# Prestressing in Coventry Cathedral

Chris Burgoyne*

*Dept of Engineering, University of Cambridge, Trumpington St, Cambridge CB2 1PZ*

Owen Mitchell*

*Present address: Mott MacDonald, 22 Station Road, Cambridge CB1 2JD*

## Abstract

Coventry Cathedral was completed in the early 1960s and has some prestressed concrete elements to resist lateral thrust from the roof. Other prestressed structures of a similar age have had corrosion problems and this has drawn attention to the fact that there is little publicly available information about the structural system at Coventry. This paper addresses that issue and is in three sections. The first summarises the four different prestressing systems in the cathedral and estimates the amount of prestress and its purpose in each location. By placing the information in the public domain it will be useful for both historians of church architecture and engineers in future generations who may have to work on the building. Although there is no evidence of corrosion in the building at the moment, it is impossible to inspect the existing tendons, so the second section considers what might happen to the structure if corrosion of the tendons were to occur. It is concluded that very little warning of failure would be given, which would be especially important for the tendons over the Baptistry window and those in the Nave ties. The final section considers what could be monitored to give as much warning as possible about future problems. The effects of loss of an individual tendon, which would not by itself be sufficient to cause failure of the structure, would cause only very small strains that would be difficult to distinguish from the background strains caused by temperature change. Many of the principles discussed in the second and third sections would be applicable to many other prestressed concrete structures.

*Keywords:* Prestressed Concrete, Corrosion

## 1. Introduction

Coventry Cathedral was completed in the early 1960s. It is mainly constructed of unreinforced concrete, with the walls clad in stone and rendered

---

*Corresponding author
    Email address:* `cjb@eng.cam.ac.uk` (Chris Burgoyne)

blockwork, but there are some prestressed concrete elements to resist lateral thrust from the roof. The well-publicised problems with the prestressing tendons in Hammersmith Flyover, which was of a similar vintage ((BBC, 2011),(Lynch, 2012)) led the cathedral authorities to be concerned that they might have comparable issues, so the authors were asked to investigate the use of prestressing in the cathedral. Despite the architectural importance of the building it became clear that there was only a limited amount of published information about its structure. Since society expects cathedrals to be around for hundreds of years, it was felt that the basic structural mechanics of the cathedral ought to be in the public domain. This paper addresses that issue by describing the prestress in some detail.

The paper also describes how problems with the prestressing might manifest themselves, and also whether anything could be monitored to determine whether corrosion were taking place. These considerations are of more general interest because they would apply to most prestressed concrete structures.

It should be stressed at the outset that no evidence has been found that the prestressing tendons are failing, but since they cannot be inspected, neither can any guarantee be given about their condition. It must also be stressed that no faults have been found in the original design.

*1.1. Historical Context*

The present cathedral stands on a site of both historic and religious significance. There have been three religious structures here in recorded history, beginning with St. Mary's Priory, dating back to the Middle Ages, of which only a few ruins remain (Hodge, 2012). The second structure was the previous St. Michael's Cathedral, constructed in the late 14th century. It was originally the parish church but became a cathedral when the Bishopric of Coventry was founded in 1918. It was seriously damaged, largely by fire, during a very heavy bombing raid on 14 November 1940 that left the tower and most of the walls standing. The stained glass and other treasures had earlier been removed against just such an eventuality.

The widespread destruction of homes, and the great loss of life, meant that the name "Coventry", and the cathedral in particular, came to symbolise both the suffering and resistance of the British population. In his "Give us the tools and we will finish the job" radio broadcast on 9 Feb 1941, Churchill pointed out *"All through these dark winter months the enemy has had the power to drop three or four tons of bombs for every ton we could send to Germany in return. London and our big cities have had to stand their pounding."* His audience would have been well aware that he was alluding to the attack on Coventry, as he was when he said on 30 December 1941 *"Hitler and his Nazi gang have sown the wind: let them reap the whirlwind"*. The symbolism of Coventry remains potent: in the 1980s, when it became clear how much information had been obtained via decrypts at Bletchley Park, there was a furore that Churchill might have known in advance about the raid on Coventry and could somehow have prevented it. It now clear that the decrypts showed only that a major raid was being planned, not its target (Calvocoressi, 1981).
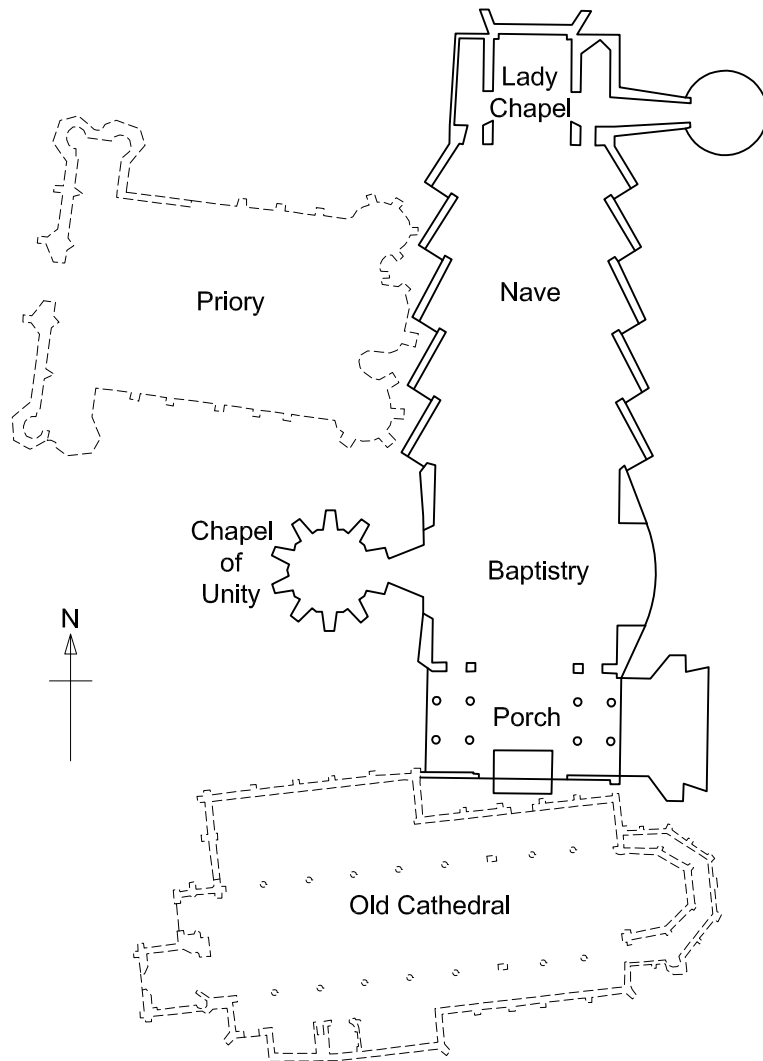
Figure 1: Layout of Coventry Cathedral (redrawn from Hodge (2012)).

The cathedral was thus far more than a church that had been destroyed in the bombing, so it was unsurprising that after the war there was a move to rebuild it, not just to re-establish it as a place of worship, but also to symbolise the recovery of the country. A commission was set up and a competition held that drew 219 entries internationally (Spence, 1962). The brief made clear that a traditional east-west orientation was not required, and Basil Spence (knighted in 1960) won the competition with a design that placed the new cathedral at right angles to the ruins. The central axis of the cathedral thus lies in a north-south orientation, which is unusual for Christian architecture in Britain (Hoare and Sweet, 2000). By connecting the new and old cathedrals he was creating a symbol of reconciliation and forgiveness in the post-war period, which the cathedral has since enhanced by the foundation of the "Community of the Cross of Nails", a reference to the cross made the day after the bombing from roof nails that were found in the burnt-out ruins. The foundation stone of the new cathedral was laid in 1956 by Queen Elizabeth II. The funding was to be provided by the War Damage Commission who specified a "plain replacement" and the original estimate was for a "total cost not exceeding £985,000" or about £18m at 2017 prices. However, once the design was complete the estimate more than doubled, which led to cost-cutting (Spence, 1962). Spence himself raised money for the cathedral by lecturing in the UK and overseas, and many of the artworks were donated. The building was completed and consecrated in 1962, and is widely regarded as one of the gems of modern British religious architecture, having been one of the first post-war structures to be Grade I listed.

*1.2. General Description of Cathedral*

Spence designed the new building to be distinctly modern, but also to have aspects of a traditional cathedral. Though not large by comparison with the mediaeval English Cathedrals, the interior has a cavernous and monolithic feel without the large columns that typically separate the nave from the side-aisles. From the outside, the cathedral is relatively simple, unlike historic cathedrals with their extravagant buttressing and carvings. The walls are clad externally with warm-coloured Hollington sandstone, similar to the locally-quarried stone that had been used in the previous cathedral. The geometric simplicity of the exterior was designed to contrast with the richness afforded by the many commissioned artworks, most notably Epstein's sculpture of "St Michael's Victory over the Devil" at the entrance and Graham Sutherland's tapestry of Christ at the northern end of the cathedral, leading Spence to describe the building as a "plain jewel-casket" (Spence, 1962). The cathedral, which for the most part has a relatively simple structure, was engineered by a team from Ove Arup & Partners, led by Povl Ahm, although Arup himself and Edmund Happold were also heavily involved.

By tradition, the altar is placed at the eastern end of the nave, with a Lady Chapel beyond. The congregation enter the nave through the west front, which is often ornately decorated. In Coventry the altar is to the north, with the entrance to the south, linking the ruins of the old cathedral to the new (Figure

1). To avoid confusion between liturgical directions and physical orientation of the building, in what follows all references to north, south, east and west will refer to geographical directions.

The cathedral is founded on bored concrete piles to a sandstone layer that is about 10 - 12 m below the structure; these are surmounted by reinforced concrete pile caps or are directly placed under the walls.

The porch is covered by a barrel vault over the space between the ruins and the new cathedral, which is entered from the south through a great glass screen that takes the place of the traditional "West Front". This leads to the large, square baptistry area, with a colourful window (Figure 2a) to the east by John Piper and the Chapel of Unity to the west. There is no screen or divider separating the baptistry and the nave, which tapers to the north, focussing attention on the altar and the Sutherland tapestry. The serrated walls of the nave were faced in concrete blocks as an economy measure and then rendered, which aids the acoustics, and appear not to be punctured by windows, but the south-facing edges of the serrations are glazed to their full height of 23 m, illuminating the congregation from behind and throwing light forward to the altar. The serrations leave triangular spaces at the edge of the nave known as Hallowing Places. A second side chapel, dedicated to Christ the Servant, protrudes at right angles at the northern end of the cathedral near the Lady Chapel.

The shallow roof spans the nave without internal support, but is hidden from view by a ceiling canopy made from pyramidal timber panels that span between thin concrete beams, which are in turn supported by slender precast concrete columns (Figure 2b). This separation of roof from ceiling significantly simplified the structural design.

The roof of the baptistry supports a 25 m high lightweight aluminium truss fleche that is surmounted by a "flying cross". Both were lifted into place by a RAF Belvedere helicopter.

### 1.3. Existing Documentation

For a structure as significant as Coventry Cathedral, it is perhaps surprising how little there is in the way of published technical information. Plenty has been written about its architecture, such as Campbell (2006), and the architect's own book (Spence, 1962) but apart from some brief articles written during the construction, giving an indication of the design intent of the Arup engineers, (Arup, 1985; Ahm, 1962a,b, 1987; Perkin, 1962), there are no papers that give enough detailed information to allow an analysis of the stress-state of the building. The project is mentioned in Arup's biography (Jones, 2006) and its superficial condition was reviewed more recently by Clarke (2000).

*RCAHMS.* Basil Spence spent many years at the beginning of his career working in Scotland and the competition entry was designed in his Edinburgh office. After his death in 1976 many of the documents that survived from his practice were donated by Anthony Blee and his wife Gillian (née Spence) to the Royal Commission on the Ancient and Historic Monuments of Scotland (RCAHMS),

(a) Concrete mullions in Baptistry window



(b) Nave interior with pretensioned columns

Figure 2: Interior views

based in Edinburgh. This archive contains hundreds of categorised folders, photos, scans and drawings relating to the design and construction of Coventry Cathedral. Amongst the many items of day-to-day communication there are some that give crucial engineering information, such as material data, letters between the engineer and Clerk of Works about tendon stressing, and stressing sheets detailing the actual extensions applied to the tendons.

*Arup Archives.* Some documents were archived by Ove Arup & Partners (now simply Arup). They hold two boxes of papers but many are rough calculations with little indication of their context. However, there were enough clear and legible prestressing calculations to inform similar calculations for the rest of the Cathedral.

*Cathedral Archives.* Coventry Cathedral archives contain much information, including many books about the architecture, but very little about the engineering. They hold a microfilm containing a set of 218 Arup drawings and also have some other microfilmed drawings including some from Spence. Although some figures and details were hard to read due to the poor resolution of the film and small size of the original annotations, there is enough information to construct a geometric model. An example drawing from the microfilm is shown in Figure 3. Some information, most notably bar schedules, is not available, although the size of the bars can be inferred because it was standard Arup practice for the first digit of the bar mark to represent the bar size (in 1/8ths of an inch).

*Laing photographic archive.* The new cathedral was built by the contractor John Laing and Co. Their archive is now located in Northampton Library.

*Those involved in the project.* Inevitably, with the passage of time, most of those involved in the project have died, but contact was made with Anthony Blee, who joined Basil Spence's practice as an architect in 1956, four years after the commencement of the Coventry Cathedral project. Having been part of the design team, he was able to shed light on the construction sequence of the Cathedral and aspects of the relationships within the team.

Ian Bedford worked as a site engineer for John Laing and was able to describe the prestressing operations for the nave ties, the canopy columns and some of the stressing in the baptistry.

Contact was also made with Prof Roger Johnson, of Warwick University, who had been involved with measurements of the behaviour of the roof, and Alf Cleugh who had worked on the site as a Clerk of Works and had been involved in some of the prestressing operations.


## 2. Prestressing System as installed

It is clear that the primary intention of the design engineer was that most of the structure would be built from unreinforced concrete, so it would largely act like a masonry building. A few courses of the external stonework and the
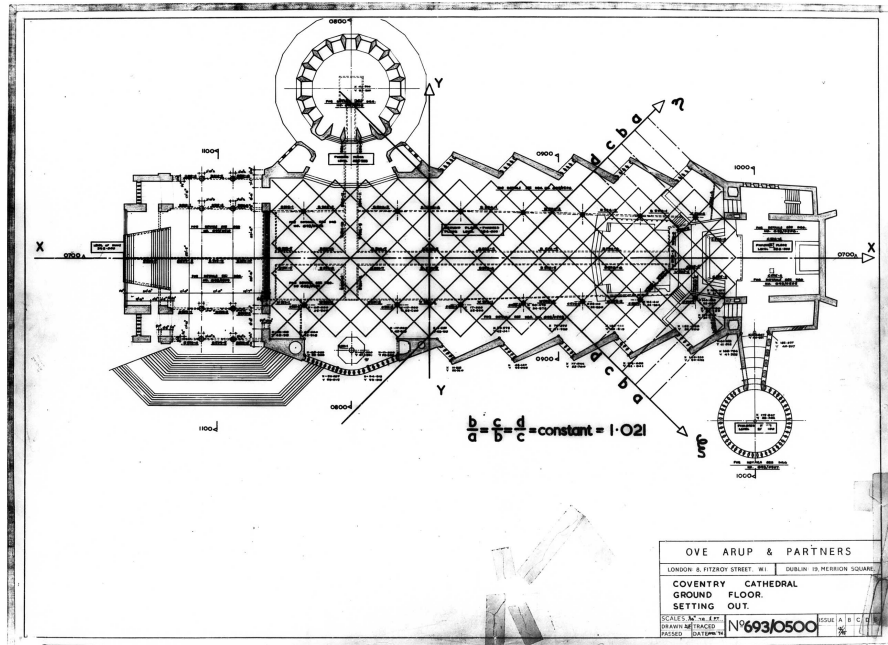
Figure 3: A microfilm drawing, showing the setting out of the Cathedral. Coventry was job number 693 for the engineering practice.

internal blockwork were built, then the stone was painted on the inside with a mastic waterproofing paint before the gap was filled with mass concrete and the process repeated. The form of construction was chosen to eliminate as many as possible of the corrodable elements, in recognition that religious buildings are traditionally amongst our oldest structures. Spence stated that

> "The materials of everything were chosen with care. This building was being designed for a life of at least 500 years and all pervious materials had to be avoided ... Concrete was not allowed to come directly in contact with the English climate." (Spence, 1962)

Interestingly, it is the concrete that was seen as the durability risk, rather than the steel that it contains.

The use of vertical walls with no reinforcement meant that the bulk of the structure could only resist gravity loads, so some method had to be found to deal with the lateral load from the roof, and this is where the prestressing comes into play.

*2.1. Locations of Prestress*

Prestressing is used for four distinct purposes in the Cathedral (Figure 4).
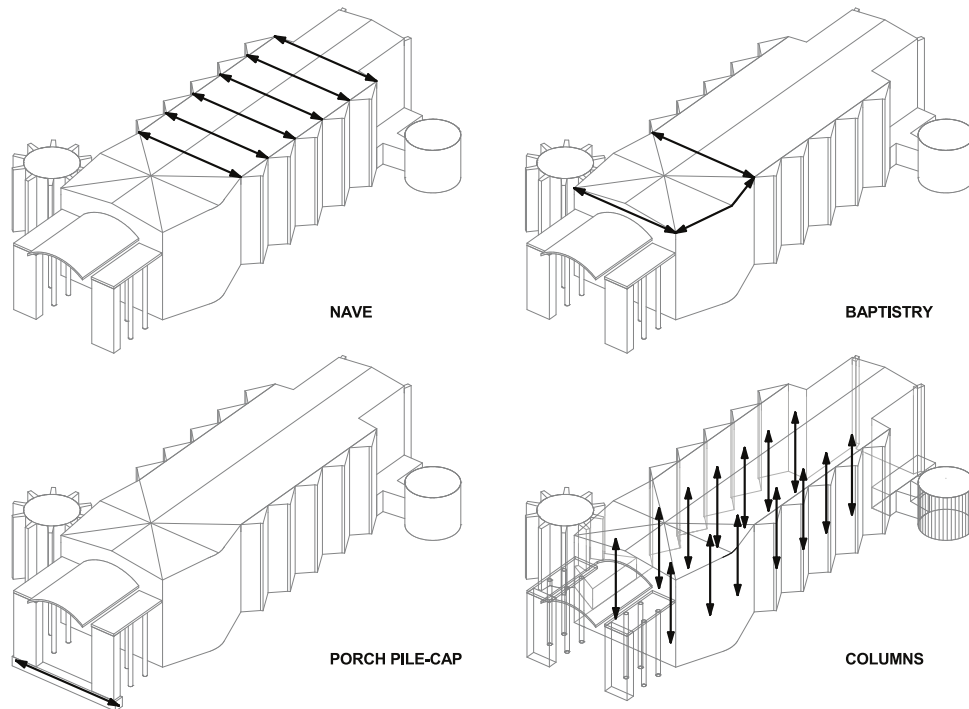
8

Figure 4: Zones with prestressed concrete; nave roof tie-beams, baptistry roof, porch pile-cap, nave columns

1. To resist the outward thrust of the nave roof by providing horizontal pre-stressed concrete tie beams.
2. To resist the thrust from the folded plate roof over the baptistry. This prestress is embedded within the walls at eaves level.
3. To resist the lateral force from the barrel vault over the porch. At one end this is carried by tendons in the south wall of the baptistry, while at the other it is carried down to the ground and resisted in a prestressed concrete ground beam below floor level.
4. To prevent buckling of the thin precast columns that support the decorative internal wooden canopy provided by Spence to improve the aesthetics of the interior (Figure 2b).

The first three sets of prestressing are critical parts of the main structure of the cathedral and used the Freyssinet system. The set in the internal columns used the Gifford-Udall system.

There is a degree of overlap between the systems. The prestressing in the southernmost nave tie-beam also serves as the prestressing element for the north edge of the baptistry roof, and the prestressing elements in the south wall of the baptistry also resist the thrust from one edge of the porch roof.

All the tendons were specified to be grouted but this is known to be an area in which poor site practice in the 1960s led to subsequent problems with prestressed concrete (Woodward and Williams, 1988). The difficulties are exacerbated in horizontal tendons where grout does not flow easily: this potential problem occurs in all the tendons with the exception of those in one wall of the baptistry and in the canopy columns. Thus, there must remain some doubt about the effectiveness of the grouting.

The prestressing in the internal columns was applied when the columns were lying flat on the ground but was adjusted when they were vertical and subsequently grouted.

## 2.2. Accessibility

None of the tendons can be inspected; even the anchorages are inaccessible. The anchorages for the nave ties and the baptistry lie behind the external stonework, and the void between the anchorage and masonry was filled with a dense mortar mix placed in a bird's-mouth shutter to stop water penetration. So even if high-level access could be provided it would require removal of significant fabric even to see the outside of the anchorage.

The ties in the nave trusses pass just below a high-level walkway that hangs from the trusses, so in theory it would be possible to drill into the ties here to find the tendons, but there seems little point since the ties are within the envelope of the building and are unlikely to get wet.

The porch ground beam is also inaccessible because it is buried below the steps that lead from the new cathedral to the old.

The lower end of each of the internal canopy columns is supported on a manganese-bronze block that holds the end of the concrete a short distance ($\approx$50 mm) above the floor; the extreme end of each strand, outside the anchorage, can thus be seen. But at the top, the grillage of beams that make up the canopy were cast on top of the columns and are integral with them, so the top anchorage is not accessible.

Inspection of the tendons is thus impossible. The condition of the tendons would have to be inferred from secondary effects, such as corrosion staining, cracking or dimensional changes consequent on the loss of some of the prestress. These issues will be addressed below.

## 2.3. Prestressing Systems

### 2.3.1. Freyssinet tendons and anchorages

At the time of construction of the cathedral the original Freyssinet system was in use, which relied on a two-part concrete anchorage: a cylinder with central hole into which a conical plug is inserted (Figure 5, Harris and Morice (1952)). The wires, typically 12 in number, are held in place by grooves in the plug, which is pushed into position by the double-acting jack used to stress the tendons. The circular arrangement of wires is maintained throughout the length of the tendon by a wire helix. An important corollary is that although each tendon is made up of a number of wires that are all stressed at the same time, they

are not twisted together as in a modern tendon and can be regarded as separate prestressing elements. The concrete cylinder is reinforced against bursting by high tensile steel wire spirals, placed towards the internal and external faces; the conical plug is not reinforced. Secondary bursting reinforcement is placed in the surrounding concrete. Provision is made in the anchorage for grouting and the central hole within the helix is designed so that grout can flow along the whole length.
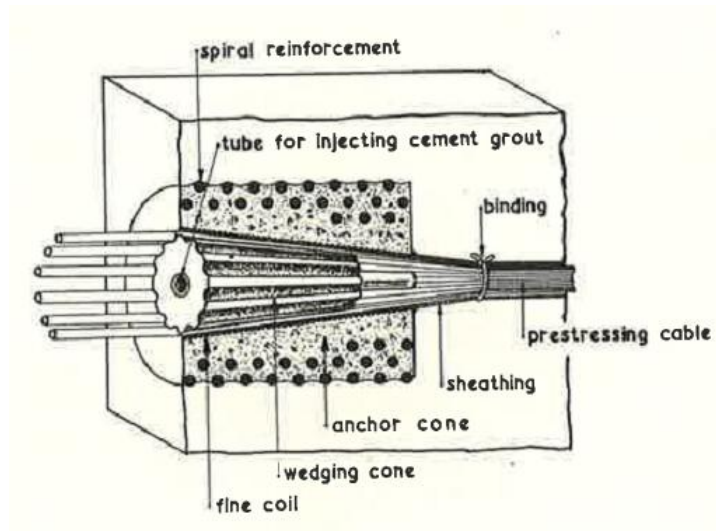


Figure 5: Freyssinet prestressing system.

### 2.3.2. Estimated Forces per Tendon

Confirmation of the tendons used in Coventry was provided by an original stressing sheet in the RCAHMS archive. It details the extensions applied to a set of prestressing tendons for one of the nave tie-beams. No further sheets were found. The sheet shows that the tendons were delivered by Richard Johnson & Nephew Ltd and were tested in October 1958, a year prior to stressing. They were described as "12 × 0.276in Freyssinet cables 100/110t/sq.in", confirming the precise version of the Freyssinet system used and the capacity per cable. Table 1 shows the key details. The stressing sheet shows that the tendons were stressed to 70% of the strength of the tendon (502 kN), but in the Arup calculations a lower figure of 60% was used (451 kN) when determining how many tendons were required, presumably to allow for losses.

### 2.3.3. Gifford-Udall System

A different system was used in the nave canopy columns. Ahm (1962a) describes how the precast elements were post-tensioned with four 0.7″(18 mm)

11

Table 1: Summary of prestress force for Tie Beam 605/3

| No. wires per tendon | | | 12 | |
|---|---|---|---|---|
| Diameter of wire | 0.276 | in | 7.01 | mm |
| Wire Area | 0.0598 | sq.in | 38.6 | mm$^2$ |
| Tendon Area | 0.718 | sq.in | 463.2 | mm$^2$ |
| Max. Capacity (stress) | 105 | t/sq.in | 1622.2 | MPa |
| Max. Capacity (force) | 75.4 | t | 751.4 | kN |
| Applied Load (stress) | 70.2 | t/sq.in | 1084.6 | MPa |
| Applied Load (force) | 50.4 | t | 502.4 | kN |

diameter Gifford-Udall tendons, one in each arm of the cruciform section. Although no information is available of the detailed construction of the strands, they were usually made from 7 wires twisted together and anchored by means of wedges and collets as in current practice (Andrew and Turner, 1985). Initial stressing sheets for the canopy columns indicate that they were stressed to a load of 14.10 tons (140.5 kN) when an extension of 2.75″(70 mm) was expected over a length of 18.85 m, which is consisitent with 7 wires, each of 5.9 mm diameter. It is likely that they were deformed to minimise the size of the strand. A figure of 140 kN will be used in subsequent calculations.

### 2.4. Structural Behaviour

At the time the cathedral was being designed, permissible stress codes were in use, so most design was carried out with nominal loads and allowable stresses. However, in prestressed concrete it would have been normal to carry out a check on the ultimate capacity of the section under factored loads, although no such calculations have been found. Since the objective of this paper is to determine the actual stress condition of the structure, no safety factors are included for the loads or the material strengths.

The structure was designed using Imperial units; where appropriate these values are quoted but all calculations here have been performed using SI units.

### 2.4.1. Roof Loading

Table 2 shows the assumed roof loading. The calculations use a concrete density of 12.5 lb/ft$^2$/in, equivalent to 2400 kg/m$^3$, for the structural concrete in the nave/baptistry and porch roofs. Non-structural lightweight aerated concrete (Celcon) was added to provide insulation. For the purposes of these calculations, as with the originals, the weight of the stiffening ribs was smeared into the general weight per unit area of the roof. The ribs have a closer spacing in the Lady Chapel and therefore the loading there was increased to 5.26 kPa from 4.73 kPa elsewhere. A value of 0.72 kPa of snow load was used, which is roughly equivalent to 1.1 m of snow depth. A catwalk, which is suspended from the apex of the roof and sits just above the tie-beam level, adds extra load, but this is

Table 2: Roof loadings

|  |  | lb/sq.ft | kPa |
|---|---|---|---|
| Nave Area | Concrete self-weight (4″) | 50 | 2.39 |
|  | Copper plating | 3 | 0.14 |
|  | Celcon | 15 | 0.72 |
|  | Snow | 15 | 0.72 |
|  | Beams | 16 | 0.77 |
|  | **TOTAL** | **99** | **4.73** |
| Porch Area | Concrete self-weight (3.5″) | 44 | 2.09 |
|  | Copper plating | 3 | 0.14 |
|  | Finishes (5.5″) | 18 | 0.88 |
|  | Snow | 15 | 0.72 |
|  | **TOTAL** | **80** | **3.83** |
|  |  | lb/ft | kN/m |
| Catwalk | Lights | 25 | 0.03 |
|  | Catwalk | 70 | 0.09 |
|  | Live | 240 | 0.33 |
|  | **TOTAL** | **335** | **0.45** |

minimal when compared to the other loads. The baptistry roof carries the fleche whose weight is small but because of its height can induce a significant moment due to wind. However, it is notable that no wind loading was considered on the building itself, so the snow load represents the only live load for which the structure was designed.

*Great Glass Screen.* The porch is divided from the baptistry by a glass screen that runs the full height of the building, and often referred to by its liturgical direction as the "Great West Screen". It seems to have been assumed that the glass at the bottom could not take the weight of the glass above, so the etched glass panels are held in a bronze framework, the mullions of which are supported at mid-height by six pairs of deltoid manganese-bronze ties to the porch roof outside and the baptistry roof inside (Blee, 2014; Crittall, 1962). Each cable was tensioned to 29.7 kN (see Figure 6) which supposedly took all the weight of the screen so that it was free to slide in a slot in the floor. The thickness of the concrete in the porch vault was increased locally where the cables are anchored. This arrangement allowed a lighter and more refined framework to be used while still being able to resist significant wind forces. These ties have been retensioned at least once since the cathedral has been in service (Clarke, 2000), most recently in 2006 when it was noted that they were completely slack. They have been tightened to act as braces to the centre of the screen but not to carry the weight of the glass which now passes directly to the ground (Fisher, 2014). The loss of tension is presumably due to creep in the concrete in the
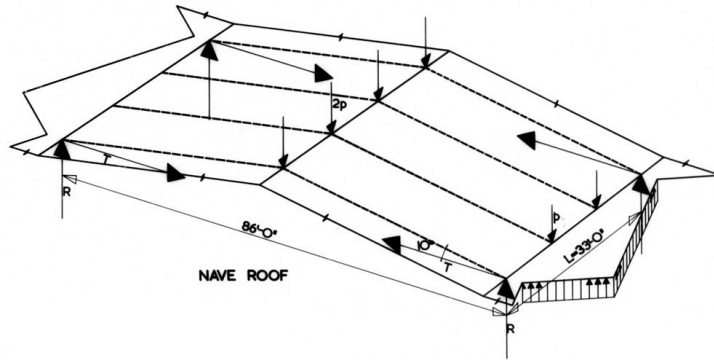
roof.



Figure 6: Underside of porch vault, showing the wires attached to West Screen

*2.4.2. Nave*

The nave roof has a shallow pitch of $10°$ which runs the whole way down the Cathedral to the Lady Chapel. It was designed as a series of flat plates that *"carry their load principally through extensional forces, tension and compression"* (Ahm, 1962b). Structurally, it consists of a $4''$(102 mm) thick concrete shell, designed to span longitudinally between reinforced ribs spaced at roughly 3.3 m centres. Every third rib has a prestressed horizontal tie, creating a "coat-hanger shaped" (Anon., 1962) tie-beam. These resist the horizontal thrusts created by the ribs, as they transfer the loads from the roof shell to the side walls. On either side of the roof, triangular reinforced slabs over the Hallowing Places sit on top of the side walls; these serve to transfer the horizontal force from the two ribs that are not tied to the ones that are (Figure 7a & 7b). Figure 8 shows a view from the catwalk that runs along the apex on the underside of the nave roof which allows maintenance access and from which lights are suspended. The shallow pitch of the roof can be seen, with a tie-beam in the immediate foreground and ribs further back. The wooden structure below the catwalk is the canopy, which is itself 20 m above floor level. The concrete shell spans between the tie-beams and ribs; the uneven lower surface is formed from acoustic tiles added later.

The amount of prestress required in the six horizontal ties depends on the roof area supported. The nave tapers towards the altar and therefore the number of tendons reduces from the baptistry (TB 603/1) to the Lady Chapel (TB 605/6). TB 603/1 also forms part of the Baptistry roof structure. The number of cables in each of the tie-beams was specified on the drawings (Table 3, using beam references given in Figure 9).

14

(a) Arup's 3D sketch of force transfer in roof
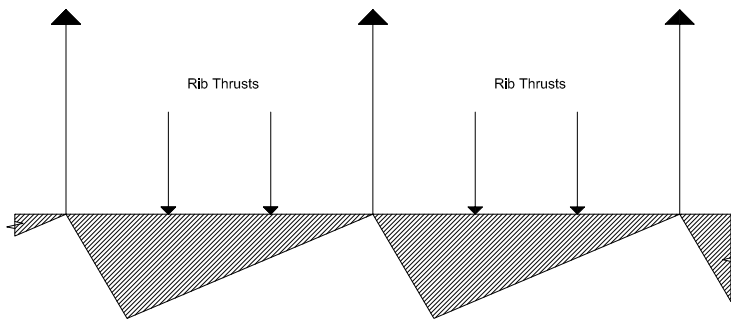


(b) Plan showing use of roof of Hallowing Places to transfer rib thrusts

Figure 7: Structural action of nave roof.

Table 3: Nave tendons

| location | cables |
| --- | --- |
| TB 603/1 | 10 |
| TB 603/4 | 5 |
| TB 603/7 | 5 |
| TB 603/10 | 4 |
| TB 605/3 | 4 |
| TB 605/6 | 4 |

15

Figure 8: Nave roof seen from catwalk above tie beams
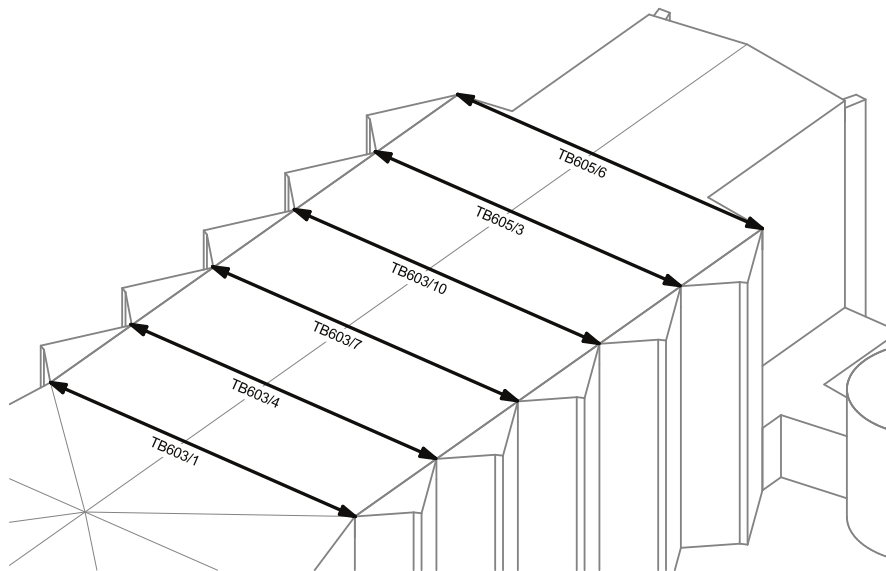


Figure 9: The tie-beam references

The structural system is straightforward and designed on the assumption that the truss was pin-jointed.

When determining the amount of prestress required a residual compressive stress of 50 lb/sq.in (0.35 MPa) was specified, even under the full design load. The number of cables was determined assuming that each tendon carried 60% of its capacity (= 451 kN), and the final prestress to be applied was calculated to achieve the desired result. For example, for the tie beam nearest the altar (TB605/6), where it was calculated that a force of 1533.3 kN was needed, 4 No. tendons were specified, with a prestress, after short- and long-term losses, of 383.3 kN applied to each.
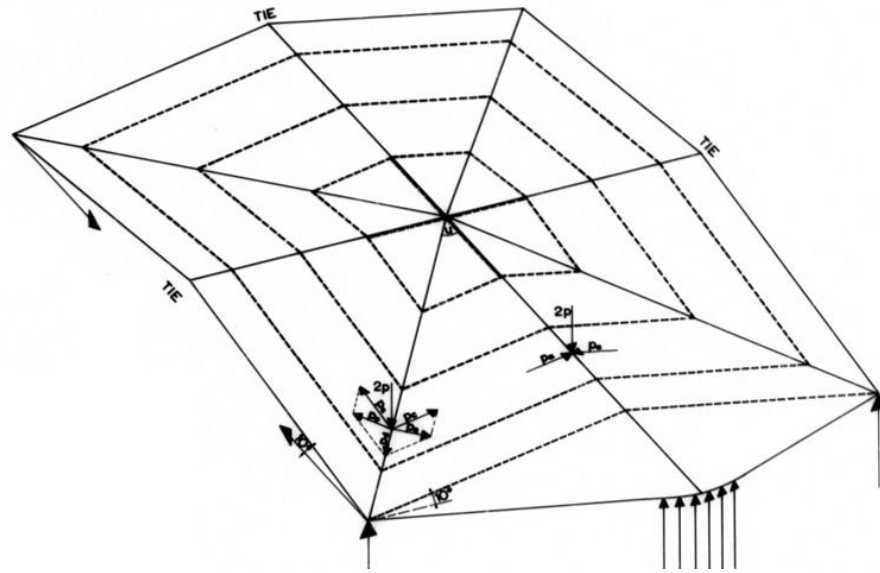
### 2.4.3. Baptistry

The roof over the baptistry at the southern end of the Cathedral spans $27 \times 30$ m and is the most complicated part of the roof structure. It is roughly square in plan, with a bulge to the east above the baptistry window. It is also a thin shell structure, with the shape of the roof defined by the intersection of the pitch from the baptistry window and the pitch from the nave. This creates a cruciform shape made from folded triangular elements, as can be seen in Figure 10a & 10b. Unlike the nave, where the prestressing spans across the roof, in the baptistry the prestressed ties run round the perimeter of the roof shell, inside the supporting walls. The function of the prestress is to maintain the rigidity of the folded shell shape, which would otherwise cause the roof to splay.

It seems likely that the design was changed at a late stage because it had originally been planned that all four sides of the baptistry would be prestressed:-

> "The three sides of the shell are tied by prestressed ties. At the fourth side the window made it necessary to avoid a horizontal tie. The tie was bent to follow the top of the window with the effect that the reaction is carried by the central part of the window instead of the flanking walls. This load from the roof increases the stability of the window structure." (Ahm, 1962b)

However, calculations in the Arup archive clearly conclude that the tension in the west wall of the baptistry could be carried by reinforcement without the need for prestress and the drawings show only reinforcement here. On the northern side of the baptistry the tie is provided by the first nave tie-beam (Table 4).

The Arup archive contains calculations showing the forces assumed to be acting in the baptistry. The structural concept is that the roof is a folded shell made up of individual triangles, with the tie forces round the sides of the baptistry keeping it in the folded position. The eight individual triangular elements were modeled as having triangular stress distributions acting on their edges, as shown in plan view in Figure 11. This analysis is based purely on equilibrium and does not take account of the various element stiffnesses. The resulting tie forces required for each of the four sides of the baptistry shell are given in Table 4. The final loads that the tie forces resist come from the baptistry
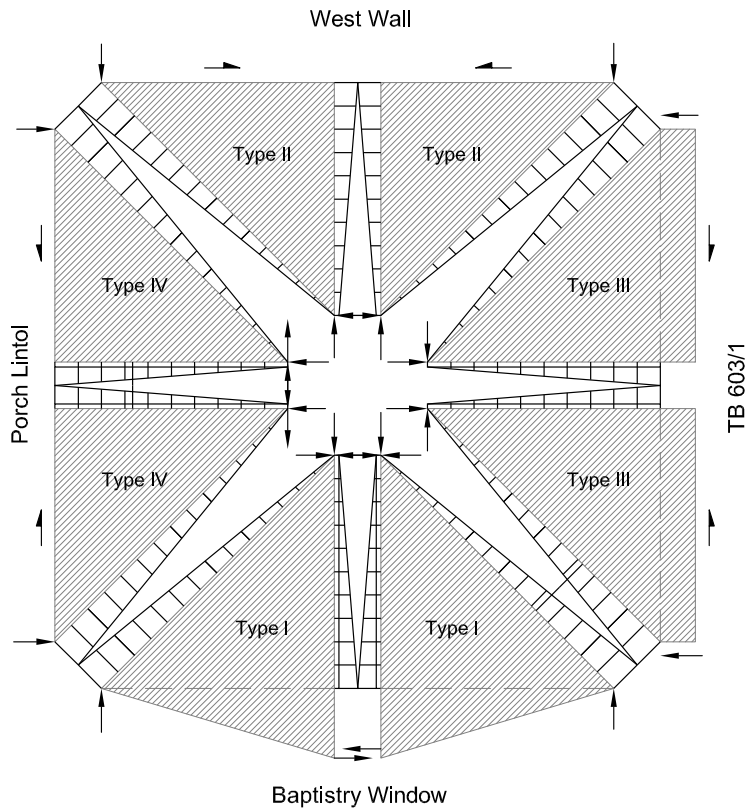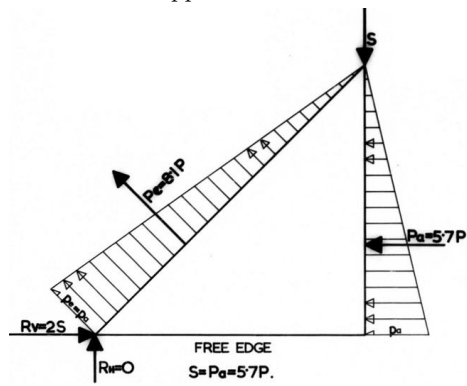
17

(a) Arup's 3D sketch
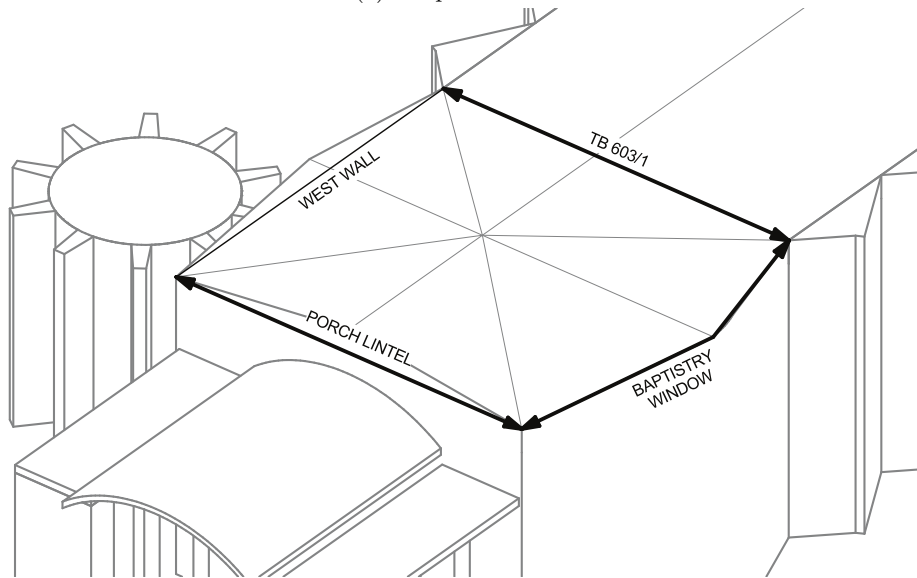


(b) Prestress locations

Figure 10: Baptistry Roof

(a) The assumed shell action of the baptistry roof, with internal stresses and tendon forces applied



(b) Triangular stress distributions shown on an individual element

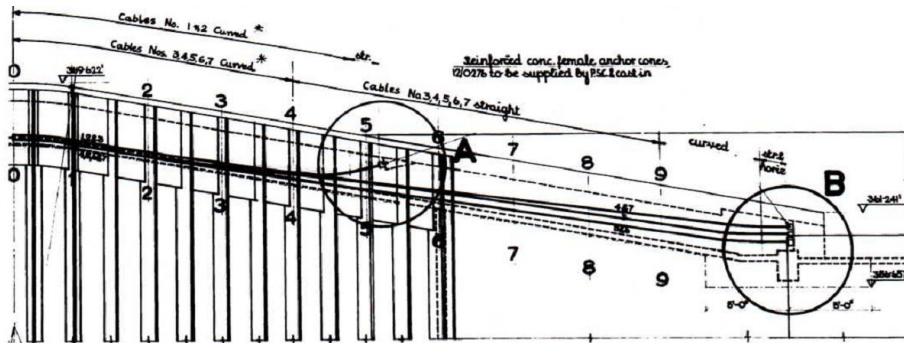Figure 11: Baptistry roof analysis

19

Figure 12: Arup drawing showing the curtailment of two shorter tendons in the baptistry window beam

shell action, the weight of the glass screen, the thrust from the porch roof and the thrust from the nave. There is also an allowance for the additional tension caused by wind load on the fleche, which it is calculated can cause variations of the tying forces by $\pm 112,000$ lbs. The shell "types" can be read from Figure 11 and relate to the four sides of the baptistry. The highest loads required are in the porch lintel (4,307 kN) and nave tie beam 603/1 (4,545 kN). The west wall has lower loads carried by ordinary reinforcement in the form of 21 high yield bars of $1''$ diameter.

Additional ribs or downstand beams were added to aid the stiffening effect and when viewed in plan are formed by three concentric squares as shown in Figure 10a. There is significant reinforcement in the ribs, and also along the diagonals in the roof itself, but this appears to have been supplied to resist local bending of the roof rather than making a contribution to its global behaviour.

Table 4: Required tensile forces in baptistry edge beams

| Type | I Baptistry window East | II West | III Nave TB603/1 North | IV Porch lintel South |
|---|---|---|---|---|
| Baptistry (lbs) | 561,000 | 764,000 | 701,000 | 740,500 |
| Screen (lbs) | 30,000 | | 23,000 | 56,000 |
| Porch (lbs) | | | | 226,000 |
| Nave (lbs) | | | 245,000 | |
| TOTAL (lbs) | 591,000 | 764,000 | 969,000 | 1,022,500 |
| TOTAL (kN) | 2,627 | 3,396 | 4,307 | 4,545 |
| No. of Cables | 7 | rebar | 10 | 11 |

The lowest force occurs on the east face but this wall is made up largely

of the baptistry window (Figure 10b). Since this window extends to the full height of the cathedral, and is bowed outwards, it can make no contribution to the support of the roof. If a tie was to be provided from the north-east to the south-east corners of the roof it would have been in the open air and would have been both obtrusive and exposed to corrosion. Instead, prestressing cables were run through a tie beam that passed along the top of this window. The drawings show that the curved baptistry window tie beam has seven cables, but only five of them run the whole length of the beam (Figure 12) and it is these five cables that provide the required reaction to the corners of the baptistry roof.

By running prestressing cables through a beam curved in both plan and elevation, the tendons cause a horizontal force tying the top of the window back into the roof, and also a vertical force down through the window mullions. The two additional tendons are anchored short of the ends of the beam and were presumably provided to increase the forces induced by the curvature of the beam. Estimates for the horizontal and vertical forces induced by the curvature can be found by simple equilibrium (Figure 13) and are given in Table 5. The horizontal force is much larger than the vertical force as the change in direction of the cable in this plane is greater ($50°$ as opposed to $20°$).
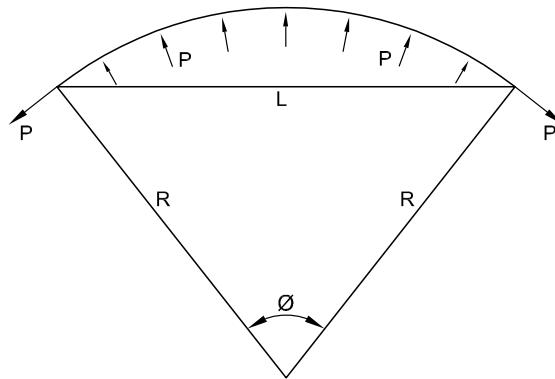


Figure 13: The assumed forces due to baptistry window curvature

Table 5: Forces from curvature of baptistry window tendons. F is the integral of the distributed forces p

|  | $\theta$ | R (m) | p (kN/m) | F (kN) |
| --- | --- | --- | --- | --- |
| Elevation | 20 | 7.50 | 378.4 | 985.6 |
| Plan | 50 | 6.99 | 406.0 | 2398.8 |

*2.4.4. Porch*

The porch connects the new Cathedral with the old (Figure 14b).

*"The roof over the Entrance Porch is a shell of the normal cylindrical type with three minor shells on either side. At one end it is carried by an inverted frame with the prestressed tie provided at ground level. This was done because it proved architecturally impossible to solve the problem of having the tie in its normal position."*
(Ahm, 1962b)

The minor shells on each side of the barrel vault are supported on eight slender columns and are structurally independent of the main shell, which is only $3.5''$(90 mm) thick, meaning that it requires horizontal thrusts for stability in a similar fashion to a masonry arch.
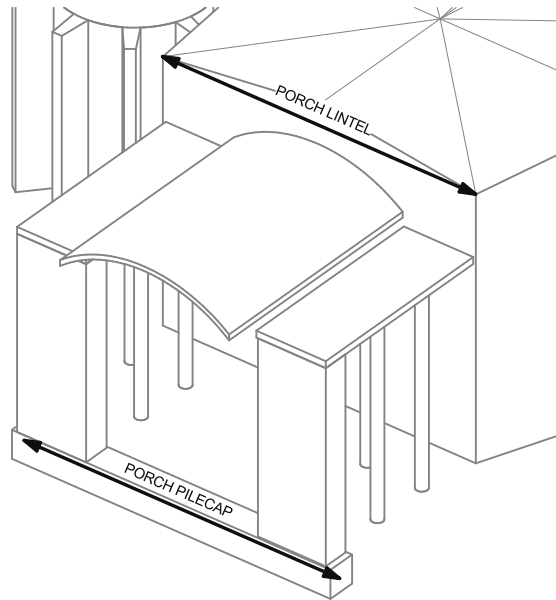
The "prestressed tie at ground level" is also known as the 'porch pile-cap' and is located under the steps that lead down from the ruins of the Old Cathedral, making visual inspection impossible (Figure 14a). The pilecap beam has twenty tendons, the highest number in any single location in the cathedral. No relevant calculations were found in any of the archives but it is clear that an estimate is needed for the lateral thrust from the porch roof onto the porch wall. The archives do include calculations for the forces onto the baptistry wall, so the same principles can applied at the other end of the vault. It was assumed that the barrel vault behaves as a beam, projecting from the south wall of the baptistry and supported on the porch side walls 16.5 m away. It extends a further 4.6 m over the steps to the old cathedral. Because of the shape of the barrel, it was reasonable to assume that it cannot rotate at its connection to the baptistry wall, but is simply-supported at its lower edges on the porch wall (Figure 15). This makes it statically indeterminate, with a moment and reaction diagram as shown in the figure.

The porch roof is actually a thin shell, so there will be significant thrust within the cross-section. The designer assumed that it behaved as a three-pin arch, with the line of thrust at the support directed towards the central pin as shown in Figure 15.

The roof loading in the porch was assumed to be 3.83 kPa (Table 2), and using the moment distribution given in Figure 15, the reactions at the supports $(V_1)$ can be found. To these must be added the loads from the wires supporting the glass screen $(V_2)$ (Figure 6) to give the total support reaction $(V_T)$. Using the relationship given in Equation (1), the horizontal thrusts $(H)$ can be now be calculated (Table 6). (The forces in the south wall have already been included in Table 4 above.)

$$\text{horizontal thrust } H = \frac{V}{2tan(\phi/2)} \tag{1}$$

The most logical place to resist the tie force from the porch roof would have been across the eaves, but this was presumably eschewed for architectural reasons. So the force had to be taken down to ground level by means of an inverted portal frame with a tie beam at ground level. The porch buttress wall is 1.3 m thick, and as with the rest of the cathedral has a largely unreinforced

22

(a) Porch prestress



(b) Porch showing link to cathedral ruins

Figure 14: Porch

54'
(16.5m)

15'
(4.6m)

0.065wL²

0.024wL²

0.444wL         -0.034wL²         0.556wL
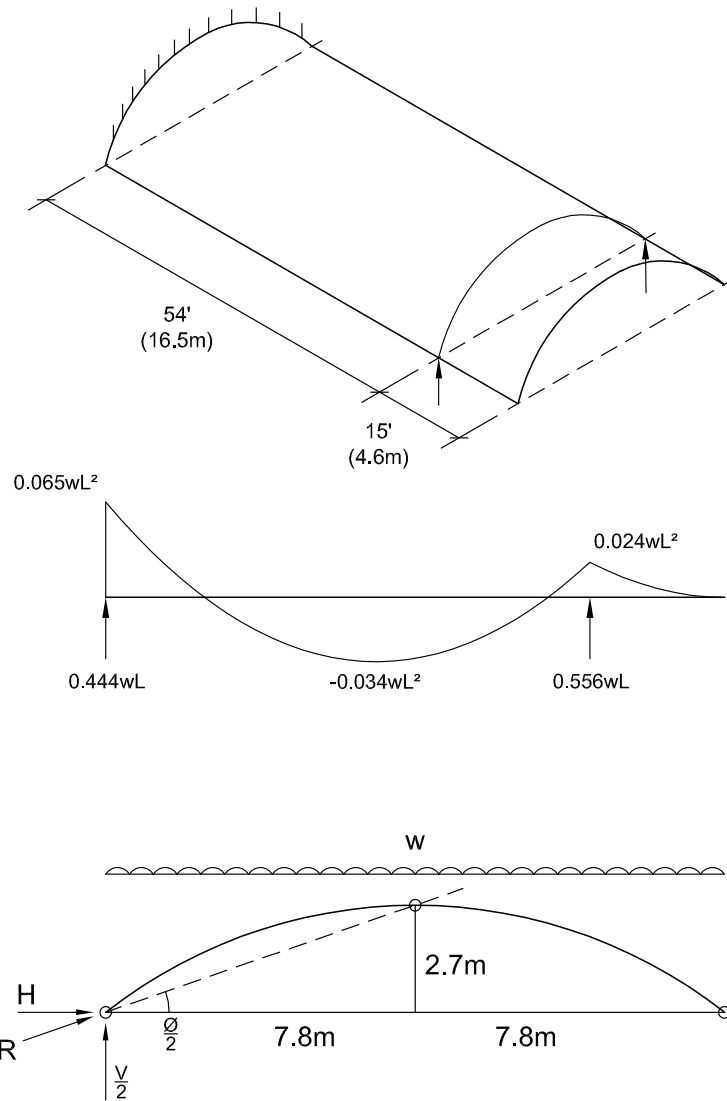
w

H
R

2.7m

Ø/2     7.8m          7.8m

V/2

Figure 15: Arup calculation of porch shell thrusts. (top) Overall porch geometry (centre) Bending moments assuming uniformly distributed load $w$ (bottom) Assumed line of action of sideways thrust

Table 6: Calculated thrusts applied to lintel and buttress walls

|  |  | lintel | buttress |
|---|---|---|---|
| Beam analysis | $V_1$ (kN) | 557.1 | 697.6 |
| Extra reaction | $V_2$ (kN) | 137.7 | 68.9 |
| TOTAL | $V_T$ (kN) | 694.8 | 766.5 |
| Thrust | H (kN) | 1012.8 | 1117.3 |

concrete core with stone facings. On each side it is supported on fifteen piles, together with two piles nearer the centre, as shown in Figure 16. It appears to have been assumed that the piles can carry no horizontal force so all the tension had to be carried in the tie. The system is statically indeterminate because some moment can be taken out through the pile cap beam ($M$), some by variations in the axial force in the 15 piles themselves ($Q$), and some by force changes in the two piles close to the centreline.

Taking moments about point A, which is at the intersection of the centre lines of the 15 piles and of the pile cap, shows that the roof loads and the weight of the porch walls cause a moment of 17,500 kNm. It is very difficult to estimate how this moment is shared between the beam and the piles, since it depends very heavily on the relative stiffnesses. In the worst case, if $Q$ is zero and the 2 central piles contribute nothing, all the moment has to be carried in the beam.

The pile cap beam itself is quite large, 1.8 m wide and 2.34 m deep, of approximately rectangular cross section (Figure 17). In the central portion of the beam the 20 tendons have an eccentricity of 0.71 m above the centroid. This places them outside the middle third, so the tendons themselves induce a net tension in the bottom of the beam. No figures are available for the actual prestress in the pile cap, but if each tendon has a residual stress of 60% of its capacity (= 451 kN), as assumed for the design of the nave tie beams, then the prestress force of 9,000 kN significantly exceeds the applied horizontal force from the roof of 1117 kN.

It has been noted elsewhere that the nave ties were designed with a minimum residual compression of 0.35 MPa, and if the same logic was applied to the pile-cap beam it must have been assumed that the hogging moment in the beam ($M$) lay between a minimum of 3,470 kNm (to avoid tension in the bottom) and a maximum of 9,320 kNm (to avoid tension in the top). So it is clear that the designers assumed that the beam had to be carrying *some* of the applied moment, otherwise it would have cracked on the bottom, but not all of it. It cannot have been assumed that the piles were rigid since the beam had to be carrying some moment, but neither can it have been assumed that they carried nothing. There was clearly no provision for adjusting the moments $M$ or $Q$.

There are many other uncertainties about these figures. The degree of fixity of the barrel vault to the baptistry wall is uncertain; less fixity would result in a higher vertical reaction onto the porch wall. The assumption that the horizontal and vertical reactions on the wall are in the ratio $2tan(\phi/2)$ is probably in error;
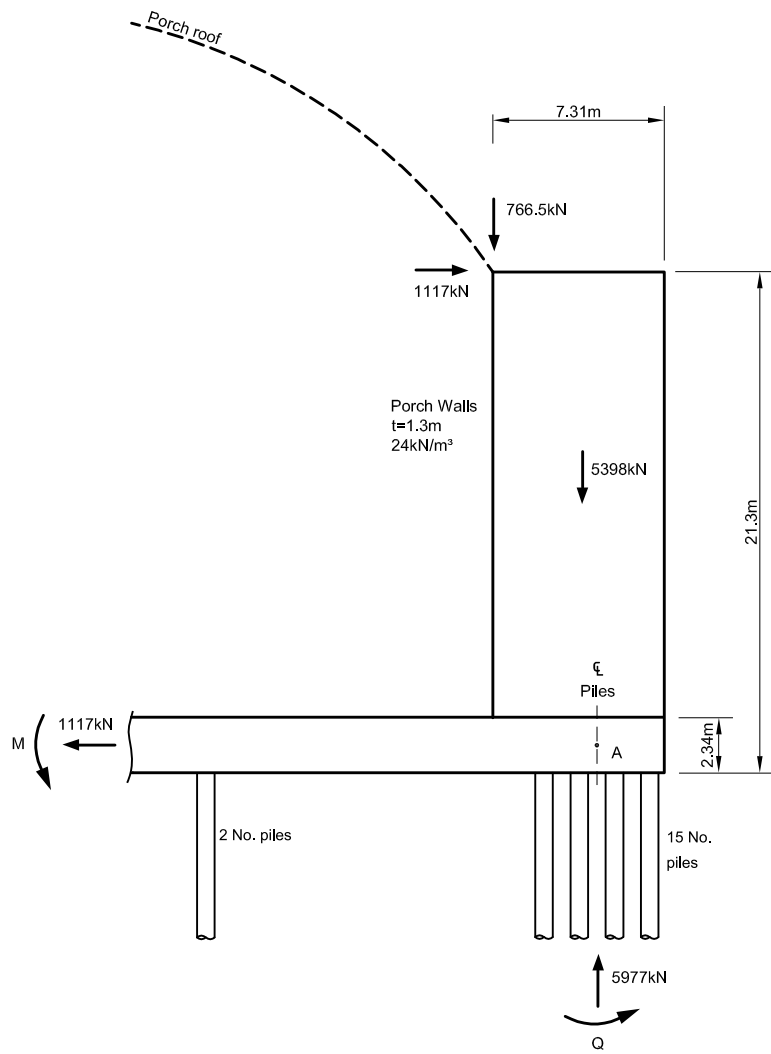
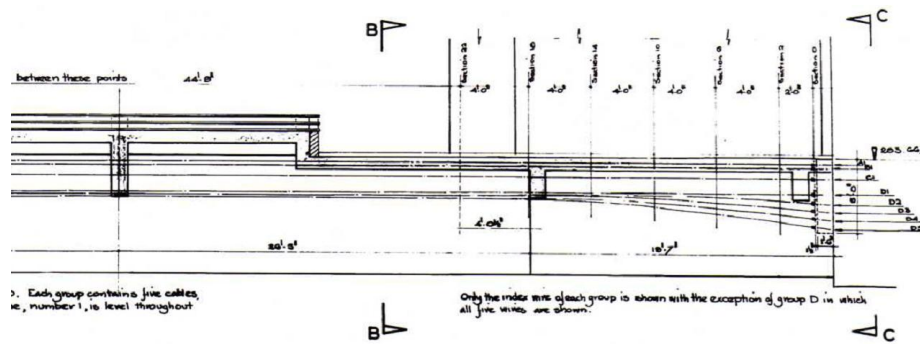Figure 16: Forces applied to porch buttress wall
Dimensions in metres

Porch roof

7.31m

766.5kN

1117kN

Porch Walls
t=1.3m
24kN/m³

5398kN

21.3m

℄
Piles

A

2.34m

M

1117kN

2 No. piles

15 No.
piles

5977kN

Q

**SECTION THROUGH PILECAP**

(a) Long section
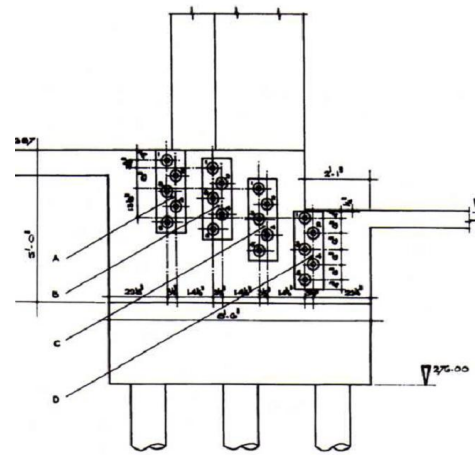


**ELEVATION C-C**

(b) Cross section

Figure 17: Original Arup drawings of tendon location in porch pilecap

27

the load on the barrel vault is fairly uniform, which would cause the line of thrust to much more closely follow the profile of the vault: this would reduce the horizontal component of the force. Because this force has a lever arm of about 23 m above the pile cap, there would consequentially be a significant effect on the moment in the pile cap. Reducing the horizontal thrust by one third would reduce the moment in the pile cap by 8,200 kNm. Cracking in the pile cap beam would make it more flexible, thus shedding moment into the piles (increasing $Q$ and decreasing $M$) and allowing some rotation of the buttress walls, which would also reduce the total applied moment.

So the conclusion is that the axial component of the prestress at about 9,000 kN is much higher than is needed to resist the horizontal thrust from the arch, but is there to reduce the tendency of the ground beam to crack. However, the actual stress state in the porch walls and the pile cap beam cannot be determined with any certainty.

### 2.4.5. Canopy columns

One of the features of the cathedral that sets it apart from a more traditional building is the use of very slender columns supporting the canopy that forms the visible ceiling. It is completely independent of the roof and consists of timber panels supported on a grillage of concrete beams, which in turn are supported by precast concrete columns of cruciform section that were precast in pieces and taper towards the base. The three precast elements that make up the columns were joined by means of epoxy adhesive, and stressed together using Tirfor winches until the adhesive had set, leaving a joint typically about 2 mm thick. The prestressing tendons were then installed and fully stressed. There is no other connection between the precast elements, so the prestress provided the only resistance to bending while the columns were being rotated to the vertical.

The grillage beams above the columns are reinforced and continuous, both with each other and with the precast columns. Starter bars were cast into the top segment of the columns and the tails of the prestressing strands also extended into the beams above. Once the falsework supporting the grillage beams was removed some moment would be induced in the columns at the top, so the forces in the column prestress were adjusted in anticipation (Ahm, 1962a). The legs of the cruciform columns are set at 45° to the central axis of the cathedral; the prestress in the two legs facing into the nave were reduced to a nominal 10 kN while that in the outer legs was adjusted. Unfortunately, there appears to be no record of what adjustments were made. The column tendons were then grouted but the annulus between the tendon and the duct was small and the tendon was not always central so there was some difficulty in completing this process (Bedford, 2014). On a number of occasions the grout had to be blown out with compressed air and the process repeated. Thus, it is believed that the grouting was effective, but the exact state of the prestress is unknown.

Despite the uncertainty of the amount of prestress, some estimate of the failure load can be made. Figure 18 shows the cross-section of the column. The leg dimension varies, being larger at the top, while the prestressing tendons are at a fixed distance from the outer edge. As the columns are slender, the primary
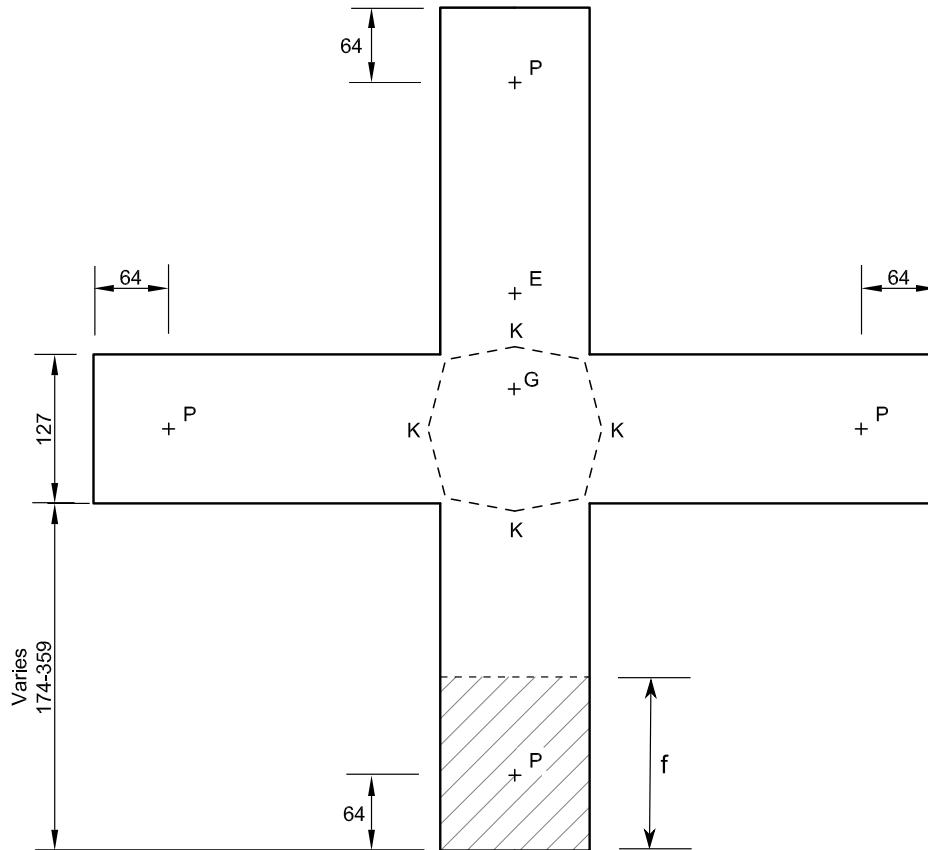
28

Figure 18: Canopy column cross-section. All dimensions in mm. The points labelled K define the central area within which the axial force must be applied to induce compression everywhere. Point E is the location of the centroid of the three remaining tendons if the bottom tendon completely corrodes away. The shaded area at the bottom would be cracked with only three tendons and the centroid of the resulting area would be at G.

concern is buckling, but since the stiffness varies over the height, the calculation of the buckling load is more complex. Estimates will be made on the assumption that the column is pinned top and bottom, and also that it is clamped at the top due to the continuity with the grillage beams. The exact situation is likely to lie between these two extremes. It will also be assumed, at least initially, that there is sufficient prestress to maintain the columns everywhere in compression, so the section is not cracked. The calculated values are shown in Table 7.

The crudest estimate is made by assuming that the column has a uniform section with the average leg length of 268 mm and a Young's modulus for the concrete of 30 GPa, (on the assumption that buckling is governed by the short-term modulus of concrete). This predicts that the Euler buckling load lies between 2,525 kN and 5,163 kN.

A better estimate can be made by taking account of the variation of the section with height and performing a Rayleigh analysis. This postulates a buckling mode and equates the strain energy with the work done by the load. The result will be an overestimate of the buckling load but by varying the shape to get the minimum buckling load a reasonable approximation to the true buckling load can be found.

If the column is pinned top and bottom, the buckling mode is assumed to be given by

$$v = a.sin\left(\frac{\pi h}{L}\right) + b.sin\left(\frac{2\pi h}{L}\right) \tag{2}$$

where $h$ is the position within the column, measured from the bottom, $L$ is the length of the column (19.3 m) and $a$ and $b$ define the shape. The absolute values of $a$ and $b$ do not affect the buckling load, but their ratio does since this defines the shape of the buckling mode. Setting $a$ arbitrarily to 1000 mm, the minimum buckling load is found to be 2,285 kN when $b$ is 90.5 mm. As expected, this value is lower than that from the uniform section and there is greater curvature in the more-flexible lower portion of the column.

A similar estimate can be made for the column clamped at the top, using a mode shape defined by

$$v = a.sin\left(\alpha_1 h\right) + b.sin\left(\alpha_2 h\right) + c.h \tag{3}$$

where the $\alpha$ functions have to satisfy the condition that

$$\alpha L = tan\left(\alpha L\right) \tag{4}$$

The first two solutions, corresponding to the lowest two buckling modes, give $\alpha_1 L = 4.4934$ and $\alpha_2 L = 7.7252$. By choosing $a$ to give the central deflection as 1000 mm and $c$ to satisfy the no-rotation condition at the top, the minimum buckling load can be found by varying $b$. The results are shown in the second column of Table 7, which again shows about 10% reduction when the effect of the taper is taken into account.

How do these values compare with the load on the column? The columns

Table 7: Buckling loads of canopy column (kN)

| Section | Pinned top and bottom | Clamped top, pinned bottom |
|---|---|---|
| Uniform section, fully prestressed | 2,525 | 5,163 |
| Tapering section, fully prestressed | 2,285 | 4,630 |
| Tapering section, 3 tendons only | 1,386 | 2,820 |

support a grillage of concrete beams that in turn support timber panels. All the columns are identical but the ones closest to the south screen in the baptistry support a larger area of canopy. The grillage beams are about 3.7 m long, on average 0.6 m deep and 127 mm wide. The weight from 25 of these beams is transmitted through the most heavily loaded column, giving a total load of 170 kN. The timber planks are about 75 mm square in cross section and are spaced about 75 mm apart. Twelve bays, each about 3.6 m square, are supported on the column, giving a load of about 5.8 kN. Since the canopy cannot be accessed and is totally within the building envelope, no live load needs to be taken into account. The self weight of the column is 71.6 kN, so the total axial load on a single column is 247 kN, well within the buckling load of the column, as would be expected from a good design. It should also be noticed that the beneficial prestressing effects of the ceiling weight, 176 kN, which is about 30% of the total prestress (600 kN), have been ignored in this analysis.

### 2.5. Discussion

#### 2.5.1. Comparison of Prestress Forces

Table 8 shows all the prestress forces in the cathedral calculated using the methods described here. These are shown in the first column, while the second column shows the forces found in the Arup archive, where available.

The results show that the estimates of the prestress forces match well those assumed in the original calculations, where these are known. In the absence of stressing schedules that would confirm the forces applied, it is assumed that these forces are the ones that were actually applied to the cathedral.

## 3. Corrosion of prestressing

The analysis so far has established why the structure is prestressed, where the prestress is, and how much prestress was present when the structure was built. The next question to consider is how that prestress would be affected by corrosion. Although the ducts are embedded in concrete and presumably passivated, the grouting may well contain voids. The anchorages are embedded in mortar, and lie behind the stone cladding, but it is not known whether

Table 8: Estimated prestress forces (kN)

| Location | Calculated | Arup archive |
|---|---|---|
| **Nave Area** | | |
| TB 603/1 | 4,417 | 4,307 |
| TB 603/4 | 1,832 | 1,848 |
| TB 603/7 | 1,739 | 1,728 |
| TB 603/10 | 1,638 | 1,616 |
| TB 605/3 | 1,450 | 1,452 |
| TB 605/6 | 1,533 | 1,533 |
| **Baptistry Area** | | |
| Baptistry Window | 2,838 | 2,627 |
| Porch lintel | 4,791 | 4,545 |
| **Porch Area** | | |
| Pile-cap | 7,910 | - |
| **Nave Canopy** | | |
| Columns | 600 | - |

any additional sealant or waterproofing layer was put onto their surfaces; certainly none was specified on the extant drawings. The outside surface of the stone is exposed to the rain, and since the walls are vertical the surface can be expected to dry out quickly. However, a visual inspection has shown that the roof is prone to flooding, especially in autumn when drains (Figure 19a) can get blocked with leaves and other debris. There are signs of water ingress and staining internally; Figure 19b shows such staining at the north-east corner of the baptistry, so clearly water can penetrate the structure. This location is immediately underneath the anchorage to tie beam 603/1 which serves to tie both the nave and baptistry roofs.

*3.1. Corrosion mechanisms*

Two corrosion scenarios can be envisaged.

- Brittle failure of the prestressing steel leading to sudden loss of the tendons

- Slow degradation of the prestressing steel which leads to a steady reduction of the prestress until the prestressed element can no longer carry the load

Page and Page (2007) identified three mechanisms for corrosion-assisted brittle failure.

1. Fracture promoted by local corrosion attack and hydrogen embrittlement.
2. Fracture as a result of stress corrosion cracking.
3. Fracture due to combined fatigue and corrosion.
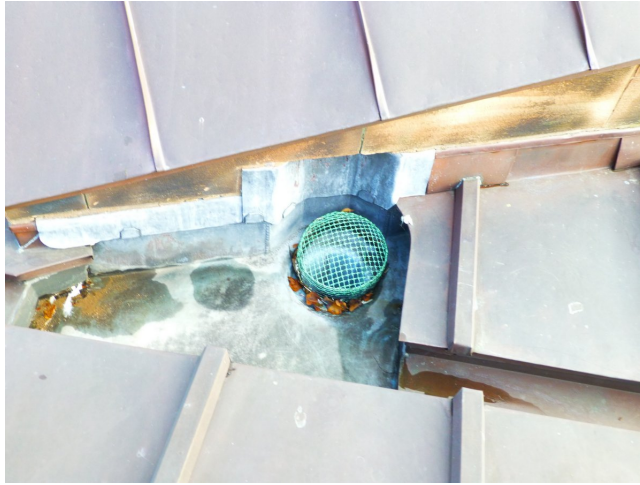
(a) Drain on roof


(b) Internal water staining

Figure 19: Roof drainage

The first is the brittle fracture of high-strength steel, particularly under the influence of rapidly applied tensile stress - i.e. failure during or shortly after the stressing procedure due to corrosion damage inflicted on the steel while it is on site and lying unprotected in ducts. The fact that the Cathedral has been standing for 50 years means that this is very unlikely. The third mechanism is also unlikely because the cathedral is not subject to significant cyclic loads.

However, hydrogen-induced stress corrosion cracking (H-SCC), remains a type of brittle fracture that is of concern. It is characterised by cracking without deformation and without apparent steel degradation or corrosion products (Page and Page, 2007). It is attributed to the presence of atomic hydrogen, which being molecularly small, is adsorbed by the steel into the interstitial spaces in the metallic structure. The atomic hydrogen may then recombine in these voids, increasing the molecular size and creating a localised pressure, leading to "hydrogen embrittlement". In the worst case scenario this can result in brittle failure of the tendon, as happened to the Berlin Congress Hall (Buchhardt et al., 1984).

The susceptibility of the alloy is important; quenched and tempered steels (which were more commonly used in continental Europe) are much more sensitive than the patented steels, which were more common in the UK. Tests on steel wire taken from Hammersmith Flyover (Austin, 2013), which is of a similar vintage, displayed the pearlitic structure that showed it had been patented. However, that steel, which was supplied by a different manufacturer, is known to be corroding, so no useful conclusions can be drawn about Coventry.

### 3.2. Nave Ties

The nave ties are probably the elements most susceptible to corrosion; Figure 20 shows half of one of the tie-beams. A more detailed view of the end anchorage is shown in Fig. 21a. There are several points to note. The section does not show the stone cladding outside the concrete, but it does show the space between the anchorage and the stone that was filled with mortar. This space tapers in the third dimension because of the serrated nature of the walls. The figure shows some the expected bursting reinforcement behind the anchorage, while other drawings show that there is only nominal reinforcement elsewhere, so the tie clearly relies on the prestress for its tensile strength.

Immediately above the anchors themselves is the parapet wall that extends upwards, but next to it is the low point on the roof, where the water collects before flowing to the drain. Figure 21b shows a photograph of the outside of the wall at one of the tie-beam anchorage locations. The approximate location of the anchorage is shown by the white circle. The stone is, to some extent, porous, as is the concrete, so it must be presumed that in certain weather conditions dampness will get to the face of the anchorage. What is not certain is whether sufficient moisture can penetrate past the anchor to allow corrosion to take place in the tendon itself.

It is also clear that failure of the waterproofing layer on the roof, which is provided by copper sheeting, would allow water to penetrate to the outside of
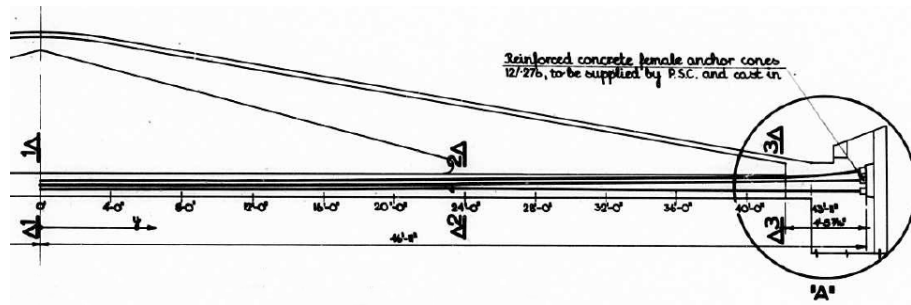
34

Figure 20: Half elevation of tie beam

the tendon duct, although whether it would penetrate further to the tendon itself is less obvious and depends on the effectiveness of the grouting.

The most likely position for corrosion is thus likely to be immediately behind the anchorage. If the tendon is properly grouted, the tendon can be expected to reanchor itself post-fracture after a suitable transition length - perhaps 30 tendon diameters. In a conventional prestressed beam, which carries load by flexure, that rebonding would probably be sufficient because the maximum flexure could be expected to occur at mid-span. But in a tension tie the force must be carried from end to end, so the anchorage itself is important. Some of the force to be reacted is imposed by the inclined rib at the tie location (Figure 20) and this could be applied along the length of the tie, so bond would be effective. However, each tie has to react the force from two other ribs, and tie 603/1 also has to carry the forces from the baptistry roof: these forces can only enter the tie via the roof of the hallowing place and then through the anchorage, which is thus critical.

### 3.2.1. Effect of corrosion on a Nave tie

How would one of the ties respond if corrosion occurred? The purpose of prestress is to ensure that the concrete itself can carry the load. Since there is much more concrete than steel, most of the load goes into the concrete; despite its greater modulus, there is very little change in stress in the tendon. Thus, provided the remaining prestress is sufficient to prevent the concrete cracking, the strains resulting from corrosion will be small. However, once the concrete cracks, all the external tensile load must be carried by the steel, which may have little reserve of strength and failure may occur with little warning. Because this is a tie, a "weakest-link" argument applies; load cannot be redistributed and "worst-case" scenarios must be taken into account.

As an example, consider tie 603/4, which is the longest tie that just supports the nave roof. It is nominally $15''$ square with five prestressing tendons and four $\frac{1}{2}''$ rebars at the corners. The assumed properties of the materials are shown in Table 9; variations in the values chosen would alter slightly the detailed results that follow but they do not change the general conclusions. A low modulus is
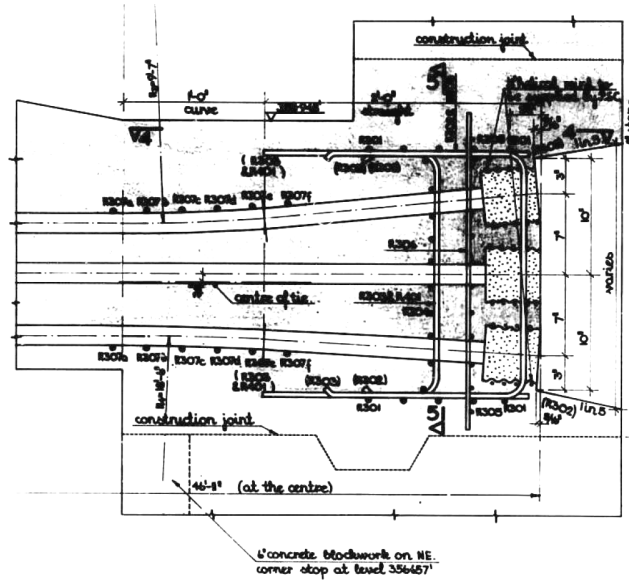
(a) Details of tie beam anchor



(b) Approximate location of tie beam anchor as viewed from outside

Figure 21: Tendon anchorage at end of a tie beam

used for the concrete because most of the loads are permanent and they will be affected by creep. It is assumed that each tendon was stressed to 451 kN, giving a prestressing force of 2,255 kN or about 14.5 MPa in the concrete. Each tendon has a strength of 751.4 kN.

Table 9: Assumed material properties in tie 603/4

| Material | Area mm$^2$ | Strength MPa | Modulus GPa |
|----------|-------------|--------------|-------------|
| Concrete | 142,300 | 3 (tension) | 8 |
| ps steel | 2,316 | 1,622 | 200 |
| rebar | 507 | 250 | 200 |

Figure 22 shows the expected strain response of this tie to an externally applied axial tension. The dead load of the roof causes a tension of 1,510 kN, but this increases to 1,781 kN when the snow load is added. Several lines are shown on the figure, corresponding to different numbers of effective tendons; the upper solid line shows the expected response as built. If the external tension were somehow increased, the concrete would go into tension at about 3,100 kN, or almost double the normal dead load. If the concrete immediately cracked, it would follow the solid line until the rebars yield at a tension of 3,800 kN, and the tie would continue to carry load until the prestress yielded at about 3,900 kN. This ignores the tensile capacity of the concrete, the effect of which is shown as a higher dotted line. The original stiffness of the tie would be maintained until the tensile strength of the concrete was reached, at which point the section would crack very quickly. It is assumed that all tension stiffening would have been eliminated by the time the rebar yielded.

This result shows that the section almost certainly had a large reserve of strength as originally designed, which is to be expected. But if corrosion were to take place, how much corrosion must occur before the effects become apparent, and how much reserve is there then before the structure would collapse?

Figure 22 also shows similar lines for different numbers of tendons; these reflect how the tie would behave if some tendons were removed. With four the section would still be fully prestressed under the dead and snow loads, but if only three tendons remained the concrete would be in tension under snow load. With two tendons it would be severely cracked and would be relying on the rebar and the very high strength of the remaining prestressing tendons.

What could be observed if the tendons corroded slowly over time? Figure 23 shows the same data plotted in a different way, with two lines showing the strains in the tie, measured relative to the initial condition after prestressing, for the two loading cases: dead load only and dead load plus snow. The as-built condition is at the extreme right of the diagram, when all prestressing losses have occurred but there has been no corrosion, so all five tendons are effective. The tie has a tensile strain of 0.000886 relative to its prestressed but unloaded state. It would only increase by a further 159 $\mu\varepsilon$ if the full snow load were applied.
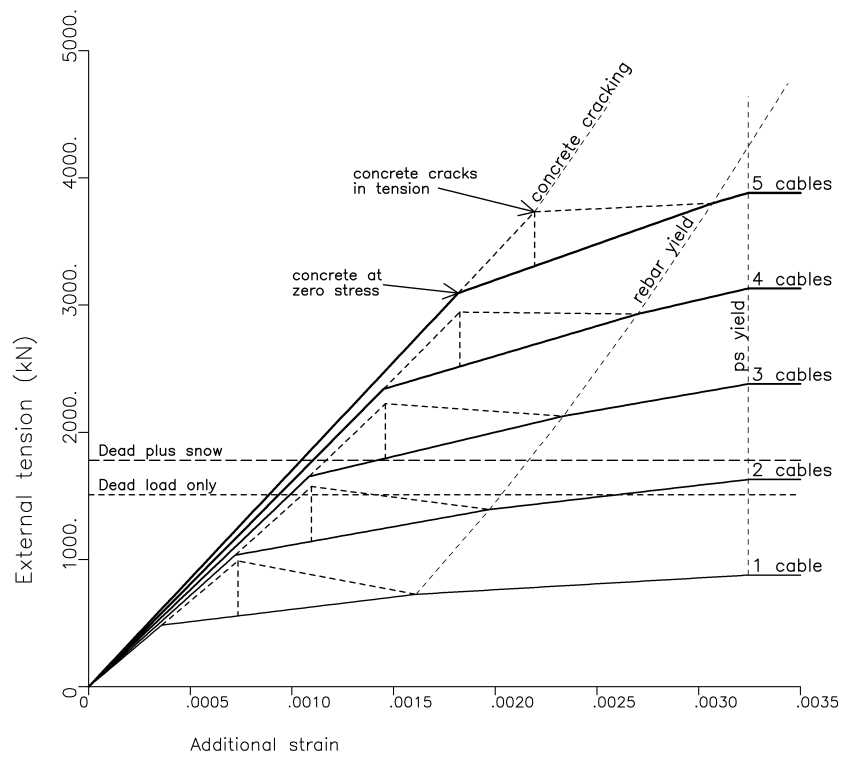
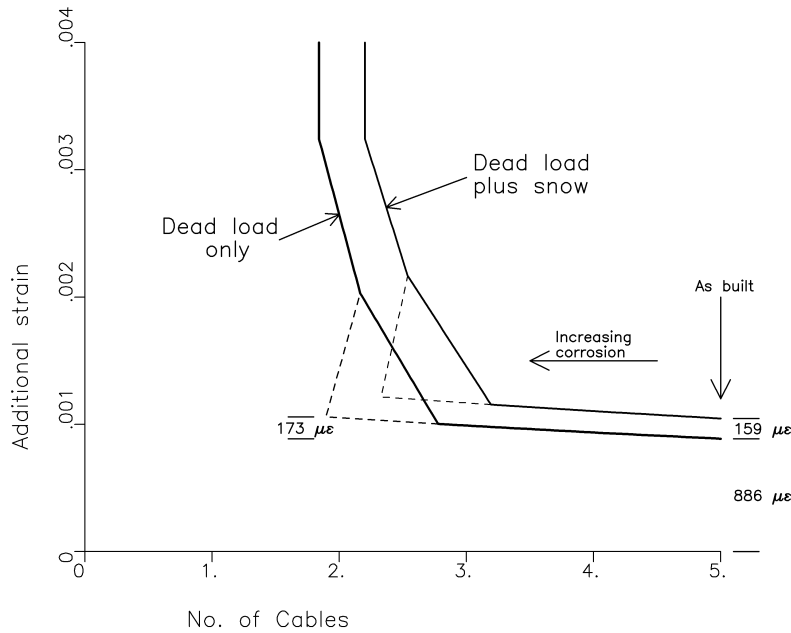Figure 22: Variation of stress in tie 603/4

Figure 23: Variation of strain at constant load for Tie 603/4

If corrosion occurs, the number of effective tendons decreases, so the loading moves to the left on the diagram; fractional loss of tendons is possible, at least locally, because individual wires can break or lose sectional area. As with Figure 22, the solid line shows the response if the tensile strength of the concrete is ignored, with the dotted lines showing the likely effect of tension stiffening. With the tie subject only to the permanent dead load, the strain would only increase by 173 µ$\varepsilon$ when the concrete cracks in tension, by which time only about 1.9 of the tendons would be effective. Although some of the prestress is being lost, enough is present for the concrete still to carry the load.

The important conclusion from this result is that the effect would almost certainly not be noticeable. Tie 603/4 is 28.1 m long, so a strain of 173 µ$\varepsilon$ corresponds to a change in length of 4.9 mm. But that would only occur if the tendon were unbonded, or to corrode uniformly over its whole length, which is very unlikely. If the broken wires reconnected with the concrete over a length of 1 m, which seems not unreasonable, the change in length of the tie would only be 0.17 mm. This would not be noticed and is the same extension that would be caused by a temperature change of only 0.6°C, assuming that the concrete has a coefficient of thermal expansion of $1 \times 10^{-5}$ per °C.

If corrosion continued, with more tendons corroding, the section would move further to the left on Figure 23. The strain would increase very rapidly because the rebar and the prestressing tendons would be unable to take up the tensile stresses released when the concrete fails in tension. A similar phenomenon was

39

noted by Wheen (1979) who carried out tests on prestressed concrete ties with different combinations of prestressed and untensioned reinforcement, with the aim of determining the stiffness of the ties after the concrete had started to crack. As would be expected, in most cases he found that the ties did not immediately lose the stiffening effect of the concrete when the first cracks appeared, but by the time an additional strain of about 0.001 had been applied, the stiffness was effectively that just of the steel bars themselves, with no effective contribution from the concrete. This additional strain is about the same as that needed to make the rebar yield in the Coventry ties. In some cases, however, when there was very little untensioned reinforcement, Wheen noted that failure took place as soon as the concrete cracked because the additional capacity of the steel was less than the tension being released by the cracking concrete.

This is why failures in prestressed concrete structures are reported to occur without apparent warning, as happened to both the Berlin Congress Hall and Ynys-y-Gwas bridge in south Wales (Woodward and Williams, 1988). That bridge was inspected shortly before it failed but no warning signs were seen, even though it is almost certain that the vast majority of the corrosion that was present at the time of failure had already occurred.

### 3.3. Porch Lintel

The prestressing tendons in the south wall of the baptistry resist the thrust from both the baptistry roof and the porch roof. This wall is probably the most exposed to driving rain, which might mean that corrosion is more likely to occur here than elsewhere. But the eleven tendons are embedded in a concrete wall rather than a beam (about 4.5 m deep and 0.66 m thick). There are a large number of small rebars (typically forty $3/8''$ dia) parallel to the tendons, and although they were probably designed as nominal reinforcement they could make a significant contribution. The relatively large area of concrete would also be capable of carrying most of the applied load without assistance, but only if uncracked.

### 3.4. Baptistry Window

The tendons in the curved beam over the window on the east face of the baptistry are anchored with a similar detail to those in the nave tie-beams, and must be presumed to be exposed to a similar risk of corrosion. Five tendons extend the full length of the beam and their anchorages are critical for resisting the corner forces imposed by the baptistry roof. The two additional tendons in the central curved region appear to be present to induce forces that hold the tie back into the baptistry roof and to apply of downward force through the window mullions. For these tendons, distributed bond through the grout would be effective.

The edge beam is larger than the nave tie beams, with six rather than four reinforcing bars. It can be expected to behave in much the same way as the tie beams, in that it would show very little response to corrosion while the concrete remained in compression. Unlike the nave ties, however, there is an alternative

load path through the baptistry roof itself, which contains a relatively large number of small reinforcing bars. The detailed layout of these, and the extent to which they have enough anchorage to be effective if the window prestress fails cannot be ascertained from the microfilm reinforcement drawings.

### 3.5. Porch Pile Cap

This beam is below the paved ground surface, which is itself exposed to rain, but is in the made ground above the piles. Thus it can be expected to get wet occasionally but to be relatively free draining, which could mean that it is wetting and drying, making it more vulnerable to corrosion. As has been explained above, the detailed stress state in this beam is impossible to determine, so it is not known whether or not it is cracked. There is plenty of prestress to resist the tie force that is applied, but corrosion would have a significant effect on the ability to carry moment. This makes it very difficult to know the actual condition.

If corrosion did take place it would probably occur if the beam were cracked on its top surface due to the applied hogging moment from the porch roof loads. That would make the pile cap more flexible, which would then mean that more moment had to be carried by the piles which would cause the porch buttress walls to rotate outwards slightly. Whether or not this was critical, or would be noticed, depends on an analysis of the piled foundation, which is beyond the scope of this paper.

### 3.6. Nave Canopy Columns

The canopy columns in the nave are thought to be at least risk of corrosion because they are entirely within the fabric of the building so are dry, and they were grouted in the vertical position so the tendons should be in an alkali environment. That alakalinity will eventually break down as there is moisture in the air, so on very long time scales corrosion might occur and it is worth considering how corrosion would manifest itself.

The most critical loading case for slender columns is likely to be buckling. If corrosion were to occur some of the prestress would be lost but the amount of prestress in a column does not affect its buckling load if the tendon has to move with the column, as happens here. However, if one of the four tendons in a column were to corrode away completely, the remaining prestress could induce tension in one leg of the column, which might then crack. This would reduce the buckling load since the column flexural stiffness would reduce. It is important to know by how much.

Suppose that one of the four tendons corroded completely. Figure 18 shows the Kern points (K) for the section which define the area within which an axial force has to be applied if the section is to remain everywhere in compression. Also shown in the figure is the location (E) where the resultant of the three remaining tendons would act if one tendon (the bottom one in the figure) completely failed. E lies outside K so the prestress would cause the section to go into tension, and if it cracked completely in tension it would crack to a depth $f$. The

stiffness of the column would then reduce, making buckling more likely. Because the section varies over its height, $f$ varies with height, being about 47 mm at the bottom but nearly 200 mm at the top. The reduction in stiffness varies from 24% at the bottom to 47% at the top, and when the Rayleigh buckling analysis is repeated for this case the critical load drops to 2,820 kN if clamped at the top (Table 7), which is still significantly in excess of the applied load of 247 kN.

The assumptions made here are quite severe and conservative. It is unlikely that the prestress would be lost for the full length of the column, and the concrete does have some tensile strength, so less of the section would crack. The axial component of the canopy load has been ignored, which would also have reduced the amount of cracking. The assumed Young's modulus for the concrete is quite high but is not unreasonable for short-term effects on mature precast concrete, such as those that occur during a buckling increment. However, because the loads are permanent, the possibility of these causing creep (equivalent to using a lower elastic modulus) might mean that a lower modulus should be used. The P-$\Delta$ effect has been ignored, which is non-conservative, but the lateral deflection caused by integrating the curvatures induced when one tendon is removed is only 23 mm so the effect would be small.

The big uncertainty is the amount of prestress left in the columns after the readjustment that took place following erection, but the saving grace is that the grillage of beams is continuous over all the columns, so buckling of one column would simply cause its load to be redistributed to the remainder. It can thus be concluded that the canopy columns are unlikely to be significantly affected by the loss of one tendon, and that corrosion is anyway unlikely.

## 4. Possible monitoring

If Coventry Cathedral were suffering from corrosion of its prestressing, how would one know? It is clear from previous experience that only a small amount of steel needs to corrode before catastrophic consequences can occur so there would be few tell-tale marks from rust staining, as commonly occurs when rebar corrodes. In recent years it has become fashionable to invest in so-called "Structural Health Monitoring" (SHM): are there any techniques that could usefully be employed at Coventry?

Some monitoring did occur during construction, in particular of the baptistry roof. It was planned to measure strains using Demec gauges at about 100 points, shown on an archived plan, sometimes on both the top and the bottom surfaces. At some locations the gauges were arranged as equilateral triangles with a 16″ baseline that allowed strains in all directions to be determined. These must have been measured while scaffolding was still in place to provide access, and were presumably monitored before and after the falsework was removed. The gauges on the top surface of the roof were also monitored when the fleche was installed. However, if a report was produced, no copy of it seems to have survived. Since the cathedral was designed before the advent of computer analysis, and the design methods employed were essentially lower bound methods to ensure structural adequacy, there was no predicted set of strains with which

42

comparison could be made. The purpose of the monitoring was to check that the deflections were not excessive; the strain changes due to loads were found to be small by comparison with effects of temperature and humidity variations (Johnson, 2014).

There is some correspondence showing that a small number of vibrating wire gauges were to be provided that could be monitored remotely after normal access was not possible, but there is no indication of whether they were actually installed. Vibrating wire gauges can be stable for many years and would have allowed the effects of creep on the roof to be measured. Even if they could be located, they are of little value without records of their calibration factors and the initial readings.

Can anything be determined from a visual inspection, and if so, how would it be done? Exposing the external surfaces of the anchorages by removal of the stone cladding and covering mortar is possible, but would be detrimental to the external appearance of the cathedral. It is possible to drill into tendons to determine how effectively they were grouted and to inspect a piece of tendon, but what would that achieve? Only a small length of tendon could be exposed in this way, which would give little information about the rest. It would also risk damaging the tendons in the process and could provide a route in for future corrosion. So it appears pointless to damage the structure for no useful purpose.

Technology has moved on significantly since the cathedral was built, so is it possible to monitor the structure by fixing instrumentation to the outside of the structural elements? It must be borne in mind that this would be potentially a very long term study; there is no evidence that the structure is deteriorating now and nothing might happen for a very long time. The cathedral is almost certainly much more robust than any electronic device used to monitor it. So any system would have to allow the measuring device to be replaced while still preserving the integrity of the measurement.

There is no technology available that can measure stress, only the strains or the changes in length that a change in stress causes. Strain gauge measurements of concrete beams are often of little value because the results depend on how far away the gauge is from a crack, and almost by definition the location of these is not known when the gauge is installed. The most likely scenario for a failure would be if one prestressing wire (not tendon) snapped. If the wire was unbonded there would be a change in strain along the whole length of the tendon, so the effect could be measured with strain gauges or by measuring the overall length of the element very accurately. However, if the wire could rebond itself the effect would be localised and could only be detected if a gauge had fortuitously been placed at the same section as the wire break.

From the results in the previous section, failure of one complete tendon in one of the nave ties would cause a tensile strain change of only 56 $\mu\varepsilon$ at the crack location and failure of one wire would cause a strain change of only 4.7 $\mu\varepsilon$. There have been various techniques based on the use of optical fibres for measuring strain, but most do not offer the combination of accuracy of strain measurement, small gauge length, and the ability to measure strain along the fibre. However, recently a new technique making use of Rayleigh backscattering

has become available, which claims to be able to achieve 1 με accuracy using a 10 mm gauge length at 10 mm spacing using optical fibres up to 70 m long. A good discussion of the various techniques is given in Hoult et al. (2014). This resolution would be suitable but the equipment to carry out the measurements is at the moment prohibitively expensive and while short-term experiments have taken place it is not yet clear how stable the optical fibre system is over time, how sensitive it is to temperature changes or how it responds to creep.

An alternative would be to continuously to monitor the length of critical elements, such as the tie beams. If fixed points could be attached to the structure at each end, and a device installed that could monitor the distance between them accurately over time, any subsequent movement of the building could be monitored. This has the advantage that failure anywhere in the element would cause an effect, but that effect would be small and it would not be possible to detect *where* the failure occurred. The relevant changes of length would be of the order of a tenth of a millimetre over lengths of 28 m, as shown in Table 10, and crucially, they are orders of magnitude smaller than the changes in length caused by temperature variations.

Table 10: Changes of length of tie 603/4

| Loss of | over length of | Length Change |
|---|---|---|
| One tendon | 28 m | 1.56 mm |
| One tendon | 1 m | 0.06 mm |
| One wire | 1 m | 0.005 mm |
| Temperature change 10 deg C | | 2.8 mm |
| Snow load | | 4.45 mm |

If a strand or a wire failed, a step change in the distance could be expected to occur. How big a change would depend on what failed, and also on the quality of the grout, so it would probably be difficult to say precisely what had happened. But it might be possible to say that *something* had happened. Physical measurements of this accuracy over this distance are very difficult. It would be possible to support a wire outside the element being measured to transmit the displacement from one end to the other so that it could be measured by an LVDT or similar device, possibly using a lever arrangement to magnify the movement. Manufacturers quote a dynamic range for LVDTs of about 20,000:1, so a gauge with a range of 5 mm should be accurate to about 0.25 microns. However, allowance would have to be made for thermal effects in the wire itself and in the structure being measured, which would have different time constants, and since the LVDT would rarely move it would be difficult to ensure that it was not affected by dust.

An optical device, such as a laser rangefinder, could also be used. Most of these work using "time-of-flight" and quote accuracies of a couple of millimetres,

which is clearly insufficient for the present purposes. Interferometers work to the accuracy of the wavelength of the laser light being used, so therefore of the order of 1 μm, but need high precision mirrors at both ends of the distance being measured and are very sensitive to relative rotations. Larger displacements are measured by counting the number of interference fringes, so the measurements would need to be continuous. An optical interferometer over such a long baseline would also suffer from drift due to refractive index changes in the air from pressure and temperature changes; a 10% change in barometric pressure over a distance of 28 m, would be equivalent to a length change of 0.84 mm. Compensation for this would be tricky.

What has been successfully monitored elsewhere? Transport for London were aware that there was some corrosion on Hammersmith Flyover in London and had instituted an acoustic monitoring system that was actively listening for wire breaks. When the number of confirmed wire breaks reached a threshold value, a pre-determined action plan was brought into effect. The extent and severity of the problem were investigated in more detail that led to the closure and strengthening of the flyover. Acoustic monitoring has to be continuous; once the sound has gone it leaves no trace. The system works by detecting the energy that is released when the wire snaps, which implies that it must be unbonded, at least over some distance; slow corrosion of a bonded wire would release very little energy. There is also the difference that corrosion was known to be happening at Hammersmith and there were many more wires than at Coventry. At critical locations there were ninety six 19-wire strands, giving 1824 wires. In tie beam 603/4 there are five 12-wire strands, giving only 60 wires, so each wire is proportionally much more important in Coventry.

The one thing that could be monitored fairly easily would be the out-of-straightness of the canopy columns, although it has been concluded above that these are unlikely to corrode. Excessive deflection, indicating significant curvature, or changes in the deflection with time, would indicate a cause for concern.

It is clear from the discussion above that it would be very difficult to set up a monitoring system. Corrosion is likely to be very slow and the distinct wire breaks would be few and far between. The strain changes caused by wire breaks would be small and very difficult to distinguish from the much larger and more frequent thermal changes.

## 5. Conclusion

This study has allowed an evaluation of the use of prestressing systems in Coventry Cathedral. Two systems resist the outward thrust of the roof in the six ties that span the Nave, and in two walls of the Baptistry walls. A third system is in a ground beam that forms part of an inverted portal frame to resist thrust from the shell roof over the porch. The final system is in the slender columns that support the architectural canopy over the Nave.

There is no evidence that there is any corrosion taking place in the tendons in the Cathedral, but none can be inspected without causing damage to the fabric of the structure. Thus it is impossible to say that corrosion is *not* occurring.

The prestressed ties in the Nave have very little untensioned reinforcement and it has been shown that as a result the ties would have very little reserve of strength once sufficient loss of prestress had occurred to allow the concrete to crack. A corollary is that there would be little visible evidence that loss of prestress was taking place. A similar condition applies in the prestressed beam in one of the baptistry walls.

The stress state in the prestressed ground beam is difficult to ascertain because the system is statically indeterminate, with an unknown moment being taken out through the pile cap. This makes it very difficult to predict either the likelihood of corrosion or its effect.

The prestressing in the canopy columns primarily ensures that the section remains uncracked, thus ensuring that it has the stiffness to resist buckling. The worst effect of corrosion would be if the prestressing became asymmetric, but even in that circumstance it is unlikely that the columns would fail. The fact that the columns are entirely within the enclosed envelope of the building means that corrosion within these elements is thought to be unlikely.

Consideration has been given to what monitoring could be installed to give warning of any potential corrosion. The strain changes caused by a single tendon failing would be small, and any measurement of length changes would be very difficult to distinguish from the much larger changes caused by temperature variations in the building. An alternative would be to monitor the acoustic emissions caused by a tendon failure, but the number of tendons, or even wires, in the cathedral are small, so failures would be very rare. Both length and acoustic monitoring would need to be continuous because they monitor failure events rather than giving information about the actual state of the prestress.

From the available literature, and discussions with those involved, it is clear this was seen as a prestige project by all concerned, and that care was taken to ensure that the elements were as durable as possible; in the choice of materials, in the design and on site. The principal risk to the structure is water penetration near the anchorages of the nave tie beams, so maintenance of the waterproofing of the roof and such simple measures as clearing the roof gutters, outlets and drainpipes is the best strategy for the cathedral authorities.

The study has raised an interesting question of how society should deal with structures that cannot be monitored but which may be deteriorating with time. The structure seems to have been properly designed, and there is ample evidence that it was built with more than normal care. It can be assumed that the structure was properly prestressed at the time and was perfectly adequate for its intended purpose when handed over to the client. There is no evidence of any corrosion of the prestressing tendons, and no evidence of any cracking that might indicate there has been any loss of force. But it has been shown above that there would be no evidence of corrosion, prior to failure. So what would happen if the owners asked Consulting Engineers to "guarantee" that the structure was safe. How many engineers would put their hands on their hearts, or perhaps more importantly, their Professional Indemnity Insurance, on a statement that the structure is safe for another 10, 50 or 100 years? This actually raises quite serious philosophical questions for the profession; at what

point does a perfectly adequate structure become compromised simply because of ignorance of what is going in internally?

## 6. Acknowledgements

## 7. References

Ahm, P., May 1962a. The New Coventry Cathedral. Civil Engineering and Public Works Review 57 (5), 614–616.

Ahm, P., March 1962b. The Structural Design of the New Coventry Cathedral. Ingenioren - International Edition 6 (2).

Ahm, P., 1987. Design and Structure. In: Campbell, L. (Ed.), To Build a Cathedral. Warwick, pp. X–XII.

Andrew, A. E., Turner, F. H., 1985. Post-tensioning systems in the UK: 1940-1985. Tech. Rep. 106, Construction Industry Reasearch and Information Association.

Anon., 25 May 1962. The Construction of Coventry Cathedral. Engineering, 682–685.

Arup, O., 1985. Coventry Cathedral: how the plan took shape. Arup Journal 20 (1), reprint of article in The Times supplement, 25 May 1962.

Austin, M., 2013. Strengthening strategies for Hammersmith Flyover. M.Eng. Project Report, Cambridge University Engineering Department.

BBC, Dec. 2011. Severe road delays as defect closes Hammersmith Flyover. URL http://www.bbc.co.uk/news/uk-england-london-16316283

Bedford, I., 2014. Personal communication.

Blee, A., 2014. Personal communication.

Buchhardt, F., Magirea, G., Matthees, W., Plank, A., 1984. Structural investigation of the Berlin Congress Hall collapse. Concrete International 6 (3), 63–68.

Calvocoressi, P., 1981. Top Secret Ultra. Ballantine Books.

Campbell, L., 2006. Coventry Cathedral: Art and Architecture in post-war Britain. Oxford.

Clarke, N., 2000. Coventry Cathedral revisited. Concrete, 38–40.

Crittall, 18 May 1962. Coventry Cathedral - the Great West Screen. Crittall Bulletin 89 (10).

Fisher, P., 2014. Personal communication.

Harris, J. D., Morice, P. B., 1952. Prestressed Concrete. Cement and Concrete Association.

Hoare, P., Sweet, C., 2000. The orientation of early medieval churches in England. J. of Historical Geography 26 (2), 162–173, doi:10.1006/jhge.2000.0210.

Hodge, J., 2012. Coventry Cathedral. Scala Publishers Ltd.

Hoult, N., Ekim, O., Regier, R., 2014. Damage/deterioration detection for steel structures using distributed fiber optic strain sensors. J. of Eng. Mech. 140 (12).

Johnson, R. P., 2014. Personal communication.

Jones, P., 2006. Ove Arup : Masterbuilder of the Twentieth Century. Yale U.P.

Lynch, D., 2012. Heavy duty repairs planned for closed Hammersmith Flyover ahead of Olympics. New Civil Engineer 3rd Jan 2012.

Page, C. L., Page, M. M., 2007. Durability of concrete and cement composites. Woodhead.

Perkin, G., Oct-Dec 1962. The Concrete of Coventry Cathedral. Concrete Quarterly (55).

Spence, B., 1962. Phoenix at Coventry. Fontana.

Wheen, R. J., 1979. Prestressed concrete members in direct tension. ASCE Journal of the Structural Division 105 (ST7), 1471–1488.

Woodward, R. J., Williams, F. W., 1988. Collapse of Ynys-y-Gwas bridge, West Glamorgan. Proc. Instn. Civ. Engrs, Part 1 84, 635–669.