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Controlled intervention: monitoring the dismantlement and reconstruction of the flying buttresses of two Gothic churches

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Abstract
This paper on “controlled intervention” focuses on the interest of monitoring during structural interventions on historical constructions. In addition to being a mechanism for control or validation of the intervention, in some cases, an adequate monitoring system can be a prerequisite for the execution of complex structural interventions. In this contribution, the methodology of controlled intervention is applied on the dismantlement and reconstruction of flying buttresses and illustrated with two case studies. In one case, the St. James church in Leuven, the monitoring process governs the intervention. For the other case, the church of Our Lady in Laken, monitoring data are used to control and validate the design of the intervention measures. In both cases, thanks to the monitoring systems, the dismantlement was controlled in real-time and the disturbance to the structure could be kept to a minimum.

Keywords
monitoring, controlled intervention, observational method, flying buttresses, vaults, dismantlement, restoration works, (neo-)Gothic churches, case studies

1 Introduction

The design of monitoring systems as part of a structural health monitoring approach towards historical constructions is gaining more and more interest (ICOMOS ISCARSAH, ISO 13822:2010). The planning of such a monitoring program for historical structures is less straightforward compared to newer civil structures and infrastructure, due to the wider time frame and the complex and heterogeneous nature of the structural elements and materials.

Monitoring is also a mean to control and validate the impact of structural interventions on historical constructions. It may even be a prerequisite for the execution of complex structural interventions. As stated in the ICOMOS ISCARSAH Charter “Any proposal for intervention must be accompanied by a programme of control to be carried out, as far as possible, while the work is in progress. Measures that are impossible to control during execution should not be allowed” (ICOMOS 2003).

The idea of controlled intervention is an established methodology in soil mechanics, where it is referred to as the “observational method” (Peck 1969). When the geotechnical behaviour of the subsoil is difficult to predict, the observational method can be applied to review the design during construction, based on the input of a continuous monitoring campaign. This idea was described in the works of Terzaghi and Peck (Terzaghi and Peck 1967) and is now also included in the Eurocodes (NBN EN 1997-1).

Also for civil works, the methodology of controlled intervention is used during the construction of for example tunnels and bridges; works which involve construction and soil mechanics. A well known example of a controlled intervention on a heritage structure is the stabilisation of the inclined Tower of Pisa. Works were carried out in different phases and during the installation of the temporary counterweight (phase 1) as well as during the soil extraction (phase 2), works were carried out in small, controlled steps and an extensive monitoring system was used to control the intervention and permit each next step in the process (Burland et al. 1998).

For the two case studies described in this paper, this methodology of controlled intervention has been applied during a rather complex structural restoration, being the dismantlement of flying buttresses. The dismantlement was judged to be necessary due to severe stability problems. During the intervention, displacements and forces were monitored, which allowed to control the dismantlement in real-time and keep the disturbance to the structures to a minimum.

In this contribution, the principles and design of the structural intervention itself and the impact of the removal of the flying buttresses will be discussed and illustrated. For each case study, a description of the monitoring system and a discussion of the monitoring results will be presented.

The first case study is the St. James church in Leuven, a building with numerous construction phases (dating back to ~1220), which has suffered and still suffers many structural problems. In this case, the monitoring process governed the intervention as will be discussed in Section 3.

The second case study is the neo-Gothic church of Our Lady in Laken. Here, the installed supporting structure was designed, based on thrust line analysis and the monitoring system was applied as a control and validation tool, see Section 4.

A comparison and discussion of the intervention on both case studies is presented in Section 5.

2 Flying buttresses

Flying buttresses are built to resist the horizontal thrust of the vaults. They are also an important architectural element of (neo-)Gothic churches. The decision to remove them is very serious and has important structural implications. If an intervention is decided, a substitution supporting system is needed to resist the horizontal thrust of the vaults.

The most straightforward way to secure stability and take up the horizontal thrust during the dismantlement of flying buttresses is to replace them by tie-rods. Figure 1 presents a sketch of the layout of a provisional supporting system used in both case studies described below. For each section with a pair of flying buttresses, four tie-rods are placed, two above and two below the contact zone of the flying buttress with the wall of the nave. The tie-rods pass through the windows to avoid drilling holes. They are anchored on horizontal U profiles, fixed on two vertical beams framing the flying buttress.

Figure 1: Basic layout of supporting system applied during the dismantlement of flying buttresses (a. flying buttress, b. provisional supporting system with tie-rods, c. vaults, d. outside wall (shown transparent to leave the shape of the vaults visible), e. location of the infill behind the springing)

This type of provisional supporting system has the advantage that it is composed of small and light pieces and is therefore easily assembled on-site. Additional damage to the structure is minimised and the provisional system can be fully disassembled after use. The horizontal thrust is resisted at the correct position, being the connection of the flying buttresses with the wall, and therefore, the working mechanism of the flying buttresses is well taken over without unnecessary reduction of the structure's flexibility.

For background information on the structural behaviour of arches and vaults, the reader is referred to works on masonry arches and vaults and studies on the general behaviour of Gothic structures (Heyman 1969, 1997; Smars 2000; Brencich and Morbiducci 2007; Barthel 1993; Block 2009; Huerta 2004).

3 Case 1: St. James church, Leuven

The church of St. James is a medium size church with a complex building history (Van Balen et al. 1995). It is a three nave church with a height of about 20 m below

the vaults of the main nave and a height of 8 m for the side aisles. The church includes parts from the 13th to the 19th century. The Romanesque bell tower was built around 1220, the Gothic nave, transept and chapels were built in at least six construction phases from the 14th to the 16th century and the neoclassical choir was built in the 18th century, Figure 2.

Figure 2: St. James church in Leuven (left) and steel support structure (right)

3.1 Structural problems

The structural problems of the church will only be summarised here very briefly, as they were thoroughly discussed in a previous contribution (Schueremans et al. 2007).

The church was built in a swamp area and has suffered important differential settlements. The critical intervention happened at the turn of the 15th to 16th century when the walls of the nave were heightened by about 8.4 m and the original wooden roof was replaced by brick vaults with a thickness of 22 cm (Van Balen et al. 1995). This surcharge caused important new settlements. After periods of concern, alterations and partial restorations (the oldest ones were traced back to the 15th century when the vaults of the aisles were rebuilt), the church was closed to public in 1963 due to overall stability concerns. Hereafter, the vaults in the aisles were dismantled and a steel tube shoring was inserted to laterally support the bell tower and to unload the columns of the nave, Figure 2. The original idea was to replace the columns, but restoration works stopped long before completion and the provisional steel frame is still in place today. The metallic structure, which altered the structural behaviour of the masonry construction, has complicated and held back restoration works in recent decades. Furthermore, since the church has been out of service since 1963, serious deterioration has occurred over time.

3.2 Controlled intervention

Due to large differential settlements between the main nave and the piers of the flying buttresses, the flying buttresses experienced very important cracks and deformations in vertical but also in horizontal plane, Figure 3. Due to water infiltrations and time-dependent degradation, the masonry lost most of its coherence. Calculation and engineering judgment revealed a situation particularly critical: small movements could have lead to collapse. Because of their dangerous shape and overall bad condition, stability could not be guaranteed with a sufficient margin of safety and it was decided (in 1999) to dismantle the flying buttresses and install provisional tie-rods to ensure stability. Before the intervention, the flying buttresses were photographed with a semi-metric photogrammetric camera and the stones were numbered. The dismantled elements are now stored in the chapels of the church, ready to be reused if reconstruction is decided in the framework of an overall restoration plan.

Figure 3: Buckled flying buttresses (left, photo: P. Smars 1994, in Van Balen 1995) and provisional system with tie-rods to replace the flying buttresses (right, photo P. Smars, 2000).

For each set of flying buttresses, four tie-rods were installed as indicated in Figure 1 and 3. Each tie-rod was equipped with a load cell to monitor the forces during dismantling. The tie-rods were post-stressed during the intervention, to control the distance between both walls of the main nave. The latter was recorded by means of invar-wire measurements.

Measurements ran for a period of 56 days, after which the data logger was removed as long-term monitoring was unfortunately not included in the project planning and the equipment was needed for other projects. The evolution of the forces in the tie-rods during the first week is presented in Figure 4. The dismantling of the four flying buttresses is clearly identifiable on the graphs. It is worth noting that forces increased relatively regularly during the dismantlement and that works on one section had a strong influence on the forces in the tie-rods of the other section.

Figure 4: Forces in the tie-rods during the dismantling of the flying buttresses
F4: sum of the forces in the four western tie-rods; F5: sum of the forces in the four eastern tie-rods; a & e, b & f: dismantlement of two western flying buttresses (section F4); c & g, d & h: dismantlement of two eastern flying buttresses (section F5)

Sudden changes (a and b in Figure 4) correspond to fastening of the tie-rod screws. The strongest increase happened in a, when the screws were tightened up by 2 mm. The forces increased by 92.5 kN, which is consistent with a deformation of 2 mm of the four tie-rods. But the displacement of the wall, measured with the invar-wire system was only 0.29 mm. The drop in c is an attempt to balance the forces in section F4 and F5. During the intervention, the maximum displacement was 0.86 mm in section F4. It has to be noted that the displacements were measured just under the level of the lower tie-rods and correlation is therefore difficult. This position was chosen for practical reasons. The invar measurements were made manually and could only be made from an accessible point: a metallic walkway built previously.

Figure 5 indicates the results of the measurements over the whole period (almost two months), together with the smoothed signals (F4S-F5S) and the temperature measurements. Unfortunately, the temperature data are not of the same quality as the force data. The thermometer inside the church did not work as expected and data from a nearby weather station had to be used.

Figure 5: Forces in the tie-rods over a period of 56 days (F4-F5); smoothed signals (F4S-F5S); exterior temperatures: daily minimum (Tmin) and maximum (Tmax)

It can be observed that the forces in the tie-rods slightly decrease over a longer period of time. Forces also appear to decrease when temperature increases. A study on the influence of temperature was presented by Smars (Smars et al. 2006). Temperature effects are complicated to analyse, as they trigger deformations of the tie-rods and of the walls, which are not perfectly rigid. In order to better understand the influence of the temperature variations on the thrust of the vault, it is advisable to perform long-

term monitoring of the deformations of the walls (also before the structural intervention takes place) to distinguish between fluctuations caused by the environment and effects of the intervention.

Twenty four days after the start of the works, four steel props were installed as a replacement of the flying buttresses. Since then, it is difficult to assess the exact significance of the monitoring results, as the forces can be transmitted both through the tie-rods and the props. It has nevertheless to be noted that the behaviour after installation of the props did not change significantly. Thus it appears that, in the mid-long term, the influence of the props was limited.

3.3 Outlook for St. James church

Urgent overall restoration works are necessary to revalorise the St. James church and preserve this monument for future generations. At present, these overall restoration works are in a planning phase. An adequate monitoring campaign will be an essential part of the restoration works as the impact of the structural interventions is to be verified during and after the works.

The experience which was gained during the monitoring and “controlled intervention” in the past will be most useful during the dismantlement of the steel tube shoring which was installed in the 1970s (Schueremans et al. 2007; Smars et al. 2006).

The monitoring system should not be used only to record the impact of the works but possibly also to guide the works, to provide feed-back and actively control the settlements. This manner of working is the main interest of controlled interventions.

4 Case 2: Church of Our Lady, Laken

The church of Our Lady in Laken, Brussels, is one of the largest neo-Gothic churches in Belgium. It was constructed in the second half of the 19th century, following the design of architect Joseph Poelaert. The crypt holds the tombs of the Belgian royal family. The Church of Our Lady is a five nave church, with a height of 27 m under the vaults of the main nave. It is currently undergoing a thorough restoration of its exterior, including a restoration of the ornamental front façade (Figure 6).

The restoration of the western and eastern side walls is carried out in two phases. In each phase, six flying buttresses are dismantled and replaced. At reconstruction of the flying buttresses, stones which are damaged or show too much deterioration are replaced by newly quarried and shaped stones. The decision to dismantle the flying buttresses was taken by the restoration team in charge. The K.U.Leuven was then contacted to devise a provisional supporting system to substitute the flying buttresses during dismantlement. In addition to the design of the supporting system, it was advised to monitor the structural intervention. At present, the first restoration phase has been realised and the next set of six flying buttresses is to be dismantled subsequently. After completion of the first phase, the tie-rods as well as the monitoring system are re-used for the second phase. In this paper, the monitoring results of the first phase will be presented and discussed.

Figure 6: Front façade of church of Our Lady in Laken (left) and view on flying buttresses to be dismantled during the second phase (right)

4.1 Flying buttresses: calculations

During the dismantlement of the flying buttresses of St. James church in Leuven, the forces in the tie-rods were gradually increased to keep the deformations minimal. In case of the church of Our Lady in Laken, the necessary forces in the tie-rods are fully applied before removal of the flying buttresses.

The calculation of the necessary forces in the tie-rods is based on a thrust line analysis which only requires information on the geometry and weight of the vaults. The resulting horizontal forces are calculated by dividing the vaults into small, horizontally equidistant arches which are supported by the ribs (Huerta 2004; Heyman 1997), see Figure 7. The vault ribs are thus subjected to their own weight and to the resulting horizontal and vertical forces of the vault sections.

Figure 7: Division of the vaults into small equidistant arches which are supported by the vault ribs, in order to simplify calculations.

The calculation of the thrust lines in the arches (the arches being the equidistant sections of the vaults as well as the ribs) is performed with a limit analysis software tool developed in the framework of the Ph.D. thesis of P. Smars, called Calipous (Verstrynge et al. 2007; Smars 2000).

The position of the thrust line, and thereby the value of the resulting horizontal force, is not unique. It depends also on the height of the infill behind the springing. For the calculations, the maximum lateral thrust is assumed. It is also one of the advantages of the tie-rod system that the vertical beams lining the flying buttresses are able to spread the reaction load. As such, the knowledge of the actual position of the thrust line is not required and a certain deviation can be dealt with.

This simplified calculation scheme could be applied here as the geometry of the quadripartite or four-celled rib vaults was rather straightforward and conservative assumptions are made. More advanced models can take into account three-dimensional effects and friction to calculate a surface of thrust for masonry vaults (D'Ayala and Tomasoni 2011; Smars 2008). Even more complicated nonlinear analysis tools are available, which demand knowledge on the masonry characteristics. However, in the case presented, only information on the geometry of the vaults was at hand and a practical calculation method which would give a reliable result, based on restricted input, was required.

The reader should be aware that these calculations can only provide an estimate of the thrust, as in reality, the vault is not sliced, acts as a 3D surface and has some tension resistance. Therefore, the real minimum and maximum thrusts are respectively lower

and higher than the estimates (approach from the interior). Hence the importance of controlled intervention and displacement measurements to keep additional damage in the vaults to a minimum.

4.2 Controlled intervention

4.2.1 Temporary support structure

Again, for each pair of flying buttresses, four tie-rods are applied, resulting in twelve tie-rods for each restoration phase. In addition to this scheme, it was requested by the restoration team to elongate the vertical profiles and connect them to horizontal profiles and steel props in order to form a stiff support and take up possible deformations from wind loading. The support structure therefore is composed of two systems; one to take up the lateral thrust and one for the wind loading. A drawing of the cross section of the church with the temporary support system is presented in Figure 8. The necessity of this additional structure can be discussed. Fact is that the added complexity of the support system makes it difficult to interpret the monitored forces in the tie-rods and deformations of the structure.

Figure 8: cross section of church of Our Lady in Laken, with temporary shoring structure and tie-rods. The smaller figure in the right corner provides a view on the tie-rods from the inside of the church

The necessary pre-stress in the tie-rods was calculated according to the analysis scheme discussed in Section 4.1. The span of the vaults is 10 m for the main vaults and 5 m for the vaults of the side aisles. The webs of the vaults, constructed in clay bricks, have a thickness of 8 cm and the sandstone ribs are 40 cm thick. The height of the infill behind the springing is 2 m. The dimensions of the vaults were checked with in-situ measurements. Using this analysis scheme, a design pre-stress of 67.5 kN is calculated for each set of four tie-rods. This is the minimum pre-stress to be applied before dismantlement of the flying buttresses. In addition to this requirement, a maximum relative displacement of the nave walls of 2 mm is aimed at to avoid additional cracking in the vaults. Therefore, the diameter of the tie-rods is not governed by their ultimate strength, but by the required stiffness to limit deformations of the tie-rods. If the lateral thrust of the vaults does not fully coincide with the applied pre-stress, the deformation of the tie-rods will be less for larger diameters. Tie-rods with a nominal diameter of 25 mm are chosen.

During pre-stressing of the tie-rods, the steel props were not yet fixed and all forces were applied solely on the tie-rod system. After pre-stressing, the props are fixed. Then, the upper part of the pinnacles is dismantled before dismantlement of the flying buttresses. A reversal of these operations would introduce an increased slenderness in the pinnacles and pier buttresses, which is to be avoided.

4.2.2 During dismantlement

During dismantlement of the flying buttresses, it was observed that long, curved iron anchors were present in the flying buttresses, in a groove above the voussoirs of the inferior arch, Figure 9. These iron bars were anchored to a vertical bar both in the nave wall and in the pier. It could be argued that these anchors enable the flying buttresses to take up tensile loading, such as introduced by wind loading, or would enable the flying buttresses to work as a beam. However, this would not be necessary due to the overall compression state of a flying buttress. Also, the shape of the anchors makes that this resistance to tension would be very limited.

Some of the anchors were heavily corroded, especially the ones which were present in the flying buttresses positioned on the western wall (which has the highest wind and rain load). On this side of the wall, also cement mortar was found in the joints of the flying buttresses from past restoration works. At reconstruction, a hybrid cement-lime mortar was applied.

One of the anchors broke during dismantlement of the flying buttress and its properties were tested in the lab. The anchor had a diameter of 29.8 mm and the maximum tensile force was 189 kN, at which the metal anchor ruptured in a brittle manner.

As the strength of the anchor was higher than expected (270 MPa), the broken metal bar was replaced by a new one and all anchors were, after corrosion treatment, re-encased in the brickwork of the flying buttresses.

The exact motivation of the original designers for inserting the iron reinforcement bars is not known. However, the presence of the iron bars had an advantageous effect during pre-stressing of the tie-rods; the risk of instability of the flying buttresses upon tensioning of the tie-rods was largely reduced, as the masonry could, to a certain extent, react as a beam upon reduction of the horizontal thrust due to the presence of the iron bars.

Figure 9: Three phases in the reconstruction of the flying buttresses and close-up of one of the iron anchors

4.2.3 Monitoring results

In order to control the required maximum displacement and minimum pre-stress, the deformations of the nave walls and the stresses in the tie-rods had to be monitored on a continuous basis during the structural intervention. The strains in the tie-rods were monitored by means of electrical strain gauges which were spot welded on the metal rods before their placement in the nave (see small circles in Figure 8). The relative displacements of the side walls are measured automatically at regular frequencies with invar-wires from one side of the nave to the other, close to the middle set of tie-rods, at a height of 11.7 m (ch4) and 19.6 m (ch3 – at the same height as the tie-rods). The temperature was only measured at one location inside the church, close to the outside wall at a height of 5m. The location of the temperature sensor, strain gauges and invar-wires is indicated on the ground floor plan in Figure 10. Displacement

measurements were also performed between the nave walls and fixed points, these will not be discussed here as little information was obtained from these measurements.

Figure 10: Ground floor plan of the church of Our Lady in Laken, with the location of the temperature sensor, strain gauges and invar-wires.

Measurement of the forces in the tie-rods started just before tensioning of the tie-rods and stopped after unloading. Temperature and deformation measurements were continued for one week after unloading. Hereafter, the monitoring system was transferred to the second phase of the restoration in which the subsequent six flying buttresses will be dismantled. During the first phase, measurements were carried out for 6.5 months. Forces, temperature and displacement data were recorded at a frequency between 1 recording each 10 seconds (at tensioning of the tie-rods) up to 1 recording every 5 minutes (after a certain time).

The “zero” point for the strain gauge measurements was taken after connection of the electric wire system, which is just before tensioning of the tie-rods. As the tie-rods were already installed at this point, tension was present in the tie-rods at the moment the strains were set to zero. For a deflection of 30 cm, a horizontal force of 10 kN will have been present in each tie-rod and on the nave walls. This force could not be measured with the strain gauges. Therefore, the pressure in the hydraulic jacks, used to pre-tension the tie-rods, was used to monitor the actual force increase during pre-tensioning. After removal of the hydraulic jacks, the long-term evolution of the forces was monitored by means of the strain gauges in a relative manner.

Figure 11 presents the monitored temperature, relative displacements and forces in the tie-rods during the first week. The presented forces are the sum of the forces measured by the strain gauges in each set of four tie-rods. These forces are calculated from the measured strains, by assuming a Young’s modulus of 205 GPa for the steel bars. This value will be checked by a tension test on the steel bars after completion of the second phase of the restoration.

Pre-stressing was applied with hydraulic jacks and the sequence of pre-stressing (F3-F2-F1) is clearly visible on the graph. Pre-stressing of F2 was interrupted and proceeded the next day. The forces in F2 were lowered (point ‘a’ in Figure 11) to decrease the displacements of the walls; however, this unloading did not influence the displacements. A pre-stress of 140 kN was applied on set F1 and F3, a slightly lower force of 120 kN was applied on set F2 to limit deformations. The average applied force of 100 kN, as presented in Figure 11, was measured by means of the strain gauges. This value is lower than the actually applied forces as strain gauge measurements could only be started after installation of the tie-rods, as explained above. The difference between the data from the pressure in the hydraulic jack and the strain gauges is approximately 4*10 kN = 40 kN, as estimated before. The results of the strain gauge measurements are therefore only used to comment on the evolution of the forces on the long term in a qualitative manner.

At the end of the pre-stressing process, the deformations were limited to 2.6 mm, which is slightly higher than the preset value of 2 mm.

The applied pre-stress is twice as high as the minimum stress estimated in Section 4.1. The decision to apply a higher pre-stress is based on following observations:

- Pre-stress losses are expected and were observed immediately after pre-stressing and on the long term (in the joints of the provisional support system, due to temperature fluctuations and due to time-dependent deformations in the tie-rods and the vaults);
- It was observed that, when the next section is pre-stressed, the stresses in previously stressed tie-rods decrease slightly;
- The target value of 67.5 kN was still present after pre-stress losses (which were almost 50 %, see discussion on long-term monitoring below).
- Monitored displacements were within limits, except for ch3 which is only slightly higher than the preset maximum displacement;

Again, as was observed at St. James church, the forces in the tie-rods are not equal as they are very sensitive to small differences of pre-tensioning. Additionally, not all strain gauges were positioned on the neutral axis of the tie-rods, which could have caused small fluctuations. The effect of bending can be calculated from the stresses measured by one strain gauge on the surface of the tie-rod. Therefore, the exact position of the strain gauges and the rotations at the supports are to be known. This information was not available as the strain gauges were not accessible after installation of the tie-rods. This problem can be overcome by placing two strain gauges at opposite sides of each tie-rod, which has the disadvantage that the amount of sensors, cables and channels needs to be doubled. This option could unfortunately not be opted for due to practical and budget restrictions.

Figure 11: Monitored temperature, relative lateral displacements of the nave walls and forces in the tie-rods during the first week (the ticks on the X-axis are positioned each day at midnight)

The subsequent fluctuations in the tie-rod forces during the first week are caused by works such as dismantlement of the pinnacles and connecting of the steel props. After this first week, an average pre-stress force of about 70 + 40 = 110 kN is left in the tie-rods. During the dismantlement of the flying buttresses, which took place two weeks after pre-stressing of the tie-rods, no additional deformations or fluctuation in the tie-rod forces were noticed, which indicates that the flying buttresses were effectively brought in a passive state before dismantlement. The horizontal force which is imposed on the nave wall by a flying buttress in its passive state was estimated to be around 20 kN, but due to the iron reinforcement in the flying buttress, this force might have been smaller. Additionally, some of this horizontal force has been taken up by the steel props which were fixed before dismantlement of the flying buttresses.

Since the temperature was not measured at the location of the strain gauges, a dummy strain gauge was provided which was fixed to an unloaded steel bar close to the middle set of tie-rods. In all graphs shown, the measurements made by this dummy have been subtracted to filter out the effect of temperature fluctuations.

After the first week, the forces in the tie-rods and the displacements remain rather constant, see Figure 12. To smooth the curve, the presented force data are taken each day at 1 am. Unfortunately, no data are available during two rather extensive power cuts (a and b). The continuation of the long-term measurements after the power cuts was not significantly affected by these problems. A slight decrease of the forces and increase of the displacements is noticeable over the measuring period of 6.5 months. At the end of the monitoring period, an average force of about 50 + 40 = 90 kN remains present in each set of tie-rods, which is still higher than the preset value of 67.5 kN. Again, the larger fluctuation in forces at the end of the monitoring period is due to re-installation of the pinnacles and disconnection of the steel props.

Figure 12: Monitored temperature, relative lateral displacements and forces in the tie-rods during the full monitoring period (no data available during power cuts: a and b)

After reconstruction of the flying buttresses, the tie-rods were unloaded and the steel support structure was dismantled. During the unloading, the measured strains in most of the tie-rods decreased to zero, however, some of the measurements did not. After inspection, it was observed that the tie-rods had displaced and bent after unloading. Thereby, larger deflections had occurred than those which were present at the moment the zero-point had been taken for the strain gauge measurements. During unloading, only small displacements of the walls (less than 0.5 mm) were detected and it is assumed that the structure will gradually adapt to the new situation again.

4.3 Outlook for church of Our Lady in Laken

During the second phase, in which the remaining six flying buttresses will be dismantled and reconstructed, experience gained from the first phase will be taken into account: extra dummy strain gauges will be provided to apply a dummy for each set of four tie-rods for better temperature compensation; two strain gauges will be installed on one of the tie-rods to estimate the effect of bending on the location of the strain gauges; slightly lower pre-stress could be applied in the tie-rods, sufficient to bring the flying buttresses in a passive state and to take into account pre-stress losses while limiting deformations; a more careful unloading of the tie-rods might prevent bending and enable to estimate a possible drift of the strain gauges data and correct the measurements accordingly; and finally, as the monitoring system and support structure is not needed for a next phase, displacement measurements should be carried out for a longer period after reconstruction of the flying buttresses and unloading of the tie-rods.

The second phase will also bring along new challenges as, for example, no window is present on one side of the flying buttresses which are positioned closest to the choir, so tie-rods will have to be positioned through drilling holes.

5 Comparison and discussion

In both case studies described above, the main points of interest for the design and execution of provisional strengthening during the dismantlement of flying buttresses are given as follows:

- Design: the estimation of the necessary forces in the tie-rods is based on a thrust line analysis, which is a simplified approach. This approach enables to calculate the required information based on restricted input. Although the horizontal thrust cannot be calculated exactly, the structure behaved as expected during intervention and additional damage to the structure was avoided.
- Provisional support: the tie-rod system is a flexible and adequate manner of providing temporary support during dismantlement if properly installed and carefully (pre-)stressed. The dimensioning of the tie-rod section is based on stiffness requirements to limit deformation and cracking.
- Control: to control the execution, two sources of information are necessary: the forces in the tie-rods and the displacements.

At the St. James church in Leuven, the tie-rods were post-tensioned, which means forces were increased as needed to limit deformations. In this process, the effect of the flying buttresses was slowly replaced by the effect of the tie-rods. As the tie-rods were tensioned by tightening the screws, load cells and invar-wire measurements were applied to provide the necessary two sources of information to control the intervention.

At the church of Our Lady in Laken, hydraulic jacks were used to apply the pre-stress. In this process, the pressure in the jacks could be used to monitor the forces in the tie-rods. On the long term, the forces in the tie-rods could be monitored by means of twelve strain gauges, which are less reliable but cheaper than load cells. Displacements were also monitored by means of invar-wire measurements.

Based on our experience with both methods, post-stressing and pre-stressing, it cannot be stated that one method has a distinct preference over the other. Post-stressing can minimise deformations in a more controlled manner, as it is a step-by-step process, provided that the steps taken are very small and the whole dismantlement process is very closely controlled. This method requires important on-site engineering judgement. For pre-stressing, deformations depend on the accuracy of the estimation of the horizontal thrust and loss of pre-stress has to be taken into account. This method is to be preferred if the dismantlement process takes a long time and cannot be permanently monitored up-close. Therefore, the installation and tensioning of the support system on beforehand is to be preferred in this case.

Similarities and observations which are valid for both case studies are:

- The relative displacements of the nave wall, and therefore the deformations of the vaults, are comparable in terms of strain: a relative displacement of 0.086 $*10^{-3}$ (0.86 mm on 10 m, case 1) versus 0.113 $*10^{-3}$ (2.6 mm on 23 m, case 2);
- As these strains were small, deformations were limited to linear-elastic behaviour and no significant damage was introduced by the intervention;
- The force in the tie-rods, and thus the lateral thrust is comparable for both case studies. Although the St. James church is smaller, its vaults are also lower and

thicker which results in a comparable lateral thrust for the vault system of both churches;

- The different sections do interact, as tensioning of one set of tie-rods influenced the other sections (both in terms of displacements and of stresses in the tie-rods). This interaction is due to the diaphragm action of the vaults, which is not accounted for in the design;
- It is difficult to correlate all factors (forces, displacements, temperature, work) and to quantify the reliability of the monitoring results. Therefore, it is necessary to gather a large number of effective data, improve the placement of the sensors and increase redundancy of the monitoring system;
- It was remarked for both case studies that it is most valuable to monitor before, during and after intervention to quantify the effects of temperature and relative humidity fluctuations and the influence of the structural intervention in the long term.

Although in both cases, additional damage to the structure could be avoided, it should be stressed that, if possible, repair is to be preferred over replacement of historic material and the necessity of the dismantlement of flying buttresses is to be judged carefully.

6 Conclusions

By means of two case studies, the interest of an integrated monitoring approach for the controlled execution of complex structural interventions was discussed. Although the two churches did not have the same scale or structural history, it was found that it was possible to reduce the negative effects of significant interventions by keeping deformations and forces within limits.

Horizontal thrust could only be estimated by means of a thrust line analysis and therefore, the exact values cannot be known. However, both churches reacted as expected during the structural interventions and responded very well to the new load situation. This also illustrates the remarkable capacity and flexibility of vaults to accommodate a certain range of load situations.

Although the interpretation of the monitoring results in both cases was complicated by the placement of metal props, the evolution of displacements and forces could be followed efficiently by means of relatively simple monitoring techniques.

For St. James church in Leuven, the monitoring provided the means to keep the deformations as small as possible and to control the force increase in the tie-rods during and after the dismantlement. In case of the church of Our Lady in Laken, the monitoring was used as a control mechanism to validate the force calculations made in advance. In this case, the tie-rods were pre-stressed before dismantlement took place.

In the presented case studies, monitoring was a prerequisite for the controlled execution of the structural interventions. Thanks to the monitoring systems, the

dismantlement was controlled in real-time and the disturbance to the structure could be kept to a minimum.

Finally, it should be remarked that in case of the church of Our Lady in Laken both monitoring systems (forces and displacements) were installed and operated by different parties. This adds complexity to the swift follow-up and processing of the data. The efficiency of monitoring during controlled interventions therefore largely depends on the close collaboration between all parties involved, including the contractor and structural engineers.

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Figure captions

Figure 1: Basic layout of supporting system applied during the dismantlement of flying buttresses (a. flying buttress, b. provisional supporting system with tie-rods, c. vaults, d. outside wall (shown transparent to leave the shape of the vaults visible), e. location of the infill behind the springing)

Figure 2: St. James church in Leuven (left) and steel support structure (right)

Figure 3: Buckled flying buttresses (left, photo: P. Smars 1994, in Van Balen 1995) and provisional system with tie-rods to replace the flying buttresses (right, photo P. Smars, 2000)

Figure 4: Forces in the tie-rods during the dismantling of the flying buttresses
F4: sum of the forces in the four western tie-rods; F5: sum of the forces in the four eastern tie-rods; a & e, b & f: dismantlement of two western flying buttresses (section F4); c & g, d & h: dismantlement of two eastern flying buttresses (section F5)

Figure 5: Forces in the tie-rods over a period of 56 days (F4-F5); smoothed signals (F4S-F5S); exterior temperatures: daily minimum (Tmin) and maximum (Tmax)

Figure 6: Front façade of church of Our Lady in Laken (left) and view on flying buttresses to be dismantled during the second phase (right)

Figure 7: Division of the vaults into small equidistant arches which are supported by the vault ribs, in order to simplify calculations

Figure 8: cross section of church of Our Lady in Laken, with temporary shoring structure and tie-rods. The smaller figure in the right corner provides a view on the tie-rods from the inside of the church

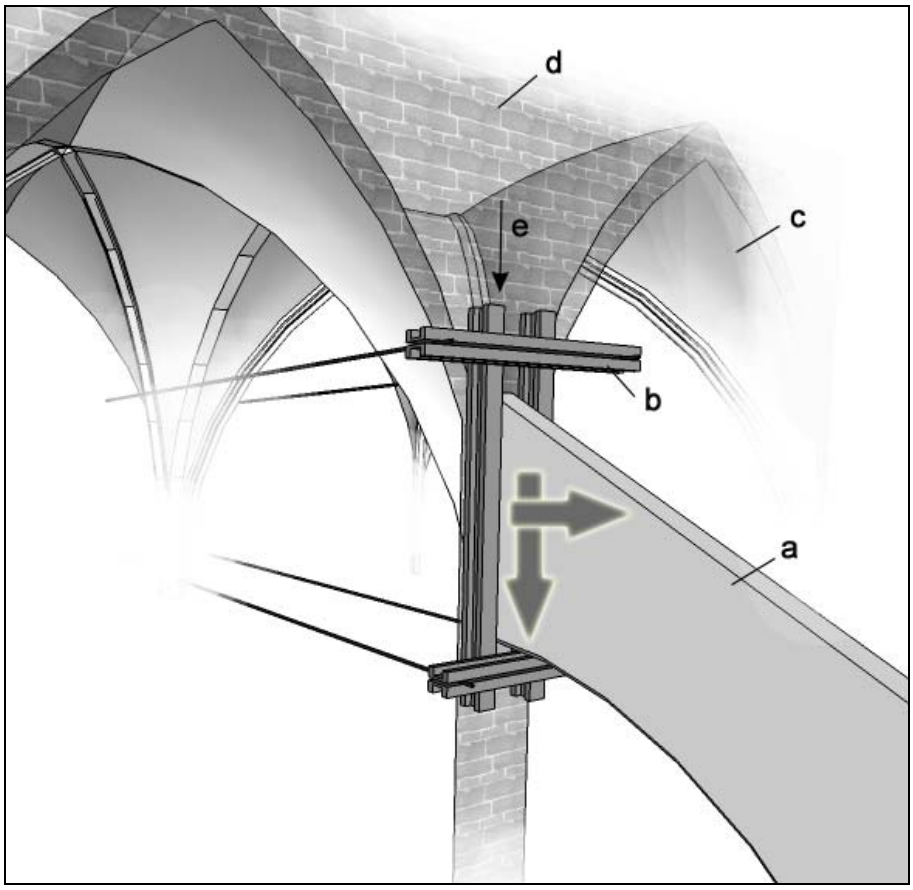
Figure 9: Three phases in the reconstruction of the flying buttresses and close-up of one of the iron anchors

Figure 10: Ground floor plan of the church of Our Lady in Laken, with the location of the temperature sensor, strain gauges and invar-wires

Figure 11: Monitored temperature, relative lateral displacements of the nave walls and forces in the tie-rods during the first week (the ticks on the X-axis are positioned each day at midnight)

Figure 12: Monitored temperature, relative lateral displacements and forces in the tie-rods during the full monitoring period (no data available during power cuts: a and b)

Figures



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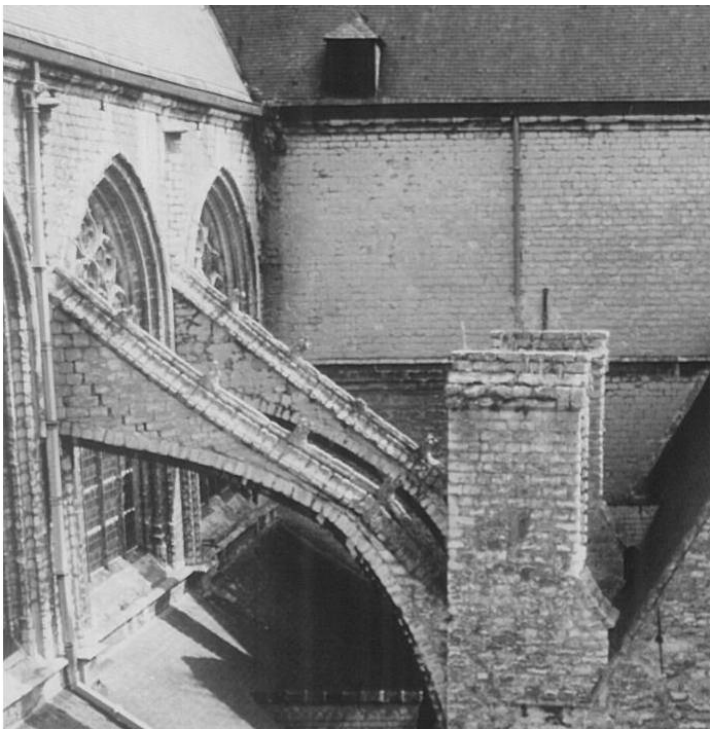

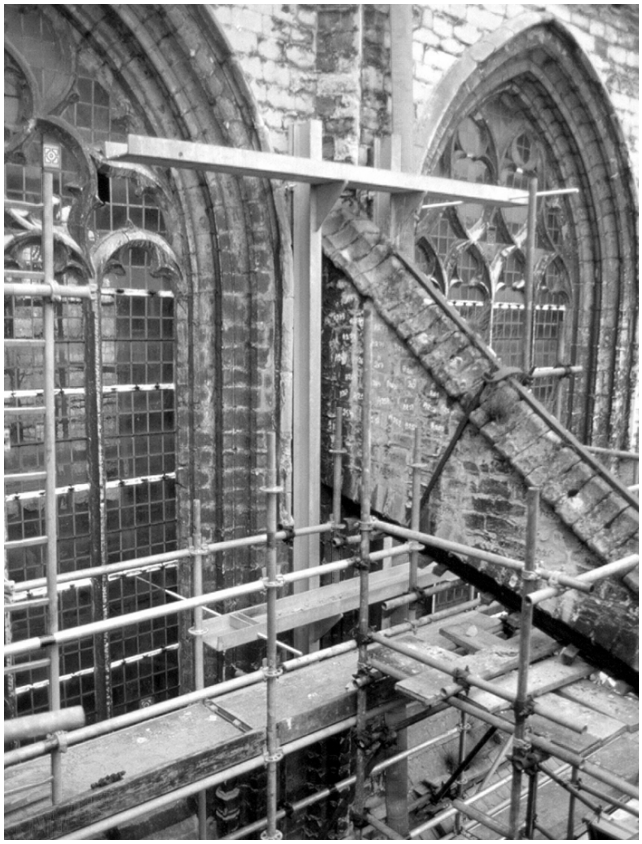

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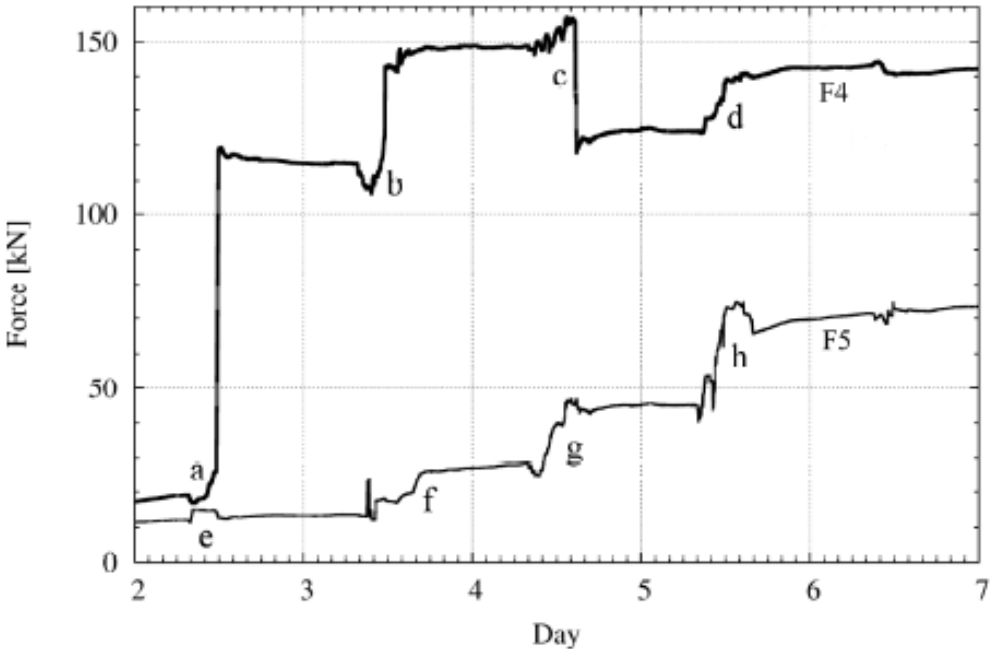

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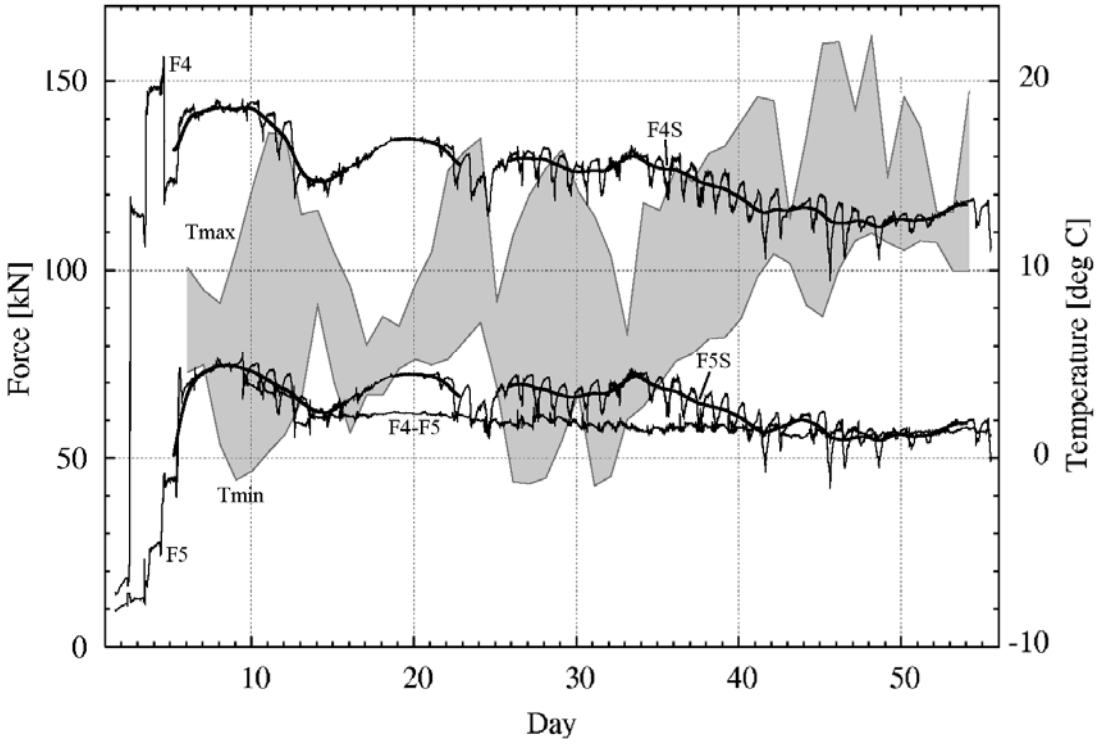

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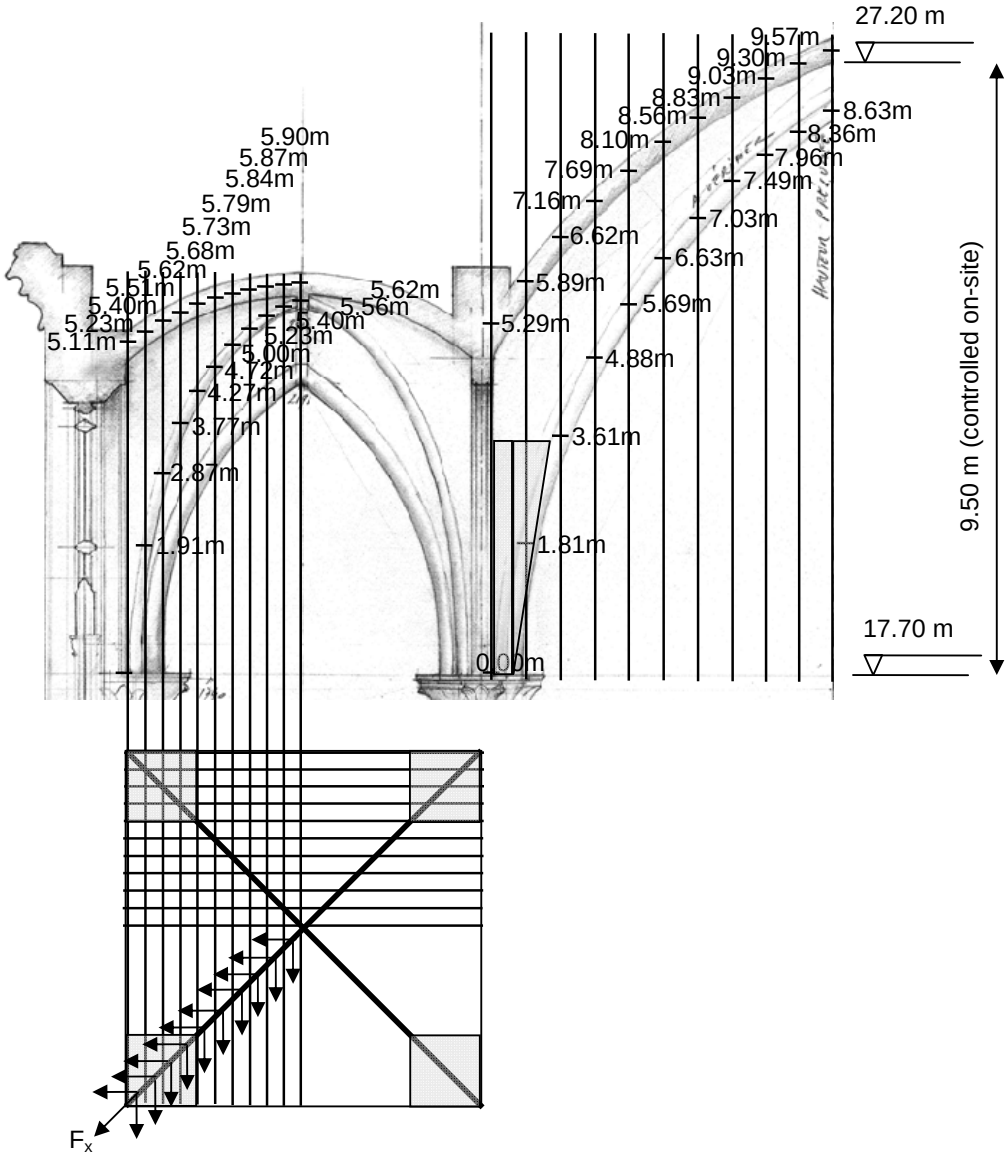

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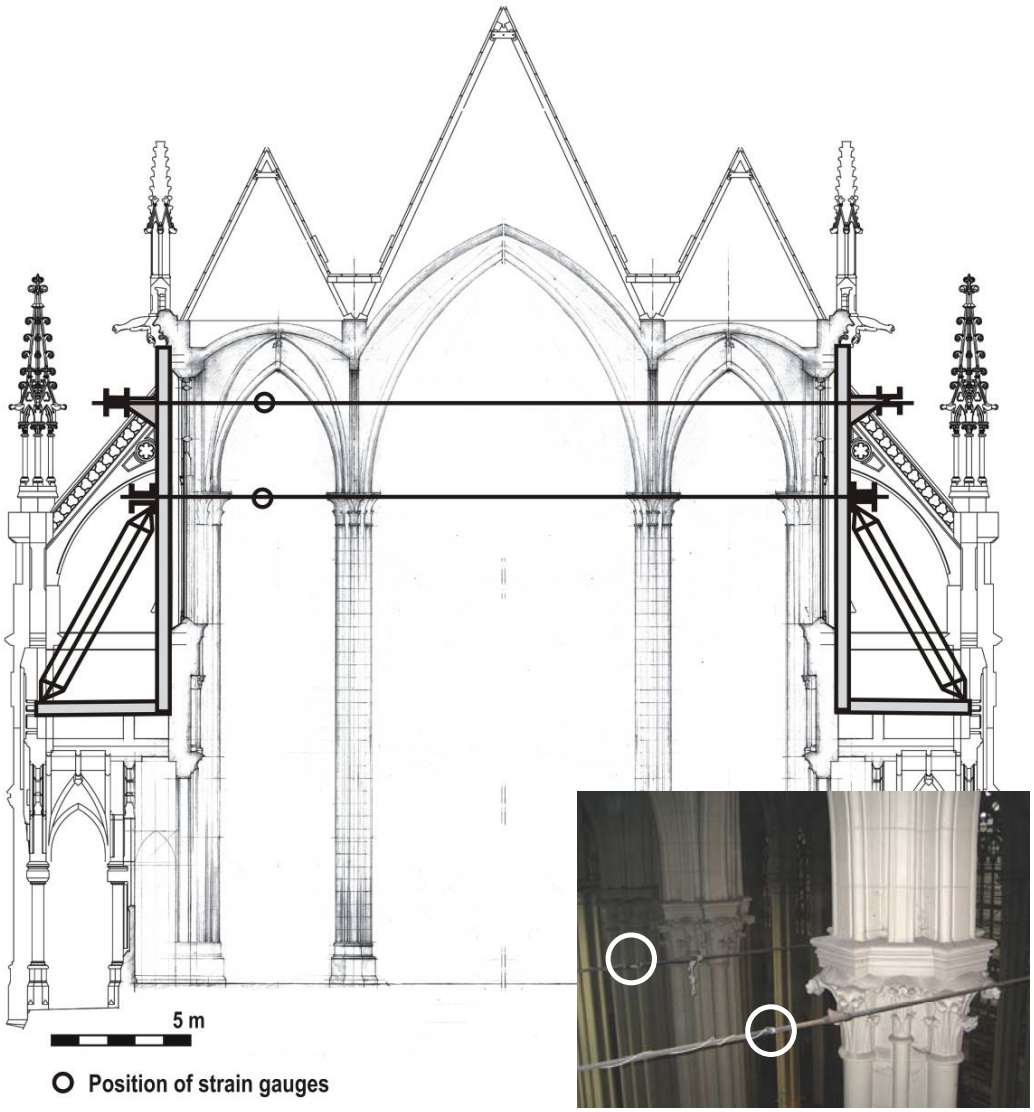

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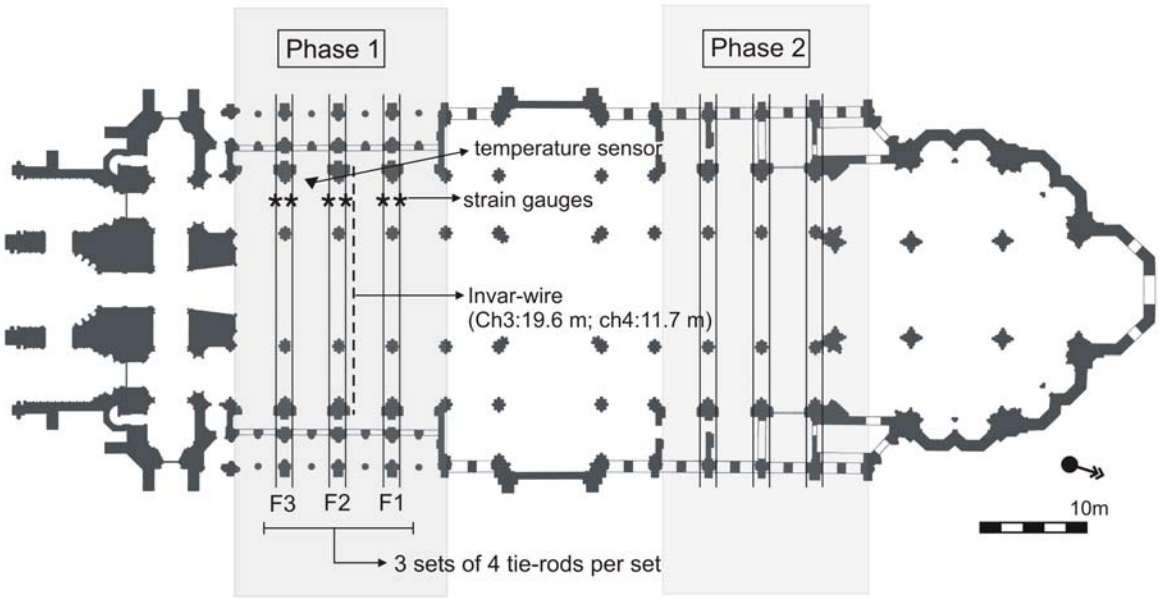

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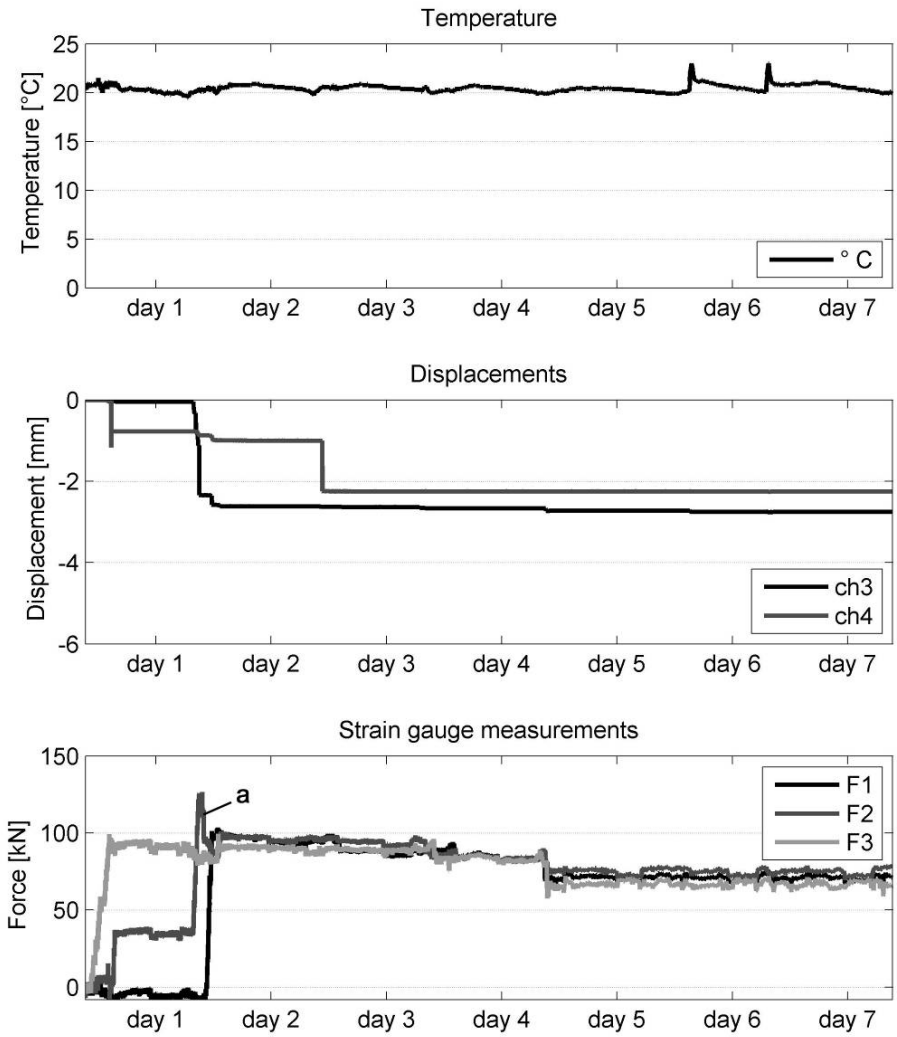

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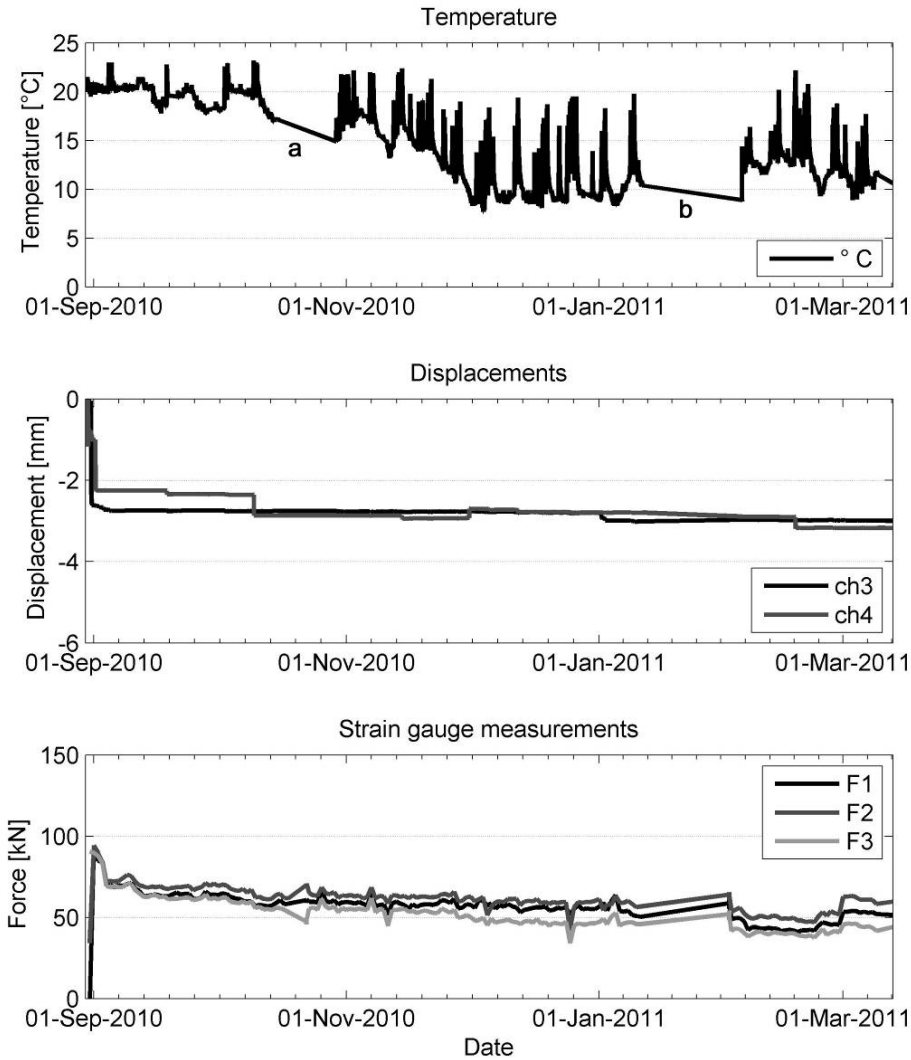

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