

Collapse of a Segmental Post-Tensioned Concrete Bridge

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This paper describes an investigation into the collapse of a single-span, segmental post-tensioned concrete bridge. The structure consisted of precast units stressed together both longitudinally and transversely. It was built in 1953 and collapsed at approximately 7:00 A.M. on December 4, 1985. There had been no evidence of distress before failure, and there was no traffic on the bridge when it collapsed. The investigation showed that the bridge had deteriorated because of corrosion of the tendons where they passed through the segmental joints. Corrosion had occurred because the tendons had been inadequately protected at the joints, and this had enabled chlorides to enter the ducts. It is believed that de-icing salts were the major source of chlorides, although it is possible that dune sand had been included in the mortar used to fill the transverse joints. The results cause concern about the condition of other post-tensioned concrete bridges in which tendons pass through joints.

Segmental post-tensioned concrete bridges are made of a number of units stressed together. The individual units may be either precast or cast in situ, and they are stressed together in such a way that the whole section remains in compression under both dead and live loading.

Since the 1940s there has been rapid growth in the construction of segmental post-tensioned concrete bridges. Some of the first bridges of this type built in the United Kingdom were made of small precast units. A number of footbridges and at least one highway bridge are known to have been constructed in this manner (1). Following the first application of segmental construction, various methods have been developed to enable longer spans to be built using both precast and cast-in-place segments (2). A large number of such bridges now exist on trunk roads and motorways in the United Kingdom and many more worldwide.

Segmental construction is popular for numerous reasons. It can be used for long-span structures, over long stretches of water or difficult terrain, and in congested urban areas where traffic disruption can be kept to a minimum. Structures can be produced to take up any required horizontal or vertical curvature, and the accuracy of the section and profile can be closely controlled during construction. It is possible to use the formwork repeatedly and thereby achieve a greater economy. Precast units can be produced under factory conditions, inspected, and if necessary rejected without seriously interfering with the progress of the work. They also have disad-

vantages, however. The materials have to be handled twice. There are additional costs in aligning and jointing the units. Special plants are often required for erection and handling, and construction stresses make the design calculations more complex. But these problems have not deterred bridge engineers from using this type of construction.

During the last decade there has been increasing concern about the ingress of chlorides into concrete bridge decks as a result of the application of de-icing salt, and the consequent risk of corrosion to the reinforcement. The joints between segments in segmental structures may be more vulnerable to the penetration of chlorides than dense, high-quality concrete; hence, these structures could be most at risk. It was assumed, however, that this presented a serviceability problem rather than one likely to cause sudden structural failure (3). This assumption was proved incorrect when a segmental, post-tensioned concrete bridge built in 1953 collapsed without warning early one morning in December 1985 (4).

This paper describes the bridge and its inspection history, gives an account of the investigations performed to determine the cause of failure, and discusses the implications for other structures of this type.

BRIDGE DETAILS

Location

The bridge was located in South Wales and carried an unclassified road over the River Afan approximately 6 km upstream from Port Talbot. It was situated at the bottom of a 1 in 6 hill, although until 1975 it had been separated from the hill by a humpback bridge over an unused railway line. Despite its low classification, the road is heavily trafficked and is on one of the local highway authority's primary winter gritting routes.

History

Most records on the bridge have been lost as a consequence of the local government reorganization in 1974. It was ascertained, however, that it was built in 1953 when it replaced a steel lattice girder bridge that was dropped into the river and used as falsework. Other surviving material included preliminary design calculations and a drawing showing the general arrangement of the structure.

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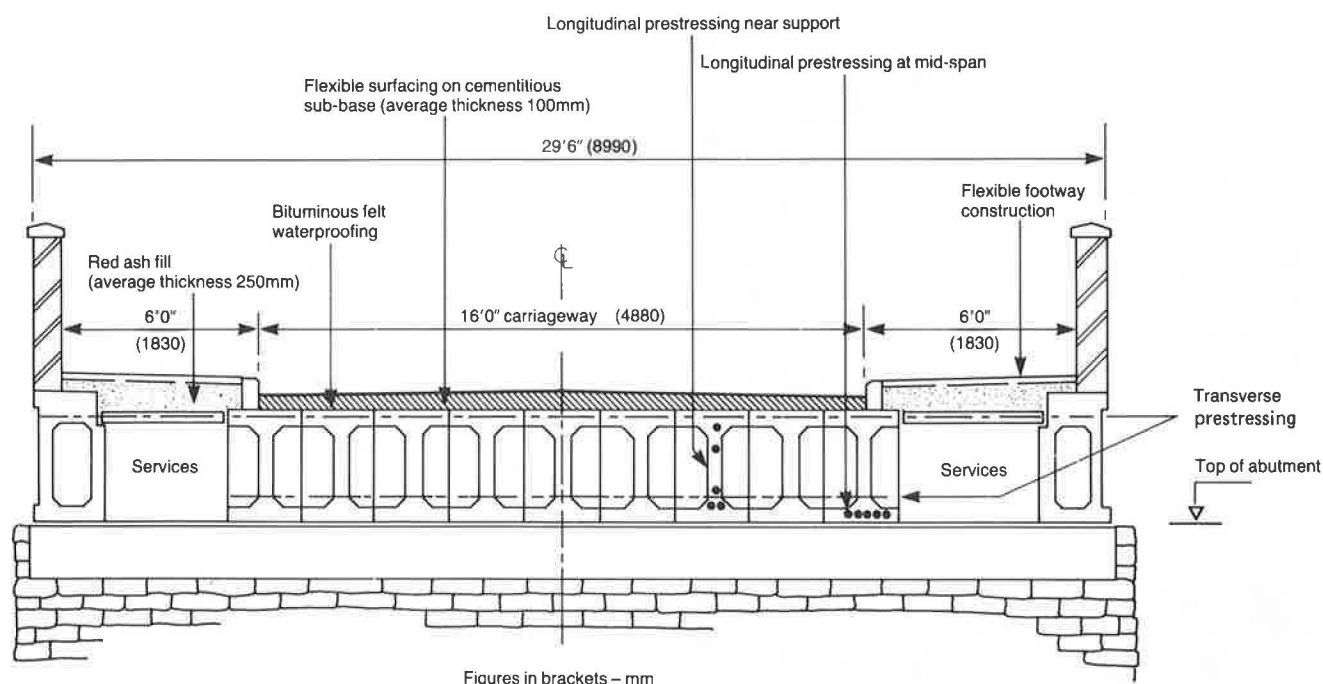


FIGURE 1 Cross section of Ynys-y-Gwas bridge (1953-85).

Description

The bridge had a single span of 60 ft (18.3 m) that sat on bearing paper on mass concrete abutments. It consisted of nine longitudinal I-beams, two box section edge beams, and two slabs 5 ft (1.52 m) wide that spanned the gap between the I-beams and the edge beams (see Figure 1). The I-beams carried an 18-ft (5.49-m) carriageway, and the slabs and edge beams carried 6-ft (1.83-m) sidewalks and parapet walls. There were service voids under the sidewalks between the outer I-beams and the edge beams.

The beams were made up of eight segments 7 ft 11 in. (2.41 m) long, post-tensioned together longitudinally with Freyssinet tendons consisting of twelve 0.2-in. (5-mm) diameter wires. The I-beams contained five tendons and the edge beams, ten. The slabs were also made up of eight segments but were not prestressed in the longitudinal direction. The ducts were unlined within the segments, and the drawings showed metal sleeves around the longitudinal tendons where they passed through the 1-in.-wide (25 mm) transverse joints. The joints were packed with mortar.

Transverse distribution of load had been provided by diaphragms 5 in. (127 mm) wide at one end of each segment and transverse post-tensioning, using the same type of Freyssinet tendons. The tendons ran through the top and bottom flanges of each I-beam at the diaphragms and near the anchorages, and through the top flange near the center of each segment. The lower tendons were anchored in the outer I-beams, whereas the upper tendons also passed through the slabs and were anchored in the edge beams. There was no filler in the longitudinal joint between the beams, although a white fibrous material had been packed around the transverse ducts where they passed through the joints, presumably to prevent leakage during grouting.

The running surface was made of a flexible surfacing and base course on a cementitious roadbase. Records showed that a bituminous felt waterproofing layer had been included during construction.

Inspection History

The bridge was not subject to any weight restrictions and had been regularly inspected. These inspections, which included an examination of the whole soffit, had not revealed any warning signs of distress, such as rust staining, cracking, or deflection of the beams.

The bridge was closely examined in 1981 when it was required to carry 50-tonne concrete beams, but nothing untoward was observed during the passage of nine of these beams. No attempt was made to confirm the internal integrity of the bridge nor to assess its load-carrying capacity, since outwardly it appeared entirely satisfactory. The loadings were arranged so as to ensure that they were not in excess of the maximum permitted HA loading. [HA loading is a formula loading representing normal traffic in the United Kingdom (5).]

Collapse

The deck collapsed when there was no traffic on the bridge; all nine internal beams failed (Figure 2). The line of failure is shown in Figure 3. The edge beams and blockwork parapets remained standing and apparently unaffected, despite the inward and downward forces exerted by the top transverse, prestressing tendons as the central deck collapsed. Some of the slabs that spanned the gap between the I-beams and the edge beams were left hanging from the edge beams.



FIGURE 2 Ynys-y-Gwas bridge after collapse (photographs courtesy West Glamorgan County Council).

INVESTIGATIONS ON THE CAUSE OF COLLAPSE

The bridge deck segments were carefully inspected after they had been lifted out of the river. It was evident that the prestressing tendons were severely corroded at both the longitudinal and transverse joints, indicating that this had been the cause of collapse. There was no evidence of rust staining that could be attributed to tendon corrosion on the external concrete and mortar.

The beams and segments were well aligned. A waterproofing system of felt sheets laid on bitumen was found under the cementitious roadbase. It is believed that this was the original system laid in 1953. All the prestressing tendons had been installed complete and in accordance with the drawings, except for the edge beams, where ten tendons were found but only nine had been shown. There was no evidence to suggest that the tendons had not been fully stressed, and the anchorages appeared sound.

The only intact members were the edge beams, which were in good condition. There was no evidence of transverse cracking, although longitudinal cracks, up to 0.5 mm wide in places, were observed along the line of some of the deflected ducts. The mortar cover over the transverse anchorages did not always fill the recess in the side of the beam, and where wires pro-

truded they were severely corroded. This was also the case for wires at the longitudinal anchorages at the eastern end of the downstream beam. Some rust staining was apparent on the inner face of one beam. It appeared to be due to tie wire near the surface and was no more serious than on many other structures. It would have been difficult to see during routine inspection as it was hidden behind service ducts. There was evidence of grout leakage on the surface of the concrete adjacent to where some of the transverse ducts crossed the joints between the beams and the slabs. It may have leaked out during construction and may account for the large voids found in some of these ducts. Evidence of packing was found at all the transverse prestressing points along both edge beams.

After the initial inspections had been made, some of the segments and the two edge beams were taken to the Transport and Road Research Laboratory (TRRL), where they were carefully broken up to expose the tendons where they crossed the joints. The tendons, grout, concrete, mortar, and longitudinal packing were examined. Samples were taken for analysis, including measurement of total chloride concentration using the extraction technique described in BS1881 (6). The remaining segments were taken to West Glamorgan County Council's Earlswood Depot, and samples were drilled for measurement of chloride concentration.

Duct Grouting

The degree of grouting in a sample of ducts is illustrated in Figures 4, 5, and 6. These results were obtained by:

- Viewing ducts at the failure section,
- Inspecting selected ducts along their length by drilling 15-mm diameter holes and inserting a borescope, and
- Examining ducts as the segments were broken up.

Vacuum/Pressure Technique for Examining Ducts

The vacuum/pressure technique for examining ducts was used. Void volumes and continuity of void in a duct were measured by drilling two 15-mm diameter holes into the top of the duct and applying a vacuum to each hole in turn. Continuity of void was recorded as the pressure drop measured at one hole as a vacuum was applied to the other.

Void volumes were measured by connecting the evacuated voids to a water gauge consisting of a perspex tube dipping in water. The volume of water drawn into the tube was measured and the volume of the void calculated. These measurements were repeated at each hole. Three sources of error were:

- The pressure within a partially grouted duct after it has been evacuated is not uniform.
- Leakages into ducts can occur through cracks and at joints.
- Only two holes were drilled into each duct, so there may be undiscovered voids.

All these errors lead to an underestimate of void volume; thus the values obtained will be only approximate.

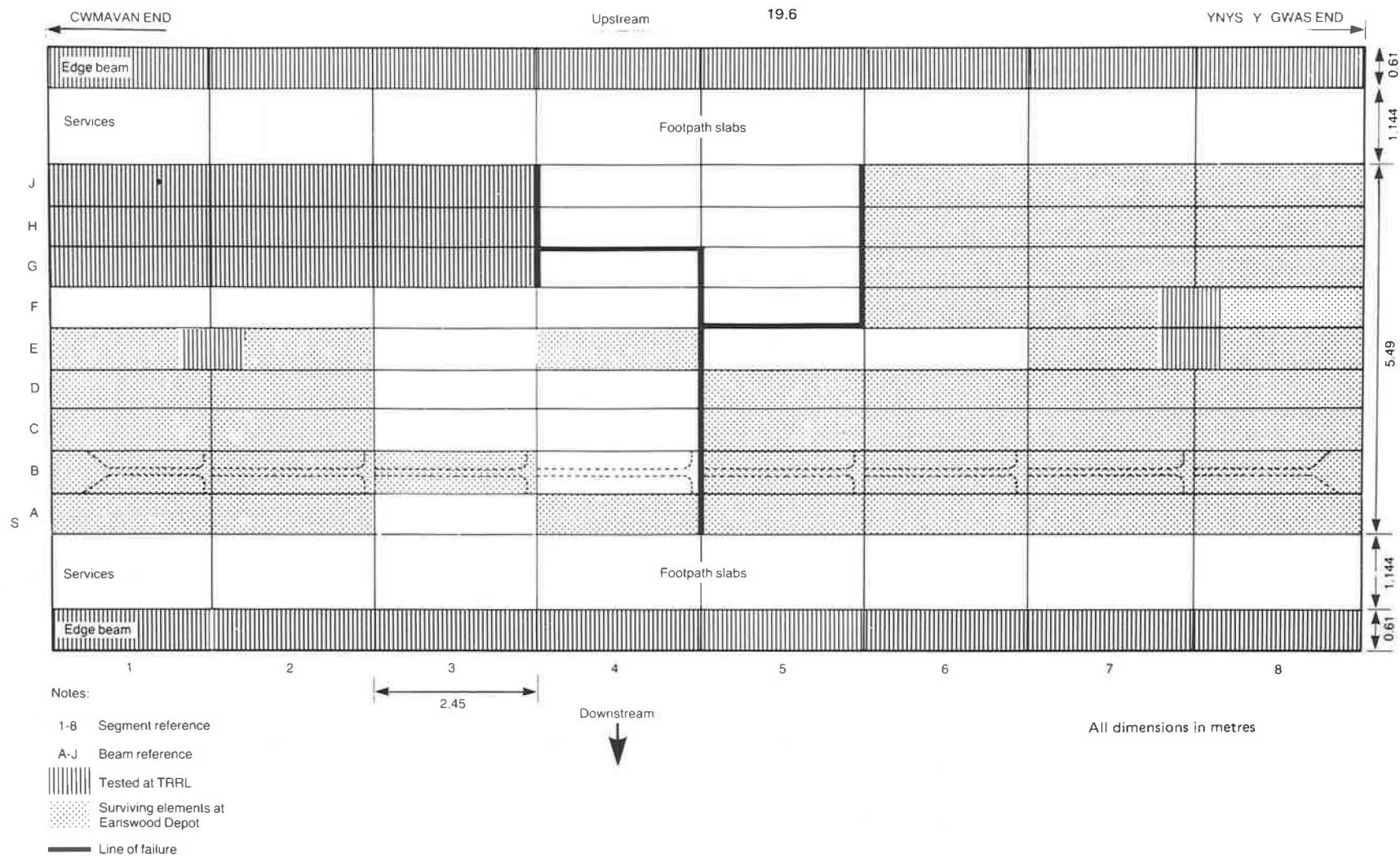


FIGURE 3 Ynys-y-Gwas bridge (1953-85) plan view of deck.

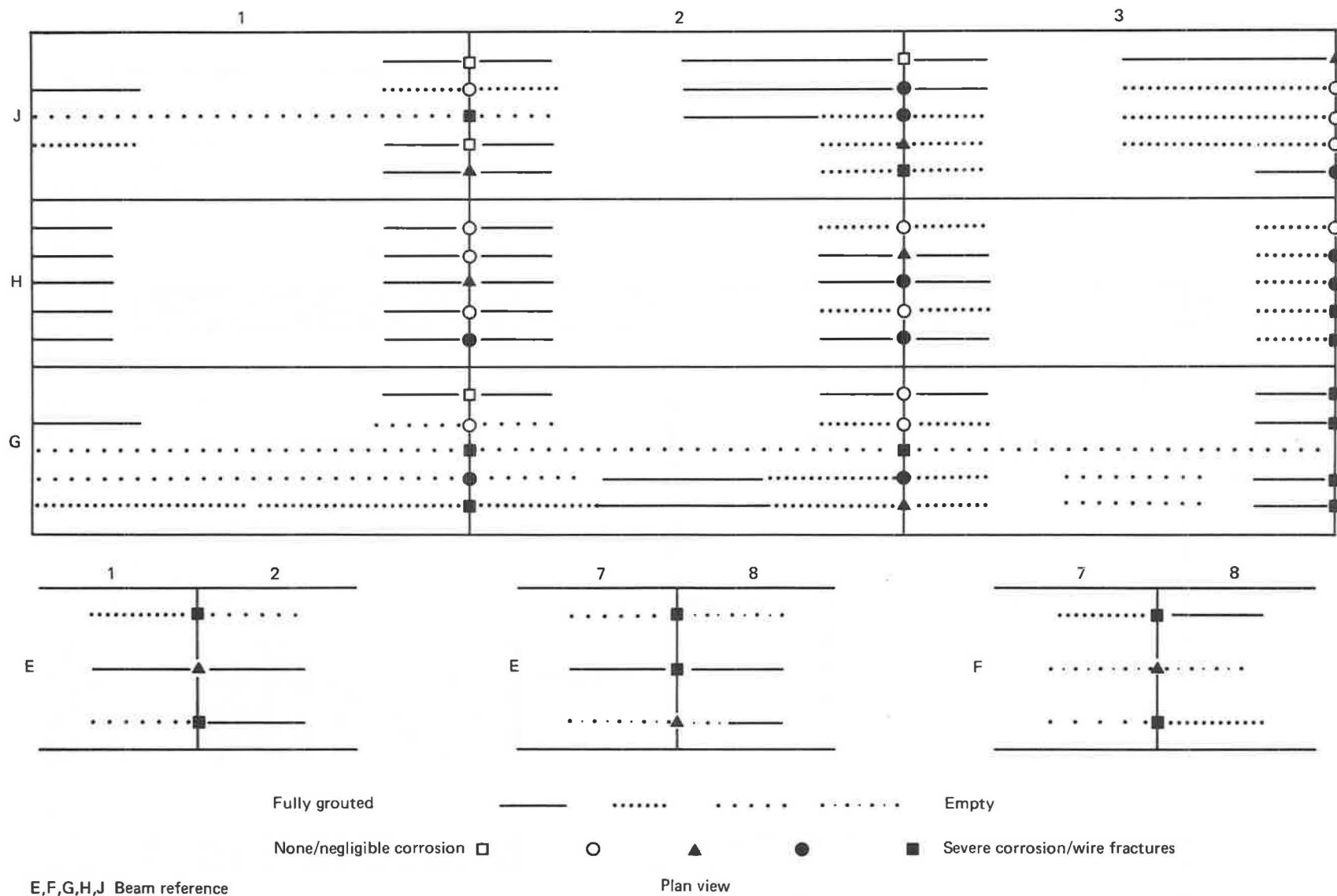
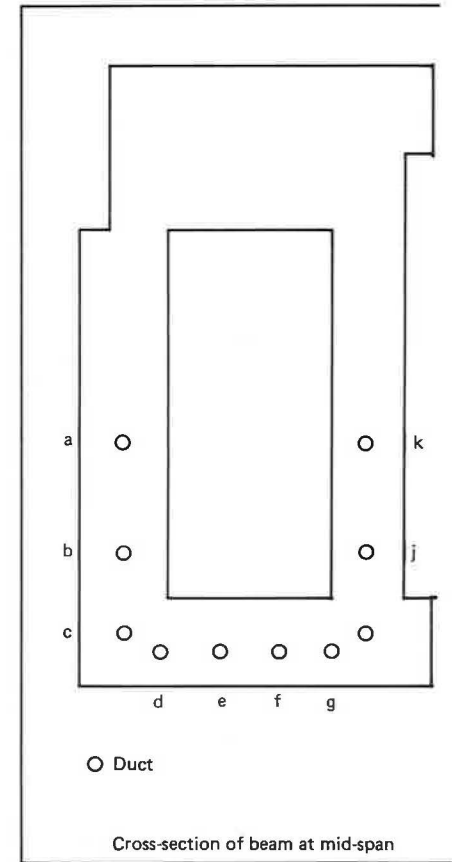
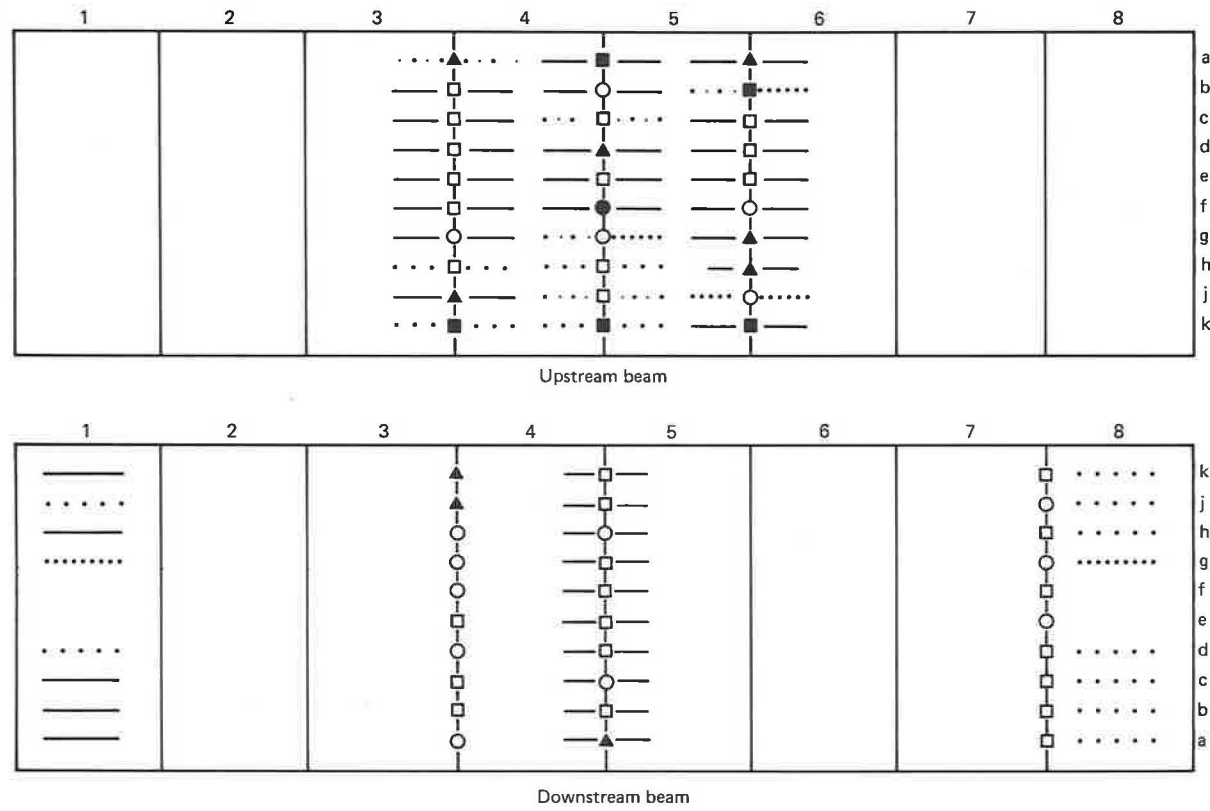


FIGURE 4 Condition of longitudinal tendons and grouting of ducts, I-beams.



1 - 8 Segment reference

Fully grouted



Empty

a - k Duct reference

None/negligible corrosion □



■ Severe corrosion/wire fractures

FIGURE 5 Condition of tendons and grouting of ducts, edge beams.

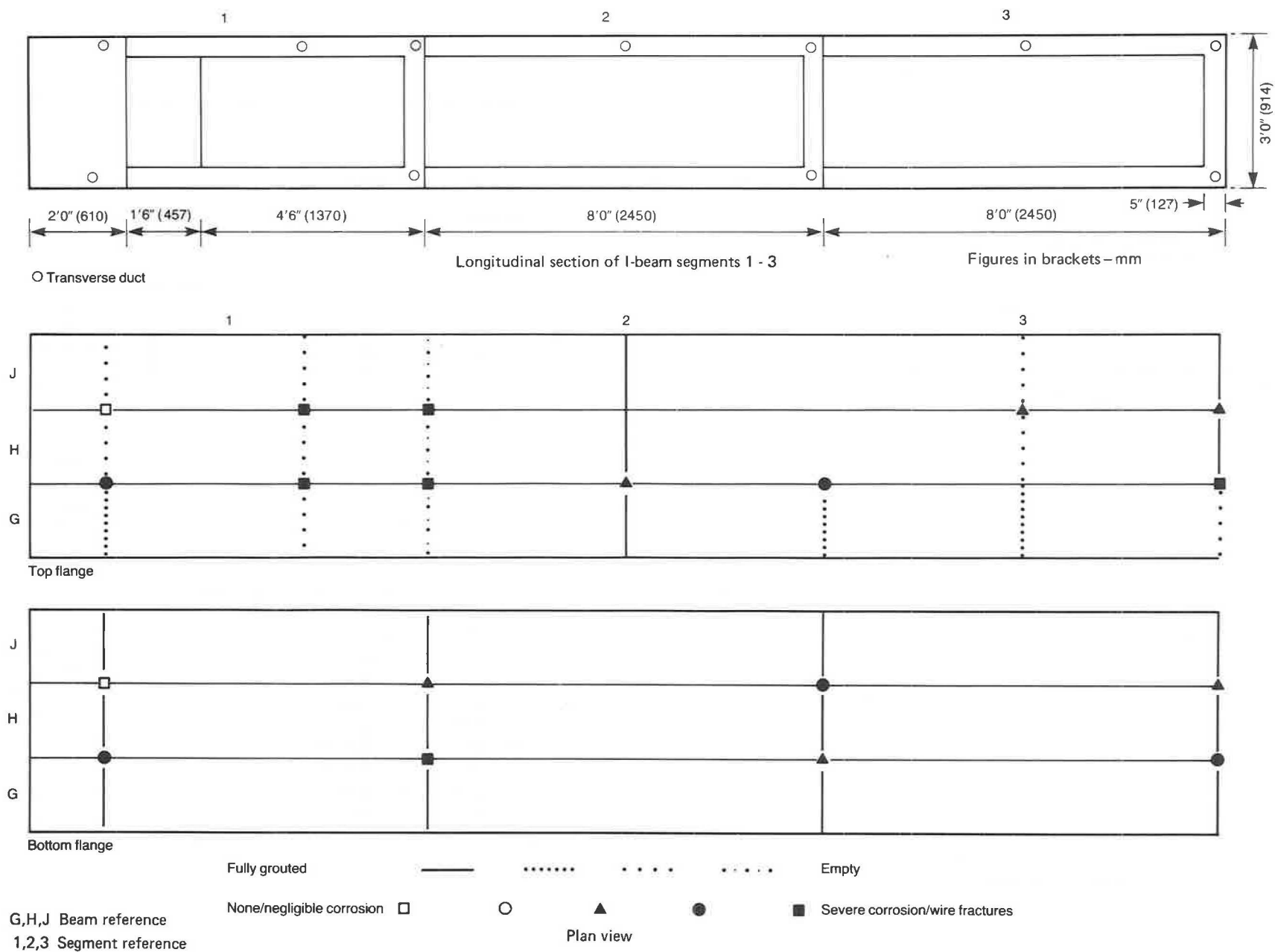


FIGURE 6 Condition of transverse tendons and grouting of ducts.

Leakage from ducts was measured by pressurizing them to 0.2 N/mm^2 and measuring the flow rate. Points of leakage can sometimes be located by painting soap solution on the beams.

Longitudinal Ducts

Eighteen of the 24 ducts examined in the I-beams were either fully grouted or contained small air pockets, either at the top of the duct or in the center between the wires. Of the other six ducts, four contained large voids with exposed wires and two were empty for part of their length (Figure 7). The largest voids were found near the anchorages in deflected ducts, although not all deflected ducts contained voids.

Holes were drilled into eight of the ten ducts in the end segments in the downstream edge beam. The ducts were inspected using a borescope, and the volume and continuity of voids were measured using the vacuum/pressure method just described. The results obtained are summarized in Table 1. Exposed wires were observed in ten of the sixteen holes drilled; they were covered with a layer of cement paste, and there was no evidence of corrosion. The continuity measurements showed that voids in the deflected ducts were isolated at the high points, whereas voids in the two straight ducts examined were continuous along the length of the beam. When the beam was demolished, all ducts appeared to be well grouted at midspan, although it was difficult to detect whether small voids were present because the ducts were unlined. It was not possible to measure void volumes in some ducts because of high air leakage, presumably through the joints. An attempt was made to locate points of leakage by pressurizing the ducts, painting soap solution on the joints, and looking for air bubbles, but without success.

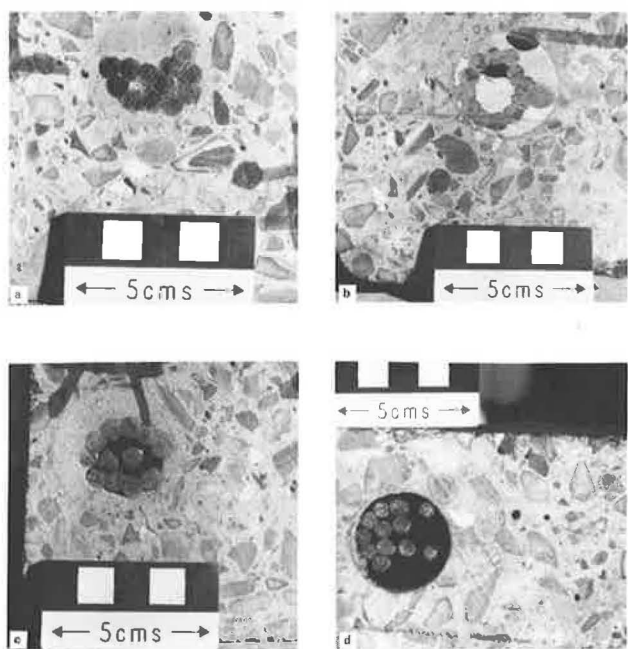


FIGURE 7 Examples of grouting. (a) Fully grouted, (b) small air voids, (c) void in center between wires, (d) empty.

The grouting in the upstream edge beam was examined at the middle three joints as the beam was broken up. Voids were found in the deflected ducts but, with one exception, the four straight ducts appeared to be fully grouted.

Grout was found in the bottom of the hollow box in both edge beams. It was about 15 mm deep in the downstream beam and 190 mm deep in the upstream beam (Figure 8d). It presumably had leaked from the ducts during construction.

Transverse Ducts

Eight of the fourteen transverse ducts examined were either fully grouted or contained only small air voids. Of the other six, three contained large voids with exposed wires and three were almost empty. Grout stains on the concrete adjacent to where ducts crossed some of the longitudinal joints indicated that leakage had occurred during construction.

Composition

The grout was dry, hard, and grey or sometimes white. All but two of the longitudinal ducts contained a grout consisting of only cement and water, whereas a fine aggregate had been added to the grout used for the transverse and the other two longitudinal ducts examined. One of the latter samples was analyzed and found to consist of Ordinary Portland Cement and fine quartz sand with an aggregate:cement ratio of 0.15:1.0.

In both of the longitudinal ducts containing aggregate, the grout was layered, one layer containing aggregate and the other not. Tests on one of these samples showed that both layers contained Ordinary Portland Cement, so the layering was not due to leakage of mortar into the duct. It may have been due, however, to the ducts being "topped up" after the initial grouting operation.

Samples of grout were taken from segments and joints of both longitudinal and transverse ducts, and depth of carbonation and chloride content were measured. Depth of carbonation was measured by cleaving the samples and spraying the exposed fracture surfaces with phenolphthalein. None of the samples examined had carbonated throughout, but the outer 1 to 2 mm of most of those taken from the joints had carbonated. It was noted that in both of the layered samples the boundary between layers was also the boundary between carbonated and uncarbonated grout. With one possible exception, grout from within the segments in grouted ducts had not carbonated; the exact origin of the carbonated sample had not been recorded.

The results of the chloride analyses are summarized in Table 2. With one exception, only trace amounts of chloride were found in samples taken from within the segments, whereas those from joints under the carriageway contained between 0.11 percent and 1.91 percent chloride by weight of sample, with an average of 0.42 percent; this is reduced to 0.33 percent, however, if the sample containing 1.91 percent is excluded. Less chloride was found in samples taken from the edge beam joints; one contained 0.30 percent, but the concentration in the remainder did not exceed 0.02 percent. There was no evidence that the chloride concentration in grout containing aggregate differed from that in grout that did not.

TABLE 1 DEGREE OF GROUTING—DOWNSTREAM EDGE BEAM

Duct	Segment	Visual inspection	Continuity N/mm^2 $\times 10^3$	Void volume	
				Litres	% of duct excluding tendons
1I	1	FG	0	0.1	1
	8	LV	14	0.3	3
2I	1	LV	0	0.3	3
	8	LV	0	NM	NM
3I	1	FG	0	0.3	3
	8	LV	0	3.3	30
4I	1	FG	81	3.7	34
	8	V	44	0.7	6
10	1	FG	0	0.3	3
	8	LV	0	0.7	6
20	1	LV	0	NM	NM
	8	LV	0	NM	NM
30	1	FG	0	0.3	3
	8	LV	14	2.6	24
40	1	FG	81	6.3	57
	8	LV	81	6.3	57

FG - Fully grouted

V - Void

LV - Large void.

NM - No measurement

Continuity - high figures indicate continuity of voids between segments.

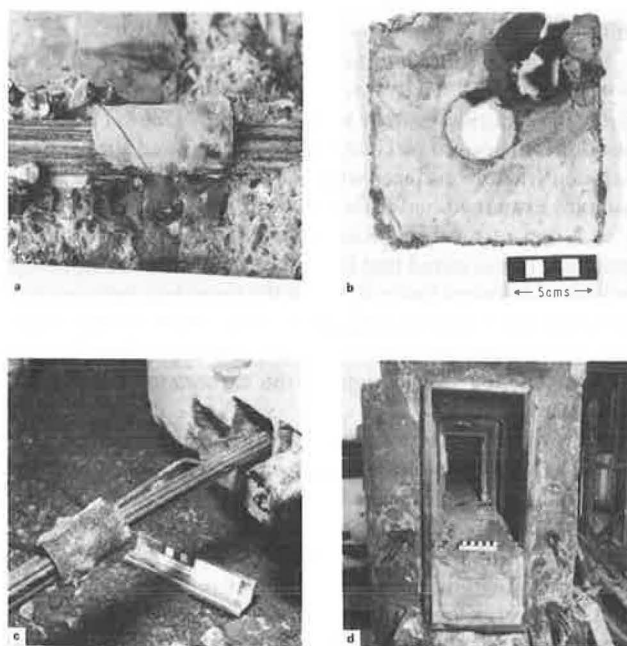


FIGURE 8 Protection of tendons at joints. (a) Cardboard tube over tendon, (b) asbestos packing with localized rust staining around hole through which tendon passed, (c) steel sleeve with corroded wires underneath, (d) leakage of grout from transverse joints into box section edge beam.

Condition of Prestressing Wires

Longitudinal Tendons

There was no serious corrosion within the segments. Wires viewed through holes drilled into longitudinal ducts were in good condition (Figure 9a). Exposed wires in large voids were covered with a layer of cement paste that varied from large patches to a few speckles dotted over the surface. There were patches of light surface rust on some wires, but these may have been present since construction. Patches of cement paste adhered to wires taken from the ducts, and while some uncoated lengths were covered with a layer of surface rust, others were in pristine condition.

Most wires were corroded where they crossed the joints. The condition of wires at the failure section varied from surface corrosion with no loss of sectional area to severe loss of section, the latter usually being accompanied by heavy scaling and pitting. Those with little or no loss of section had suffered ductile fracture. It was difficult to determine which of the other wires had fractured as the bridge collapsed and which had corroded through before collapse.

Several joints away from the failure section were carefully broken up to expose the wires. The amount of corrosion varied considerably, even on wires in the same duct. On the majority it extended over lengths of 50 mm to 100 mm (Figures 9c and 9d), but on some it covered up to 250 mm. It appeared to progress from slight surface scaling and pitting to severe,

TABLE 2 CHLORIDE CONCENTRATIONS (PERCENT BY WEIGHT OF CEMENT)

		Average	Standard deviation	Maximum
Concrete (1)				
I-beam	deck	0.20	0.19	0.73
	top flange	0.12	0.08	0.32
	web	0.20	0.18	0.64
	diaphragm	0.43	0.22	0.78
	bottom flange	0.58	0.33	1.15
	soffit	0.59	0.33	1.01
edge beam-ds	deck	0.09	(2)	0.09
	outer web	0.12	0.03	0.14
	inner web	0.40	0.28	0.83
edge beam-us	outer web	0.06	0.01	0.09
	inner web	0.26	0.25	0.92
Mortar				
	I-beam	0.66	0.63	2.08
	edge beam	0.09	0.15	0.96
Grout				
I-beam	at joint (3)	0.50	0.51	1.91
	" " (4)	0.22	0.11	0.36
	within segment (3)	0.01	0.01	0.03
	" " (4)	0.01	0.004	0.02
Edge beam-us		0.07	0.13	0.30
Corrosion products (5)		0.31	0.34	0.84
Asbestos (5)		<0.01	<0.01	0.01

1. Samples drilled from a depth of 2-10mm
 2. Insufficient samples
 3. Longitudinal ducts
 4. Transverse ducts
 5. Chloride content - % by weight of sample
- ds - Downstream. us - Upstream.

localized loss of section. On many wires loss of section was in excess of 50 percent, accompanied by heavy scaling and, in some instances, fracture of the wires. In extreme cases wires had lost more than 90 percent of their cross-sectional area but had not fractured. Only about 8 percent of the wires examined had actually fractured. On most wires corrosion occurred only on one side, the other often being in near perfect condition even after significant loss of section had occurred on the corroded side. The corrosion products were usually a mixture of the black and reddish brown oxides of iron; the former is magnetite (Fe_3O_4), the lower oxidation state of iron commonly known as black rust, and the latter haematite (Fe_2O_3), the higher oxidation state. Magnetite was also found on fracture surfaces. It is produced where the supply of oxygen is restricted; it is much more dense than the more common red rust and does not therefore produce the same expansive forces.

Three joints in both edge beams were dissected and the wires examined. Corrosion was again localized to the joints, but it was less severe than at the I-beam joints. After the wires had been removed from the beam, it was difficult to distinguish where some tendons had crossed the joints. Cor-

rosion was more severe in the upstream edge beam than in the downstream beam; the proportions of wires that suffered loss of section were 32 percent and 16 percent, respectively. This compares with 70 percent in the I-beam joints examined. The most severe corrosion was observed on wires that had been pulled against the cardboard tubes surrounding the ducts.

An attempt was made to assess the condition of the tendons where they crossed the joints. This was done by classifying the degree of corrosion on each wire and giving it a mark on a scale of 0 to 8 (Table 3). The total marks for each cable were then subdivided into five groups. The results obtained for the longitudinal ducts examined are illustrated in Figures 4 and 5. The estimates of loss of section are approximate as the heavy scaling associated with many of the more severely corroded wires made it difficult to estimate the amount of steel remaining.

Transverse Tendons

Wires extracted from within the segments of grouted ducts were partially covered with patches of cement paste. Some

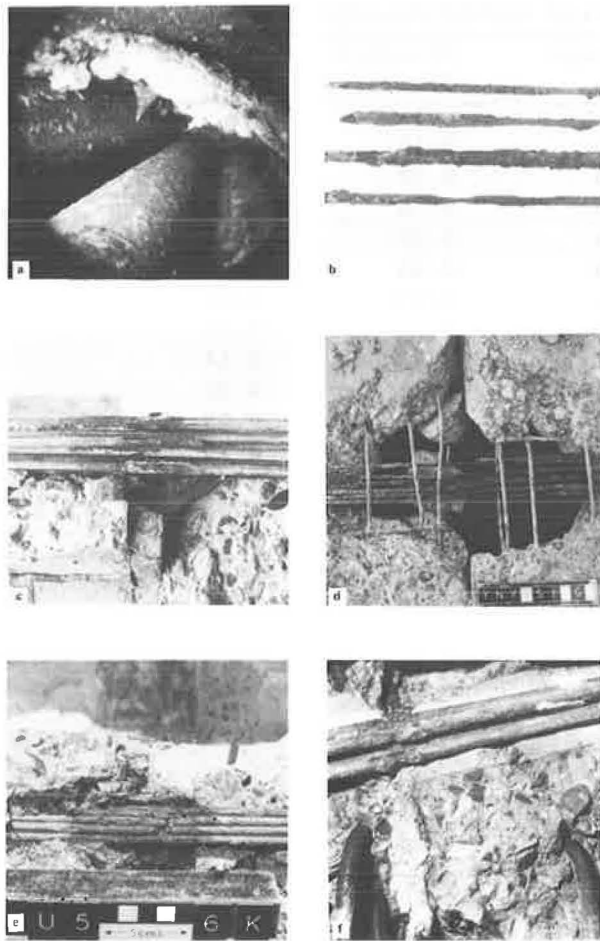


FIGURE 9 Condition of tendons. (a) Wires in poorly grouted duct within a segment, (b) wires from poorly grouted duct within a segment, (c) corrosion on wires at joint, (d) wires in poorly grouted duct at joint, (e) fractured wires at joint, (f) wires in well-grouted duct at joint. (a, c, e, longitudinal tendons; b, d, f, transverse tendons.)

surface rust was present, but in general uncoated areas were in perfect condition. Wires were severely corroded at the joints. Corrosion usually occurred on one side of the wires, and in some cases more than 90 percent loss of section had occurred over distances of a few millimeters (Figure 9f).

In poorly grouted ducts corrosion that had started at the joints extended along the width of the beams (Figure 9d). It varied from extensive surface rust to heavy scaling, loss of section, and fracture. Loss of section was usually almost uni-

form around the circumference of the wires. Most fractures occurred at the joints, although some were found within the segments, and short lengths of wire that had corroded through at both ends were found in some ducts (Figure 9b).

A summary of the condition of wires at the joints examined appears in Figure 6.

Material Properties

A metallographic examination on selected wires showed that they consisted of high-carbon, unalloyed, cold drawn steel with no evidence of faults that would have contributed to early failure. Chemical analyses of two wires are shown in Table 4. The tensile strengths of fourteen uncorroded wires were measured; they were in the range of 1,550 to 1,700 N/mm², with an average of 1,600 N/mm² and a standard deviation of 46 N/mm². This agrees with the design calculations, which assumed a strength of 100 to 110 tons/in.² (1,545 to 1,700 N/mm²).

Chemical analyses of corrosion deposits showed that chlorides were present in significant quantities. No evidence was found for the presence of sulfides. Fracture surfaces were covered with a black oxide that contained significant quantities of magnetite; its presence showed that oxygen access had been restricted. X-ray energy dispersion analysis of selected areas on the fracture surfaces showed the presence of chlorides and calcium.

Protection at Joints

The metal sleeves that were meant to have covered the tendons where they passed through transverse joints were found at only a small number of locations. Most ducts were surrounded by a flakey laminar material that looked like cardboard, and in several cases cardboard tubes could clearly be seen (Figure 8a). Where metal covers were observed, they did not appear to have offered much protection (Figure 8c). A soft, pasty substance was found at several of the transverse joints. Analysis of the material showed that it was a clayey soil; it may have been used to position the sleeves that covered the tendons where they crossed joints.

The squares of white fibrous material that had been placed between the beams at the positions of the transverse ducts were identified as asbestos. They were found at all the joints examined. They were usually damp despite the otherwise dry surroundings, indicating that they were acting as sponges, soaking up any moisture that penetrated the waterproofing

TABLE 3 CLASSIFICATION OF CORROSION

Degree of corrosion	Condition of wire	Marking
None	No corrosion	0
Mild	Slight surface corrosion with some scaling and pitting	1
Moderate	Mild corrosion plus up to 25% loss of section	2
Severe	25 to 50% loss of section	4
Very severe	>50% loss of section	8

TABLE 4 CHEMICAL ANALYSIS OF WIRES

	Element % by weight							
	C	Si	Mn	P	S	Cr	Ni	Cu
Wire 1	0.84	0.09	0.56	0.034	0.042	0.02	0.03	0.09
Wire 2	0.84	0.10	0.79	0.034	0.026	0.03	0.04	0.04

TABLE 5 CONCRETE PROPERTIES, MEASURED ON CORES DRILLED FROM THE STRUCTURE

I-beam		
Estimated in-situ cube strength (N/mm^2)	Web	67.0, 71.0
Density (kg/m^3)	Top flange	56.0, 64.5, 64.5
Composition	Web	2323, 2302
	Top flange	2291, 2267, 2328
	Cement	OPC
	Fine aggregate	Chert
	Coarse aggregate	Flint
Air voids	Cement/aggregate	1:3.6, 1:3.4 and 1:3.7
	Water/cement	0.37, 0.38.
		2%, 3% and 5%
Coefficient of water permeability (1)		2.08×10^{-11} m/sec to 7.29×10^{-11} m/sec
Depth of carbonation		0.4mm to 1.5mm
Chloride concentration(2)		Up to 8mm along microcracks
		< 0.05% by weight of cement
Edge Beam		
Estimated in-situ cube strength (N/mm^2)	Web	64.5, 67.0
Density (kg/m^3)	Top flange	66.0, 65.5
	Web	2300, 2360
	Top flange	2380, 2310

1. For guidance the permeabilities can be compared with typical values given in the reference 9, these are as follows

low $< 10^{-10}$
medium 10^{-10} to 10^{-12}
high $> 10^{-12}$

2. These measurements were made on samples taken from the centre of the concrete.

layer and providing a damp environment around the duct. In one instance the packing had slipped, thus restricting the flow of grout along the duct (Figure 8b).

Concrete

Examination of Cores

Eleven 100-mm-diameter cores were drilled from the top flange and web of some of the segments. Five were used to measure compressive strength and density, in accordance with BS 1881 (7,8); surface permeability measurements were made on three, and the rest were subjected to a petrographic examination.

In addition, the cement:aggregate ratio, water:cement ratio, and chloride concentration were measured on samples taken from the cores. The results obtained are summarized in Table 5.

In general the concrete was in excellent condition. It consisted of dense, high-quality paste, and there were no serious defects. The petrographic examination revealed minor microcracking that was attributed to long-term drying shrinkage. On one of the cores taken from a web there were more extensive microcracks in the surface zone, accompanied by microfractures that were at a low angle or parallel to the surface. The origin of these microfractures was not evident, but they may have been caused by the collapse. There was some surface corrosion on reinforcement in one of the cores but no

significant loss of section. Other pieces of exposed reinforcement were in excellent condition.

Chloride Concentration

Samples for chloride analysis were obtained by drilling holes 13 mm in diameter and collecting the resulting powder. To ensure that they were representative, several holes were drilled at each location; to avoid erroneous results, powder from the first 2 mm was discarded. Samples were taken from both the I-beams and the edge beams. They were drilled from the following positions:

- Deck (upper surface of top flange),
- Sides of the flanges,
- Webs,
- Diaphragms, and
- Soffit.

At a number of locations profiles of chloride concentration as a function of depth from the surface were obtained by drilling samples from depths of 2–10 mm, 10–20 mm, and 20–30 mm, respectively. Samples from the remaining locations were drilled from depths of 2 to 10 mm. The results obtained are summarized in Table 2, and normalized frequency distributions are shown in Figures 10 and 11. In addition, two samples were taken from the center of cores drilled from the concrete. These contained 0.05 percent and less than 0.05 percent chloride by weight of concrete, respectively.

Chlorides were found over the whole concrete surface. The concentrations in samples taken from the upper surface of the top flanges of the I-beams varied considerably, ranging from 0.05 percent to 0.64 percent by weight of cement (Figure 12). Samples drilled from the sides of the flanges contained between less than 0.05 percent and 1.00 percent chloride by weight of cement. The chloride concentrations in the bottom flange were generally higher than in the top flange. Chlorides were also found in concrete samples taken from the edge beams; the concentrations in the inner webs were higher than those in outer webs.

The profile measurements showed a decrease in concentration with increasing depth from the surface. This demonstrates that the chlorides had entered the concrete since construction. At some positions they had reached a depth of more than 30 mm (Figure 13).

Mortar

The mortar was found to consist of a 2:1 mix of sand (95 percent of which passed a 600- μ m sieve) and high-alumina cement. The aggregate was composed of quartz, with smaller concentrations of quartzite and shell. With one exception the samples examined were found to be carbonated.

The 24-hour water absorption was measured on three samples using the method given in BS 812 (10). The results ranged from 6.1 to 7.3 percent. These values are not unusually high for mortar. Permeability measurements were made on samples taken from joints 1/2, 2/3, and 7/8; the results obtained were 1.36×10 , 8.7×10 , and 1.46×10 m/sec, respectively.

More than 100 samples were drilled from both I-beam and

edge beam joints for measurement of chloride concentration. They were obtained by drilling horizontally into the mortar at different depths from the deck. The results are summarized in Table 2, and Figure 11 plots normalized frequency distributions of chloride concentration. Samples were also taken from the mortar cover over three anchorages. They contained 0.06 percent, 0.27 percent, and 0.66 percent chloride by weight of cement, respectively.

The chloride concentrations in samples from I-beam joints ranged from 0.06 percent to 2.04 percent, with an average of 1.01 percent. Some joints contained high concentrations throughout, whereas others contained very little (Figure 12). The edge beam joints contained less chloride, although there were localized areas of high concentration. There was no decrease in concentration with increasing depth from the deck surface, which would have been expected if de-icing salts had penetrated from above (Figure 14). Indeed, in joints containing large amounts of chloride, the higher concentrations tended to be nearer the soffit. Horizontal chloride profiles were also measured (Figure 14). There was no discernible decrease in concentration with distance from the surface through the I-beam joints, but there was a decrease in concentration with distance from the outer surfaces of the two edge beam joints examined.

Asbestos

Samples of asbestos from around transverse ducts where they passed through longitudinal joints were examined. Most samples were moist, and those from around corroded tendons were rust stained. The pH of water-soluble constituents ranged from 8.0 to 9.2. Eleven pieces of asbestos were analyzed for chloride; one was found to contain 0.02 percent chloride by weight of sample and the rest, 0.01 percent or less (Table 2).

STRUCTURAL ASSESSMENT

A reanalysis of the structure was performed by Travers Morgan and Partners under contract to TRRL. It showed that the shear capacity was marginally deficient, but the bending capacity was well below current requirements and may also have been below contemporary requirements. The soffit was only just in compression under dead load, and it was in tension under live load. Thus the joints may have opened under traffic, encouraging the passage of moisture and oxygen to the tendons, although no evidence of cracking had been observed during the routine inspections.

CAUSE OF COLLAPSE

Corrosion of Tendons

The prestressing tendons were severely corroded where they crossed the longitudinal and transverse joints. The structure collapsed when the sectional area of the tendons had been reduced to the point where they could no longer carry the imposed load. The mechanism by which collapse may have occurred has been discussed elsewhere (4).

Chlorides were the primary cause of corrosion. They were

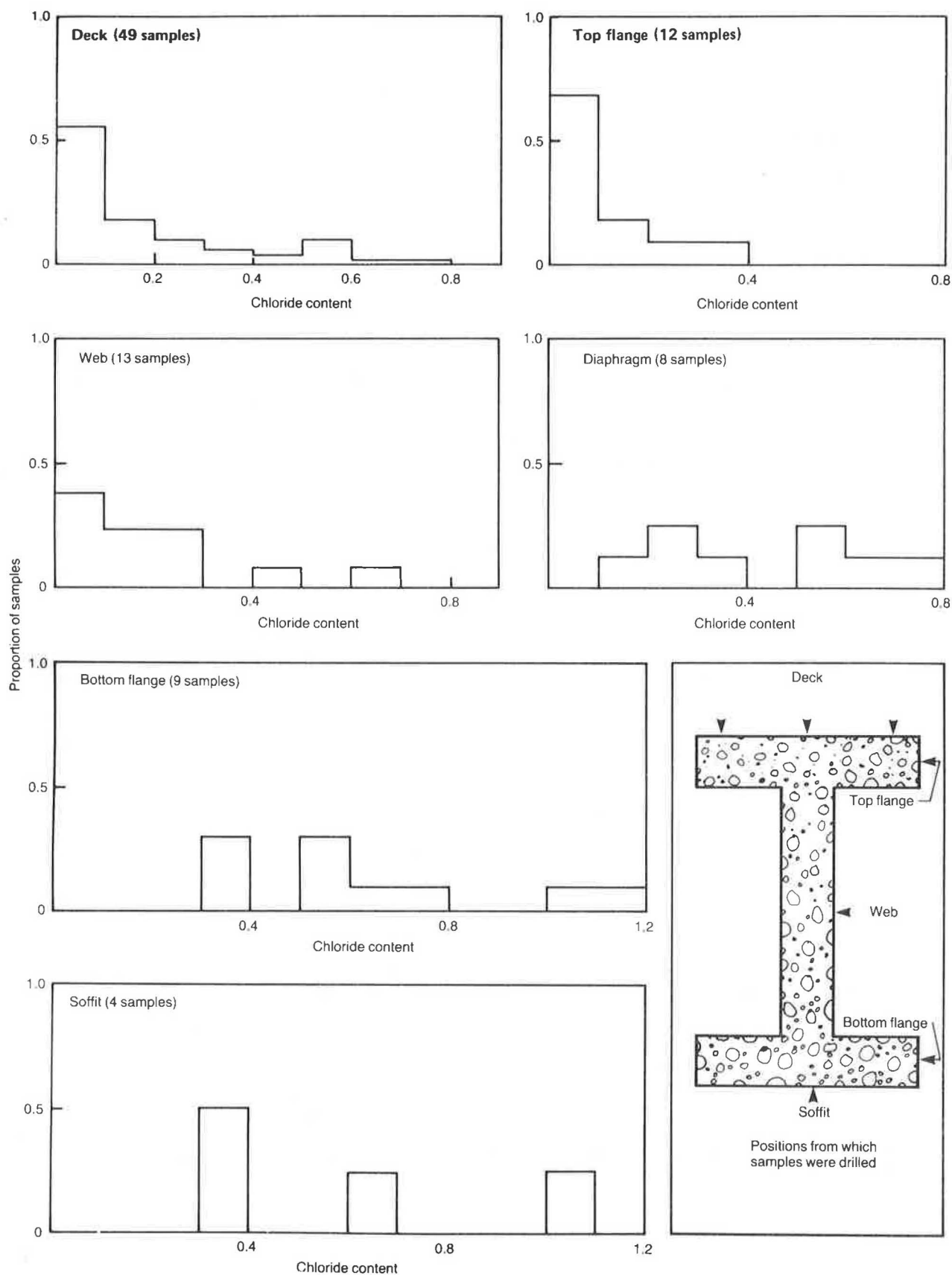


FIGURE 10 Normalized frequency distributions of chloride content (percent by weight of cement)—I-beam surface concrete.

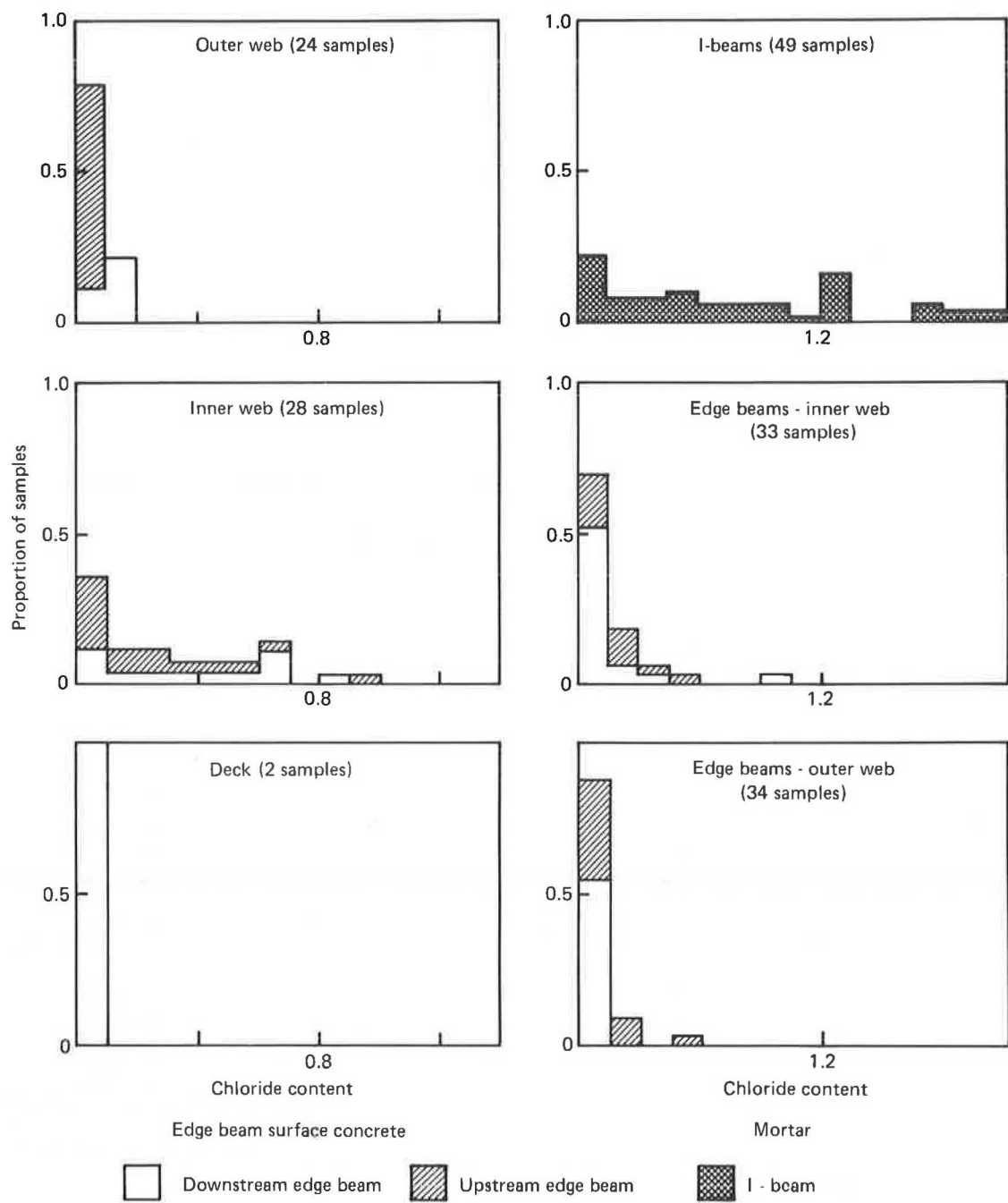


FIGURE 11 Normalized frequency distributions of chloride content (percent by weight of cement) in the edge beam surface concrete and mortar.

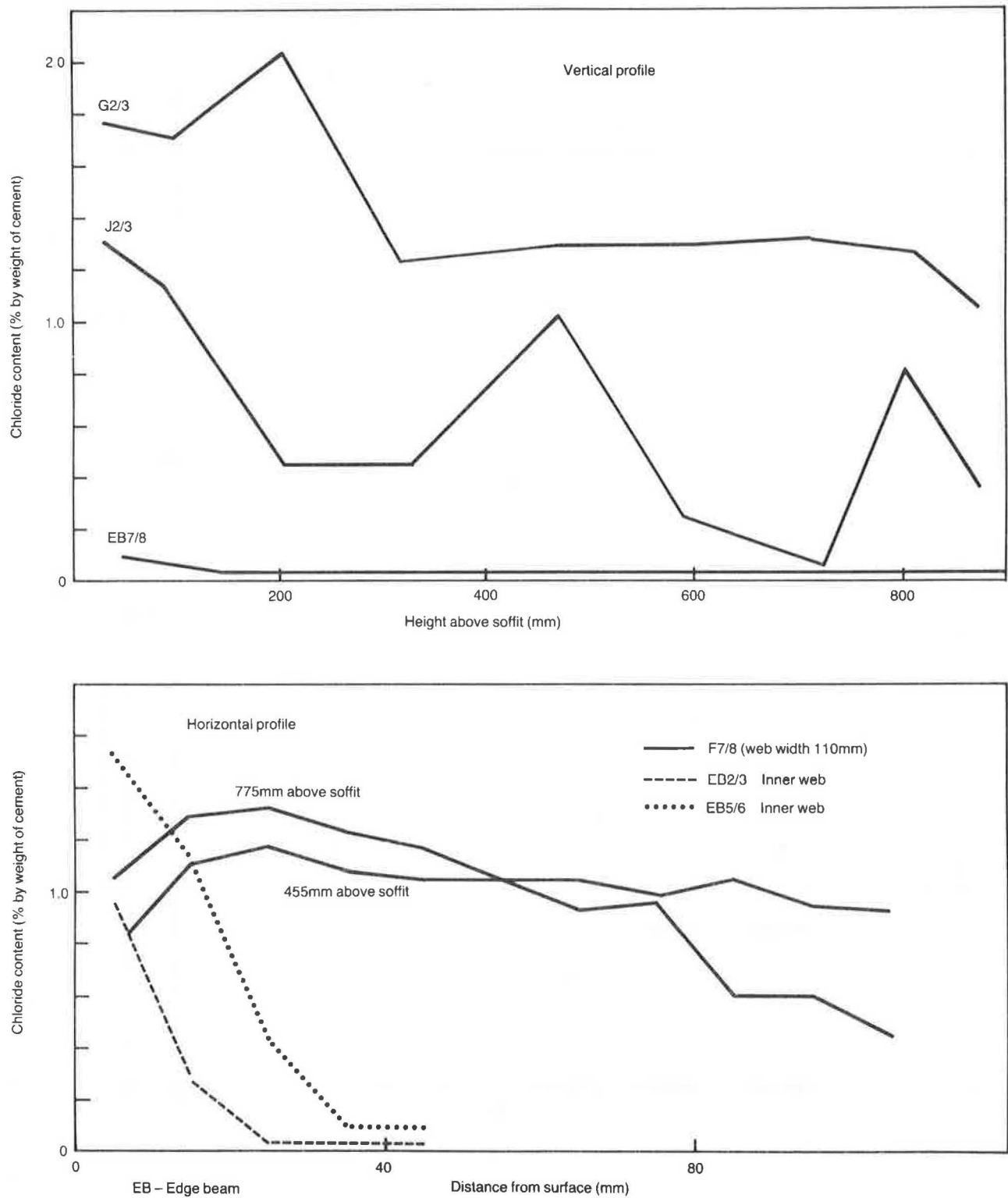


FIGURE 13 Horizontal and vertical chloride profiles through the mortar.

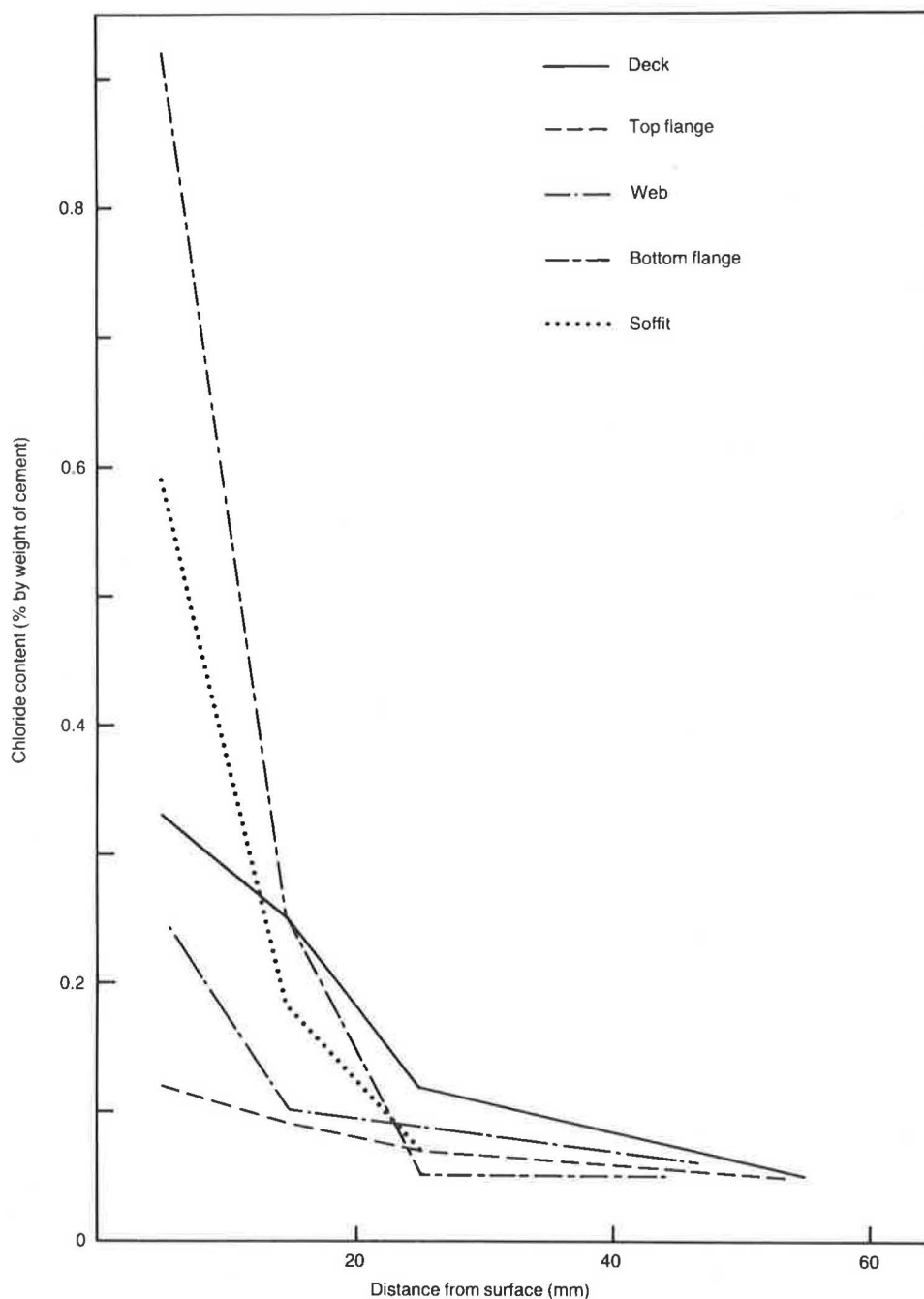


FIGURE 14 Average chloride profiles through concrete.

found in the surface concrete, transverse joint mortar, asbestos packing, grout at both longitudinal and transverse joints, corrosion products, and on fracture surfaces. The edge beams had not collapsed because the tendons had suffered less corrosion than those in the I-beams and, although chlorides were found in the corrosion products and on wires taken from one of the edge beams, their concentration in the surrounding mortar was much lower than in the I-beam mortar.

Source of Chlorides

Only trace amounts of chloride were found in grout from within the segments, whereas much larger concentrations were

found at the joints. This indicates that it was not present during construction but had subsequently entered the ducts where they crossed the joints. There are three possible sources of chloride:

- Calcium chloride added to the mix during construction,
- De-icing salt, and
- Chloride present in the aggregate.

Calcium chloride had not been added to the concrete, but the high concentration of chloride in the mortar between some of the I-beam segments suggests that it may have been added to the mix during construction. This is unlikely since its use

in high-alumina cement is not recommended and would serve no useful purpose.

Chlorides were found over the whole concrete surface, and their concentration decreased with depth from the surface. This shows that they had diffused in after construction and almost certainly originated from de-icing salt that had penetrated from the deck through the longitudinal joints between the beams. The higher concentration measured in the concrete in the inner webs of the edge beams compared with the outer webs suggests that de-icing salt had also passed through the joints between the edge beams and the sidewalk slabs. This percolation of chlorides between the beams accounts for those found in the asbestos packing and grout at transverse joints.

The presence of de-icing salt over the entire concrete surface implies that it must also have diffused into the mortar, and this would have occurred through the webs as well as from above. The horizontal chloride profiles through the inner web of an edge beam mortar joint decreased in concentration with depth from the surface, showing that chlorides had penetrated after construction. Similar profiles were not found through the I-beam mortar; this may have been because the webs in these joints were thinner than those in the edge beam and chlorides were able to penetrate from both sides.

The mortar in some I-beam joints contained a much higher chloride concentration than the surrounding deck concrete. This could be attributed to differential permeability. There were other joints, however, where the chloride concentration in the mortar was less than surrounding concrete. A possible explanation is that de-icing salt was not the sole source of chlorides. For example, it is possible that dune sand was added to the mix used for the mortar. The evidence is not conclusive, but dune sand was frequently used in the early 1950s, the bridge site is only a few miles from the sea, and the aggregate used for the mortar was found to contain quartz with a few fragments of shell, which is consistent with the use of dune sand.

The chloride concentration in dune sand is very variable depending on its origin and subsequent treatment. Only a small quantity of sand would have been required, and if this had been stored on site for some time, most of the chlorides in the sand at the top of the pile may have been washed away by the rain. Indeed it is known that severe storms occurred at the time of construction. This could account for the wide variations in chloride concentration in the joints. On the other hand, there was no evidence of chlorides in grout from within the segments even where sand had been added, and it is likely that the sand used would have been from the same source as that used for the mortar. Only six pieces of grout containing sand were analyzed, however, and they may not have been a representative sample.

To summarize, while it is possible that dune sand was used, it is almost certain that most of the chlorides originated from de-icing salt.

Effect of Chlorides

Although the presence of chlorides is a major cause of corrosion, it is difficult to quantify their effect. Chloride salts are

present to some extent in most structural concrete, and much research has been directed at attempting to define a safe threshold value below which the risk of corrosion is negligible. Unfortunately, the threshold value is dependent upon a number of factors, including the type of cement, cement content, degree of carbonation, permeability, and the ratio of soluble to insoluble chloride.

Various limits have been put on allowable chloride concentrations in reinforced concrete. The United Kingdom Specification for Highway Works (11) requires that the total chloride content shall not exceed 0.30 percent by weight of cement, whereas ACI Committee 201 recommended a limit of 0.2 percent (12). The latter figure was based on research done for the Federal Highway Administration (FHWA) and assumed that 75 percent of the chlorides would be water soluble. Limits for prestressed concrete have been more restrictive, and the United Kingdom Specification for Highway Works gives a limit of 0.1 percent.

In practice the greater the chloride concentration, the greater the risk of corrosion. Data from a corroded bridge in the United Kingdom have been analyzed to calculate the number of times corrosion was associated with a given value of chloride concentration (13). It was found that a concentration of 0.2 percent represented a probability of corrosion of less than 10 percent, although it was pointed out that such figures are likely to vary from bridge to bridge and may also vary with age.

The chloride concentration at the level of the steel in the precast units was well below 0.2 percent; this, along with the good quality of the concrete, suggests a low risk of corrosion, consistent with the observations. The isolated cases in which corrosion was observed may have been due to local increases in chloride concentration or pockets of poor-quality concrete. The chloride concentration in grout at the I-beam joints was much higher, ranging from 0.11 percent to 1.91 percent, although 53 percent of the samples tested contained less than 0.3 percent.

The average chloride concentration (by weight of cement) of all samples taken from the surface 10 mm of the precast concrete was 0.31 percent, with a maximum of 1.15 percent. This is less than has been found on many other bridges in the United Kingdom. Vassie (13) has recently performed a survey of the chloride concentration in eight reinforced concrete trunk road bridges after 50 years' service. Samples were drilled from the surface to a depth of 25 mm. The mean chloride concentration in each bridge varied from 0.33 percent to 1.2 percent. If samples had been drilled to a depth of only 10 mm, the concentrations would presumably have been considerably higher. Other investigators have reported chloride concentrations in excess of 5 percent (14).

Factors Contributing to Corrosion

The effect of chlorides on corrosion is illustrated in Figure 15, which shows increasing corrosion as the chloride content in grout adjacent to the tendons increases. There is considerable scatter on these results, and this is partly because the chloride content measured is an average value and the concentration adjacent to the steel may differ from this. In

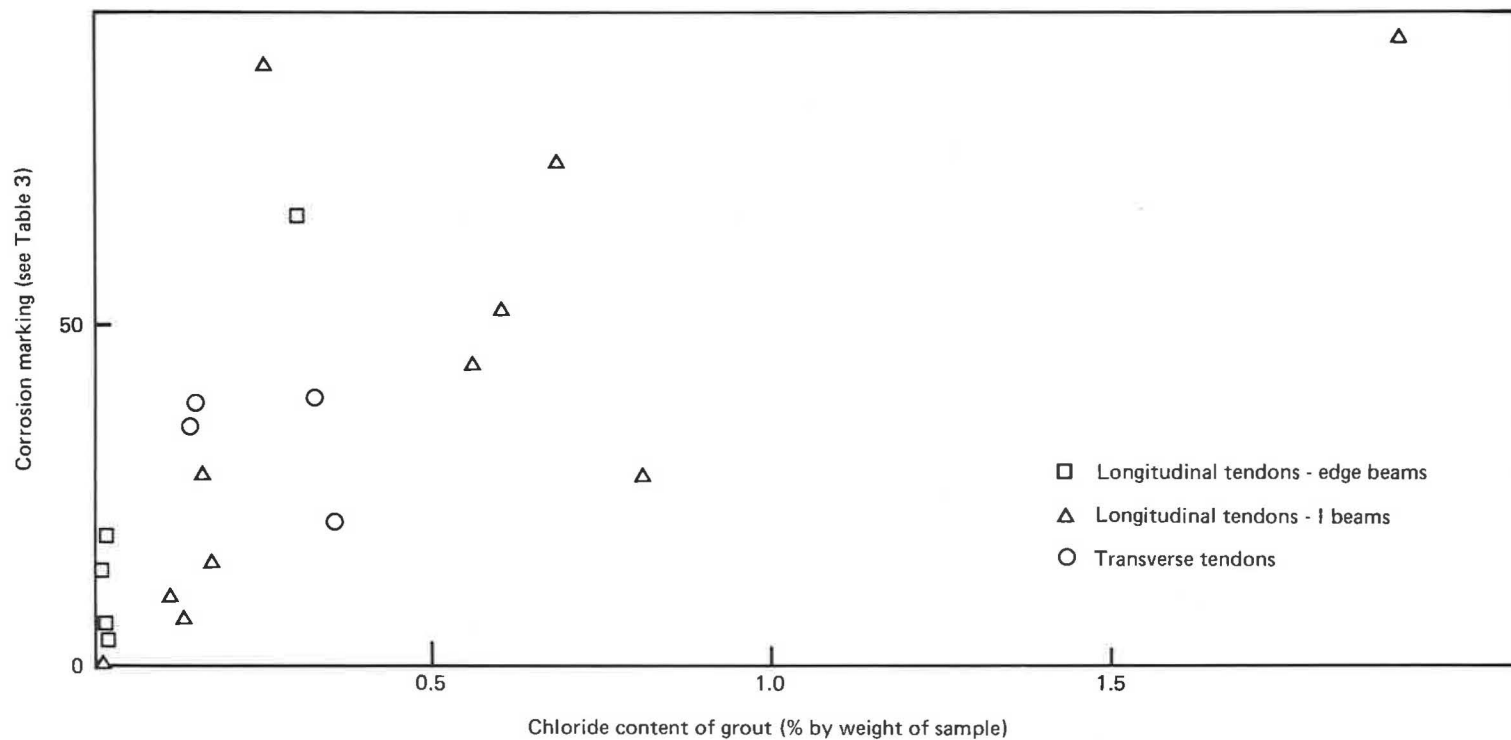


FIGURE 15 Degree of corrosion of tendons versus chloride content of adjacent grout.

addition, there were numerous other factors that contributed to corrosion, some of which are discussed below:

- The absence of an in situ deck: This makes it possible for moisture (as well as chlorides) to enter the segmental joints and penetrate to the tendons.
- Ineffective waterproofing: The bridge records indicated that a waterproofing layer was applied during construction, but this had not prevented the ingress of chlorides into the concrete. The variation in chloride concentration showed that its performance varied over the deck.
- Inadequate protection at the transverse joints: The tendons were protected by grout, steel, or cardboard sleeves and 40 to 50 mm of mortar. The mortar was well compacted and of reasonable quality, but a 40- to 50-mm covering would not give the same level of protection as good-quality dense concrete. It was between a hundred and a thousand times more permeable, which would make it easier for oxygen and moisture to penetrate to the steel. The use of high-alumina cement may have exacerbated this effect as porosity increases after conversion, although the mortar samples tested were only partially converted. This was illustrated during the vacuum pressure measurements when it was found that it was difficult to maintain a vacuum in ducts containing large voids. This had not been the case during previous work on monolithic beams (15).
- Underdesign: It is probable that the segmental joints had opened under live load, and this may have accelerated further the passage of moisture and oxygen to the steel, especially at midspan.
- Inadequate protection at the longitudinal joints: The squares of asbestos packing were the only protection given to transverse tendons, and these appeared to act as sponges soaking up moisture.
- Carbonation of the grout: Inadequate protection at the joints resulted in carbonation of the outer surface of the grout. Corrosion was usually more severe where tendons were pulled against the sides of the duct and embedded in carbonated grout.
- Poor grouting: The large voids present in some of the transverse ducts allowed corrosion at the joints to extend along the length of the tendon.
- The damp environment provided by the river beneath the structure.

Condition of Structure—Excluding Corrosion

The structure possessed numerous redeeming features. The concrete in the segments was a good-quality dense material with no serious defects. The mortar in the transverse joints was well compacted. Most longitudinal and some of the transverse ducts were well grouted; indeed, the incidence of voids in the longitudinal ducts examined was less than has been found in more recent structures (15). With the exception of partially grouted transverse ducts, wires from within the segments were in excellent condition. Exposed wires in partially grouted ducts were covered with a layer of cement paste. Surface rust was observed on some wires, but this may have been present since construction. Where cement paste had not

adhered to the wires, the exposed steel was often in pristine condition, after more than 30 years in service.

IMPLICATIONS

The absence of visual evidence of corrosion in the form of cracking and spalling at joints adjacent to grouted ducts is of major concern. Evidence of corrosion was not visible because corrosion was localized at the joints. The low level of metal loss and high density of some of the corrosion products (magnetite) would have produced limited expansive forces that must have been accommodated by the surrounding material. The absence of such evidence is consistent with the very localized rust staining on the asbestos packing shown on Figure 8. In partially grouted ducts the corrosion products would not have produced expansive forces. It would therefore appear that routine visual inspection is inadequate for detecting corrosion at joints in segmental bridges. Structures at risk could be identified by checking to determine whether they contain features similar to those on the Ynys-y-Gwas bridge. For example:

- Segmental construction,
- Narrow infilled joints between segments,
- Absence of a reinforced concrete deck slab,
- Transverse stressing,
- Location,
- Regular winter gritting, and
- Date of construction.

Where structures are identified as being at risk, those techniques that can be used to determine whether the tendons are corroding should be ascertained. At present this cannot be done directly, so inspections must take full advantage of available investigatory techniques for assessing the corrosion risk. These include measurement of chloride concentration at and near the joints, load testing to determine whether segments are opening under load, and drilling holes into ducts to observe the condition of the tendons. This latter technique must be used with caution. Other techniques, such as electrode potential and resistivity surveys, that can provide useful information on reinforced structures are less appropriate for tendons in ducts. They may detect corrosion of adjacent reinforcement, although this would not have been the case at Ynys-y-Gwas. In the longer term, improved inspection and testing techniques are needed (3,16).

To avoid the occurrence of corrosion problems in the future, a greater emphasis must be put on the protection given to prestressing systems in new structures, especially those of segmental construction. In addition more thought needs to be given to designing structures in which the tendons can be inspected.

CONCLUSIONS

The structure collapsed because of corrosion of the tendons where they crossed the joints. Failure occurred when the sectional area of the steel had been reduced to the point where

it could no longer carry the imposed load. The bridge had been regularly inspected, but there had been no evidence of deterioration before failure.

Chlorides were the primary cause of corrosion. They were found over the whole concrete surface and in the transverse joint mortar, asbestos packing, grout at both longitudinal and transverse joints, corrosion products, and on fracture surfaces on the tendons. Most chlorides almost certainly originated from de-icing salt that had penetrated through the longitudinal joints between the beams. It had diffused into the mortar, both vertically from the deck and horizontally through the webs. The concentrations in both the concrete and mortar were very variable, however, and the high concentrations throughout some joints indicated that chlorides had been present in the aggregate used for the mortar. The bridge site was near the sea, and dune sand may have been used, although the evidence is not conclusive.

The maximum chloride concentration in the surface concrete was 1.15 percent by weight of cement, and the average was 0.31 percent. This is lower than has been found in other structures in the United Kingdom. In addition to the presence of chlorides, corrosion had been exacerbated by the following:

- Lack of an in situ slab over the beams,
- Ineffective waterproofing,
- Inadequate protection to the tendons,
- Opening of the segments under live load,
- Poor workmanship, and
- Dampness of the environment over the river.

The concrete in the precast units was a high-strength dense material with a low permeability to water, and the reinforcement in the units was in good condition. Exposed tendons within the segments in partially grouted, longitudinal ducts were covered with a layer of cement paste, and there was no evidence of corrosion. This shows that where protection against corrosion is adequate, post-tensioned concrete can provide long-term durability.

To date no other bridges of similar construction have been identified in the United Kingdom. It is known, however, that there are about a hundred segmental bridges without an in situ deck and possibly many more worldwide. The vulnerability of the tendons to chloride corrosion needs to be assessed. In the longer term, improved inspection and testing techniques for prestressed structures need to be developed.

At present the advantages of quick and efficient construction procedures have led to an increase in the number of segmental bridges being built. The findings reported in this paper, however, illustrate how vulnerable joints in segmental construction can be to salt attack; and this must have implications for modern-day segmental construction. Greater emphasis is needed on the protection given to prestressing systems and the design of structures in which tendons can be inspected.

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