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Church of Saint-James at Leuven (B) – Structural Assessment and Consolidation Measures

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ABSTRACT: The construction of the church dates back from 1220. During several subsequent building phases, the wooden roofs in the central and side naves were replaced with masonry vaults, and flying buttresses were added. The structure itself is located on a former swamp, resulting in large differential settlements. In the past, restoration works took place. However, due to the excessive cracks observed, it was decided in 1963 to close the church for service, to remove the severely cracked masonry vaults of the side naves and to shore up the pillars of the main nave. Additionally, in 2000, the remaining flying buttresses were removed and replaced by tie-rods. The focus of this paper is on the structural understanding of this historical building, to determine the most appropriate strengthening and consolidation measures, respecting the authenticity of the building and preserving it for the future.

1 INTRODUCTION

In 1995 the Lemaire Centre for the Conservation of Historic towns and buildings, performed an initial investigation on the actual state of this listed historical building, ordered by the Flemish Government. In 2005, the city of Leuven started a global restoration plan. Triconsult was asked to investigate, under the supervision of KULeuven, the actual safety of the structure and to provide a means to remove the shoring, added in 1963. Final objective is to provide a multi-purpose re-use of the building in the future.

An extensive survey of all possible (historical) sources was performed in 1995 to get a complete picture of the structural history of the building itself. These sources already reveal a lot of the causes of the structural problems that need to be solved today. During the study performed in 1995, the stability of the vaults was studied in detail. Additionally, the differential settlements of the main columns, the walls and pillars were monitored during half a year. Cone penetration tests enabled to quantify the limited load bearing capacity of the subsoil.

In 2005, an in site survey took place, in which additional information was gathered on the layout and depth of the foundation by digging investigation holes and performing endoscopes in drilled boreholes through the foundation. Furthermore, information was gathered on the cohesion of the soil using in site vain shear tests. Properties of brick, stone and mortar were measured. This information was used to model and estimate the time dependent vertical settlements of the major structural elements that took place over several centuries accounting for the different building phases of the structure. These settlements can be related to the observed slant of the western tower and explain the present state of the structural crack pattern, present in the main nave and transept of the church.

In this contribution the different steps in the preservation process are outlined: anamnesis, analysis, diagnosis, therapy and control according to the Krakow Charter. Attention is paid to the relation between the information of different sources, the quality of numerical modeling and its experimental verification in practice by (semi)- or non-destructive testing and monitoring. A more detailed description of several items can be found elsewhere (Schueremans et al., 2006a,b; Schueremans and Van Gemert, 2006a,b).

2 ANAMNESIS

2.1 Historical building phases and evidence of malfunctioning

The history of this church reveals interesting items related to its structural problems. Therefore, the main building phases are briefly commented, Figure 1 (Van Balen et al., 1995):

- 1220: construction of the western tower of the church. The structure itself is located on a swamp, reclaimed by the monks at the time of construction;
- 1290-1300: construction of main pillars and arcades, intended for a flat wooden ceiling;
- 1305-1317: period of prosperity for the city of Leuven (B). Construction of wall in between arcades on top windows and construction of side naves + masonry vaults;
- 1465-1535: construction of wooden barrel vault above main nave;
- 1465-1488: construction of transept and columns at crossing of main nave and transept;
- 1485-1488: reconstruction of masonry vaults in side naves at both north and south side;
- 1534-1535: Construction of additional level with windows in main nave; construction of supports and abutments of flying buttresses; removal of wooden barrel vault and; construction of masonry vault in main nave (arching: Gobertange stone).

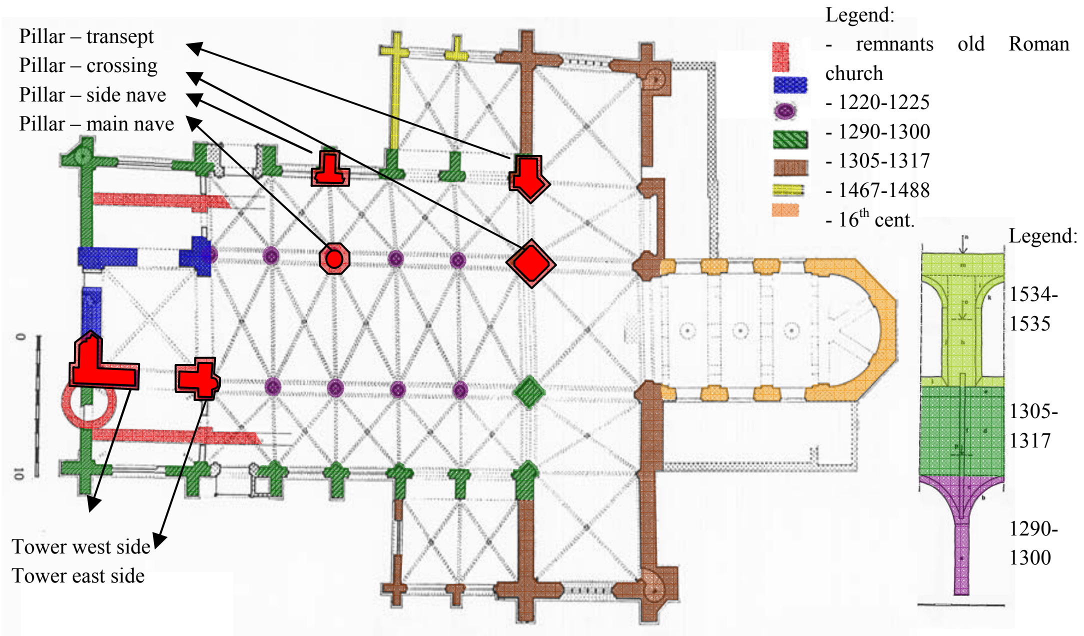

Figure 1 : Historical building phases

Already during history, structural problems were encountered at regular time intervals:

- 1485-1487: vaults of side naves rebuilt – cracking due to large differential settlements urged for dismantling of original vaults;
- 1806: the pillars are strengthened with steel rings;
- 1963: the church is closed down for service;
- 1965-1971: phase 1 of restoration 1961-1974, see further;
- 2000: dismantling of flying buttresses.

In 1961, the collapse of the church was feared. To prevent further deterioration, it was decided to take immediate action. The plan contained 3 subsequent phases, Figure 2:

- Phase 1: Support of cross-columns against the lateral loading of the tower. Two steel tube structures are added in the crossing and concrete columns near the crossing pillars are constructed to get a distribution of the horizontal loading over the cross-columns;

- Phase 2: Goal is to replace the masonry pillars of the main nave by means of reinforced concrete columns and add a facing of natural stone at the outside of the columns. Additionally the foundation surface has to be increased to limit the soil stresses. At that moment in time, a pile foundation was considered not feasible. To enable the replacement of the columns, a temporarily shoring of the main nave is required. In the mean time, the masonry vaults of the side nave are dismantled, since these demonstrate a severe state of decay;
- Phase 3: Remedy the initial cause of the differential settlements: uncouple, detach the western tower from central nave.

Only preparatory phase one and phase two (partly) were executed. Until now the structure is supported, already during 40 years, by means of a temporarily shoring.

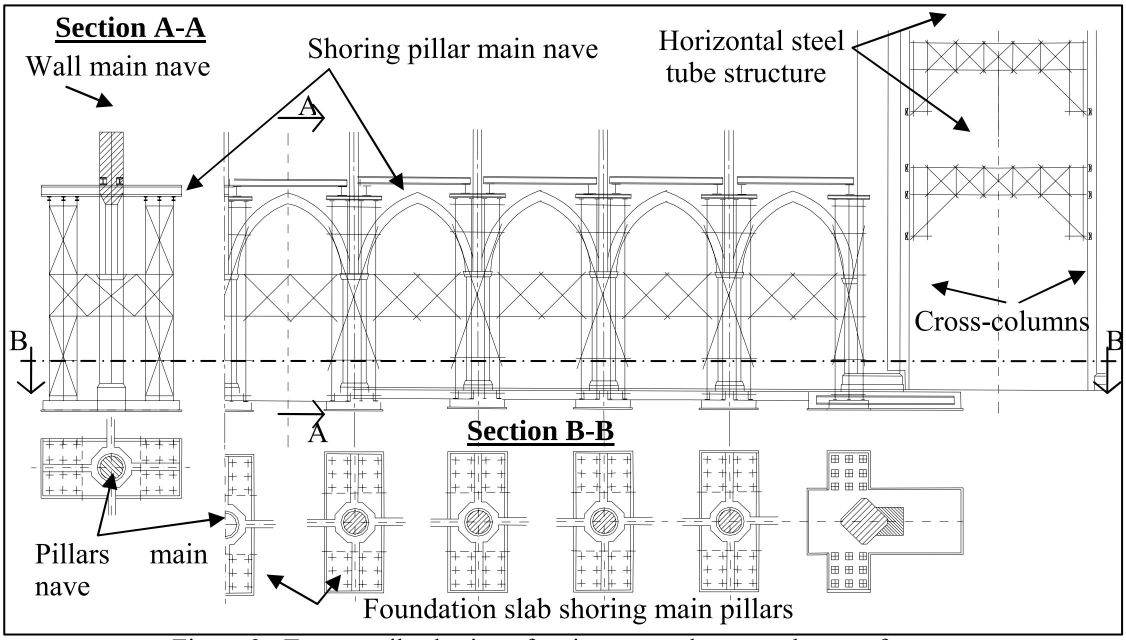

Figure 2 : Temporarily shoring of main nave and cross-columns of transept

2.2 Data collection

In the starting phase, all necessary data are gathered to verify the assumptions made in the aforementioned analysis process, dating from 1961. For that purpose, it was clear that the overall geometry, layout of the masonry pillars, slant of structural elements and their differential settlements must be included. Because of the specific location, the soil properties as well as foundation layout are main issues. Furthermore, information related to the strength of the different load-bearing elements and the quality of the brick and stone masonry have to be determined, to check the remaining safety of the structure. In the analysis phase, this information provides the data required to verify the load-bearing capacity of the different structural elements, as well as the time dependent settlements.

2.2.1 Geometry and structural layout

Sufficient data related the general geometry in plan and elevation were available from different sources, such as the city of Leuven (B), archives in the central library of Leuven and at the Monuments and Landscapes division of the Flemisch Government as well as from former studies performed by the Raymond Lemaire Centre for the Conservation of Historic Towns and Buildings at KULeuven.

In addition, and related to the three dimensional layout of the vaults of the main nave and side naves, their layout was acquired in detail in 1995 in the Phidias project (Van Balen et al., 1995). By means of a demo – a point cloud was acquired from a part of the vaults of the main nave using the promising 3D-scanning technique (Schueremans et Van Gemert, 2006a).

From a visual inspection of the masonry pillars in the main nave, the former strengthening is clear. To increase the load-bearing capacity, a hoop reinforcement and vertical reinforcement is added (1802-1806). At several locations, the original natural stones (Ledian lime-sandstone) (B) have been replaced with a local Doornikse (B) (E: Tournaisian) stone, having a higher strength and stiffness. Endoscopes in boreholes (ϕ=25 mm) learn that, in contradiction to the initial assumption, the cross-section of the main pillars is made out of regular blocks of natural stones. No traces of a core filled with rubble masonry could be found.

2.2.2 Settlements

Large differential settlements have taken place over a period of almost 8 centuries. Three different techniques have been used to quantify these settlements:

- In 1994 63 geodetic leveling points have been installed around the church. A topographic leveling has been performed at several discrete moments in time. This results in a mid-long term record of vertical settlements of the main structural elements of the church;
- In 1994 a Hydrostatic Leveling System (HLS) (Schueremans et al., 2006b) has been installed in the main nave as well as in the tower. The 8 measuring units allow following the relative displacements of these points with an accuracy of 0.01mm. These measurements were captured during a period of 5 months, resulting in a short time record of highly accurate data;
- In addition to these monitoring systems, the resulting slant of the tower and pillars has been recorded. A topographic survey was conducted resulting into 27 vertical profiles indicating the slant of the major structural components (Schueremans et al., 2006a).

From these data, the slant of the tower and its evolution as a function of time can be retraced. Furthermore, since different techniques are used, the accuracy of the different techniques could be compared (Schueremans et al., 2006). It could be concluded that the leaning is still continuing, despite of the present shoring: 1.5 mm/m/century to the north and 2.5mm/m/century to the east. The actual speed is lower than the global average, which is not surprising since the clay-containing subsoil has been able to consolidate nearly for 100% over a period of 8 centuries.

Additional information regarding the stability of the masonry pillars of the main nave is obtained from the geodetic leveling points. At the north side of the main nave near the tower, large differential settlements occur, with an order of magnitude of 9 mm, compared to the former campaign 5 years ago. These are acquired not only from the leveling points on the pillar, but also from the leveling points on the reinforced concrete foundation slabs of the steel tube shoring. This indicates a continuous settlement of this specific area.

2.2.3 Foundation system

Investigation pits were dug at a pillar of the main nave, a cross-column and a pillar from the side nave. From these in site investigations the geometry of the foundation was retrieved, Figure 4. The quality of the masonry and the bottom level of the foundations are searched for by means of a core drilling through the foundation masonry. The cores revealed that the masonry was in excellent condition, built with regular blocks of natural stone and lime mortar joints. At the bottom of the masonry pillars, wooden elements were retraced. It is not clear whether these elements originate from beams or pillars. More important however is the fact that these wooden elements are rotten, limiting the overall load-bearing capacity of the columns. This is surprising, since these elements are located far below the phreatic surface.

2.2.4 Subsoil

To enable a good estimate of the differential settlements as well as the load-bearing capacity of the known foundations, four 200 kN cone penetration tests (Van Balen et al., 1995) as well as four in site vane shear tests are performed (Schueremans et al., 2006a), Figure 4. The latter are used to determine the cohesion of the clay-containing subsoil at foundation level. In fact the contribution of the cohesion in the load-bearing capacity is significant.

2.2.5 *Material properties*

The XIX°C strengthening of the main pillars with steel rings was inspired by doubt in their load-bearing capacity. Therefore, the compressive strength of the masonry has been determined. To do so in a least-destructive way, representative samples of bricks and natural stones are collected in site. Additionally, mortar samples are taken at 10 different locations, overlapping with the different building phases during history. From the chemical analysis of the mortar samples, their composition is looked for. Based on the type and amount of binder, an estimate of the compressive strength is made. The compressive strength of the composite masonry is then retrieved from the components' compressive strength by means of a numerical relationship proposed in EC6 (EN1996,2002; Schueremans, 2001; Schueremans and Van Gemert, 2006a,b). Detailed values are reported elsewhere (Schueremans et al. 2006a).

3 ANALYSIS

To get an overall idea of the remaining structural safety, the main structural components are analysed in more detail. More specifically, their load-bearing capacity and time-dependent settlements caused by vertical loading originating from proper weight and permanent loads are looked into. The behavior of following structural elements is investigated, Figure 1:

- Western tower – west side;
- Western tower – east side: both sides of the tower are investigated separately for two reasons. Due to the main vault at the first floor, the loads of the tower are guided towards the corners of the tower. At the west side, the cross-section of the load-bearing structure is significantly larger. At the east side of the tower, half of a column is present as well as the additional load coming from half a section of the main nave;
- Column main nave: the columns of the main nave are heavily loaded;
- Cross column: the cross-columns have a significantly larger cross-section compared to the masonry pillars of the main nave;
- Column transept: different loading compared to the columns of the main nave;
- Column side nave: different loading compared to the columns of the main nave.

3.1 *Load-bearing capacity of structural components – Ultimate Limit State*

Table 1 presents the results of the vertical stresses in the masonry, compared to the compressive strength of the masonry of the aforementioned structural components.

Table 1: Results of load-bearing capacity of masonry structural components

Structural component	Design stresses in masonry $\sigma_{m,d}$ [MPa]	Design masonry strength $f_{m,d}$ [MPa]	Check (safety margin)	Failure probability p_f	Reliability index β
Tower – west side	0.64	1.81	ok; (1.6)	$5.8\ 10^{-14}$	7.4
Tower – east side	1.33	1.81	ok; (1.36)	$2.8\ 10^{-9}$	5.8
Pillar – main nave	3.14	2.21	not ok; (0.70)	$2.2\ 10^{-5}$	4.09
Pillar – crossing	1.28	2.21	ok; (1.72)	$6.0\ 10^{-11}$	6.44
Pillar – side nave	0.45	2.21	ok; (4.91)	$< 10^{-16}$	>8
Pillar - transept	0.66	2.21	ok; (3.35)	$< 10^{-16}$	>8

From table 1, it is clear that the load-bearing capacity of the masonry is sufficient in all cases, except for the pillars of the main columns. Since this structural element is no longer in accordance to the general methodology, according the partial safety method, outlined in Eurocode 6, the actual failure probability is estimated as well. From the analysis, it is clear that the remaining safety is acceptable. A target value for the failure probability has been determined to be $p_{f,t} = 5\ 10^{-5}$ (Schueremans, 2001; Schueremans and Van Gemert, 2006b).

3.2 *Load-bearing capacity of the foundation – Ultimate Limit State*

Because of the large settlements encountered during history, a profound analysis of the load-bearing capacity of the foundation of the different structural components is indispensable. Table 2 summarizes the results of this analysis. The subsequent columns present the actual soil stresses, the load-bearing capacity of the subsoil and the remaining safety as a ratio between the latter ones. It accounts for the geometry of the foundation, the ground water level and the soil properties derived from the in site vane shear tests and cone penetration tests. From Table 2, it is clear that in none of the examples – except for the pillars of the side nave - the capacity fulfills the current standards. Additionally, one can clearly see that the size of the remaining safety margin is most critical for the pillars of the main nave, the east side of the tower and the pillars of the crossing. These are the foundation elements focused on in the consolidation.

Table 2 : Results of load-bearing capacity of foundation subsoil for different structural components

Structural component	Design stresses at foundation $\sigma_{g,d}$ [MPa]	Design strength subsoil $f_{g,d}$ [MPa]	Check (safety margin)
Tower – west side	0.43	0.24	not ok; (0.56)
Tower – east side	0.76	0.24	not ok; (0.32)
Pillar – main nave	1.01	0.28	not ok; (0.28)
Pillar – crossing	0.89	0.40	not ok; (0.45)
Pillar – side nave	0.22	0.26	ok; (1.18)
Pillar - transept	0.45	0.26	not ok; (0.58)

3.3 *Time dependent settlements of structural components – Serviceability Limit State*

The time-dependent settlements are based on the formula of Buisman and Koppejan (Schueremans et al., 2006). In Figure 3 the settlements are outlined as a function of time for the west and east side of the tower as well as for the pillars of the main nave. It is clear that large differential settlements can be observed in between the different load-bearing elements.

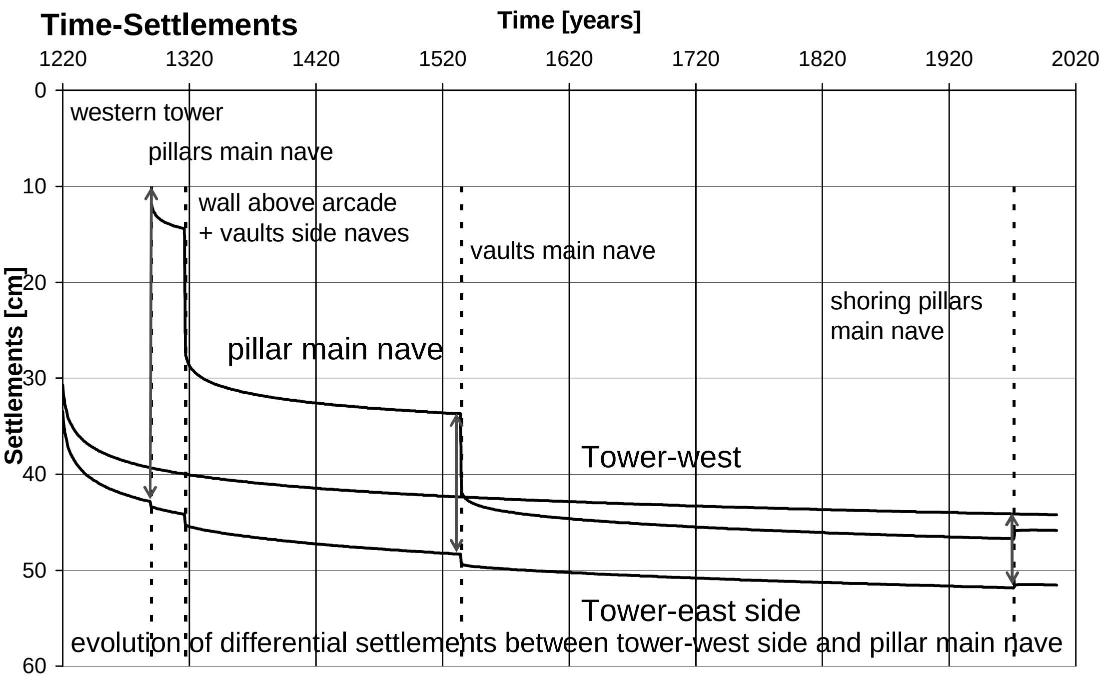

Figure 3 : Time dependent settlements

The east side of the tower shows larger vertical deformations compared to the west side, causing its leaning over to the east side. The large settlements of the pillars of the main nave, constructed approximately 70 years after the construction of the tower, are significantly larger then those of the tower itself. This clarifies that the tower is not pushing against the main nave, since the latter is moving down faster. These analyses results are very relate to the observed crack pattern, the instability of the flying buttresses and of the vaults of the side naves.

4 DIAGNOSIS

The main structural problems of the church originate from a lack of load-bearing capacity at foundation level. The foundation system of the main pillars originally has not been designed for the actual loads. On top of that, material degradation of the wooden elements at the bottom level of the foundation, leads to punching of the pillars through the foundation subsoil and thus increased differential settlements. These differential settlements clearly exceeded the structural ductility of the vaults of the side naves, the walls above the arcade as well as the flying buttresses. Since the settlements of the north and south side of the main nave are comparable, the damage to the vaults of the main nave is limited. Although the pillars of the main nave are heavily loaded, their design strength is judged to be sufficient. The tower is leaning to the east because of the higher loads at the east side in combination with a smaller load-bearing cross-section. Although the tower has a symmetrical layout, it is leaning to the north. This might originate from the fact that the tower has been constructed on top of an old Romanesque church, from which remains of the foundation have been found mainly at the south side. In that the foundation system is no longer symmetric although the superstructure of the tower itself is.

5 THERAPY

An increase of the load-bearing capacity of the foundation of the pillars in the main nave is indispensable. Even with the actual shoring, some of the pillars as well as the added shoring tube structure shown significant vertical settlements, as observed from the geodetic leveling survey.

For the pillars of the main nave, a micro-piling with 8 piles corresponding with the octagonal cross-section of the pillars is proposed, Figure 4. In addition the foundation of the cross-pillars is strengthened with 12 micro-piles. The on set of the piling, with a length of 12 m, will be in the tertiary highly consolidated, or moderately consolidated clay-containing sand to provide sufficient point-resistance.

After consolidation of the pillars of the main nave and cross-pillars (west side) a new risk of the leaning of the tower is present, since both elements will be on a different foundation system. The vertical settlements of the pillars after consolidation will be very limited; the additional settlements of the tower are much more difficult to estimate. As a result, what is not the case at this moment in time – the pushing of the tower against the walls of the main nave - might become possible in the future. Therefore, three options are proposed. The first is to provide also a pile foundation under the tower, resulting in a similar foundation system for both elements and stabilizing all central movements. The second is to provide a long term monitoring of the slant of the tower. Based on the evolution of the slant, the necessity of additional strengthening of its foundation will be judged on regular time intervals. The third option is similar to the initial proposal, made in 1961. Detaching the tower from the main nave will prevent the risk of the tower pushing against the main nave. Since the foundation of the tower has proven to be sufficient for over more then 780 years, its load-bearing capacity might be judged to be sufficient, although it does not comply with actual design standards. Based on the calculations of the time-dependent settlements, the additional settlements of the tower are estimated to be limited: 0.3 cm for the oncoming century. Before, during and after detaching, a permanent monitoring of the slant of the tower enables to accurately follow up the consolidation measures taken. It provides the possibility to interfere and take action whenever required.

After consolidation of the foundation, the steel tube shoring can be dismantled in a controlled way. The horizontal support of the reinforced concrete columns against the cross-pillars will be replaced by means of horizontal anchors in the longitudinal direction (in the wall above the arcades) and in transversal direction (in the wall of the transept). In this way, the slender columns will again encounter sufficient lateral support without the presence of the reinforced concrete columns.

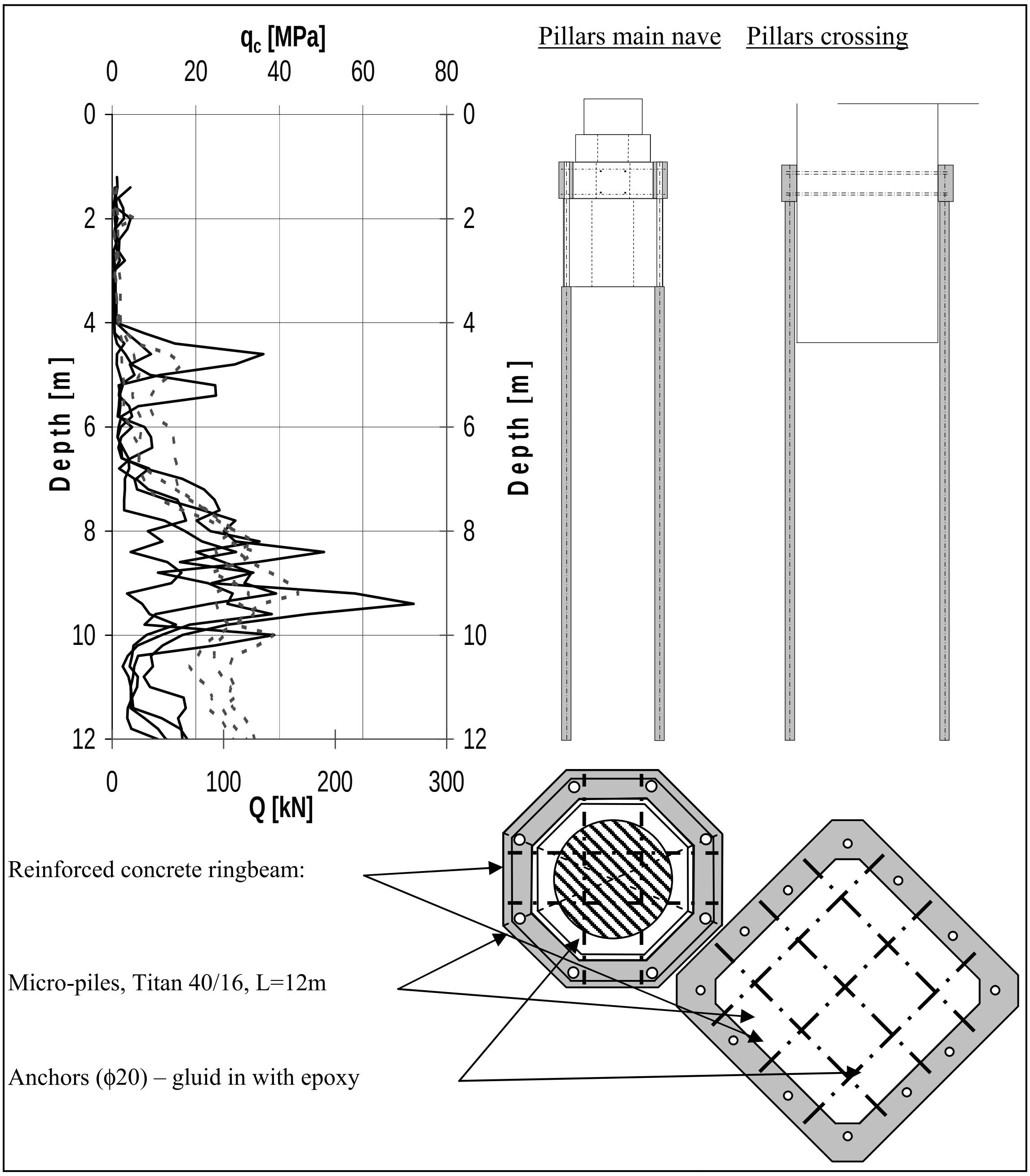

Figure 4 : Foundation strengthening of pillars

6 CONTROL

The impact of the consolidation measures will be monitored. This monitoring system will be operational one year before the actual consolidation of the foundation will be executed. In doing so, the order of magnitude in the evolution of the measured variables as a function of time, temperature and relative humidity are known beforehand. This is necessary for the responsible structural engineer to correctly judge the acquired data during the consolidation itself as well as afterwards. The devices will be installed at the abutments of the vaults in the main nave and transept. Target is to measure the main longitudinal and transversal distances. From that the evolution of the slant of the tower can be followed as well as the impact caused by the different foundation systems between the main nave and the choir that will appear once the foundation consolidation has been executed. The transversal distances in the main nave allow monitoring the effectiveness of the new tie-rods or reconstructed flying buttresses.

The dismantling of the steel-tube shoring of the pillars in the main nave will be used to experimentally verify the compressive stresses in the tubes and pillars. Strain-gauges will be fixed to the tubes and pillars and the deformations will be recorded before and during dismantling of the shoring structure. This enables to quantify the actual vertical force acting on the columns, as well as give an indication of the part of the loading still acting on the pillar and the part transferred to the steel tube shoring.

7 CONCLUSIONS

The church of Saint James revealed to have substantial lack of load-bearing capacity, resulting into structural problems and fragmented strengthening at several moments in history. In 2005, the city of Leuven ordered a structural safety analysis with the objective of a multi-functional re-use of the building. In this paper, the main findings are presented, outlined in correspondence with the general framework of the preservation process.

Initially the information is gathered form in site data, historical sources and former studies to get an overall idea of the actual situation the structure is in. In the analysis phase, the load-bearing capacity of the different structural components is checked as well as their time-dependent settlements. The combination of in site data and numerical simulation allows diagnosing the present crack pattern, the instability of the dismantled flying buttresses and the vaults of the side naves. Starting from a critical survey of the strengthening concepts dating from 1963, a new consolidation concept is proposed that aims at preserving the church for its future multi-functional re-use. With an adequate monitoring campaign, it is planned to verify the impact of the proposed measures on the short and mid-long term.

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