PERFORMANCE OF CONCRETE SEGMENTAL AND CABLE-STAYED BRIDGES IN EUROPE

U.S. Department of Transportation
Federal Highway Administration

International Technology Exchange Program

MAY 2001

NOTICE

The metric units reported are those used in common practice by the persons interviewed. They have not been converted to pure SI units because in some cases, the level of precision implied would have been changed.

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16. Abstract

The main objective of the scanning activity was to evaluate the European inventory of prestressed concrete segmental and cable-stayed bridges. On average, European structures are a decade or two older than similar ones in the United States. Members of the scan team examined durability; identified possible future needs for maintenance, repair and retrofit, and replacement; and compared trends and current practice.

The team visited four countries: Switzerland, Germany, Denmark, and France. Representatives from Norway and the United Kingdom also met with the team.

In general, segmental and cable-stay technology and developments in Europe and the United States are moving in parallel directions. Early performance problems from the 1960s and 1970s have been eliminated through new codes and practices on both sides of the Atlantic. Technical advances continue to be made with respect to corrosion, external and internal prestressing tendons, inspection methods, use of new composite materials, and construction techniques.

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Performance of Concrete Segmental and Cable-Stayed Bridges in Europe

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FHWA INTERNATIONAL TECHNOLOGY EXCHANGE PROGRAMS

The FHWA's international programs focus on meeting the growing demands of its partners at the Federal, State, and local levels for access to information on state-of-the-art technology and the best practices used worldwide. While the FHWA is considered a world leader in highway transportation, the domestic highway community is very interested in the advanced technologies being developed by other countries, as well as innovative organizational and financing techniques used by the FHWA's international counterparts.

INTERNATIONAL TECHNOLOGY SCANNING PROGRAM

The International Technology Scanning Program accesses and evaluates foreign technologies and innovations that could significantly benefit U.S. highway transportation systems. Access to foreign innovations is strengthened by U.S. participation in the technical committees of international highway organizations and through bilateral technical exchange agreements with selected nations. The program has undertaken cooperatives with the American Association of State Highway Transportation Officials and its Select Committee on International Activities, and the Transportation Research Board's National Highway Research Cooperative Program (Panel 20-36), the private sector, and academia.

Priority topic areas are jointly determined by the FHWA and its partners. Teams of specialists in the specific areas of expertise being investigated are formed and sent to countries where significant advances and innovations have been made in technology, management practices, organizational structure, program delivery, and financing. Teams usually include Federal and State highway officials, private sector and industry association representatives, as well as members of the academic community.

The FHWA has organized more than 40 of these reviews and disseminated results nationwide. Topics have encompassed pavements, bridge construction and maintenance, contracting, intermodal transport, organizational management, winter road maintenance, safety, intelligent transportation systems, planning, and policy. Findings are recommended for follow-up with further research and pilot or demonstration projects to verify adaptability to the United States. Information about the scan findings and results of pilot programs are then disseminated nationally to State and local highway transportation officials and the private sector for implementation.

This program has resulted in significant improvements and savings in road program technologies and practices throughout the United States, particularly in the areas of structures, pavements, safety, and winter road maintenance. Joint research and technology-sharing projects have also been launched with international counterparts, further conserving resources and advancing the state of the art.

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Executive Summary

European technology and experience with prestressed concrete segmental and cable-stayed bridges is one to two decades longer than in the United States, and the purpose of the Federal Highway Administration (FHWA) scan was to examine durability; identify possible future needs for maintenance, repair, retrofit or replacement; and compare trends and current practice. Cooperation was excellent, with a mutual exchange of ideas and information, transcending normal business practice. Early performance problems from the 1960s and 1970s have been eliminated through new codes and practices on both sides of the Atlantic.

All countries reported some corrosion of prestressing tendons or reinforcing steel, though in a minority of structures. Sources of corrosion were associated with poor-quality grouting of tendons or honeycombed concrete allowing water, oxygen, and chlorides to penetrate. European policy is to apply waterproof membranes, drainage, and protective overlays to bridge decks of all types of construction. The Europeans attribute the prevention or control of more serious damage from de-icing salts to this widespread policy. Less use of or alternative, less-aggressive de-icing chemicals are slowly being introduced. For new construction, more stringent controls and practices have been introduced on the grout installation operation to ensure that ducts are sealed and free of voids. Similar specifications are being introduced in the United States, for example, through the Post-Tensioning Institute (PTI), American Segmental Bridge Institute (ASBI), and the Florida Department of Transportation (DOT).

Both external and internal tendons have been and continue to be used in European segmental construction. More robust ducts of durable polyethylene are being introduced in Switzerland, and new greased-and-sheathed mono-strands are preferred in Germany, France, and Switzerland — particularly for repair or retrofit.

European cable-stay structures are performing well and, although there is a trend toward new, greased-and-sheathed mono-strands in high-density polyethylene pipes, all types of stays have been used and are allowed under new performance-based specifications currently being developed. New stay pipes are ribbed to eliminate wind-rain vibration effects. Wind-rain and other vibrations are suppressed by the installation of various types of damping systems during or after construction (e.g., hydraulic shock absorbers, cross-connectors, ties).

Inspection and condition assessments of existing structures are made in a similar manner to the U.S. practice, but there is greater commitment to implementing effective maintenance, repair, and up-keep in Europe. Structures have been replaced mostly for lack of adequate traffic flow (functional obsolescence), rather than for deterioration. Where appropriate, new fiber composite materials are used for repair and retrofit, which is similar to U.S. practice.

Available non-destructive evaluation techniques are the same as in the United States (e.g., gammagraphy, x-ray, ultrasonic, electrical resistance, magnetic perturbation, georadar, etc.), and their use requires engineering expertise and interpretation. Nowadays, instrumentation with remote monitoring is occasionally installed on major structures.

European contract procedures and public administration are different from that in the United States, but technical advances are similar. Many earlier issues from before the 1970s,[1] such as greater creep deflections of cantilevers with midspan hinges, have since been resolved on both sides of the Atlantic, through the introduction of new calculation techniques, codes, and practices. The visit confirmed current European practice, experience, and problems. Although the Europeans have not necessarily resolved all issues, they are proceeding in the same direction as current industry trends in the United States.

Although the scan was primarily intended for segmental and cable-stayed bridges, the information reported by the host countries was often more broad — covering reinforced and post-tensioned concrete structures in general and, occasionally, other types — particularly with respect to concrete materials, corrosion protection, bridge maintenance, bridge management systems, and non-destructive inspection methods.

In conclusion, segmental and cable-stay technology and developments in Europe and the United States are moving in parallel directions. The Europeans are satisfied with their structures. They continue to develop the technology and build segmental and cable-stayed bridges. With appropriate attention to improved grouting procedures, these structures will continue to serve well into the foreseeable future. The scan was an informative and worthwhile investment. It is recommended that similar scans and exchanges of ideas and developments be made on a regular basis, at least once per decade.

Glossary of Abbreviations

AASHTO ...... American Association of State Highway and Transportation Officials
ACP .... Asphalt Concrete Pavement
ASBI .... American Segmental Bridge Institute
ASCE .... American Society of Civil Engineers
BASt .... Bundesanstalt fur Strassenwesen (Institute for Highway Research, Germany)
BBRV .... Bureau of BBR Ltd.
BMS .... Bridge Management System (Maintenance)
CFRP .... Carbon Fiber Reinforced Plastic
CMA .... Calcium Magnesium Acetate (de-icing chemical)
COWI .... Danish consulting engineering and planning company (Chr. Ostenfeld & W. Jønson)
DIN .... Deutsche Norm (German Standard)
DOT .... Department of Transportation
DSI .... Dywidag Systems International
EMPA .... Eidgenossische Materialprufungs und Forchungsanstalt (Switzerland)
ETH .... Swiss Federal Institute of Technology, Zurich
EU .... European Union
FSU/FAMU .. Florida State University/Florida Agricultural and Mechanical University
FHWA .... Federal Highway Administration
GIS .... Geographic Information System
HDPE .... High-Density Polyethylene
LMC .... Latex Modified Concrete
NCHRP .... National Cooperative Highway Research Program
NDE .... Non-Destructive Evaluation
NDT .... Non-Destructive Testing
PC .... Prestressed Concrete
PE .... Polyethylene
PP .... Polypropylene
PT .... Post-Tensioning
PTI .... Post-Tensioning Institute (U.S.)
QA .... Quality Assurance
SETRA .... Service d'Etudes Techniques des Routes et Autoroutes (Roads and Motorways Department), France
SLS .... Serviceability Limit State
TRL .... Transport Research Laboratory (UK)
ULS .... Ultimate Limit State
UK .... United Kingdom
UT .... University of Texas (at Austin)
VPI .... Vapor-Phase Inhibitor

Part A: Overview

BACKGROUND

Contemporary prestressed concrete segmental and cable-stayed bridges began being built in Europe in the 1950s. The technology was later imported to the United States. On average, European structures are a decade or two older than similar ones in the United States. The purpose of the scan was to evaluate and learn from European experience with bridges of long service life, in order to identify durability and needs for maintenance, repair, retrofit, or replacement. The scan team visited four countries: Switzerland, Germany, Denmark, and France. Representatives from Norway and the United Kingdom met with the team in Denmark and France, respectively. The team also shared current U.S. practice, experience, and research with European counterparts. The goal was also to develop and promote mutual understanding of the technology, maintenance needs, and durability of these structures.

OBJECTIVES

The main purpose of the scanning activity was to evaluate the European inventory of these bridge structures, which have a longer in-service history than similar ones in the United States. Researchers can then determine what problems have surfaced with respect to accessibility for inspection, maintenance, repair and retrofit, long-term durability, demolition, and replaceability. The scanning team set out to determine what solutions have been used and which ones were effective in resolving problems that have arisen. This research will indicate what problems may be anticipated and what technologies might be imported into the United States to solve similar problems that have or may occur in the U.S. inventory of these types of structures.

PANEL COMPOSITION

The following persons participated in the study:

Dr. Walter Podolny	FHWA, Washington, D.C.
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Majid Madani	Caltrans, Sacramento, CA
Dr. Mohsen Shahawy	Florida DOT, Tallahassee, FL
Doug Edwards	FHWA, Tallahassee, FL
Randy Cox	Texas DOT, Austin, TX
Brett Pielstick	Parsons Transportation Group, Jacksonville, FL
Kent Montgomery	Figg Engineering Group, Denver, CO
Alan Moreton	Figg Engineering Group, Tallahassee, FL

Method of Study

The scanning team was primarily interested in the following:

- Repair, retrofit, and maintenance problems associated with specific design, construction methods, and structural details.

- Successful and unsuccessful methods to overcome identified problems.
- Enhanced or emerging technologies relating to inspection, maintenance, repair, or retrofitting.
- Performance of cable stays and post-tensioning (PT) tendons.
- Corrosion protection methods.
- Methods for demolition, in the event of functional obsolescence or structural deficiency, and methods of widening.

To focus the study, the team developed specific questions in its areas of interest and sent the questions to the hosting agencies in advance of the tour. The amplifying questions appear in the appendices.

Part B: General Background

HISTORY AND PERFORMANCE OF SEGMENTAL AND CABLE-STAYED BRIDGES IN EUROPE

Switzerland: History and Performance

Salginatobel Arch Bridge

One of the very first reinforced concrete bridges with a fairly large span is the spectacular Salginatobel Bridge, an arch bridge that was conceived and built by Robert Maillart in 1929 in the mountains of Switzerland.[2] It has a span of 90 m (295 ft) and rise of 13 m (42 ft 8 in) and carries a local, single-lane road high above a precipitous gorge (figures 1, 2, and 3). In 1991, it was recognized as an International Historic Civil Engineering Landmark by the American Society of Civil Engineers (ASCE). Subsequent to a recent period of renovation and rehabilitation, the scan team was able to visit and examine this bridge closely. It is a classic example of

Figure 1. The Salginatobel Arch Bridge, with its reinforced concrete arch, was built in 1929 in Switzerland by Robert Maillart.

Figure 2. In 1991, the Salginatobel Arch Bridge was recognized as an International Historic Civil Engineering Landmark by the American Society of Civil Engineers.

Figure 3. Some hosts and members of the scan team pictured near the Salginatobel Arch Bridge [left to right]: Willi Schuler, H. Figi, Majid Madani, Randy Cox, and Peter Matt.

pioneering bridge technology that was followed in later years by prestressed structures.

First Applications of Prestressing Technology

The first applications of prestressing technology in Switzerland included:

1943 – World's first precast bridge.

1950 – First precast road bridge.

1951 – First rock anchors.

1962 – First soil anchors.

1962 – First cable stay.

The consumption of prestressing steel increased significantly, from almost zero in 1950 to more than 200,000 tonnes by 1997. The majority of prestressing steel has been used in bridges (68 percent), followed equally by buildings (15 percent), anchors (15 percent), and miscellaneous applications. Various commercially available systems have been used, including button-headed wires, strands with wedges, and bars with nuts.[3]

Swiss National Corrosion Survey

In Switzerland, a national survey into the corrosion of prestressing steel, undertaken for the Swiss Federal Roads Office, culminated in a report (No. 534) that was published in 1998.[4] The survey involved all Cantons (Swiss states) and other authorities. Of 143 structures of various types that were identified, about 50 percent had small to significant damage to steel or rebar. Of these, 38 were analyzed in detail, including 27 bridges, 10 anchored structures, and 1 agricultural silo. (The latter was the first structure to reveal corrosion, induced by the action of bacteria. Grease on the mono-strands reacted with the bacteria to create acetic acid that attacked the steel.) Of the 27 bridges, 12 were dismantled: 9 for inadequate traffic function and 3 for lack of structural serviceability. Of the 12, only 2 had significant corrosion of prestressing tendons, which was less than 2 percent of all the older structures.

In 1999, a 4-year research project, known as "ZEBRA" (an acronym in German for Assessment of Bridges Being Demolished), was begun by Prof. Thomas Vogel and others of ETH (the Swiss Federal Institute of Technology, Zurich) to garner more information. Along with previous studies, preliminary findings indicate the following:

- Cases of significant tendon corrosion have been found in Switzerland.
- Water (humidity) gains access with chlorides to weak points.
- No stress corrosion has been found.
- Corrosion damage can be avoided by early recognition of potential risks during inspection and maintenance.

- The quantitative extent of corrosion has not yet been established.

The general qualitative conclusion is that structures of prestressed concrete behave very satisfactorily and, so, continue to be built.

Performance of Existing I-Girder Decks

The Swiss found significant corrosion in a few post-tensioned concrete girder decks. Chloride infiltration of the concrete occurred through leakage of de-icing salts at joints between deck panels. Chlorides penetrated through the deck slab and caused corrosion of the webs and post-tensioning tendons within. Chloride-induced corrosion attacked from both inside and outside of the ducts, where concrete had not been properly consolidated and where ducts contained voids in the grout.

Figure 4. I-Girder from demolished bridge under test at ETH, Switzerland. Hosts [left to right]: Peter Matt, Aurelio Muttoni, Franco Lurati, Thomas Vogel, and Peter Marti.

Measures have been introduced to prevent or mitigate corrosion damage in new construction, including careful application of waterproof membranes and overlays, with attention to water flow and sealing of possible avenues for chloride penetration. In addition, grouting is now very carefully controlled with vents, at all high points, and additives to reduce bleed and the consequent formation of voids. Re-grouting is used, where appropriate and necessary, if and when voids are suspected. The grout itself is a simple mixture of water and cement with a bleed-reducing additive. Currently, routine inspection and extensive use of electrical potential mapping are used to check for any such damage to existing structures. Testing of I-girders removed from structures confirms the structural adequacy of post-tensioned technology (figure 4).

Durability of Cable Stays

The first cable-stayed bridge in Switzerland, a pedestrian bridge at Birsfelden, was completed in 1962. There are now 25, with spans up to 140 m (460 ft) at the most recent, the Sunniberg Bridge, near Klosters (figures 5, 6, and 7). Although the oldest cable-stayed bridge is 37, the average age is only 6 years.

BBRV, VSL, and EMPA have been major contributors to the development of cable stays in Switzerland. There are no Swiss standards dictating design; design is the engineer's decision. So far there have been no durability problems with any of these

Figure 5. Sunniberg Bridge near Klosters, Switzerland, completed in 1999, from a concept by Prof. Christian Menn.

Figure 6. On the Sunniberg Bridge, short pylons rise away from the deck to accommodate the curve of the highway.

Figure 7. A curved structure of five spans, Sunniberg Bridge is fully monolithic between the piers, pylons, and abutments. All movements are accommodated laterally by tall, flexible piers.

structures and no wind-rain or other vibration problems. (This may be due to the fact that the spans and, consequently, the cable stays are relatively short, whereas long stays usually tend to be more susceptible to vibration.)

The Wurenlos Cable-Stayed Bridge (1971) proved to be durable, despite suffering longitudinal cracks in the plastic stay pipes. The damage was attributed to the use of recycled material and absence of a proper construction standard. The cracked pipes were repaired in 1990, and all the stays were replaced in 1998. When the dismantled stays were examined, the grout material and filling was in generally good condition, and the stay wires (BBRV system) were centrally located. Although some voids were found at the top of the stays, resulting from grout set and bleed, there was no significant corrosion damage and, therefore, no loss of strength. So despite the difficulties, the system proved to be durable.

Germany: History and Performance

In Germany a segmental bridge is considered to be one mainly made of precast concrete segments and, as such, there are none. The Germans were reluctant to pursue precast segmental technology due to concerns for potential weakness at joints for water, salt ingress, and corrosion. Consequently, most concrete structures are of cast-in-place concrete, with reinforced joints and internal longitudinal post-tensioning. Many bridges comprise deck slabs that span and cantilever over widely spaced, underlying longitudinal structural concrete girders. The girders may be a hollow box or simple solid rectangular section (figure 8). The use of expansion joints is minimized by making structures continuous, with expansion joints only in between or at end abutments.

One source of problems in older construction was the use of coupling joints, where 100 percent of the post-tensioning tendons were coupled. Although continuous mild steel reinforcement was present across the joint, the structures exhibited cracking at the joints after the bridges had been in service. The problem has been resolved by stipulating that a maximum of 70 percent of the tendons may be coupled at a joint. Some older structures that exhibited cracking have been retrofitted. The most common retrofit is the addition of external post-tensioning continuous across the joints showing distress (figure 8).

Figure 8. Underside of a typical post-tensioned concrete girder highway viaduct on the A59 route near Cologne, Germany, recently rehabilitated with external longitudinal tendons.

In the past, transverse prestress was required when there was longitudinal prestress. Now, however, either reinforced or transverse prestress design is permitted. Transverse prestress is used particularly for long, deck-slab wing cantilevers. Transverse internal tendons used to be grouted; however, now, unbonded tendons are encouraged to permit replacement, if necessary. A typical unbonded tendon is made up of greased-and-sheathed strands. The sheathing is polyethylene (PE), and up to four such sheathed strands are contained in a smooth oval (flat) PE duct with no filler.

In general, the long-term structural performance of decks and overlays in Germany is good. Decks do not need replacing unless for other reasons, such as widening. Bridges are examined and maintained regularly, and any damage from carbonation or chlorides is repaired as necessary. So far, it has not been necessary to replace any structural decks in Germany.

Denmark: History and Performance

General

The Danes cited the results of a recent worldwide survey of 17,612 post-tensioned bridges of various ages, showing that 351 (only 2 percent) have durability problems due to corrosion of tendons.[5] The causes are attributed to many factors, such as the types of work, leaks, inadequate grouting, and poor coupling of tendons. Following are results of the survey:

Age range (yr)	<10	10-20	20-30	30-40	>40	Total
Total bridges	3965	4543	6015	3000	89	17,612
Percent	23	26	34	17	0.4	
Total damaged	11	81	136	119	4	351
Percent	0.3	1.8	2.3	4.0	4.5	2.0

The statistics and details broadly agree with the Swiss national survey and are in general agreement with similar findings from the UK. Such findings demonstrate generally good overall performance for post-tensioned structures.

Figure 9. Sallingsund (1979) was the first bridge in Denmark built in balanced cantilever with precast segments and epoxy-glued joints.

Performance of Sallingsund Bridge

The Sallingsund Bridge was the first bridge in Denmark built in balanced cantilever with precast segments and epoxy-glued joints (figure 9). After 20 years of operation, it has demonstrated excellent performance with no durability problems and, so far, has been a maintenance-free bridge. (Concepts from this structure were later incorporated in the much larger bridge over the Northumberland Strait in Prince Edward Island, Canada.[6])

The following features are considered to have contributed to Sallingsund's good performance:

- Although there are stiffening ribs under the top slab inside the box there is no transverse prestress. Plain, mild steel reinforcing was used throughout, with a minimum concrete cover of 40 to 50 mm (1.5 to 2 in), with 50 mm for edge beams and aggressive areas. Because the deck is only reinforced transversely, it has suffered some shrinkage cracks through the top slab. Nevertheless, there is no leakage of water because the deck is sealed with a membrane and wearing course.
- On the bridge deck, cross-fall is maximized with slopes down from the central crown and inwards from the deck edges to drainage channels at the edges of the roadway, approximately over the webs of the box girder. This allows water to run off as quickly as possible. In addition, the deck is sealed with a waterproof membrane, consisting of two layers of a bitumen sheet with an additional plastic-reinforced bitumen sheet layer. It is mechanically connected using thin, stainless steel plates folded over to secure it at the edges of the bridge deck, where a separate concrete parapet encases the edge. The deck is covered with an asphalt wearing surface over an open-textured, asphalt base-course drainage layer over the membrane. The membrane and drainage layer have small vertical drainage pipes through the top slab to carry water away. However, deck drainage holes do become blocked with vegetation and need periodic cleaning.
- The epoxy-glued joints between precast segments remain intact with no evidence of loss of epoxy, cracking, or leakage. During construction, tests were run at the Danish Technical University and in France to establish the best way to make watertight joints. The tests established the correct epoxy resin mixture and demonstrated the need to sand blast the joint faces to lightly expose the aggregate and to have low humidity during application. For field production, the most effective seal was obtained by applying epoxy to one face, except for the top 200 mm (8 in), where it was applied to both faces. In addition, a small channel at the top of each joint provided an extra seal with a bead of epoxy. A minimum temperature of 5°C (40°F) was allowed for epoxy application.

- Potential cross grouting of ducts through segment joints was a concern because the ducts were too close to each other at some joints. Consequently, whole families of similar tendons were grouted simultaneously using a special grout with a pot-life of more than 24 hours. Operations were closely monitored and controlled, step by step, for each duct at each joint, and there was a thorough inspection of all vents. Spot checks were made by gammagraphy, and a special pressure device was developed to detect any air voids and re-grout, if necessary (a technique also used occasionally by the Swiss).
- Damped hinges at the quarter points have been fitted with vermin guards to keep out creatures. Structurally, the damped hinges have performed well, with no cracks in any of the details. Some insignificant thermal cracking has been noticed in the cast-in-place pier columns, but otherwise the piers and monolithic connections with the superstructure are performing well. Sacrificial cathodic protection has been installed on some substructures. Precast concrete, lost-form ice protection shells at the water line are performing well in an aggressive climate. Chloride corrosion has occurred in the shells in the splash zone but the shells are sacrificial in any case. All concrete was air-entrained for frost resistance. In every respect, the bridge is performing very well.

Further information on Sallingsund Bridge appears in appendix c and Ref. 7.

Performance of Vejlefjord Bridge

Opened in 1980, the Vejlefjord Bridge (figure 10) is a cast-in-place balanced cantilever with spans up to 110 m (361 ft). Each cantilever has 108 tendons anchored in the top web/flange location (figure 11). Continuity tendons in the top and bottom slabs connect the cantilevers. Transversely, the deck has deep ribs under the top slab at intervals of 6.87 m (22.5 ft) and is stressed by 36-mm (1 -in) DSI PT bars.

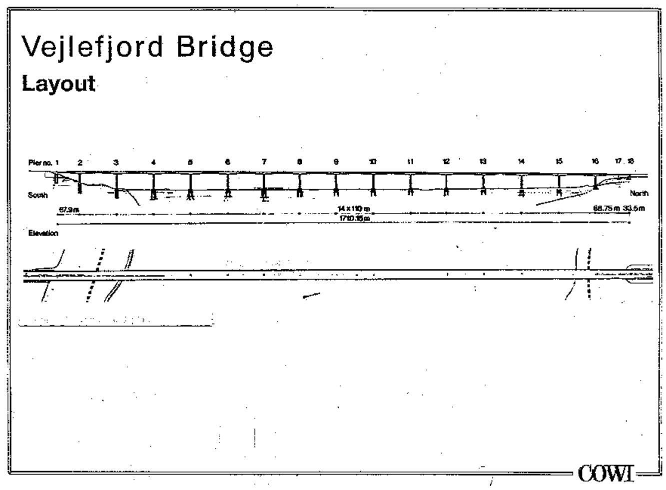

Figure 10. Opened in 1980, the Vejlefjord Bridge has a total length of 1,710 m (5,610 ft) of 110-m (361-ft) spans of cast-in-place, balanced cantilever construction in three continuous portions. (Courtesy of COWI).

The deck has a crown at the center and drains to the edges, where a narrow strip with a back-fall creates a drainage channel at a low point near each edge (figure 12). The deck surface is sealed with a primer and a bonded waterproofing membrane, consisting of two layers of bituminous sheet. There is a 20-mm (¾-in) porous asphalt drainage layer, a 30-mm (1¼-in) mastic asphalt base course, and a 40-mm (1½-in) stone-matrix asphalt wearing course. A tar epoxy drainage channel at the lowest point connects the drainage layer to vertical weep pipes through the slab (figure 13).

Vejlefjord is subject to heavy traffic and has required some repair of the pavement overlay at a cost of approximately $34,000 per year, for a total cost of $370,000 over

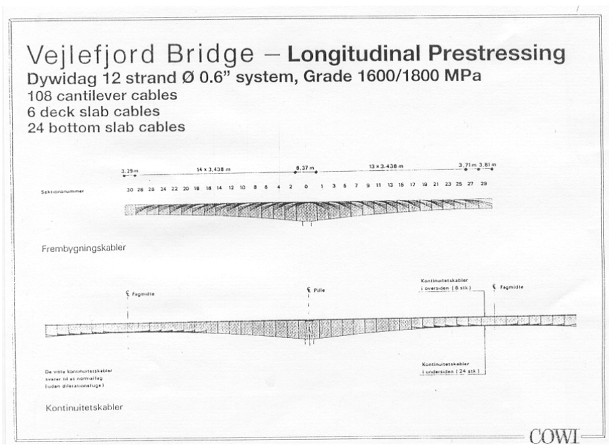

Figure 11. Cantilever and continuity tendon post-tensioning layout of the Vejlefjord Bridge cross section. (Courtesy of COWI).

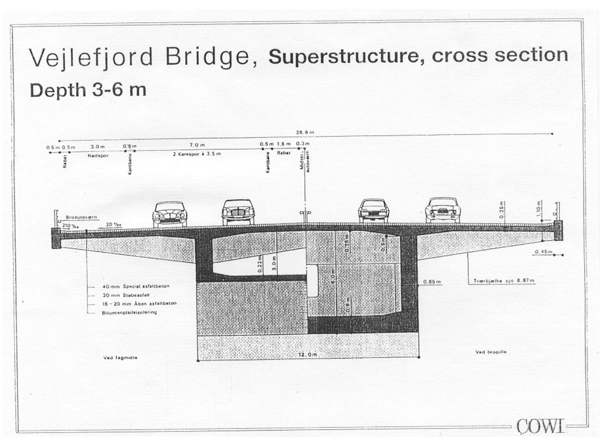

Figure 12. Cross section of Vejlefjord Bridge. (Courtesy of COWI).

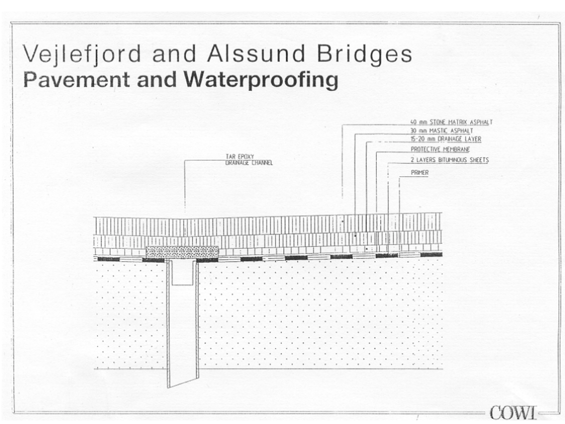

Figure 13. Pavement and waterproofing of Vejlefjord and Alssund Bridges. (Courtesy of COWI).

a period of 11 years. A complete replacement of the wearing course was undertaken in 1999, at a cost of approximately $920,000. Structurally, some concrete spalled from a web near one of the piers because of frost in a non-grouted duct. That repair cost $15,000. A total replacement of both the membrane and wearing course is projected for 2019, after a lifetime of about 39 years. Otherwise, no further repairs are foreseen, based on recent inspection.[8]

Further information on Vejlefjord Bridge appears in appendix C and Ref. 7.

Performance of Alssund Bridge

Alssund Bridge, near Sonderborg, is a cast-in-place cantilever, with spans up to 150 m (492 ft) (figure 14). The longitudinal PT system is very similar to Vejlefjord Bridge, with top cantilever tendons over the piers and continuity tendons in the top and bottom slabs through the midspan closures. The main and sidespans were constructed in free cantilever, using form travelers and the approaches on falsework. A temporary pier in each sidespan provided stability for the main span cantilevers. There is a hinge at the center of the main span.

Repairs to concrete have not been necessary and none are foreseen, because no damage is evident. There have been minor repairs to the pavement overlay for a total cost of $30,000. Replacement of the wearing course is anticipated around 2006, and replacement of the waterproofing in about 2016, after about 35 years in service.

Further information on Alssund Bridge appears in appendix C and Ref. 7.

Norway: History and Performance

Standardized Cast-in-Place Segmental Construction

The Norwegians have adopted and standardized a form of segmental construction in a manner that is slightly different from the

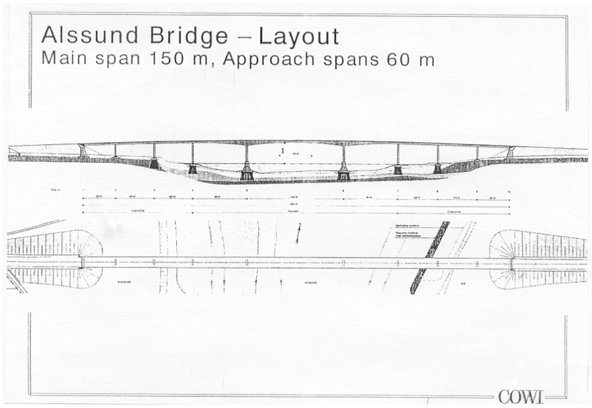

Figure 14. Alssund Bridge, near Sonderborg, Denmark, which opened in 1981, has a main span of 150 m (492 ft) with symmetrical side spans of 85 m (279 ft) and three approach spans of 60 m (197 ft) at each end. (Courtesy of COWI).

rest of Europe. Most structures are in remote locations where cast-in-place, balanced cantilever construction using form travelers is more convenient than precast construction. The superstructure consists of a standard box beam for two traffic lanes plus a sidewalk or bicycle path (figure 15). The out-to-out spacing of the vertical webs is 5.4 m (17 ft 9 in), which supports decks from 7.8 to 10.2 m in width (25 ft to 34 ft). Two boxes are used for wider structures. The box depth of the cantilever spans ranges from 1/20th of the span, at the pier, to about 2 to 3 m (6 to 10 ft) at midspan. Post-tensioning is by families of longitudinal, internal cantilevers and continuity tendons.

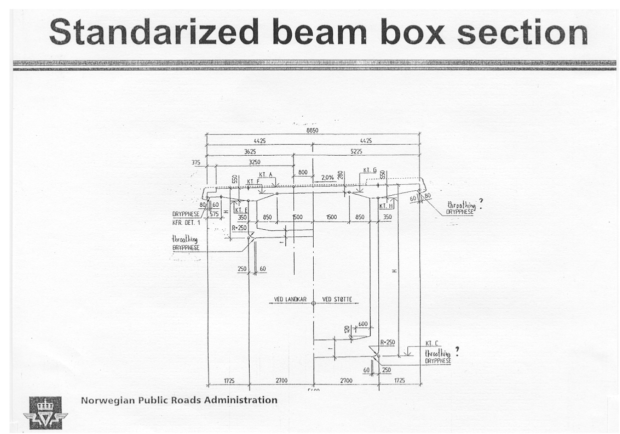

Figure 15. Typical cross section of standardized cast-in-place segmental cantilever construction in Norway. (Courtesy of Norwegian Public Roads Administration).

On average, these structures are about a kilometer long, with three or more main spans, ranging up to 301 m (988 ft). Typically, shorter approach spans are of cast-in-place, span-by-span construction using 2- to 2.5-m- (6-ft 7-in- to 8-ft 3-in-) deep box sections. The first bridge of this type was built in 1961; there are now 110 completed, with others planned.

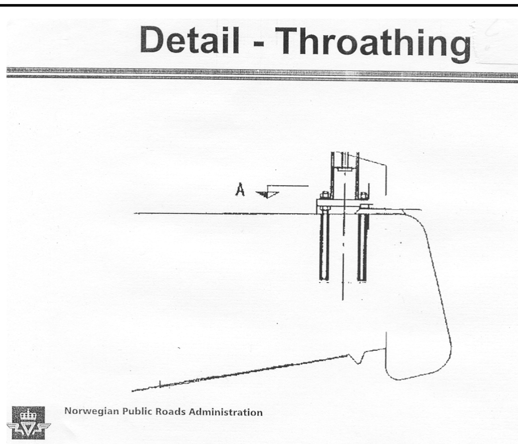

Figure 16. Detail of edge barrier and drip nose under deck. (Courtesy of Norwegian Public Roads Administration).

To enhance the durability of the deck, the deck slab has a male drip nose (figure 16) on the underside of the top edges, which was primarily for shedding water during construction. Parapet edge beams were cast after construction and have a down-stand below the top slab for shedding water. More recently, rounded bottom corner edges, with a radius of 250 mm (10 in) have been introduced, as this technique has been found to minimize

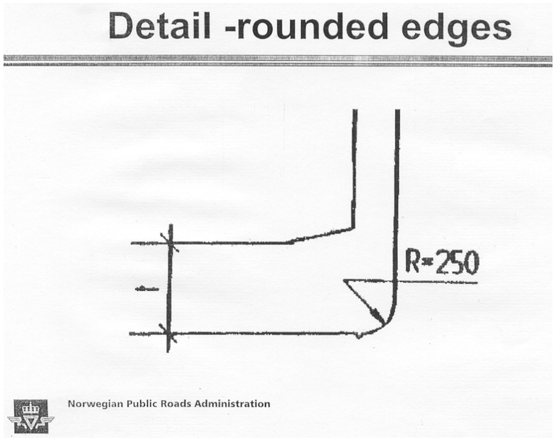

Figure 17. Detail of rounded corner edges to reduce frost attack, chloride penetration, and carbonation of the concrete. (Courtesy of Norwegian Public Roads Administration).

chloride attacks at sharp corners. The cost of these details was minimal (figure 17).

Durability and Norwegian Solutions

Many concrete bridges were built from 1970 to 1990. The first condition survey, in 1986, uncovered some chloride and carbonation problems. Further surveys that were conducted in 1991 and 1993 brought the total surveyed to 277, of which 143 are in exposed, rough marine areas. The remainder are in inner coastal areas with less ocean spray. Of the 143 bridges, 66 had high levels of chlorides and significant corrosion, and 54 had high chlorides but no significant corrosion. Corrosion of reinforcement had begun in some, but not all. The surprise was that problems were found on bridges less than 10 years old. This is attributed to the concrete design code at that time, which had no serviceability limit-state (SLS) checks that would have influenced the placement and spacing of reinforcement. Only ultimate-state design was used, with concrete cover generally around 25 mm (1 in).

It took some years to implement changes, including introducing limits on crack widths, increasing concrete cover on decks from a previous 35 mm (1.4 in) to 40 mm (1.6 in), and restricting the water/cement ratio to a maximum of 0.40. In 1973, code changes had relaxed requirements for crack-width control; however, in 1987 and 1989, the codes were strengthened again for both concrete cover and crack control. There was a concerted commitment to investigate any damage and implement new practice, based on the findings of the surveys.

New strategies apply experience and observations to develop solutions by addressing geometric shape, clearances, surface treatment, reinforcement, concrete composition, cover, and crack widths. This led, for example, to the adoption of rounded bottom corner edges and forms the basis of the current approach to design and construction. There was also a commitment to educate site workers regarding proper fixing of reinforcing steel, concrete placement, consolidation, curing, and so forth, to improve the overall quality of construction.

Features of the new practice also include a minimum clearance above water; the use of closed-box sections to minimize exposed surface areas; the use of rounded exposed corner edges; and large drainage pipes made of stainless or acid-proof steel that extend well clear of the structure.

Puttesund and Sorsund Bridges

Two bridges, the Puttesund (1970) and the Sorsund (1962), suffered excessive long-term deflection [up to 0.5 m (20 in)] at the center hinges of the main-span cantilevers. This resulted in a slope discontinuity and a noticeably bumpy ride.[8] Up to 180 mm (7 in) of extra surfacing to ease the ride made the deflection worse. On

Puttesund, some inclined cracks occurred near the quarter points, indicating potential shear problems.

Repair solutions were investigated using non-linear finite-element, time-dependent (creep) calculations. Solutions involved weight reduction, by replacing concrete sidewalks with aluminum; replacing some portions of slabs and surfacing with lightweight aggregate concrete; and adding external prestressing. The addition of cable stays was considered for Puttesund, because it would have solved the shear and deflection difficulties. That was, however, ruled out as too expensive, because of the towers and details needed. Since transverse moment effects in the webs contributed to the reinforcement stress and cracking, shear strength was improved by adding vertical prestress.

Longitudinal flexure, rather than shear, controlled Sorsund Bridge. Because this structure had an open-bottom slab in a portion of the span, strengthening involved casting concrete is this area.[8]

Excessive deflection at midspan hinges was also reported in similar structures in Europe in the early 1970s[1] and, by the mid-1970s, had led to significant changes in design practice around the world that included improved calculation methods for creep and the elimination of central hinges. Repetition of such problems has been avoided in structures built after this period.

France: History and Performance

Prestressed Bridges

n France, the Luzancy Bridge (1941-1945) is the most famous of the first prestressed concrete bridges built by Freyssinet. This bridge, over the River Marne, is a portal frame structure with a span of 54 m (177 ft). The first regulations for prestressed concrete were drafted in 1953, and the period from 1955 to 1970 saw rapid progress spurred by motorway construction. Spans increased to more than 190 m (623 ft) and single-box widths up to 25 m (82 ft). External prestressing and mixed internal and external followed in major structures in 1980, although four bridges had been built with external prestress in the 1950s. [9] By the end of the 1990s, based upon plan area, 47 percent of the bridges of the French national road network were of prestressed concrete (this is 18 percent, by number).

As with many new technologies, early prestressed structures experienced problems and needed repair. About 50 large structures (out of an inventory of several thousand) were strengthened, mostly with additional external prestressing tendons. Problems were attributed to many sources, such as concrete beams with congested internal tendons that caused honeycombed concrete and led to corrosion. First-generation bridges suffered when there was no waterproof layer, especially if there were anchors in the top of the slab. There was a lack of attention to details that allowed water to leak into anchors, lack of protection of anchors, or exposure of anchors at expansion joints, and so on. Other problems were related to creep and shrinkage or thermal gradients in bridges built by phases (in cantilever). Also, problems arose from poor construction, such as incomplete grout. However, although many duct voids were found, where the ducts were sealed with no water path, the post-tensioning tendons were not corroded.

Figure 18. Brotonne cable-stayed bridge over the River Seine, France.

Over the years, some lack of proper care and maintenance allowed deterioration. In some cases, too many overlay layers had been added and exceeded the design dead loads. Many lessons have since been learned and disseminated throughout the industry, worldwide.[1, 8, 10] Considerable progress has been made and incorporated in new criteria and standards, so repetition of problems has been avoided.

Precast Piers

Precast, post-tensioned segmental piers are sometimes used in France, occasionally with architectural-shaped or complex caps. Many bridges on the TGV railway line have precast piers, and slipped-formed piers are also permitted. The appearance of piers is becoming a very important issue with communities and owners, and slender, aesthetic shapes are often needed.

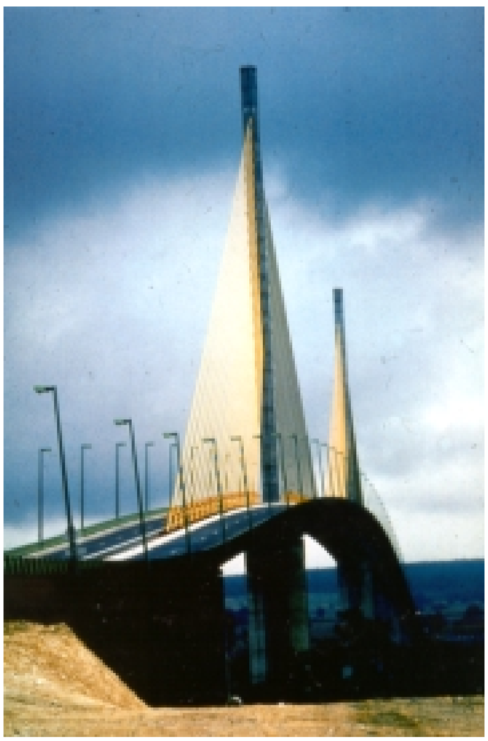

Figure 19. A detail built into the design enabled the cable stays of the Brotonne Bridge to be re-tensioned about 10 years after construction to adjust for long-term creep deformation.

Cable-Stayed Bridges

Many cable-stayed bridges have been built in France. One, at Brotonne over the Seine, (figures 18 and 19) became the forerunner of several similar structures around the world. It was partially precast and features a single, central plane of stays with a main span of 320 m (1050 ft). A mechanism built into the design enabled stays to be re-tensioned to compensate for creep deflections, years after construction.[11] The most recent notable structure is Normandie Bridge, which features a new type of external ribbed high-density PE (HDPE) stay pipe, introduced to reduce wind-rain vibrations.[12]

A bridge carrying the A9 Autoroute over the Isere River has a single saddle for all tendons at the top of the single, central pylon. Cable stays, made up of multiple strands, lie side by side in the saddle and do not bear upon each other. The stays are sheathed but have no outer pipe. A similar saddle to that on the Isere Bridge was also used on the Bourgogne Bridge at Chalon-sur-Saone.

United Kingdom: History and Performance

Ynys-y-Gwas, South Wales

The collapse of a small bridge at Ynys-y-Gwas, in South Wales,[13] caused by corrosion of PT strands passing unprotected through porous mortar joints, prompted investigations into the durability of bridges in general, especially those with grouted post-tensioning. The need for a study was further highlighted by reports of corrosion damage to other structures in the UK and Europe. A subsequent survey of 200 reinforced concrete and post-tensioned bridges identified a number of factors that contributed to inadequate durability.[14]

Although fears were expressed in the UK about the potential leakage of de-icing salts through joints between precast segments, there appears to be little evidence to support this fear on modern, precast segmental bridges. It should be pointed out that the small, 52-ft span bridge at Ynys-y-Gwas was made of short (7-ft-long) pieces of T-beam with a single strand in the bottom of each web. The T-beams were jointed with thick, sand-cement (porous) mortar joints and a tube of cardboard was used to form the duct at the joints. The strands were susceptible to attack at the porous joints and this is where they failed. The bridge was built in 1954. It was not made of large trapezoidal concrete segments, did not have multiple-strand tendons, did not have tight fitting, match-cast, epoxy-sealed joints and, in general, cannot be described as a segmental bridge in the manner that the term has come to mean today.

Findings of UK Survey

Results of the survey indicated that most defects and damage occurred in areas where amendments to existing specifications, along with improved inspection and maintenance, would significantly improve future durability. The most serious source of damage was salt water leaking through expansion or construction joints in the deck or utility ducts and poor or faulty drainage systems. The report concluded that there was a need for good, bonded deck waterproofing and better protection of structural surfaces exposed to salt splash such as piers, abutments, and parapets.

In 1992, until findings were known and new measures could be implemented, the UK Highways Agency placed a moratorium on bonded, grouted internal tendons. The announcement, in a press release by the Minister of Transport, provoked a reaction by the entire industry. A working party was set up by the Concrete Society of the UK, and various studies led to the publication of Technical Report No. 47 (TR47) on *Durable, Bonded, Post-Tensioned Concrete Bridges*.[15] The Highways Agency introduced higher standards for all structures, which were outlined in a new design manual.[16]

Subsequent reports reveal that the UK bridge stock is in reasonably good condition with only a few significant defects in a minority of cases. In most bridges, the post-tensioning system is in good condition with few, if any, voids and no corrosion — no bridges are in imminent risk of collapse. The moratorium did, however, provide the opportunity to reconsider design and details and implement new standards and bridge management techniques.[17]

According to a recent investigation, the segmental bridge carrying the M180 Motorway over the River Trent, which was completed in 1979, is in good condition with little or no signs of corrosion or tendon deterioration.[18] This bridge is contemporary with Sallingsund, in Denmark, and has similar epoxy joints and waterproofing.

OVERVIEW OF POLICIES AND PRACTICES OF EUROPEAN COUNTRIES

Europe in General

Currently, European Union (EU) countries have various rules of their own and most refer to the CEB-FIP code for creep, shrinkage, etc. Most, however, are converting to the new Eurocode,[19] which will have features similar to SEB-FIP and current French and British codes.

France

In France, SETRA is the technical department under the Directorate of Roads, Ministry of Public Works, that has responsibility for infrastructure planning to maintenance, including design, construction, operation, safety, and environmental protection. SETRA actively promotes innovation, development, and cooperation with the EU research and standards. In 1990, SETRA published a comprehensive document called *External Prestressing*, which presented the state of knowledge gathered over many years with clear diagrams and explanations illustrating typical applications.[20] The report contains outlines of specifications and details for ducts, deviators, anchorage areas, and implementation studies.

Current regulations for prestressed structures in France take into account both serviceability and ultimate limit states (ULS). At the SLS, the main features for bridges are as follows:

1. G = forces under dead load, taking into account the different stages of construction, including creep and shrinkage; accounting for losses due to friction and relaxation; taking all conditions to time infinity.

2. Q = live load (1970s criteria to be replaced by new Eurocode = approximately 1 tonne/m/lane).

3. Thermal gradient is applied, in combination with only dead load (TG = 12°C) and with live load (TG = 6°C). The top is always hotter than the bottom, with a linear gradient. The new Eurocode, however, will have non-linear gradients with some reverse gradient.

4. Normal stresses are considered as follows:

   Compressive stresses:

   $0.50\ f_{c28}$ under permanent loads
   $0.60\ f_{c28}$ under frequent and infrequent loads
   $0.60\ f_{cj}$ under construction or $0.55f_{cj}$ if "j" is less than 3 days except for well-controlled precast factory production when limit can extend to $0.67f_{cj}$

   Tensile stresses are considered for three classes:

   Class I In service - no tension

   During construction - $0.7\ f_{tj}$
   (f_{tj} = tensile strength at "j" age)

   Class II Stresses calculated on the basis of a non-cracked section

In service – infrequent load combinations

f_{tj} at cover depth
$1.5\ f_{tj}$ elsewhere (extreme fiber)

In service – frequent load combinations

No tension at depth of cover

Under construction –

$0.7\ f_{tj}$ at depth of cover
$1.5\ f_{tj}$ at the extreme fiber

Class III In service – no tension under permanent loads

(Also, limits are placed on stresses in prestressing steel and any passive reinforcement under frequent and infrequent load combinations according to detailing.)

Shear stresses at the SLS are also limited. The advantage is that it provides a minimum web thickness for shear. The limit is approximately 2.5 to 3.5 N/mm^2 (362 to 507 lbf/in^2) applied to the shear stress (not the principal tensile stress); however, the code formula derives from Mohr's circle for principal tensile stress. There is no limit on principal tension itself.

At the ULS the main feature is:

The load factor at ULS = 1.35G + 1.5Q

There is no explicit difference between bonded and unbonded tendons nor between internal and external tendons. The calculation considers no elongation or increase of stress in an external tendon at the ultimate state, because the SLS governs. Stress in the prestressing at ultimate is only evaluated for bonded tendons. Otherwise, the only difference between internal and external tendons is the friction. There is also a document that provides various recommendations on the detailing of tendons.

Compare the above formula to that of the American Association of State Highway and Transportation Officials (AASHTO), i.e., LF = 1.3D + 2.17(L+I). However, because live loads are somewhat heavier in Europe, the end product may be similar.

United Kingdom

In the UK, the Highways Agency has overall responsibility for the safety, economics, and durability of all highway structures. The agency issues various design guides and memoranda to supplement criteria contained in British Standards and Codes of Practice, such as BS5400 (Bridges) and CP110 (Concrete Structures).

In 1992, the Highways Agency placed a moratorium on the use of internal grouted tendons until design and construction standards could be reviewed. In 1994, it released Parts 9 and 10 of the *Design Manual for Roads and Bridges*,[21] which provided guidance for the design of concrete highway bridges and structures with external and unbonded prestressing. Factors influencing external tendons are presented together with guidance on their use, maintenance, replacement, robustness, flexure shear and torsion capacities, losses, anchors, deviators, and corrosion protection.

The Highways Agency introduced improved durability design and detailing requirements in 1995 for all structure types[16] with a basic philosophy to:

- Consider whole-life costs and do not design solely for the low bid.
- Use continuous decks and eliminate expansion joints.
- Avoid runoff of salty water onto abutments and piers.
- Avoid using in-span half-joint hinges (dapped hinge joints).
- For bridges less than 60 m long and 30° skew, use integral abutments unless articulation is otherwise necessary for settlement (e.g., mining).
- Consider using buried structures as often as possible.
- Consider eliminating reinforcement entirely by using more mass concrete for abutments and arch structures.
- Use non-corrodible external fasteners.
- Provide access for maintenance, painting, bearing replacement, etc.
- Provide corridors in abutments [preferably 1 m × 1.8 m (approx. 3 ft × 6 ft)].
- Increase previous concrete covers (BS 5400) by 10 mm (¾ in).
- Use external PT that can be replaced in service.
- Keep discharge drains clear and use robust pipes (not embedded in piers).
- Improve construction inspection, testing, and records [quality assurance (QA)].

Various ways of introducing and detailing the above improvements are discussed further in the reference manual. It also encourages use of improved concrete materials and attention to crack widths, detailing of reinforcement, and use of sealers on exposed surfaces. While much attention is given to concrete structures, requirements are also outlined for enhanced durability of steel structures, such as implementing simple, easily inspected weld details; avoiding intermittent fillet welds; making the steelwork self-draining; and paying proper attention to protection and maintenance.

When TR47[15] was published in 1996, the Highways Agency lifted the moratorium on grouted post-tensioning, with the exception of internal tendons in precast segmental construction. The current policy does not directly exclude this technique but rather lays out rigorous performance standards for the tightness of ducts, proven by a pressure test, but does not specify details. This has spurred parts of the industry to begin developing duct connector details for internal tendons.

Commercial products may be available soon. Internal tendons are permitted in the UK with cast-in-place segmental construction, provided that the ducts are detailed and made to prevent ingress of water.

The policy requires a multi-layer system of tendon protection, that anchors are not placed in areas exposed to de-icing salts, and that installation is done under a certified scheme. The number of grout vents and similar details were not specified prior to TR47 — now recommendations are provided. A test mock-up is required during construction. For external post-tensioning, the current UK opinion favors a sealed duct filled with grout or wax, but this is not a specific requirement. External tendons, however, must be inspectable and replaceable.

Recent UK bridges, such as the Second Severn Crossing, have external tendons in HDPE pipe with cementitious grout, in accordance with TR47. To simulate the behavior of fully bonded tendons, the tendons on the Severn Bridge are relatively short. The structure was required to be robust, to allow for a 25 percent loss of tendons and still carry the dead load. However, the 25 percent loss provision was also applied to localized details — not solely averaged over a whole cross section — and led to much more PT. (By contrast, the current AASHTO Guide Specification for Segmental Bridges requires 5 percent extra strand for potential construction difficulties and 10 percent for future load increases.) At the time the scanning tour took place, no major bridges were under construction in the UK, but the emphasis now is on designing for maintenance.

Based on whole-life costs, the Highways Agency found that galvanized tendons were the least expensive, but there were concerns about the galvanizing process. The galvanized tendons would have to be inside a low-humidity duct, within a waterproofed structure, with no cement grout. In France, only one or two bridges have used galvanized tendons for temporary repairs. In the United States, the FHWA will not approve galvanized strand if it is in contact with cementitious material. Although there has not been a documented case of hydrogen embrittlement in the United States, concern about it keeps galvanizing from being permitted.

Neither epoxy-coated strand nor polypropylene (PP) ducts have been used in France or the UK. However, stainless steel PT bars are available from the firm of McCall in England.

OVERVIEW OF MAINTENANCE INSPECTION

Switzerland

Information from maintenance inspection from each Swiss Canton is contained in a central database. These data are used for evaluation and to issue guidelines. The minimum inspection cycle is once every 5 years.

Germany

Many old bridges are not capable of carrying modern loads, so new classes of lighter truckloads have been established for load-rating an old bridge to a lower load class, as necessary. There are five classes in this lower load level (DIN 1072).

However, for heavy loads, trucks weighing more than 50 tonnes require a special permit for the vehicle and the route it is to take.

A German standard (DIN 1076) covers bridge inspection. Rules for inspection and maintenance were issued some 27 years ago and have been updated, based on feedback and experience. The standard requires a main, close, inspection every 6 years and a minor inspection at least every 3 years. Regular inspections three to four times a year may be made of known problem areas. Bridge inspection is carried out by experienced and qualified engineers aided by assistants. The inspection is primarily visual and must be of the whole surface area at no more than arm's length.

A computer software program, SIB-Bauwerke, is used for inventory, inspection reports, and for the Bridge Management System (BMS). It is coupled to a geographic information system (GIS) map. Detailed information on a bridge and its various components is input in the inspection part of the program. The engineer then identifies and classifies various conditions of different parts. The condition rating of various parts and components ranges from 0, very good, to 4, which is serious and needs attention. An overall condition rating is then calculated for the whole structure. Reviews of the inspection reports and the central database enable decisions to be made for further inspection or repairs. The BMS is under continual development and improvement.

Denmark

In Denmark, bridge inspection falls into three main types, which are as follows:

1. Routine
2. General/Principal
3. Special

Routine inspection is primarily for function and control, with the emphasis on traffic safety, and is classified into three categories:

- Category 1: The purpose is safety only. It involves an observation of the bridge by an experienced or trained road inspector, two or three times per week, per bridge.
- Category 2: The purpose is for traffic safety. It is a visual inspection made by an experienced observer/engineer to report on functional defects, for example, painting needed or filling of potholes. The frequency is twice per year, and the cost is minimal, estimated at US$50, per occasion.
- Category 3: This is a routine inspection of mechanical and electrical installations for traffic safety and functioning of devices (signs/signals) conducted by an experienced engineer or technician.

The purpose of a general or principal inspection is to register those bridges that need rehabilitation or replacement. It is done directly by the Danish Road Directorate or a consultant engaged for the purpose and must be made by an experienced engineer. It is carried out at intervals ranging from 1 to 6 years, depending on the circumstances and judgment of the engineer. Any damage is described and the condition is rated. Advice is rendered as to whether a special investigation is warranted. An economic evaluation and estimate of the cost of any repairs is made, and the date of the next principal investigation is established. The normal maintenance inspection, decision process, and execution takes about 3 years for a typical bridge. Manual and automatic monitoring are used occasionally, as necessary.

A special (ad-hoc) inspection is primarily to prepare repair and rehabilitation strategies. It is done, as needed, by a consultant bridge expert. The extent and cause of damage is established along with the repair strategy and estimate of cost. The cost of postponement is also evaluated — including costs to users in waiting time, detours, fuel, and delays — all according to predetermined vehicle user costs. All analyses are made in net present worth costs and provide the prioritization of repairs.

For the Vejlefjord and Alssund Bridges, routine inspections involve a walking tour, every 3 months, of the interior to look for any obvious problems and, perhaps, make a few measurements. Spot checks of the exterior concrete surfaces are made from a cherry-picker, biannually. Routine traffic safety inspections of the pavement are made weekly. Principal inspections of the structural concrete are carried out every 10 years and of the pavement every year. Special inspections are made as necessary depending on the evaluation of any damage and repair strategies resulting from the principal inspection. Present and cumulative costs are evaluated. Annual costs for all inspections are approximately as follows:

- Vejlefjord: Average US$85,000, but might range from $60,000 to $150,000.
- Alssund: Average US$33,000, but might range from $20,000 to $60,000.
- For both bridges, cleaning, alone, is approximately 25 percent of the annual costs.

The annual costs for Vejlefjord are higher than Alssund, because non-destructive evaluation (NDE), using georadar, under site conditions is being examined in order to gain experience and establish comparisons for subsequent inspections.

Norway

The Bridge Department of the Norwegian Road Directorate uses the Brutus BMS.

France

In France, a central inventory of bridges on highways is maintained and updated every year. Information is available on materials, area, age, and condition assessments. There are 280 box girders on the national road network and another 2,200 elsewhere.

In 1996, there were 21,549 bridges of all types (masonry, reinforced and prestressed concrete, steel, etc.); 3,984 were prestressed concrete (PC). The ages of the PC bridges by percentage was as shown below:

Age	10 yr	10-20 yr	20-30 yr	30-40 yr	40 yr	Not Evaluated
Percentage	22	33.5	32.7	5.3	0.7	5.8

Of these structures, two were demolished because of corrosion. Although the corrosion was not serious, traffic was heavy and increasing, so the decision was made to replace the bridges. There remain, however, four deteriorated bridges: one is very old and the other three are 30 years old. Two slab bridges have been transformed into stayed bridges by adding towers and cables. Most of the bridges are precast segmental bridges of relatively small spans constructed under a concession arrangement with the precast production industry.

Visual inspection is made every 3 years, and any defects are recorded and categorized according to a grading system known as the IQOA method. IQOA is the acronym for Image Qualité des Ouvrages d'Art, or assessment of the quality of civil engineering structures.

Class 1: Bridges in apparently good condition, but requiring ordinary maintenance.

Class 2: Bridges with defective components or protection elements or minor structural defects that need specialized maintenance, but with no urgency to repair.

Class 2E: Class 2, but repair is urgent to prevent further defects; without repair risks upgrading to Class 3.

Class 3: Needs repair but is not urgent.

Class 3U: Needs urgent repair.

Class NE: A bridge not yet graded.

Any urgent repairs that might impact the safety of road users are given a "safety mention," which is added to the condition grade.

The policy for the frequency and methods of inspection for ordinary bridges was established under technical instructions issued by the Transport Ministry. The goal is to make an inspection and a condition assessment, using the least effort, by qualified engineers and technicians, every 3 years according to the IQOA method. Also, a simple routine visit and overview of each bridge is made every year to assess if anything needs doing such as cleaning or further inspection.

For large structures, such as box girders and cable stays, a more rigorous inspection is carried out every 6 years. The inspection is primarily visual; however, additional separate and detailed inspections are made if there are any signs of a need. Various guides outline the details of the inspection needed for different components, such as cables, anchors, bearings, hinges, and joints. Detailed deck inspections that examine all parts of the bridge (sometimes including measurements of the tension in cable stays or suspension cables) may involve a few weeks of work.

In particular, a detailed inspection of a new bridge is made at the end of construction to establish a benchmark and again after 9 years. (There is a 10-year warranty for new construction.) Additional inspections are made for exceptional events such as fire or flood. Depending on the pathology, if defects are found, additional scientific investigations may be pursued (i.e., radiography, vibration measurements, etc.) to provide a complete diagnosis of the structure. To provide time for decisions and the design of repairs, a bridge may be monitored and temporary load restrictions introduced, while routine inspections continue.

United Kingdom

Maintenance inspections are primarily visual, with limited coring to explore suspected problem areas. Extensive reviews of concrete bridges were undertaken during the moratorium on post-tensioning.[14]

OVERVIEW OF REPAIR, RETROFIT OR REHABILITATION, DECONSTRUCTION, AND REPLACEMENT

Switzerland

Chillon Viaduct

Erected in 1966-69, the Chillon Viaduct is the only precast segmental bridge in Switzerland[7] (figure 20). It is 2 km (1¼ mi) long and comprises two parallel structures of 92-, 98-, and 104-m (302-, 321-, and 328-ft) cantilever spans with 1,376 precast segments. Expansion joints are at the center of every third span. Excessive long-term deflection occurred at some of the midspan joints. Initially, the riding surface was leveled up with an extra asphalt overlay, but the added weight caused more deflection. Consequently, the affected expansion joint cantilevers required retrofitting with additional post-tensioning.

Figure 20. Chillon Viaduct (1969) is the only precast segmental bridge in Switzerland. It has 1,376 match-

Research into external prestressing was undertaken to identify suitable retrofitting methods, since there were no existing design procedures. An external PT scheme was designed and installed and has proved successful. Monitoring of the expansion spans continues, using a hydrostatic leveling method. In other respects, there have been no problems and no leakage of epoxy joints. There is a waterproof membrane and asphalt overlay on top of the deck.

It should be noted that the deflection problems of the Chillon cantilevers were already known to the U.S. industry in the 1970s.[1] Since then, structures in the United States have been designed with a greater understanding of long-term deflectionsresulting from creep.

Viadotto sopra le Cantine, Capolago

This 30-year-old viaduct on the A2 Expressway in Southern Switzerland comprises two structures of 16 and 15 spans of 20 m (66 ft). Each span is made of precast I-girders, each 10 m (33 ft) long and joined at midspan with 50-mm- (2-in-) thick mortar construction joints at midspan. The deck is a cast-in-place, reinforced concrete deck slab, and the spans are continuous. Longitudinal PT tendons drape in the webs and pass through the bottom of the midspan closure joints. There is no longitudinal reinforcing through the narrow midspan joints.

In 1986-7, cracks opened at the midspan joints. Additional inspection revealed other cracks at pier heads and damage to expansion joints. There was concern about the possibility of fatigue of the prestressing tendons, because of the opening of the midspan joint. However, the PT duct is continuous through the joint, and there was no evidence of any corrosion of the prestressing, although corrosion of the deck reinforcing, resulting from cracking over the piers, was noted.

Proposals for rehabilitation were received in 1988. Reconstruction of the deck was considered in 1989 and of the full bridge in 1990. However, no action was taken and systematic monitoring continued through 1993-95. Rehabilitation with carbon fiber, external tendons, or full replacement was again studied in 1998-9, with possible deck replacement in 2001-2. Meanwhile, it is continuously monitored with remote sensors.

Continuous monitoring of crack openings is made at intervals of 2 seconds, over a period of 10 minutes. Results show that cracks open by 0.2 to 0.3 mm (0.008 to 0.012 in), under normal traffic, with an estimated increase of 200 N/mm^2 (29 ksi) stress in the prestressing tendons. Dead and live load moments amount to only one-third of the prestressing moment.

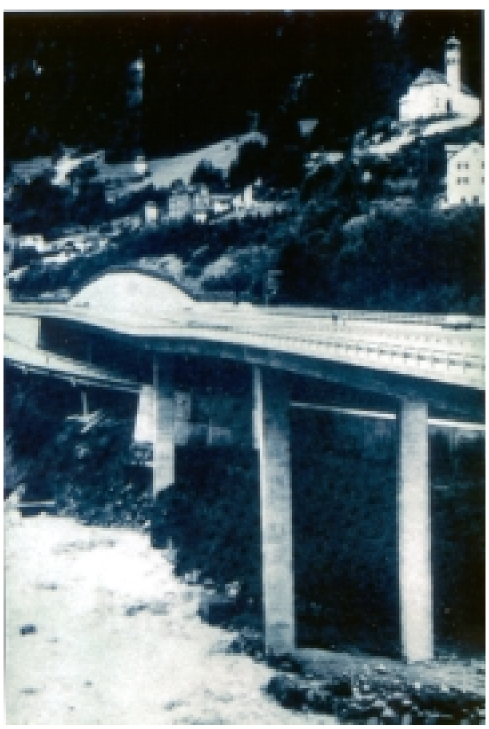

Figure 21. A main pier of the Reuss Bridge, at Wassen, was undermined by scour from excessive flash floods in 1987, but the bridge remained intact.

Analysis of the structure and stress in the tendons under traffic has enabled a prediction of the remaining service life. The analysis model comprised a grillage of longitudinal beam elements and transverse slab elements. The longitudinal post-tensioning consists of two tendons of 26-7-mm- (0.28-in-) diameter wires per beam. The concrete density is 25 kN/m^3 (160 lb/ft^3), and the asphalt overlay is 22 kN/m^3 (140 lb/ft^3). A 540-kN (122-kip) concentrated traffic load was applied. This analysis also revealed that the secondary effect of the prestressing increased positive moment at midspan, causing higher stress in the prestressing steel than assumed in the design. Monitoring continues. Although the center joint is similar to that of the Ynys-y-Gwas Bridge in Wales, monitoring and remedial action have prevented serious consequences. However, because analyses have shown that long-term fatigue could be a problem, the bridge will be replaced.

Reconstruction of the Reuss Bridge at Wassen

In 1987, heavy flash floods in the mountains caused considerable damage to many local roads, bridges, and villages. The foundations beneath a column of a post-tensioned concrete box-girder bridge were completely undermined, leaving the column and its base hanging from the structure.[22] The superstructure suffered considerable deflection and cracking, but did not collapse (figure 21), testifying to the inherent redundancy and safety of continuous post-tensioned concrete structures.

Figure 22. The pier of Reuss Bridge was shored-up, the post-tensioned box superstructure was jacked up to level, repaired, and load-tested.

Geotechnical studies and structural analyses led to a scheme of installing new, underpinned foundations and footings. Temporary shoring enabled the deck to be jacked back up to level (figure 22). The structure was repaired, external post-tensioning tendons were added to compensate for the forces and moments induced by the accident, and it was re-opened in 1988 (figure 23). The extreme nature of this accident truly demonstrates the redundancy of this type of construction.

Germany

Figure 23. The reconstructed Reuss Bridge was re-opened in 1988. This case demonstrates the advantages of the inherent redundancy of continuous, post-tensioned, concrete construction.

Since 1984, 186 older prestressed bridges in Germany have been evaluated. Of these, 30 were strengthened, 22 with external prestressing. The remaining 156 needed no strengthening. Bridges were strengthened with external tendons using anchor blocks, cross beams, and/or steel frames to create the tendon trajectory. Tendon layouts are profiled with deviators and are also straight. The external prestress is anchored in reinforced concrete blocks or cross beams between webs attached by transverse local prestress. There have been no service problems with shear or transverse tendons in Germany.

In most cases, external prestress was needed to restore serviceability and strength after flexural cracking at coupling joints; in others, it was to add traffic lanes. A coupling joint is a construction joint in the superstructure, frequently at the quarter point of continuous spans, at which all the longitudinal post-tensioning tendons have been coupled. The concept was popular in the 1970s; however, it gave rise to a plane of weakness at the joint, where the full tendon force was not properly mobilized, and led to cracking. Previous standards did not specifically address the design and details. Although the problem was first discovered in 1977, it took a few years to analyze and resolve. Initial attempts to simply seal the joints by epoxy injection proved unsuccessful. A few years after treatment, the cracks would reappear, indicating the need for extra prestress or reinforcement.

Consequently, the Germans conducted further investigations and analyses and developed a four-stage treatment program:

Stage 1: Examination of the joints under different loads, thermal conditions, and prestressing forces to determine those at risk.

Stage 2: Based on calculations, although the likely durability life of the joints is not identified at this stage, the joints are assigned to one of three categories:

1. Low risk.
2. Durability problems, but no safety concerns.
3. Serious risk, with potential fatigue of tendons.

Stage 3: Observations are made for short- and long-term thermal effects and possible stress conditions. Although the actual conditions in the joints cannot be known, due to unpredictable effects from shrinkage and creep, measurements offer indications.

Stage 4: A treatment program is developed and implemented.

Of 4,300 bridges under the administration of the Rhineland, about 1,050 coupling joints needed to be inspected and treated. Of these, 604 needed waterproofing, 312 also needed the addition of unbonded reinforcing, and some needed extra prestress.

Sealing of joints was effected using localized layers of different commercial products. Additional reinforcing was added in trenches cut by high-pressure water jetting. To cut a precise trench, the machine was mounted on rails and the nozzles were carefully directed. Local passive reinforcing bars, up to 28-mm (1 -in) diameter, added in this manner, significantly increased the ultimate moment capacity. Where only one joint needed reinforcing, extra rebar was placed in hydro-blasted trenches. Long reinforced trenches were used in only two cases. However, if several joints needed treatment in a structure, then external post-tensioning was used to avoid having to cut too many long trenches.

Normally, additional prestress was applied. It was found to be easier and more effective to apply unbonded, concentric, rather than eccentric (deflected), prestress, because it greatly simplified the details. New anchors were installed on extra cross beams, added between the webs if they could not be installed on the ends of the girders. Where necessary, the added cross beams were also post-tensioned. Also, additional anchor blocks (external buttresses) were constructed against webs and slabs, where necessary, for the new tendons. At existing diaphragms, it could be difficult to find locations for coring for the new tendons, because of the large amount of reinforcement and transverse post-tensioning. Reference was made to plans of the existing details as well as inspection of the bridge itself. Mostly, plans accurately reflected the details. Nowadays, only concentric, straight, external PT tendons are applied.

Rehabilitation began about 10 years ago in the Rhineland, and only a few bridges now remain to be done. A German federal publication has been issued to guide and specify treatment for other areas of the country. Under previous design rules, up to 100 percent of the tendons could be coupled at a joint. However, this is now limited to a maximum of 30 percent; the remainder must be continuous through the joint. The effectiveness was demonstrated when the new rules were implemented part of the way through construction of a new bridge. The first four joints done to the old rules cracked; the remainder did not. There have been no problems with any joints made under the new rules.

Figure 24. Anchor block for external tendon retrofit to continuous, post-tensioned concrete viaduct (L124 extension of A59), near Cologne, originally built with tendon coupling joints at the quarter points.

The rehabilitation and retrofit of structures with coupling joints spurred the development and installation of external tendons. Experience led to a preference for concentric, straight, unbonded tendons, to provide an extra stress of 1 to 1.5 N/mm^2 (140 to 220 lbf/in^2) (figures 24, 25, and 26). Typically, the extra prestress force can be kept to less than 3 MN (675 kips) per tendon, which can be achieved using up to twenty 15-mm (0.6-in) strands in up to five rows of four strands per tendon. Most external retrofit tendons are sheathed mono-strands in ungrouped, larger ducts. Details allow for de-stressing and re-stressing of these tendons, if necessary.

Maintenance repairs are carried out by both maintenance forces and specialized contractors, as necessary. Repairs to corrosion-damaged concrete are made by first removing the damage, using high-pressure water blasting [at 850 bar (12,000 lbf/in^2)]. The replacement material is a concrete mix with a high resistance to frost. Small areas are repaired with shot concrete to a thickness specified, according to the application, for either static or dynamic loads. Repair materials sometimes contain polymers to enhance the performance and resistance. Additional reinforcement is included if the thickness of a new shotcrete layer exceeds 50 mm (2 in). All materials are pre-tested to establish their proven durability for the application concerned. Epoxy injection is used to fill cracks. Sometimes, if repairs are needed, carbon fibers or plates are bonded to structures to

Figure 25. German hosts and members of the scan team examine the external tendon retrofit to the L124 viaduct near Cologne.

Figure 26. Scan team examines anchor blocks for external tendon retrofit of L124 viaduct.

increase the capacity; for example, on the underside of beams. Carbon fiber materials are much lighter and easier to apply and bond than steel plates.

Denmark

No rehabilitation or retrofit was reported from Denmark except for very minor repairs to frost-damaged concrete.

Norway

Various repair strategies are used on existing bridges in Norway. Strengthening or replacement is considered in conjunction with possible repair methods to determine the best strategy. Preventive maintenance and repair methods used include the following:

- Surface treatment/impregnation
- Mechanical repairs using spray-on methods
- Concrete jackets to columns or piles
- Re-alkalization
- Chloride extraction
- Cathodic protection

It is essential to clean the surface well before any surface treatment, taking care to restore exposed nails, pores, or cracks that can degrade coatings. Water-repellent impregnation should be done under dry conditions, which is difficult for existing bridges. Caution must be exercised when making mechanical repairs that require the removal of material by chisels, sequential operations, or the addition of spray mortars (shotcrete). Static calculations are required to verify sequential steps for mechanical repairs and shotcrete.

Re-alkalization has been tested on three bridges. It was found to work well at first, but not necessarily in the long term, because of local variations in cover, etc. Spray mortar may be needed to increase the cover before re-alkalization. In 1994, a trial was conducted of chloride extraction from 965 m^2 (10,400 ft^2) of substructure columns on the Salvoy Bridge. A reduction of 50 to 70 percent of the chloride level was achieved. The method looks promising, if certain conditions are fulfilled. If there are variations in cover, extra spray mortar may first be needed. The trial is limited, and there are no long-term results.

Between 1984 and 1994, cathodic protection was installed on 15 structures. The systems are based on paint, titanium mesh, and single anodes. So far, the conclusions are that they must be inspected and maintained continuously, with possible remote telephone-line monitoring. The technique effectively prevents corrosion, and the older installations are still performing well. Long-term results are awaited, and other techniques are being developed.[23, 24]

France

The French have many examples of long-term, durable structures of classic stone materials (figure 27) — some are a few hundred years old. Since the first post-tensioned concrete bridge was built over the Marne in the 1940s, the French have had 50 years of nationwide experience with prestressed construction of all types. Since the 1970s, 50 bridges (all built prior to 1975) have been strengthened by external prestress. Extra longitudinal post-tensioning has been added to bridges that needed widening or where the original prestress proved insufficient, as in some originally built in cantilever where two phenomena were not properly taken into account: creep and creep redistribution and the effect of thermal gradients.

Creep effects revealed themselves through vertical cracking of the bottom slab in the midspan region. Radial tendon forces due to continuity post-tensioning in curved intrados sometimes contributed to the cracking. Some cracks extended up into the webs. This was repaired using external PT with straight or deflected profiles (figures 28 through 31). A deflected profile is more efficient, but difficult to install, because it requires adding ribs to carry deviators.

In one extreme case (Blagnae Bridge near Toulouse), where a bottom slab was very badly cracked, the cracked areas were completely removed by cutting with a saw, but were not replaced. Instead, lateral stiffeners were added to carry the necessary extra longitudinal external tendons. Additional anchor blocks were constructed beyond the original ends of the superstructure boxes at the abutments.

Figure 27. The Pont Neuf Bridge in Paris (circa 16th century), which was recently cleaned, is a classic example of durable material.

Figure 28. The Corbeil Bridge, Francilienne, Essone, France, on which external transverse post-tensioning was added to the midspan region.

Figure 29. Internal frames on Corbeil Bridge brace the bottom slab and webs. External longitudinal tendons run inside the box near the webs.

Figure 30. External, greased-and-sheathed mono-strand tendons pass through a diaphragm on Corbeil Bridge.

Figure 31. Anchor protection for new external tendons on Corbeil Bridge.

New anchor blocks were attached to the existing superstructure by external PT bars.

Other local defects included transverse cracks arising from the local tension effect behind a row of PT anchor blocks arranged across the top of some bottom slabs. Repairs required adding transverse exterior mono-strand PT to help confine the anchor zones. The added transverse PT strand was secured with external blocks. In some cases, bars were used in instead of strands.

Shear cracks occurred in some webs. Subsequent analyses revealed that the cracks were a consequence of an improper assumption that the three webs of the box each carried an equal share of the shear force, when, in actuality, the center web carries 40 percent of the dead load shear. There was also a lack of tension in internal PT stirrup bars. The webs were repaired by adding vertical, external PT bars. Accurate drilling of central vertical holes down through webs to install vertical PT bars proved difficult. However, in one case, a web 4.2 m (13 ft 9 in) deep was drilled. The vertical PT bars were anchored with external steel plates and blocks at the bottom. The top anchors were recessed in block-outs and sealed under new waterproofing. On one occasion, the vertical PT bars were made of stainless steel. To prevent recurrence of cracking and to restore the integrity of the concrete, the cracks were injected with epoxy. External bars to either side of the webs have also been used.

Revised technical standards, introduced in 1975, prevented recurrence of the above types of defects. It should be noted here that the vast majority of segmental construction in the United States occurred after 1975 and, as a consequence, the types of problems described above have been avoided, due, in large part, to the 1977 FHWA report.[1]

Because prestress from additional post-tensioning may cause over-compression, rather than add extra PT, where possible, the existing is replaced in some way. For example, corroded transverse tendons were removed by trenching and then replaced with new ones. Similar transverse prestressing strands have been added to tops of deck slabs to enhance their capacity for higher loads. The strands are individually waxed (greased) and sheathed with HDPE and then grouted inside PE ducts (as used in Germany).

Similar prestressing has been added longitudinally, vertically, and transversely, as necessary, to retrofit existing structures. Webs may first be x-rayed in order to identify locations for drilling to attach anchors, and such types of repairs have been made without interruption to traffic. For example, additional external tendons were installed on precast segmental bridges in Vietnam by attaching them to webs with steel brackets.

In an unusual departure, in France, a bridge was strengthened by installing steel lattice trusses around the two precast I-beams of the existing superstructure to avoid closing the bridge. External transverse PT was also installed underneath the top slab to increase its local capacity.

When it came to widening another existing prestressed I-girder bridge, a question arose as to how to deal with the longitudinal shortening of the new girders, relative to the mature existing ones. The answer was to allow the new girders to stand in place for several months to permit most of the creep and shrinkage to take place. It is also possible to use steel beams — as was done to widen some I-girder bridges on the A7 motorway.

The widening of an existing arch bridge is currently being studied by SETRA. Built in 1980, the arch has a span of 260 m (853 ft) and carries a composite deck atop spandrel columns at 29-m (95-ft) intervals. The spandrel columns vary from 1.6 m (5 ft 3 in) in diameter to an oval section 2.0 m by 2.5 m (6 ft 7 in by 8 ft 3 in). The deck consists of longitudinal steel girders with a composite-reinforced concrete, transversely haunched, deck slab with a total width of 14 m (46 ft). When built, the concrete in the arch was 50 percent stronger than necessary, and the foundations are on solid rock. Consequently, the favored solution is to add transverse steel girders over each spandrel and then install longitudinal steel box girders to carry the new, extended-width, deck slab. An alternative solution would be to add post-tensioned column caps and replace the deck with an orthotropic deck. Another alternative would be to construct another, similar, parallel structure.

United Kingdom

There is no information concerning repair and rehabilitation.

DECONSTRUCTION/DEMOLITION

Switzerland

There is no information concerning deconstruction/demolition in Switzerland.

Germany

So far, in Germany, only post-tensioned bridges that were functionally obsolete for traffic capacity have been demolished. If an existing prestressed bridge has to be removed, then calculations and a plan of the removal method are needed for approval.

Denmark

In Denmark, one continuous, voided-slab deck was tested to ultimate during demolition. The initial failure was by shear, whereupon it was simply demolished. However, the Danes recognize that, whenever staged construction is used, caution is needed to take a structure down in a controlled and engineered manner.

Norway

To the knowledge of the team, no prestressed bridges have been taken down in Norway.

France

In France, the old Beaucaire Bridge developed sag at the center of the cantilever spans. It was decided to demolish the bridge, but river traffic could not be interrupted. The bridge consisted of twin parallel boxes with a slab between. A special frame, similar to a form traveler, was made and set up on top of the top slab. The top slab was partially saw-cut at the locations of the existing segment joints. Holes 80 mm in diameter were drilled through the slab on the saw cut to receive special wedge shaped, hydraulic splitters. These were used first to split and separate the slab and the first segment, at the cantilever tip. Then deconstruction proceeded in this manner for each segment. The segments were lowered onto a barge, taken to shore, and then broken up. This "reversed balanced cantilever" deconstruction worked well for this bridge, originally built in balanced cantilever. Figure 32 shows the new bridge (left foreground) and the deconstruction of the old bridge.

Figure 32. Beaucaire Bridge deconstruction. (Courtesy of SETRA).

For the demolition of a typical, three-span, highway overpass the major problem was removal while traffic remained fully operational on the motorway below. Similar to most highway overpasses, this bridge was a prestressed flat slab with no girders. Consequently, the slabs were removed en masse. A temporary steel girder was installed along each side of the entire length of the deck. Then two lateral girders were set on the longitudinal steel girders to support the span at the original pick-up points. Then the entire assembly was slid longitudinally back behind the abutment in a process of "reverse incremental launching."

United Kingdom

There is no information from the UK concerning deconstruction/demolition.

Part C: Current and Proposed Practice for Segmental and Cable-Stayed Bridges

DECK ELEMENTS

Deck Riding Surface

In Europe, with few exceptions, and for bridges of all types, traffic is not allowed on the structural deck slab. The majority of decks have a bituminous asphalt wearing surface. There is very little use of the structural concrete surface as the riding surface (some in Norway) or the use of concrete overlays. The primary purpose of the overlay is to protect waterproof membranes, bonded to the concrete deck slab. The membrane and overlay provide drainage to carry away de-icing salts and protect the structural deck. Riding-surface overlays are maintained and replaced approximately every 15 to 20 years. High-performance concrete overlays are not used in Europe.

Environment and Use of De-Icing Materials

All European countries use de-icing salts and chemicals. The Swiss typically use road salt; other materials are used only on a trial basis. De-icing salts have caused a lot of damage in Germany, so now, less salt is used. A sufficient, not excessive, amount is applied as a brine solution, along with salt mixed with sand. In Denmark, de-icing salt is normally calcium chloride. On the Great Belt, however, calcium magnesium acetate (CMA) is used to reduce corrosion. In France, de-icing salts are currently used, but other materials are being considered; however, most is salt, and the amount is not regulated. Retention basins are required by law on major French highways to contain salt-laden runoff. This requirement is being extended to other, older roads.

Deck Protection

For all types of bridges, Swiss policy is to apply a waterproof membrane, continuously bonded to the concrete deck surface, with a bituminous tack coat. The membrane is protected by a coarse-textured, asphalt drainage and base-course layer. This is topped by a bituminous wearing course. The drainage layer and continuous waterproof membrane protect the deck by carrying salt-laden runoff to the bridge drainage system. The top cover to reinforcing steel in the deck slab is typically 40 mm (1½ in).

Likewise, in Germany, traffic is never allowed on the concrete deck surface. A bituminous sheet membrane is always used to waterproof the slab with a minimum 50-mm (2-in) dense asphalt overlay. Concrete overlays, such as latex-modified concrete (LMC) are never used. Typically, an asphalt overlay lasts 15 to 20 years, then it is replaced. As such, the long-term structural performance of decks and overlays is generally good, and decks do not need replacing, unless there is a need for other reasons. If any damage from carbonation or chlorides is found, then it is repaired appropriately. With regular examination and maintenance, it is not necessary to replace structural decks.

In Denmark, waterproofing of the decks is similar. It consists of a waterproof bituminous membrane bonded to the concrete, then a thin drainage layer, followed by an intermediate course and wearing course for a total thickness of about 110 mm (4.3 in). Water is carried on the membrane to weep holes for drainage.

Norwegian practice is mixed. On some bridges the structural deck surface is the riding surface. Others, however, have a concrete wearing surface on top of the structural deck, but some problems have been encountered from shrinkage. On others, waterproof membranes with asphalt overlay are used, but not to a great extent. In general, the policy is to have the concrete provide the necessary performance.

In France, bridge deck waterproofing has been mandatory since 1968, and all concrete bridge decks must be protected by waterproofing, including those on steel girders. Standards were published by SETRA[25-32] for various waterproofing systems. Waterproof membranes must have the following qualities:

- Prevent ingress of water
- Resist chemicals
- Span cracks
- Bond to the concrete regardless of irregularity of surface
- Resist puncture
- Have good temperature performance
- Resist impact resistance

Four types of membranes and overlays are described in the specifications. The first consists of a primer, with a mastic asphalt membrane layer. The second employs a chemical polyurethane spray membrane about 2 to 3 mm thick. The third comprises prefabricated bituminous layers with fibers and is grit bonded, by heat, to the concrete. The fourth is a mechanical system laid by trucks using bituminous concrete, polymer-modified bituminous layers, and then a wearing course. All membranes are protected by an overlay of natural asphalt, found in France, and extended with bitumen. It is quite rigid. Special structures, with significant deflection, require a more flexible material.

The cost of the waterproofing layer is about 100FF/m^2 (US\$17.50/m^2); the overlay wearing course is extra. The working life for all types of waterproofing and overlays is about 25 years. Normally, the waterproofing is replaced when the wearing course needs replacing. If only the surface of the waterproofing is damaged, then the surface may be scarified and replaced. For more serious damage, such as rutting, the membrane is replaced. During installation, many samples are taken and tested for tension and bond. In-situ tests are also made, especially to check the quality of the waterproofing. If poor installation is found during construction, it is simply cut out and replaced.

When encountered, problems are not of the waterproofing materials themselves. Indeed, these are usually good and precertified. Problems usually arise from the design of details, application, and site installation. For example, it is essential to seal and close the edges of the waterproofing, to prevent intrusion of water behind or under the product. Wearing course materials are usually the same as the adjacent highway (unless there is a special need for a very flexible bridge, such as Normandie Bridge), and the criteria are in the guide. There is no limitation on the longitudinal grade.

By comparison, in the United States, waterproof membranes are rarely used. Colorado and Maine have used bitumen membranes. Where overlays have been used, most are latex-modified or high-density concrete placed directly on top of the deck slab. However, there is a tendency for delamination from differential shrinkage between the overlay and the underlying (older) concrete. Texas has experimented with asphalt concrete pavement (ACP) overlays, but finds that they soften and slip in hot weather. (For further information see Ref. 33.)

Barriers and Parapets

Currently, the Swiss typically use New Jersey-shaped barriers. In some cases, an additional granite curb is provided, at the base of the Jersey barrier, in the aggressive salt-laden runoff area. Barriers are considered salt splash zones and, as such, contain stainless steel reinforcing on the splash face. Surfaces of barriers are also sealed with applied coatings or hydrophobic sealants such as silanes and siloxanes. Concrete for barriers may contain micro-silica (silica fume) for added protection against chloride penetration.

Sealants (Penetrating Sealers and Coatings)

The Swiss generally apply sealers, such as silanes and siloxanes, to surfaces exposed to de-icing salts; i.e., faces of barriers. External, decorative coatings (usually epoxy-based paint systems) are applied to concrete surfaces as necessary for both color and surface sealing, as on the outside faces of the parapets of Sunniberg Bridge.

In Germany, concrete surfaces are not generally coated, except for exposed components, such as barriers, or repaired surfaces, when the repair is deeper than the rebar. Again, silane and siloxane sealers are used. Polyurethane, rather than epoxy-based chemicals, is used for coatings. A guide strictly controls coating materials to those tested and approved (similar to, for example, the Florida DOT's "Qualified Products List").

In Norway, surface sealers are applied on new bridges only where the environment makes it necessary, for example, on parts of columns close to or in the water, especially the tidal/splash zones, and edge beams of decks where de-icing salts are used in winter. Three different surface treatments are available: surface coatings (paints), water-repellent impregnation (silane/siloxane based), and a cement-based wash. The main purpose is to protect against moisture, chlorides, and carbonation.

Florida DOT has used silanes for a number of years and concluded that they are only effective for a short time. By contrast, Texas DOT found that linseed oil is as good.

STRUCTURAL CONCRETE

Concrete and Durability

Switzerland

The Swiss consider durable concrete important for three reasons: functional operation, safety, and overall appearance of the structure. Their approach is to consider the concrete as a structural member and establish the properties needed for good maintenance. As a structural component, concrete needs protection from chlorides in contact and splash zones. Because chloride penetration and concentration increase with porosity and frequent wetting and drying, use is made of waterproof membranes, coatings on concrete and rebar, stainless steel, and increased concrete cover — in appropriate combinations (above).

For good maintenance, concrete needs resistance to frost and de-icing chemicals, for which it needs a low capillary pore volume. This is improved by increasing the percentage of fines (e.g., adding micro-silica), using a low w/c ratio, air-entrainment, and an assured minimum curing time. All concrete is cured for a minimum of 7 days, with insulation and warmth, if necessary. (Note that most concrete is cast-in-place, rather than precast, so other curing methods, such as steam, are not typically used.)

Germany

In Germany, the typical structural concrete strength is 45 N/mm^2 (6,500 lbf/in^2), but it may range from 35 to 55 N/mm^2, for which the allowable tensile stresses are shown in the table below.

Concrete Strength N/mm^2 (lbf/in^2)	Final Tension N/mm^2 (lbf/in^2)	Erection Phase N/mm^2 (lbf/in^2)
35 (5,000)	2.8 (406)	2.2 (320)
45 (6,500)	3.2 (464)	2.5 (360)
55 (8,000)	3.5 (508)	2.8 (406)

Fly-ash is used to reduce the heat of hydration, but micro-silica is seldom used. Although the assumed design life of a bridge is 80 to 100 years, it is obviously not known for sure, since most bridges are relatively young (circa 1960s and 1970s).

Denmark

Corrosion of reinforcement commences with a gradual initiation phase, where chlorides migrate into the concrete. The initial corrosion may not be evident. However, rapid corrosion of reinforcement commences when the concentration of chlorides in concrete reaches a threshold level, a well-known phenomena.[34] Consequently, it is normal practice in Denmark to incorporate fly-ash and micro-silica to make concrete dense and impermeable. The corrosion inhibitor calcium nitrite is not used in Denmark, because of some evidence that it washes away faster than chlorides penetrate, making it ineffective (appendix C and Ref. 35).

Design for durability requires the consideration and application of many aspects, such as the required service life, the nature of the environment, the available budget and costs, the needs of current codes of practice, standards of workmanship, and aesthetic requirements. Various technical bulletins are available to assist with the design of durable concrete structures.[36, 37] The general approach is to develop solutions for the structural design, details, construction, and controls by considering the following factors:

- De-watering, waterproofing, and drainage.
- Castability, including the shapes of members, corners, and edges.
- Concrete mix design including:
  - Type of cement and pozzolanic materials.
  - Type of aggregates, reactivity, and porosity.
  - Water/cementious ratio.
  - Additives, chloride, and alkali content.
  - Air content and frost resistance.
- Reinforcement needed:
  - For crack control; i.e., bar size, spacing, and cover.
  - Possible skin reinforcement, stainless steel, galvanized, epoxy, non-metal.
- Location and type of construction joints.
- Workmanship and control including:
  - QA/QC processes and logistics.
  - Training, pre-testing, and trial casting.
  - Consolidation, vibration, and curing protection.
- Special measures such as:
  - Installating cathodic protection.
  - Using permeable and impermeable form liners.
  - Impregnating the surface with silane/siloxane.
  - Monitoring corrosion.
  - Identifying requirements for possible repairs.

The broad-based approach applies to all types of structures, not solely segmental and cable-stayed bridges.

Norway

The Norwegians focus attention on clearances and optimum geometrical shapes, to provide maximum protection and mitigate salt and ice effects, and other factors. Techniques include the following:

- Ensure a minimum clearance of superstructure above water.
- Adopt closed box sections.
- Use rounded edges.
- Use drip noses rather than drip grooves.
- Use large drainpipes of stainless steel extending well out of the structure.

An enhanced concrete mix is used with micro-silica (silica fume) for aggressively exposed portions of structures, and ordinary concrete is used elsewhere. The enhanced mix has a maximum water/cementitious ratio of 0.38, with a minimum cement content of 370 kg/m^3 (625 lb/cy), micro-silica content of 8 to 10 percent of cementitious material, and an air content of 5 $\pm$ 1.5 percent. By comparison, their normal concrete has a maximum water/cementitious ratio of 0.4, a minimum cement content of 350 kg/m^3 (560 lb/cy), micro-silica content of 3 to 5 percent, and the same air content.[38, 39]

When it was first introduced, the enhanced composition caused workability and placement problems. A training program was implemented for all in the industry and the difficulties were successfully overcome. The focus of the training is on why and how to use the new material; it is largely an information issue. Altogether, in the 1990s, a total of 1,870 people from county governments and industry went through the program.

Also, in Norway, research and development is currently under way to determine performance of the new materials and techniques, through systematic tests and measurement of chloride penetration. Research on the performance of chloride-resistant concrete began in 1991 with an investigation into eight bridges, built from 1952 to 1991, using different exposures and concrete composition. The results from the first phase of the study showed large variations in chlorides for similar conditions, and no significant conclusions could be drawn. A second phase will extend the study beyond 2000. For the second phase, two reference mixes from early 1970s and 1986 concrete are being used, along with studies of 187 different types of members, involving 15 additional concrete mix combinations at four different exposure and chloride attack conditions. So far, the performance of the new mixes is generally better than the older reference mixes, but the observations must continue for some time yet.

In a related area, the Norwegian Defense Construction Service is responsible for the design, construction, and maintenance of all concrete quays and jetties owned by the Ministry of Defense. In June 1998, the Service conducted a workshop of experts to develop possible measures to ensure cost-optimal marine structures with a long service life. Maintenance costs in marine areas have been very high. Savings could have been made through more attention to initial durability design and appropriate commitments to maintenance. The Service recognized that broad cooperation was needed to address multi-disciplinary and organizational issues that contribute to durability, long life, and minimal overall costs. The workshop produced broad recommendations for cost-optimal design and construction for various types of construction, materials, reinforcement, details, mitigation of corrosion effects, reduced damage, and special corrosion-protection measures to extend service life in marine environments. This plan includes the commitment by all parties to good-quality execution of construction; owners must select competent, qualified, experienced contractors, and not always by the lowest bid![40]

France

France has guidelines for the resistance of concrete to freezing and chlorides that include minimum cement ratios, etc. Fly-ash and micro-silica and other additives are not compulsory, but are used as necessary. Air-entrainment is used for freezing conditions, even for ordinary concrete. Corrosion inhibitors are not used. There are no "rules" for concrete to last a specific time, because it is not possible to apply regulations to things that cannot be measured (i.e., lifetime). Strength is used as a means of gaining durability.

United Kingdom

The scan team sought no information.

Reinforcing Steel

Cover

In Switzerland, different concrete materials and treatments are used, according to zones of exposure of the parts of the structure. The criteria listed below are applied to the cover to reinforcement and have been in place for about 10 years.

- Contact zone = where water may stand; cover is 60 to 80 mm (2.4 to 3.1 in).
- Splash zone = such as the barrier face; cover is 60 to 80 mm.
- Deck slab = cover is 40 mm minimum, plus waterproofing (1.6 in).

For concrete in contact with soil = 60-mm cover with bitumen seal to buried surfaces.

In Denmark, Dr. Rostam (COWI) pointed out that thicker cover to rebar is not necessarily wise, as concrete crack widths increase with cover. In general, owners like to limit crack widths. Epoxy-coated reinforcement leads to less structural bond and larger crack widths.

In Norway, as with construction anywhere, condition surveys indicated that specified minimum covers were not always met. Consequently, new rebar fixing techniques were introduced using special "fixing bars" to ensure correct covers and spacing. Some fixing bars are of stainless steel, making it easier to lay out and fix reinforcement. In addition, they are expected to provide visual indication of any onset of corrosion in the future. Cover to reinforcement is now controlled with a required minimum, and a nominal amount to allow for tolerances for cutting and bending bars, as shown in the table below.

Portion of Structure	Required Minimum	Nominal Cover
Underwater Casting	100 mm (4 in)	120 mm (4.75 in)
Splash Zone	100 mm (4 in)	120 mm (4.75 in)
Above Splash Zone	60 mm (2.4 in)	75 mm (3 in)
Elsewhere	40 mm (1.5 in)	55 mm (2.2 in)

For serviceability, concrete crack widths were first limited to 0.2 mm (0.008 in) in severe conditions. However, this led to large amounts of rebar and congestion. In 1993, the width was revised to 0.3 mm (0.012 in) to avoid the problems. Crack widths depend on cover, service-level stress, and bar size. The criteria were derived from the CEB-FIP code and were adapted to limit the stress in the bars at the service level.

Reinforcing and Coatings (Plain or Epoxy-Coated Rebar)

In Switzerland, some epoxy-coated rebar has been used in splash zones such as the faces of jersey barriers. Stainless steel rebar has also been used for some applications. Neither epoxy-coated reinforcing nor epoxy-coated prestressing strands are allowed in Germany. In Norway, reinforcement and fixing bars were epoxy coated; however, after reports from the United States, the use of epoxy-coated bars was discontinued in favor of ordinary, non-coated, mild steel and concrete enhanced with micro-silica.

Although epoxy-coated rebars were used for the Storbaelt Tunnel, they are not generally recommended in Denmark, because of poor results in the Middle East and other places where their use has been discontinued. Epoxy-coated rebar is not used in France either. In the United States, the Florida DOT no longer allows the use of epoxy rebar but has, instead, introduced enhanced concrete (more corrosion-resistant) mix designs.

Virginia DOT and some other States are looking into solid stainless steel or clad steel. Stainless steel rebar is now available in Europe at the proper strengths, sizes, and costs. Although it is more expensive than mild steel, it is not necessarily required throughout a structure, but can be used in combination with black steel in appropriate locations, such as barriers and similar areas.

As an indication of the long-term performance of stainless steel, Dr. Rostam (COWI) cited the Progresso Bridge on the gulf coast of Mexico. It was built 60 years ago using stainless steel rebar. There is no corrosion of the reinforcing, even though the chloride content of the concrete is approximately 10 to 20 times the threshold level, and the concrete is in relatively good condition. Current Danish thought is that, in the long term, the use of stainless steel is worthwhile. Although the performance of stainless steel is good, galvanizing of ordinary mild steel is not because it is affected by chlorides and the concrete.

Other possibilities for improving durability include the use of non-corrodible reinforcement, such as glass, aramid, and carbon fiber, but there is little experience and information so far in Europe.

Joints Between Segments

European practice is to apply epoxy to the joints of precast, match-cast segments — usually to one surface — but application may be made to more than one surface, as in the case of the Sallingsund Bridge, where it was applied to both faces of the top slab. There is no distinction between the performance of different types of joints.

In the United States, current practice is to classify joints between segments, as follows:

- Type A = epoxy or wet joints (as in cast-in-place)
- Type B = dry joints

Slightly higher-strength reduction factors (phi-factors) are allowed for type A joints.[10] Dry joints between precast segments are sometimes used in non-freeze-thaw zones. However, they must be sealed across the top to prevent unsightly water seepage. Different sealing techniques have been used.

Recent durability testing at the University of Texas (UT)[41] might have given the impression that dry joints are used with internal tendons. This is not so. In the United States, the practice is to never pass internal tendons through dry joints. Only external tendons are used with dry joints (i.e., tendons are outside the concrete, but inside the box section). Tests at UT clearly showed that non-sealed dry joints allow seepage, leading to corrosion of any internal tendons passing through the joint, and that epoxy-sealed joints provide superior corrosion protection for such internal tendons. Similar tests at Florida State University/Florida Agricultural and Mechanical University (FSU/FAMU), for the Florida DOT, provide additional data on different epoxy joint details.[42]

Applications of Lightweight Concrete

In Europe, there have been limited applications of lightweight concrete. High-strength or lightweight concrete is used in Germany only for an occasional experimental case. [Lightweight concrete generally has a density of about 18 kN/m^3 (115 lb/ft^3), compared with normal concrete of about 24 kN/m^3 (153 lb/ft^3)]. No structures of lightweight aggregate are built in Denmark.

Recently, lightweight concrete has been used on the center portions of some long spans in Norway. The aggregates came from Germany, Norway, and the United States. For long spans, higher strengths have been used, ranging from 60 to 65 N/mm^2 (8,700 to 9,400 lbf/in^2). Normal concrete strength is 55 N/mm^2 (8,000 lbf/in^2). Some experiments were made with very high strengths, up to 120 N/mm^2 (17,000 lbf/in^2); however, these proved to be impractical for construction.

Lightweight concrete is not often used in France. It was used, however, to lighten the central, 400-m (1,300-ft) span of the Iroise River Bridge, near Brest, in Brittany. This bridge has a central, single plane of cable stays to a steel-strutted box with two vertical concrete webs and outriggers under a concrete deck slab.

PRESTRESSING

Transverse Prestress Design and Details

In Germany, edge anchor pockets of transverse deck-slab tendons are recessed with a thin portion of slab over the top of the pocket. The anchors are capped and sealed with a grout backfill to the pocket. On a recent project, only 3 transverse tendons, out of 500, suffered any cracks during construction, reflecting generally good construction quality control (occasional cases have occurred in the United States and elsewhere). The cracks were satisfactorily repaired. Throughout Europe, anchors for transverse tendons on the edges of decks are protected under sidewalks or membranes, and water cannot get to them.

Longitudinal External and Internal PT

An external post-tensioning tendon is one that lies outside the concrete section; an internal tendon is one that is embedded within the concrete.

Traditionally, in the United States, an external tendon in segmental bridges consists of a number of 12- or 15-mm- (0.5- or 0.6-in-) diameter strands, housed in a black, smooth wall and HDPE duct with a wall thickness of approximately 1/20th of the overall diameter. Steel pipe (schedule 40) prefabricated to the desired radius is used through diaphragms, deviators, and anchor blocks. The steel pipe may be galvanized, plain, or epoxy coated. The steel pipe projects a short distance out of the concrete in which it is embedded, so that the PE duct can be connected to it with a thick, neoprene sleeve, secured with banding clamps. After installation and stressing, the tendons are grouted from end to end. This type of external tendon is most often used in new construction. It is very different from the greased-and-sheathed strand tendons now being used in Germany and France for the repair and strengthening of older bridges.

In Germany, the first prestressed bridge using external tendons was built in 1936, and the first with internal tendons was built in 1950. Thereafter, internal prestress was used for all bridges. External tendons were again tried 15 years ago, and, in 1999, new guidelines were issued. However, maintenance personnel cannot directly inspect internal tendons. Consequently, a few defects (honeycombed concrete and grout voids), revealed by rust stains, raised concerns. On one occasion, areas around grout vents on the top slab of a box were not properly smoothed out and prevented bonding of the waterproof membrane, allowing water to seep into the tendons. Such concerns prompted moves to external tendons using greased-and-sheathed strands. That said, the Germans consider that there is nothing inherently wrong with internal tendons, and they are still used, provided that they are made and installed properly. Internal tendons, however, are no longer permitted in the webs of concrete box girders, but they may be used in top and bottom flanges. Mixed construction is allowed using external draped tendons, along with internal tendons in top and bottom slabs.

Heavy truck traffic in Germany has doubled in the last 10 years, leading to more frequent and higher stresses. Consequently, retrofit by external tendons is necessary to increase the capacity of existing concrete and steel box girder bridges. This led to many applications for a new, greased-and-sheathed type of external tendon. These are easy to inspect and, if necessary, to replace. The grease is an approved material made by Shell. Examination revealed no signs of corrosion or other problems with the system on a bridge in service for 15 years.

The tendons consist of a grease-filled, extruded HDPE sheath around each strand. Four sheathed strands are laid side by side in a flat, oval HDPE duct. However, there is no filler between the sheathed strands and duct. Four or five ducts may be layered, making up a tendon of 16 to 20 strands. For installation, open, rectangular section HDPE housings are provided at deviator saddles, and a trumpet-shaped, flare cone is provided at anchors, so that the strands may separate and flare out to the anchor plates. At anchor plates, the sheath is removed and each strand is secured with a wedge in the normal manner. Tendons are stressed by pulling all the strands together with a multi-strand jack. After tensioning, the anchor cone and tendon are sealed with cement grout for a length of about 0.5 m. When first introduced, the new external system was about 20 percent more expensive than previous systems. To encourage its use, a disincentive of 5 percent was added to bids that used only internal tendons.

In Denmark, most post-tensioning tendons have been internal, using metal ducts, but some HDPE duct has been used. External tendons have been used for continuity tendons on a cable-stayed bridge in Norway; however, most tendons are internal.

Provisional Tendons (Construction Contingency/Future Tendons)

There are no requirements in Swiss codes for either provisional or future tendons. In Germany, provisions are required for the addition of tendons in the future, if necessary. In Denmark, provisions for future tendons are made on a case-by-case basis for new construction. However, as there are few new post-tensioned structures in this small country, no nationwide policy has been established. In France, provision has been made in the new Chevire Bridge for 10 to 20 percent additional (future) tendons, and all tendons are in PE sheaths (appendix E).

Corrosion Protection of PT

The only precast concrete segmental bridge in Switzerland is the Chillon Viaduct, which was completed in 1970. It was erected in cantilever using internal tendons. Since then, most new structures have been constructed cast-in-place with internal, bonded tendons.

The period from 1969 to 1994 saw the first generation of plastic (PE) ducts in approximately 70 bridges totaling more than 300,000 m^2 (3,227,000 ft^2). However, work at ETH, Zurich, undertaken by Thurlimann et al., from 1980-1987 and complemented by Marti et al. (1992-1996), culminated in 1994 in a new generation of PP ducts. Currently, approval is being sought for a new generation of robust duct known as "PT Plus," by VSL Corp. The advantages of the new system are enhanced corrosion protection and increased durability, along with reduced friction losses; for example, coefficient of friction = 0.14 for strand on plastic versus 0.18 for strand on steel duct. The main disadvantage of the new system is increased initial cost — incentives are needed.

The first applications of electrically isolated tendons were in ground anchors, in 1985, for which the anchor heads were made of an epoxy material. Electrically isolated tendons were approved for structural PT in 1997.

Consequently, the Swiss now address three possible duct types for different levels of protection for internal tendons: metallic ducts (spiral wound), corrugated (robust) plastic, and electrically isolated tendons. New Swiss guidelines under development will establish criteria for the selection and application of the tendon systems, broadly, as shown in the table below:

	Type	Longitudinal	Transverse
Cat (a)	steel ducts	only simple beams	not to be used
Cat (b)	corrugated, robust plastic	continuous girders and cantilevers	flat PE duct
Cat (c)	electrical isolation	compulsory on structures carrying direct current and for buried ground anchors	not required (DC does not affect transverse PT)

Quality assurance is strictly enforced by Swiss authorities. Various test standards must be met, especially for electrically isolated tendons where the electrical resistance must be high. The Swiss do not want to use epoxy-coated strand. In France, electrically isolated tendons are being considered for ground anchors, but not for bridges, except for stray current protection on electric railway bridges.

In the United States, PP, rather than PE, ducts are now used in some applications. However, PP ducts tend to be brittle in cold weather and may break if abused during construction, so care is needed. They perform well under warm conditions.

In Switzerland, for temporary corrosion protection of tendons during construction, a vapor-phase inhibitor (VPI) powder was used initially and found to last for about 8 months. However, it was never uniformly distributed through the length of the tendon and permitted local corrosion and pitting. Consequently, in 1999, VPI powder was prohibited. Water-soluble oil can be difficult to remove. (Also see references 43 and 44.) Inert gas (nitrogen) was tried with mixed results. The Swiss concluded that the best way is simply to keep the tendons as dry as possible by plugging vents and removing water.

In Denmark, only two or three of the many bridges of post-tensioned construction are suffering significant corrosion. Now, however, Denmark follows new European guides for improved quality assurance and methods of grouting of post-tensioning tendons that address both the grout material and procedures. In particular, the guidelines address the following:

- Avoid grout voids and bleeding during injection.
- The grout must create a passive environment for the tendon.
- The grout must be non-toxic, safe, and “green”; i.e., environmentally friendly.
- The execution of injection must be in a controlled manner with proper proportioning; long pot-life; ability to restart injection, if necessary; ability to wash out grout, if unsatisfactory; and grout must have good thermal properties (no excess heat of hydration or rapid set).
- Checking of injection should be simple, reproducible, and easy to inspect to ensure no voids from the injection process and grout bleed.
- Grout must be able to penetrate between the strands under a reasonable pumping pressure.
- Grout needs a long pot-life.
- Injection should be done free from frost.
- Consider economic benefits versus the costs — the benefits of good grouting are structural safety, durability (corrosion protection), and good environmental provisions.

Tests on the Brande Bridge, in Jutland, of a proprietary, cementitious thixotropic grout are currently in progress (in 1999), and results will be available later. Grout material itself, however, is only part of the issue; the rest is in installation of tendons, concrete, and grout. Good grouting is also supplemented with waterproofing of bridge decks.

In France, the 3-km bridge to Ré Island has tendon grout with micro-silica (silica fume), and double ducts are provided for possible future replacement tendons (appendix E).

Currently, there is no continuous duct system at joints of precast segments. However, the Freysinet company is developing a three-piece duct connector for internal tendons, in response to the UK ban on internal tendons through precast segments. Usually, grout is injected into all tendons at the same time to avoid crossover. Nowadays, the largest possible tendons are used, to minimize the total number of tendons passing through joints and the number of grouting operations.

A comprehensive conference was held in the UK, September 23-24, 1999, on Targeted Research Action for Environmentally Friendly Construction Technologies (TRA EFCT); the conference featured a section on the Quality Assurance of Grouting of Post-Tensioned Concrete Structures. The Proceedings contain information on new and high-performance grout, post-tensioning techniques, laboratory and field trials, and aids to inspection.[45] The conference was sponsored by the European Commission on Industrial and Materials Technologies Programme and the European Council for Construction Research, Development and Innovation (ECCREDI).

CABLE STAYS

Design Parameters

Cable-stayed bridges are popular and continue to be designed and built in Europe. One of the most recent is the Sunniberg Bridge, near Klosters, Switzerland (figures 5, 6, and 7 and appendix A). The cable stays are relatively short, up to 70 m long (230 ft), and are arranged in a parallel-harp pattern.

In Germany, locked-coil ropes are preferred for cable stays, suspension cables, and hangers, although there is a bridge at Mannheim with parallel-wire cable stays. To date, no stays have had to be replaced, but the design standard requires that stays be replaceable for both steel and concrete bridges.

The Norwegians published a *Technical Specification for Cables for Suspension Bridges and Cable-Stayed Bridges* in the form of a handbook as part of their Process Code[46] for administering construction projects. It comprehensively covers technical details of materials, different types of cables and ropes, anchor sockets and other components, corrosion protection, testing, workmanship, sampling, measurement, and so forth.

A new European code for cable stays is being developed. It will be similar to the new document by PTI, due in 2000.

Corrosion Protection of Cable Stays

For the Sunniberg Bridge, the cables were prefabricated in a factory by the firm of Stahlton and consist of a bundle of parallel, 7-mm (0.27-in) wires embedded in a

Figure 33. Cable stay of Sunniberg Bridge, Switzerland, consists of parallel wires in a matrix inside a thick-walled PE pipe.

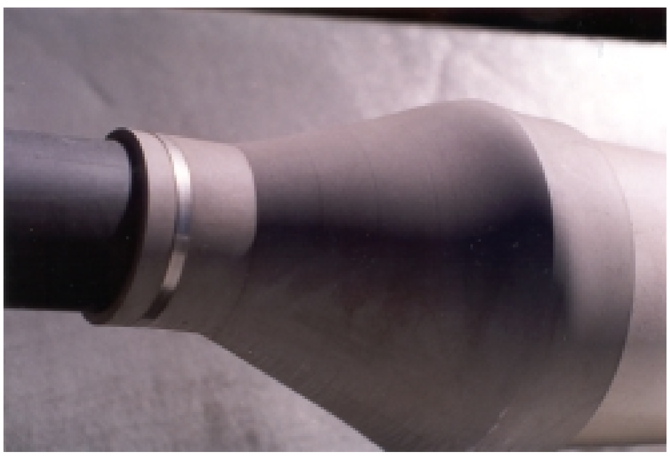

Figure 34. Cable-stay boot connects the external stay pipe to the transition pipe at the anchors on Sunniberg Bridge.

Figure 35. Scan team and German hosts visit the Rodenkirchen Bridge in Cologne, Germany.

corrosion protection compound, inside an HDPE sheath (figure 33). The outer pipe is connected and sealed to the transition pipe by a neoprene boot (figure 34).

Most cable-stayed bridges in Germany are in the Rhineland. Most cables are locked-coil ropes, but are not inside any polyethylene or steel pipes. Locked-coil ropes are also used for suspension cables and hangers (figures 35-39). In the past, different amounts of galvanizing and coatings were used. However, for the last 15 years, all wires within the locked-coil ropes have been completely galvanized. The wires are trapezoidal and “z”-section, high-strength, interlocking steel. However, since it appears that the trapezoidal section had some associated fatigue, only the “z” section is used now. Protective coatings of locked-coil ropes consist of primary coats of epoxy over the galvanized wires, with topcoats of polyurethane. Features where stays penetrate the deck are made watertight to leave no standing water. Minor, localized exterior corrosion has been found on the outer anchor clamps at the bottom of some suspension hangers, but it is insufficient to affect the hangar function and capacity. The corresponding upper clamshell hangers have no corrosion.

Early difficulties were encountered with some cables that had polyurethane (foam type) filling and coating. One on the bridge at Mannheim, with parallel-wire stays, was replaced with a stainless steel duct filled with inert nitrogen, at a slight overpressure. The pressure is monitored and a warning light indicates loss. At first, leakage occurred, but was detected and fixed. Since that time, more than 6 years ago, there has been no loss.

In Denmark, Anton Petersen (COWI) reported on the Zarate-Brazo Largo Bridges in Argentina. These are two similar cable-stayed structures in line along a highway, opened in 1978. The cables

Figure 36. Locked-coil cable hanger and saddle, Rodenkirchen Bridge.

Figure 37. Saddle for main suspension cables inside anchor buttress, Rodenkirchen Bridge.

comprised 7-mm (0.3-in), non-galvanized wires in cement grout in PE tube. In 1996, one cable suddenly broke as a consequence of corrosion (reducing the section) and fatigue. Many wires (about 70 percent) had corroded through prior to failure. The remaining stays were also affected and are now being replaced. There are also reports of vibrations, due to rain-wind, and sympathetic vibrations with the deck. Dr. Walter Podolny (FHWA) observed that the prefabricated, multiple, single-wire main tension cable elements were in storage for a long time and took a permanent coil set. So some damage may have been done during unwinding of coils.

Figure 38. Anchor assembly for main suspension cables, inside anchor buttress, Rodenkirchen Bridge.

The Faroe Bridge in Denmark, opened in 1986, has a main span of 290 m (951 ft), made of a steel, orthotropic box girder with a central plane of stays anchored to the central, upper pylon of a steel A-frame tower and to the center web at the deck. The cables are made of non-galvanized, 7-mm wire, grouted with cement inside a PE tube. The installation was carefully controlled during construction. The design, by Prof. F. Leonhardt, was patented at the time of construction.

The Normandie Bridge, in France, has stays of individually greased-and-sheathed, 15.2-mm (0.6-in), seven-wire mono-strands. Each wire has a thin, galvanized surface. The bundle of mono-strands making a cable is encased in a split-shell, circular section, outer plastic pipe with molded spiral ribs to reduce wind-rain vibrations. The plastic shell pipes were prefabricated in approximately 3-m (10-ft) lengths. Each half shell clips together around the strands and couples to the next

Figure 39. Deutzer Bridge, Severins Bridge (cable stay), and South Bridge; view from the Cathedral, Cologne.

section at a widened pipe bell-mouth. The detail leaves a tiny, triangular opening on the longitudinal seams at each coupling. A centralizing spacer wire wraps around the strands.

The Oresund Bridge between Denmark and Sweden (figure 40; appendix C) has a seven-wire, galvanized, greased-and-sheathed mono-strand system similar to that on the Normandie, but with a continuous outer HDPE pipe. There is no centralizing spacer wire, and the void is not grouted. A small portion of the cable near the deck anchors is filled with a wax compound. With this system, a single strand can be removed, if necessary. The cable stays of the Oresund Bridge are in harp formation, with pairs of cables spaced 600 mm (2 ft) apart, in the horizontal direction.

Figure 40. Model of main cable-stay pylons of Oresund Bridge, Denmark.

On the Normandie Bridge, oscillation suppression wind ropes (cross ties) are attached with special clamps to the outer PE pipes, where they are fitted with a local, internal neoprene cushion at each clamp. Locked-coil hanger ropes of 98-mm (3.9-in) diameter on the Stoerbelt Suspension Bridge, in Denmark, have an extruded HDPE surface sheath installed at the factory. Lengths up to 190 m (623 ft) were prefabricated and shipped on coils of about 2.5-m (8-ft) diameter.

In Denmark, Mr. Ejgil Veje, of COWI, reported that the cables on the 30-year-old bridge at Tancarville, in France, were found to be corroding and had to be replaced. Loss of section had reduced the factor of safety to about 2.0 for the main cable. The main cables were monitored with an acoustic system, which could detect the breaks in individual wires. The reflection of the sound provides a qualitative indication of the most damaged hangers, which is not an absolute measurement, but it helps prioritize repairs.

In France, Mr. Benoit Lecinq, of SETRA, reported that a committee will soon issue recommendations, similar to PTI, for cable stays and corrosion protection. The key issues being addressed by the committee are as follows:

- Corrosion
- Fretting at inter-wire contacts, deviation saddles, and anchors
- Ultraviolet damage to PE pipes

- Thermal effects
- Live loads

To reduce fretting, it is necessary to introduce a third body between the two contact surfaces. This could be the galvanic coating and could be an alloy of zinc or aluminum.

To protect against corrosion, the philosophy is to provide two complementary barriers. This could be an expendable galvanic coating and an outer sheath to protect it. For example, a complementary wax medium may surround the galvanized wires within a sealed HDPE or steel pipe, but there must be no holes in the sheath or pipe, so that it contains the wax. An alternative would be dry air — not yet used in France, but used on the Akashi Bridge in Japan and the Oresund Bridge in Denmark. Dry air is generated and circulated continuously, but does not count as a “barrier” in the new rules.

Another system comprises galvanic coating, within an extruded sheath inside an outer pipe, for the outer profile, for aesthetics and rain-wind characteristics. The outer pipe is not considered a corrosion barrier and is not sealed; it is vented (e.g., Normandie Bridge). The greased-and-sheathed mono-strand of this system suffers no reduction in strength from the galvanizing process. The typical strand strength is 1,860 MPa (265 ksi). The galvanized layer is 190 to 300 gms/m^2 and provides a coating of 10 microns. It is applied to the wire before winding and stress relieving of the strand. The wax/grease fills the interstices of the voids between the individual wires to eliminate water circulation. The wax is applied by opening and closing the wires. The outer polyethylene sheath is extruded onto the outside of the seven-wire strand and, because of the geometry of the shape of the strand section, it bonds to the strand. By comparison, locked-coil rope has galvanic coating, which constitutes one barrier; an extruded sheath, or similar, would make up the two required barriers. Epoxy-coated strands are not used in France; however, the new specification will be open to new technology.

Proper protection of the strand at the bottom anchor is critical. Where all strands are galvanized, the sheath is removed in the anchor zone and replaced by a filling material in a stuffing box. A compression seal is provided on the upper end of the stuffing box, around the individual strands. Filler in the stuffing box may be any suitable filler of resin or wax or dry air. The strands are anchored in a wedge plate. The ends are capped and sealed with the filler.

At the anchorage area where the stay passes through concrete deck, an outer transition pipe passes through the concrete. It can be an “anti-vandalism” pipe. A deviator ring is provided inside the top end of the pipe, which could be a damping device, too. A continuity sheath connects the outer stay pipe over the transition pipe. The anchor is accessible from below deck and the stay is replaceable.

In the new Eurocode, cementitious grouting of cable stays will be permitted, but the grout will not count as a barrier (same as PTI).

Vibrations of Cable Stays

Rain-wind induced vibrations were first observed in Japan as a low wind speed phenomenon on inclined stays on relatively smooth PE pipes. Vibrations were initiated by the wind on rivulets of rainwater running down the pipes. The introduction of a small protruding spiral rib on the surface of the stay pipes eliminated the effect. Subsequent work in Japan indicates that other surface irregularities can also be effective.

The Swiss report no wind-rain vibrations, probably because their cable stays are relatively short, whereas vibrations are more likely on long stays. Wind-rain vibrations have not been noticed on stays of locked-coil ropes, perhaps because the surface has a spiral shape reflecting the lay of the rope beneath the coating, and rivulets do not form in the same manner as on a smooth pipe.

Excessive rain-wind vibrations have been suppressed on German bridges; in some cases by installing cross ropes, in others, heavy-duty vehicle shock absorbers. Another method involves installing a relatively small star-shaped cross-connection device on a group of very close, parallel back-span anchor stays to damp vibration.

No recent wind-rain induced vibration problems have been experienced in Denmark. Some vibrations, however, were initially experienced on the Faroe Bridge, with amplitudes up to about 1.5 m (5 ft). The vibrations were eliminated by adding small-diameter wind ropes between the stays and spiral strips around the stay pipes. Monitoring verified the cause as rain-wind induced sympathetic vibrations from the interaction of the cables and supports. Despite the amplitude, the consequences were relatively minor. Initially, some fatigue was experienced by the wind ropes themselves. A detail was revised and the ropes replaced. The wind ropes are not attached to the deck. There are now no excessive vibrations.

So far, the Danes have been unable to develop any models to predict vibrations induced by rain-wind, as such, since other phenomena also contribute. Sympathetic excitation may occur due to periodic motion of a stay support or girder, leading to significant amplification of the motion of the support. Critical ratios are integer multiples of the support frequency. Wake interference (wake galloping) can occur because of close spacing of up- and down-stream stays. The cable stays of the Oresund Bridge are in harp formation with pairs of cables spaced at 600 mm (2 ft) apart, horizontally. Cable motions may also be induced by vortex shedding and buffeting. The Danes found that dirt or abrasion of cable pipes, over a long time, can change the surface and may lead to vibration. This was confirmed by wind-tunnel tests on new as well as abraded and weathered pipes.

There are five cable-stayed bridges in Norway. Three have locked-coil ropes, and there have been no vibrations. A few cables, made of strands, vibrated under wind conditions. Wind ropes (cross ties) were installed and eliminated the effect.

On the Brotonne Bridge, in France, cable vibrations were eliminated by installing heavy-duty vehicle shock absorbers, a short distance from the bottom anchors, to damp the motion of the stays. Brotonne has a single, central plane of stays anchored in the median of the deck. This solved the vibration problem. The dampers were installed 2 years after construction. The stays are strands grouted in steel pipes. An adjustment was made to the tension in the stays after being in service for 10 years; this provision was built into the design. (A similar stay-damping system was installed on the Sunshine Skyway Bridge in Florida.) Similar hydraulic dampers have been fabricated and supplied to the Normandie Bridge. However, they have only been installed on a few of the longest stays. Wind ropes (cross ties) of a special, damping material (not steel) are attached to the stays, anchored to the deck, and tensioned to a calculated load.

Visco-elastic dampers have been designed by Freysinet for installation at the upper end of stay transition pipes. One type was installed on the Dee River Bridge in the UK. Another type has been installed on the Vasco da Gamma Bridge over the Tagus River in Portugal. The dampers are tuned to the desired logarithmic damping coefficient; 3 to 4 percent damping is sufficient to prevent vibration. Both types are removable, without removing the stay.

EXPANSION JOINTS AND BEARINGS

Expansion Joints

Watertight modular joints with a test requirement and a design life of the joint itself of 40 years are the preferred system in Germany. The life of a seal on a modular joint is required to be 20 years. So they work for a few years and are then changed if found to be leaking. However, modular joints are noisy under traffic. The alternative — finger joints — is not as noisy, but leak and must be fitted with drainage channels. Modern practice in Germany and other EU countries is to make bridges as continuous as possible, preferably with expansion joints only at the abutments, in order to eliminate expansion-joint devices. In the United States, most expansion joints eventually leak, and water then gets to PT anchors and bearings, causing corrosion or other difficulties. Consequently, the use of continuous construction is encouraged.

Bearings

Bridges in Germany are always designed for bearing replacement, with an allowance of 10 mm (0.4 in) for raising the structure. When early stainless steel roller bearings were found to be insufficiently ductile and very brittle in cold weather (some failures occurred), they were replaced with pot bearings or neoprene pads. It is common practice in Germany to install a pointer and scale on bearings so that offsets can be read from a distance — a useful device for maintenance checks (figure 41).

Figure 41. Bearing movement gauge, L124 Viaduct, Cologne.

Laminated neoprene and pot bearings are commonly used in France. They are approved by certification, based on a given standard specification (T-47-811). The specification covers resistance to sliding; ozone; water; creep; bonding of layers; stress relaxation, under shear; and testing, under an imposed angle of deflection. Laminated bearings are limited in size, depending on the number and thickness of the laminated layers. Pot bearings have an elastomer inside a shallow cylindrical pot and piston. Sliding is provided by polytetrafluoroethylene (PTFE) on stainless steel plates, guided as necessary.

There are three issues with bearings, which are as follows:

1. Quality of the product: this is assured by certification of the product.

2. It is not possible to certify the installation process for any bearing, because it depends on site control. So a special inspection halt is made before and after setting bearings.

3. Calculations of the movements of bearings are susceptible to uncertainties in assumptions and errors. (In the United States, AASHTO imposes a multiplier of 1.3 to all calculated movements.)

Because the cost of bearing replacement after construction far exceeds the initial installation, care taken is worthwhile.

Concrete plinths are used to adjust for slopes and cross grades on precast beams. Tapered neoprene layers are not used. Neoprene bearings are mostly used for seismic areas, using multiple bearings on each pier, as necessary, but lead cores are not used. A technical committee in Europe has an Italian subcommittee to investigate seismic bearings. On high-speed rail bridges, longitudinal (hydraulic) dampers are used to damp seismic motion.

Part D: Emerging Technology and Management Systems

ENHANCED MANAGEMENT AND METHODS

Enhanced Bridge Management (Maintenance)

The replacement cost of the 35,272 bridges of all types on the federal roads in Germany is estimated at about DM90,000 million (US$50 billion), with a need for about DM900 million per year (US$500 million) for maintenance. Such resources are not available — costs rise and funds fall. Consequently, a comprehensive BMS is currently being established to address planning, control, implementation, and rehabilitation, with a large, up-to-date database and corresponding operational methods. Within the BMS, a “Structures Road Information Database” and corresponding “Federal Road Information System” were introduced in 1998.

The planning process includes the evaluation of damage and the selection and evaluation of remedial measures. At the network level, possibilities are evaluated according to urgency and financial needs. At the federal level, the data are used to provide prognoses and evaluate scenarios. Attention then shifts to implementation and realization.

Damage and condition assessments are first made, by visual inspection, by an experienced civil engineer. The degree of damage is reported in a systematic manner, in accordance with established guidelines, providing exact definitions of damage, locations, and components affected. The inspector does not perform structural calculations. Each instance of damage or deterioration is assessed separately, on a scale of 0 to 4, for impact on traffic safety, structural condition, and durability (0 = no damage, 4 = needs immediate attention). From the information, a condition rating is calculated in a manner standardized for all German states.

The German BMS is still being developed.The Federal Ministry coordinates the activities of the individual states, but optimization procedures are not yet in place. Prioritization, in terms of needs and resources, should be standardized for all States by 2005.

In Denmark, a comprehensive, web-based BMS involving coordination, planning, reporting, emergency assistance, and overview of ongoing work grew out of general needs and was prompted by needs of the recently completed Storbaelt and Oresund crossings. Results from inspection and monitoring are connected through normal Internet servers. All authorized users, owners, and consultants have immediate access with a standard personal computer.

The on-line BMS enables identification, planning, and reporting of maintenance and management activities, in coordination with other authorities. It provides an overview of all activities and secures updated information. Anyone can see if maintenance is up to date or if inspection is needed. Projected maintenance activities, details, and costs are listed for each bridge and each contract and contractor. Projections of future damage are based on results of previous inspections, costs of repairs, etc. All information is continuously updated. It is a comprehensive and powerful tool, in which needs are apparent, and it can be used to enforce inspection programs and funding, despite political pressures and other priorities. All relevant statistical data, organizational information, and needs are known, so budgets and work orders can be determined.

The BMS includes modules for minor bridges that address inventory, routine maintenance, principal and special inspections, monitoring systems, optimization of repair work, budget and cost control, long-term budgeting, rehabilitation, and pricing. Management systems for large bridges are more extensive, and modules in the BMS integrate organizational features, information, reports, management tools, traffic, operations, maintenance, equipment, and materials. It provides access to the management system for the owners, users, public authorities, contractors, consultants, and operators.

Concerns about security are addressed with access passwords for approved users. Access is available to all interested parties, and it is easy to expand and incorporate new partners. In the United States, the corresponding system is “Pontis,” but only by State.

Safety-Based BMS

In Denmark, a safety-based BMS is under development and pilot introduction. It is not a network-level system, but a model for evaluation of single bridges. A safety-based system is one that takes into account enhanced calculation methods, actual material properties, along with probabilistic loads and occurrences to obtain a safe assessment of the load-carrying capacity of weak or deteriorated structures. The motivation for a safety-based system is to provide realistic projections to reduce costs for strengthening without compromising safety. Cost savings are realized through the postponement or reduced degree of rehabilitation or replacement.

The general approach is based on codes for bridges and is applied to both new and existing structures. It takes into account the levels of uncertainty factors built into modern limit-state codes, their intents, probabilities, and conservatism in order to evaluate current and projected capacities, based on measured rates of deterioration. It involves proper modeling of stochastic variables, i.e., the nature and probability of loads, material strengths, redundancies, and modeling techniques. The safety of a bridge is calculated for conditions ranging from new through various degrees of deterioration, under different loads and traffic probabilities, in order to scientifically quantify the uncertainties. The redistribution of loads, under increasing load conditions, is also taken into account when determining the safe capacity.

By contrast, practices based on using “deterministic analyses” (that is, those based only on strict application of code formulae), provide a relatively low level of capacity. On the other hand, a safety-based, probabilistic analysis will result in a higher — but still safe — load level. The result can be a significant saving in maintenance costs. (For further information, see appendix C.)

The safety-based probabilistic approach arose from insurance needs for mega-projects, such as offshore platforms, pipelines, nuclear projects, tunnels, dams, harbors, coastal protection, airports, space, and similar industries.

In addition, there was a need for a rational risk analyses in potential accident situations, taking into account requirements for structural safety, to consider the probability of occurrence and the ability of the structure to withstand a certain action with a certain probability. This led, for example, to probabilistic-based capacity assessment for ship collisions. The technique involves the following steps:

1. Collect ship data at a location.

2. Estimate frequency of collisions.

3. Calculate capacity and consequences of impact.

4. Perform risk and reliability assessments.

5. Identify, evaluate, and implement risk-reducing measures.

The reasons for collisions are taken into account, including human, mechanical, and weather factors, in order to generate the probability of risk and likely level of impact load. Structural capacity and potential damage are determined using non-linear, dynamic analysis and actual material properties. Risk-reducing measures may involve improved navigation aids through the provision of fenders or impact caissons.

Traffic-Load Rating Using Reliability Methods

A load rating is made for various standard truck loads. Actual vehicles are related to standard trucks by the induced moments and shear forces. Using the same trucks, a reliability analysis is made, using probabilistic models of material properties and actual traffic patterns and loads. Results of material tests, inspections, and measurements are factored into the analysis. Failure probability is determined according to Bayes rule.[47] Each bridge is assigned a class, relating to the heaviest standard rating truck that can cross the bridge. The bridge class is shown on network maps for load routes. Actual vehicles are compared with the standard-rated trucks for internal forces and moments in each span to establish the class.

The benefit of the system is a consistent and rapid, nationwide evaluation and administration that prevents overloads. Most inquiries for the transport of heavy loads are handled by police.

Existing codes or regulations may not be applicable to the bridge and circumstance, in which case, a reliability-based bridge rating is conducted, taking into account the effects of various actions and observations, and leads to a structure-specific rehabilitation design basis. The daily distribution and standard deviation of truck types and weights is used to formulate probabilistic models and statistical load effects. Results of material and structure inspections are considered. Load resistance factor design is used, with characteristic values for loads and resistance. Partial safety factors are examined for actions and resistance provided by the materials of the bridge. The first partial safety factor studies were made on the Storbaelt project. Permanent loads of cables, found from direct force measurements, allow for low partial safety factors, where the known tensile strength defines the characteristic resistance. Fatigue results are converted into damage development. The monitoring interval determines the minimum material safety factor. Ultrasonic testing inspection is required to confirm it with calibration, through the reliability method. The method has a foundation in ISO codes and Eurocodes.

The Danes now have a flexible reliability design basis that can be adjusted and updated by new findings and results. For example, more traffic observations improve the reliability of the model and enable a safe reduction in the safety factor.

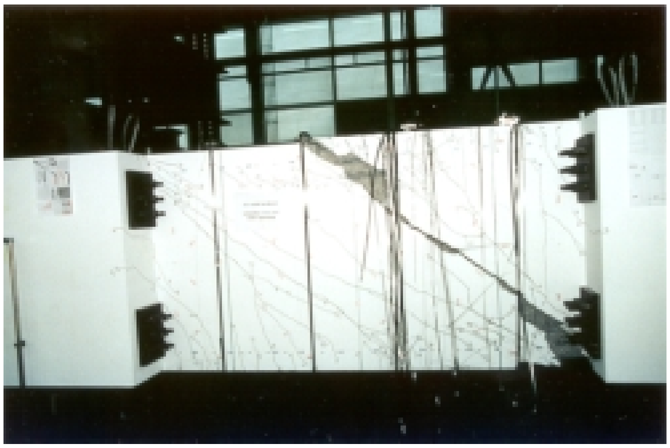

Figure 42. Carbon fiber shear reinforcing test, Swiss Federal Institute of Technology (ETH), Zurich.

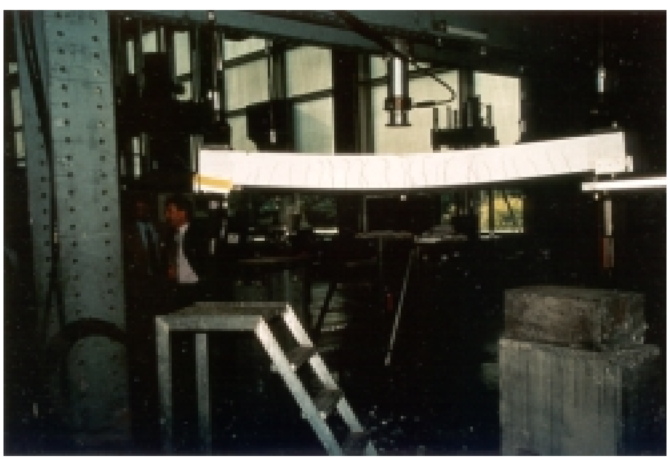

Figure 43. Fiber-reinforced concrete slab test at Swiss Federal Institute of Technology (ETH).

EMERGING STRUCTURAL MATERIALS

As in the United States, the use of non-corrodable reinforcement and post-tensioning material (such as carbon, glass, aramid fiber) for concrete bridges is under consideration in Europe (figures 42 and 43). In Germany, a bridge in a private-enterprise facility was developed and post-tensioned with fiberglass tendons. It was instrumented and some empty ducts were provided, in case the concept did not work.[48]

In Denmark, the cable-stayed Herning Footbridge was built as a trial project. It spans rail tracks and has a pier and pylon of “Corten” steel. The superstructure is a dish-shaped, voided concrete slab with normal transverse reinforcing steel. Longitudinal post-tensioning in the deck consists of 50 percent carbon fiber and 50 percent steel tendons, some non-grouted. Carbon-fiber reinforced plastic (CFRP) has been used for post-tensioning in the bridge deck. The stays are all carbon fiber with a 3-m- (10-ft-) long tube of stainless steel at the bottom to protect against vandalism. Longitudinal CRFP PT is inside the large-diameter voids.

The purpose of the Herning Footbridge is to develop experience in corrosion resistance and construction using new materials. Observation of Herning will enable projections for bigger bridges and economic assessments for comparison with normal highway bridges, using the total life cycle for carbon fiber or steel. Carbon fiber costs need to be reduced by about 25 to 50 percent to compare with either ordinary or stainless steel.

Danish engineers decided not to proceed with large-scale tests, at the moment, as there is a need to first pursue more studies of bond anchor capacity, linear elastic behavior, and so on. To date, only studies of longitudinal-fiber tendons themselves have been conducted. Test sections of the deck have been made to test the bond of the carbon fiber, too. Measurements will be made over the next 10 years. Corrosion cells have been installed, along with elevation bolts, for precise level observations, strain gauges in the cables, and load cells.

At this time, comparisons of costs in Denmark indicate that stainless steel reinforcing is less expensive than CFRP, if an overlay is not required. The cost of a new overlay is about US$80/m^2. Even so, stainless steel reinforcing has not yet been adopted as a standard practice. (In Europe, stainless steel PT bars are available from the firm of McCall in England.)

In most European countries, as in the United States, bonded carbon sheets and other artificial fiber materials have only been used for strengthening and repair. Aramid and CFRP have not yet been used for cable stays, except in limited research prototypes, in France.

Mohsen Shahawy, of the Florida DOT Structures Research Laboratory, reported that glass and aramid are not suitable for prestressing, because aramid is susceptible to moisture absorption and glass fiber is susceptible to alkali reaction. Glass is used at a low stress level (25 percent), but it has a low modulus, so deflection controls. Carbon fiber tendons from Japan have been used in model tests in Florida, and two bridges were built in Canada, for test and research purposes.

NON-DESTRUCTIVE INSPECTION METHODS

The Swiss have examined various methods and found the following results:

- Electrochemical: some success
- Endoscopy (fiberscopy): limited success
- Georadar: doubtful results
- Impact echo: mixed results
- Infrared scan: unknown
- Magnetic perturbation: not good for anchors or massive sections
- Radiography: impractical, too bulky, and gives limited imaging
- Reflectometrical impulse: unknown
- Ultrasonic: does not work very well

The overall conclusion by the Swiss is that there is no simple, reliable method for examining internal post-tensioning tendons and cable stays.

In Germany, some research into ultrasonic testing is being pursued to detect voids in grout. It is, however, generally difficult to detect voids with NDT methods, so such methods are used only if and when needed for a second stage of inspection. Current methods include:

- Half-cell potentials to identify possible onset of corrosion.
- Ground penetrating radar (or georadar) using equipment conveniently suspended behind a survey vehicle.
- Impact echo has been used to detect defects in walls and tunnels.
- Infrared laser scanning is under study for possible use in tunnels. It may detect voids or water behind tunnel walls, and it may be possible to detect cracks wider than 0.5 mm (0.02 in) with an improved system. The technique produces contour plots of an area's potential problems.
- Magnetic field measurement (perturbation) may identify ruptures of prestressing. However, it is complicated on an actual structure by the presence of rebar and other metal components. It is a slow process and not practical on a large scale. Research in Munich is aimed at a method for thick coatings.
- Ultrasonic testing methods have also been used.

For cable stays, the Germans have found no single, simple technique. Magnetic perturbation is used to monitor corrosion condition of locked-coil ropes for hangers and stays. An initial observation, at the time of construction, however, is vital for a baseline reference. The massive nature of anchor sockets is unsuitable for magnetic perturbation; however, cross-sonic logging has been used on some anchors.

In Denmark, endoscopes have had limited success for examining voids in grout inside tendon ducts. However, this is invasive, because access must be cut into the ducts. An alternative air-pressure device has been developed to test the sizes of voids and can also inject grout into a void. It is used on new bridges on a case-by-case basis.

Non-destructive investigations by georadar were tried on the Vejlefjord Bridge. The process involved the transmission of radio waves into the concrete in a 45 cone of propagation. Reflections of different strengths, indicating different materials, were displayed on a monitor. The equipment comprised an antenna with a frequency of 1 GHz and a computer with software for interpretation and display. The investigation focused on local areas in draped-cantilever cables suspected of not having been properly injected. Only 10 cables were examined. Borings and cores calibrated the observations. Although cable trajectories were plotted on the webs ahead of the tests, they were not always in their intended locations. Of the 10 cables inspected, only one known voided area was found. Georadar was unsuitable for use on the bottom slab, due to a rough top surface and because the PT tendons were inaccessible in the remote corner of web and bottom slab.

The typical result of georadar is a color contour plot, called a radargram, which allows interpretation of cover and integrity of grout. However, it requires calibration and skill to interpret the images. Complications arise from congested reinforcing bar, unknown embedded objects, honeycombing, variations in concrete composition, contact surfaces between old and repair concrete, uneven surface (air under antenna,) and the thickness of the cover layer, which affects the ideal placement of antenna. Georadar is a time-consuming process but is suitable for inspection if there is good knowledge of the structure and smooth surfaces to facilitate calibration. In addition, an experienced operator is needed. It offers possibilities for detailed investigations after inspection by other methods.

In France, magnetic perturbation is occasionally used on cable stays. X-rays have been used on anchors; however, they do not penetrate the steel of the anchor and can only penetrate about 400 mm of concrete.

Similar radiographic and magnetic flux methods have been attempted in the United States according to Mohsen Shahawy of the Florida DOT. The latter can only detect a significant loss of section or wire break. For internal tendons, magnetic flux has been attempted only for concrete thickness of less than 200 mm (8 in).

In France, some grout voids in a few early bridges were detected by radiography. However, it is not easy to detect broken strands and impossible to detect corrosion of strands. To verify the radiography, intrusive observations were made by excavating the concrete and removing pieces of duct. This cannot be done everywhere, because it is only feasible for local cases where the duct is near the surface, not in massive areas around anchors. Occasionally, an endoscope would be inserted near anchors to verify a grout void. Forces in PT strands were measured by prying strands sideways, with a small jack, and monitoring the deflection calibrated to similar deflections in laboratory control samples. All such methods are very intrusive; reliable non-invasive ways would be preferred, but few options exist.

Subsequent to the concerns about post-tensioned bridges by the Highways Agency in the UK, a 5-year special inspection program highlighted the need for non-destructive testing (NDT) methods for detecting voids and damage. Several tests were carried out at the Transport Research Laboratory (TRL) on beams purposely made with defects to investigate the effectiveness of various NDT methods.[49]

The techniques studied at TRL included the following:

- Radiography (x-ray with a 3 Curie cobalt 60 source) detected voids but could not detect corrosion.
- Impulse radar at 900 MHz and 1 GHz, using analog and digital systems, could not see beyond metal ducts, but can be used to detect metal. This technique is reliable and is often used to detect voids in ducts.
- Electrical reflectometry trials were disappointing (RIMT = Reflectometric Impulse Measurement Test, developed in Switzerland and Italy).
- Impact echo showed promise but needs further development to be reliable.
- Ultrasound might have potential for further development, but requires direct access to the anchors, which may be difficult in practice.
- Metal duct and tendon detectors were partially successful.

An important conclusion was that caution is needed with all the above methods, because they are susceptible to misapplication and misinterpretation.

INSTRUMENTATION

Instrumentation involves the use of various invasive and non-invasive techniques to gather data on structural performance. It involves strain measurement, fiber optics, measurement of chloride ion penetration, remote data logging, and intelligent structures.

In Germany, bridges are instrumented only if there is a special need, such as a defective bridge with no immediate means of repair. On the whole, German bridges are of smaller scale than long-span bridges in the United States and Japan, where instrumentation is more common.

Monitoring and instrumentation are pursued in Denmark to reduce inspection costs, obtain early warning of the onset of damage, corrosion, moisture damage, or the effects of temperature, and to improve overall knowledge of structure performance and durability under the operating traffic conditions. However, existing structures are limited to examination of cracks, materials, freeze-thaw effects, alkali-silica reaction, chloride and moisture ingress, carbonation, deflections, and effects such as vibrations and temperature that can be monitored with probes. Nevertheless, relatively realistic service-life models for the projection of deterioration are possible based on observations of existing, mature structures with a history of periodic inspection, along with sampling, testing, and probes. “Smart” monitoring systems can be introduced in new structures, such as Oresund, to provide actual feedback on performance and durability.

SEISMIC CONCERNS

Significant seismic conditions are not normally encountered in the northern European countries that the team visited. Seismic activity occurs in southern Europe, around the Mediterranean.

Although seismic conditions are not normally considered in Denmark, because of the magnitude of the project, they were examined for the Storbaelt Bridge, but were found not to govern the design. Consequently, within the purview of the countries visited, there is no direct experience, opinions, or research with the use of grouted or ungrouted tendons, or the performance of cable-stayed bridges, under seismic conditions.

Part E: Findings and Conclusions

A significant finding of the scan is that the majority of the problems with the early-generation segmental and cable-stayed bridges reported to the scan team were those that had already become known to the engineering community through previous scans, publications, or revised codes. One example is excessive long-term creep deflections in long-span cantilevers, particularly those with midspan hinges. Such problems have been solved by the addition of external, supplementary post-tensioning. Repeat difficulties have been avoided through new codes and procedures introduced in the 1970s (e.g., CEB-FIP 1978 and 1990) and carried forward into later guides such as the *AASHTO Guide Specification for Segmental Bridges*. Consequently, repeat problems, requiring strengthening by supplementary, external post-tensioning, have since been avoided both in Europe and the United States.

Despite serious accidental flash-flood scour undermining a post-tensioned concrete box girder viaduct in Switzerland, the structure remained intact and was repaired, which clearly demonstrated the inherent redundancy of modern post-tensioned construction.

Recent issues reported to the scan team concerned corrosion of reinforcing-steel and post-tensioning tendons in bridges of all types — not solely segmental or cable-stayed bridges. The majority of corrosion cases arise through the penetration of corrosive agents (usually chlorides from de-icing salts or seawater) into the concrete and, particularly, into inadequately consolidated (honeycombed) concrete or voids in tendon grout. In all such cases, improved detailing and stricter quality control at the time of construction would have prevented the damage. Nevertheless, the number of serious cases is small (typically fewer than 2 percent of all post-tensioned bridges), and all authorities have active maintenance plans and solutions. New practices, guides, training programs, techniques, and procedures are being introduced to improve or solve these issues for new construction.

The majority of European countries apply waterproof membranes and protective overlays to bridge decks of all types. They consider this essential for enhanced durability and attribute the prevention of more serious chloride penetration and corrosion to this practice. In practices similar to those in the United States, concrete is air-entrained and is usually enhanced by the use of fly-ash, micro-silica, or other additives to improve durability. Concrete surfaces exposed to severe environmental conditions (faces of barriers, tidal splash zones, etc.) are mostly sealed with penetrating sealers. Visible surfaces subject to corrosive agents are sometimes coated with epoxy- or polyurethane-based paints.

The use of de-icing salts continues throughout Europe. However, reduced or regulated quantities or new materials (such as CMA) have been introduced or tried in some areas.

New, more "robust" post-tensioning ducts and, occasionally, sealed or isolated PT systems are being introduced. New, greased-and-sheathed mono-strand systems are popular for external tendons, for new construction, retrofits, and strands for cable stays. New standards and codes are being developed for post-tensioning and cable stays, similar to measures in the United States, through the PTI, for the grouting of tendons and cable-stay design and construction.

Most countries have or are introducing formal maintenance inspection programs for all bridges, coordinated through a central office or database BMS. Current inspection methods and periods vary but are somewhat similar from country to country. The impression of the scan team is that there appears to be more intense interest and commitment to long-term durability and, particularly, maintenance of structures in Europe than for comparable structures in the United States.

Some novel methods of bridge-load rating using safety-based and/or reliability methods are being applied in Denmark to provide safe but reliable and cost-effective alternatives to demolition, rehabilitation, or reconstruction of older bridge stock. The technology arose from insurance needs for very large projects (airports, offshore platforms, etc.).

Emerging materials, such as carbon- and glass-fiber, are finding similar application in Europe as in the United States. They are mostly for strengthening, repair, or retrofit of existing structures, where the application of a bonding resin and fiber mat is easier than traditional methods, involving, for example, drilling and doweling for rebar. To date, only experimental projects have been built using new composite materials.

All countries that the team visited have tried various non-destructive inspection methods (as in the United States). However, there is no simple and reliable method — no “magic bullet” — for inspecting either post-tensioning tendons or cable stays. Certain parts of post-tensioned and cable-stayed bridges, such as anchors, for example, are usually embedded in a mass of concrete with much reinforcing and other items that make it difficult to examine using non-destructive equipment and sensing methods. Nevertheless, many techniques are available; each can be useful when applied appropriately. For the most part, non-destructive methods are used as a second-phase approach to supplement routine inspection by other normal (visual) means.

In general, segmental and cable-stayed bridges in Europe are performing well and there are no insuperable problems. Some areas, such as installation of grout, are being improved to avoid corrosion. More rational, scientific, and reliable methods of evaluation and systematic maintenance are being introduced. Progress has been made toward more durable (long life) structures through enhanced concrete materials, construction methods, and corrosion prevention technology.

Generally, precast and cast-in-place segmental and cable-stayed are proven ways to build modern bridges; they are efficient, economical, competitive, practical, and aesthetic, and most designs provide considerable redundancy against extreme or unforeseen events. The Europeans continue to design and build these structures.

There is a need for a continuing review process to keep abreast of developments and track performance. With the insights from this scan tour, the team recommends that similar scans and exchanges of knowledge be made on a regular basis, preferably at intervals not exceeding once per decade.

IMPLEMENTATION OF FINDINGS

Distribution of the report should be made through the following organizations and publications:

ASBI	Newsletter/Conference/Seminars (Cliff Freyermuth)
AASHTO	T10 Bridge Committee and Conference Spring 2000
PTI	Notes/Meetings (Jerry McGuire)
PCI	Journal (George Nasser)/Meetings (John Dick)
TRB	Annual Meeting/5th International Bridge Conference, Tampa 2000
ACI	
ASCE	
ARTBA	
NW Bridge Seminar	
FIB	
IABSE	

Possible magazine articles or notices:
- Civil Engineering Magazine - ASCE (Virginia Fairweather)
- Concrete International
- ASCE Bridge Journal (Dennis Mertz)
- Bridges Magazine - UK - Helena Russell
- Engineering News Record
- Journal of IABSE (Zurich)

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2. "Weltmonument Salginatobelbrucke saniert" Tiefbauamt Graubunden, Die Weiterverwendung von Bild und Text mit Quelenangabe ist erwunscht. Weitere Exemplare erhaltlich unter Tel. 081-257-3715.

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Appendix A: Switzerland

BRIEF FACTS

Switzerland has a population of 7.3 million, a GDP of US$172.4 billion, and is 41,290 sq. km (15,940 sq. mi). It is divided into three areas: the Jura, the Plateau, and the Alps. The Jura is the northern, lower-elevation countryside. The Plateau is the middle area, characterized by urban zones and major highways. In the Alps, there are many smaller bridges and retaining walls. Swiss government includes 26 Cantons, roughly equivalent to U.S. States. There are approximately 71,000 km (44,000 mi) of various types of roads: 1,590 km (990 mi) are national roads (Interstate type) and 2,200 km are (1400 mi) other main roads.

The Federal Roads Board is responsible for all aspects of design, construction, and maintenance of the national and main roads and provides 65 to 90 percent of the funding; the balance comes from the individual Cantons. Cantons submit plans and tenders to the Board for approval, appoint consultants for planning and design, and administer contracts. Design competitions with jury selection are used for projects where aesthetics and local interests are important. Large projects are often let as a design and construction competition, again, with a jury selection process.

The responsibilities of the Bridge Section of the Federal Roads Board include the following:

- Approval of projects and approval concepts for projects.
- Approval of measures for maintenance.
- Formulation of technical guidelines.
- Initiation of research.
- Promotion or introduction of the results of research in practice.
- Coordination between Cantons.
- Management and working groups.

The Cantons develop projects and concepts and are responsible for inspection, maintenance, and inventory.

The approval of concepts for bridge structures by the Board involves the following steps:

1. The roadway width and decision on single or double structures.
2. Establishing the robustness, i.e., minimum thickness for typical cross sections, boxes, beams, full slab superstructures, and the use of monolithic connections.
3. Provision of access for maintenance, such as corridors or chambers behind expansion joints at abutments.
4. Establishing the durability characteristics to meet a target life of at least 80 years.

SUNNIBERG BRIDGE

Sunniberg Bridge (figure 5) has five cable-stayed spans up to 140 m (460 ft); it is situated about 62 m (200 ft) above the bottom of a rugged valley. The bridge is curved in plan to a radius of 503 m (1650 ft). Very tall but slender piers, extending to short pylons above the deck, are fully monolithic with the superstructure, and the deck is monolithic with the end abutments. Consequently, all movement is accommodated by lateral displacement of the curved deck on the flexible piers.

The pylons, which rise 14 to 16 m (46 to 53 ft) above the deck, incline to the outside to provide clearance from the stays for the curved roadway. The piers and pylons have a complex section that varies in size with height and reflects the shape of the moment diagrams. Forming required special rectangular elements with insert pieces that could be shifted to change the section. The deck is a slab with relatively slender edge beams and was built in balanced cantilever, in segments of 4 m, using form travelers.

There is a plane of stays on each side of the deck at each pylon. The statical scheme provides considerable redundancy. Calculations were made using a three-dimensional computer model of finite elements with linear elastic properties. The lateral deflection forces, due to curvature of the deck, produce large lateral bending moments in the lower parts of the piers. Vertical deflections under live load were to be limited to 1/400th of the span. The assumed live load consists of a distributed load of 2 kN/M^2 and a concentrated load of 360 kN for which the maximum deflection was 1/600th of the span.

The bridge was entirely cast-in-place; the superstructure was built in cantilever using form travelers. Site-produced concrete attained a 28-day strength of 64 MPa (9300 lbf/in^2) compared with a required 43 MPa (6200 lbf/in^2). The bridge was completed in two and a half years at a cost of Sfr20 million (US$14.3 million).

Appendix B: Germany

BRIEF FACTS

The population of the united Germany is about 82 million (1998), and the GDP is approximately US$1.74 trillion. There are 16 individual states, and Germany borders 9 European countries. The land area is approximately 357,000 sq. km (138,000 sq. mi), and the total transport infrastructure is estimated to cover about 5 percent of the land.

As of 1998, there are 35,272 bridges on federal roads in Germany and about 2.3 times this number throughout the country. By plan area, the vast majority of the federal stock (70 percent) is prestressed concrete, with a smaller number of reinforced concrete (19 percent), some steel (7 percent), and composite structures (4 percent). The current replacement cost of the federal bridges is estimated at approximately US$50 billion.

The Bundesanstalt fur Strassenwesen (BASt) is the Federal Highway Research Institute and has six departments:

- Administrative Services
- Behavior and Safety
- Traffic Engineering
- Automotive Engineering
- Highway Construction Technology
- Bridges and Structural Technology

There are four sections under Bridges and Structural Technology:

- Concrete Structures
- Steel Structures and Corrosion Protection
- Tunnels and Structural Foundations
- Maintenance of Engineering Structures

BASt provides support to the Federal Ministry of Transport, participates in about 100 European or national committees on codes and regulations, and performs extensive research for the Federal Ministry of Transport, Building and Housing. About 60 research projects are currently in progress by BASt personnel.

Appendix C: Denmark

BRIEF FACTS

Denmark's population is 5.25 million. There are 14 counties and 275 municipalities comprising an area of 43,094 sq km (16,638 sq mi). The highest point is 173 m (567ft) above sea level. With 406 islands nestled between the entrance to the Baltic from the North Sea, ferries are essential, and Denmark has a long history as a seafaring and trading nation.

There is a total of 71,560 km (44,465 mi) of roads, divided into three types:

- State 1650 km (1025 mi)
- Counties 9900 km (6150 mi)
- Municipalities 59,950 km (37,250 mi)

The roadway budget for 1999 was Dkr1.9 billion (US$284 million).

The Danish Road Directorate is part of the Ministry of Transport. The Directorate has responsibility for the following areas:

- 45 major bridges, worth US$1.4 billion; plan area of 465,000 m^2 (5,003,000 ft^2).
- 1,295 minor bridges, worth US$1.2 billion; plan area of 760,000 m^2 (8,176,000 ft^2).

Minor bridges are those with a length between 2.0 m and 200 m (7 ft and 656 ft). Typical highway overpasses are sloped-leg, filled frames, and underpasses have three spans. Major bridges are concrete and steel structures more than 200 m long and include the very large Storbaelt and Oresund crossings. The Storbaelt connection is currently carrying 18,000 vehicles per day, compared with the estimate of 11,000. The Oresund connection to Sweden is due for completion in 2000.

The tasks of the Road Directorate include the administration of roads, construction, plans, maintenance, road safety, services, and filling stations. The Export Division of the Directorate operates, worldwide, to promote the BMS.

SALLINGSUND BRIDGE

Sallingsund is 1,700 m (5,610 ft) long and is situated across a North Sea estuary. There are 17 spans of 93 m (305 ft) and two end spans of 51 m. The overall width is 16 m (52.5 ft), which provides a single lane in each direction, with shoulders and sidewalks. Two of the main, 93-m spans provide separate navigation channels with a clear width of 60 m (197 ft) and clear height of 26 m (85 ft). The bridge was constructed between 1973 and 1978. Recently, the main problem has been the risk of impact from sailing vessels, because current vessels are three times larger than estimated at the time the bridge was designed.

A significant feature of this structure is that the foundations for the deep-water piers were made of very large, precast, truncated conical shell segments that rest on a precast template in the seabed, through which piles were driven from the surface. The conical base is topped, at water level, with a special, elongated hexagonal precast icebreaker shell made solid with reinforced concrete. Hexagonal-section, cast-in-place pier columns of 3-m (10-ft) lifts rise from the icebreaker bases. (This bridge was the forerunner for the much larger Confederation Bridge across the Northumberland Strait, in Prince Edward Island, Canada.[6]

Over water, the piers are monolithically fixed to the superstructure, but in the last three spans over land, the superstructure is on neoprene bearings. There is a dapped hinge at the quarter point of every other span, fitted with Maurer expansion joints. During erection, the hinges required temporary locking, until the cantilever was completed. Split pier segments were used to reduce size and weight for erection. These were erected on temporary steel bearings, adjusted to elevation, stressed to each other, and secured with vertical PT bars. The gap between the pier and pier segments was temporarily grouted until final jacking and adjustment of the cantilevers to relieve some erection effects, and then finally fixed. Fixed connections are needed for resistance to vessel impact.

The superstructure was erected using a self-launching erection gantry. All the original longitudinal post-tensioning tendons are internal and arranged in families of cantilever tendons, over the piers, and families of top- and bottom-continuity tendons, through the midspan regions. During construction, in the first two spans, friction was higher than expected, due to some tangential discontinuity at the joints. Some external tendons were added in these two spans to compensate for the loss of prestressing force.

The original design and tender for Sallingsund was based on cast-in-place cantilever construction, using the Dywidag Systems International (DSI) post-tensioning system. The constructed alternative, by Campernon Bernard, used precast segments. It was selected because of the innovative deep-water foundation system and because it was the most economical bid. However, there were concerns about the watertight integrity of the proposed epoxy joints and grouting of the tendons. Consequently, significant attention was given to many details that contributed to durability.

VEJLEFJORD BRIDGE

Vejlefjord Bridge has a total length of 1,710 m (5,610 ft), made up of 110-m (361-ft) spans of cast-in-place, balanced cantilever construction, arranged in three continuous portions. Two vertical webs, 0.85 m (2 ft 9 in) thick, vary from 6 m (20 ft) deep at the piers, to 3 m (10 ft) at midspan. The width of 26 m (85 ft) provides for two lanes and shoulders in each direction, with additional access ways. Form travelers were used for construction of the balanced cantilevers, and a 105-m- (344-ft-) long temporary girder provided stability. The bridge was built from each end, simultaneously.

Vejlefjord was constructed for the Danish Road Directorate by a consortium of Monberg and Thorsen, Denmark; Dyckerhoff and Widmann, Germany; and A. Jespersen and Son, Denmark.

ALSSUND BRIDGE

The Alssund Bridge, near Sonderborg, was opened in 1981 (figure 14). The main span is 150 m (492 ft), with symmetrical side spans of 85 m (279 ft), and three approach spans of 60 m (197 ft) at each end. It is 16 m (52 ft 6 in) wide and provides two lanes, with small shoulders, in each direction. Two vertical webs, each 0.65 m (2 ft 1 in) thick, vary in depth from 8.0 m (26 ft 3 in) at the piers, to 2.9 m (9.5 ft) at midspan.

Alssund was designed by COWI for the Danish Road Directorate and was built by a consortium comprising C.G. Jensen, Denmark, and Skanska Cementgjuteriet, Sweden.

DURABILITY CONSIDERATIONS

Dr. Steen Rostam, Chief Engineer for COWI Concrete Technology, offered the following insight into the Danish approach to durable concrete, which was largely based on experience with recent large projects, such as Storbaelt and Oresund.

To ensure durable concrete structures, careful analysis of the potential causes of corrosion and concrete deterioration and a well-thought-out protection plan is recommended. Deterioration occurs due to the corrosion of rebar, alkali aggregate reaction, salt scaling, impact, and so on. The transport of corrosion products needs proper modeling; it is often ignored or not studied, since only visual observations have been made. Corrosion begins by a gradual initiation phase, involving the migration of chlorides, and increases in concentration up to a critical level that marks the onset of corrosion. The initial phase is then followed by rapid propagation of corrosion and visible damage. Similar findings have been reported from studies of bridges in the Florida Keys.[34] Chlorides act as a catalyst to corrosion and, therefore, are not consumed. To prevent damage, chloride concentration must always be kept below a critical threshold level.

Over the past 15 years, laboratory tests on slabs, at Florida DOT, indicated that calcium nitrite was effective. The problem is, however, that the amount of nitrite needed is proportional to the chloride concentration. So, over time, nitrite is consumed or migrates out of the concrete. Dr. Rostam's opinion is that the calcium nitrite would migrate out of the concrete before chloride concentrations reach critical levels.

Quality of construction is governed by site activity and quality-assurance procedures. Poor quality will not be seen in the post-construction liability period, because it takes years for visible signs of corrosion to become evident. Consequently, a construction handover "birth certificate" should be developed, including a thorough mapping of concrete cover and initial chloride penetration levels, for future reference and comparison. Subsequent periodic monitoring would indicate the likely onset, or not, of critical-threshold levels, enabling improved prognoses and maintenance plans.

The outer 10 mm (0.4 in) of concrete surfaces tend to contain less coarse aggregate and may not be as well consolidated, leading to increased permeability to chlorides. Proper use of form liners can improve surface impermeability.

Cracking of concrete is a natural phenomenon that should be accepted, but controlled, through appropriate measures. Also, concrete properties change with time and must be factored in. To ensure enhanced durability, the following are recommended:

- Provide full bituminous tanking of substructures in soils.
- Apply coatings and sealers.
- Be selective with materials and avoid aggressive or reactive components.
- Use low-alkali cements.
- Use of reinforcement coatings, such as epoxy-coated rebar, is not recommended.
- Use of stainless steel for critical areas such as barriers is economical because of price and improved availability.
- Use cathodic protection.
- Design and detail the structure and materials to prevent corrosion.

In general, it is necessary to identify the environment and aggressive chemicals and design a specific multiple barrier protection system, such as for the Oresund Tunnel where a 100-year service life was required. The multiple barrier system included the following features:

- An outer ground injected grout with high percentage of fly-ash (a partial barrier).
- A 400-mm (16-in) lining material of 0.35 w/c, micro-silica, dense concrete.
- Epoxy-coated reinforcement, but only of fully fabricated and welded cages, coated by the fluidized bed technique. (During the same period, epoxy coating was still favored in the United States. However, subsequent findings might have changed the approach adopted for Oresund.)
- With a welded reinforcing cage for electrical continuity, cathodic protection can be installed in the future.
- A total of 480 corrosion sensors were installed in the concrete, both inside and outside cover.

For the Oresund crossing, the bridge superstructure was constructed of full precast cantilevers (similar to the Confederation Bridge in Prince Edward Island), erected using the Svannen floating crane. Expansion joints are at midspan, a decision made by the designer/builder. The piers and pylon shafts were slip formed using actively moving forms. The piers suffered high microcracking from a very thixotropic mix; 15 m (49 ft) of one pylon column had to be demolished and re-cast. Full-scale tests for different slip-forming parameters were made, but it was found that some microcracking of slip-formed concrete is unavoidable. Preventive cathodic protection, using sacrificial zinc anodes, was added to precast caisson foundations.

Dr. Rostam also emphasized the worldwide need for sustainable and environmentally friendly construction. In particular, he pointed out that carbon dioxide from cement production is enormous and an environmental hazard. It is essential to adopt lower cement contents. In this regard, the pursuit of HPC is not environmentally friendly, in addition to its having other problems associated with workability and site control. Therefore, it is necessary to consider adapting concrete to both the project and environmental needs. Then, the reliability of various corrosion-enhancement methods must be evaluated, on a systematic basis, for the whole project.

(For further information, see Reference 35, "Durability of Concrete Structures," Workshop report for CEB-RILEM, May 1983, by Dr. Steen Rostam, editor, Technical University of Denmark, Copenhagen.)

ORESUND BRIDGE

The Oresund Link extends from Copenhagen, Denmark, to Malmo, Sweden, and is a combination bridge-tunnel. There is about 15 m (50 ft) depth of water over an immersed tube tunnel, an artificial island, and a cable-stayed bridge with a 490-m (1,608-ft) main span, comprising a double deck of combined road and rail. It was constructed under a design-build contract of 4.5 years' duration. Its features also include the following:

- An immersed tunnel of two rail tracks and two highway sections.
- Prefabricated, precast caissons and pier shafts fabricated on land and erected by the Svannen floating crane. The precast caissons are rectangular in plan, but otherwise similar to Confederation Bridge in Prince Edward Island.
- Superstructure steel truss of 20-m lengths.
- The road is on the top concrete deck, the rail on the bottom, in concrete troughs on steel cross-frames.
- Steel trusses, of grade 480 N/mm^2 steel, were fabricated in Spain by Dregarros.
- Full-approach spans and portions of main span were erected in 140-m lengths, by the Svannen floating crane.
- There is 1,260 m (4,134 ft) between expansion joints. (By contrast, on the Storbaelt, there is 3 km between expansion joints.
- It was completed on time and within budget!

SAFETY-BASED BMS

The technique involves the following phases:

1. Fact finding from previous inspections and analyses.
2. Formulation of the task in close cooperation with the owner (Road Directorate).
3. Identify safety requirements for the bridge.

4. Develop deterministic models for failure:
   - Identify critical failure modes.
   - Main girders, moment, and shear conditions for failure.
   - Moment or shear failure of transverse members, ribs, slabs.
   - Consider substructure columns, caps, connections, etc.
   - Identify the most loaded areas.
   - Identify areas of heavy deterioration.
   - Identify critical combinations of failure modes and deterioration to determine critical points in the structure to analyze in detail.
5. Develop a probability-based safety-model for critical failure modes.
6. Model the stochastic variables:
   - Loads: consider the total load, amplification factors, and distribution.
   - Strengths of concrete, reinforcement, tendons, etc.
     – Modeling uncertainties related to the model itself and response used.
     – Predict deterioration, based on previous inspections or repairs.
7. Calculate the safety of the non-deteriorated bridge:
8. Calculate the reliability index for critical failure modes.
9. Perform sensitivity analysis.
10. Calculate the safety taking deterioration into account:
    - The model must be able to take into account deterioration of individual reinforcement, areas of concrete, and tendons.
    - Develop the model to optimize capacity by redistribution of internal forces.
    - Make a sensitivity analysis to determine important parameters and reinforcement groups.
11. Analyze various repair and rehabilitation options to be used when deciding on the optimal management plan, which may involve:
    - Traffic weight restrictions.
    - Repair and strengthening options.
    - Improvement of information, such as:
      – More inspection; extended routine, or special inspection.
      – Load testing.
      – Determine actual weight of bridge.
      – Install monitoring.
      – Develop more advanced analysis and response models.

- Make technical and financial comparisons of options.

12. Determine the requirements for visual appearance in cooperation with owner.

13. Make a cost-optimal management plan using the decision analyses to choose between several rehabilitation options. This might involve:

    - More inspection and updating of deterioration models.
    - When to make minimal or essential repairs.
    - Install waterproofing, improved drainage.
    - Continue routine and special inspections.
    - Make projections of future needs, lifetime costs versus new bridge, etc.

    Within the study, various possible repair and rehabilitation options are considered and evaluated. The costs and the effect of taking various actions are then estimated so that appropriate decisions can be made. Major advantages of the probabilistic approach are savings and proper management decisions, including future inspection requirements. The greatest uncertainty is in the projection of the rate of deterioration. At present, this is somewhat subjective, because it depends on the experience and opinion of the observer. Over time, as information is systematically accumulated, a rational basis will develop.

Appendix D: Norway

BRIEF FACTS

Much of Norway is covered by mountains, and the western coast is gouged by deep fjords and dotted with islands. To facilitate modern communications, many roads, bridges, and tunnels have been built by the Norwegian Road Directorate. The Directorate oversees the work of 19 County Road Offices throughout the country. There are approximately 15,000 bridges (with spans over 3 m). The total annual budget is Nkr11 billion (US$1.65 billion), of which Nkr205 million (US$31 million) is for bridge maintenance. The Bridge Department employs 50 people, with primary responsibilities for approval, planning, design and construction, research and development, maintenance, and advice. The Department has adopted the Brutus BMS for maintenance operations.

The Directorate establishes regulations for codes, loads, standardization, cables for suspension and cable-stay bridges, and is involved in research and development into chloride-resistant concrete, non-metallic reinforcement, high-tensile steels, and methods of maintenance.

Additional information may be obtained from the Norwegian Public Roads Administration, International Division, P.O. Box 8142, Dep. N-0033 Oslo, Norway. Tel: +47 22 07 39 00 – Fax +47 22 07 32 65.

Appendix E: France

BRIEF FACTS

France has a population of 58.5 million, an area of 551,000 km^2 (212,700 sq mi), and a GDP of US$1.3 trillion. France is divided into 22 regions, with a total of 100 counties and 36,400 communes. Highway statistics are in the table below.

	Kilometers	Miles	Vehicles/Day	No. of Bridges
Motorways	7,900	4,900	29,410	8,000
National Roads	24,000	14,900	29,000	20,000
County Roads	361,200	224,400	1,300	86,000
Local Roads	578,900	360,000	150	120,000

The national network of roads is administered and maintained by the Directorate of Roads through 22 Regional Public Works Directorates. The work of the Regional Directorates involves planning, concepts, maintenance, construction, operations, engineering, and safety.

Scientific, engineering, and technical services are provided through seven nationwide areas headed by Service d'Etudes Techniques des Routes et Autoroutes (SETRA), Technical Service for Roads and Motorways. The primary roles of SETRA are to:

- Develop and provide technical knowledge for structures and roads.
- Act as a central service to the decentralized regions.
- Manage the various regional Technical Centers.

The central office of SETRA is primarily responsible for:

- Methodologies, software, design and checks, construction oversight, research.
- Standards, technical approvals, and regulations.
- Advice, appraisals, technical assessments for the Directorate.

CHEVIRE BRIDGE

The Chevire Bridge, at Nantes, over the Loire River, provides 55 m clearance for navigation and includes a variable-depth concrete cantilever, projecting 40 m into the main span, with a 162-m suspended, orthotropic, steel box section span, 25 m wide. The two-web concrete box section has webs 60 mm thick, to accommodate anchors and transverse ribs under the top slab, prestressed with twelve 15-mm strands (2.35-m spacing). Longitudinal post-tensioning comprises straight cantilever tendons with some internal draped tendon anchoring in the webs. Continuity tendons are external and draped. At the main cantilever, the tendons drape and anchor in the bottom of the dapped hinge bearing support to the suspended steel span. The concrete dapped bottom is 2.5 m deep and performs well (no cracks). All segments were cast in place except the dapped end segment; this was necessary for quality control of the heavy and complex reinforcement — it weighed 300 tons.

Provision has been made for 10 to 20 percent additional (future) tendons. All tendons are in PE sheaths. Constant diameter pipes and diablo-shaped steel tubes are used for tendon deviators.

RÉ ISLAND BRIDGE

The bridge to Ré Island is 3 km from the coast on the Atlantic and has 30 spans of 110 m made of precast cantilever segments, 15.5 m wide, with webs varying from 460 mm thick at the top to 360 mm at the bottom, to save weight. The spans have single point deviator segments. Prestressing is a mix of internal and external tendons. Some internal top tendons are anchored in top blisters at the web/slab junction. Most cantilever tendons are anchored in the top slab at the web. Continuity tendons are external of twelve and nineteen 0.6-mm strands. The external tendons are two spans long. The tendon grout includes silica fume, and double ducts are provided for possible future replacement tendons.

Amplifying Questions

The purpose of the visit is to observe the European inventory of the structure types, which have a longer in-service history than similar ones in the United States. In this manner, the panel hopes to determine what problems have surfaced with respect to accessibility for inspection, maintenance, repair and retrofit, long-term durability, demolition, and replace-ability. The purpose is to determine what solutions have been used and which ones were effective in resolving problems that have occurred. This may give U.S. practitioners some indication of what problems to anticipate. It may also provide technological solutions that might be imported into the United States, to solve similar problems that have or may occur in the U.S. inventory of similar structures.

I. Repair, retrofit, and maintenance problems associated with specific design, construction methods, and structural details of prestressed concrete segmental and cable-stayed bridges

1. Is there a comprehensive summary of the structural condition of segmental and cable-stayed bridges in your region or country and may we have a copy?
2. Have any construction methods or structural details been identified as being either detrimental to or contributing to long-term durability?
3. Has the need arisen for significant repair or retrofit of a segmental or cable-stayed bridge and, if so, for what reason? What repair or retrofit was made, and how well did it perform?
4. What has been the long-term performance of structural decks and overlays, and has there been need for deck replacement? If so, do you have replacement procedures that have been carried out. What were the results?
5. What types of sacrificial wearing surfaces have been used, how well have they performed, and what has been or is your maintenance policy?
6. Have any large bridge bearings been removed and replaced? What provisions have been made in designs for inspection and replacement, and are there any preferences for certain types of bearings?
7. Transversely, how have segmental bridge decks performed? Were they reinforced only with mild steel or were they transversely prestressed (by either pre- or post-tensioning)? Has there been any longitudinal cracking, and what were the allowable design service level tensile stresses in top slabs when built?
8. When transverse tendons have been used, has there been any evidence of corrosion, and what corrosion protection measures were or are taken to enhance durability? For example, what are the types of materials, ducts, components, grouts, additives, procedures, etc.?
9. Have there been any local failures in transverse anchors in edge copings? How were they repaired? Was it effective, what have been the results, and have you implemented any new practices as a consequence?
10. Longitudinally, what has been the experience with internal and external post-tensioning tendons? (Internal tendons are those embedded in the

concrete and may be bonded or unbonded; external tendons are outside the concrete and unbonded.)

11. Have you found any significant difference in the long-term performance of internal (within the concrete) versus external (outside the concrete) post-tensioning tendons?
12. Has there been any cracking near anchors, deviator blocks, or diaphragms, and what action was taken to repair the damage or seal the cracks?
13. Has the filling of tendon anchor pockets, temporary block-outs, lifting holes, and so forth affected either the rideability or durability of the deck? (That is, have they allowed moisture ingress, leakage or corrosion?)
14. What jointing techniques have been used between precast segments, and what has been their durability and performance history? Has there been any evidence of deterioration, leaky joints, corrosion, or differences in performance among joint types? What has been the treatment and how well has it performed?
15. What types of tendons and tendon protection have been used in connection with different jointing techniques between segments, and how well have they performed?
16. As regards longitudinal tendons, what were your design parameters? Did they depend upon the type of joint between precast segments and environmental factors; i.e., epoxy or dry joints, aggressive or non-aggressive environment, allowable service level tensile stresses? Did these parameters include thermal gradients?
17. Has lightweight concrete been used to any extent in the construction of segmental and cable-stayed bridges, and what have been the results?
18. What methods have been used to treat structural and non-structural cracks?

II. Successful and unsuccessful methodologies developed to overcome identified problems

1. How often and to what depth of detail are maintenance inspections, condition assessments, or other performance checks conducted?
2. To what load level and how have load ratings been made? (In the United States, a load rating is the determination of the actual characteristic vehicle load level to which the structure can safely be loaded. It may be different from the specified design loads.)
3. Do you have generic or bridge-specific maintenance and inspection manuals for segmental and cable-stayed bridges? May we obtain copies?
4. Has additional post-tensioning been added to any structures for any reason, and what provisions have been or are made in new structures to facilitate additional post-tensioning to address possible future needs such as heavier loads, additional traffic lanes, etc.?
5. What considerations have influenced decisions to use internal or external tendons; i.e., considerations of durability, ease of construction, inspection or

maintenance, type of corrosive environment etc.? What guidelines or recommendations regarding this decision have been developed and for what reason?

III. Enhanced or emerging technologies relating to inspection, maintenance, repair, or retrofitting of segmental concrete and cable-stayed bridges

1. What has been or is being done to enhance durability? For example, different types of cement, aggregates, use of fly-ash, blast furnace slag, micro-silica, corrosion inhibitors, such as calcium nitrite, other additives, applications of coatings to reinforcement (i.e., epoxy), prestressing strand or to concrete surfaces such as penetrating sealers etc.?
2. Have you used or do you recommend the installation of strain/load monitoring or warning systems and other remote sensing systems, or do you rely on visual inspection only?
3. What non-destructive evaluation (NDE) methods, types of instrumentation or remote sensing devices have been used to check, inspect, or monitor cable stays, post-tensioning tendons, or their components?
4. What has been your experience with repair technologies using artificial fibers, composite materials, bonding of steel or other materials with epoxy, or similar agents, and what have been the results?

IV. Performance of cable stays and post-tensioning tendons

1. Has there been any excessive vibration of cable stays? Has this been a concern, and what retrofit measures have been considered or used, and what is the current practice?
2. Have any cable stays been replaced and for what reason? How was this done and what provisions, if any, were or have been made for future stay replacement?
3. How have cable stays, anchors, saddles, components, and protection systems been inspected and checked for water tightness, corrosion protection, and integrity of the system?
4. What has been the performance, in general, of cable-stay anchors and saddles?

V. Corrosion protection methods for cable stays and post-tensioning tendons

1. What corrosion protection features or details have been used for cable stays? Do these vary for different primary tensioning elements (strands, bars, wires). What has been their performance history?
2. What features or details have been used, where cable stays penetrate the superstructure deck or pylon, that have led to corrosion or other problems? Do you have any preferences or recommendations?
3. What has been your experience with the corrosion protection of grouted post-tensioning tendons?

4. Have you experienced any grout voids or similar problems, and what has been done to fill, seal, and protect them? What policies have been put into place to prevent recurrence of problems?

5. Have any special additives, inhibitors, other grout materials, fillers, duct and anchor component materials, or details have been used to improve durability? What has been the performance?

VI. Methods for demolition in the event of functional obsolescence, structural deficiency, and methods of widening concrete segmental or cable stayed bridges

1. Have any segmental or cable-stayed bridges had to be demolished and for what reason? Was this because of structural deficiency or functional obsolescence? What techniques were used and how were they related to the erection method?

2. Have any segmental bridges been widened and what methods were used?

VII. Miscellaneous

1. If external tendons have been accepted or required, what has been done or experienced in regions of high seismic activity, if any?

Biographical Information

Walter Podolny, Jr., is a Senior Structural Engineer in the Office of Bridge Technology of the Federal Highway Administration (FHWA), in Washington, DC. In this position, he exercises managerial and technical responsibility for the review and approval of major, unusual, and complex fixed and moveable bridges, tunnels, and related structures. He has been with the FHWA for 28 years. Dr. Podolny is a graduate of Cleveland State University and holds a Master's degree in Civil Engineering from Case-Western Reserve University. He received his Ph.D. from the University of Pittsburgh. Dr. Podolny is a registered engineer in four States and is a Fellow of the American Society of Civil Engineers and the American Concrete Institute. He is also a member of the Precast/Prestressed Concrete Institute, the Post-Tensioning Institute, and the International Association for Bridge and Structural Engineering. Dr. Podolny retired from FHWA in 1999.

Randy Cox is a Structural Engineer for the Texas Department of Transportation in Austin, Texas. Mr. Cox currently serves as the Director of the Bridge Construction and Maintenance Branch of TxDOT's Construction Division and is involved with providing Statewide assistance regarding bridge construction, structural steel erection inspection, and bridge maintenance. In addition, Mr. Cox serves as a member of TxDOT's structural research committee and as an advisor to several on-going research projects on bridge durability. Prior to his current position, he was a bridge design engineer with TxDOT. Mr. Cox is a graduate of the University of Texas at Austin with a Bachelor of Science degree in Civil Engineering. He is a licenced professional engineer in Texas.

Douglas Edwards is the Bridge Management and Structures Engineer for the Florida Division Office of the Federal Highway Administration (FHWA). Mr. Edwards is currently the FHWA bridge specialist for design, construction, and inspection/maintenance issues in the State of Florida. Florida's segmental bridge inventory includes stacked interchange flyover structures, long marine water crossings in semi-tropic environments, and two large cable-stayed bridges. Prior assignments with the FHWA include similar duties in the States of Michigan and Illinois. Mr. Edwards is a graduate of Purdue University in Indiana. He is a licensed professional engineer in the State of Indiana and serves as an ex-officio member on the T-4 Technical Committee for Construction of the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures.

John M. Hooks is the team leader for deployment of innovative bridge technology in the FHWA Office of Bridge Technology, in Washington, DC. His expertise and experience are in identifying, evaluating, testing, and deploying bridge technology to the domestic U.S. bridge engineering community. He will be the person responsible for implementing promising technology identified in the bridge durability scan. In recent years, he has developed extensive documentation relating to bridge inspections, including for segmental bridges. These include the Bridge Inspector's Training Manual 90 and the report *Guidelines for Inspection Manuals for Segmental Concrete Bridges*. He received his BSCE in 1966 from Clarkson University and his MSCE (Structures) in 1968 from the same university.

He is a licensed professional engineer in the State of New York and a member of the American Society of Civil Engineers.

Majid Madani is a senior Bridge Engineer for the California Department of Transportation (CalTrans) in Sacramento, California. Mr. Madani currently directs the oversight engineering of the new Benicia Martinez bridge across Carqquniz Strait in California. His current assignment is establishing design criteria and engineering oversight for this structure, which is a lightweight, concrete-balanced, cantilever segmental bridge. As part of this project, he also monitors and directs the testing of precast segmental superstructure and pier-cap behavior in seismic events. Mr. Madani is a graduate of California State University, Sacramento, and holds a Bachelor's degree in Civil Engineering. He is a licensed professional engineer in California and serves as the chairman of the cast-in-place/prestressed, segmental construction for the California Department of Transportation, Division of Structures.

Maurice D. Miller is a bridge design engineer for HNTB Corporation in Kansas City, Missouri. He is currently semi-retired. Prior to his retirement, Mr. Miller was responsible for design of long-span bridges for the Kansas City Office of HNTB. Mr. Miller is a civil engineering graduate of Iowa State University, Ames, and holds a Master's degree in Structural Engineering from the University of Kansas, Lawrence. Mr. Miller is active in the American Segmental Bridge Institute and has authored a paper on the durability of segmental bridges in the United States. He is a licensed professional engineer in 12 States and a licenced structural engineer in two States.

R. Kent Montgomery is a Principal Bridge Engineer for Figg Bridge Engineers, Inc., at its Western Regional Office in Denver, Colorado. Mr. Montgomery provides technical direction for the Western Regional Office, which is currently involved in the design and construction of many segmental bridges, as well as other bridge types. Mr. Montgomery is a graduate of the University of Colorado and holds a Master's degree in Civil Engineering with an emphasis in structures. He is a licensed professional engineer in Colorado and a licensed structural engineer in Illinois and Massachusetts. He is a member of the American Segmental Bridge Institute (ASBI) and serves on a joint Precast/Prestressed Concrete Institute (PCI) - ASBI committee to develop standard superstructure and substructure segmental sections, as well as on an ASBI seismic research committee.

Alan J. Moreton is Principal Bridge Engineer for the Figg Engineering Group in Tallahassee, Florida. Mr Moreton has 30 years of experience in design and construction of concrete segmental and cable-stayed bridges. Prior to arriving in the United States, Mr Moreton was a bridge designer and Resident Engineer for the UK DoT. He graduated from Nottingham University, is a Chartered Engineer, and a Fellow of the Institution of Civil Engineers, UK. Mr. Moreton is a professional engineer in Florida and six other States and is a member of ASBI, AASHTO, the Post-Tensioning Institute, and the TRB Concrete Properties Committee.

Brett Pielstick is a Structural Engineer for Parsons Transportation Group (PTG) Construction Services in Daytona Beach, Florida. Mr. Pielstick is currently the

Resident Engineer for the Broadway Bridge project and serves as an Area Manager for PTG. The Broadway Bridge is a segmental bridge being built by balanced cantilever and will feature mosaic tiles around the elliptical columns. Previous assignments with PTG included Resident Engineer for the precast segmental Seabreeze Bridge and Assistant Resident Engineer for the Acosta Bridge in Jacksonville, Florida, featuring a 630-ft main span. Mr. Pielstick holds a Bachelor's of Science degree in Civil Engineering from Brigham Young University. Mr. Pielstick is the PTG representative for the American Segmental Bridge Institute, and he is also affiliated with the American Society of Civil Engineers, the Florida Engineering Society, the National Society of Professional Engineers, and the American Road & Transportation Builders Association.

Mohsen A. Shahawy is the Chief Structural Analyst for the Florida Department of Transportation, and he currently directs the FDOT Structures Research Center in Tallahassee, Florida. His duties include load capacity evaluation, field testing, load rating of existing bridges, and developing new design concepts. Since joining the FDOT in 1986, Dr. Shahawy has been involved with the design, construction, and load rating of Florida's major bridges, including the Sunshine Skyway Bridge. He is an expert in non-destructive strength and serviceability evaluation of prestressed concrete bridges and pioneered research on the use of composites in civil engineering applications. Dr. Shahawy is a graduate of Cairo University, Egypt, and holds a Master's and Ph.D. degrees in Structural Engineering from the University of Manitoba and Queen's University, Canada. He is a licensed professional engineer in Florida, Georgia, and Canada and serves on several technical committees of the American Concrete Institute (ACI), the Prestressed Concrete Institute (PCI), and the Transportation Research Board (TRB).

Dr. Man-Chung Tang is Chairman of the Board of T.Y. Lin International, Inc. He is a member of the National Academy of Engineers of the United States and an honorary member of the American Society of Civil Engineers. Dr. Tang has worked on the design or construction of many long-span, cable-stayed, and segmental bridges. Several of them included world-record spans at the time of their completion. Dr. Tang is a past president of the American Segmental Bridge Institute (ASBI) and former chairman of the ASCE Committee on Cable-Suspended Bridges. Dr. Tang has presented more than 100 technical papers on bridges and other major structures.

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