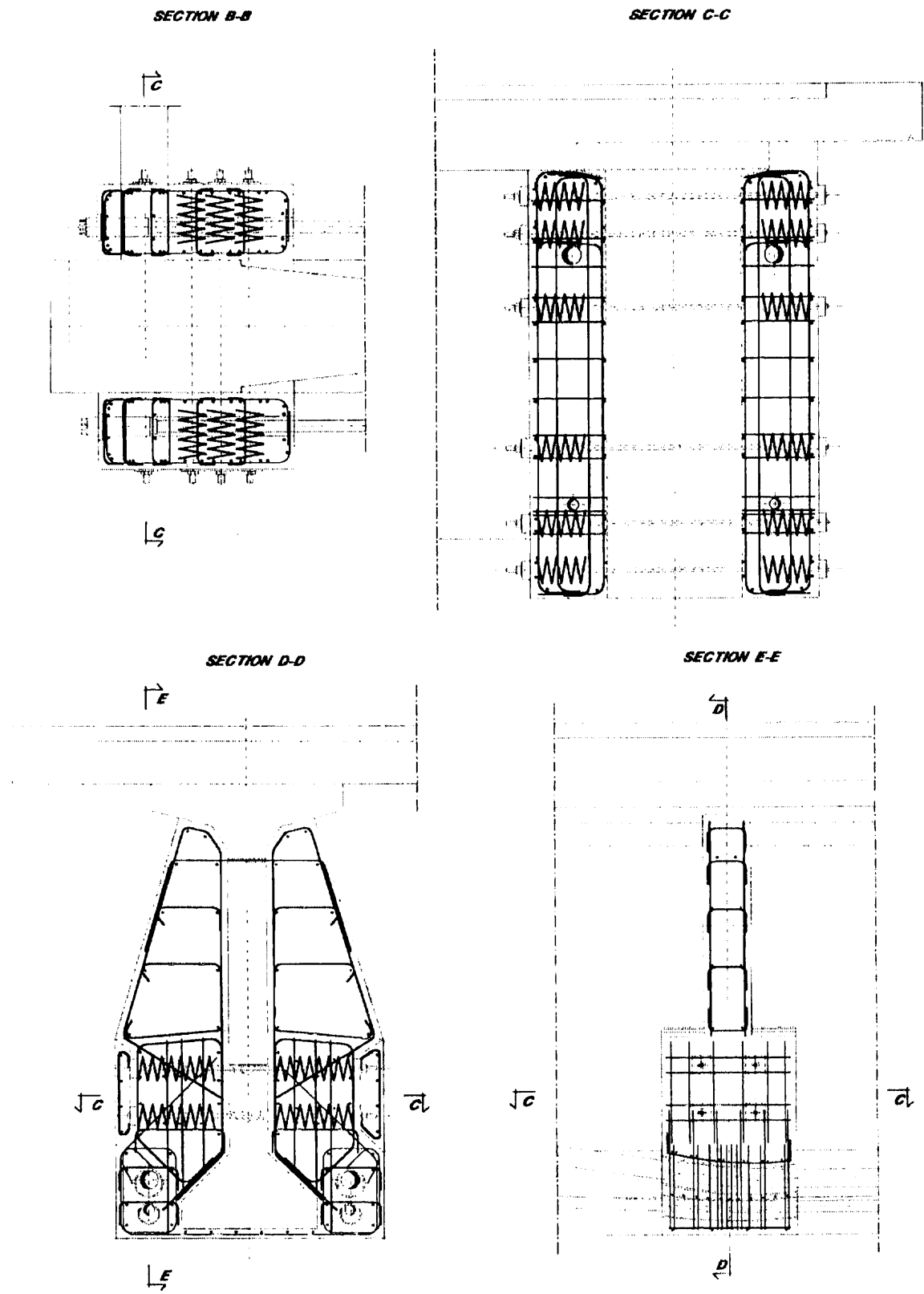
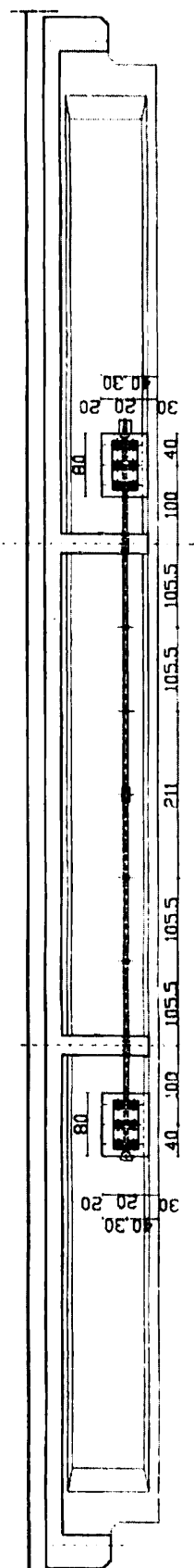


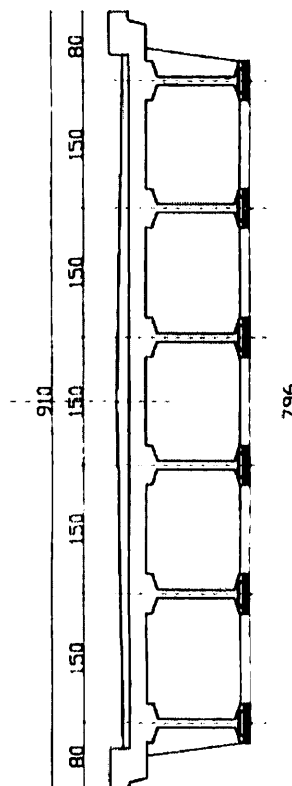
Figure 2



Figure 3



SECTION D-D



fib Bulletin 15: Durability of post-tensioning tendons

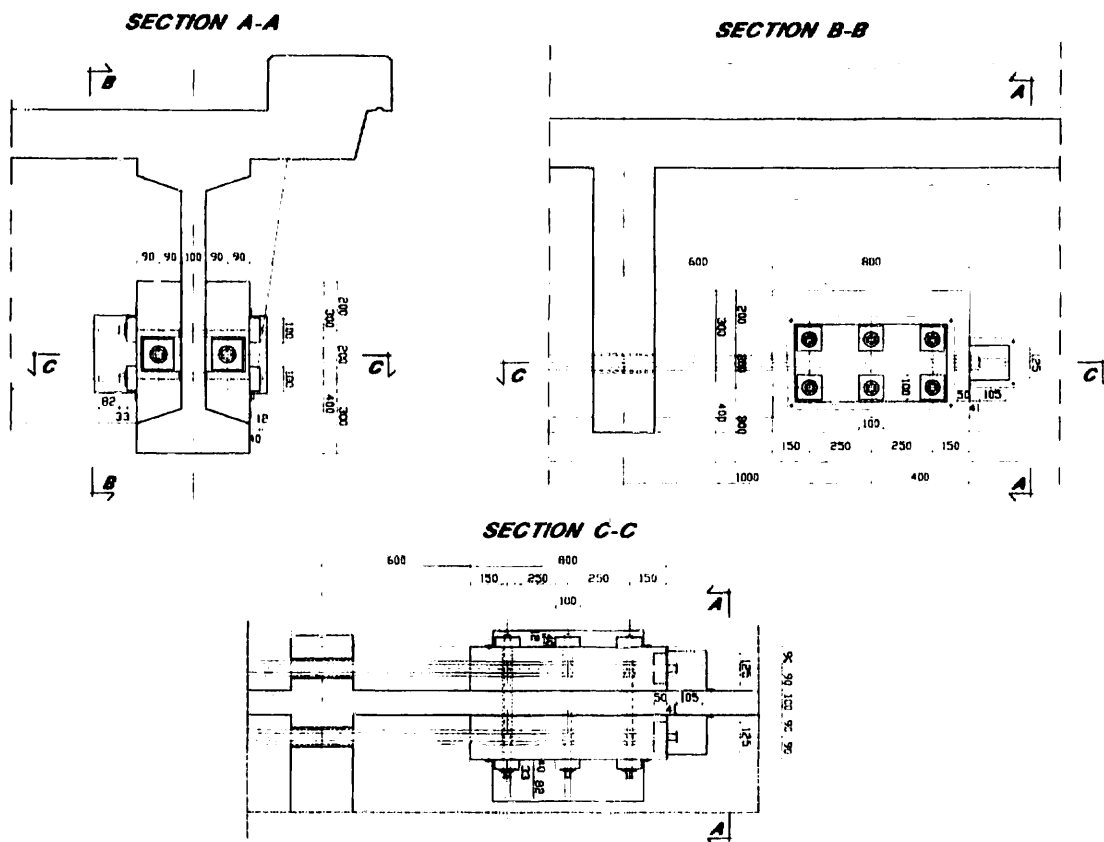


Figure 5



Figure 6

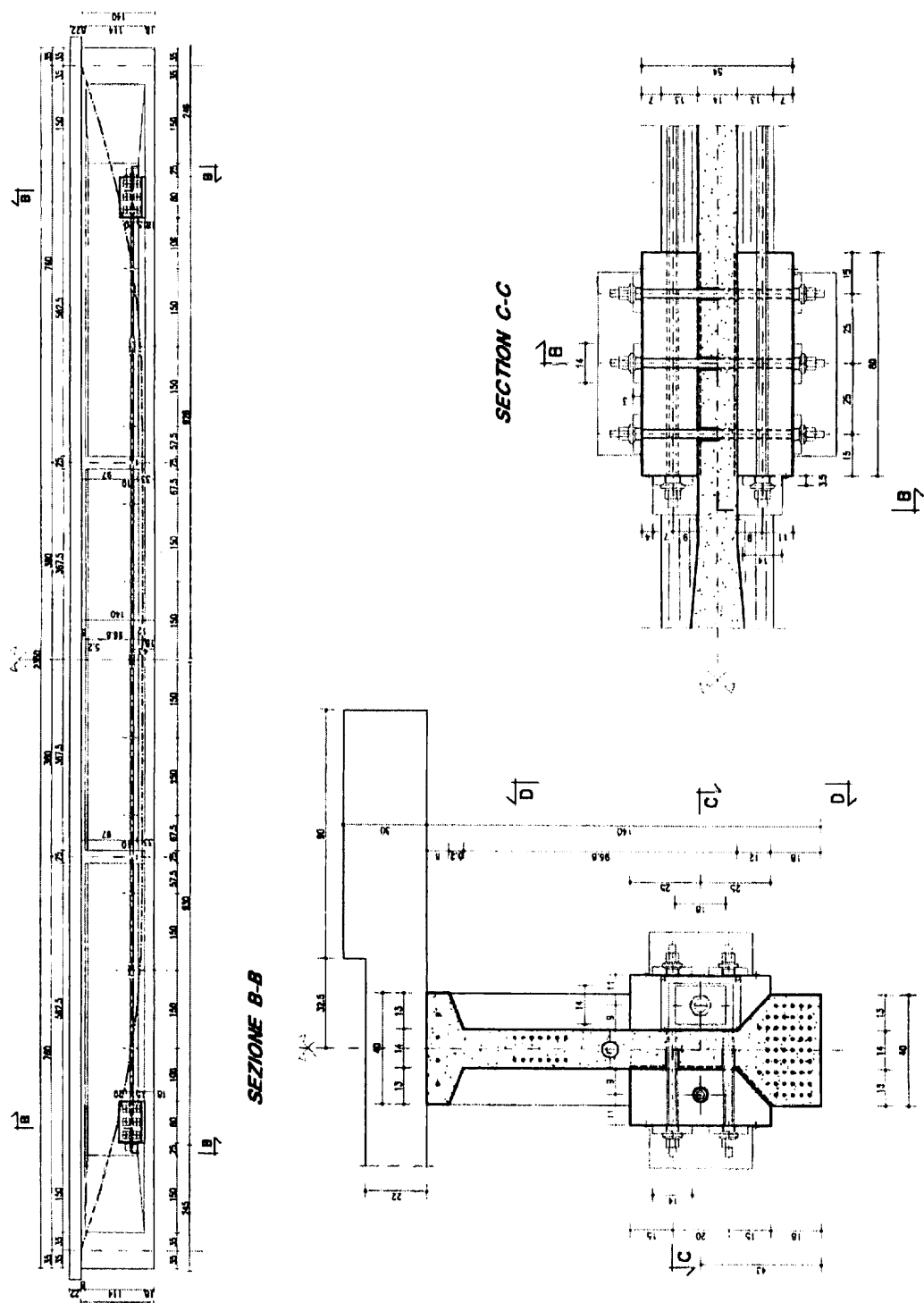


Figure 7

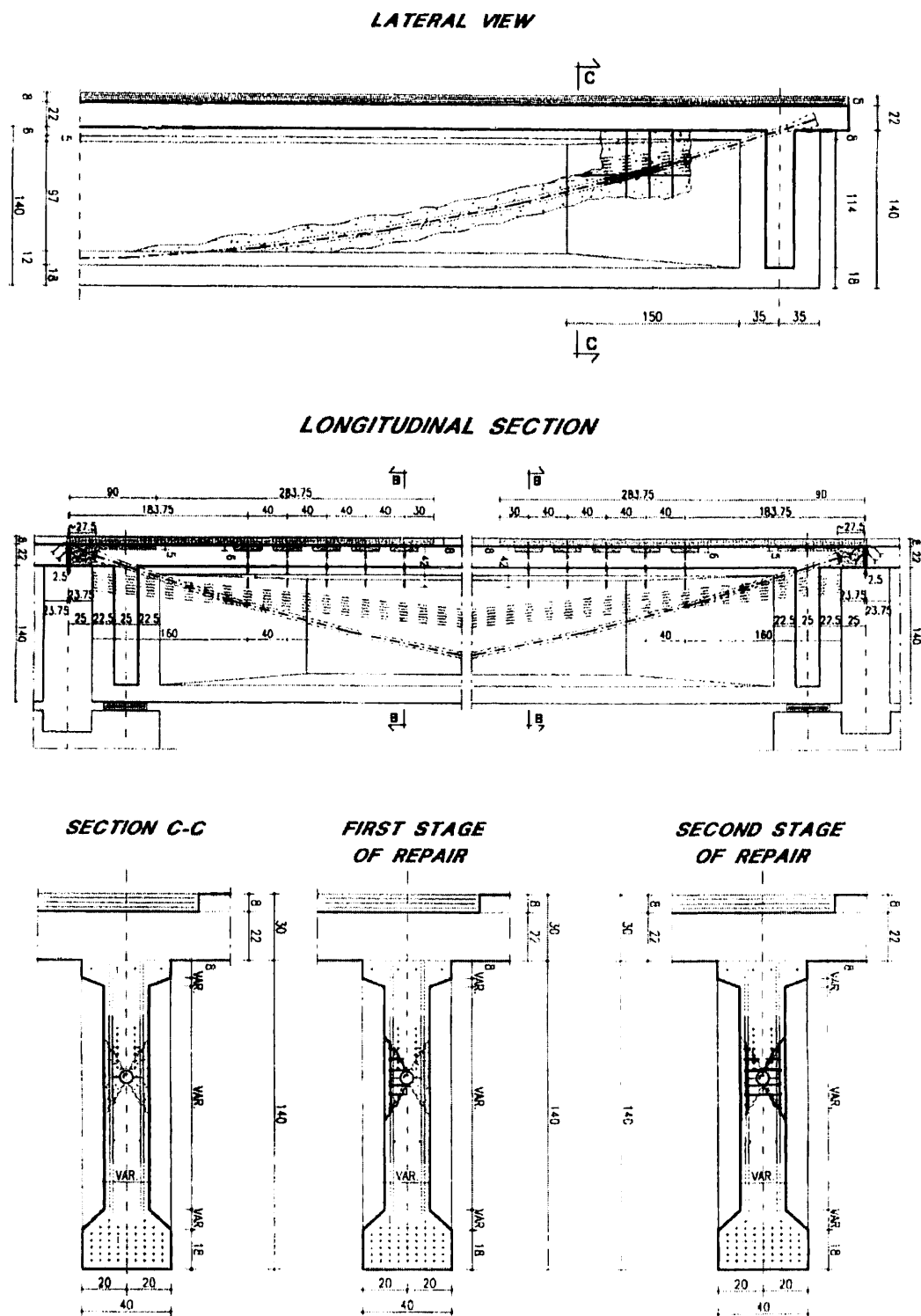


Figure 8

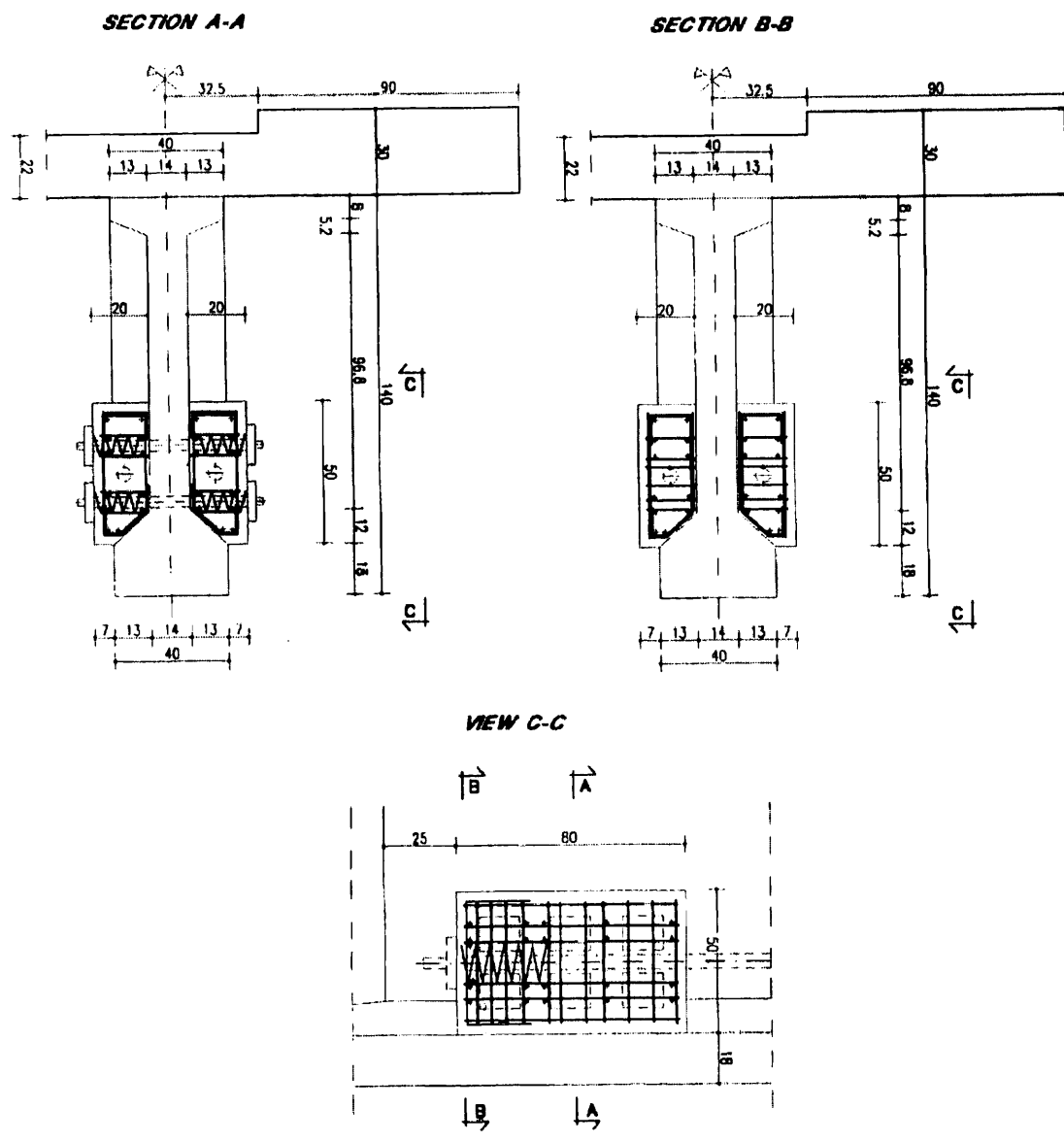
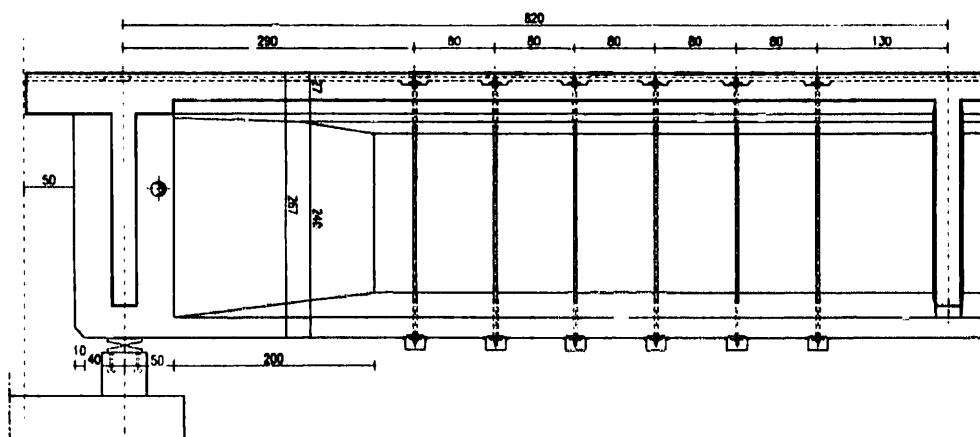


Figure 9

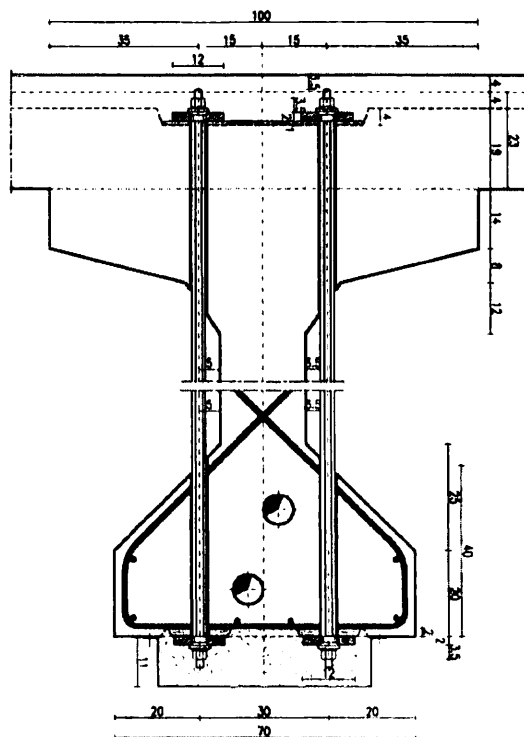


Figure 10

LATERAL VIEW



DETAIL 1



DETAIL 2

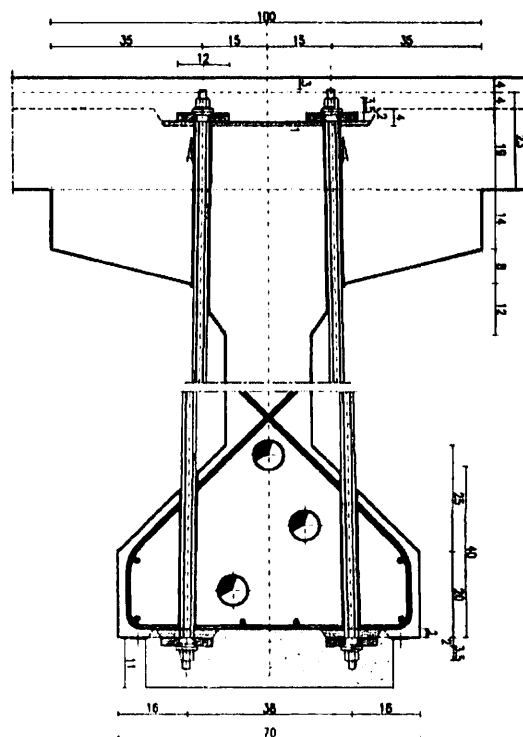


Figure 11

Evolution of Prestressing Systems

H.R. GANZ

VSL International Ltd., Lyssach/Switzerland

1. Introduction

The idea of prestressed concrete goes back more than 100 years. Modern development of prestressed concrete however, is attributed to Eugene Freyssinet of France who started using high strength steel wires for post-tensioning concrete beams in 1928. By 1939, he had designed the Freyssinet system with conical wedges for anchoring $\varnothing$ 5mm wires at the ends of prestressed members as well as special jacks for use in stressing and anchoring the wires. From the 1940's onwards he designed and built a significant number of impressive post-tensioned structures including the famous bridge over the river Marne in Lucancy. Other engineers contributed significantly to the development of post-tensioned concrete and prestressing systems such as F. Dischinger with the introduction of external post-tensioning. Dyckerhoff & Widmann developed a $\varnothing$ 25mm stressbar system (Dywidag system). Leonhardt & Bauer in Germany and Magnel in Belgium introduced their wire prestressing systems. In the second half of the 1940's the Swiss engineers Birkenmaier, Brandestini, Ros and Vogt created the BBRV prestressing system for $\varnothing$ 7mm wires anchored in a steel anchor head by cold formed button heads. This early period after World War II brought a rapid development, and in 1953 some 25 different post-tensioning systems were known, two third of them in Germany, the remaining in Europe except for two systems in the USA, see Müller¹⁾. Only relatively few of those early systems survived the introductory stage, and actually are available still today.

The early 1950's saw the creation of the International Federation of Prestressing (FIP) who subsequently contributed significantly to the development of prestressed concrete through the preparation of recommendations and guides for prestressing materials and systems, and design and construction of prestressed structures.

The 1960's brought the introduction of 7-wire strand prestressing systems for monostrand tendons, particularly in the USA, and for multistrand tendons, e.g. VSL post-tensioning system.

Post-tensioned tendon forces were limited initially to about 400 kN but increased rapidly in the late 1950's and 1960's into the several thousands kN range. Today, the maximum tendon capacity typically found in post-tensioning system approvals is in the 10,000 - 15,000 kN range.

Today's market size for post-tensioning steel is estimated to 600,000 - 800,000 tonnes/year. The USA alone contribute about 100,000 tonnes. Between 50-75% of this steel goes into bridge construction however, a significant part in the order of 25% is used in buildings. Other typical applications for post-tensioning tendons include engineering structures such as reservoirs, silos, offshore structures, containment structures, etc., and slab on ground. For reference only, the pretensioning steel consumption is estimated at 3-5 times the size of the post-tensioning market.

Most of the prestressed structures built in these first 50 years in accordance with the rules for good design, detailing, and practice of execution have demonstrated an excellent durability of prestressed concrete, in general, and of prestressing tendons in particular. The FIP Recommendations²⁾ for corrosion protection of prestressing steels state that "It must be emphasised that instances of serious corrosion in prestressed concrete structures are rare when one considers the volume of prestressing steels that have been consumed world wide over the years".

The prestressing / post-tensioning systems have continuously evolved since their introduction about 50 years ago. This paper gives a summary of today's status and recent evolutions considered important for the performance and durability of these systems. The emphasis will be on improved materials, products, specifications, and procedures for post-tensioning tendons. While similar in many aspects, ground anchors, stay cables, and pretensioning are not considered in this paper.

2. Post-tensioning system components

2.1 Prestressing steels for post-tensioning systems

Today three main forms of prestressing steels are used for post-tensioning:

- Cold drawn plain wire of 5-7 mm nominal diameter, and specified tensile strength of 1670-1860 MPa.
- Cold drawn plain prestressing strand of 13-16 mm nominal diameter, and specified tensile strength of 1770 - 1860 MPa.
- Hot rolled plain or ribbed prestressing bars of 15-40 mm nominal diameter, and specified tensile strength of 1030 - 1230 MPa.

The most important steel for use in post-tensioning is the 7-wire prestressing strand. The three types of prestressing steel are available as uncoated, bare steel or with metallic and non-metallic coatings applied in the factory. The 7-wire strand is also available with plastic sheathing.

2.1.1 Uncoated, bare prestressing steel

These steels have been standardised since long, and reference is often made to ASTM, British, Euronorm, or Japanese standards, see Table 1. These standards assure high quality prestressing steels. The following is a brief summary of the main relevant characteristics of the prestressing steels for post-tensioning, and recent evolutions.

Type of steel	ASTM Standards	British Standards	Euronorm Standards	Japanese Standards
▪ Cold drawn wire	A 421	BS 5896	Draft EN 10138-2	JIS G 3536
▪ 7-wire strand	A 416	BS 5896	Draft EN 10138-3	JIS G 3536
▪ Hot rolled bar	A 722	BS 4486	Draft EN 10138-4	JIS G 3109

Table 1: Frequently referenced standards for prestressing steel

- (1) Geometry of prestressing steel: This is usually defined with a nominal diameter, and nominal cross sectional area. Important for use in post-tensioning systems are the tolerances in cross sectional dimensions. Unfortunately, some standards such as ASTM A416 still permit quite large tolerances which may cause problems with the behaviour of anchorages during stressing operations, e.g. excessive friction or scrapping of steel. Recent standards such as European EN 10138 permit a tolerance of $\pm 2\%$ on the mass of the steel which avoids such problems.
- (2) Specified strength of prestressing steel: This is usually defined with a characteristic breaking load, and a characteristic proof load (0.1 - 0.2 % permanent elongation). Drafts of EN 10138 now also include an upper limit of the maximum breaking load to avoid excessive cold working of the steel and consequential loss of ductility. This limit is about 15% above the characteristic breaking load.
- (3) Ductility of prestressing steel: This is usually defined with a minimum elongation of 3.5% of the steel at maximum load over a given specimen length. In addition to the minimum elongation, the steel shall break in a ductile manner with corresponding reduction of area, visible to the unaided eye. For wire, the reverse bend test has proven to be practical and reliable. A minimum number of bends is specified around a given bend radius. A corresponding test has been developed for 7-wire strand by FIP and IPTI, these are the deflected tensile test and one-pin test, respectively. This test has recently found its way into standards. This test limits the reduction of strength of a strand deviated around a specified mandrel to 28% for post-tensioning applications.
- (4) Relaxation of prestressing steel: This is usually defined as a percentage loss of force of a prestressing steel held at constant elongation over 1000 hours, initially stressed to 70% or 80% of its specified strength. While in the past several relaxation classes did exist, the trend is clearly to one class for very low relaxation only for all products, e.g. 2.5% for 7-wire strand in 1000 hours when initially stressed to 70% of the characteristic strength.
- (5) Fatigue strength of prestressing steel: This is usually defined as a stress range which can be sustained by the steel without failure over 2×10^6 cycles at an upper load corresponding to 70% of the characteristic breaking load. Specified fatigue stress ranges are in the order of 180-200 MPa.

- (6) Stress corrosion test for prestressing steel: This test shall confirm that a particular type of prestressing steel is sufficiently resistant to stress corrosion. Different test methods have been proposed and used. The test method most commonly referred to is the one developed by FIP in which the prestressing steel is immersed under tension into an ammonium thiocyanate solution. A minimum time of exposure before failure is specified for each type of prestressing steel. Since the test results may show significant scatter, a minimum and average value is specified. These values are in the order of 1.5 - 5 hours for wire and strand, but are significantly larger for bars with large diameter.

The FIP test has been developed and is well adapted for cold drawn wire and strand. However, there is some doubt of its suitability for other types of steel such as quenched and tempered prestressing wire. Other procedures such as the DIBT-test in Germany, and the test in distilled water in France are also used.

Cold drawn wire, 7-wire strand, and hot rolled bar which comply with standards including the above characteristics (1) to (6) can generally be expected to demonstrate properties suitable for post-tensioning, and good durability. Other types of prestressing steel not covered by the above should be considered with caution before using for post-tensioning. This comment applies e.g. for quenched and tempered wire which is widely used in pretensioning in Japan, but which is not recommended for use in post-tensioning, see FIP Notes³¹.

2.1.2 Coated prestressing steel

Mainly three types of coatings have been used. These are zinc, zinc and aluminium, and epoxy. Detailed information on these methods of protection of prestressing steel can be found in a fib report⁴¹.

2.1.2.1 Zinc coating

Zinc coated prestressing steel is relatively rarely used for post-tensioning. Its main use is in stay cables. However, it has occasionally been used, e.g. for temporary applications over extended periods of time.

Zinc provides a sacrificial protection of the prestressing steel. Corrosion protection by zinc depends primarily on the consumption rate and the available thickness of the zinc layer.

Galvanised prestressing steels have been standardised in the French standard NF A 35035. It is understood that a European standard and a ISO standard are being prepared on galvanised prestressing steel for prestressing. These standards have been specifically written for post-tensioning applications and include e.g. tolerances on the thickness of the zinc layer. Final drawing of the coated wires permits to achieve relatively tight tolerances on the coating thickness. This is important for the performance of wedge anchorages in particular. There are other standards for zinc coated steel but these have not been written for prestressing steel specifically, and should be used with caution.

Prestressing steels may be galvanised by a number of processes. The safest process to avoid the risk of hydrogen embrittlement is hot dip galvanisation, see FIP Notes⁵¹. According to the same reference, cold drawn wire and strand is the least sensitive steel to hydrogen embrittlement.

2.1.2.2 Zinc and aluminium coating

The same comment as for zinc coating applies to the use of this product in post-tensioning. Coatings of zinc containing approximately 5% aluminium appeared in the 1980's. This type of coating provides an improved protection to the prestressing steel compared to pure zinc, as determined in accelerated salt spray tests. In addition, it has a higher hardness than pure zinc which is seen as an advantage for anchorage by wedge.

2.1.2.3 Epoxy coating

Epoxy coated prestressing steel has been mainly applied for pretensioning applications. However, some applications for temporary use, and for permanent use as ungrouted external post-tensioning have been reported, e.g. in Japan.

Experience has shown that epoxy coated strand needs to have the interstices between wires completely filled for long term corrosion protection. The thickness of coating needs to be carefully controlled for reliable anchorage by wedges. There is some discussion whether the coating thickness needed for protection, and the corresponding tolerances, make this product suitable for post-tensioning anchored by wedge. ASTM has issued a standard A822 for epoxy coated strand.

2.1.3 Sheathed prestressing steel

Prestressing strand protected with a layer of grease and an extruded HDPE sheathing has been used for several decades primarily as monostrand for post-tensioning of building slabs. Specifications for this type of tendon have been introduced by the Post-Tensioning Institute (PTI) in the USA. These specifications have been amended recently between PTI and ACI⁶⁾, and are considered to provide a reliable level of long term protection. Key aspects for the durability of these tendons are the material specifications for grease and HDPE. France and Belgium are working on a draft standard for monostrands which is being used also by a European working group for the guideline for European approval of post-tensioning systems⁷⁾. The novelty of these two drafts is that performance characteristics are specified for the raw materials grease and HDPE before and after manufacturing of the sheathing. This permits a control of the quality of the manufacturing process.

For applications where the sheathed monostrand is not encased in concrete or grout, and allowed to move freely inside a duct, the grease is often replaced by wax, and the HDPE sheathing applied tightly around the strand. However, this type of monostrand is suitable only for ungrouted applications since it would produce excessive friction in grouted conditions. For such ungrouted applications e.g. in stay cables, the wax and HDPE sheathing has also been combined with zinc coating.

2.2 Anchorage systems for post-tensioning systems

Anchorage are the heart of any post-tensioning system, and are usually based on proprietary designs by the system owner. It is important to recognise that the basic concepts of anchoring prestressing steels mostly are quite simple, however, that the actual behaviour is significantly more complex, and often is beyond mathematical or even numerical analysis. Hence, much of the design and confirmation of the anchorage concepts still relies on experience and detailed testing.

Post-tensioning anchorages can be grouped into active or stressing anchorages, i.e. allowing stressing of the tendon, passive or dead end anchorages, i. e. not permitting stressing, and coupling anchorages, to connect adjacent segments of tendons. In this paper, the presentation of post-tensioning anchorages will be based on the concept of anchoring the prestressing steel, and the method of transfer of the tendon force onto or into the structure.

2.2.1 Anchorage of the prestressing steel

2.2.1.1 Anchorage by wedge effect

The most common concept of anchoring 7-wire strand, wire, and sometimes even bars is by wedges. Wedges typically come in two or three parts. Wedges interact in a complex manner with the anchor head confining the wedge, and the prestressing steel. Wedges have a smooth surface in contact with the anchor head to permit relative movement, and a saw tooth surface in contact with the prestressing steel to hold the prestressing steel. Not enough wedging effect may cause slippage of the steel, too much may cause premature failure of the prestressing steel. Fig 1. illustrates a typical anchorage by wedge in an anchor head.

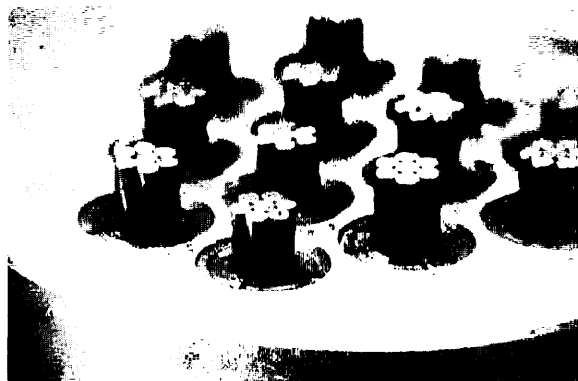


Fig. 1: 7-wire prestressing strand anchored by wedge

2.2.1.2 Anchorage by bearing

This is the concept used for button headed wire anchored inside an anchor head and for nuts sitting on a bearing plate. Since the contact surface between anchor head and button head is comparable to the section of the wire, the contact stresses have the same order of magnitude as the strength of the prestressing steel. While these stresses are beyond any limits given in standards for the lower strength anchor head material, the anchorage concept performs perfectly satisfactory anyway. Fig. 2 shows a typical anchorage for button headed wire.

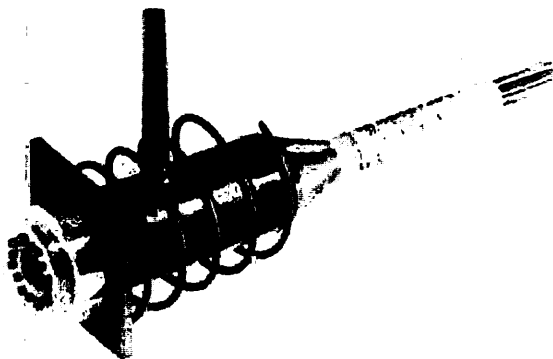


Fig. 2: Prestressing wire anchored by button head

2.2.1.3 Anchorage by nut and thread

This is the common anchorage concept for prestressing bar. The transfer of load across the thread is statically highly indeterminate, and actually only a fraction of the threads is effective in transferring the load at one time. Fig. 3 shows typical nut and thread anchorages on ribbed bar.

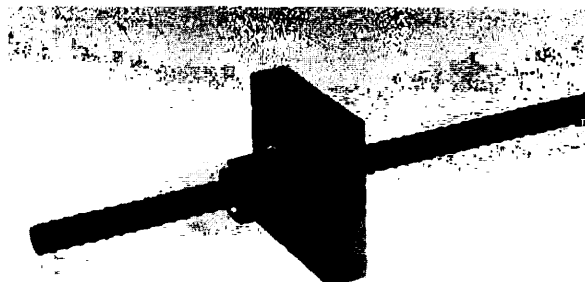


Fig. 3: Prestressing bar anchored by nut and thread

2.2.1.4 Anchorage by compression fitting

This is a special anchorage concept mainly used for 7-wire strand. It consists of a metallic cylinder with a hardened insert liner into which the strand is installed. The strand and cylinder with liner are then pulled through a die which plastically deforms the cylinder to provide a tight fit onto the strand and liner.

2.2.1.5 Anchorage by bond

Another method of anchoring prestressing steel in a dead end is by direct bond to the surrounding concrete similarly to pretensioning. Often bond of the prestressing steel is improved by deformations on the steel, or by forming a bulb from the 7 wires at the end of the strand.

The behaviour and performance of all the above anchorage concepts depend significantly on the manufacturing tolerances, and the consistent quality of these tolerances. They also depend on the preparation of the contact surfaces, e.g. lubrication, oxidation/corrosion, and cleanliness. It is therefore, indispensable to have specialists for installation

and assembly of the anchorages on site who are knowledgeable of these aspects and in the behaviour of the anchorages, and who can take appropriate measures, if needed.

2.2.2 Transfer of load to the structure

2.2.2.1 Basic bearing plate

This is a simple steel plate which transfers the tendon load applied by the anchor head by direct bearing into the concrete or onto other materials such as steel, masonry, and timber. This concept has been used since the start of post-tensioning, and is used for tendons of all types of prestressing steel. Fig. 4 illustrates such an anchorage with basic bearing plate.

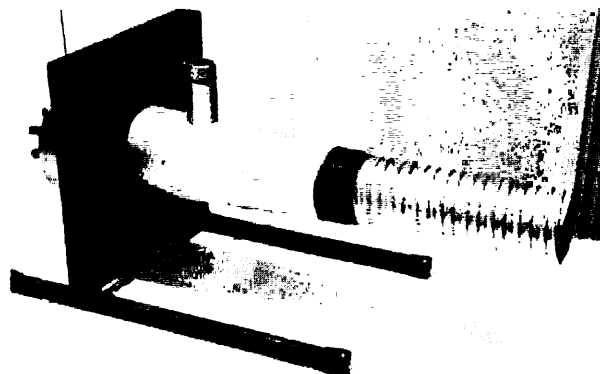
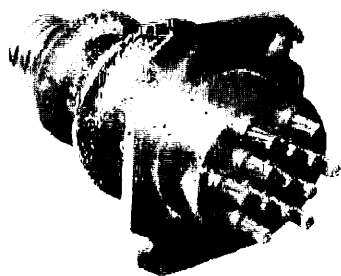


Fig. 4: Basic bearing plate for multistrand tendon

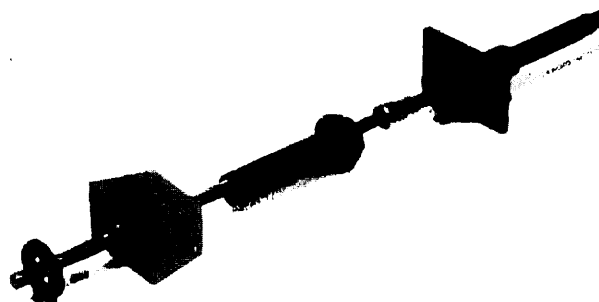
2.2.2.2 Special bearing plate

These are either machined or typically cast iron anchorage bodies which transfer the load from the anchor head to the surrounding concrete at several levels via flanges, etc. More recently high strength concrete has also been used. For small tendon sizes there are also special bearing plates into which the anchor head is integrated, i.e. one component for both functions only. Fig. 5 presents such special bearing plates for multistrand and monostrand tendons.

Fig. 5: Special bearing plates:



a) Multistrand tendon



b) Monostrand tendon

The contact stresses between bearing plate and concrete can reach several times the concrete strength, already at the time of transfer. This is only possible by adequate confinement of the concrete in the immediate vicinity of the bearing plate. This area which needs confinement is often called local zone of the anchorage. Confinement is typically provided by spiral reinforcement or stirrups. This confinement reinforcement is often considered part of the system. The behaviour of the local zone and the concrete beneath it is complex. Therefore, verification of the load transfer is generally based on testing. However, for basic bearing plates, PTI⁽⁸⁾ permits design of the anchorage zones using basic bearing plates by empirical design procedures. This is however, not permitted elsewhere.

2.2.3 Coupling of tendons

Coupling of tendons is common in certain parts of the world but typically avoided in others such as the USA and France. A common type of coupling tendons is by having two anchor heads, one for each tendon segment, back to back, and connected by a threaded sleeve. Another common type is by overlapping the tendon ends in a common coupling anchorage, usually one tendon segment anchored in the centre, the other along the perimeter. Fig. 6 illustrates such coupling anchorages.

Fig. 6: Tendon couplers:



a) By threaded sleeve

b) By overlapping of tendon

Couplers form a discontinuity in the tendon for the flow of the tendon force and particularly for grouting. Special procedures and care are needed to assure reliable filling of couplings with grout. Couplers also introduce discontinuities in the flow of forces in the structure, and require careful detailing and supplementary reinforcement at the joints. This discontinuity has caused problems in structures in the past. It is therefore, common practice to not couple more than 50% of the tendons at one particular section, and add sufficient non-prestressed reinforcement.

2.2.4 Performance of post-tensioning anchorages

While many post-tensioning anchorages still look similar to those at the early stages of prestressed concrete, they have gone through a continuous evolution. Their quality has steadily improved, and has reached an exceptionally high level of performance and reliability. Acceptance of the post-tensioning anchorages is today widely based on detailed testing with stringent acceptance criteria. Such test procedures have been developed in particular by FIP⁹⁾ and have been implemented in many countries in Europe. They will form the basis of the future European approval of post-tensioning systems⁷⁾. They have also been widely accepted throughout Asia including Japan. Also the acceptance tests in the USA by PTI⁸⁾ are converging gradually towards the FIP procedures.

These system acceptance tests include three basic test procedures: The static tensile test which primarily confirms the reliable anchorage of the prestressing steel; the load transfer test which verifies the transfer of the tendon load from the anchorage into the surrounding concrete; and the fatigue test which assesses the performance of the anchorage under stress variations in the tendon. Acceptance criteria as per FIP are stringent since the anchorage is required e.g. to achieve an ultimate load of 95% of the actual tensile strength of the prestressing steel with a minimum of 2% elongation. This corresponds approximately to the specified ultimate tendon capacity. In the USA typically 95% of the specified tendon strength is required, see e.g. PTI⁸⁾. There is still disagreement whether fatigue tests shall be required for all types of tendons, unbonded tendons only, or bonded tendons only. It is the opinion of the experts who prepared the FIP recommendations⁹⁾ that the fatigue requirements should apply to all systems since it is considered confirmation of the good detailing and robustness of the anchorage even though some types of tendons may not be subjected to significant fatigue loading.

Apart from the above discussed performance, post-tensioning anchorages absolutely need to have a reliable quality to permit safe stressing of the tendons on site, every time. This is achieved by design of the components with sufficient safety margin, consistent quality of manufacturing, and installation by experienced and well trained personnel. However, it also requires selection of suitable materials for each of the anchorage components. The selected materials need, apart from strength, to offer in particular sufficient ductility. They also need to be tough and insensitive to minor defects which are hard to detect visually. A suitable choice of material often provides larger safety margins than those typically provided by design.

It may be worthwhile mentioning that in addition to the above performance testing each post-tensioning anchorage is finally test loaded on site during stressing of the tendon, sometimes to values within 15-20% from the specified tendon capacity. Often, this is the highest load the anchorage ever sees during the life of the structure because long term losses will reduce the tendon force by 10-20%, and force increase due to overload is rarely occurring in post-tensioning tendons.

2.3 Ducts

Ducts serve different objectives in post-tensioning. For internal tendons, they first create the void in a concrete structure, in a defined alignment, which allows installation and the free movement of the prestressing steel during stressing. They also form the contact surface which determines the friction losses during stressing. Finally, they form the interface between prestressing steel and structure to transfer bond forces. For external tendons, the ducts serve as a guide for installation of the prestressing steel. They again form the contact surface which determines the friction losses during stressing. Last but not least the duct needs to provide a robust encapsulation of the tendon for long term protection to compensate for the lack of concrete cover. Obviously, duct materials shall not be harmful to the prestressing steel.

While not structural, ducts are an essential component of any post-tensioning system for reliable installation, stressing, and grouting, and for long term durability of the tendon. This has not always been recognised. In particular in the early years, no ducts at all or improvised solutions with cardboard were used. Ducts made of materials which are potentially harmful to the prestressing steel such as PVC have also been used.

The following is a brief review of different types of ducts considered suitable for post-tensioning.

2.3.1 Corrugated metal ducts

This is the most widely used type of duct for internal bonded post-tensioning. It is made from steel strips, spirally wound into a continuous duct. The thickness of the sheet metal strips is usually between 0.2 and 0.7 mm depending on the duct size, whether tendons are prefabricated or installed on site after concreting, and depending on the expected concrete pressure or other loads during construction. Individual segments of duct are typically joined with a sleeve made of an oversized duct which can be threaded onto the corrugation, and sealed with tape or equivalent products. Fig. 7 shows such a typical corrugated duct with sleeve and grout vent.

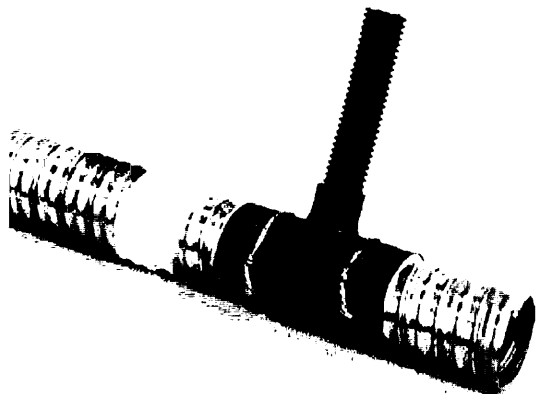


Fig. 7: Corrugated metal strip duct with duct sleeve and grout vent

In Europe, this type of duct has been standardised in EN 523 and 524. This standard can be considered to assure use of state-of-the art corrugated duct. Performance specifications cover the essential requirements, and detailed test procedures permit the verification of the actual behaviour. The main characteristics specified in the standard include duct geometry; stiffness of the duct to avoid excessive deflection of ducts between supports during concreting; flexibility of the duct to allow defined smooth curvatures; resistance to transverse loads to avoid collapse of the duct during concreting; resistance to longitudinal loads during construction; and leak tightness of the duct to water. Similar characteristics are specified e.g. by the PTI⁸⁾ in the USA.

Corrugated ducts must be free of rust to assure reliable friction losses during the stressing of the tendon. The use of galvanised duct is common in many parts of the world to avoid the corrosion of the duct surface. The galvanising also serves as a lubrication and thus, reduces the friction losses compared with a dry and clean black steel surface. However, there are countries who do not permit the use of galvanised duct because of the fear that galvanising could cause hydrogen embrittlement of the prestressing steel. Even though widely used there is no evidence known to the author where the use of galvanised duct would have caused damage to tendons made of cold drawn wire and 7-wire strand.

2.3.2 Corrugated plastic ducts

Corrugated plastic ducts of the type typically used as drainage pipes have been installed already more than 30 years ago for bonded internal tendons. However, some of this type of duct has not performed as expected and has led to excessive wobble of the tendons, excessive friction losses during stressing, and leakage of grout during injection. These problems were mainly due to the flexibility of this type of duct, and the small wall thickness.

At the begin of the 1990's a new generation of thick-walled corrugated plastic duct for bonded, internal tendons has been developed and introduced in the market. This type of duct was specifically designed for use in bonded tendons. It was verified for equivalent characteristics as the corrugated metal duct. In addition, the duct wall was designed such as to positively avoid cutting through by the prestressing strand. The duct system included specific couplers to connect adjacent duct segments, or to connect onto anchorages. Fig. 8 shows this type of plastic duct, with coupler and vent.



Fig. 8: Corrugated plastic duct with duct coupler and integrated grout vent



Fig. 9: Coupler for plastic ducts to seal joints in precast segmental construction.

This duct system has been confirmed by testing to provide a complete leak tight encapsulation of the prestressing steel even across relatively wide cracks, see Abel^[10]. This duct system can also be detailed to provide an electrical isolation of the prestressing tendon from the surrounding structure. This permits electrical resistance measurements to confirm the intactness of the encapsulation at any time during the life of the structure, see Swiss guidelines^[11]. In addition, electrical isolation protects the tendon from the effects of stray currents. The plastic duct system has also been confirmed to reduce the risk of fretting fatigue of the prestressing steel, see Escola^[12].

Several similar duct systems were gradually introduced in the market. However, no commonly accepted performance specification was available. fib Commission 9 has recently prepared a technical report^[13] on these new types of plastic ducts. It includes material specifications, acceptance criteria and test methods for component testing, and acceptance criteria and test methods for system testing. The report recommends to subject these ducts to an approval procedure until sufficient experience has been collected which then may permit to standardise these duct systems. This approach has been integrated into the draft guideline for European approval of post-tensioning systems^[7] which requires approval of these ducts based on the procedures specified in the fib Report^[13].

Most countries consider epoxy resin to provide sufficient protection to post-tensioning tendons crossing joints between precast segments. However, the UK has specified full encapsulation for the tendons by plastic in these joints, see TR 47^[17]. A special plastic duct coupler has now been brought on the market which permits continuity of the encapsulation across segment joints, see Fig. 9.

All these duct systems are seen as a complementary protection of the prestressing tendon in addition to the primary protection by a thick and dense concrete cover over the tendon, and the cementitious grout in contact with the

prestressing steel. The idea of this second barrier against corrosion has since many years been successfully applied to ground anchors.

2.3.3 PE pipe

Thick walled PE pipe is the commonly used type of duct for external post-tensioning. The pipe serves partly similar purposes as the internal ducts. However, since external tendons are not protected by a thick layer of concrete, there are some differences which warrant attention. PE pipe for external tendons may be subjected to significant internal pressure during grouting, and must be designed to safely carry this pressure. The draft European approval for post-tensioning systems⁷⁾ e.g. specifies 1 MPa (10 bar) design pressure for the pipe. PE pipe for external tendons may be subjected to significant temperature variations. In particular, low temperatures may introduce significant restraint stresses into grouted PE pipes. Pipes with local defects or made from recycled material have cracked under such exposure. Unlike for internal tendons, the PE pipe is the only element providing encapsulation to an external tendon. Hence, it must be completely leak tight to assure proper grouting and to assure long term protection and durability of the tendon. Even relatively small defects or holes in the pipe have been found to permit tendon corrosion in aggressive environment.

Careful selection of the PE material is a key element for successful use and durability of the pipes. Only new PE material (granulate) should be used. Material recycled from previously used PE components shall not be used. PE pipes in accordance with the guidelines for European approval for post-tensioning systems⁷⁾ and ASTM D3035 or ASTM F714 can generally be considered suitable for use in external tendons.

Recently, the Japan Highways Administration has started the use of transparent pipe for external tendons to facilitate control of the quality of grouting.

2.3.4 Steel pipe

For certain special applications, smooth steel pipe is used as duct for tendons. Such applications include e.g. long vertical tendons which are intended to be grouted completely at one time. The use of steel pipe permits to confine the grouting pressure and thus, protect the surrounding concrete. Smooth steel pipe is also used in locations of tight tendon curvature such as loop anchorages, and external tendon deviations. The smooth pipe can be bent in the workshop to the specified geometry. It is sufficiently rigid to maintain the specified geometry even under relatively rough conditions. The smooth surface also reduces the secondary transverse stresses applied to the prestressing steel in tight tendon curvatures.

The wall thickness of such pipe is often specified as 2% of the pipe diameter but should not be less than 3 mm to permit welding, if needed.

As for any duct, sealing to provide a leak tight system is also important for steel pipe. Since the steel pipe is rigid compared to other types of duct, it can only adapt to placing tolerances at the pipe segment joints by creating a gap between adjacent pipe ends. Therefore, careful detailing, execution and control of the quality of pipe joints is essential to avoid problems with leaking joints, and pipe blockages.

2.3.5 Duct diameter

The duct diameter must be sufficiently large for proper installation of the prestressing steel and for grouting of the tendon. The required duct size depends on the type of prestressing steel (diameter of steel, sheathing, etc.), the chosen installation method (prefabricated tendon, or pushing of individual strands in-situ, etc.), and the tendon length and geometry. As a general guidance, the cross sectional area defined by the nominal diameter of the prestressing steel, including eventual coating or sheathing, should not exceed 40% of the cross section defined by the inside diameter of the duct. This percentage however, is larger for bar tendons.

Too small duct sizes are considered bad practice since this may cause problems during installation. Perhaps more importantly, too small duct sizes may cause problems during grouting, and leave parts of the tendon incompletely filled and thus, incompletely protected for long term durability. This has been considered the cause of grouting defects found in a recent investigation in Austria, see Eichinger et al¹⁴⁾.

2.4 Accessories

Accessories of post-tensioning systems include such non-structural components as temporary or permanent anchorage caps, grout inlets and outlets (called vents in this paper), coupling connections between duct segments or between duct and anchorage, etc.

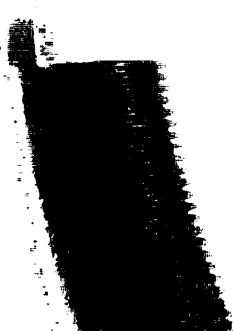
Most of these components serve temporary purpose during installation, and in particular during grouting, and have often not received the attention they actually deserve. Complete grouting of tendons, and consequently good durability, is only possible in a leak tight duct system including all connections and anchorages.

Grout vents traditionally consist of a plastic hose, closed with tie wire, and cut flush with the concrete surface after setting of the grout. Carefully detailed systems however, are available with threaded connections including valves and caps. Such systems provide positive connections which resist site conditions, and which allow proper sealing of the vent after grout setting. Fig. 10 illustrates different types of vents.

Fig. 10: Different types of grout vents:



a) Smooth hose closed with tie wire



b) Corrugated hose with cap

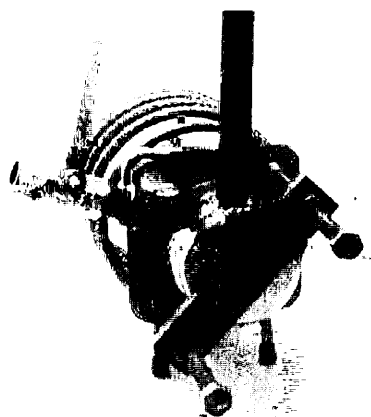


c) Corrugated hose with valve

Commonly used accessories and connections for corrugated steel duct are shown in Fig. 7. These details often employ the use of tie wire and duct tape. For the recently introduced plastic duct systems which offer complete encapsulation, special coupling and connection devices have been developed which are fully compatible with the duct. Such details as shown in Fig. 8 leave no room for improvisation on site and assure consistent quality of all connections.

It is still common practice in many countries to seal anchorages for grouting either with quick-setting mortar (dry packing) or by concreting the anchorage recess before grouting. These procedures do not permit a control of the quality of grouting at the anchorage. Most modern post-tensioning systems now include solutions with temporary and/or permanent caps to seal the anchorage, see Fig. 11. Such solutions are recommended and positively contribute to the durability of tendons.

Fig. 11: Caps for sealing of anchorages:



a) Temporary cap for grouting



b) Permanent cap

Monostrand tendons with extruded sheathing provide excellent corrosion protection along the tendon length. However, the sheathing needs to be stripped at the tendon end for the anchorage of the prestressing steel. It is still common practice in some countries to leave a short monostrand length near the anchorage unprotected, without sheathing. Others consider this as taking unnecessary risks even for benign environment, and always require complete encapsulation. The author is of the opinion that the cost for a protective sleeve and cap to fully encapsulate the monostrand is well spent money, and that full encapsulation should be required in all cases. Fig. 12 shows the two solutions.

Fig. 12: Monostrand tendons:



a) Fully encapsulated strand

b) Tendon with exposed strand at anchorage

Tendon couplers are located by definition at a construction joint. Such joints by experience are known to permit easier ingress of water and eventually aggressive substances such as chlorides into the structure, and to the tendon. Tendon couplers can be quite large, and may be subject to some relative movement during stressing of the second tendon segment. Hence, protection and encapsulation of couplers, in particular in aggressive environment, is important but not necessarily easy. Some engineers are of the opinion that the use of couplers should be avoided for improved durability of tendons.

It seems worthwhile mentioning again that the intactness of the encapsulation along the tendon, at anchorages and couplers, can be verified by electrical resistance measurements if plastic ducts and accessories are used, see Swiss guidelines⁽¹⁾.

2.5 FRP tendons

FRP materials have been successfully used for a number of applications in construction. A considerable amount of research and testing has been carried out on the use of FRP for prestressing tendons. Different groups work on the development of guidelines for the design of structures with FRP tendons, e.g. fib Task Group 9.3, ACI Committee 440, and Japanese Engineers Association. A small number of projects has been carried out with FRP post-tensioning tendons, and experience is being collected on their performance in practice.

It is the opinion of the author that there is insufficient experience available to consider FRP tendons generally suitable for post-tensioning in today's practice. Therefore, FRP tendons are not further covered in this paper.

3. Post-tensioning equipment

Reliable installation of post-tensioning systems on site requires the use of suitable specialised equipment, adapted to the particular post-tensioning system, and operated by experienced and well trained personnel. There are three main types of equipment used in post-tensioning work, i.e. for installation of the prestressing steel, for stressing of the tendon, and for grouting of the duct.

Equipment for installation of the prestressing steel must ensure firm gripping of the steel without damaging it. This aspect is in particular important for coated and sheathed prestressing steel.

Stressing equipment for post-tensioning tendons consists of stressing jacks and accessories to temporarily hold and then release the prestressing steel during stressing. Stressing equipment must ensure reliable stressing of the

prestressing steel to the specified load, and reliable transfer of the load from the jack to the permanent tendon anchorage. Stressing jack and accessories must be fully compatible with the specific post-tensioning anchorage to ensure such things as complete release of the wedges during stressing to avoid excess friction in the anchorage or scratching of the prestressing steel, and reliable seating of the wedges at transfer of the tendon load to the anchorage to assure the expected anchorage performance. Stressing equipment must also permit multiple stressing stages to accommodate the elongation of long tendons. In addition, it must allow detensioning of a tendon, if a problem occurs during stressing. Stressing jack and accessories such as the gripping devices must permit simultaneous stressing of the entire tendon, and assure equal forces and elongations in all tendon elements even for multiple stage stressing. Stressing equipment shall be regularly calibrated by a qualified body. Many specifications require that the calibration of the stressing equipment is not older than 6 months. Above all, stressing equipment must allow safe operation on site, every time a tendon is stressed. An unintended sudden release of the energy stored in a prestressing tendon stressed to 70-80% of its strength could be disastrous.

Suitable grouting equipment is important to ensure complete filling of the duct with a properly mixed and homogeneous grout. The equipment must permit the filling of a tendon duct in an uninterrupted operation, at the expected speed, and up to the expected maximum pressure. Again, safety aspects are important since cementitious grouts are potentially harmful to the human skin and in particular to the eyes.

For all the above reasons, post-tensioning systems typically are provided with specific proprietary equipment. It is important that the equipment is included in the independent assessment by a qualified body during the post-tensioning system approval. For its potential effect on the performance of the tendon anchorage, the FIP recommendation⁹⁾ and the draft guideline for the European approval of post-tensioning systems⁷⁾ require that the tendon in the static tensile test discussed in Section 2.2.4 be loaded to 80% of the specified tendon strength with the stressing equipment. Only at this 80% level, the load is transferred to the testing rig for further loading up to ultimate.

4. Corrosion protection of post-tensioning tendons

Corrosion protection is probably the most important aspect to achieve durable post-tensioning tendons. The objective must be to achieve a design life of the tendon comparable to that of the structure in which it is placed. The design of the corrosion protection systems should take into account that most parts of the tendon are not accessible during the design life, in general, and that individual components or the entire tendon, are not replaceable, in general. Even if special details are provided to allow replaceability of the tendon during the design life, it is the author's opinion that this should be considered an exceptional case which is not normally expected to really happen, i.e. the corrosion protection shall be designed for the entire design life. A detailed discussion of all aspects of corrosion protection of prestressing steels is provided in a FIP recommendations²⁾. Corrosion protection of post-tensioning systems includes temporary protection of the components, and starts at the manufacturing place of the components.

4.1 Temporary corrosion protection

Materials and components for post-tensioning shall be protected and stored such as to avoid their corrosion and staining. Typically, light superficial corrosion which can be removed by wiping with a soft cloth is acceptable. However, any more severe corrosion must be avoided, in general, but in particular on surfaces where corrosion may impair the proper functioning of the component. This applies in particular to the anchor head wedge cavities, and the wedges, but also to the ducts. Temporary protection by oils is often sufficient since these surfaces will receive permanent protection at a later time, e.g. by grouting.

Due to its high strength, prestressing steel is more susceptible to corrosion than other components, in general, and therefore, must be stored and protected carefully. Bare prestressing steel can be packed and wrapped by the manufacturer such as to provide specified temporary protection. The prestressing steel should be stored on site in a dry and clean location, off the ground, and with sufficient ventilation to prevent condensation. In special cases, it can be stored in an air-conditioned area.

For extended periods of storage on site, the temporary protection can be improved by application of oils onto the prestressing steel. These oils should not contain substances potentially harmful to the prestressing steel. These oils should be applied at the manufacturing place, and renewed on site as needed. Preferably, products should be specified which do not excessively reduce the bond properties of the steel, and which do not need to be removed before application of the permanent protection. Temporary corrosion protection used for wire and strand starts to deteriorate through evaporation after its application, and is effective in moderate atmospheric conditions for up to approximately 8

weeks. Bars can be coated with types of oils which can be effective for up to 6 months. In any case, the actual product and expected period of protection should be verified with the supplier.

Once the prestressing steel is installed in the structure, it should be stressed and permanently protected, e.g. by grouting, as quickly as possible. Guidance on the maximum period of time between installation of the prestressing steel, stressing of the tendon, and final protection by e.g. grouting of the tendon made of bare prestressing steel may be found in selected standards and publications. Without taking additional protective measures the AASHTO Standard Specifications for Highway Bridges¹⁵⁾ give 7, 15, and 20 days as permissible intervals between tendon installation and grouting, for very damp (> 70% relative humidity), moderate, and very dry atmosphere (< 40 % relative humidity), respectively. The final draft European Standard on "Execution of concrete structures" proposes a maximum interval of 12 weeks between tendon fabrication and grouting, a maximum period of 4 weeks for installation of the tendon into the formwork before casting the concrete structure, and a maximum interval of 2 weeks between tendon stressing and grouting. If grouting needs to be delayed beyond the above proposed intervals, particular protection methods need to be provided for post-tensioning tendons. These particular protection methods include the use of water soluble oils sprayed onto the prestressing steel as discussed above for storage. As mentioned, only oils should be selected which do not need to be removed by flushing with water before grouting. Flushing with water is undesirable for environmental reasons and because it is likely to leave water in the duct which may negatively affect the quality of grouting.

Other temporary protection methods include the blowing of dry air or inert gas such as nitrogen through the tendon. All the mentioned temporary protection methods can also be used to protect post-tensioning tendons in winter time when temperature does not allow grouting.

Leaving prestressing steel inside the duct without permanent corrosion protection for extended time, has been reported as contributing factor or even as cause for durability problems and delayed failures of post-tensioning tendons, e.g. in Germany. Pitting corrosion on the prestressing steel must be avoided since it may in addition affect the durability of the tendon through a reduced fatigue life of the prestressing steel.

4.2 Semi-permanent corrosion protection

Other temporary protection methods which are more durable than those presented in the above section, include the coating of the prestressing steel as presented in Section 2.1.2. As mentioned in the referenced section, such coated steels may be considered e.g. for temporary tendons installed over extended periods of time.

4.3 Permanent corrosion protection applied at the factory

The most frequently used type of permanent corrosion protection applied in the factory is the greasing and sheathing of the strand with HDPE as presented in Section 2.1.3. These so-called monostrand tendons are typically used for building slabs. They have also been used for transverse tendons in bridge decks, and for hoop tendons in silos and reservoirs. Sheathed monostrand provides a reliable corrosion protection along the tendon length. However, as mentioned in Section 2.4, the ends of the monostrand at the anchorage need to be suitably protected by encapsulation.

Epoxy coating of prestressing steel has been proposed as permanent corrosion protection. However, experience with post-tensioning tendon is quite limited. A more detailed discussion of factory applied corrosion protection methods can be found in a recently published fib report⁴⁾.

4.4 Permanent corrosion protection applied on site

The by far most common permanent corrosion protection method for post-tensioning tendons applied on site is by injecting the tendon duct with a cementitious grout. This method has been used since the beginning of post-tensioning and has, when done according to the rules of good practice, performed extremely well.

For external tendons, occasionally injection of the tendon duct with grease or wax has been used for permanent corrosion protection. Both materials need to comply with stringent specifications to assure long term durability. Such specifications have e.g. been proposed in the draft guideline for European approvals of post-tensioning systems⁷⁾.

A detailed review of all aspects of grouting of post-tensioning tendons on site is given in a separate presentation at this workshop, see Fuzier¹⁶⁾.

5. Installation of post-tensioning systems

Durable post-tensioning tendons are only achieved if the post-tensioning systems are installed on site in accordance with proven procedures for assembly of the components, installation and stressing of the prestressing steel, and grouting. Installation must be carried out by experienced and well qualified personnel intimately familiar with the particular post-tensioning system and the corresponding equipment. The personnel must be knowledgeable of the potential consequences of poor installation on safety and durability, and must be able to react correctly if problems are detected. Only suitable equipment adapted to the particular post-tensioning system, and operated by experienced and qualified personnel shall be used.

The above basic principles are assured best if the installation of the post-tensioning system is done by a specialist contractor who has either developed the system, or has been adequately trained by the system developer. The personnel on site in charge of post-tensioning works should be specifically trained as "post-tensioning craftsman" for the specific system. This concept has been recognised twenty years ago in France, and has led to the creation of a new profession, the "Chargé de Mise en Précontrainte (CMP)". In addition, national approvals for post-tensioning systems in France include the notion of qualification of the specialist contractor and its personnel (CMPs), and includes the verification of this aspect in the regular audits performed by the certification body.

These above mentioned principles have been applied in the early stages of post-tensioned concrete since the company who developed a system normally installed it also. However, this has changed later on due to different reasons. Post-tensioning has probably been considered by some as a commodity which can be done by everybody. In the author's opinion today's durability problems are at least partly a consequence of this "commodity / do-it-yourself approach".

The UK has recognised the importance of qualification of companies and personnel during the period of the temporary ban of post-tensioning, and has subsequently introduced a certification procedure for post-tensioning companies, see TR47 report¹⁷⁾. The Post-Tensioning Institute (PTI) has introduced a certification procedure for field personnel for unbonded monostrand post-tensioning. The working group preparing the guidelines for European approval of post-tensioning systems⁷⁾, has proposed to combine the qualification of the post-tensioning companies with the system approval. However, this proposal has unfortunately been blocked by a minority of member states. Also FIP has published a recommendation¹⁸⁾ on the qualification and approval of prestressing contractors and system suppliers.

6. Conclusions

Through a continuous process since the begin of post-tensioning, the materials and entire systems have gradually evolved and have achieved an extremely high level of safety, and reliability, in general. We know exactly what we need to do to achieve safe and durable post-tensioning systems.

Since the post-tensioning systems are proprietary, non-standardised products, they need to be reviewed by a qualified independent body in an approval procedure before they are first placed on the market. This review has to include all aspects of components specification, anchorage performance, installation procedures, equipment, traceability, quality assurance, etc. Once initially approved, the system must be subjected to a continuing certification procedure including technical audits and independent testing of components.

There are enough countries who have implemented such approval and certification procedures for post-tensioning systems which have proven to work and to assure reliable quality of the components and systems. These models could be used in countries which do not have such approval procedures yet. In the author's opinion such approval procedures are an "absolute must" for all post-tensioning systems in all countries.

We generally acknowledge the importance of the quality of installation of the post-tensioning systems and the qualification of personnel to achieve durable and safe post-tensioning tendons and thus, durable post-tensioned structures. However, with a few exceptions, owners and countries have not shown enough determination to implement an approval and certification scheme for the post-tensioning companies and personnel yet. Again, in the author's opinion, the risks involved in the installation and stressing of post-tensioning tendons are such that this work should be left to specialists, specifically trained for a particular post-tensioning system such as the CMP in France.

The author is convinced that we can assure durable and safe post-tensioning tendons with the today available knowledge. We just need to rigorously apply the knowledge and only accept the combination of approved post-tensioning system with approved specialist contractor, both being approved and subsequently certified by a qualified independent body.

References

- [1] Müller, H. R. Post-tensioning materials and systems from the past to the year 2000. Proceedings, FIP Symposium Brisbane, pp 353-359, 1995.
- [2] Corrosion protection of prestressing steels. FIP Recommendations. Fédération Internationale de la Précontrainte (FIP). Published by SETO, London, 1996.
- [3] Bruggeling A.S.G., Jungwirth D., Ganz H. R., Müller H. R. Brittle failure of prestressing steels - Statement by FIP Commission 2. Fédération Internationale de la Précontrainte (FIP). Leaflet attached to FIP Notes 1998/1.
- [4] Factory applied corrosion protection of prestressing steel. State-of-art report. fédération internationale du béton (fib), 2001.
- [5] Hampejs G., Jungwirth D., Morf U. Timiney P.. Prestressing materials and systems: Galvanisation of prestressing steels. Commission reports. Fédération Internationale de la Précontrainte (FIP). FIP Notes 1991/4.
- [6] Specification for unbonded single strand tendons. Draft report by ACI Committee 423. American Concrete Institute (ACI), February 20, 2001.
- [7] Guideline for European technical approval of post-tensioning kits for prestressing of structures. Draft January 2001. European Organisation for Technical Approvals (EOTA). Brussels, 2001.
- [8] Acceptance standards for post-tensioning systems. PTI Guide specification. Post-Tensioning Institute (PTI). Phoenix, 1998.
- [9] Recommendations for the acceptance of post-tensioning systems. FIP Recommendations. Fédération Internationale de la Précontrainte (FIP). Published by SETO, London, 1993.
- [10] Abel M. Zur Dauerhaftigkeit von Spanngliedern in teilweise vorgespannten Bauteilen unter Betriebsbedingungen (On the durability of tendons in partially prestressed structures in service conditions). Lehrstuhl und Institut für Massivbau IMB. Aachen, Heft 4, 1996.
- [11] Massnahmen zur Gewährleistung der Dauerhaftigkeit von Spanngliedern in Kunstbauten (Measures to assure the durability of post-tensioning tendons in bridges). Guidelines. Bundesamt für Strassen und SBB AG. Draft August 2001.
- [12] Eskola L. Zur Ermüdung teilweise vorgespannter Betontragwerke (On the fatigue of partially prestressed concrete structures). IBK Report No. 223. Institut für Baustatik und Konstruktion (IBK). ETH Zürich, 1996.
- [13] Corrugated plastic ducts for internal bonded post-tensioning. Technical report. Bulletin 7. fédération internationale du béton (fib), 2000.
- [14] Eichinger E. M., Diem J., Kollegger J.. Bewertung des Zustandes von Spanngliedern auf der Grundlage von Untersuchungen an Massivbrücken der Stadt Wien (Assessment of the condition of post-tensioning tendons on the basis of investigations on concrete bridges of the city of Vienna). Institut für Stahlbeton- und Massivbau, Heft 1. Technische Universität Wien, 2000.
- [15] Standard specifications for highway bridges. Fifteenth edition. American Association of State Highway and Transportation Officials. Washington D.C., 1992.
- [16] Fuzier J. Ph. Development of grout und grouting techniques. Proceedings. Workshop on Durability of Post-Tensioning Tendons. fédération internationale du béton (fib) and International Association for Bridge and Structural Engineering (IABSE). Ghent, 2001.
- [17] Durable bonded post-tensioned concrete bridges. Technical Report No. 47. The Concrete Society. Slough, 1996.
- [18] Qualification and approval of prestressing contractors and system suppliers. FIP Recommendations. Fédération Internationale de la Précontrainte (FIP). Published by SETO. London, 1998.

Referenced standards:

1. ASTM Standards:

- A 416: "Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete".
- A 421: "Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete".
- A 722: "Standard Specification for Uncoated High-Strength Steel Bar for Prestressed Concrete".
- A 882: "Standard Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand".
- D 3035: "Standard Specification for Polyethylene (PE) Plastic Pipe (DR-PR) Based on Controlled Outside Diameter".
- F 714: "Standard Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Outside Diameter".

2. European Standards:

- EN 523: "Steel strip sheaths for prestressing tendons - Terminology, requirements, quality control", 1997.
- EN 524: "Steel strip sheaths for prestressing tendons - Test methods", Parts 1-6, 1997.
- pr EN 10138: "Prestressing steels", Parts 1-4, Draft 1999.
- pr ENV 13670-1: "Execution of concrete structures - Part 1: Common", Final Draft, July 1999.

3. Japanese Standards

- JIS G 3109: "Steel bars for prestressed concrete", 1994.
- JIS G 3536: "Uncoated stress-relieved steel wires and strands for prestressed concrete", 1999.

4. British Standards

- BS 4486: "Cold worked high tensile alloy steel bars for prestressed concrete".
- BS 5896: "High tensile steel wire strand for the pre-stressing of concrete".

5. French Standards

- NF A 35035: "Fils lisses et torons de précontrainte galvanisés à chaud", April 1993.

Development of grout and grouting techniques

J.-Ph. FUZIER

Freyssinet International (France)

1. Introduction

The highly stressed steels must be protected from external corrosive and aggressive agents such as water oxygen and, especially in the case of bridges, the infiltration of de-icing salts. This requires that the tendons are completely surrounded with a protective barrier.

A first barrier is evidently the concrete surrounding the tendons: concrete quality, concrete cover, waterproofing and drainage constitute the first line of defence.

In the case of post-tensioning, the tendons are placed either in ducts within the body of the concrete section (i.e. internal prestressing) or outside the concrete section (i.e. external prestressing). These ducts may constitute another barrier, a second line of defence.

Then the ducts are filled in with a protective material. This is the third line of defence. In case of bonded tendons, cement grout is normally used.; it plays the two roles:

- protecting the steel tendon from corrosion. One important feature, they are strongly alkaline (pH up to 13); this alkalinity will inhibit the corrosion of the steel provided that it is not neutralised (pH falling below 9) by reaction with atmospheric carbon dioxide or infiltration of chlorides.
- bonding the tendon to the concrete member throughout its length.

In case of unbonded tendons, other materials could be used: grease, petroleum waxes, etc.

Experience in many countries of the world has shown that past practices, on some occasions, did not achieved a complete filling of the ducts. In response, suppliers and specifiers have recently introduced new systems, tests and procedures which have the objective of improving durability and long term performance of post-tensioned structures.

Objectives

Improvements on the current processes of grouting post-tensioning ducts cannot be achieved by any single action on its own, but require parallel developments in the grouting materials, grouting techniques, grouting equipments and QA/QC systems.

It is recognized, that proper installation and use of a system, is as important as the performance of the system components for the successful application of prestressing.

It has therefore been the objective of the various actions to develop:

- better grouts which have improved properties: flowabilities, bleed, volume change, stability
- more realistic tests of grout performance
- improved grouting techniques, equipments and specifications
- QA systems which can be applied during and immediately after grouting
- inspection and long term monitoring systems

2. Grout Materials

An extensive work has been recently done in order to characterise the properties of the grout components. PTI (USA) has issued end of last year (2000) its: 'Specification for grouting of post-tensioned structures'. The fib task group 9.8 is finalising its 'guidelines for grouting'.

2.1. Cement

Cement used for grouting should be ordinary Portland cement (type CEM 1). [ENV 197-1].

Furthermore, cement has to comply with the following requirements:

- $Cl^- < 0.5\%$
- $S^{2-} < 0.01\%$
- no lumps
- not contain any substances harmful to the prestressing steel
- additives $< 0.1\%$. Only grinding agent should be allowed
- cement has to be correct to the Tusshenbroeck flash set test.

It must be delivered in sacks of defined weight, usually 50kg or 25kg and disposed on pallets wrapped, or in containers and stored in a dry place to protect them from humidity and moisture. Tolerance in weight should be less than 2% (1% less than defined weight and 1% more than defined weight).

To preserve a constant quality of grouting, the particle size of cement can be defined by a nominal Blaine specific area and tolerances have a significant effect on the properties.

Furthermore, the manufacturing date of cement should be printed on each sack to control age of it before using in grout.

2.2 Mineral additives

Mineral additives such as fly ash, or silica fume, can be used to improved the properties of the grout; the potential benefits include:

- reducing the maximum temperature developed during cement hydration reactions
- increasing long-term strength
- decreasing permeability of the grout with the effect of slowing down the rate of chloride ion migration.
- control of bleeding

2.3 Admixtures

They should not contain any substances harmful to the prestressing steel, especially thiocyanates, nitrites, nitrates, formiates and sulphurs and they have to comply with the following requirements:

- $Cl^- < 0.10\%$ in weight of powder admixtures
- $Cl^- < 1$ gram per litre of liquid admixtures
- pH has to be included in limits which are defined by the manufacturer
- dry extract: included in a scale of $\pm 5\%$ from the defined value

Well proven admixtures should be used which comply with appropriate standards. The admixtures are usually a plastifier and a retarding agent which improve:

- flowability at a given W/C ratio
- elimination of bleeding water
- prevention of segregation in high pressure grouting
- retarding of the setting of the grout

The admixtures may also provide additional properties such as:

- expansion of grout to compensate the shrinkage
- air entrainment to improve freeze resistance during the setting period
- thixotropy

The product should be in bags or container or within a pre-bagged mix with the identification and the instructions of the manufacturer (product sheet, safety data sheet...). Retarders delay the setting of the grout, and are particularly useful for grouting of long ducts. By using methanol or glycol, the grout can be made freeze-resistant during the setting period, but this may cause some reduction in strength and problems with setting, segregation and bleeding. Compatibility and suitability checks should be made for the various components products.

2.4 Water

The water should not contain ingredients harmful to the prestressing steel or the cement grout. It should not contain more than 300mg of chloride ions per litre or 200mg per litre of SO_4^{2-} ions, or any organic materials. Drinking water from the public water supply may be used.

When public drinking water is not available, a chemical analysis should be performed.

Water should comply with EN 1008.

2.5 Grout mix performance

The cement grout has to be designed considering all materials included in it. The grout should not contain more than:

- $\text{Cl}^- < 0.10\%$ by weight of cement
- SO_3 (expressed) $< 4.50\%$ by weight of cement
- $\text{S}^{++} < 0.01\%$

Furthermore, the following performances should be achieved.

- Flowability

The flow time with the marsh cone (diameter: 100mm) has to be less than 25 seconds in the range of temperatures defined by the manufacturer. For thixotropic grouts other limits and test methods will apply.

- Maintenance of the flowability

The flow time has to be under 25 seconds until at least 30 minutes after the grout manufacturing or until the time defined by the manufacturer or specified by the designer.

- Bleeding requirements

The quantity of bleeding water and air from the grout after 3 hours kept at rest, has to be less than 0.3% in the bleeding test tube. This requirements should be the same with grout kept at rest during 24 hours. The grout has to be designed to preserve these requirements in the range of temperatures defined by the manufacturer.

- Mechanical strength

This is not considered an overriding parameter but typically the following are guidelines:

- the flexural strength after 7 days has to exceed 4Mpa
- the compressive strength after 7 days has to exceed 27 Mpa on 100mm cubes.

- Setting time

Beginning of setting time of the grout: > 3 hours, in the range of temperature defined by the manufacturer.

Ending of setting time of the grout: < 24 hours, in the range of temperature defined by the manufacturer.

- Capillarity absorption

After 28 days, capillarity absorption has to be less than 1g per square centimetre.

- Volume change

Volume change should be between 1% and + 5% from the initial volume.

- Shrinkage and cracking

Shrinkage measured after 28 days according to the NF P 18-361 should be < 0.1%.

2.6 Thixotropic grout

This is the property of a grout that enables it to stiffen in a short time while at rest, but to acquire a lower viscosity when mechanically agitated, the process being reversible.

The interest of such a grout is that still using the same grouting equipment, it flows in the duct with a quasi perpendicular head avoiding thus the creation of air voids.

The fluidity of this grout cannot be measured with the Marsh cone, but rather the shear stress is measured with a 'scissometer' and expressed in g/cm^2 . Viscosity will be $> 120 \text{ g/cm}^2$ and $< 200 \text{ g/cm}^2$.



Thixotropic grout



Marsh cone

3. TESTING

3.1. Testing Regime

The purpose of testing is to ensure suitability of the grout to achieve the aim of full protection and bond to the prestressing steel. The testing regime may include proving tests (either in a laboratory or site conditions), suitability test (generally on the site) and acceptance tests (on site). Additionally in some countries system type approvals are given by the authorities which may include the testing of groutability and of the grout itself.

- **Proving tests**

Experience has shown that full-scale trials of grout and grouting procedures are valuable to prove adequacy. For some projects it may be advisable to carry out project specific trials of grouting which should be planned and carried out well in advance of site operations; however, recent test results of identical grout mixes for similar tendon configurations may be acceptable for many projects. Proving tests should include results of examining the completeness of filling of the duct, particularly at anchorages and at high points of the tendon profiles where air voids may form. This can be achieved by coring into the trial after the grout has set and by physical examination.

- **System type approvals**

Some countries, especially in Europe, have requirements for prestressing systems to undergo rigorous testing to be given a system type approval for use, which may include the grouting. In this case, provided the details proposed for use on a project are considered to be typical, testing may be limited to suitability and acceptance as described below.

- **Suitability tests**

In all cases it is advisable to perform the testing of the suitability of the grout for the site conditions before using the grout in the project. This may either be carried out on site or supplied by the manufacturer and certified for the batch delivered to site. Suitability testing includes testing strength, bleeding, volume change, fluidity and sedimentation and demonstrating compliance with the specified limits.

- **Acceptance tests**

Routine acceptance testing is carried out on all projects during grouting operations to demonstrate compliance and consistency with the project requirements and confirm that the required durability should be achieved. The testing includes tests for strength, bleeding, volume change, fluidity and sedimentation.

3.2 Testing methods

According to EN 447 (1996), the grout should as a minimum be tested for:

- Fluidity
- Bleeding
- Volume change

- **Strength**

Appropriate test methods for determining these properties are presented in EN 445/ 1996. Other grout performance criteria commonly tested but not required by EN 447/ 1996, are:

- Setting time of liquid grout
- Density of liquid grout
- Permeability of liquid grout
- Density of hardened grout

Grout performance criteria less frequently tested are:

- Sedimentation of liquid grout
- Frost resistance of grout in the plastic state
- Frost resistance of hardened grout
- Uniformity of grout structure
- Strength of bond between grout and post tensioning steel as well as between grout and duct

3.2.1 Strength test

The compressive strength of the grout is measured on cubes of 50 x 50 x 50 millimetres up to 100 x 100 x 100 millimetres or on cylinders, diameter 50-100 millimetres with a length similar to their diameter. The samples should be cured in site conditions. Generally it is sufficient to test one set of cubes for each day of site operations but testing frequency should be in relation to the volume of grout being used.

3.2.2 Bleeding test

The test consists of measuring the quantity of water remaining on the surface of the grout, which has been allowed to stand protected from evaporation. Simple tests using small glass or plastic cylinder have been shown to inadequately represent the true conditions inside ducts and are no longer recommended.

It has been observed that the type of bleed test currently specified in most National Codes, and in the UK Concrete Society Report TR47, fails to identify potentially unstable grouts. The important feature missing from these tests is the destabilising effect of the 'wick action' which is imposed by the presence of strands. This shortfall has been addressed by LCPC in France by the development of an inclined tube test. It has also been addressed in a BRITE Euram project 'QA of Grouting', by the development of a 1.5m vertical test.

It has also become evident that current specifications that require bleed water to be reabsorbed in 24 hours have no logical basis. If bleed water develops, reabsorption will merely create an air void.

Likewise, requirements to measure the bleed water after three hours are not necessarily relevant to modern thixotropic grouts.

An inclined or vertical tube test containing prestressing steel as used on site has been shown to better represent the conditions inside ducts. The presence of this steel provides a passage for any bleed water to rise to the surface. Tube tests are described in paragraph 3.3.

3.2.3 Volume change test

This test measures the expansion or contraction of the grout and is conventionally carried out using small glass or plastic cylinders protected from evaporation. The volume change is

reported after 24 hours. For acceptance testing the test should be performed at least twice each day.

3.2.4 Fluidity test

The fluidity of the grout, expressed in seconds, is a measure of the injectability and flowability of the grout. It can be measured by three methods; either measuring the time for a stated quantity of grout to flow through a cone. Alternative test methods are measuring the time for an immersion body to plunge down a tube filled with grout or by use of a shear vane ('scissometer'). The cone test is more commonly used and is simpler to use and appropriate for most applications.

It is important in suitability testing to measure the fluidity at times after mixing which represent the likely length of time the grout will be required to remain fluid for injection through the ducts. For acceptance testing the tests can be used to demonstrate that grout expelled at vents and anchorages remote from the injection point is of satisfactory quality.

3.3 New bleeding (stability) tests

3.3.1 Vertical tube

(1) Equipment

The test also enables volume change to be measured.

Vertical duct of transparent material with one end sealed

- Internal diameter d_i = 60 to 100mm
- Height h = about 1.5m

Prestressing strands

- Effective diameter d = about 16mm
- Length l = 1.4m
- Number of strands N = sufficient to fill about 30% of cross sectional area of the duct, i.e. $3Nd^2 = d_i^2$

(2) Procedure

- Set up transparent duct in a vertical position with its open end at the top. Provide rigid fixing preferably on a concrete floor so that no movement or vibration can occur.
- Install the strands in the duct, ensuring that they are all firmly located on the base.
- Pour the grout into the top of the duct at a steady flow rate to ensure there is no trapped air. Fill the duct to a height, h_0 , about 10mm above the tops of the strands. Put plate on to top of duct to minimise evaporation.
- Record starting time and height of the grout.
- Record height of grout, h_g , at 15 minutes intervals for first hour and subsequently at 30 minutes intervals until there is no further change in 3 consecutive readings.

NB: The purpose of recording intervals instead of simply taking the final value is to be able to follow the behaviour of expansive grouts. If, for one reason or another, the grout has been left for a time before being poured into the duct, expansion may be

completed and measured behaviour will be affected. Measurements should ignore the meniscus on the grout.

- Record height of bleed water, h_w , at the same times as the grout.
- Record any inhomogeneties that may develop in the appearance of the grout as seen through the transparent duct. Examples of inhomogeneties are:
 - formation of lenses of bleed water below top of grout
 - segregation leading to areas of different coloured grout

(3) Results

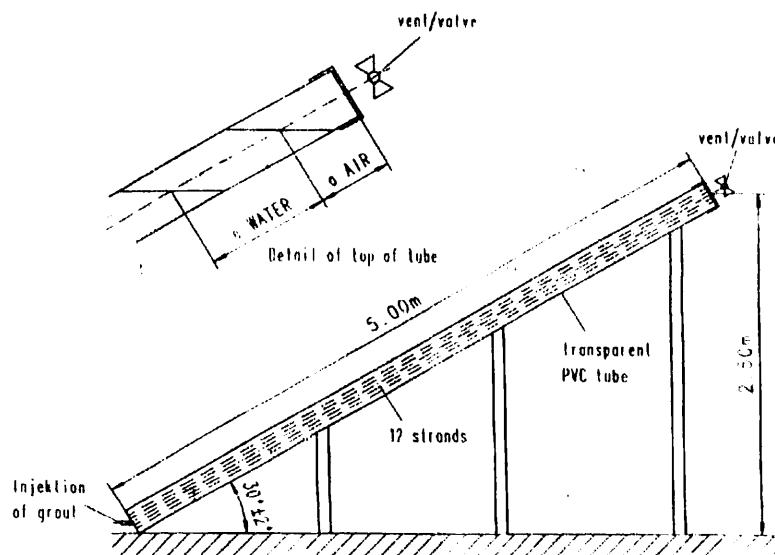
Bleeding is expressed as $\frac{h_w}{h_o} \times 100 \text{ percent}$

Volume change is $\frac{h_g - h_o}{h_o} \times 100 \text{ percent}$

3.3.2 Inclined tube test

(1) Objective

This test serves to determine the bleed properties and stability of grout which has successfully passed preliminary testing, at full scale and including the filtering effect of strands. It also allows to confirm the proposed grouting procedures, in particular the effect of time between ending an initial grouting and starting of re-grouting on site, if specified, and equipment used on site. The intent of the test is to confirm that a duct on site can be completely filled with the proposed grout, equipment and procedure without excessive bleed of the grout.



Inclined tube

(2) Test method

In a first phase, the bleed water and air accumulated on top of a tube filled with grout should be determined. The grout is injected under pressure and is setting such that water losses due to evaporation are prevented. In a second phase, the effect of re-grouting of a tube on bleed water and air accumulated can be determined, if such a procedure is envisaged.

(3) Test equipment and set-up

- Two transparent PVC tubes, of approximately 80mm diameter and 5m long, equipped with caps at each end including grout inlet at the lower end, and grout vent at the top. The tubes should be able to sustain a grout pressure of at least 1 Mpa.
- 12 prestressing strands $\varnothing$ 15-16mm per tube, i.e. a total of 24.
- Grouting equipment as per the grouting method statement.
- A thermometer with automatic recording.

(4) Test procedure

- The two tubes are fixed on their supports such as to avoid noticeable deflections, at an inclination of $30^\circ \pm 2^\circ$ against a horizontal reference line. 12 strands should be installed in each tube. The caps are subsequently installed on the tube ends (fixed with glue).
- The grout is prepared as per the grouting method statement. Specimens should be taken from the grout mix to confirm flow time. In case of a thixotropic grout, other methods must be used.
- Grouting of first tube.

Grout is injected into the first tube (tube 1) from the bottom end. When the grout exits from the vent at the top with the same consistency as it enters at the bottom, the valve should be closed, and the grout pressure should be maintained at the value and for the duration specified in the method statement. Subsequently, the valve at the bottom is closed, and grouting of tube 1 is considered complete.

The level of air, water and any other eventual liquid on top of the grout should be measured. Such eventual liquid on top of the grout can be distinguished from the grout by its whitish to yellowish colour, usually clearer than the grout. A minimum of 4 measurements of levels should be taken between 0 and 24 hours after completion of grouting, with one measurement just before re-grouting of tube 2 is started. The following 4 measuring intervals are suggested: 30 minutes, 1 hour and 24 hours after grouting.

- Grouting of second tube

Grouting of tube 2 should follow the same procedure as used for tube 1 and should be done quasi simultaneously with tube 1. At a time specified in the method statement for re-grouting, the mixing of grout in the equipment is started again, and the flow time of the grout is determined again.

Subsequently, the valves of inlet and vent of tube 2 are opened again, and grouting is started again. This will allow any liquid accumulated on top to be replaced by grout. When grout exists from the vent on top, the valve is closed, and the grout pressure is maintained for the duration specified in the method statement. Subsequently, the valve at the bottom is closed, and re-grouting of tube 2 is considered complete.

The time between initial grouting and re-grouting, and the duration for the second mixing activity, should comply with the grouting method statement. Typically, this time will be between 30 minutes and 2 hours.

Similar to tube 1, the measurement of levels are done between 0 and 24 hours after completion of the initial grouting. One of the measurements should be taken just prior of re-grouting tube 2 followed by measurements 30 minutes, 1 hour and 2 hours after completion of re-grouting.



Pressure grouting

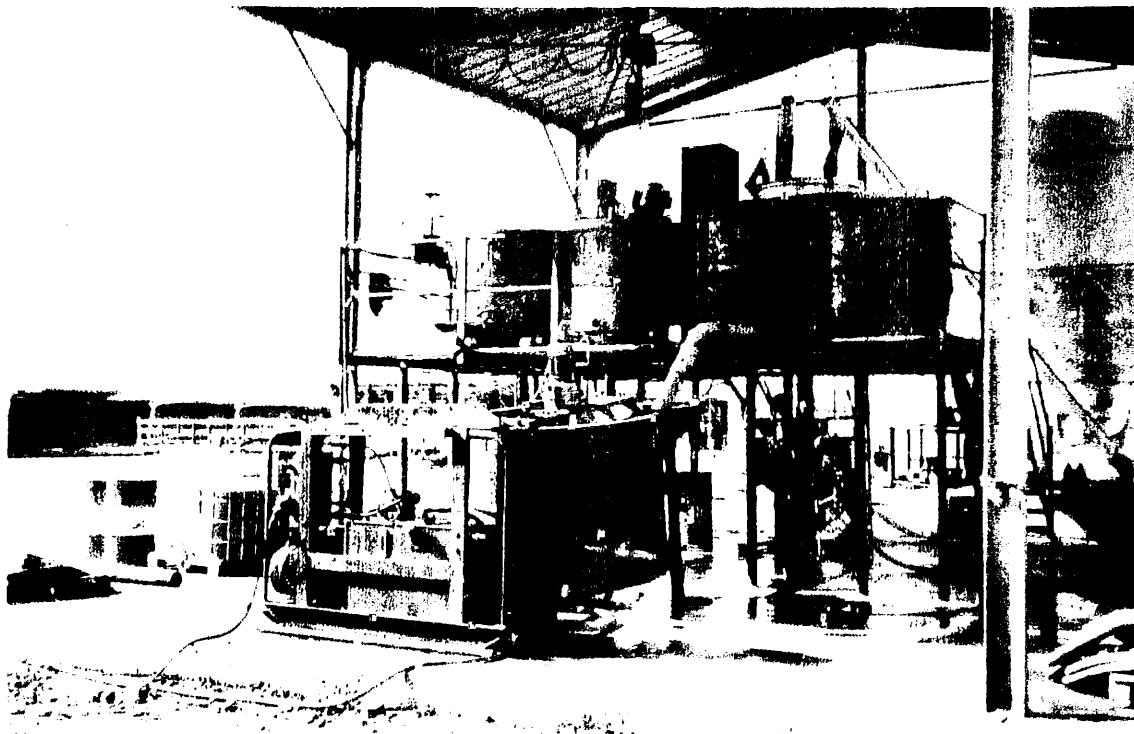
(5) Measurements and observations

The following measurements and observations should be made and recorded:

- description of test set-up
- grout mix designed, origin and certificates of all grout constituents
- mixing procedure of grout
- flow time of grout mix before initial grouting, and before re-grouting (or viscosity of a thixotropic grout)
- method statement for grouting
- measurements of level of air, water and eventual liquid on top of the grout

- any observations and comments on the formation of bleed or liquid, or on difficulties encountered during the test
- development of temperature during the entire test period
- photos illustrating test set-up, and details of top end of tube with air, water and eventual liquid.

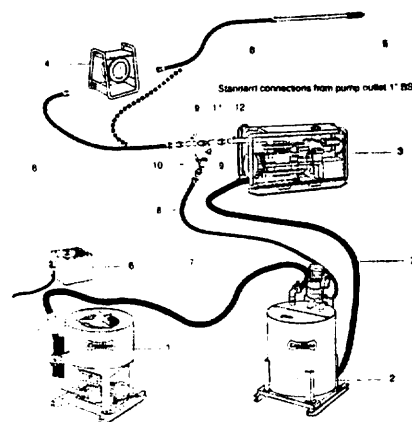
4. Grouting operations



Grouting plant

A proper grout mix, a well-functioning grouting equipment and correct grouting procedure are a prerequisite for a durable corrosion protection of the prestressing steel and the bond behaviour between the tendon and the structure, hence for the long-term integrity of the structure. The grouting operation should only be carried out by qualified and trained personnel, directed by a grouting specialist. Grouting work should be under the responsibility of the prestressing system supplier.

4.1 Equipment



Grouting equipment

4.1.1 Mixer and related components

- Efficient mixing is carried out in a tank sized to correspond to the energy transmitted by the motor to the turbine or the screw. Sufficient power shall be available to correctly shear the entire mass of grout, without depositing cement in the bottom of the tank, and to obtain a colloidal mixture. All manufacturers have adopted the shearing or deflocculating turbine.
- The most efficient mixers are fitted with recycling systems. The grout is pumped across the turbine and discharged under pressure in the top part of the tank.
- The equipment shall be designed to allow feeding of cement continuously so as to prevent the formation of clods of badly mixed cement, and shall comprise a coarse sieving device.
- Water dosage
Water is dosed by means of water counters, pans or by weighing, and the required accuracy shall be ensured. Preferably, the operation shall be carried out automatically, and not manually, so as to prevent errors.
- Cement dosage
When cement is added from bags, the cement manufacturer shall guarantee the accuracy of the indicated weight.
When cement is delivered in bulk, the weighing system shall also ensure that the required accuracy is obtained.
- Mixing time control
It is recommended to fit the mixing equipment with a timer in order to automatically stop the mixer motor, thus preventing any errors.

4.1.2 Agitator and related components

The agitator shall be designed for:

- Storage of quantities of grout greater than the volume of the sheath to inject
- Slow mixing of the grout without creating any air bubbles
- Grout suction by means of the injection pumps pipes without accidentally incorporating any air, even when the grout level is very low (bottom at an angle).

The cement grout shall be conveyed from the mixer to the agitator through a sieve (mesh opening: 2 mm).

4.1.3 The pump and related accessories

The pump shall be designed to inject grout with the least possible amount of jerking. The pump shall be fitted with an adjustable pressure switch allowing for automatic stopping at a maximum injection pressure (generally 15 bars) and precise adjustment of such. Also, it is recommended that the maximum capacity be at least 30 bars.

The equipment shall be designed for injection at variable rates, so as to be able to satisfy working conditions in the best possible manner (injection rate modulated in function of the type of sheath and tendon, slow final pressure increase, etc.). The pump rate selected depends on the selected injection speed. Lastly, to prevent air pockets forming, due to the grout falling in down sections of sheaths, or water forming on the grout feed front, the following values, to within the ranges given, must be obtained;

8 to 14 m/min.	Tendons up to 19 T 15
16 to 20 m/min.	Sections greater than 19 T 15

The pump shall be fitted with:

- an accurate pressure gauge (readout accuracy: 1 bar).
- a valve system preventing grout returning to the suction tank or grout return under pressure in the pump when operations are shut down.
- a drain to gradually drop the pressure
- possibly, a pulse counter to calculate the injected volume

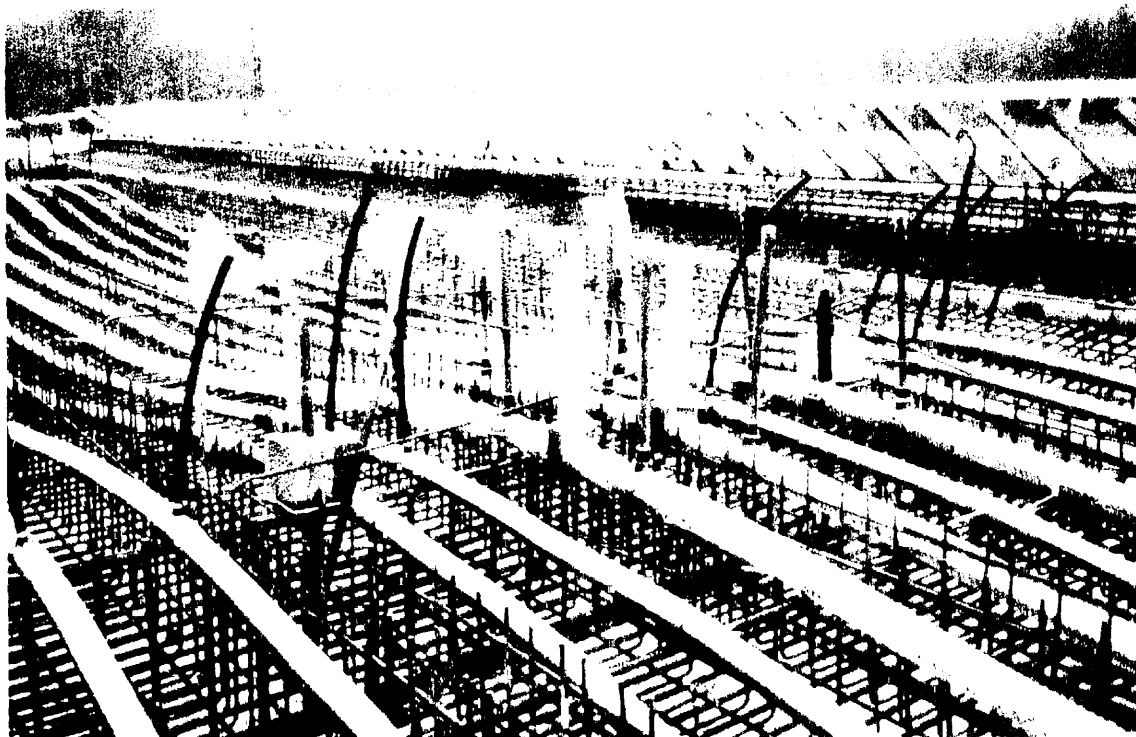
Grouting hoses shall be designed to resist to a 40 bar service pressure.

When the grouting pump is located away from the tendon inlet, it is necessary to install a pressure measurement instrument at the line end. This instrument is fitted with a pressure gauge, two valves and a support.

4.2 Grouting of tendons

4.2.1 Operations prior to grouting

- Checking installation and position of vents and drains
- Sealing of sleeve-duct and trumplate-duct connections
- After approval of elongation results, cutting of stands with a disk-cutter
- Installation of grout caps
- Air tightness test of the ducts
- Required testing



Vents installation

4.2.2 Grouting operations

Grouting of a tendon should always follow the method statement set up for this specific tendon, whether it is:

- an horizontal tendon with little curvature
- an horizontal tendon with important curvature
- a cantilever or a continuity tendon
- a vertical cable

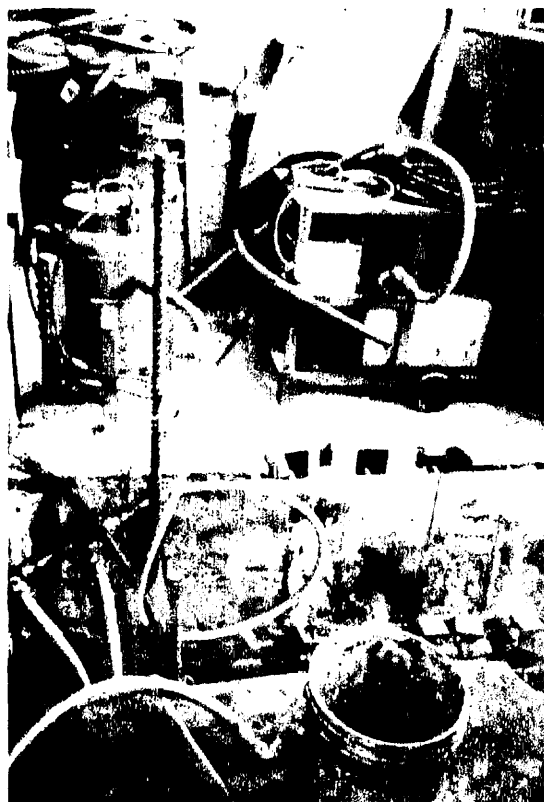
Typical procedure implies:

- grouting until grout is flowing out of the first vent with a consistency similar to the one previously measured at the pump
- progressively close all vents until to the end of the tendon. When proper grout of correct consistency is flowing out of last vent, close the vent, increase the pressure by not more than 3 bars and hold the pressure for about one minute. Then close the inlet.

Special considerations and method statement will be prepared for movable couplers, vertical tendons, horizontal and external tendons when special precautions have to be observed.

4.2.3 Vacuum grouting

The grout is fabricated as previously said; grouting hoses are installed on the anchorage: inlet is closed. Vacuum is carried out from the other end (< 200 mbar); the inlet is opened and the pump is activated. Pumping and vacuum are cumulated; this allows to speed the grouting operations since there is no chance to get air bubbles.



Vacuum grouting

4.2.4 Grouting of vertical tendons

Vertical ducts can be required in bridges, for example as tie-down tendons of unbalanced cantilever and for deep webs in box beams and piers. They may also be used in off-shore structures and nuclear pressure vessels. Vertical grouting presents somewhat different problems and special measures are specified in the code. In fact, we have seen that the physical characteristics of the grout are modified while in contact with the strands. Water is separated by capillarity and absorbed through the voids between the wires: the separated water volume depends on the differential pressure between lower and higher point of the duct. This may induce water and air pockets inside the ducts.

The water carries fine particles of cement and admixtures and after setting and drying, it may remain in these areas some white deposit. Of course, this is more important with vertical or highly deviated cables. In order to solve these difficulties and improve the final quality of the cement grout, several methods have been developed:

(a) Vacuum grouting

Grouting is done after having pumped the air out of the duct; Prior to the operation, an air tightness test is carried out on each duct; This will permit to locate any leaks and select the correct pump flow.

(b) Drainage between wires

At the top of the vertical cable, the drained water is collected in a larger size duct. Then re-grouting is done 2 or 3 hours after the first operation.

(c) Vacuum assisted grouting

After complete filling of the entire tendon with grout (once the air has been evacuated), 4 bar pressure was applied with the grout pump until exudation of water through the strands ceased and no more grout could be pumped into the tendon (also called strand filter effect). This method will only work with standard strand as it allows water to filter through. Compact strand section is too tight for the filtration to be effective.

4.3 Wax and grease injection

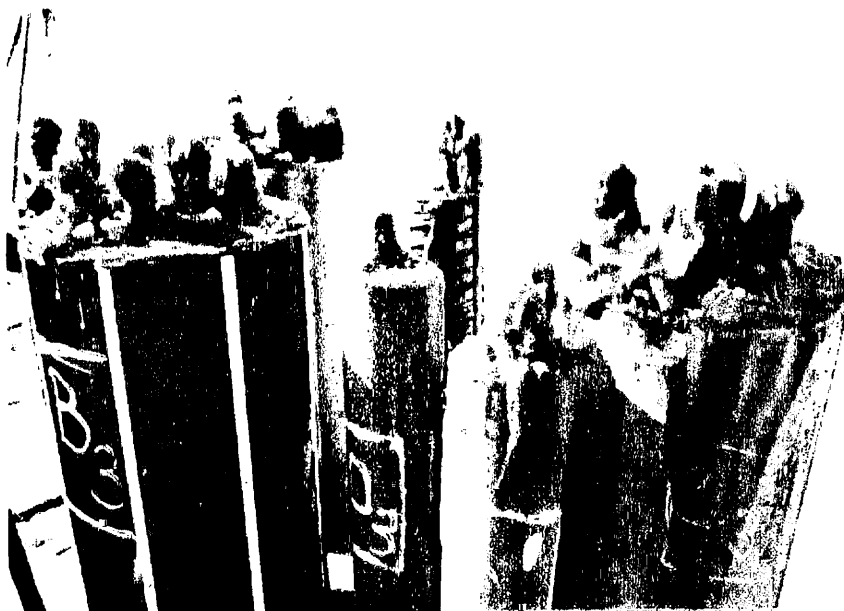
4.3.1. Corrosion protection of unbonded tendons

Unbonded systems have been specified and used whether internal or external to the concrete section. In this case a flexible corrosion protection material is requested. For instance, the nuclear containment in United States consists of prestressed concrete structures with vertical, horizontal and dome tendons. In order to cope with the surveillance programme issued by the Nuclear Regulatory Commission which requests to check at random the tension in the cables over 40 years, grease injection has been used. This allow the lift off operations and the removal of cables.

4.3.2. Materials

Several materials would be considered: polymer, grease, waxes, polyurethane, etc. It is necessary to define the various petroleum products for clarification:

- Gatch: mix of paraffin and waxes including a high percentage of oil (up to 30%)
- Crystalline paraffin: this product is obtained by disoiling of the gatch during crystallisation
- Microcrystalline wax: this product is obtained from heavy components of distillation. Waxes are made of saturated hydrocarbures according to formula $C_n H_{2n+2}$ types with a high percentage of ramified chains and a microcrystallisation. This explains its flexibility, and its adhesive properties to various supports. Some specific additives provide excellent behaviour under extern pressure (fretting corrosion) [INJECTELF]
- Petrolatum: a mix of paraffin and oils
- Vaseline: result of refining and dilution of petrolatum
- Grease: mix of oil and soap with anti-corrosion additives. Wax is today often preferred to grease because of its crystalline structure providing a greater stability: grease bleeds, and then oil leaks.



Petroleum wax

4.3.3 Grease and wax injection

These products have a high viscosity at ambient temperature; they need to be heated in order to get a good injection. Temperature should be 30° C higher than the melting point to take into account the heat exchange with the various materials.

- Grease: 50 to 100° C
- Wax: 90 to 120° C

These products shrink while cooling and this can create cracks and voids. However the wax has a very high wetting capacity which allows it to cover every parts and to be injected rapidly at a lower pressure than grease.

4.3.4 Other products

A series of two components products have been also proposed by various suppliers.

Injection should be carried out with specific pumps. Vents and drains are installed at the same location as for cement grouting. But the tightness of the duct must be perfect (only plastic ducts are today recommended): an air pressure test is requested prior to the injection.

5. Quality control and quality assurance

As previously said, improvements on the current process of grouting post-tensioning ducts cannot be achieved by any single action on its own, but require parallel development in the field of grouting materials (chapters 2 and 3), grouting techniques (chapters 3 and 4) and QA/QC systems.

5.1 Certification of post-tensioning operations and training

It has been recognised that good quality workmanship is fundamental to the production of durable post-tensioned concrete bridges. This requires good procedures and appropriate training. In the past, grouting of ducts has sometimes been undertaken by inadequately trained staff and the importance of good grouting has not been properly recognised on site. Consequently, Certification Schemes are used now in Europe (France, UK).

5.2 Testing

Paragraph 3.1 is describing tests which have to be carried out prior the grouting operation (suitability tests) and during grouting operation (acceptance tests). In fact prior to use, the post-tensioning contractor must assess the grout properties using the materials, material sources, plant and personnel proposed for use on site. Grout preparation must be undertaken under the temperature conditions expected on site. The assessment must be made sufficiently in advance of the grouting operations to allow adjustments to the materials, procedures or equipment.

During grouting, the post-tensioning contractor must have a procedure to control the grouting process. The procedure must include:

- (a) The planning of resources, supervision, material and the attendance required.
- (b) Preparation of the duct
- (c) Production of grout including the grouting equipment, admixtures and grout components
- (d) The injection of grout and venting of the duct.
- (e) An assessment of the amount of cement/grout.
- (f) Corrective action in the event of blockage or breakdown
- (g) Grouting in cold weather and grouting in hot weather.
- (h) Tests for the measurement of the grout properties including: fluidity (immediately after mixing and at the end of the injection period), volume change, bleeding, segregation and compressive strength.
- (i) Sealing of ducts and vents.

Minimum test requirements - as per the TR 47 Rev. 2 - are shown in the following table:

1. Suitability Testing	
Fluidity	Samples immediately after mixing, one test. After estimated time to grout duct or minimum of 90 min from initial mixing. Two tests averaged in both cases
Bleed Volume change Sedimentation Strength	Each sampled immediately after mixing, 3 tests averaged.
2. Acceptance Testing	
Fluidity	Sampled immediately after mixing, one test from mixer. After flow through duct, one test from each anchorage outlet. On completion, one test from mixer.
Bleed Volume Change	One test per day or none per 1.5 m ³ of grout, unless specified otherwise.

Strength	
Sedimentation	One test per day for site-mixed grout, or one test per pre-bagged supplied batch (by manufacturer's reference number); subject to a minimum of one test per continuous grouting operation.

'Minimum test requirements for Grout (TR 47 Rev. 2)

5.3 Monitoring

After grouting, it is difficult to inspect visually internal cables. This means that reliance must be placed on indirect methods to confirm the adequacy of the corrosion protection system. Different tests and monitoring systems have been suggested:

Stage when tested	Feature under test	Test method
Before grouting	Sealing of duct	1. Duct leakage under pressure
During grouting	Progress of operation	2. Pressure measurements
	Voids within and above grout	3. Grout stiffness test
After grouting but before acceptance	voids above grout	4. Gamma & Radiography/Radioscopy 5. Impulse radar 6. Impact echo 7. Ultrasonic transmission 8. Ultrasonic reflection 9. Tomography 10. Thermography 11. Pressure/volume/leakage testing
During service life	Degree of corrosion risk	12. Half-cell potential sensors 13. Resistivity sensors 14. Electrical continuity measurement 15. Corrosion cell sensors 16. Polarization measurements
	Tendon integrity	17. Fiber optic integrity 18. Acoustic emission 19. Ultrasonic pulse 20. RIMT electronic pulse
	Effects of prestress loss	21. Magnetic flux exclusion 22. Strain by fibre optic sensors 23. Strain by VWG or Demec 24. Vibration monitoring 25. Magnetoelastic stress monitoring 26. Smart-drilling & endoscopy

'Testing techniques in relation to QA, QC and Monitoring of prestressing elements (BRPR-Brite euram Q/A of grouting - Dec 1999)'

6. Conclusion and perspective

This paper has summarised the remarkable work which has been done by owners, engineers and specialised contractors during the last five years, to improve the durability of post-tensioned tendons. As previously said such a result cannot be obtained by any single action on its own, but it requires parallel development in the design, the post-tensioning technologies, the grout and grouting and the overall construction. As far as the grout materials and grouting operations are concerned, the achievements are as follows:

- Development of high performance grout with no bleed, no shrink, pre-tested, sometimes pre-bagged
- Use of thixotropic grout
- New bleeding and volume change suitability tests (inclined tube in France) required
- Trained workmanship and certification of specialised contractors required
- Grout, grouting procedure and grouting equipment are acting together
- Interest of vacuum grouting
- Improved quality assurance, testing and monitoring

7. REFERENCES

- (1) fib (CEB-FIP) Recommendations (October 1998), *'Qualification and approval of prestressing contractors and system suppliers'*.
- (2) prENV,3670 (July 1999), *'Execution of concrete structures'*.
- (3) fib TG 9.8, *'Guidelines for grouting' Draft 2*.
- (4) PTI 2000, *'Specification for grouting of post-tensioned structures'*.
- (5) Lapsley, R.D., (September 1996), *Grouting of Post-Tensioned Structures - Recent Advances in Vacuum Grouting Techniques and Grout Mix Design'* - fib Symposium, London.
- (6) *PTI Guide Specification- Acceptance Standards. for Post-tensioning Systems'* (1998), Post-Tensioning Institute, Phoenix. AZ.
- (7) Gulyas, Robert J., Sprinkle, Michael M. (2001), *'Laboratory Evaluation of Duct Grouts for Precast and Cast-in-Place Prestressed Concrete Members'* - Precast/Prestressed Concrete Institute Journal
- (8) Schokker, A.J. (May 1999), *'Improving Corrosion Resistance of Post-Tensioning Substructures Emphazing High Performance Grouts'*, Ph. D. Dissertation, the University of Texas at Austin.
- (9) Schokker, A.J., Koester, B.D., Breen, J.E., and Kreger, M.E. (December 1999), *'Development of High Performance Grouts for Bonded Post-Tensioned Structures'*, Research Report 1405-2, Center for Transportation Research, Bureau of engineering Research, the University of Texas at Austin.
- (10) West, J.S., Vignos, R.P., Breen, J.E. and Kreger, M.E. (October 1999), *Corrosion Protection for Bonded Internal Tendons in Precast Segmental Construction'*, Research Report 1405-4, Center for Transportation Research, Bureau of engineering Research, the University of Texas at Austin.
- (11) Sprinkell, Michael M. (October 1997), *'High Performance Silica Fume Grout for Post-Tensioning Ducts'*, International Symposium on high Performance Concrete, New Orleans, Louisiana, FHWA, pp. 343-354.

- (12) de Cuyper, J., Stouffs, A., and Tilly G.P. (august 1999), *Assessing the stability of grout*
- (13) Tilly, G.P. and Woodward, R.J. (1996), development of improved grouting for post-tensioned bridges, *'Post-tensioned concrete structures 1996'*, Proc. FIP Symposium.
- (14) British Standards Institution (1996), *'Grouting for prestressing tendons - test methods (BS EN 445/1996), Grouting for prestressing tendons - grouting procedures (BS EN 446.1996)'* and *'Grouting for prestressing tendons - specifications for common grout (BS EN 447/1996)*.
- (15) Fédération Internationale de la Précontrainte (1990), *Groutings of tendons in prestressed concrete*, FIP Guide to good Practice, Thomas Telford, 16 pp.
- (16) Federal Highway Administration, Germany (1998), *guidelines for Concrete Bridges with External Tendons*.
- (17) Highway Agency, TRL, SETRA and LCPC (1999), *'Post-Tensioned Concrete Bridges'*, Thomas Telford, London.
- (18) Fédération Internationale de la Précontrainte (1996), *'Materials and systems for External pre-stressing*.
- (19) Chaussin, R., Chabert, A., Cussigh, F. (1998), La maîtrise des problèmes d'injection des câbles de précontrainte, *la technique française du béton précontraint*, AFPC-AFREM, XIIIème congrès FIP. pp. 91-98, Amsterdam.
- (20) Luther D.C., Huynh, T.L., Mellier, P. (1996), the Hibernia offshore platform, *'Comptes rendus du symposium FIP'*, Londres, pp. 82-91.
- (21) Note n° 21 (1996), Note d'information LCPC/SETRA n° 21
- (22) Norme NF EN 445 (Mai 1996), *'Coulis pour câbles de précontrainte'*.
- (23) Ministère de l'Équipement, des Transports et du Logement (Septembre 1999), *'Circulaire n° 99-54 du 20 août 1999 instituant un avis technique des coulis d'injection pour conduits de précontrainte, délivré par la commission ministérielle de précontrainte'*.
- (24) Chaussin, R., Chabert, A. and Jartoux, P. ((1996), *' Prestressed Concrete - Quality and Durability'*. FIP Symposium, London.
- (25) Ganz, H.R. (1996), *'Plastic ducts for enhanced performance of post-tensioning tendons'*. FIP Symposium, London.

Latest Technology Applied to The Pre-Grouted Prestressing Strand

Toshihiko Niki

Manager, PC Engineering Group, PC Department, Special Steel Wire Division
Sumitomo Electric Industries Ltd., Iiyogo Japan

Yoshihiko Touda

PC Engineering Group, PC Department, Special Steel Wire Division
Sumitomo Electric Industries Ltd., Hyogo Japan

1. Introduction

The Pre-Grouted Prestressing Strand is composed of prestressing steel coated with special epoxy resin and covered with a corrugated plastic sheath, as shown in Fig. 1. With controllable hardening time, initially it behaves as an 'unbonded' prestressing strand and finally behaves as a grouted (bonded) prestressing strand. ⁽¹⁾

The Pre-Grouted Prestressing Strand provides (A) anti-corrosion properties, due to the epoxy resin and the seamless polyethylene plastic wrapped around the bare material, (B) effective site work by elimination of the grouting process, which sometimes creates serious problems in quality, through improper grouting. After several evaluation programs, the Pre-Grouted Prestressing Strand has been applied to various concrete structures and is now regularly applied to transversal tendons for prestressing bridge decks. In this context, it was considered desirable to apply the Pre-Grouted Prestressing Strand to the main cable (longitudinal tendon) of the bridges girders as an alternative to the conventional grouting process. Normally the main cable should contain several temperature zones - in the massive concrete section, for example, the concrete generates excessive heat due to a hydration reaction during the concrete cure. To apply the Pre-Grouted Prestressing Strand to the main cable safely, new technology had to be developed to reduce the temperature sensitivity of the product.



Fig. 1 Pre-Grouted Prestressing Strand

This report discusses the latest technology applied to the Pre-Grouted Prestressing Strand; specifically, the 'Moisture Reactive Resin Technology' and its evaluation test results.

2. Development of the new Pre-Grouted Prestressing Strand

2.1 Application to Main Cables

During the curing process, prestressing materials placed in the massive concrete area may be heated up to 90°C, which exceeds the temperature range required to control the hardening time of the conventional Pre-Grouted Prestressing Strand (Fig. 2). To apply the Pre-Grouted Prestressing Strand to the main cable (which normally contains a massive concrete zone), the resin used in the Pre-Grouted Prestressing Materials has to exhibit low temperature dependence.

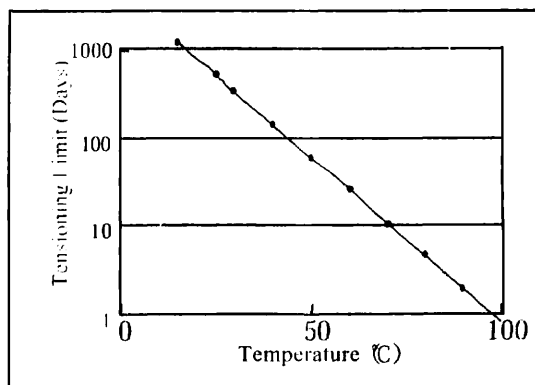


Fig. 2 Influence of temperature on Tensioning limit of the conventional resin (Samples exposed to constant Temperature)

2.2 Moisture Reactive Resin Technology

To reduce temperature dependence, 'Moisture Reactive Resin Technology' was introduced in the new Pre-Grouted Prestressing Strand. Generally, when an epoxy resin hardens, molecules of epoxy form 3-dimensional structure with a support of a hardener that works as their 'connector'.

As shown in Fig. 3, the developed resin contains the hardener that originally sealed by 'capsule'. As the Step-1, the each capsule is gradually removed by small quantity of moistures contained in the epoxy resin. This process is not influenced by temperature. The desiccant material contained in the resin absorbs extra moisture, which may come from the strand end or penetrate through the polyethylene, possibly accelerating epoxy curing. In Step-2, the epoxy molecule starts bridging (hardening) by the hardener. The conventional resin relied only on this process.

With 'Moisture Reactive Resin Technology', even prestressing strands are exposed to an environment of 100°C, the tensioning work can be done for up to 10 days as shown in Fig. 4.

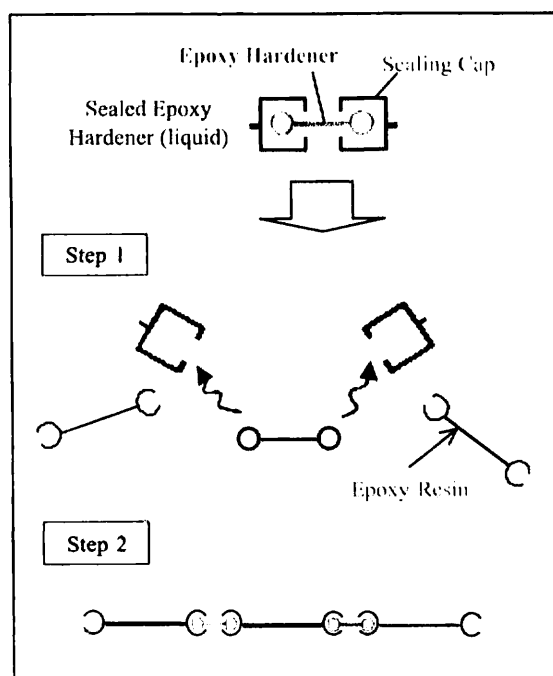


Fig.3 Hardening mechanism of the new resin

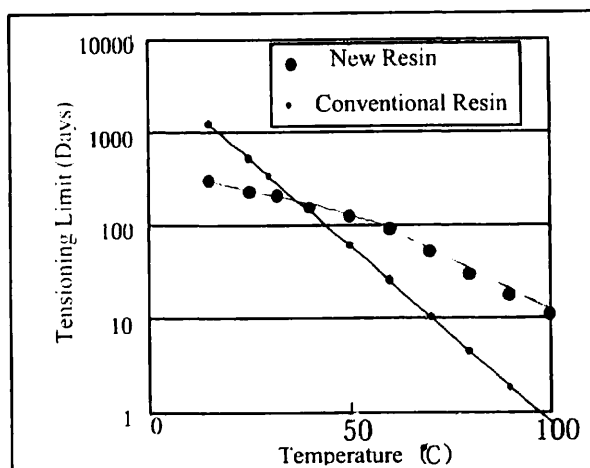


Fig. 4 Influence of temperature on tensioning limit of the new resin and conventional resin. (Samples exposed to constant temperature.)

2.3 Estimated Tensioning Limit and Complete Hardening time in Actual Situations

Fig. 4 shows the tensioning limit when the Pre-Grouted Prestressing Strand is exposed to constant temperature. During actual bridge construction, after the temperature reaches its highest point, it then decreases to the outside temperature. Table 1 shows the estimated tensioning period limit and the complete hardening time as applied to the typical range of temperature in actual bridge constructions. These estimations are based upon the laboratory test result.

	Maximum Concrete Temp	Average Outside Temp.	Tensioning period Limit	Hardening Period
Massive Concrete at hot region	95°C	28°C	1 Month	1-1.5 Years
Slab Concrete at cold region	75°C	8°C	3 Months	2 Years

Table 1 Estimation of Hardening Time at Actual Use

3. Practical tests

3.1 High Temperature Situation - Massive Concrete at hot area

To verify the estimation (Table 1), a substantial test was performed, as shown in Fig. 5. To

make a situation where the test is performed in a hot region, the special testing laboratory (Fig. 6) was constructed in Oyama-City, Tochigi Prefecture. The temperature inside laboratory was kept 30-40°C by the air conditioning equipment and the concrete temperature was as shown in Fig. 6. The test shows that 21 days after concrete pour, 100% of the stress applied at one-end was transferred to the other (fixed end), which means that stressing work can be performed for at least after 21 days even in high-temperature situations.

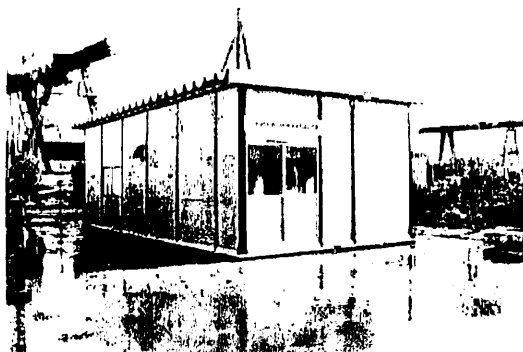


Fig. 5 Laboratory for the Substantial Test

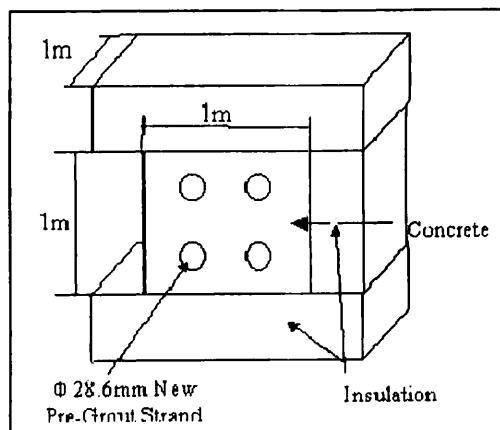


Fig. 6 Test Specimen

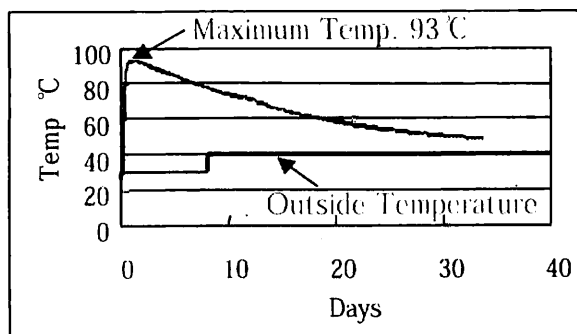


Fig. 7 Applied Temperature Pattern

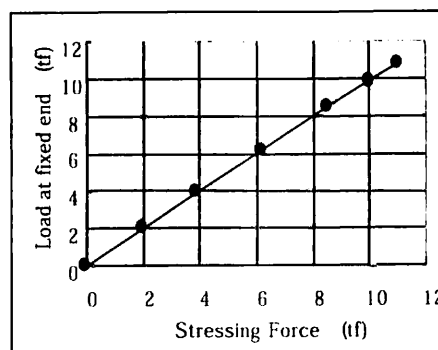


Fig. 8 Stress Transfer to Fixed End 21 Days after Concrete Pour.

3-2 Low-Temperature Situation - Slab Concrete at cold area

It was confirmed that the new Pre-Grouted Prestressing Strand with Moisture Reactive Resin provided enough period till stressing work after a concrete pour in a high temperature condition. The next step was to verify the complete hardening period in a low-temperature condition. An actual test was performed in Hokkaido, in the northern part of Japan, as shown in Fig. 9 and Fig. 10.

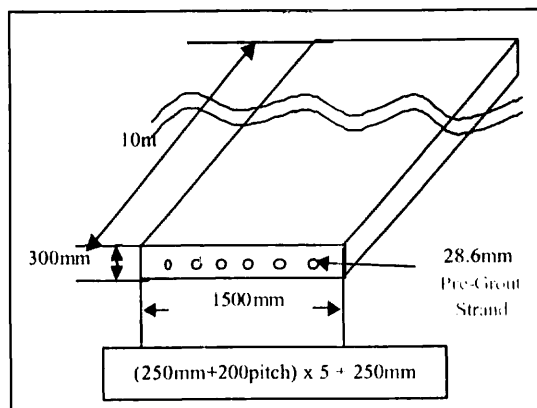


Fig. 9 Test Specimen – Actual Slab Concrete

The load transfer rate from the stressing end to the fixed end has been measured each week since the concrete was poured. It was observed that load transfer offset started to occur 11 months after concrete pour, and that no load transfer was measured after 12.5 months. Although in the laboratory (hardness measurements), the resin has not hardened completely yet (as of August, 2001, after 12.5 months), the resin can obviously harden completely within 24 months, as estimated.

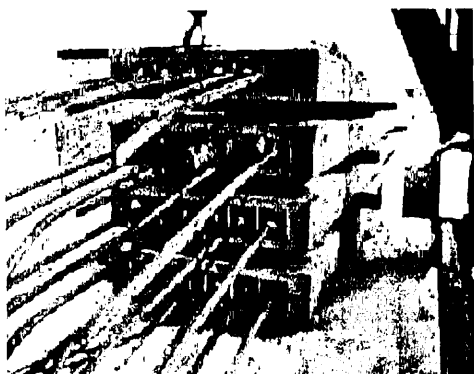


Fig. 10 Test Specimen in the Snow

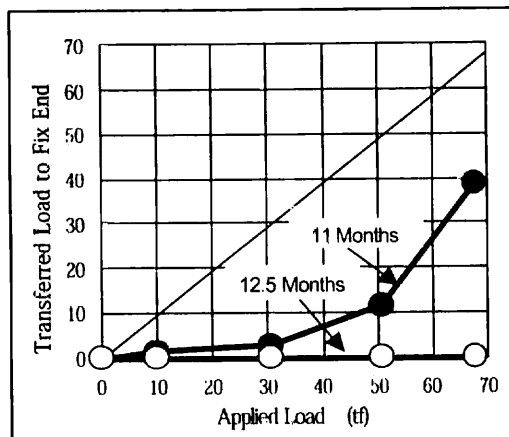


Fig. 11 Stress Transfer to Fixed End

3.3 Friction Coefficient

Friction (Wobble) coefficient per unit length of prestressing strand was measured under the following test conditions;

Prestressing Tendons:

270K 28.6mm-diameter Pre-Grouted Prestressing Strand; both a conventional type and the new type with Moisture Reactive Resin Technology.

Measurement Length:

30m. Stresses are applied at one-end (jacking end) and the transferred load was measured at the other end (fixed end).

As the result, the friction coefficient value of the new Pre-Grouted Prestressing Strand was 0.0020-0.0023, while the conventional one exhibited a value of 0.0022-0.0029. The new Pre-Grouted Prestressing Strand, as well as the conventional Pre-Grouted Prestressing Strand, have values identical to that of the 'Unbonded Prestressing Strand'.

4. Mechanical Property after Complete Hardening of Resin

4.1 Compression Strength

Compression strength after the resin hardened completely was measured. As shown in Fig. 12, the new Pre-Grouted Prestressing Strand exhibited better compression strength than that of the conventional one.

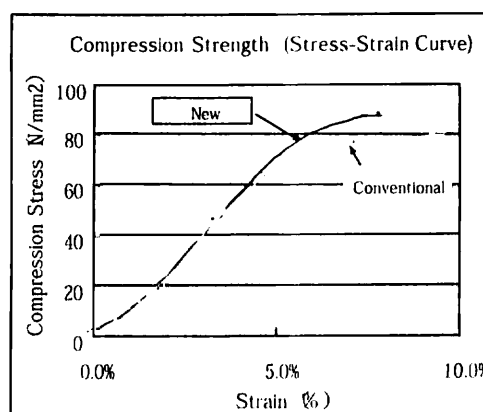


Fig. 12 Compression Stress Strain Curve

4.2 Bond Property

Comparing the bare strand to the conventional one, the bonding properties between the prestressing strand and the concrete were evaluated, by a 'Pull-Out Test'.

As shown in Table 2, the bonding ability of the new Pre-Grouted Strand exceeds those of the conventional strand and the bare strand.

Specimens	Max Pull-Out Load (KN)
New Pre-Grouted Strand 28.6 mm Dia.	158.2
Conventional Pre-Grouted Strand 28.6 mm Dia.	131.9
Bare Prestressing Strand 28.6mm Dia.	130.4

Table 2 Pull-Out Test Results

4.3 Anti-Corrosion Property

Anti-corrosion property of the new Pre-Grouted Prestressing Strand was evaluated by the Salt Splay Test. Specimens were exposed for 3,000H to 5% NaCl solution. Fig. 13-15 shows the result after 3,000H exposure to the Salt Spray Test; no evidence of corrosion was observed.

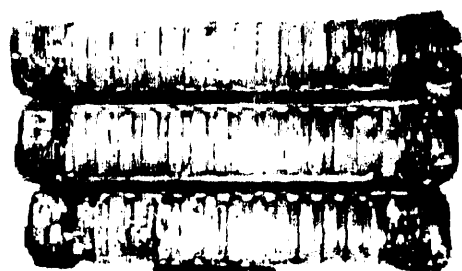


Fig. 13 Overview of Specimens After 3000H exposure

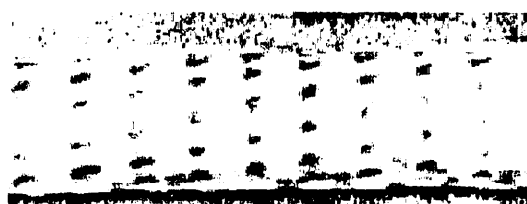


Fig. 14 Inside of polyethylene sheath

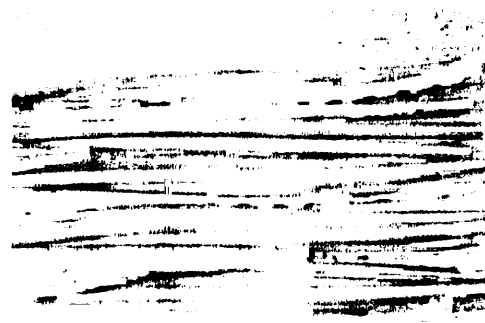


Fig. 15 Condition of steel surfaces

5. Application to Actual Constructions

As of August 2001, the Pre-Grouted Prestressing Strand has been applied to the main cables in the actual constructions at Ichinomiya Kouka Bridge, Murahigashi Daini Kouka Bridge, Zaikeduka Kouka Bridge, Okoba River Bridge, Nobeno Kouka Bridge, Iburodani Bridge, and Nobeno Kouka Bridge. Fig 16 and 17 show views of these construction sites.

To avoid damages to the polyethylene sheath, special attention is paid to using tools such as turn table for pay-off (Fig. 18), guide rolls for installation pass line, and the like.



Fig. 16 Site View of construction site-1

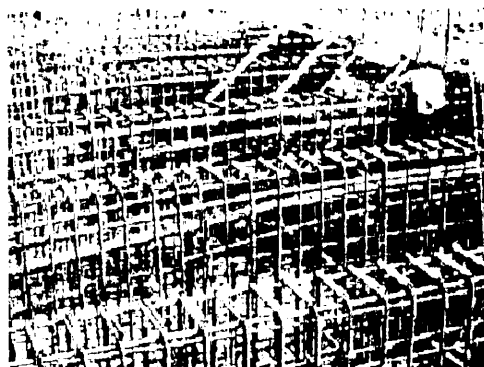


Fig. 17 Site View of construction site-2

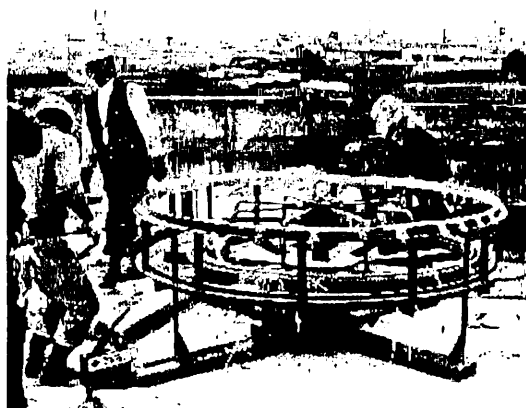


Fig. 18 Pay-Off Turn Table

6. Conclusion

- (1) The 'Moisture Reactive Resin Technology' was developed for application to the Pre-Grouted Prestressing Strand.
- (2) Laboratory tests and practical tests have been conducted on the new Pre-Grouted Prestressing Strand. It was confirmed that the new resin allows one month for the

tensioning period in the highest-temperature situation, and that resin hardens completely within two years in the lowest-temperature situation.

- (3) It was also confirmed that the new Pre-Grouted Prestressing Strand exhibits mechanical properties equal to or better than those of the conventional one.
- (4) Because of its low temperature-dependence, the new Pre-Grouted Prestressing Strand can now be applied to the main cables in bridge constructions.
- (5) The Pre-Grouted Prestressing Strand, which is produced at a manufacturing plant under severe quality control, eliminates the risk of improper grouting at construction fields.

Reference

- (1) Development of 'After-Bond' Prestressing Steel, *Kazuo Suzuki, Osaka University, Takeshi Kobayashi, Sinko Wire Company Ltd., Toyokazu Minami, Shinko Wire Company Ltd., FIP – XIth International Congress on Prestressed Concrete, June 1990.*

Appropriate detailing in the design process for durable post-tensioned bridges

M.E. RAISS

Dr., Managing Director, Benaim (UK) Ltd

1 INTRODUCTION

The durability of post-tensioned concrete bridges depends on a number of factors including:

- the overall approach to design for durability
- the use of appropriate design details,
- the specifications for duct and grouting systems, and
- a quality assurance scheme for supply and installation of post-tensioning systems.

This paper considers the approach to design for durability and the use of appropriate design details. It is based on the work of the UK Concrete Society working party which drafted the Concrete Society Technical Report TR47 "Durable Bonded Post-Tensioned Concrete Bridges" [Concrete Society (1996)]. When TR47 was originally published it led to the lifting of the temporary ban on internal grouted bonded construction in the UK. It is currently being up-dated in the light of experience gained in the last 5 years.

2 DESIGN FOR DURABILITY : MULTI-LAYER PROTECTION

A full discussion of the mechanisms of deterioration of prestressed concrete structures is beyond the scope of this paper, but the main concern with the prestress cables is external chlorides penetrating to the steel, from sources such as de-icing salts or seawater. It follows that, in developing a design strategy, the nature and intensity of the aggressive actions – and how they might penetrate to the steel – is of fundamental importance. This applies both to conceptual design and to the evolution of design details. The transport mechanisms for chlorides are much influenced by the combined effects of wind, water and temperature, in both ambient and micro-climate terms. The provision of resistance to those requires an integrated approach, involving:- design concept, detailing, construction quality and material selection. The importance of this cannot be over-emphasised.

In recognition of the difficulty of achieving perfect protection by any individual means, TR47 recommends a multi-layer protection approach to the design for durability of post-tensioned construction. This has been used for ground anchorages and requires the provision of a number of protective measures, on the basis that the total integrity of one layer will maintain the integrity of the whole even if another one of the layers of protection becomes partially ineffective.

In preparing the durability strategy for a bridge the following items should be considered:

- a) The bridge location and exposure conditions. Chloride-induced corrosion is one of the major concerns in concrete bridges, wherever the structure is subject to road salts or wind-borne chlorides.
- b) Structural articulation with a view to minimising movement joints and provision of structural continuity, possibly in the form of integral bridges.
- c) Access for inspection, testing, maintenance, and possible replacement of short-life elements.

- d) Type of cross-section, and its shape, particularly at its boundaries, to minimise the risk of contaminants penetrating the structure.
- e) The method of construction, with its associated buildability and workmanship factors.
- f) The overall layout of the prestressing tendons to suit the methods of construction and maintenance. A main decision here is whether the tendons are external, internal or a mixture of the two systems.
- g) Location, detailing and protection of the anchorages.
- h) Detailing the deck waterproofing system and drainage design to avoid ingress of water - the normal transport mechanism for contaminants. This needs to consider the expected life for the systems components, and the risks associated with component failure.
- i) Specifications for the quality of the concrete and its cover.
- j) Specifications for the duct system – corrosion-resistant duct material designed to exclude contaminants.
- k) Filling the ducts with cement grout or wax as appropriate.
- l) Quality assurance and testing during construction.

3 AVAILABLE PROTECTIVE MEASURES

There are a large number of layers of protection available which may be summarised as given in this section. In this context a layer of protection in anything whose purpose is to ensure the overall long-term integrity of the prestressing cables. Hence methods of inspection (and subsequent maintenance) are layers of protection, just as much as grouting of the ducts.

3.1 General

There is much that can be done, both quantitatively and qualitatively, assuming that the major threat is corrosion due to chlorides. In 'load' terms, the source of chlorides can be either de-icing salts or seawater. To reach the bridge, transport mechanisms are required. In general climatic terms, this involves a combination of water and wind. In local exposure terms, water can interact with the outer surfaces, in terms of vapour, spray, driven rain, run off or ponding. The effect of that interaction can be exacerbated by the influence of temperature, causing joints to open or cracks to form.

Designers therefore need to carefully consider the location of the bridge, what local conditions can form, and how these interact with the outer surfaces. A prime concern is to minimise the uptake of water, and to get rid of what water does reach the bridge, as quickly as possible. This involves a combination of conceptual design, structural detailing and attention to bridge 'finishings', such as drainage, waterproofing and surfacing. In extreme cases, there may be a case for controlling local conditions via external barriers. There is certainly a case for looking carefully at both the profile and texture of the outer surfaces.

3.2 Access for Inspection and Maintenance

One of the main concerns with regard to internal prestressing systems is the inability to inspect the tendons visually. It should be remembered however that external prestress can usually only be inspected in the straight sections between anchorages and deviators and then only if it is not enclosed in a grouted duct; however, tapping of external ducts can be helpful in detecting voids. Enclosure of external prestress within a duct with holes for inspection using an endoscope is possible. The concern is that without being able to inspect the tendons, corrosion may proceed undetected and lead to collapse without warning. One of the advantages, however, of internal prestress is that the concrete itself forms one of the layers of protection. Although it is not possible to see the tendons, the use of non-metallic ducts can facilitate inspection using radar and other non-destructive techniques. Radiography can be used to inspect tendons within metal ducts, but is less convenient than radar and has safety implications.

Access for inspection and maintenance should be regarded as an essential element in the multi-layer protection strategy, and should always be provided. In particular, inspection galleries should be provided, so that anchorages (and their protective systems) can be inspected; provision is also required at or near to key locations such as deviators and joints. These key elements should feature strongly in any inspection checklist, together with checks on changes in moisture conditions, caused (say) by failed expansion joints or blocked drains. Maintaining the exposure condition assumed in design is an important element in management and maintenance.

It may be possible to partially offset the difficulty of inspection of internal cables and anchorages by the installation of sensors or probes which allow remote monitoring of the potential for corrosion at the location of the probe. The TR47 Working Party has not yet adequately investigated the usefulness and reliability of such probes and cannot yet recommend their use. However, it is anticipated that with some development they may be used as an additional assurance, if required, by giving advanced warning of any potential corrosion.

3.3 Materials and Components

The quality of materials and components are of great importance, and therefore the derivation of good specifications is crucial. This should be done with a clear idea of performance requirements, and of a methodology to ensure that the chosen items do in fact comply.

3.4 Construction Quality

Poor workmanship and construction defects are major issues which strongly influence the level of durability actually achieved. A good example of this, in the past, was ineffective grouting for post-tensioned work. However, the issue is wider than that, ranging from poor compaction and failure to achieve specified covers to cases where joints are poorly made (either in the structural elements themselves, or in fitting together the various pieces of hardware involved in prestressing operations). Substantial loads and forces are involved in casting and stressing prestressed concrete bridges – often involving significant pressures and strains. There is therefore a design element involved in this, in ensuring that temporary conditions during construction are properly considered, and in deriving details which enable materials and components to be fitted together on site.

Of particular interest to post-tensioned bridges is the quality of the prestress system installation, stressing and grouting. A detailed specification is given in TR47, which requires all critical operations to be undertaken under a Quality Assurance Certification scheme, and this has been an essential part in the increase in attention to the all important details of many of these operations.

3.5 Bridge Deck Waterproofing Systems

The waterproofing system is the first line of defence against ingress of road salts applied from the bridge road surface. Unfortunately there are no systems available which can be guaranteed to remain waterproof throughout the life of the bridge. It is understood that the range of modern high quality liquid-applied membranes are likely to be more effective than earlier systems. These membranes can be applied in either one or two coats; however applied, they should be proved using ‘pin-hole’ detection equipment which will give reasonable assurance of the integrity of the membrane. Careful preparation of the surface of the concrete and application of the membrane are important, and checks should be carried out for adhesion and thickness.

3.6 Drainage

It is essential that the drainage system should work efficiently to remove water from the road surface as well as the water which passes through the surfacing down to the bridge deck waterproofing system. The details of the drainage paths should be such that if items of the

equipment fail, leak or become blocked, then the water does not find access to the prestressing system. Further advice is given in Department of Transport (1995).

3.7 Expansion Joints

A high proportion of expansion joints leak and their effectiveness and life span are very dependent on the quality of installation and maintenance. Wherever possible consideration should be given to the use of continuous bridge decks and integral abutments to eliminate expansion joints, and hence reduce the risk of contaminants reaching sensitive parts of the structure.

Where expansion joints are used, provision should be made for inspection of the joint and the structure underneath the joint, and the details should be based on the assumption that the joint will leak and will not provide protection against ingress of water and road salts [Department of Transport (1994a)]. Appropriate drainage paths for the leakage should be provided which ensure that it cannot get access to the prestress anchorages or the bearings and that the water is not allowed to pond. This is especially important if intermediate joints have to be located over piers, in ensuring that drainage paths are kept clear of anchorages, because here it is often difficult to provide an inspection gallery.

3.8 Coatings

The use of surface treatments on concrete will provide a protective barrier against aggressive agents [Concrete Society (1997)]. In selecting a surface treatment, whole life cycle performance should be taken into account as the cost considerations of application, maintenance, expected life and possible re-application can be significant.

3.9 Structural Design

The structural design should be to an internationally recognised Code (eg BSI (1990)) and other authoritative guidance documents. These give guidance on stress levels, and the control of cracking, both at early ages and in service due to the influence of loads, creep and temperature.

3.10 Cracking

Cracking in concrete can occur for a number of reasons [Concrete Society (1992)]; its relevance to durability is largely related to corrosion, and depends on the type and magnitude of the cracks [Concrete Society (1995)]. Care is required, when considering the layout and sequencing of concrete pours and prestressing, to minimise the risks of cracking particularly in the vicinity of anchorages. The application of a low initial prestress at an early age can help counteract early-age cracking. The reinforcement provided in the direction of the prestress is usually much less than that used in reinforced concrete bridges and should be checked for adequate distribution of early age cracking [Department of Transport (1987)].

Cracks parallel to, and on line with, the ducts – due, say, to transverse bending in reinforced sections, or to thermal effects at significant changes in cross-section – can occur, and may require consideration, as potential planes of weakness. Such cracks may either be limited by design of reinforcement, or, alternatively, an extra layer of protection introduced into the multi-layer protection system. Cracks at right angles to ducts are less likely to be critical, in terms of affecting the integrity of the ducts in durability terms, provided that their widths are limited in accordance with normal design practice.

3.11 Construction Joints

Well-made construction joints should not leak, particularly when protected by waterproofing membranes. However, waterproofing membranes often do not provide a complete seal, and do not last indefinitely, and joints leak. It is therefore advisable to keep construction joints in deck slabs away from anchorages and prevent, by means of drips, any access for the leakage to the anchorages. If possible, joints in ducts should also be kept away

from construction joints. Where, in sequential or segmental construction, the prestressing anchorages are inevitably located at construction joints, care should be taken in detailing. The whole emphasis is not to create planes of weakness, which permit easy access to water (spray, run-off, driven or ponding) as a transport mechanism for contaminants, and to detail protection for the anchorages, which prevents ingress of water.

3.12 Concrete Quality and Cover

Internationally recognised Codes and Specifications (eg BSI (1990), Department of Transport (1992)) give recommendations for minimum concrete strength and cover requirements for post-tensioned concrete bridges, and concrete specifications which will give a reasonably dense, impermeable concrete protection to the ducts. Such documents are continually under review and it has been proposed recently in the UK that the cover to the ducts be increased by 10mm [Department of Transport (1995)]. Whilst this may be considered, feedback from service has demonstrated that the specified cover is not always achieved in practice; good quality control is essential.

As with all concrete structures, it is possible in special circumstances to improve the concrete protection by increasing the cover or reducing the permeability of the concrete. However, increasing cover would often require increased section thickness and increased prestress adding overall weight and cost to the structure. Reducing the permeability of the concrete is possible by reducing the water/cement ratio or by cement replacement using either pulverised fuel ash or ground granulated blast furnace slag. Cement replacement can also have other benefits such as reducing the heat of hydration (and consequent cracking) and improving the workability and finish – although some concerns have been reported in Belgium and France, with the interaction of high slag cements and high tensile steel. However, the author is not aware of any specific cases of such problems.

3.13 Prestress type – internal or external

It is not possible here to fully explore the relative merits of internal and external prestress. A brief summary of the differences is as follows:

Advantages of external prestress:

- Generally offers a reduction in self weight, and makes concrete placing easier in the webs
- Permits simpler tendon layouts
- Reduces prestressing losses, especially due to friction
- Facilitates inspection, restressing or replacement
- Gives early warning of failure
- Reduces importance of cracking in the concrete in terms of corrosion protection for the tendons.
- Can permit more rapid construction in certain circumstances.

Advantages of internal prestress:

- The available eccentricity of prestress is generally greater
- The tendons reach higher stresses at ultimate limit state, requiring less additional untensioned reinforcement.
- Increased protection of the tendon by the structural concrete.
- The structural role of anchorages and deviators is less critical in the long term performance
- The safety and security of the prime structural members is less at risk - in relation to vandalism for example
- Internal prestressing systems can be cheaper.

From the point of view of durability, the main differences are the ability to inspect, re-stress and replace the tendons in external prestress, and the added protection given to the tendons by the concrete surround in internal prestress. Otherwise the layers of protection are similar in principle. Further details of available protective measures are given in FIP (1996). The following specific requirements are necessary for designs incorporating external prestress in the UK [Department of Transport (1994b)];

- the prestressing and protection system should permit easy inspection, and tendons should be capable of being re-stressed or replaced, if necessary.
- the failure of either two external tendons or 25% of the external tendons at one section should not lead to collapse.

The choice of re-stressable and replaceable or only replaceable has a significant impact on the design of external prestressed structures, as to whether provision is made to de-tension by jacks and whether the consequent projecting lengths of strand are provided at permanent anchorages. The decision should be taken with reference to the number of tendons, the spans and the consequential costs.

3.14 Prestressing components

In simple terms, the essential elements here are:-

- a) the prestressing tendons
- b) the ducts, containing the tendons
- c) the anchorage system
- d) the protective system overall

It is possible to consider (a) and (b) in a general way. Ducts will be different, depending on whether internal or external prestress is used. The primary protective system for bonded internal prestress is the cement-based grout. For external unbonded construction, a wider range of protective systems is available; the anchorage methods may also be different, and there is the further need to consider special features such as deviators and couplers.

3.14.1 Prestressing Tendons

A number of steel strand types are available which normally consist of seven wires. The diameter of the strand, its compactness, strength and metallurgical properties may all vary. Of particular concern in the past has been the susceptibility of some types of strand to hydrogen embrittlement. It is understood that strand manufactured to BS5896 [BSI(1980)] prEN 10138 or ASTM A416 [ASTM (1999)] is not likely to be subject to hydrogen embrittlement, and any of these strands may be used. There have also been concerns about the effectiveness of epoxy coating particularly for reinforcement. Any small defect in the coating can lead to an increased likelihood of local corrosion and the use of epoxy-coated reinforcement in the USA is being reviewed. It is not clear whether the problems associated with epoxy-coated bars also apply to epoxy-coated strand.

Strand can also be made available with an already built-in layer of protection. This may be a physical layer such as galvanising or epoxy coating or an additional means of inspection such as 'intelligent strand' in which a fibre-optic sensor is passed down through the centre wire of the strand and can be used to monitor strains and breakages in the strand. 'Intelligent strand' is currently being tried on a bridge in the UK. The effectiveness of these special inspection and monitoring facilities needs to be carefully considered and cannot be recommended yet for use on all structures pending feedback.

3.14.2 Ducts

During the development of post-tensioning systems, a number of different methods have been used to form ducts. For straight tendons this has sometimes meant using formers which are subsequently removed leaving unlined ducts. Lined ducts have normally been formed

using either welded or spirally wound steel tube. Unlined ducts and biodegradable ducts such as cardboard tubes should not be used. The advantages of spirally wound tubes are that it is flexible and can be bent on site to match the required profile. More recently, non-metallic ducts have been used. These have been made out of High Density Polyethylene (HDPE), now known as PE80, or Polypropylene, and have a number of advantages:

- Corrosion-resistant.
- Better sealing against ingress of contaminants.
- Can be pressure-tested during construction to demonstrate integrity.
- More potential to be 'seen' through by some testing techniques.

If the duct is to be used as one of the protective layers making up the system, then it should itself not be subject to corrosion which would make the protection ineffective. The main advantage of non-metallic ducts is that they can form a sealed system around the tendon and minimise the risks of contaminants reaching the tendon. The advantages of being able to penetrate non-metallic ducts with testing techniques remains to be proved. The test methods currently available all have limitations and only provide partial information about conditions within the duct. However, with development of the techniques, more detailed detection of voids and corrosion may be possible.

The TR47 specification requires the use of corrosion-resistant ducts which for internal tendons are bonded to the surrounding concrete. The ducts should be pressure-tested prior to concreting to verify the assembly. Minimum wall thicknesses are specified which have to allow for the effect of the stressed tendon 'biting' into the duct wall.

The use of non-metallic ducts requires a re-assessment of some of the properties of ducts and their effects on the whole protection system. The main differences for the materials currently used for non-metallic ducts compared with steel are as follows:

- The duct material does not corrode.
- The duct material is effectively resistant to passage of chloride ions.
- The duct material does not conduct electricity.
- The duct material has a high coefficient of thermal expansion (which is typically $140 \times 10^{-6}/\text{deg.C}$).
- The duct material has a low Young's modulus (typically 800 N/mm^2).

Although there are few examples of ducts corroding from the outside the fact that a non-metallic duct material cannot corrode is obviously an advantage. Ducts which corrode do not provide a physical barrier between any contaminants and the tendon. Ingress of chloride ions through the duct material is a theoretical possibility, however, in practice it is not likely to occur given the thickness of the material.

Another issue to be considered is the risk of stray current corrosion. Overhead electric power systems (AC) can induce voltages and current into adjacent metal objects, which, if not controlled, can generate harmful potentials on exposed conductors. In addition, third rail power systems (DC) can leak currents into the ground, which attempt to return to the power supply via the lowest impedance route; if not controlled, the resulting stray currents can cause accelerated corrosion of prestressing tendons at the point where the current exits the tendons. Adjacent to AC lines, conductive paths should not exceed 500m. This is longer than normal post-tensioning tendons, but it would be possible to envisage straight external tendons of this length. Generally the principal control measure is to ensure that the tendons and anchorages are electrically isolated. This needs particular care and is probably only totally reliable with a plastic encapsulation of anchorages. The use of plastic ducts, together with a nominal concrete cover of 45mm around all metal parts of the anchorages may be sufficient to achieve this. If it is necessary to expose metal parts at either or both ends of the tendons, then the tendon system should be earthed at one end only. This is common practice for railway viaducts employing DC power supply.

The coefficient of thermal expansion of non-metallic ducts is greater than that of concrete and there has been some concern that the increase in temperature during hydration of the concrete, and subsequent cooling, would leave a gap around the duct. This has not been found to be the case in the grouting trials undertaken to date.

One claimed advantage for non-metallic ducts is the ability to form a sealed system to the tendon which will exclude contaminants from the duct. Whilst it would be possible to design such a duct system, the currently available ducts cannot be assumed to be guaranteed sealed and fully watertight. The pressure-test acceptance criteria given in the TR47 specification are related to what is possible using the currently available non-metallic ducts and demonstrate that the duct has been properly assembled. Tests undertaken by the TR47 Working Party show that the acceptance criteria specified will not guarantee a full barrier to the ingress of contaminants at the joints in the duct. Difficulties were experienced with joints between lengths of ducts, and with seals between the ducts and anchorages, and also with venting tubes. It is for this reason that multi-layer protection is required. It is hoped that fully sealed ducting systems will be developed in due course in which case the acceptance criteria would be very much tighter than currently specified; on the other hand, less dependence on multi-layer protection would then be required.

For external prestress the locations where the ducts connect to the anchorages or pass over deviators need careful consideration; detailing in these areas is especially important. Where it is necessary to join lengths of duct, electrofusion or butt fusion jointing can normally be used for PE ducts; again detailing is important and such joints should preferably be kept clear of construction joints, (up to 1 metre is suggested), to reduce the risk of penetration. For longer tendons, consideration needs to be given to duct movements during stressing and the provision of a joint with a sleeve coupler which allows the duct to shorten.

The development of prestressing systems based on non-metallic ducts is still at a comparatively early stage, especially for bonded construction. Much work has been done on the properties of the basic materials, but the design and manufacture of plastic ducts can vary between suppliers. *fib* has published a Technical Report [FIB (2000)] on duct properties which is sufficiently broad to distinguish between ducts used for external unbonded construction and for bonded construction, but the emphasis is on defining performance requirements for bonded tendons and on suggested test methods to ensure that these are met. This is the basis of the proposed revisions to TR47 and for a systems approval approach, which should become the norm in the future.

The duct should be large enough both to allow threading of the prestress tendon and to facilitate grouting. The diameter depends on the size of the prestress tendon, the overall length and the curvature of the duct. Normally a maximum tendon to duct area ratio of 0.40 to 0.45 should be used, increasing with tendon size. The tendon to duct area ratio is defined as the area of the strand based on its actual steel area divided by the internal cross-sectional area of the duct. For short tendons with little change in direction, the tendon to duct area ratio can be increased provided that grouting trials show that the duct can be satisfactorily grouted.

3.14.3 Grouts and Grouting

One of the main advantages of a properly grouted duct is the alkaline environment created within the duct. Major concerns with existing bridges have been the quality of the grouting and the protection provided to the tendons. The grouting operation has often been undertaken by inadequately trained personnel who have not understood the importance of the grouting. There are even instances of ducts being left completely ungrouted. The TR47 specification gives details of improved grouting methods, layout of vents, and the materials to be used. Site operations should be carried out by companies and operatives satisfying the Certification Scheme. TR47 also recommends full scale grouting trials to "prove" the systems, equipment and operatives being used. This has been useful in concentrating the site on the need for good quality control. However, it is probably not necessary to have full scale trials on those sites where the systems, equipment and operatives have been used in a similar way on a recent previous site.

The measures proposed have provided a substantial improvement on previous grouting operations and give assurance that the ducts are adequately filled with grout. They cannot, however, guarantee complete filling of all ducts and some small voids may still occur. It must however be remembered that voids themselves do not cause corrosion of the tendons.

3.14.4 Vents and Grout Injection

Trials have shown that an additional grout vent just beyond the crest in the tendon profile, in the direction of grouting, can be beneficial as that area is particularly vulnerable to trapping of air. Any air or bleed water carried past the vent at the crest during grouting should migrate back up the duct and be removed at this additional vent. The additional vent should be closed before the crest vent. The trials also showed that larger vents are desirable. The intention of having vents, where possible, raised at least 500 mm above the duct profile, is to provide an effective head of grout. The TR47 Specification gives guidance on spacing, location and size of duct vents based on these trials. If the direction of grouting is not known at the time of casting, additional vents will be required on both sides of the crest vent.

All anchorages are supplied with a tapped hole for use either as the grout injection location or as a vent. The end cap to the anchorage is also supplied with an air vent and the cap is to be filled with grout during grouting of the cable. For grout injection, the anchorage hole must be positioned so that grout is injected from the bottom. At the far end of the cable the anchorage hole could be either on the low side or the high side. It acts as a vent during grouting but the final vent for the anchorage and the end cap is the tubed vent on the end cap which must always be placed at the highest point. Anchorages with two holes would avoid the possibility of the grout hole being placed incorrectly.

In the design of the vents it must be remembered that the vent is part of the protective system and needs to be sealed to the same level of air-tightness as the duct. This requires an engineered connection between the vent and the duct and a sealable stopper where the vent exits the concrete (see TR47). If this is the top surface of the deck and the vent is extended above deck level for grouting purposes, the extension will have to be removed before stopping up the duct. Vents should be sealed on completion of grouting and also on removal of any extension tubes with a properly engineered detail.

3.14.5 Other Protection Systems for External Prestress

There are three typical systems for protection of external tendons:

(1) Unbonded tendons protected by a cement grout:

In this case, the duct is a continuous tube e.g. of PE, and the grouting principles are similar to those given above. At anchorages the duct passes through the concrete, usually, but not necessarily, via an outer pipe and trumpet to the anchorage – the so-called double envelope system. This is illustrated in Figure 8 & 9 below. At deviators, it again passes through an outer tube, often with a bell mouth to prevent damage to the plastic. At high points it is not feasible to extend vents above the duct, as this would require holes through the slab and connection of vent tubes to the duct within the deviator. However the ability to impact echo the duct to check for voids after grouting gives added confidence which heavily outweighs this disadvantage. This situation is illustrated in Figure 11. The location of such vents (for either grouts or waxes) should be considered at the design stage, and will depend, in part, on the prestressing and protection systems to be used. Such vents by necessity have to be smaller diameter than the specification minimum of 15mm. It may also be necessary to locate drains at low-point deviators, to remove water which may have accumulated in the duct. For easy removal, it is important that the tendon can move freely in these zones; in this regard, it is important to have a good seal between the anchor head and the duct. Protection of the anchorage itself is also important, and may be done using a long cap to accept the extra tendon length, if required, for re-stressing. These tendons are not normally suited to restressing unless they are straight and a shimmed or screwed anchor head is used.

(2) Unbonded tendons protected by a flexible product (e.g. wax):

The most common filler material is petroleum wax, which is heated to about 90°C and poured while still liquid. In general, only a single duct is used (no double envelop), and care is necessary to prevent the wax running out of the duct. There has been limited feedback on

some waxes being brittle. There have been a number of reported instances of wax products becoming unstable and leaking out, especially at high temperatures and care should be used when considering these products, particularly as they are flammable.

(3) Tendons made with sheathed and greased monostrands, within a grouted system:

Greased strands are enclosed in a PE sheath, and pass through pipe assemblies both at the anchorages and the deviators. One advantage is that the strands can be stressed (and destressed) using a monostrand jack, requiring less working space behind the anchorages. The strands can also be extracted individually and replaced. A system is also available of sheathed and greased strands within a PE pipe, which is then grouted prior to stressing. The grout is used to fix the strands in position, thus reducing the risk of displacement and of tears in the sheaths. In this case the duct must be supported to avoid displacement by the weight of grout. The grouting details are similar to those given above.

3.14.6 Detensioning and replacement of external tendons

Detensioning and replacement of external tendons requires special consideration with particular emphasis on safety. The operation should be carried out only by trained and experienced personnel directed by a competent supervisor. Exclusion of personnel from the area immediately adjacent to the detensioning operation as well as the entire tendon length is of paramount importance. Investigations prior to detensioning should include the following:

- Removal of anchorage caps to inspect condition/length of strand protruding.
- Tapping of the duct to check the adequacy of grouting.

The details of the de-tensioning procedure will depend on the type of prestress layout, and whether or not an extra strand length has been left at the anchorage to facilitate re-stressing and de-tensioning. Further advice will be given in the revision to TR47.

3.14.7 Anchorage Locations

The method and form of construction should be considered at the preliminary design stage. They will often significantly affect the layout of prestress tendons and the location of anchorages. For example, the layout of tendons for span-by-span construction will be different to that for structures cast in one pour. The significance for durability of tendon profiles and anchor locations, should also be considered at an early stage. The tendon profile and duct size affects the ease of grouting. The location of anchorages affects the ease of stressing and subsequent inspection, as well as their susceptibility to water ingress. For example, anchorages in top pockets in the deck have often been used in span-by-span construction. They are easy to construct, stress and subsequently fill, but there is a concern that by their shape and location, they provide a path for contaminants to the prestress cables. Anchorage layouts are especially important for external systems, as is the detailing of ducts where they pass through deflectors and diaphragms.

Feedback from inspections clearly demonstrates that the ingress of contaminants is most common at anchorages, with corrosion being initiated in the tendon immediately behind the anchorage, often in the presence of imperfect grouting. The anchorages are particularly vulnerable as there is a tendency for bleed water and air in the duct to migrate to the high points such as anchorages. It is of major importance, therefore, that attention is given to the location of anchorages, and to detailing to prevent the access of water to the ducts. Anchorage systems depend both on tendon type and on supplier. The guidance given in this Section relates to anchorages used for internal grouted tendons and external unbonded systems. In both cases, the TR47 Working Party felt that the subject was so important that illustrations are given in the Report of preferred solutions. Some of these illustrations are repeated here, but for full information refer to TR47.

The layout of prestress and location of anchorages is dependent on method of construction. In the case of a simply supported beam cast in situ the anchorages would

normally be at the ends of the beam (Figs 1-2), but there are many forms of construction which require anchorages at various different locations. These may be broadly sub-divided as follows:

- Dead-end anchorages within the body of the concrete.
- Anchorages in blisters within the span, either inside a box girder or below the slab in a beam and slab deck (see Figures 3 and 4).
- Face anchorages on the joint between segments in either span-by-span or segmental construction. (see Figure 4)
- Anchorages in pockets in the top surface of the deck (see Figure 5).

Wherever anchorages are located the designer should ensure that an adequate multi-layer protection system is provided.

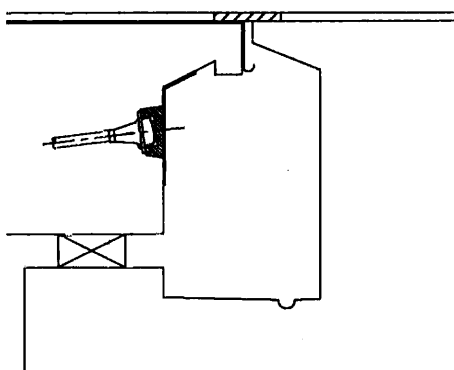


Fig.1 Buried anchorage for internal prestress at end of deck with abutment gallery

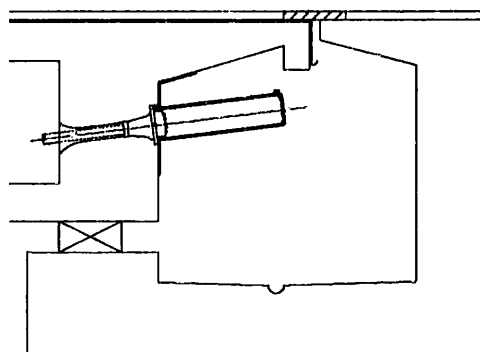


Fig. 2 Restressable anchorage for external prestress at end of deck with abutment gallery

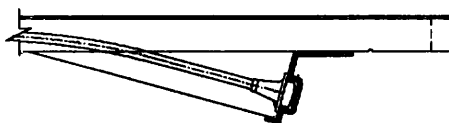


Fig. 3 Exposed anchorage for internal prestress in blister

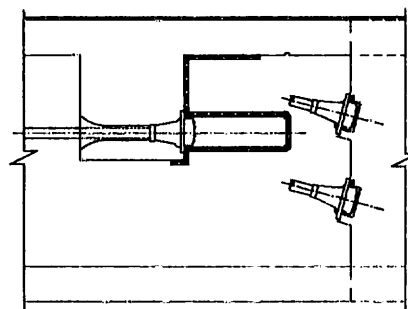


Fig. 4 Restressable anchor for external prestress with internal face anchorages for staged construction

There has been much discussion as to the merits of anchorages in pockets in the top surface of the deck. There have been examples of severe corrosion of tendons with top pocket anchorages but none have occurred, to the knowledge of the author, where the tendon has been properly grouted. It is not clear whether or not top pockets constitute an additional level of risk. However, there is a strong feeling amongst many engineers that the top pocket

provides an unnecessarily convenient route for contaminants into the anchorage and tendon. There are few situations in which an alternative arrangement to the use of top pockets cannot be found, and it is recommended that they should not normally be used. If there is no alternative (for example in some bridge refurbishment schemes) then the design and construction should ensure that contaminants are excluded both during construction and in service by taking additional protective measures (see Figure 5).

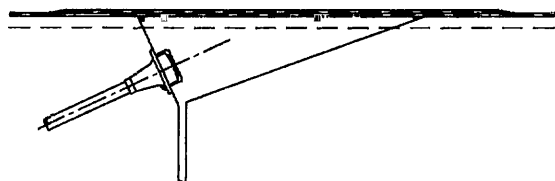


Fig. 5 Top pocket anchorage. This detail is **not** recommended unless additional protective measures are used

The location and detailing of anchorages are dependent on the choice of whether or not to leave the anchor head exposed. Exposed anchorages go some way to allaying fears of lack of inspectability of prestressing systems, but they do increase the risk of corrosion by reducing the number of layers of protection. The end cap of the anchorage has to be removable to inspect the ends of the strand and once removed it may be difficult to replace and ensure an airtight seal. The advantages of burying the anchorage in a concreted pocket are that the end cap can be filled with grout rather than grease and the end cap is itself surrounded in concrete. Inspectable or buried anchorages are equally acceptable and the designer should choose whichever type is most appropriate to the location under consideration. Dead end anchorages deep within the body of the concrete are also acceptable but have the practical disadvantage of requiring the tendon to be in place prior to concreting.

At the location of expansion joints an 'abutment gallery' should be provided for inspections of the joint and the concrete below the joint. This gallery can also be used for inspection of exposed anchorages and for inspection of the concrete and waterproofing covering buried anchorages (see Figures 1 and 2). For external prestress, the gallery dimensions should be chosen to allow replacement of the tendon.

The TR47 Working Party has not yet concluded which are the most appropriate measures to protect anchorages at the ends of an integral bridge. One possibility is that an abutment gallery is provided, similar to that required at expansion joints, so that the end of the deck can be inspected. It may also be possible to have a smaller gallery for inspection using a TV camera although such a gallery would not facilitate maintenance should this be required. Alternatively it may be possible to dispense with visual inspection by placing a corrosion probe adjacent to the anchorage and using either a buried anchor detail, or encasing the anchor in a sufficiently well detailed concrete cover. It is anticipated that with the greater use of integral bridges, new details will be forthcoming.

3.14.8 Anchorage Details

Figures 6 to 10 give typical anchorage details for a number of situations. In developing the details the following points have been taken into account:

- a) The anchorage should have multi-layer protection. No individual layer can be assumed to be effective for the life of the structure.
- b) The detailing of expansion joints should be such that when the joint leaks, the water is directed away from the anchorages and into a properly detailed drainage system.
- c) The anchorage end cap should be filled with grout during grouting of the tendon. The end cap is shown bolted to the anchor plate in the figures, but in some systems the cap is bolted to the anchor head. Provided that the anchor plate and head are machined for a close fit, this is an acceptable detail. The end cap vent should be at the top.

- d) Any concrete used to infill pockets should have flowable non-shrink characteristics (see Item 4 of Department of Transport (1986)) and should be held in by reinforcement. The edges of the recess should be protected with a sealant. The minimum cover to the end cap should be 50mm.
- e) The anchorage or capping concrete should be covered with the bridge waterproofing membrane.
- f) Exposed anchorages should be protected from seepage through construction joints in thin slabs by keeping the joints at least 500mm from the anchorage, and by means of drips.

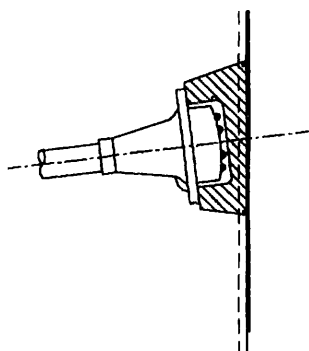


Fig. 6 Buried anchorage for internal prestress dead end or stressing end

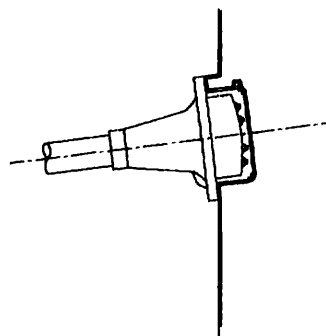


Fig. 7 Exposed anchorage for internal prestress dead end or stressing end

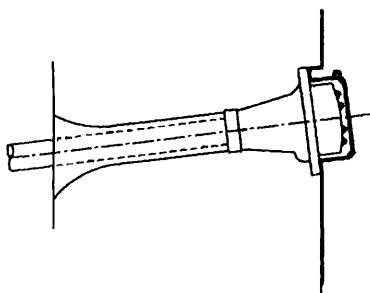


Fig. 8 Exposed anchorage for external prestress dead end or stressing end (non-restressable)

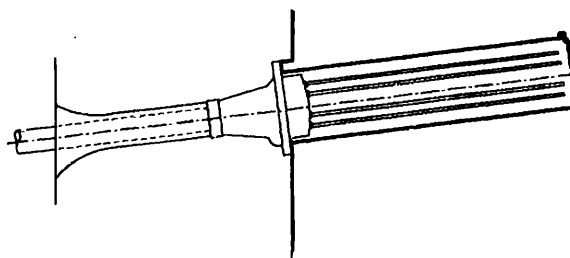


Fig. 9 Exposed anchorage for external stressing end (restressable)

It should also be noted that overhangs at the ends of the deck without construction joints (in order to comply with item f) above), necessitate special devices to enable stressing jacks to access the anchorages. Normally a crane with special lifting device or a jacking trolley would be required. In cases of very large tendons this can be prohibitive and consideration could be given to use of a construction joint with a waterbar and sealant in a surface groove.

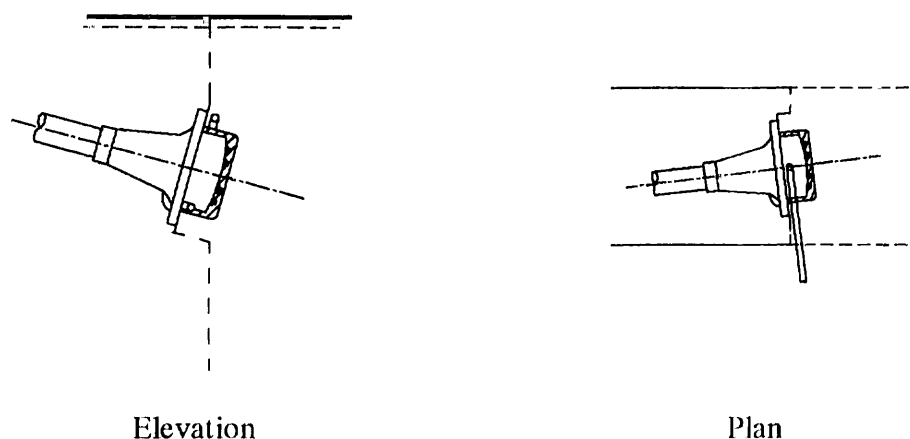


Fig. 10 Face anchor details for insitu stage by stage or segmental construction



Fig 11 Vent for external prestress at crest in deviator

4 PRECAST SEGMENTAL CONSTRUCTION AND JOINT TYPE

Precast segmental construction is considered separately, since particular care is needed. Only general advice is considered here. The only two bridge collapses in the UK due to tendon corrosion have been in structures constructed using precast segments. Both had thin mortar joints and incompletely grouted ducts. When considering the risks associated with segmental construction, it is important to understand the significance of the different types of joint. Joint types in previous use may be sub-divided as follows:

- Thin mortar joints
- Wider, in situ concrete joints
- Match-cast joints – epoxy or dry

Sufficiently wide in situ concrete joints, and match-cast joints properly sealed with epoxy resin, can be satisfactory in durability terms. The main durability problems have been with thin mortar joints. Difficulties in forming these have led to a joint material with high permeability - and they should not be used. Special consideration has also to be given to the continuity of the ducts across the joints. The TR47 Working Party believes that duct continuity across the joints is vital when grouted tendons are used unless some other protective systems are proven. Ongoing research may help to form definitive advice on this in the future [University of Texas (1999)].

Modern precast segmental methods generally involve match-cast segments with thin epoxy joints, and wider insitu concrete stitches. In some countries, dry joints have been used, but these are not recommended where de-icing salts may be used.

One of the principles of the multi-layer protection strategy is that a continuous sealed duct is a key layer, in preventing contaminants reaching the prestressing steel. At the time when the first edition of TR47 was published, the TR47 Working Party was unaware of any detail which could guarantee duct continuity – directly or indirectly. At that time (1996) the recommendation was that only external unbonded prestressing should be used for precast segmental construction.

Since 1996 the TR47 Working Party has sought reliable solutions to this problem. There would appear to be three possibilities: -

- (1) The development of proprietary splicing sleeves for the duct at the joints. It is believed that one such solution has been developed, but full details have not yet been launched onto the market. Any such solution would be proven via a Technical Approvals system, which would require both the development of a suitable test and acceptance criteria. Pending further experience with this system, it may be prudent to assume that some percentage of the internal tendons is lost (say 5-10%).
- (2) Further research on the use of epoxy resins in thin match-cast joints and corrosion protection, both in the laboratory and in the field [University of Texas (1999)]. Precast segmental bridges have been in service now, in the UK and especially in the USA, for up to 30 years.
- (3) The development of a design-based solution, based on the multi-layer protection strategy, possibly involving a combination of external unbonded and internal bonded tendons (enough, say, to carry all dead loads), possibly with the introduction of additional protective layers, to give an overall reliability comparable to that for having continuous ducts as part of the system. Current practice with this approach is to have a minimum of 75% of the tendons as external unbonded.

What is being described here is a developing situation. However, the TR47 Working Party feels that internal prestressing should be reconsidered for precast segmental construction.

There are some special considerations for the location and detailing for anchorages in precast segmental construction, and these mainly relate to face anchorages at joints. A typical layout is given in Figure 12. The particular concern is to protect the anchorage, including the cap, from any possible water leakage in the joints.

Special care needs to be paid to the following details:-

- Sealing of the anchorage recess or box-out, particularly if the distance from the edge of the recess to the edge of the segment is small (this concrete is easily damaged). Some engineers have recommended the use of an additional sealing strip around the anchorage box-out. However, there is concern that introducing a different material into the joint will interfere with the proper compression on the liquid epoxy resin during making of the joint, and therefore interfere with the bond between the epoxy resin and the segments on either side. The author recommends that the epoxy resin should be used as the seal around the box-out.
- Location of inlet/outlet/vent pipes. In addition to the injection hole/vent in the anchorage and the end cap vent, it is necessary to allow for an inlet and vent for grouting the box-out. Hence there are four pipes to be detailed in such a way as to prevent these pipes becoming damaged during the erection process. There are a number of ways in which this can be achieved, one such is shown in Figure 12.

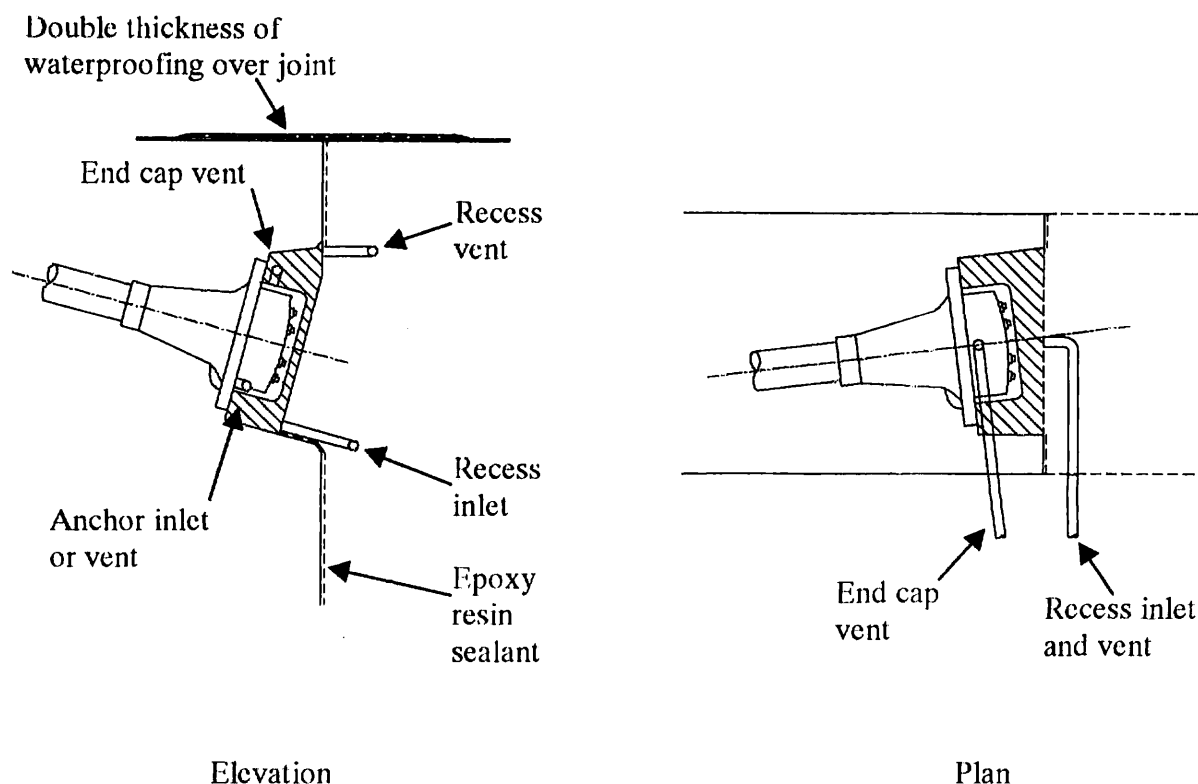


Fig. 12 Face anchor details for precast segmental construction

5 RECOMMENDED MULTI-LAYER PROTECTION SYSTEMS

5.1 General

In proposing the multi-layer protection concept, an outline has been given in general terms of all the factors which could come into the design equation, in consciously designing for durability. A number of different layers have been considered, not all of which are practical in all situations. The following layers are recommended as the minimum requirements for a typical road bridge subject to de-icing salts. These protective measures must be read within the philosophy of this report with regard to overall design principles. The requirements for rail bridges or road bridges in countries where de-icing salts are not used may be less onerous.

5.2 Prestressing System

- Ducts and vents should be in accordance with TR47, corrosion-resistant and pressure-testable. The ducts and vents should be pressure-tested.
- Full-scale representative grouting trials should be used to prove the grouting method, materials and personnel, where there is no previous history.
- Method statements should be prepared in advance for all prestressing operations and should be approved by an appropriately experienced Chartered Engineer.
- All operations associated with the installation, stressing and grouting of tendons should be undertaken under an approved Quality Assurance scheme.
- Anchorage and vent locations and detailing should follow the logic outlined in the Figures included in TR47, some of which are re-produced in this paper.

5.3 The Deck and its Elements

- a) Concrete strength and cover requirements should be in accordance with current international standards.
- b) Specification should be in accordance with current international standards as amended by the TR47 Specification.
- c) A sprayed deck waterproofing system should be used, and double thicknesses specified at critical locations as specified in TR47. It should be checked for integrity using appropriate non-destructive test equipment, including pin-hole detection equipment for liquid-applied waterproofing systems. Where a double thickness is used, the first layer should be proved before the second layer is applied. In addition, the membrane should be used to protect any anchorages left exposed in abutment galleries, inside box girders and on bridge deck soffits.
- d) The expansion joints and drainage system should be detailed to ensure that in the event of equipment failure or leakage, water cannot find access to the prestressing system.
- e) Strand should be in accordance with BS5896 [BSI (1980)] or similar.

5.4 Possible Additional Measures for Exceptional Structures

For exceptional structures in unusually aggressive environments (for example, bridges over the open sea), additional protective measures could be considered taken from the list below. Each structure will have to be considered on its own merits.

- Increased cover.
- Reduced concrete permeability.
- Perfectly sealed ducts.
- Corrosion-monitoring devices.
- Special strands.
- Concrete coatings.

6 ACKNOWLEDGEMENTS

This paper is based on the work of the Concrete Society TR47 Working Party, and its various sub-groups. The efforts of the members of the Working Party are gratefully acknowledged, particularly Prof. George Somerville who took over as convenor from the author in 1995 and Gordon Clark who took over as convenor in 1999.

7 REFERENCES

A more detailed list of related reference documents may be found in TR47 [Concrete Society (1996)].

ASTM, 1999. ASTM A 416/A416M-94, re-issued 1999. Specification for steel strand, uncoated seven-wire for prestressed concrete. American Society for Testing and Materials.

BSI, 1980. BS5896: Specification for high tensile steel wire and strand for the prestressing of concrete. British Standards Institution. See also prEN 10138. Prestressing steels. Part 3. Strand. (Final version not yet published.)

BSI, 1990. BS5400: Part 4: 1990. Steel, concrete and composite bridges. Code of practice for design of concrete bridges. British Standards Institution.

CONCRETE SOCIETY, 1992. Non-structural cracks in concrete. Technical Report 22, third edition. The Concrete Society, Slough, UK

CONCRETE SOCIETY, 1995. The relevance of cracking in concrete to corrosion of reinforcement. Technical Report 44. The Concrete Society, Slough, UK.

CONCRETE SOCIETY, 1996. Durable Post-Tensioned Concrete Bridges, Technical Report TR47. The Concrete Society, Slough, UK.

CONCRETE SOCIETY, 1997. Guide to surface treatments for protection and enhancement of concrete, Technical Report TR50,. The Concrete Society, Slough, UK.

DEPARTMENT OF TRANSPORT, 1986. Materials for the repair of concrete highway structures, Departmental Standard BD 27/86. DoT, London.

DEPARTMENT OF TRANSPORT, 1987. Early thermal cracking of concrete, Departmental Standard BD 28/87 and Advice Note BA 24/87. DoT, London.

DEPARTMENT OF TRANSPORT, 1994a. Expansion joints for use in highway bridge decks, Departmental Standard BD 33/94. Highways Agency, London.

DEPARTMENT OF TRANSPORT, 1994b. The design of concrete highway bridges and structures with external and unbonded prestressing, Departmental Standard BD 58/94 and Advice Note BA 58/94, Highways Agency, London.

DEPARTMENT OF TRANSPORT, 1995. Design for durability, Departmental Standard BD57/95 and Advice Note BA 57/95. Highways Agency, London.

DEPARTMENT OF TRANSPORT, 1992. Manual of contract documents for highway works, Volume 1: Specification for highway works. DoT, London. Reprinted 1993 with amendments.

FIB, 2000. Corrugated Plastic Ducts for internal bonded post-tensioning. Bulletin No. 7, Technical Report. Fédération Internationale du Béton.

FIP, 1996. Materials and systems for External pre-stressing. State of the art report. Fédération Internationale de la Précontrainte.

UNIVERSITY OF TEXAS, 1999. Corrosion Protection for Bonded Internal Tendons in Precast Segmental Construction. J S West, R P Vignos, J E Breen and M E Kreger. Research Report 1405 -4. Center for Transportation Research, University of Texas at Austin.

Strategies for improvement - Approach in Europe - The UK strategy and *fib* developments

G. CLARK

Director, Gifford and Partners, UK
Convenor Concrete Society WP for TR47
Convenor fib Commission 9 TG 9.8 “Grouting”

1 Introduction and background

The 25 September 1992 was a milestone in the development and use of post-tensioned concrete construction in the UK, when the Department of Transport (now the Highways Agency) announced that it would not commission bridges with grouted duct post-tensioning pending a review of standards.⁽¹⁾ Many thought this was an over-reaction but it is interesting now, 9 years later, to see the worldwide interest in durability of post-tensioning tendons, what developments have taken place, and what strategies there are for improvement.

Most engineers actively involved in the post-tensioning industry will be aware of UK Concrete Society Technical Report TR47 “Durable Grouted Post-tensioned Concrete Bridges”⁽²⁾ which was published in 1996 following intensive industry collaboration through a Working Party, and led to this form of construction being allowed again, although a ban still remained on precast segmental construction with internal tendons.

In essence through introducing a strategy of multi-layer protection, improving the characteristics of the grout and the quality of the processes both in design and in construction, and requiring a third party certification scheme, confidence returned in this form of construction. An important aspect of this was the major surveys of the bridge stock, which is reported elsewhere, which showed that things were perhaps not quite as bad as had been feared.⁽³⁾ Whilst there had been a small number of bridge collapses, the studies had established which were the most vulnerable types of bridges and inspections had focused on these initially.

Since 1996 there has been considerable further work and the UK Concrete Society will be publishing a second edition of TR47 in the near future, with updated and extended content, which will broadly accord with new *fib* Guidelines in course of preparation. Elsewhere in the world there is extensive work in this field to improve the quality of post-tensioned construction, especially the grouting.

2 UK 1996 Requirements

The strategy adopted in 1996 included a range of improvements to past practice, covering design detailing, construction workmanship and materials properties. It introduced the use of continuous corrosion resistant ducting, specifications for the grout and procedures to ensure filling of ducts and removal of air. At the same time a certification scheme was devised by UK CARES to ensure that only properly qualified and trained personnel are entrusted with the work.⁽⁴⁾

A requirement was introduced for full scale trials on every project, which were to be grouted using the proposed materials, equipment, personnel and procedures for the project. After the grout had set, sections of the trials were cut open to physically prove that the ducts had been filled. There was some resistance to this measure, as it is quite expensive and time-consuming, but it was considered essential to restore confidence and has been successful in doing so.

2.1 Ducts and Design details

The need for the duct systems to be sealed and made of polyethylene or polypropylene was based on the multi-layer protection approach, whereby no single item is relied upon solely to protect the prestressing tendons against corrosion. At the time the plastic duct systems were in their infancy and initially some ducting was on the market which did not fully meet the required tests for air-tightness and wall thickness, but there are now a compliant selection available on the market from all the major suppliers of post-tensioning systems.

Anchorage location and detail had been inappropriate in the past, given the knowledge that expansion joints and waterproofing can leak, and suggestions were made for improvement, which is reported elsewhere.

2.2 Materials for Grouting

Grout had commonly been made with just water and ordinary bagged cement, with admixtures. The recognition of the problems with bleeding and stability of the grout resulting in imperfectly grouted ducts was acknowledged by introducing a special category of grout, which had a more demanding specification. Trials had been carried out under a LINK-funded project on how to achieve the ideal grout with no bleed, no shrinkage, long setting-time and repeatable quality.

Here there was a typical problem. We could not immediately introduce a single specification for grout for which there was no product available on the market. The specification therefore introduced two categories of grout, Common and Special. For special grout we defined the preferred acceptance criteria were defined and the intention was stated to require this at some time in the future. It is gratifying to see that now there are several products available which can satisfy these more demanding requirements.

2.3 Test methods

Test methods for shrinkage, bleeding, fluidity and sedimentation were included in the recommendations, mainly being tests that were already in common use and included in the EuroNorms, although the latter was a new test. This was to check the density of the hardened grout taken from the top and from the bottom of a sample and was an early effort at checking for stability of the grout.

2.4 Training and Certification of Procedures

The introduction of the certification scheme by UK CARES was supported by the Highways Agency and needed careful thought. Firms could not be eliminated from carrying out post-tensioning if they were already in the market but had no current projects for their procedures to be vetted in order to achieve approval. So for an interim period it was accepted to have a provisional qualification.

3 Research and Development

Since 1996, two main areas of development work have been focused on; improving the characteristics of the grout, and investigating better test methods. Much of this work has been on an international basis and the knowledge has been shared. A Britc-Euram funded project “QA of Grouting” was carried out with input from Denmark, Belgium and the UK, and the results presented in the final report to the European Commission in 2000.⁽⁵⁾

This project investigated new formulations for grout and new test methods and a Seminar was held in Cambridge in September 1999 at which the preliminary results were explained to an invited audience.⁽⁶⁾

3.1 Grout characteristics

Trials using transparent tubes had clearly shown the different behaviour of grouts in the real grouting process and the importance of the fluidity being not too liquid.

3.2 Test methods

Up until then, the conventional test for bleeding of grout had been a simple vertical cylinder about 200mm high, but it was observed that in transparent tubes with strands, considerably greater bleeding was occurring. The strands were acting as a wick to let the water rise up the slope of the duct and collect at the high points.

It was decided to include a strand in a vertical cylinder to simulate this and clearly greater bleed was induced by the “wick effect”. Grouts that had passed the old test failed this new test badly. In France an inclined tube test had been developed which simulates the same effect but on a larger scale.⁽⁷⁾

4 Forthcoming 2002 UK Recommendations

A Second Edition of the report TR47, completely revised and extended, is near to publication. This will include particular guidance for external tendons, new test methods and revised specifications, and will represent a further improvement in standards, based on the experience of the last 5 years, and the further research and development work that has been going on, not just in the UK.

It will also include a new section on grouting of voids in existing partially grouted ducts, as some of the procedures for this operation are different and require special attention.

4.1 Ducts and Design details

Where TR47 previously covered details only for internal tendons, it will be extended to include recommendations for different types of external tendons, including aspects such as methods of de-tensioning, anchorage arrangements and suitable smooth-walled ducts and vents.

4.2 Materials for Grouting

Experience has shown that the improved qualities of “designed” grout mixes, previously referred to in the specification as special grout, are of sufficient importance to warrant the adoption of a single standard in the new edition. Should use of common grout be preferred, (and it is cautioned against), the reader is referred to EN 445, 446 and 447,⁽⁸⁾ which although generally now agreed to be inadequate, will be updated in the future.

The standards for the material properties will be adopted such that “designed” or formulated grouts will be essential, either in pre-bagged form or as materials combined on site in a controlled procedure.

One important factor that has been recognised is the importance of the compatibility of the grout with the particular equipment to be used for the grouting, and the two must not be separated.

UK acceptance criteria will be revised as follows:

- Bleeding – any one test < 1% and the average of 4 consecutive tests < 0.3%
- Volume change – between 0% and +5%
- Fluidity <25s and >10s at duct outlet
- Sedimentation – variation <5%

4.3 Test methods

The new bleeding and stability tests will be included in the recommendations, as these are fundamental to the design of the grout. The vertical 1.5m bleed test is preferred for ease of site use for verification testing. The importance of compliance to a specific water/cement ratio has been removed because research has shown that any grout meeting the new tests will have a suitably low water/cement ratio.

4.4 Training and Certification of Procedures

Certification requirements remain largely unchanged although the guidelines will be re-written to the form of a guide as to what the Quality System should include to be in line with TR47. This makes it easier for CARES to alter the requirements without having to revise TR47 in the future.

5 Forthcoming *fib* Guidelines for Grouting

5.1 Background

Currently the FIP guide to good practice “Grouting of Tendons in Prestressed Concrete”,⁽⁹⁾ published in 1990 is the latest international guidance, and is therefore overdue for revision, as it pre-dates the development work of the last 9 years or so. An *fib* Task Group convened by the author is currently preparing new guidelines, which will be based on the latest research and experience internationally, and it is understood that once these are published, CEN will review EN445, 446 and 447.

Significant development work in Europe has assisted reaching a consensus view over the best way to get good grouting, by selection of materials, tests and procedures.

5.2 Ducts and Design details

For international use, it is important to recognise the different exposure and environmental conditions of the various parts of the world, in terms of degree of aggressiveness. It is not considered necessarily the case that plastic ducting is always required to provide an extra barrier, so the guidelines will include discussion of use of metal ducting, as well as plastic ducting complying with the new *fib* Technical Report Bulletin No7 “Corrugated Plastic Ducts for Internal Bonded Post-Tensioning”.⁽¹⁰⁾

5.3 Materials for Grouting

A single acceptance standard has been agreed upon as being preferred, and will be such that “designed” or formulated grouts, as is described elsewhere, will be necessary in order to meet the criteria. The most important criteria will be the requirement that bleeding must be less than 0.3%. Other criteria are broadly similar to those previously described for the UK and now adopted elsewhere.

5.4 Test methods

Test methods are known to vary internationally, and grout is no exception. Therefore the guidelines will describe only the new tests that have been found to be crucially important, especially the bleeding/stability test.

5.5 Training and Certification of Procedures

Worldwide practice varies here, although there is a common theme, that some form of training and certification is desirable. The guidelines are not prescriptive as to the form of the scheme but will discuss the importance of these aspects and describe in some detail the

typical procedures that are expected to be included in the process, as well as the type of records of the operations that should be kept. The *fib* guidelines will describe in some detail the stages of operation preferred during grouting.

6 Conclusions

There is a vast wealth of knowledge and experience within the international bridge community and more is being learnt every day. The UK experience sparked off a process which is gradually spreading through the rest of the world. The strategy for improvement developed in the UK in 1996 is fundamentally unchanged although some of the individual parameters have moved more towards common ground with developments in other countries.

One thing is certain, that the durability of the post-tensioning is improved and much better assured for the future.

References

1. DEPARTMENT OF TRANSPORT. Standards for post-tensioned prestressed bridges to be reviewed. Press Notice No. 260, 25 September 1992.
2. THE CONCRETE SOCIETY. Durable Post-tensioned concrete bridges. Technical Report 47 The Concrete Society, 1996.
3. HIGHWAYS AGENCY, TRL, SETRA AND LCPC. Post-Tensioned Concrete Bridges. Thomas Telford, London, 1999.
4. UK CERTIFICATION AUTHORITY FOR REINFORCING STEELS. Certification scheme for supply and installation of post-tensioning systems.
5. BRITE-EURAM Project BE95-1675, Contract BRPR-CT95-0099 QA of Grouting – Final Report 2000.
6. GIFFORD AND PARTNERS, QA of Grouting of post-tensioned Concrete Structures. Proceedings of Seminar, Cambridge, 23-24 September 1999.
7. Bulletin officiel Marchés publics de travaux. Modalités d'application du fascicule No. 65A CIRCULAIRE No. 99-54 du 20 août 1999 instituant un avis technique des coulis d'injection pour conduits de précontrainte, délivré par la commission interministérielle de précontraintes September 1999.
8. BRITISH STANDARDS INSTITUTION. Grouting for prestressing tendons - test methods (BS EN445:1996), Grouting for prestressing tendons - grouting procedures (BS EN 446:1996) and Grouting for prestressing tendons - specification for common grout (BS EN 447:1996).
9. FÉDÉRATION INTERNATIONALE DE LA PRÉCONTRAÎNTE. Grouting of tendons in prestressed concrete, FIP Guide to Good Practice, Thomas Telford, 1990, 16pp.
10. FÉDÉRATION INTERNATIONALE DU BETON. Corrugated Plastic Ducts for internal bonded post-tensioning. Bulletin No. 7, Technical Report, January 2000.

External prestressing of German bridges and its further development

J. EIBL

University of Karlsruhe, Germany

1. The situation at present — Introduction

Due to the necessary renewal of the German road system after World War II many bridges had been built between the fifties and the mid sixties. Already twenty years later between the seventies and eighties a significant number of damages had been discovered, mainly corrosion at the prestressing steel due to unsatisfactory grouting. Big discussions started. At that time a leading state official, Mr. W. Voss from the Federal State of Nordrhein-Westfalen, addressed the author and asked whether we could not look out for a new type of prestressing method. We were both convinced that the grouting process in countries with a wet and cold climate would always incorporate some risk of hollow spaces and remaining cavities within the sheathing tubes. At least one can state that the overcome method of prestressing is at least not very maintenance friendly. As an alternative we discussed external prestressing as done by the French and as applied by Prof. Dischinger already in 1934 at the Aue Bridge in Germany.

As a consequence then, since 1985 first in cooperation with the Swiss authorities, then with the mentioned Federal State, the German Railways and of course in agreement with the German Federal Government about 10 prototype bridges have been built by this method. The cables used were mainly developed by BBRV-Suspa, Vorspanntechnik and Dyckerhoff and Widmann together with the author's Institut für Massivbau at the University of Karlsruhe.

In 1998 the Federal Government having accompanied this first bridges with great interest, finally decided [2,3] that hollow box type bridges are to be built exclusively with external prestressing. This decision was mainly supported by the very thorough cost investigations done by GROßMANN [1] coming to the conclusion that the easier repair justifies a higher erection price for external prestressed bridges than for conventional ones. The difference was estimated to be more than 10%.

However due to opposing arguments mainly from the building industry it was then allowed to use conventional grouted cables still within the top and bottom plate. In such straight cables grouting was estimated to be much easier. No cables are allowed within the webs with regard to sagging of concrete under internal cables, as had been found in many cases. The main, until now used external cables types are shown in Fig. 1. The cable type shown in the midst of this Fig. is meanwhile also used, substituting the monostrand by pure steel strands in mortar with regard to economic reasons. Remembering that many of former mentioned damages resulted from unsatisfactory grouting, this type is not very much appreciated by the author.

With these guidelines meanwhile altogether about 40 bridges have been built with external prestressing as nearly all bridges with spans larger than 50m consist of hollow box type continuous girders. Only the German Railways go on building conventional prestressed bridges except the one conceptually designed by the author at the beginning of this first period of external prestressing.

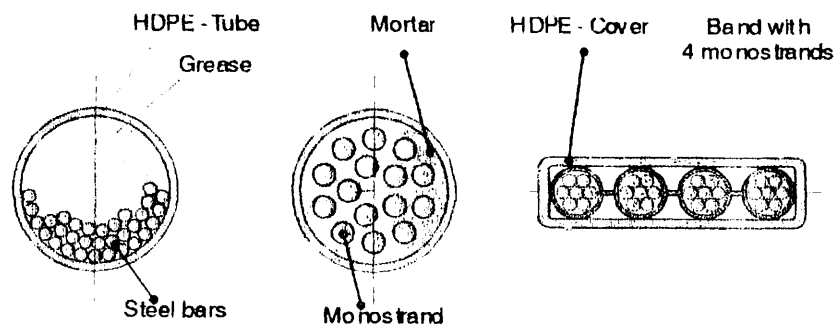


Fig. 1 : Used cables

2. Gathered experience

In the meantime there is no more any objection of the contractors against this type of construction. What had to be learned was the necessity to carry out the geometrical layout very thoroughly especially at the saddle points, as the governmental institutions, for good reasons, do not allow any kinking of the cables at the ends of these saddles. Meanwhile three degrees of additional tolerance to the theoretical cable inclination at both ends is demanded. As after some first difficulties all participants are now accustomed to these regulations no more principle discussions take place.

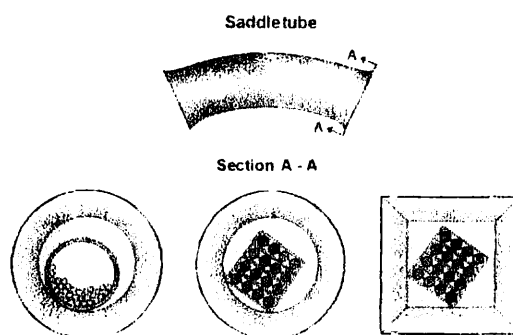


Fig. 2 : Monostrand cables and problems

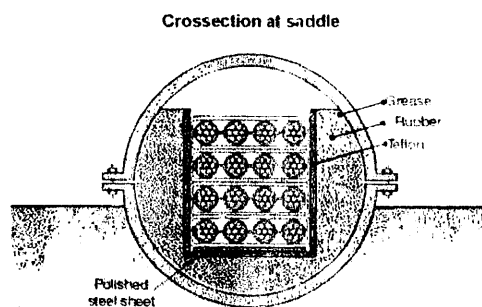


Fig. 3 : Proposal to avoid problems as shown in Fig. 2

Nevertheless the authors feels that one should strive for more failure tolerant saddle structures. Therefore research is going on at our institute for the most used 4 strand-band cables

(see Fig.1) financially supported by the government to avoid kinking and the coupled edge problem (Fig. 2) at penetration tubes. The latter rises due to unsatisfactory cable arrangements especially in case of a necessary double curvature in the vertical and horizontal plane. The idea is that the surrounding of multilayered bands by a rubber "torus", which may be distorted with little resistance, and adding an easily sliding layer of Teflon may help to make a less damage sensitive system.

One has also had to learn that the cable layout needs a very careful planning in case of bridges with big spans and a rather small available height. The bridge in Fig.4 and 5 designed by the authors consulting office was won together with German contractors in a financial competition against an already planned composite structure. The dimension of which we had to accepted. As the given span of this composite structure was 100 m at maximum with a total outer height at midspan of only 3.50 m the cable arrangement had to be done very thoroughly.

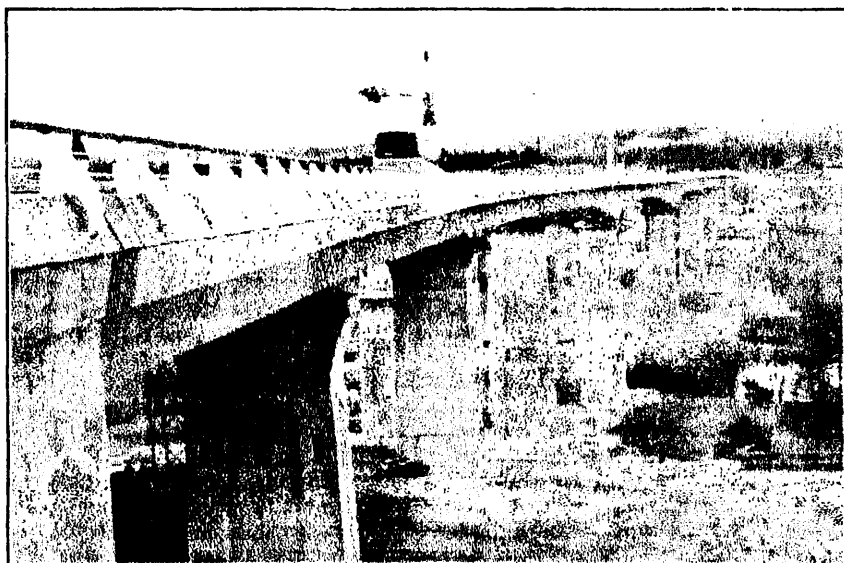


Fig. 4: View of Trockau-Bridge

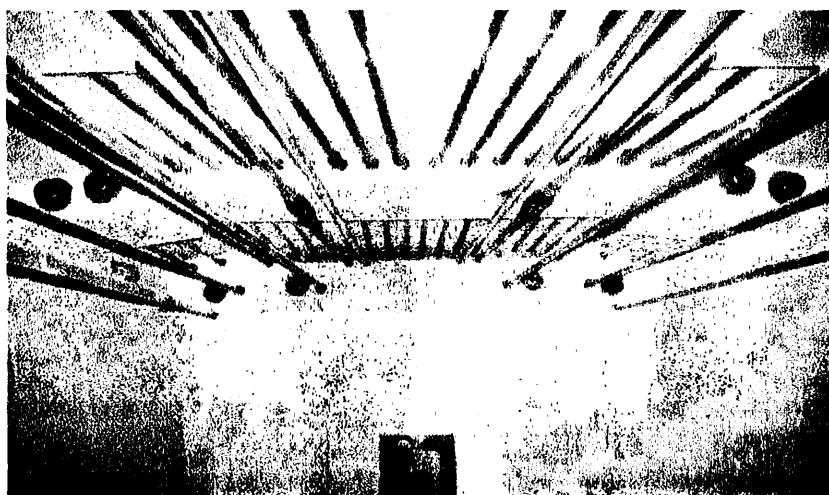


Fig. 5 : Interior view of cable-layout – Trockau-Bridge

It generally proved that anchoring blocks (Fig.6) cause a problem not yet completely solved. According to German regulations these blocks they have to be designed with a safety factor of 2.8, regarding steel failure at ULS. However the intention behind was more to limit crack-width along reentrant edges and corners than to avoid a steel failure. This regulation proved to consume a rather high amount of mild steel, causing high labour costs also due to the resulting binding work in highly reinforced areas which are penetrated by cables and demand thorough concreting.

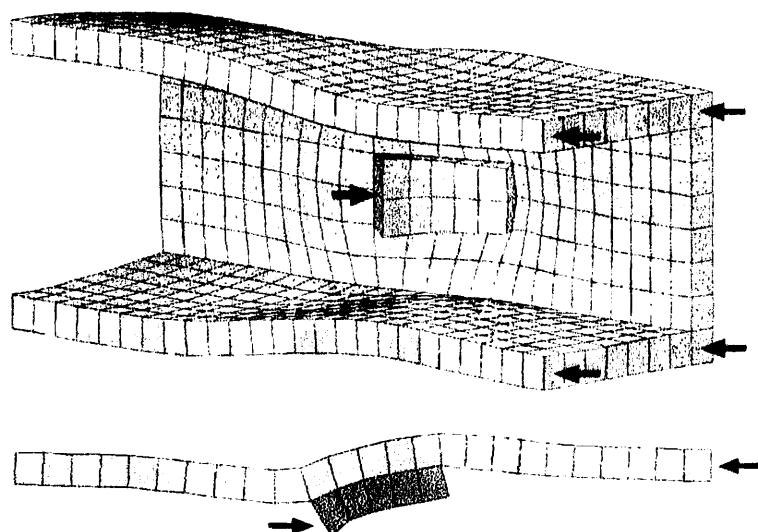


Fig. : 6 Anchoring blocks

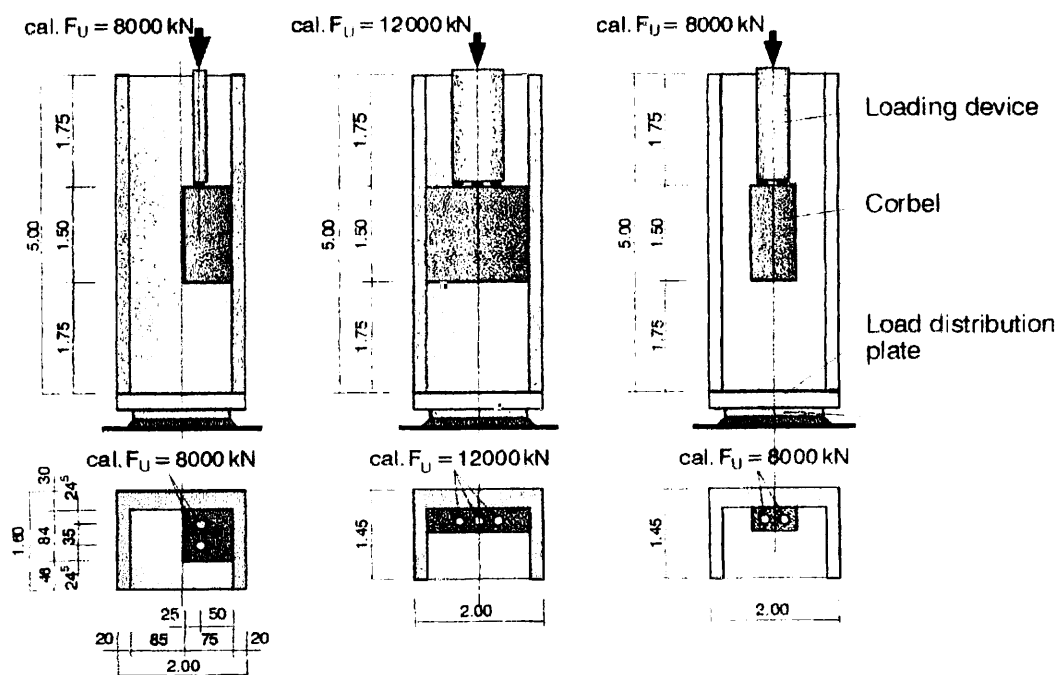


Fig. : 7 Experiments to study the crack behaviour in anchoring blocks

Necessary plate anchoring blocks below the upper and above the lower plates also reduce the internal lever arm considerable. The additional demand that the webs have to remain free of any cables, a condition which however has nothing to do with the type of prestressing, increases the space problem further.

Therefore the author has carried out experiments at his institute within a research program of the German Federal Government to cheque whether this high safety factor is really necessary (Fig.6 and 7). Fig 7 shows the big specimens $1.45 \times 2 \times 5,00 \text{ m} \times 2,00 \text{ m}$ implying a maximum load capacity of 18 MN which to the author opinion were necessary as otherwise no realistic crack behaviour could not be expected. The resulting scale to reality was about 1:3. A typical crack pattern is shown in Fig.8 for the right specimen in Fig.7. The reinforcement was chosen according to a reasonable assumed strut and tie models. The tests showed that a "safety factor" of about 1.9 is appropriate, reducing the mentioned drawbacks.

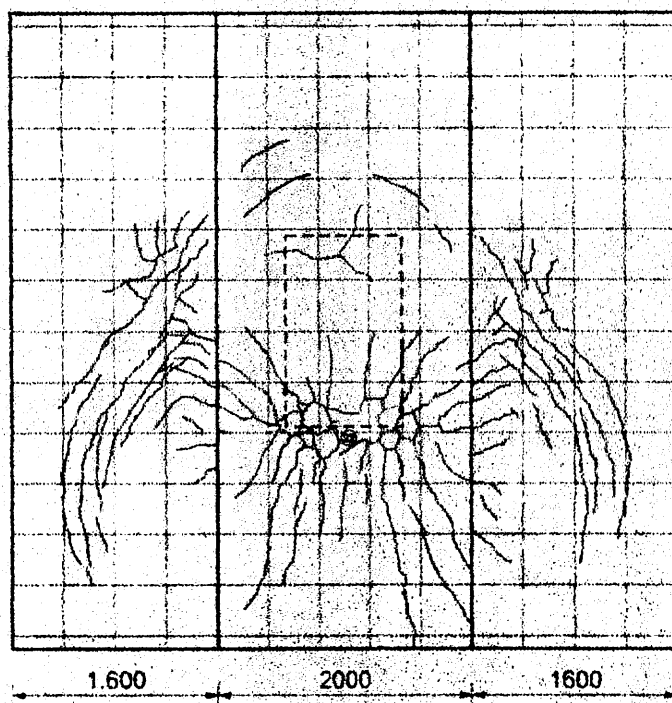


Fig. 8 Crack pattern of specimen 3 in Fig. 7

Prof. Hegger from the University of Aachen accompanied these experiments with FE.- Computations and elaborated design tools for practitioners.

Further discussions concerned the use of monostrands in different cable versions. As we demand always at least two corrosion barriers around the prestressing steel and have mostly additional material layers at the saddle, there arises the question what moves against what at those points during prestressing. We found that it is not possible to answer this question by simple reasoning. Experience shows that without a future physical modification of friction between the different layers, it is not possible to make a safe prediction. It causes not only problems to lengthen the protection layers in the end trumpets at the cable heads correctly, but

may also lead to uncontrolled movements between the steel strand and the surrounding monostrand material. Tests at our institute have shown that already relative movements of about 30-80 cm, depending on the pressure perpendicular to the saddle surface, can damage the monostrand mantle (Fig.10).

3. New solutions

With every reached state of development the question raises, what can we still improve? To the authors opinion the main problem is the space one needs for external cables and their anchoring, the reduced internal lever arm and the costly anchoring corbels. It is a discrepancy that on the one hand we do not trust in conventionally grouted cables, thus using external prestressing, and on the other hand, within the concept of mixed design, we use conventional grouted cables. Why not combining the advantage of only exchangeable cables with the advantage of an arrangement inside the concrete (Fig.9).

Since quite some time the author propose to put newly developed exchangeable cables inside the concrete part of the crosssection again. Then we have a higher internal lever arm, and again the appreciated protection against fire, vandalism etc. This leads per se to a much easier and less costly anchoring of the cables, as shown in Fig.(9), avoids the high amount of mild reinforcement in this regions and most of the time consuming labour for installing the high amount of mild steel. How can this be done?

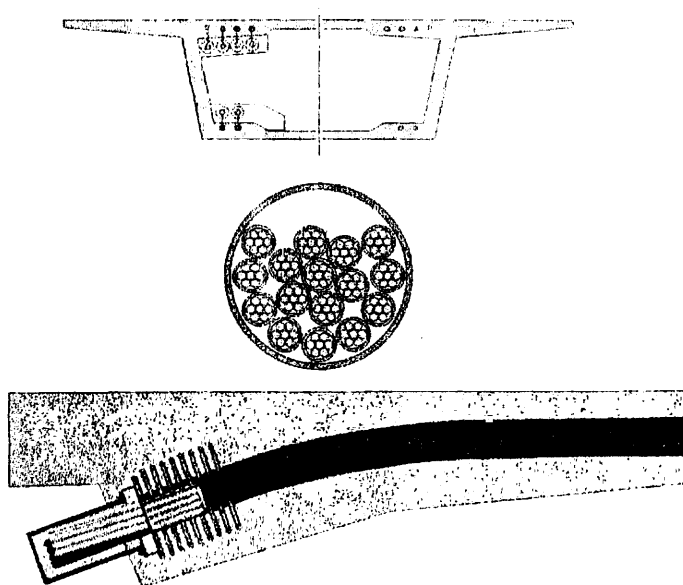


Fig.: 9 Anchor at unbonded cables inside concrete

We need steel strands with a first cover of corrosion protection and — in Germany demanded anyhow — a second layer around it, separated by a reasonable distance of about 3mm on each side to remove easily the complete inner "monostrand" not just the pure steel strand as is some times proposed. The reason for the latter is, that in long cables the steel strand cannot be removed without a complete destruction of the corrosion protection material— grease— and the monostrand mantle (Fig.10). It is also not possible to draw in a new steel strand retaining intact the corrosion protection material over whole length of the cable. One further has to take care that no water, whenever entering between the two protection layers, leads to freezing and

splitting of the surrounding concrete. Such cables can easily be developed using e.g. a bigger outer tube with monostrands in it and an appropriate foam layer between the first and second layer allowing a 10 % water expansion in case of icc formation. Such type of cables could also be used building segmental bridges.

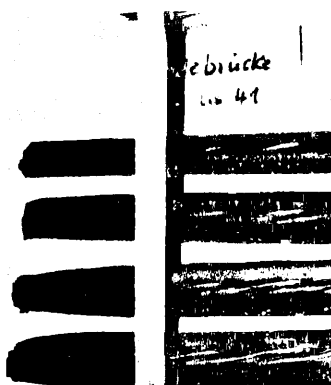


Fig. : 10 Damage in monostrand mantle due to perpendicular pressure and relative movement

Completely new cables of this type, however with rectangular steel bars having the same strength as the common strands, are already under licencing discussions. Such cables have a much smaller cross-section with a high force capacity due to the fact that their available area is about 30% higher than that of classical strands with the same outer diameter.

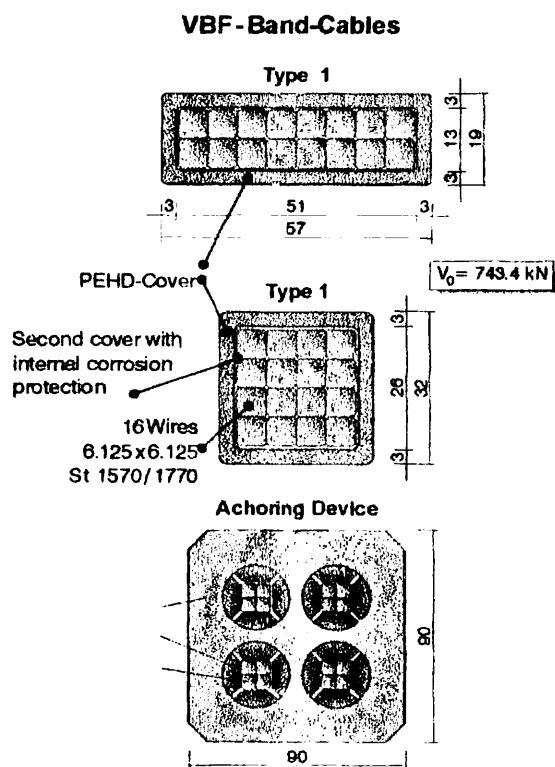


Fig. : 11 New cables system

4. Summary

As a consequence of unsatisfactory experience with bridge structures in Germany one has started to build external prestressed box type bridges around the nineties. In the meantime about 40 bridges have been built and helped to gain experience. Disadvantages as possible kinking at saddles, problems with saddle geometries and such with anchoring blocks have been discussed and partially removed.

The gathered experience gave cause to look for further progress with unbonded, exchangeable cables, now arranged within the concrete area again. This is a full turn, back to the old type of prestressing and just avoiding mortar as a corrosion protecting material and all the problems connected with grouting.

Eventually new strands will be substituted by thin rectangular steel bars in new unbonded prestressing cables.

Literatur:

- [1] Großmann, F., Wirtschaftlichkeit von extern vorgespannten Brücken,
 Externe Vorspannung und Segmentbauweise, Workshop an der
 TU. Karlsruhe Oct.1998, Verlag Ernst u, Sohn 1998

- [2] ARS 17/1999 Spannbetonbrücken-Richtlinie für Betonbrücken mit externen
 Spanngliedern *Ausgabe 1999 mit Anlagen und Musterzeichnungen*

- [3] Standfuß, Erläuterungen zur Richtlinie für Betonbrücken mit externen
 Abel, Spanngliedern, *Beton-und Stahlbeton 93(1998), Heft 9*
 Havaresch

Strategies for improvement - Approach in France -

R. CHAUSSIN

Mission d'Inspection Spécialisée des Ouvrages d'Art, France

A. CHABERT

Chargé de Recherches au L.C.P.C., France

1 General

The major concern of any organization involved in the commissioning and management of a structure is to obtain an appropriate level of serviceability and of safety during an **intended life-time... and beyond**, which implies, in particular, that the full service life is achieved with only routine maintenance. This requirement, based on economic reasons, is important but not sufficient because it is well known that when a structure reaches its theoretical life-time, it is not scrapped except if it shows evidence of a very poor behaviour. In other words, its retirement time is not fixed by a predetermined life-time but by its appearance. As a consequence, an additional requirement is necessary : **the collapse of a structure must be preceded sufficiently in advance by warning signs** easily detectable during periodic visits. It is of fundamental importance that the death is not sudden and that its approach is expressed by clear symptoms.

It is a widespread opinion that prestressed structures are more sensitive than other structures to brittle failure. This received idea is greatly questionable :

- even traditional robust-looking structures may suddenly collapse, such as the Wilson bridge in Tours, an old masonry bridge on river Loire, which caved in some 20 years ago due to an undermining problem
- in the other hand, prestressed structures generally show clear distress signs, readily apparent to an experienced eye before collapse. This rule may have exceptions, but very simple precautions in the design and detailing of new structures are enough to avoid such particular cases (see 5 below).

The prestressing elements (generally wires or strands) are unanimously considered as the weak link of prestressed structures, especially in post-tensioned structures. Up to now, and probably still for a long time, these elements are made of high strength steel sensitive to different forms of corrosion : classical corrosion (by dissolution or rust formation) but also stress corrosion due to hydrogen embrittlement.

In pretensioned monolithic members, the concrete cast in full contact with the prestressing elements provide them with an efficient protection.

In post-tensioning, the tendons are generally threaded in ducts which need to be sealed and filled with a protective material, and many problems may arise concerning :

- the quality of sealings
- the continuity of the ducts and their watertightness, especially when internal tendons cross joints
- the good filling of the ducts
- the quality of the filling material.

To improve the durability and the safety of prestressed structures, these fundamental items have to be taken into account. But other aspects need to be considered :

- the quality of design, in all its details
- the quality of materials
- the quality of construction, including the coordination of construction methods and installation technologies
- a clear definition of the roles of the various participants : client, consulting engineer, architect, general contractor, post-tensioning specialist company...
- a rational organization of public procedures.

2 Organization improvements

In the field of organization, two aspects are worth being emphasized :

- the fundamental role of post-tensioning specialist companies
- the importance of public procedures in matter of control.

2.1 The role of the post-tensioning specialist companies

It is desirable for prestressing to be entrusted to companies known as post-tensioning specialist companies (or PT SCs) whose main activity is post-tensioning rather than to general contractors who also have to deal with a large number of other problems. There is no contradiction between this statement and the fact that some general civil engineering contractors have developed their own prestressing system, but in such cases the necessary skills will have been brought together in a « prestressing » department which operates like a PT SC.

These PT SCs must have experts –thoroughly familiar with the potential and the limits of the prestressing system they use- who are able to develop the system, to adapt it in response to specific problems, to advise consulting engineers in their decisions, to provide technical assistance to general contractors, to react rapidly in the event of a difficulty and, finally, to be responsible for achieving quality.

Additionally, of course, they have to install the prestressing system and, therefore, they must have the resources to :

- manage procurement and acceptance procedures for incoming materials and components
- dispatch equipments to job sites
- ensure the traceability and the documentation
- maintain and repair the equipment
- prepare the work instructions for job sites
- train the personnel and particularly the « post-tensioning site supervisors » responsible for the site operations.

2.2 Public procedures

In matter of post-tensioning, an **approval procedure** is inescapable but its content may be quite different according to the intended level of reliability.

Several European countries have set up such a procedure.

In France, it is managed, since 1965, by a commission of experts known as « Commission Interministérielle de la Précontrainte » or C.I.P., with an extensive scope of activities since it includes :

- prestressing steels
- prestressing systems
- grouting materials
- stay cables.

Concerning PT systems, the CIP has in charge to :

- deliver temporary or long term approvals
- coordinate and supervise the verification of factory production control
- coordinate and supervise job site audits the aim of which is to assess the quality of procedures and of installation provided by the PT SCs. These audits must not be confused with the work supervision performed on behalf of the client.
- collect and exploit the site observations
- issue recommendations, especially when the feedback informations show problems of a general nature, so as to improve the situation.

European authorities have appointed EOTA to prepare an approval procedure for PT Systems. Unfortunately, it is strictly limited to manufacturing aspects and completely ignores installation problems.

Furthermore, the approval and certification functions are neatly partitioned and entrusted to distinct qualified national bodies. One can wonder how such a multi-stage rocket can work and what happens when a PT system holder, to get the approval, chooses and pays his approval body in a country A, then its certification body in a country B.

3 Steel protection improvements

Several barriers ensure the protection of prestressing steels.

In the United Kingdom, TR 47 introduced the concept of multi-layer protection, the philosophy being that should any layer become defective, the remaining layers would provide protection. For bridges, four layers may generally be identified :

- the filling material
- the duct and the sealings
- a good quality concrete
- an efficient waterproofing of the deck allied to a good water management.

If all these elements are important, the first ones, ensuring the close protection of the tendons merit a particular consideration.

In fact, if a perfect **watertightness of ducts and sealings** is a good safeguard against the ingress of deleterious agents it also efficiently blocks their outflow : one must be sure of the **innocuousness of the filling material** and avoid setting the fox to mind the geese.

In most cases, the filling material is a **cement grout** and one can wonder about its quality and its efficiency.

3.1 Improvement of cement grouting

There has been tremendous technological progress since the heroic post-war period : it concerns both the detail design of prestressed structures as a whole and the performance of the ducts, the development of arrangements facilitating grouting and the improvement of mixing and injecting equipment.

Nevertheless, problems do subsist and it is not uninteresting to summarize the French experience in this field during the thirty last years.

The publication of various regulatory texts, in particular a provisional LCPC-SETRA directive in March 73 on the grouting of ducts, accompanied by the development of regional grout formulas and the application of a policy of checking the filling of the ducts by gamma radiography, gave us nearly two decades of euphoria during which problems were rare and the final inspections almost always proved satisfactory.

However, the '80s witnessed a new trend : the use of superplasticizers spread, changes were made in the standardization of the cements, thereby changing the sources of supply of products suitable for these very special applications, and inspections became laxer.

By the beginning of the '90s, the first signs of problems could be detected on some structures, in the form of generally whitish substances on top of the grout filling the temporary injection caps.

These presumptive anomalies were soon confirmed on the occasion of a campaign of investigation by gamma radiography of the external tendons of a box-girder bridge built shortly before : filling defects were found at the high points, and opening a few ducts revealed, in these zones, the presence of a substance having a pasty consistency, topped by a film of water and an air pocket.

Checks on other structures under construction have since shown that the phenomenon also exists in common structures such as slab bridges and is more widespread than one might think.

On the initiative of the contractor who built the first structure surveyed, grouting tests were performed at the CEBTP on transparent tubes 5 metres long, inclined approximately 30 degrees from the horizontal, containing strands like a real duct. Several common grout compositions (including the one used on the structure concerned) were tested. It was found that :

- many of these grouts had a fluidity comparable to that of water (flow time in the Marsh cone less than 10 seconds),
- these same grouts were the site, immediately after injection, of a migration of whitish substances and air bubbles.

Chemical and mineralogical analyses revealed that :

- the supernatant liquid had a very high potassic alkalinity,
- the paste, which hardened rapidly in contact with air, had a composition rich in ettringite, portlandite, and calcite, with a large concentration of admixtures.

It seems in fact that the most alarming phenomenon is the presence in contact with the tendons of a suspension, even though strongly alkaline, which constitutes a perfect electrolyte. Unhomogeneities, at the surface of the steels can induce the formation of anodic and cathodic zones : in the first ones, dissolution and rust formation occur, in the second ones, hydrogen embrittlement arises. Such unhomogeneities may be favoured by condensation of pure water on the tendons especially external if they cross a water-air interface.

Given the extent of the problem, even if it has not been proved that the defects found actually threaten the preservation of the post-tensioning steels, the authorities have reacted in three ways :

- by publishing in 1996 a SETRA-LCPC memorandum on grouts. This document call the attention of project supervisors to the problems encountered and makes the performance of inclined tube tests as suitability tests systematic ;
- by setting up, on CIP's initiative, an approval for grouts and grouting equipments. The corresponding procedure has been officialized by a text issued in August 1999 (see [1]) ;
- by intensifying the research being done at the LCPC, in collaboration with the professionals concerned, to quantify the parameters influencing the quality of grouts and improve the mix design of these products.

3.2 The inclined tube test

The inclined tube test was thought up by the CEBTP to meet the needs of the survey mentioned above. Its initial objectives were to reproduce actual grouting conditions at reasonable cost and to compare the behaviour of some common mix designs. In fact, it proved very efficient and selective (see 3.4 hereafter).

The test, specified by the French approval procedure, now consists, schematically, of the parallel grouting of two transparent tubes 80 millimetres in diameter each containing 12 slack strands (to reproduce the drain effect of the tendons). One tube is grouted in a single pass, the other in two steps separated by some interval. The volumes of air, water and substances derived from possible segregation are then evaluated at different times.

The results are considered acceptable if the sum of these different volumes is less than 0,3 % of the total volume of the tubes.

3.3 The French approval procedure

This procedure managed by the CIP in reality concerns several interested parties : a mix designer, responsible for the composition of the grout (and for the regularity of production of its constituents) and one (or more) applicator(s), who can only be PTSCs responsible for their part for suitably controlling the proportion of water, the mixing, and the grouting of the duct using efficient equipment, in accordance with general protocols accepted by the mix designer and specified, for each site, by a quality assurance plan specific to the application concerned.

The granting of the approval depends on prior testings including, in addition to the more or less traditional tests on the constituents and on the grout itself, grouting tests on inclined tubes performed with the equipment of the PTSC(s) associated with the mix designer.

The approval certificates clearly identify :

- the mix designer and the approved mix
- the applicator(s) and the equipment he has (they have) to use for injecting the considered mix.

There is long-term follow-up of production and of the sites by periodic inspections organized along the same lines as those applied for decades to post-tensioning systems.

Naturally, the granting of the approval is preceded by a probationary period during which the product and its placement must prove their effectiveness on sites.

3.4 Research

Research of more or less fundamental character has been undertaken by :

- the cement and admixture producers, in collaboration with the PTSCs to settle mix formulations
- the public laboratories of LPCs network, working with the professions concerned, to improve the tests on cement grouts, to investigate the conditions of a good filling, to optimize the rheological properties and the formulation parameters of the grouts and at last to detect bleeding in the ducts, since free water is the most dangerous enemy of steels.

Some interesting results have been achieved :

- two mix formulations proved to be satisfactory, without any bleeding and got the approval ; the appraisal of a third one, with high thixotropic properties, is ongoing

- the various parameters of the inclined tube test were investigated, (see [2]) to verify that they suitably take into account the factors increasing the segregation and bleeding phenomena, in particular :
 - the number of strands, which actually magnifies bleeding by drain effect
 - the value of inclination, which significantly acts upon segregation ; it appeared that this phenomenon, known as Boycott effect (from the name of a British physician who discovered it about blood circulation) is maximum for an angle between 30° and 60°
 - the type and the nature of the tube (corrugated or smooth) which does not seem to have a great influence
- new tests were experimented on smaller 1 m inclined tubes, with the aim to replace the traditional bleeding test carried out on 100 ml specimens which is not selective enough
- experimentations were performed on sinusoidal shaped 20 m long transparent tubes to observe the flow, to identify the mechanisms of formation of air pockets and to appraise different dispositions or procedures (regrouting, vacuum grouting, use of expansion tanks...) likely to improve the filling.

At last a new method, based on the measure of the dielectric constant, is being developed to detect the presence of water in external ducts and seems to give promising results.

3.5 Advantages and disadvantages of cement grouts : bond

The most common and probably the less expensive of filling materials, cement grout, needs water in its composition and if free water subsists after setting, it may have a detrimental effect on tendons.

But another property of cement grout merits to be discussed : bond.

In **internal prestressing**, bond is generally considered as an **advantage** since it forces the steels to follow the deformations of the surrounding concrete and hence to contribute to the resistance of the structure. Furthermore, if a tendon locally fails, it is re-anchored on a short distance by bond, and its prestressing effect does subsist on the major part of its length. But this wonderful property may have the **drawback** of hiding the damage if the failure of the steels occur in a zone submitted to small load effects, where the integrity of the structure can be only due to the tensile strength of the concrete. As prestressing is retained in the zones which are submitted to important action effects, the structure appears perfectly sound, but any unusual load may lead to its complete collapse if its redundancy is not sufficient.

In **external prestressing**, bond is a **total disadvantage** since it turns the bundle of strands forming the tendon into a solid composite element, which strongly complicates its dismantling if it seems necessary to change it.

It is easy to by-pass the problems mentioned above in the case of internal prestressing (see 5 hereafter). Its why cement grouts probably have still a long future in this field. But in matter of external prestressing their use is more questionable and, in France, a recent official text, published, in February 2001 (see [3]) strongly recommends alternative solutions.

3.6 Alternative solutions and improvements for external prestressing

These solutions **avoid bond** as mentioned hereabove. Among them, some are very classical :

- injecting the ducts with a petroleum wax, generally preferred to a grease (mix of soap and oil) for its stability due to its crystalline structure. But these waxes need to be injected at high temperatures (about 90° C) which poses a problem of contraction on cooling especially if steam has been locked up in the ducts. Moreover, HDPE continuous tubes are commonly used, crossing the deviators without any interruption thanks to a double envelope system and a local strengthening of the tube (e.g. by placing a round annular steel duct inside) is often necessary to avoid any risk of the tendon perforating the plastic in the curved zone. At last, this solution is particularly sensitive to fire
- using unbonded, greased and sheathed monostrands to form the tendons. These monostrands may be threaded into a general PHED duct which is injected with a cement grout, before stressing the strands, to avoid the ripping of the individual HDPE sheaths due to the pressure of the top layers in the curved sections. Such a solution provides an excellent protection of the steels in full contact with grease and their individual sheathings, the cement grout acting after hardening as a spacer and hence needing no out of reach perfection. Unfortunately it is a costly solution.

Other solutions can be devised to replace cement grouting :

- flexible cold injectable materials
- protection by dried air (in conjunction with galvanization for instance).

Anyway, a general tubing seems desirable to control the whipping of a broken element.

4 Improvements concerning confinement

It is well known that traditional corrugated steel ducts are not perfectly watertight and may be corroded if they are not protected by a compact concrete. It is why **corrugated plastic ducts** were developed, some ten years ago by the system holders, strongly incited by the UK ban on internal prestressing in the middle '90s.

Nevertheless, these ducts have their weak points :

- they have a high coefficient of thermal expansion and need to be carefully designed not to slacken between their supports if temperature arises
- they are more sensitive than traditional ones to mechanical attacks and often require appropriate strengthening at the level of their supports or their mutual contacts
- they are expensive.

Due to these reasons and to a bad experience (Issy-les-Moulineaux bridge) in the early '80s, this type of duct is not much in favour in France.

In other respects, a certain number of local problems have been solved these last years :

- coupling ducts through match cast joints

- placing vents at the high points of external tendons. Up to now the double tubing through deviators or bulkhead cross-beams was supposed to be such a barrier that the vents were frequently located outside the concrete, which made unavoidable an air pocket at the upper part of the duct. In fact, very simple dispositions bring an efficient solution to that difficulty.

At last, some other details should merit attention to make easier disassembling and reassembling the anchor caps so as to inspect their filling.

5 Fallback positions

In spite of all these improvements, one must keep in mind that the failure of a tendon always may occur, and it is important to take precautions so that its effects are limited, easy to detect and to repair.

First of all, it is a good thing, whenever it is possible, not to put all one's eggs in one basket and to **combine internal and external tendons**.

In **internally prestressed members**, a particular attention must be given to zones submitted to small action effects, where prestress may totally disappear without any warning sign if the tendons are bonded, causing an insidious embrittlement of the structure as mentioned in 3.5 above. This phenomenon can easily be counteracted by placing in these zones enough reinforcing steels to withstand by themselves the action effects susceptible to occur even if cracks appear. In that case, crack formation constitutes an evident sign of distress for an experienced eye and the structure manager has enough time to take appropriate measures.

Externally prestressed structures must of course be designed so that the failure of one tendon does not seriously impair their proper functioning. The broken tendon is then easy to observe and to replace.

Furthermore, it is often advisable to incorporate, during the design phase, the potential for **additional external tendons** (anchor and deviation points) for strengthening purposes, either to respond to changes in operating conditions or to compensate the loss of part of the existing prestressing or, at last, to counteract an unexpected behaviour of the structural concrete. The additional cost of such measures when they are included in the initial construction bears no relation to the cost of installing them later in the structure when it is in service.

Bibliography

- [1] Circulaire n° 99-54 du 20 Août 1999 instituant un avis technique sur les coulis – MELTT
- [2] Leroy R., Willaert M., Rouanet D., Mirmand H. (2000). Identification des paramètres favorisant la formation des poches d'air et d'eau dans les coulis de ciment pour gaines de précontrainte. Bulletin des Laboratoires des Ponts et Chaussées n° 229 pp. 53-69.
- [3] Circulaire n° 2001-16 du 28 Février 2001 relative à la conception de la précontrainte extérieure au béton. MEL'IT-2001-5.

North American Strategies for Improving Bonded Post-Tensioned Concrete Construction

Randall W. Poston & Jeffrey S. West

WDP & Associates, Inc.

Austin, Texas, USA

1 Introduction

Post-tensioned concrete construction was introduced in North America in earnest in the late 1950's. Most of the early post-tensioning usage in North America was grouted tendon applications in bridge construction. In the mid-1960's, unbonded post-tensioned concrete for building construction evolved rapidly and has become today one of the predominant forms of construction for concrete buildings. Until only recently the use of grouted post-tensioned concrete remained for the most part relegated to the bridge industry and other specialized applications such as containment structures, tanks, and tie-backs.

There has been a major push to "re-introduce" grouted post-tensioned concrete into the building industry in the last five or so years. This is in part due to perceived durability problems with unbonded tendons. Most of the misconceptions prevalent in the industry regarding the durability of unbonded tendons are mostly unfounded, but have been fostered by relatively few "celebrated" building repair projects in which most if not all tendons were replaced. Post-tensioning manufacturers have seized the opportunity by designing a new generation of bonded post-tensioning systems as an enhanced durability feature. These new generation of bonded systems for buildings also attempt to exploit North American building code provisions by eliminating the minimum mild reinforcement required in unbonded construction.

Despite the perceived durability benefits of grouted post-tensioning construction, there have been recent observed problems with grouted tendons, primarily due to presence of voids in anchorage regions. In some cases, tendons have failed due to corrosion, although there are no known reports of actual structural failures in North America due to tendon corrosion.

This paper will explore some of the problems that have occurred in North America with grouted post-tensioned concrete construction. This will be done in the context of selected case studies. Then, the strategies that are being adopted by the post-tensioning industry in North America to address grouting issues will be summarized.

2 Background Case Studies

To provide background of the issues regarding durability of grouted post-tensioned concrete construction in North America, two bridge projects will be discussed.

2.1 Internal Tendon Bridge

An early generation precast segmental bridge had been in service in the northeastern US for more than 30 years. The pier sections of the bridge are precast, post-tensioned cantilever beams connected by “drop-in” precast, pretensioned beams as seen in Fig. 2-1.

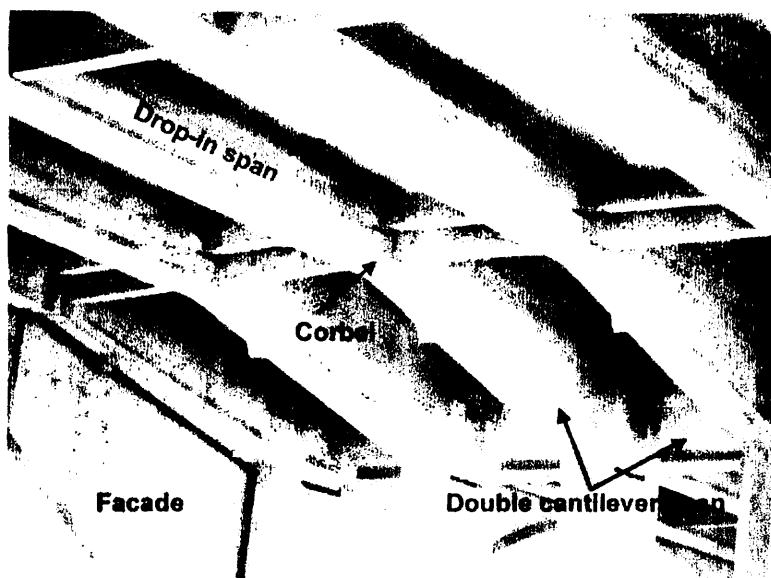


Fig. 2-1: Early Generation Precast Segmental Bridge in US

Concerns arose about the structural integrity and durability of the corbel region, as shown in Fig. 2-2, where the pretensioned girders transfer load into the cantilever due to deterioration in the corbel region and the lack of redundancy in the structural system.



Fig. 2-2: Corrosion Deterioration in Corbel Region

The bridge contained tapered, post-tensioned girders that cantilevered out from the bridge piers. Two tendons are anchored at the corbel end of the cantilever. They follow a parabolic trajectory into the corner of the web and up through the flange until they are anchored at the corbel on the opposite end of the girder. At the corbel, the tendon ducts are anchored horizontally next to each other and as they pass through the girder they change positions relative to one another until one duct is located directly above the other. Each tendon duct was composed of a smooth steel cylinder with a wall thickness of approximately 1 mm and a diameter of approximately 60 mm. Each duct contained 12, seven-wire, prestressing strands.

In order to assess the integrity of the post-tensioning tendons, a non-destructive method known as impact-echo was utilized to locate voids. At select locations where voids were determined, openings were made in the duct for inspection. The voids found ranged from completely empty ducts as seen in Fig. 2-3 to partial voids.



Fig. 2-3: Voided Tendon Duct

The remarkable finding was that although the anchorages in the corbel region had been exposed to chemical deicers from years of leaking expansion joints, no failed strands were found. The worst case was surface corrosion and some mild pitting that was judged to not have affected tendon capacity to any significant degree.

Although it was determined that the tendons in this bridge had not suffered extensive corrosion, all voids were retrofit grouted. The regrouting was necessary not only to ensure continued corrosion protection, but also because the beams had been designed for ultimate strength utilizing the increased strain capacity of a bonded tendon.

2.2 Externally Post-tensioned Precast Segmental Bridge

The bridge in question connects the mainland to a barrier island in the Gulf of Mexico in the southern US. The total crossing length is 5870m with 140 spans. The typical span length is 41.5m, with a 68.6m main span providing 19.8m of vertical clearance over the Intracoastal

Waterway. The bridge is a constant depth, single-cell precast segmental box girder post-tensioned with external tendons. The typical 41.5m long spans have six external tendons, consisting of nineteen seven-wire strands each. The bridge was erected using span-by-span construction, and is configured to be six span continuous.

Two tendon failures were discovered during an August 2000 inspection. One of the tendons experienced multiple strand ruptures within the free length of the tendon, as shown in Fig. 2-4. The second failure involved a complete failure of the tendon at an end anchorage, as shown in Fig. 2-5. An inspection of the bridge and investigation of the tendon failures revealed the following possible factors contributing to the corrosion-induced tendon failures:

- **Environment:** Several environmental factors contributed to the observed tendon corrosion. The geographical location of the bridge provides a humid, coastal environment with a high annual average temperature. In addition, the bridge is a low-water crossing for the majority of its length, placing the superstructure in relatively close proximity to the surface of the saltwater.
- **Grout voids at anchorages:** Numerous grout voids were found, particularly for anchorages at inclined locations. Varying degrees of tendon corrosion were observed where anchorage voids were present. Anchorage voids may have resulted from excessive grout bleed water accumulation, poor selection of grout materials and proportions and/or poor grouting practices.
- **Moisture and chloride penetration through anchorage end protection:** Observed conditions suggest moisture, and possibly chlorides, entered the grout void space behind several anchorages producing severe conditions for tendon corrosion. Severe corrosion of the anchorage wedge plates and strand tails, and strand corrosion within the wedges and immediately behind the wedge plate point to moisture and chloride penetration through the grout and bituminous coating used to provide the anchorage end protection.
- **Grout voids along external tendon length:** Grout voids were detected along the free length of many external tendons. In most cases, these voids did not result in tendon corrosion, as the duct appeared to provide a suitable protective barrier. However, in some instances, tendon corrosion ranging from mild surface corrosion to localized pitting corrosion with wire section loss was observed. In these situations, it is assumed that moisture, and possibly chlorides, reached the tendon through a breach in the duct (see next bulleted item) or possibly from penetration at the end anchorages. Grout voids were attributed to a possible combination of excessive grout bleed water, poor selection of grout materials and proportions and/or poor grouting practices.
- **Damage to external tendon ducts:** Damage to external high-density polyethylene (HDPE) tendon ducts was observed in the form of split ducts and mechanical damage. Numerous duct splits were found throughout the bridge. These splits were attributed to substandard duct materials. Tests on samples of the HDPE duct removed from the bridge indicated that the environmental stress cracking resistance of the material did not meet the requirements of the applicable standards in place at the time of construction. Mechanical damage to the ducts was very limited, existing primarily in the form of intentional punctures. These punctures, which had been patched with a bituminous material, were assumed to have been used to check for voids and to regrout voids. In several cases, tendon corrosion was observed coincident with duct splits and duct mechanical damage.

A thorough inspection of the bridge identified all possible deficiencies, and repairs were conducted. Repairs included removal and replacement of tendons with the most severe corrosion, regrouting anchorage voids, repairing split and damaged external ducts, and improving anchorage end protection.

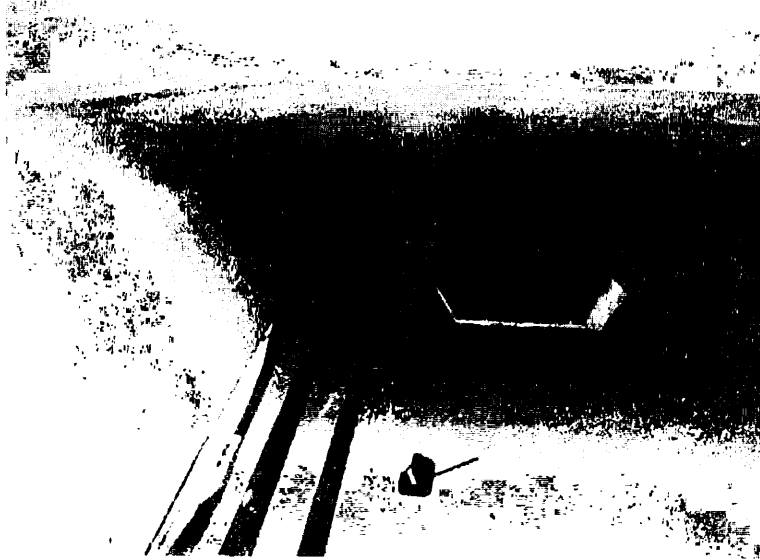


Fig. 2-4a: Multiple Strand Failures Within Tendon Free Length (photo courtesy of the Florida Department of Transportation (FDOT))



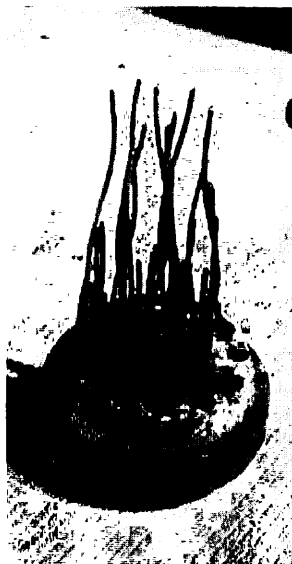
Fig. 2-4b: Corrosion Damage and Broken Strands Within Tendon Free Length (photo courtesy of FDOT)



Fig. 2-5a: : Tendon Condition Following Failure at Anchorage (photo courtesy of FDOT)



*Fig. 2-5b: Damage at Diaphragm Due to Tendon Failure
(photo courtesy of FDOT)*



*Fig. 2-5c: Broken Strands at Wedge Plate
(photo courtesy of FDOT)*

3 Grouted Tendon Research

There have been several studies in North America conducted on durability of grouted post-tensioned concrete structures and on improving the performance of grouts. The U.S. Army Corps of Engineers conducted long-term exposure tests on post-tensioned beams exposed in a tidal flat at Treat Island, Maine, since 1956 [Freyermuth (1991)]. Overall, the test specimens performed well, with no indication that the integrity of the post-tensioning has been compromised after decades of exposure to chlorides and cyclic freezing.

An NCHRP report, issued in February 1989, was based on a 1-year study on corrosion protection systems for prestressed concrete structures [Perenchio et al. (1989)]. Severe accelerated corrosion tests were conducted on various corrosion protection systems. The authors found the epoxy coated ducts, hardware and strands performed the best. However, the North American post-tensioned industry standard consisting of a bare anchorage, galvanized steel duct with duct taped joints, bare prestressing strand, and normal cementitious grout adequately protected the encased bare strand from corrosion when embedded under only about 25mm of concrete cover.

A comprehensive study was conducted at The University of Texas at Austin (UT Austin) on improving the durability of bridge decks by using transverse prestressing [Poston et al. (1987)]. In general, grouted galvanized duct was found to protect adequately the tendons between the anchorage. Thin concrete covers at anchorages were found to provide inadequate corrosion protection. The authors recommended full encapsulation of all components of the post-tensioning system. Additionally, cracking in the concrete was found to promote greatly the corrosion of the mild reinforcement in the slabs; prestressing had a significant effect in reducing the penetration of chloride ions at crack locations when the cracks were limited in width.

Since 1992, an ongoing research project sponsored by the Texas Department of Transportation (TxDOT) at UT Austin has been studying the corrosion protection of various types of grouted post-tensioning systems [West et al. (1999a), Schokker et al. (1999a), West et al. (1999b), West et al. (1999c), Schokker et al. (1999b)]. This research also included the study of stay cable corrosion protection [Hamilton et al. (1998a), Hamilton et al. (1998b)]. The salient conclusions from this ongoing research study are:

- For external tendon systems, there is uncertain and questionable protection afforded by the grout if the duct is compromised,
- Plastic duct systems for internal tendon systems eliminate secondary durability problems such as spalling and staining,
- Steel ducts may not provide an effective long-term barrier to corrosion, and
- Fluidizing admixtures and pozzolans can improve grouting operations and the protection value of grout.

Other recent North American research related specifically to improving performance of grouts includes Thompson et al. (1992), Ghorbanpoor et al. (1993), Sprinkel (1997), Hope and Ip (1988), Bastien et al. (1997) and Ip and Manning (1993).

4 Strategies for Improving Durability of Post-tensioned Structures

The primary efforts towards improving the durability of post-tensioned concrete in North America have focused on improving grout materials and practices. Although some local differences exist, most current national and state specifications for grouting in bridge and building construction are similar and contain only minimal requirements for grouting. In most cases, these specifications have experienced little or no change in some time. The grout requirements in place in much of North America in the past and at present consist of the following:

- **Grout materials:** The most commonly specified grout is portland cement based with a water/cement ratio of 0.45. The use of an expanding admixture may be optional or required. Other admixtures are permitted in most cases, although no guidance or limits are present. Some agencies allow use of fly ash for partial cement replacement.
- **Grout testing:** Grout testing requirements can be described as minimal to none. The American Association of State Highway and Transportation Officials (AASHTO) Construction Specifications for Bridges mentions the use of a flow cone test for fluidity, but it is not a specific requirement.
- **Grouting Practices:** General information is provided for the location of injection ports and vents, flushing of ducts prior to grouting, sequence of vent closing during grout injection, maximum pumping pressures and treatment of vent locations after grouting.

Although the industry state-of-the-practice described briefly above has proven to be adequate in many cases, there have been instances of grout problems leading to structural and durability concerns. As more owners and engineers become aware of grout related problems, and as more structures are being examined specifically for grout problems, the incidence of known grout problems appears to be increasing. This increased realization of existing and potential problems in North America has led to the recent development of improved specifications and practices for grouting. The most formal treatment of improved grouting specifications is the Post-Tensioning Institute Guide Specifications for Grouting of Post-Tensioned Structures [PTI (2001)]. In addition, the American Segmental Bridge Institute has developed and issued recommendations for improved grouting practices. Some governing agencies have also begun to develop stricter requirements for grout materials and practices. Individual companies have also responded by testing and developing new grout materials, specifications, testing procedures and personnel training and certification. The main aspects of the new strategies for improved grouting are discussed in the following sections.

4.1 Improved Grout Materials

Several significant changes have taken place in requirements for grout materials. The first is the use of grout classifications, dependent primarily on the exposure conditions to be encountered. The grout classifications provide specific material requirements, and dictate fresh and hardened property testing requirements. The material requirements for non-aggressive exposures are similar to the grout requirements widely used in current state-of-the-practice. For aggressive environments, increased guidance and flexibility are provided for the use of mineral and chemical admixtures.

Prior to construction, a contractor must demonstrate that the grout intended for use meets specific physical property requirements. For non-aggressive environments, grouts must meet requirements for setting time, compressive strength, pumpability and fluidity as measured by the flow cone test, and control of bleed water formation as tested using a simple wick-induced bleed test. For aggressive exposure applications, grouts must meet the same requirements as for non-aggressive exposures, and are subject to additional requirements for low permeability, volume change and control of bleed under pressure.

In addition to the physical property testing prior to construction, physical property testing will now also be required during grouting operations for quality control purposes. Again, the testing requirements are a function of grout classification. For grouts to be used in non-aggressive exposures, quality control testing is limited to grout compressive strength and fluidity. For aggressive exposures, additional testing requirements include volume change and bleed under pressure. These tests must be conducted regularly as a function of grout volume pumped.

Another change to grouting practice is the formal recognition of commercial, pre-packaged grouts. Such grouts will be subject to the same material and physical property testing requirements as grouts prepared on-site. However, these grouts allow the contractor to avoid the development and testing process prior to construction by purchasing a pre-packaged grout that has already been tested and certified to meet the applicable requirements. Physical property testing requirements for quality control during grouting operations are also reduced for commercial, pre-packaged grouts, only requiring regular testing for fluidity during production.

4.2 Construction

Numerous advances have been proposed for construction-related grout issues. These advances address issues prior to, during, and following the grouting operations.

Prior to the initiation of grouting, the contractor will be required to document and demonstrate his or her plans for grouting. This may include development of written grouting procedures to be reviewed and approved by the design engineer. In addition, the contractor may be required to conduct field grout batching and testing trials, and field mock-up tests of grouting operations. The intent of these requirements is to ensure that the contractor carefully plans all aspects of the grouting operation prior to construction, and to have the contractor demonstrate his or her personnel and equipment capabilities.

Detailed requirements are also available for issues including:

- Protection of the tendons through all stages of construction,
- Treatment of ducts and use of temporary seals and covers,
- Locations and treatment of grouting injection ports and vents,
- Storage of materials,
- Grout injection equipment,
- Grouting operations, and
- Temperature considerations.

Other developments involve new requirements for post-grouting operations. These include inspection of all high point vents on multiple occasions following grouting, with all voids to be topped off with grout. In addition, anchorages are to be inspected for voids 48 hours after grouting. These inspections may involve drilling out of the injection port, and insertion of a fiberscope or videoscope to discern conditions behind the anchorage. All detected voids are to be regouted prior to the installation of anchorage end protection. Improved protection requirements

have also been developed for grout vent and inlet locations, and for end anchorage protection. End anchorage protection requirements in aggressive environments include the use of permanent, non-corroding end caps, provision for anchoring pour-back concrete or mortar to the structure, and use of waterproofing membranes over the pour-back material instead of bituminous coatings.

4.3 Grouting Personnel Training and Certification

The North American industry has responded to this issue by developing a formal training and certification program. The first program was held in August, 2001. This is a continuing and evolving effort, with several future programs planned. In addition, experience requirements have been put in place for the contractor's personnel to ensure that persons in charge and conducting the work have previous experience with similar operations.

5 Summary

The post-tensioning industry with respect to grouted tendons in North America had virtually remained unchanged since its introduction. The failure of grouted tendons in one US bridge and the problems reported with other grouted structures has quickly changed the previous lack of attention to grout materials and grouting operations. Much needed guideline grout specifications and formalized training programs have recently been introduced in North America to improve the quality of grouted post-tensioned concrete construction.

6 References

Bastien, J., Dugat, J. and Prat, E. (1997), "Cement Grout Containing Precipitated Silica and Superplasticizers for Post-tensioning", *ACI Materials Journal*, Vol. 94, No. 4, July-August.

Freyermuth, C. (1991), "Durability of Post-tensioned Prestressed Concrete Structures", *Concrete International*, October, pp.58-65.

Ghorbanpoor, A., and Madathanapalli, S.C. (1993), "Performance of Grouts for Post-tensioned Bridge Structures", Research Report FHWA-RD-92-095, Federal Highway Administration, December, 46 pp.

Hamilton III, H.R., Breen, J.E., and Frank, K.H. (1998a), "Bridge Stay Cable Corrosion Protection. I: Grout Injection and Load Testing", *Journal of Bridge Engineering*, ASCE, Vol. 3, No.2, May, pp. 64-71.

Hamilton III, H.R., Breen, J.E., and Frank, K.H. (1998b), "Bridge Stay Cable Corrosion Protection. II: Accelerated Corrosion Tests", *Journal of Bridge Engineering*, ASCE, Vol. 3, No. 2, May, pp. 72-81.

Hope, B., and Ip, A. (1988), "Grout for Post-tensioning Ducts", *ACI Materials Journal*, Vol. 85, No. 4, July-August.

Ip, A., and Manning, D. (1993), "A New Generation Grout for Post-tensioned Bridges", *Third Canadian Symposium on Cement and Concrete*, Ottawa, Canada.

Lankard, D.R., Thompson, N., Sprinkel, M., and Virmani, Y. (1993), "Grouts for Bonded Post-tensioned Concrete Construction: Protecting Prestressing Steel from Corrosion," *ACI Materials Journal*, September-October, pp.406-414.

Perenchio, W., Fraczek, J., Pfeifer, D. (1989), "Corrosion Protection of Prestressing Systems in Concrete Bridges", NCHRP Report 313, TRB, National Research Council, Washington, D.C., 1989.

Poston, R., Carrasquillo, R. and J. Breen (1987), "Durability of Post-tensioned Bridge Decks", *ACI Materials Journal*, Vol. 84, No. 4, July-August, pp. 315-326.

PTI. (2001), Guide Specification of Grouting of Post-tensioned Structures, Post-Tensioning Institute, AZ, 69 pp.

Schokker, A. J., Koester, B. D., Breen, J. E. and Kreger, M. E. (1999a), "Development of High Performance Grouts for Bonded Post-Tensioned Structures," Research Report 1405-2, Center for Transportation Research, The University of Texas at Austin, October, 56 pp.

Schokker, A.J., West, J.S., Breen, J.E., and Kreger, M.E. (1999b), "Interim Conclusions, Recommendations, and Design Guidelines for Durability of Post-Tensioned Bridge Substructures," Research Report 1405-5, Center for Transportation Research, The University of Texas at Austin, October, 38 pp.

Sprinkel, M. (1997), "High Performance Silica Fume Grout for Post-tensioning Ducts", Proc., *PCI/FHWA International Symposium on High Performance Concrete*, New Orleans, La., October.

West, J. S., Larosche, C. J., Koester, B. D., Breen, J. E., and Kreger, M. E. (1999a), "State-of-the-Art Report about Durability of Post-Tensioned Bridge Substructures," Research Report 1405-1, Center for Transportation Research, The University of Texas at Austin, October, 172 pp.

West, J.S., Schokker, A.J., Larosche, C.J., Breen, J.E., and Kreger, M.E. (1999b), "Long-Term Post-Tensioned Beam and Column Exposure Test Specimens: Experimental Program", Research Report 1405-3, Center for Transportation Research, The University of Texas at Austin, October, 264 pp.

West, J.S., Vignos, R.P., Breen, J.E., and Kreger, M.E. (1999c), "Corrosion Protection for Bonded Internal Tendons in Precast Segmental Construction," Research Report 1405-4, Center for Transportation Research, The University of Texas at Austin, October, 78 pp.

Strategies for the improvement of unbonded post-tensioning systems in North America

H.C. WALKER

CWConsultants, I.LC, 2429 Highpointe Drive, Kalamazoo, Michigan 49008 USA

1 Introduction

The authors experience with unbonded post-tensioning systems has been largely with the design and construction of over twelve hundred long span parking structures over 35 years in the United States and Canada. Thus referencing North America in this paper means experience in the United States and Canada, but not Mexico.

Unbonded post-tensioning systems have been used in concrete building structures in North America since the early 1960's. The earliest unbonded systems were derivations of the Swiss BBR button headed systems with the wire tendons greased and wrapped with sisal kraft paper. These were marketed by competing firms, Ryerson and Prescon in the USA and Conenco in Canada. Then came a single strand system (Atlas) that was also greased and paper wrapped with a coil spring wire and plate anchorage with hardened steel grippers or wedges.

In the early 1960's, unbonded post-tensioned structures were designed with the unbonded post-tensioning tendons as the only flexural reinforcing with little or no back-up mild steel reinforcing. However, after the Anchorage, Alaska earthquake in 1964, where there were some serious failures of the Atlas system, the American Concrete Institute (ACI) Building Code (ACI 318) was revised to require backup mild steel reinforcing for unbonded post-tensioned structures. The back up reinforcing was designed to carry an ultimate load of the dead load of the structure plus 25 percent of the design live load to prevent collapse if the unbonded post-tensioning system failed. Since that code change, to my knowledge, there have been no catastrophic collapses of unbonded post-tensioned structures.

In the United States and Canada, unbonded post-tensioning is used primarily in flat slab and flat plate concrete structures for high-rise apartments, condominiums, hotels, and other residential structures; office building floors; and long span parking structures. (Unbonded post-tensioning is frequently used to prestress concrete slabs-on-grade for single-family residences where poor or expansive soil conditions exist. However, prestressed concrete slabs-on-grade are a separate and distinct challenge that is outside the scope of this paper.) While residential and office structures are enclosed and protected from the elements, tendon failures have occurred after the structure has been put into service. Parking structures, particularly in the northern United States and in Canada have had numerous unbonded tendon failures primarily related to corrosion caused by de-icing salts.

2 The evolution of unbonded PT systems in North America

2.1 End anchorages

In the 1960's and early 1970's, the unbonded BBRV button-headed system was sold as having 'positive end anchorages'. This was especially true after the Atlas system failures during the 1964 Anchorage earthquake. Gradually, single strand steel castings were developed. These castings, with approximately 2-1/2 inches x 5 inches (60 mm. X 120mm) of bearing area, were machined with a conical hole to receive a single 1/2 inch or 0.60 inch seven-wire strand which would be anchored by two tapered hardened steel grippers or wedge chucks. During the stressing operation in the field, once the strand was fully stressed, the wedges were tapped into place with a seating tool with final seating occurring with the release of the prestressing strand from the hydraulic jack. With time, the various manufacturers perfected the wedge anchorages and confidence grew that the wedges would provide a safe and reliable anchorage of the strand. The BBRV systems phased out by the end of the 1970's.

2.2 Greases

The grease has a two-fold purpose---first, as a lubricant to reduce friction losses during the stressing operation, and second, as a corrosion inhibitor. Lithium based greases have been used for this purpose for a number of years. The Post-Tensioning Institute has published a detailed specification for unbonded tendon greases based on whether or not the tendon would be exposed to corrosive conditions such as de-icing salts or sea water. Of course, the lubricating factors of the grease were important only during the time of stressing the tendon. However, the problems with the early greases were that they would dry-out or oxidize with time and lose their corrosion inhibiting capabilities. Modern greases have excellent corrosion resisting qualities.

2.3 Wrapping, sleeves, ducts, sheathing

Early unbonded tendons were spiral wrapped with sisal kraft paper to prevent the tendon from bonding to concrete during concrete placing. Torn paper wrapping was easy to patch with paper or duct tape. However, the paper wrapping had no waterproofing qualities and thus provided no resistance to chloride attack of the steel tendons. Also, most of the wrapping was done by hand and it was a messy job.

Two sheathing systems evolved. One was using a plastic ribbon as a spiral wrapping. This gave way to the "cigarette wrap" which was a plastic ribbon longitudinally wrapped around the strand and heat-sealed. The problem with this system was in cold weather, bending of the tendon would often rupture the overlapping seal and provide an opportunity for water and salt-water to have direct access to the strand.

The other system was the "push through" system where an extruded plastic tubing had the strand and grease installed into the tube simultaneously. The tube had to be slightly larger than the strand, and if the grease did not completely fill the void between the strand and the tube wall, air voids would occur. Often these voids would collect water during manufacturing, shipping, and construction. There were a number of serious tendon failures with this system.

The state-of-the-art sheathing system evolved to a 40-mil polyethylene plastic coating extruded onto a grease strand. This eliminated the possibility of air void inclusions and delamination of the sheathing seam. Engineers, however, became concerned about the lack of toughness in the field of the 40-mil sheathing. It easily abraded and was also easily cut by the

tie wires. Recently, the minimum extruded sheathing thickness was increased to 50 mils with a marked decrease in sheathing damage.

2.4 Strand

Typically, the strand used in the modern unbonded tendon is $\frac{1}{2}$ inch or 0.6 inch 270K stress relieved low relaxation seven-wire strand conforming to ASTM A-416. There have been no major changes in strand characteristics since the development of low relaxation strand.

2.5 Engineering and construction details

It was found that the engineering and construction details had much to do with the longevity of unbonded post-tensioning systems. At construction joints where tendons were coupled, early on, it was felt that the prestressing forces would keep the joint tight and leak proof. Also, not much effort was given to keep water out of the anchorage zones during construction. Often, tendon failures pointed to these areas. Sealing of construction joints with a high quality sealant, and taking efforts to protect the anchorage zone from water intrusion during construction reduced these problems.

3 Failures of unbonded post-tensioning tendons.

Failures of unbonded post-tensioning tendons occur due to many reasons, including the following:

- Strand failure due to salt water intrusion,
- Strand failure due to non-salt water intrusion,
- Strand failure due to hydrogen embrittlement,
- Strand failure due to uneven seating of anchor wedges,
- Strand failure due to misalignment of the anchorage,
- Strand slippage due improper seating of the anchor wedges,
- Anchorage failure due to low strength or improper consolidation of the concrete at the anchorage,
- Corrosion of the anchorages or wedges,
- Improper heat-treating of anchorage components,
- Improper swedging or seating of dead-end anchorages,
- Concrete distress due to misalignment or reversal of the tendon profile.
-

Many of these tendon failure modes are due to poor quality control during the manufacturing or field processes. However, it became apparent that most of the serious failures were caused by water or salt-water contact with the strand or anchorages.

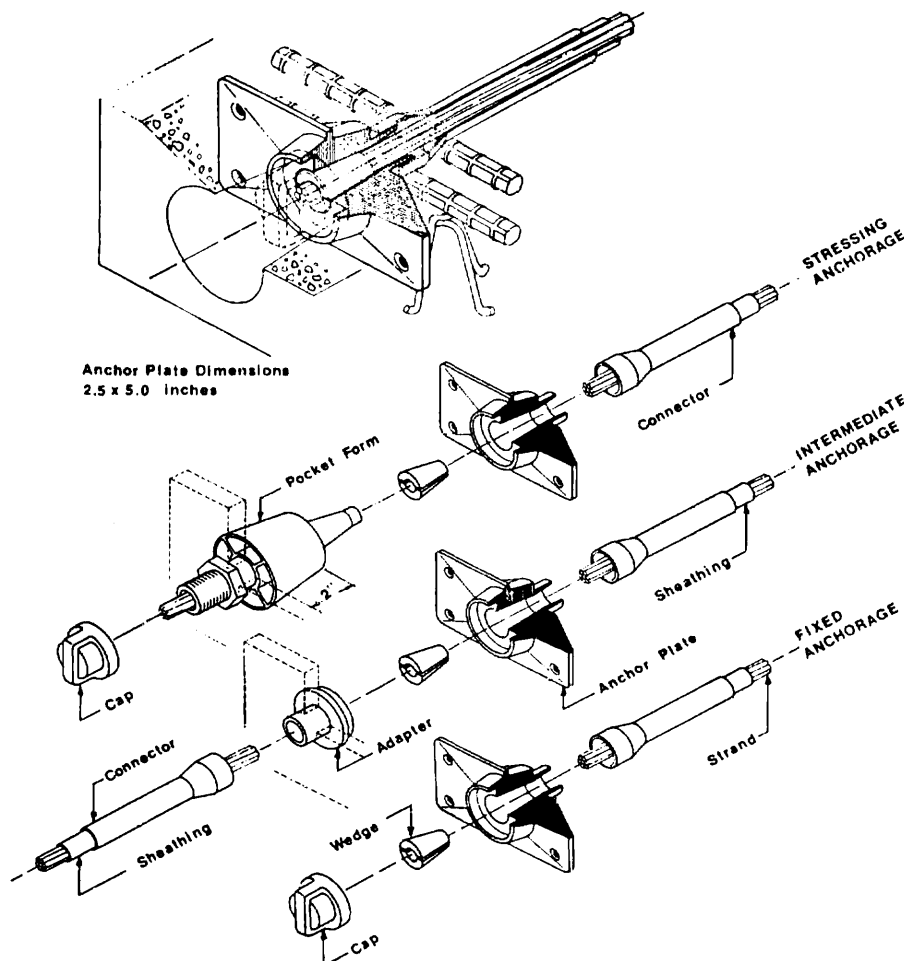
4 Electrically isolated tendon

Corrosion of steel is an electro-chemical process where water and oxygen are necessary. Chlorides serve as a catalyst to accelerate the process. Thus, the electrically isolated tendon was developed stop corrosion of tendons in a corrosive environment. This tendon uses a strand with extruded sheathing, and anchorage with a positively attached grease cap to seal the end of the anchorage, and a waterproof connection trumpet to prevent intrusion of water

between the anchorage and the sheathed strand. Also, the anchorage is coated with epoxy or plastic to insulate it from the concrete. With a properly assembled system, the steel within the tendon is sealed and protected from water or salt-water intrusion. The electrical isolation prevents the flow of the electrical currents that cause corrosion. See Sketch 1.

Sketch 1

MONO-STRAND TENDON FOR AGGRESSIVE ENVIRONMENT



5 Encapsulated tendon

Because of patents on the Electrically Isolated Tendons, a number of tendon manufacturers use the encapsulated tendon, which is similar to the electrically isolated tendon, except that the anchorage casting is not coated to insulate it from the concrete. The argument is that the anchorage casting is of greater mass than the wires in the strand and, thus, the possibility of corrosion to failure is not as likely as with strand.

6 The trumpet problem

In theory, the electrically isolated tendon and the encapsulated tendon address the cause of most unbonded tendon failures in North America. However, after installation of these systems on a number of projects, the question arose of adequacy of air-void elimination in the trumpet region. On one of our projects, a trumpet was 'dissected' and large air voids and very little grease were found. Therefore, we began specifying that the trumpets be manufactured of a translucent plastic so the extent of grease and air voids in the trumpet could be visually determined in the field. When air voids were found, a grease gun with an injection needle similar to a football air needle was inserted through the trumpet gasket to add more grease.

7 Total Durability

The unbonded post-tensioning tendon cannot provide all of the durability of an unbonded post-tensioned slab by itself. In addition to a properly manufactured and installed encapsulated or electrically isolated tendon system, durability includes:

- On parking structures, minimum constructed slopes of one percent, with two percent being desirable, to drain salt laden water off the floor slab surface.
- On parking structures, provide a minimum of 2 inches (50mm) of concrete cover over the post-tensioning tendons and the reinforcing steel.
- On parking structures, use epoxy coated mild steel reinforcing since it is secondary reinforcement.
- On parking structures, use non-corroding accessories including chairs, bolsters, inserts, and tie wires.
- Concrete should have a water/cementitious ratio of 0.40 or less to produce impermeable concrete.
- On parking structures, consider the use of corrosion inhibiting admixtures, or silane sealers as additional defense mechanisms.
- Intensive inspection and quality control procedures.

8 The future of unbonded post-tensioning in North America

Cast-in-place unbonded post-tensioned concrete construction is labor intensive. However, when compared to the potential of field problems with bonded (grouted) post-tensioned system, I believe the unbonded post-tensioned concrete will be the post-tensioning system of choice for building construction for the foreseeable future in North America. The questions of proper grease protection and the elimination of air voids have been addressed and positively answered. The ability to positive assure a correctly grouted bonded project has not been resolved in North America. Grouting is also much more cold weather sensitive than an unbonded system.

North America has a problem with the quality of its labor force. New apprentice tradesmen lean towards more sophisticated, cleaner, and less muscular endeavors. Construction labor forces come more and more from the bottom of the barrel. Thus, enhanced

field quality control and inspection are going to be necessary to assure that the constructed quality is on the same level as the engineered quality.

Non-steel, and therefore, non-corrosive, post-tensioning tendons made with glass fibers and carbon fiber are being actively researched in North America. The major problem with these products are their low fire-resistance threshold. However, with proper fire protective measures, these fibers may become the prestressing tendon of the future.

Strategies for improvement

- Approach in Japan -

Keiichi Aoki

Registered engineer, Bridge and Structural Engineering Division,
Japan Highway Public corporation, JAPAN

Masaaki Umitsu

Deputy manager, Engineering Division,
Oriental Construction Co.LTD, JAPAN

Yukio Igo

Assistant manager, Technical Department,
Japan Prestressed Concrete Contractors Association, JAPAN

1 Introduction

1.1 Present operation of the Japan Highway Public Corporation

The expressways constructed by the Japan Highway Public Corporation (JH) were initially opened to traffic in 1953 and reached a length of 6854 km in July 2001, about 60% of the officially planned length. The length of bridges is about 900 km, accounting for 14% of the expressway length. Bridges consist of reinforced concrete, prestressed concrete (PC) and steel bridges with a relative ratio of 3:3:4. The relative ratio of the bridges constructed in or after 1996 is 0.3:6:4 owing to the adoption of the partially prestressed concrete (PPC) structure.

The Daini-Tomei/Meishin expressway under construction will have a total length of about 490 km. Gentle curvature and slopes are used for the horizontal and vertical alignments and the road width has been set relatively large to enable inter-city driving at a speed of 140 km/hr. Structures therefore accounts for a large percentage on the expressway. The percentage of bridge length in the total length is about 40%, larger than on the existing expressways in service. Reducing the bridge project cost is the prime imperative because it would greatly contribute to the reduction of total project cost of the Daini-Tomei/Meishin expressway.

In addition, the cost for maintaining the existing expressways in service amounts to about 80 billion yen per year and is expected to increase further. The maintenance cost will substantially exceed the present level once the Daini-Tomei/Meishin expressway has been placed in service. Reducing the initial cost of bridges and making structures easier to maintain are therefore great challenges.

1.2 Concrete crisis and problems with PC bridges

Concrete structures were long considered maintenance-free. In recent years, however, tunnel lining concrete or concrete cover on bridges spalled in successive events due to salt damage or poor construction. The accidents have been widely reported as a social issue.

Damage or deterioration has also been discovered on PC bridges due to the lower technical level available at the time of construction or poor construction. Some cases have been reported that decreased the structural reliability. Most of the damage was caused by salt. The fracture of prestressing steel is ascribable mostly to incomplete grouting. Thus, PC bridges have been subjected to intense scrutiny.

1.3 Approach to improvement in grouting systems in Japan

Under the above circumstances, several improvements were made to the grouting system to ensure complete injection of grout.

As an improvement in materials, a complete shift was made to non-bleeding grout in April 1997. Accordingly, grouting systems were totally improved. Specifically, ventilation vents for entrapped air were installed at additional points, the percentage of voids in the sheathing was increased, and the diameters of injection and ventilation hoses were increased.

Later, flow meters were adopted in April 1999 for recording the process and amount of grout and ensuring management.

At the same time, stricter management of grouting work was sought by making the training of grouting supervisors mandatory and establishing a uniform grouting management system.

For improving the durability of PC bridges under heavy corrosion environments in near coastal areas and places where de-icing chemicals are used, polyethylene sheathing was developed and has been used in an increasing number of cases. It has been applied not only for controlling salt damage but also for enhancing the reliability of grouting.

1.4 JH's approach to new PC technology for higher durability

Greater emphasis is expected to be placed on maintenance among the jobs done by JH. It is, however, still very difficult to inspect the soundness of the inside of concrete bridges, especially of prestressing steel.

JH is therefore trying to apply the pretensioning method in an increasing number of cases that offers the most reliable anti-corrosive treatment, and is actively adopting a structure with external cables because it makes inspection easy. JH is also increasing the application of pre-grouted prestressing steel in longitudinal direction that was developed in Japan. Thus, JH has been working to construct highly durable PC bridges.

The structures with external cables are effective because they facilitate the inspection and replacement of prestressing steel and because the reduction of weight of main girders and the

use of high-capacity cables provide economy. So this type of structure has been used for box girder bridges since 1999.

In the structure with external cables, the corrosion of prestressing steel used to be prevented mainly by using polyethylene piping and grouting. Verification of grout injection was not easy and no inspection could directly verify the damage to prestressing steel, so JH developed transparent (polyvinyl chloride resin) sheathing and has been preventing corrosion by injecting grout into the sheathing as a standard practice. Other anti-corrosion methods include the use of epoxy coated prestressing steel, which has been adopted in cases where deterioration is expected during construction like in areas vulnerable to salt damage or the grout weight has a great impact on design because of the long span.

Pretensioning has long been applied to the products manufactured at the plant and its anti-corrosion capacity for steel has historically been proved. JH is considering its application to numerous PC structures, and has been applying it to the transverse prestressing of deck slabs in the precast segment construction method that has recently been used in an increasing number of cases. With the increase in the span length of deck slabs, steel with a larger capacity has been used that has a diameter of 21.8 mm with a pretensioning force of 445 kN, larger than 15.2-mm-diameter conventional steel with a pretensioning force of 200 kN.

The pre-grouted prestressing steel is manufactured at the plant by coating prestressing steel with polyethylene sheathing and injecting cold-setting resin (epoxy) into the gap between steel and the sheathing. It was developed in Japan in 1987 and has been used mainly for transverse prestressing of deck slabs since around 1992.

JH has been working on expanded application of pre-grouted prestressing steel for eliminating errors and saving manpower in construction.

JH basically prohibited the use of prestressing bars that had been used mainly for transverse prestressing of deck slabs and shear reinforcement, on ground of incomplete injection of grout. Since then, PC box girder bridges with corrugated steel webs have been actively introduced. Light-weight steel webs with high shear buckling strength replaced concrete webs that account for 20 to 30% of the self weight of a PC box girder bridge. A total of more than 30 box girder bridges of the new type will be constructed including those under planning. The structure provides such benefits as the reduction of self weight, efficient prestressing in the longitudinal direction and improvement of construction process. Long spans, however, entail numerous problems. Especially, no PC box girder bridges with corrugated webs using full external cables (no internal cables) have ever been constructed in the world. JH has been conducting various studies and tests to solve problems. It is expected that this type of box girder bridge will be used widely as the mainstream PC bridge.

Approaches to durability improvement of PC bridges in Japan

Year	Grouting improvement measures in Japan	JH's approach to new PC technology			
		Full external cabl structure (No internal cables)	Wider use of pretensioning	Pre-grouted prestressing steel	PC bridges with corrugated steel plate webs
1987				Developed Pre-grouted prestressing steel	
1992				Initially adopted for transverse prestressing of deck slabs	
1993					Japan's first PC bridge with corrugated webs: Shinkai Bridge*1 (erection girder method)
1995					Ginzan-miyuki Bridge*2 (erection by launching method)
1997	Implemented grouting management sysytem Shifted to non-bleeding circuit		Initially adopted 1S15.2 strand for transverse prestressing of deck slabs by pretensioning Shigenobu viaduct		
1998		Initially adopted full external cabl structure for a highway bridge Kaihei viaduct (all staging method)			Hondani Bridge (by balanced cantilever method)
1999	Adopted flow meters			Initially adopted for the main cabl Jyugosawa Bridge and Komagata viaduct	
2000		Kunigawa-Daisan Bridge (by balanced cantilever method)			
2001		Oidaira viaduct under construction (by launching method)	Initially adopted 1s21.8 strand for transverse prestressing of deck slab by pretensioning Nagashima viaduct under construction		

Notes

- 1.The project was implemented by the Niigata prefectural government
- 2.The project was implemented by the Akita prefectural government

2 Japan's approach to improvement in grouting systems

2.1 Measures to improve the completeness of grouting

With the revelation of fractures of prestressing steel in Japan, the completeness of filling of sheathing by grouting was examined on existing bridges by non-destructive inspection, and the actual conditions of incompleteness of grouting were realized. The actual conditions of bridges were surveyed and grout injection tests were conducted. As a result, incomplete grouting was attributed to such causes as forgetting to inject grout, bleeding water in the sheathing, downward grout flow on downgrades and plugging of the sheathing. JH has been removing as many causes as possible and continuously making improvements to ensure complete grouting.

2.1.1 Use of transparent and color grout hoses

Forgetting to inject grout is a simple human error. The error cannot be avoided mainly because the non-transparent grout hoses conventionally used make it difficult to check whether grouting has been carried out or not. In Japan, transparent grout hoses are now used to eliminate the cause and enable checking of grouting work. In recent construction of PC box girder bridges, transparent grout hoses that were colored according to the type of prestressing steel or the area of use have been used for easy identification. On bridges with numerous cables crossing one another at interior supports and other points, numbered seals are pasted on grouting hoses to avoid confusion.

2.1.2 Non-bleeding grout

For avoiding the creation of voids in the sheathing due to bleeding water in the grout or downward grout flow on downgrades, non-bleeding grout was developed and a total shift was made to the grout in April 1997.

2.1.2.1 Characteristics

The non-bleeding grout has viscosity agent added to it and generates no bleeding water. Thus the possibility of bleeding water generating voids can be eliminated, and the completeness of grouting can be increased. The grout is also so cohesive that it can be injected while filling the whole cross section of the duct on a downgrade and causes no air to be entrapped. Higher cohesiveness, however, was expected to require higher grout pressure and longer injection time. For preventing excessive grout pressure and ensuring the designated injection efficiency, therefore, the diameters of the standard sheathing were

increased for prestressing bars with a small duct void ratio. For example, the sheathing diameter for a 32-mm-diameter steel was increased from 38 mm to 45 mm. For the same reason, the standard diameter of grout hose was increased from about 15 mm to 19 mm.

2.1.2.2 Materials, mix proportion and mixing method

Ordinary Portland cement is generally used for grouting. Water is supplied from the municipal water supply system, or from rivers or groundwater as long as their quality meets the designated standards. Admixtures are available with various proportions of water reducers, viscosity agents, expanding admixtures and retarders, and the ones with the optimum proportion are selected for manufacturing grout.

Mix proportions of grout are determined so as to meet the quality requirements according to the type of prestressing steel, location of sheathing, temperature, and curing and other construction conditions. A water-cement ratio of 45% is generally used. For achieving the desired viscosity, the water-cement ratio is determined based on the temperature of mixed grout estimated from the ambient temperature at the time of grouting.

The sequence of throwing materials in a mixer and mixing time slightly vary according to the mix proportion and the admixtures used. Materials are usually mixed in a mixer with a mixing cycle of 1000 rpm or higher for about three minutes after all materials are thrown in.

2.1.2.3 Quality control test

The following three tests are generally carried out to control the quality of grouting.

1) Fluidity test

Test procedure: Flow values are measured using a cone.

2) Compressive strength test

Test procedure: Cylinder specimens are used for the compressive strength test.

Specified value: 28-day compressive strength of 20 N/mm² or greater

3) Measurement of chloride content

Test procedure: The chloride content of the sample with the water-cement ratio identical to that of grout mix is measured and the chloride contents of admixtures are added to it. Or chloride content is calculated from the list of qualities of all the materials used. (No bleeding water occurs, so no simple measurements by Cantabro tests or detector tubes are possible.)

Specified value: Total chloride in grout should not exceed 0.3 kg/m³.

2.1.3 Grouting equipment

Rotary vane grout mixers with a mixing cycle of 1000 rpm or higher are generally used.

Various types of grout pumps are available. Electric squeeze pumps are used for longitudinal cables. Whenever injection trouble occurs, vacuum grouting pumps are used. At present, flow meters are used to check the amount of grout injected, injection time and injection pressure. Chart type flow meters, whose use was made mandatory in April 1999, print start and end time of grouting and the amount of grout injected, and record flow rate and pressure in real time. Checking these data during grout injection enables early detection of trouble or malfunction.

2.1.4 Improvement of grouting systems based on grouting tests

An injection test of non-bleeding grout was conducted in 1998 to confirm downward grout flow and entrapped air for different grout temperatures and types of admixtures. Then, in 1999, a grout injection test was carried out while varying the angles of cables and grout viscosity, and, based on the entrapment of air, improvements were made to the locations of grout injection and ventilation vents, and the injection method. Fig. 2.1 shows a recommended grouting program.

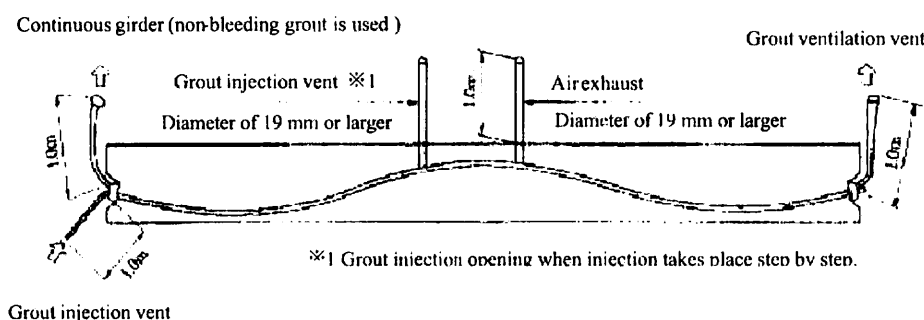


Figure 2.1 Example of a scheme of grouting

2.1.5 Grouting management system

For ensuring grouting using improved grout by improved grouting methods, training on grouting has been conducted in Japan. Instructions are being made to have supervisors who received training present at the site of grouting for management of work. A checklist has been prepared that covers considerations for grouting during bridge construction, inspection items during grouting and matters to be confirmed after work completion, for greater accuracy of management by grouting supervisors.

2.2 Other measures

2.2.1 Nonferrous sheathing (polyethylene sheathing)

For increasing the durability of PC bridges under corrosion-prone environments in near

coastal areas or places where de-icing chemicals are used, nonferrous rustless sheathing of polyethylene or other macromolecular materials are being adopted in an increasing number of cases. Polyethylene sheath couplers were also proposed in Japan for precast segment joints a few years ago, and attempts have been made to improve the durability of segmental bridges in areas requiring salt damage control measures. The performance requirements of polyethylene sheathing are listed below.

- 1) No crashing, cracking, breakage or grout leakage should occur during construction.
- 2) Sufficient bond with concrete should be provided.
- 3) Sufficient abrasion resistance should be offered when prestressing steel is inserted or Strategies for improvement stressed.
- 4) The coefficient of friction with the tendon should not exceed that of existing products.
- 5) Performance for construction should be secured during assembly.

2.2.2 Liquid analysis of grout

Japanese universities have been making liquid analysis to identify the behavior of grout in the sheathing during injection. The studies have been aiming at understanding the conditions for full-face grout flow that means grout flowing while filling the whole cross section of the sheathing, and the process of void creation and void size. Tests and analyses have been carried out using the rheological property of grout, sheathing diameter, gradient and amount of grout injected as parameters. It is hoped that the grout behavior in the sheathing in actual structures may be quantitatively grasped by numerical analysis through the collection of data by examining more injection conditions and using more parameters.

3 JH's approach to the increase of durability

3.1 Full external cables (no internal cables used)

Inspection of internal cables is very difficult in view of the present technical level. JH therefore has actively adopting full external cables for box girder bridges since 1999 because they facilitate inspection.

External cables contribute to greater economy and higher performance of construction work because they place no restrictions on the thickness of members for arranging prestressing steel and thus enable the reduction of weight of main girders and because they enable the use of high-capacity cables. They also facilitate inspection, maintenance and replacement of prestressing steel.

Despite the above benefits, external cables also involves the following problems.

- 1) Corrosion of prestressing steel owing to the placement of cables outside the girders
- 2) Evaluation of increased tension in cables under ultimate load
- 3) Safety of anchorage zone when large-capacity external cables are used
- 4) Application of external cables to balanced cantilever methods

3.1.1 Corrosion protection of external cables

Corrosion of external cables used to be prevented mainly by using polyethylene piping and injecting grout. Verification of completeness of grout injection was, however, difficult and inspection for possible damage to prestressing steel was also difficult. Then, JH developed transparent (polyvinyl chloride resin) sheathing to verify grout injection more easily. Fig. 3.1 and 3.2 shows a transparent sheath and grout injection under way.

For the epoxy-coated strand method, another corrosion control method widely used in Japan, verification tests for crashing at deviators and for brittleness in cold areas provided favorable results.

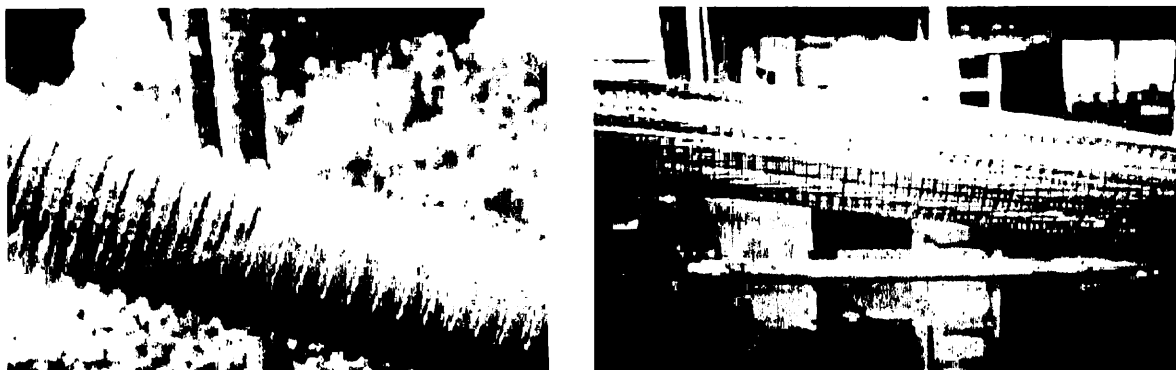


Figure 3.1 transparent sheath and grout injection under way

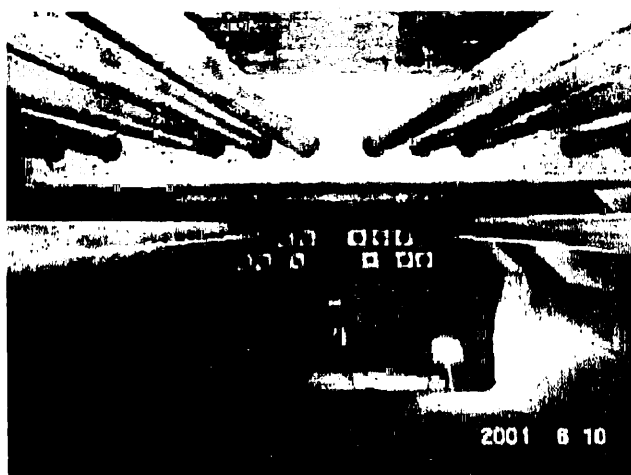


Figure 3.2 transparent sheath and grout injection under way

3.1.2 Method for evaluating increased tension in external cables under ultimate load

External cables experience a decrease in eccentricity of cables due to the bending deflection of main girders, and the ultimate strength of the girder decreases. Navier's hypothesis does not hold true with external cables, so they cannot be treated like internal cables. Calculation of ultimate strength therefore requires proper evaluation of increases of tension in external cables. Conventional design methods assumed an increase in tension of 100 to 200 N/mm². Only little knowledge was available about tension increases in varying girder depth or cables for cantilevering. JH therefore analyzed actual bridges to examine increases in tension in external cables under ultimate load. As a result of a nonlinear frame analysis using a cross section divided into fiber elements shown in *Fig. 3.3*, equation (1) was proposed to identify tension increase.

- Tension increase in cables for cantilevering
 - For $L/d_p > 50$: 0 N/mm²
 - For $L/d_p < 50$: $100d_p/L$ (200 N/mm² or less)
- Tension increase in continuous cables
 - For $L/d_p > 50$: 0 N/mm²
 - For $L/d_p < 50$: $400d_p/L$ (400 N/mm² or less)

where L is the distance between anchorages of prestressing steel, and d_p is the effective depth of an external cable.

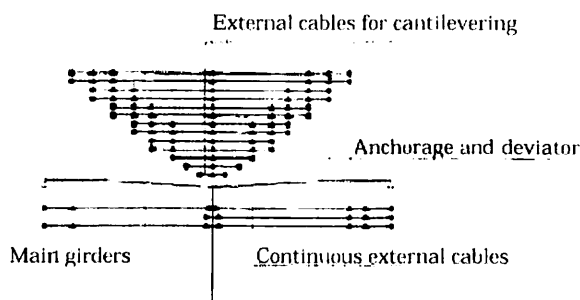


Figure 3.3 Nonlinear frame analysis model

3.1.3 Safety of anchorage when using large-capacity external cables

Large-capacity cables are generally used for external cables, so large tensioning force is transferred at one point. The safety of large-capacity external cable anchorage was of concern because no such anchorage had been used. Then, JH made a finite element analysis of anchorage and conducted verification tests to evaluate safety (*Fig. 3.4*).

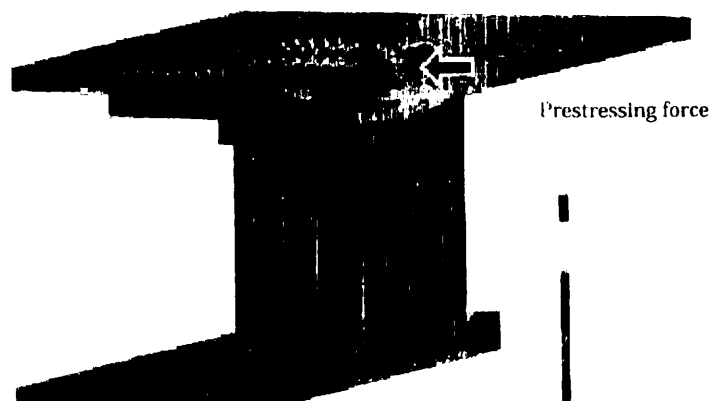


Figure 3.4 Nonlinear frame analysis model

3.1.4 Study of bridge with cantilever erection using full external cables

No bridge had been erected using cantilever methods adopting full external cables. So, the adoption of a reliable erection method was desired. Reduction of anchorages was desired to reduce weight, so the erection method described below was proposed (*Fig. 3.5*).

- 1) The bridge is erected sequentially using temporary prestressing bars as by conventional cantilevering methods. The prestressing bars are replaced with external cables in three blocks at a time.
- 2) The prestressing steel bars replaced by external cables are moved to the leading block for reuse.
- 3) A double-barrel jack with a maximum capacity of 10 kN is used because the tension in the prestressing bars is released at other points than at the end of the girder.

The above method was verified by tests and was successfully applied to cantilevering erection using external cables.

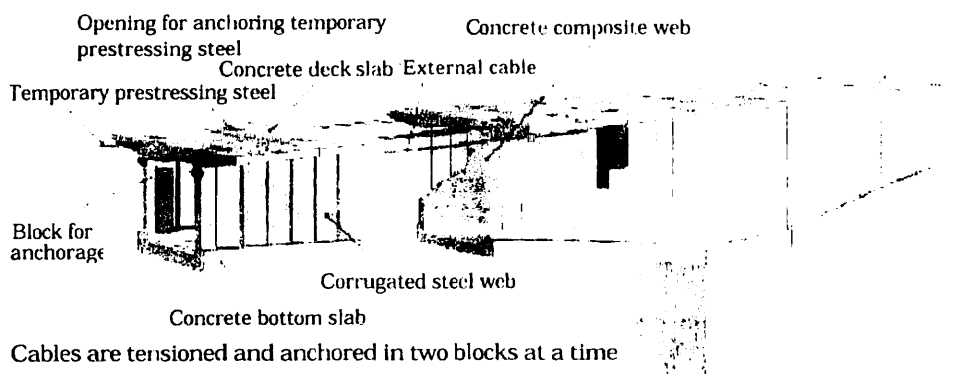


Figure 3.5 cantilever methods adopting full external cables

3.2 Transverse pretensioning

The benefits of transverse prestressing of deck slabs by pretensioning are listed below.

- No grouting at site is required, so incomplete grouting is not a problem.
- Completeness of concrete grouting is improved.
- Tensioning can be controlled more easily.

With the increase of demand for mechanization, manpower saving and quick construction, the precast segment construction method has been increasingly adopted. For the precast segment construction method by short-line match casting, concrete is placed at a fixed location, so the quality of curing equipment and concrete can be controlled better than in in-situ concrete casting.

For pretensioning, permanent abutments (permanent abutment method) or reaction beams are used (reaction beam method).

For the permanent abutment method, although it is effective when the pretensioning equipment is installed permanently at the plant, prestress is transferred in a relatively short time after concrete placement, and thus causes elastic deformation due to prestressing and subsequent creep and dry shrinkage to increase. The reaction beam method, on the other hand, enables prestressing after match casting, so it causes no variance in deformation in adjacent segments during manufacturing (*Fig. 3.6-3.9*).

Adoption of large-capacity pretensioning steel widens the range of application of pretensioning. The steel is expected to be used not only in the deck slab segment but also to various structures requiring greater durability of prestressing steel that have recently been adopted in an increasing number of cases such as precast PC deck slabs and prestressed reinforced concrete precast girders.

Comparison of prestressing strands used for pretensioning	
Prestressing strand of a diameter of 15.2 mm	Prestressing strand of a diameter of 21.8 mm
	
Two layers of sevenwire strands	Three layers of 10 wire strands
Surface area: 63480 mm ²	Surface area: 95380 mm ²
Tensioning force: 400 kN	Tensioning force: 445 kN
Tensioning force/surface area: 3.15 N/mm ²	Tensioning force/surface area: 4.72 N/mm ²

Figure 3.6 Pretensioning steel

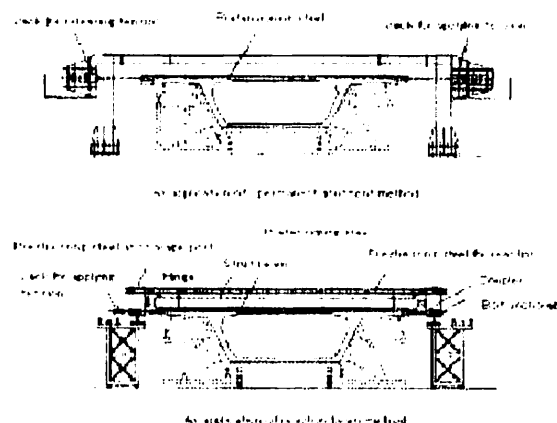


Figure 3.7 Pretensioning equipment

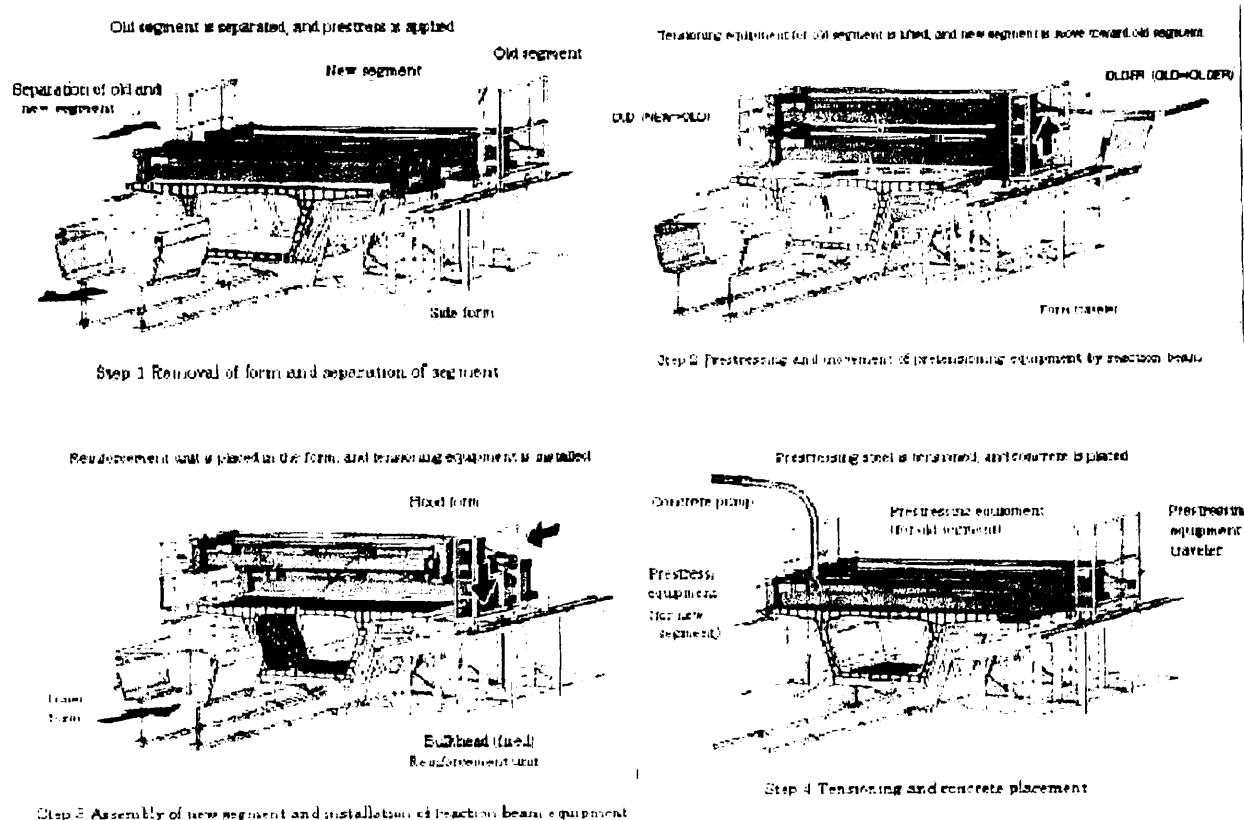


Figure 3.8 Construction by reaction beam method



Figure 3.9 Construction by reaction beam method (Nagashima viaduct)

3.3 Pre-grouted prestressing steel

Pre-grouted prestressing steel is made by coating a prestressing strand with unhardened epoxy resin and a polyethylene protective tube (Fig. 3.10). It is embedded direct into concrete with the polyethylene protective tube as a tendon for post-tensioning. The time of hardening is set for the epoxy resin that fills the polyethylene protective tube so as to provide enough time before hardening to allow post-tensioning. The resin has viscosity like grease before hardening, and it naturally hardens with time after the completion of tensioning of

prestressing steel and bonds with concrete to form an integrated object. Epoxy resin has the following characteristics.

- It is injected into the sheathing at the plant, so there are no cases of incomplete injection of grout.
- Construction work can be saved because neither in-situ insertion of steel nor grouting is required.
- Pre-grouted resin protects prestressing steel from corrosion.
- Smaller diameter of sheathing makes concrete placement easier.
- Polyethylene protective tube and unhardened resin reduce the coefficient of friction below that for conventional methods, and thus provide higher efficiency of prestressing.

Pre-grouted prestressing steel was generally applied only to transverse prestressing of deck slabs or other work. Application to main steel was started in view of the above benefits.

Pre-grouted prestressing steel is used only for single strands for economic reasons. When pre-grouted prestressing strands are used for main cable, more prestressing strands are required than when conventional multiple strands are used. Numerous prestressing strands therefore need to be anchored in a limited anchorage spacing. Then, attention should be paid in design to the arrangement of pre-grouted prestressing strands and details of anchorage. Management issues include the identification of the time of hardening of resin and of the coefficient of friction because the resin hardens at various times according to the temperature during construction and the maximum temperature of placed concrete and because the coefficient of friction during tensioning of prestressing strand varies depending on the degree of hardening of the resin (*Fig. 3.11 and 3.12*).

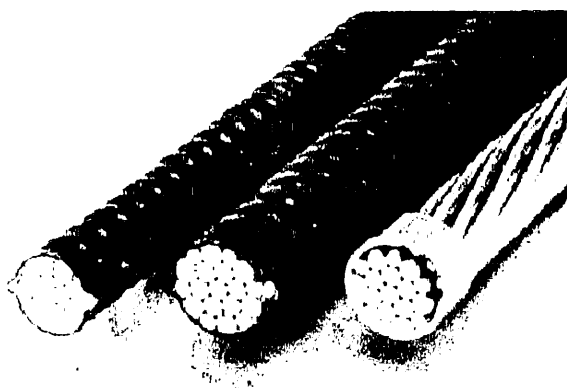


Figure 3.10 Pre-grouted prestressing steel

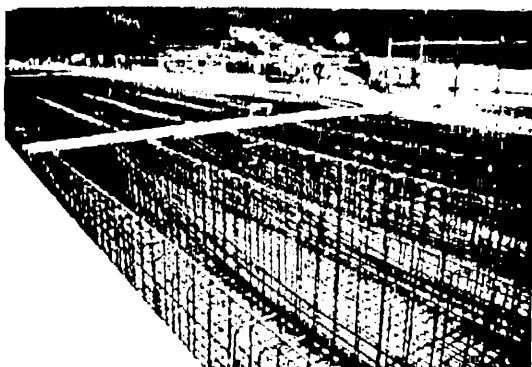


Figure 3.11 Use of pre-grouted prestressing strand for main cables

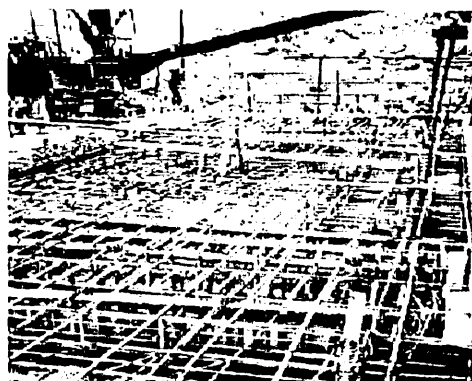


Figure 3.12 Use of pre-grouted prestressing strand for transverse prestressing of deck slabs

3.4 Corrugated steel web

PC bridges with corrugated steel webs use light-weight corrugated steel plates instead of concrete webs, whose weight accounts for 20 to 30% of the self weight of PC box girder bridges. The new structure uses the advantages of steel and concrete. It was developed in France and provides the following benefits.

- 1) The corrugated shape of steel plate produces a high shear buckling resistance, and the shape of the corrugated steel plate that does not take axial force enables efficient prestressing of top and bottom concrete deck slabs.
- 2) Use of light-weight corrugated steel plates for webs whose weight accounts for 20 to 30% of the self weight of main girders enables reduction of self weight of main girders. This then enables the use of longer spans and reduction of construction cost. The weight of a segment to be cantilevered during erection can also be reduced, so longer erection segments can be adopted and construction period can be shortened.
- 3) Use of no concrete webs eliminates assembly of reinforcement, cable arrangement and concrete placement for concrete webs. Thus, saving of construction manpower, quality enhancement and improvement of durability are expected. Replacing deck slabs expected to be damaged is easier than on present PC bridges, and durability is remarkably improved because steel plates are used for webs to facilitate repair.

4 Closing remark

As Japan's approach to the improvement of the durability of prestressed concrete bridges, an approach to the improvement of the existing prestressing concrete grouting system and JII's approach to new prestressing concrete technology were outlined.

In the future, studies will be carried out to seek ideal structures considering life-cycle cost and focusing on maintenance.

Durability of post-tensioning tendons

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