LARGE SCALE LOADING TESTS ON A 70-YEAR OLD POST-TENSIONED CONCRETE BEAM

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Abstract

The UCO textile factory was built in Ghent (Belgium) in the period 1947-1948. The roof structure covered an area of about 30 000 m² and consisted of 100 primary beams and 600 secondary beams. Corbels that were attached to the primary beams served as supports to the secondary beams. The post-tensioned beams were designed by prof. Gustave Magnel, who was a world leading expert in the field of prestressed concrete. Also unique was the fact that the beams were precast at the building site. This project was one of the first important large-scale applications of prestressed concrete in industrial buildings in the world. Recently, part of the factory building was demolished and two of the primary beams of the roof structure were transferred to the Magnel Laboratory for Concrete Research for testing at an age of 70 years. The primary beams have a span of 20.5 m and a maximum depth of 1.7 m. Only prestressing tendons (Blaton-Magnel system with Ø 5mm wires) were present and no passive reinforcement nor stirrups were provided along the length. Only in the end blocks stirrups were present to resist the splitting forces. This paper describes the results of a loading test up to failure on one of these beams. The experimental results are compared with results from analytical calculations. Furthermore, the paper presents some information related to time-dependent prestress losses after 70 years in service. The results provide important information regarding the assessment of existing post-tensioned concrete structures dating from that pioneering period.

Keywords: *Large scale testing, post-tensioning, prestress losses.*

1. Introduction

The textile factory 'Union Cottonière' in Ghent was constructed in 1947-1948 (Figure 1). At the time of construction, it was considered to be the most important work of this kind in building construction due to the large scale application of prestressed concrete and prefabrication (Magnel, 1954). The single-single story factory covers an area of about 30 000 m². The beams of the flat roof are in prestressed concrete, cast on the ground and lifted into position. The columns are spaced in a regular grid of 21.6 m x 14.4 m. The primary beams have a nominal span of 20.5 m and are supported on concrete corbels which are monolithically attached to the columns. Each side of these beams have corbels carrying the secondary beams which have a nominal span of 13.7 m. The secondary beams are placed at an inter-distance of 3.6 m and in turn carry tertiary reinforced concrete beams. The latter beams are I-shaped and the top flange supports precast slabs, while a suspended ceiling is supported by the bottom flanges.

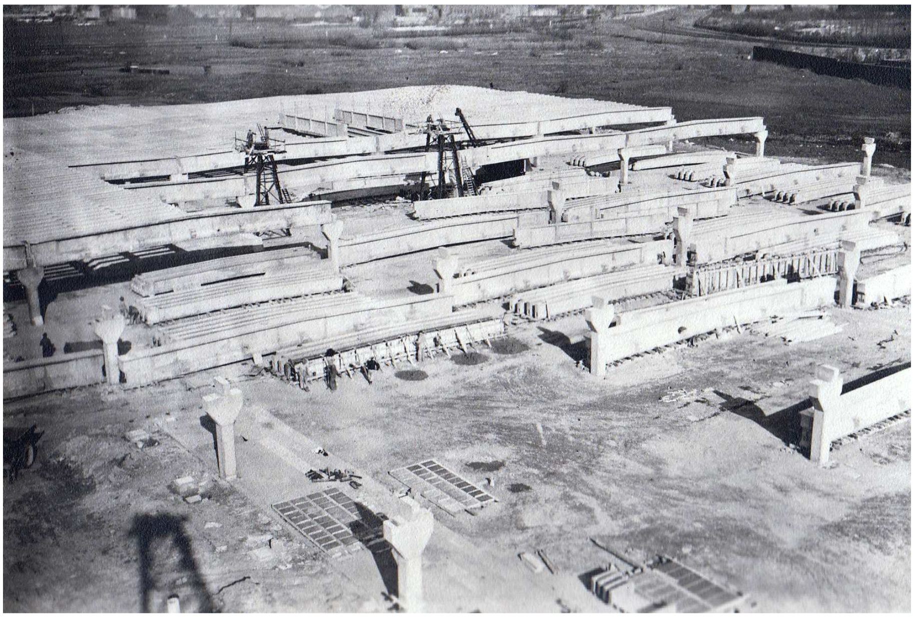

Figure 1: Construction of the textile factory ‘Union Cottonière’ in Ghent (1948)

According to (Magnel, 1948), the weight of the roof was about 260 kg/m², and an additional load of 50 kg/m² was foreseen in the design. Consequently, the secondary beams carry about 16 000 kg and their selfweight is about 6 000 kg, while the primary beams carry a load corresponding to five secondary beams, or 110 tonnes, and their selfweight is 40 tonnes. In total, about 100 primary beams and 600 secondary beams were produced for the construction of the roof structure.

The primary beams have a height of 1.75 m with a top flange of 90 cm and a bottom flange of 50 cm (see Figure 2, (Magnel, 1948)). The prestressing reinforcement of these beams consist of three cables of 48 wires with a diameter of 5 mm, working at 1000 MPa at the moment of establishing the prestress (Magnel, 1948). Also, these beams do not have other reinforcement, except for some stirrups in the anchor blocks and some lateral reinforcement at the corbels which carry the secondary beams. Note that no supporting reinforcement is provided to anchor the corbels.

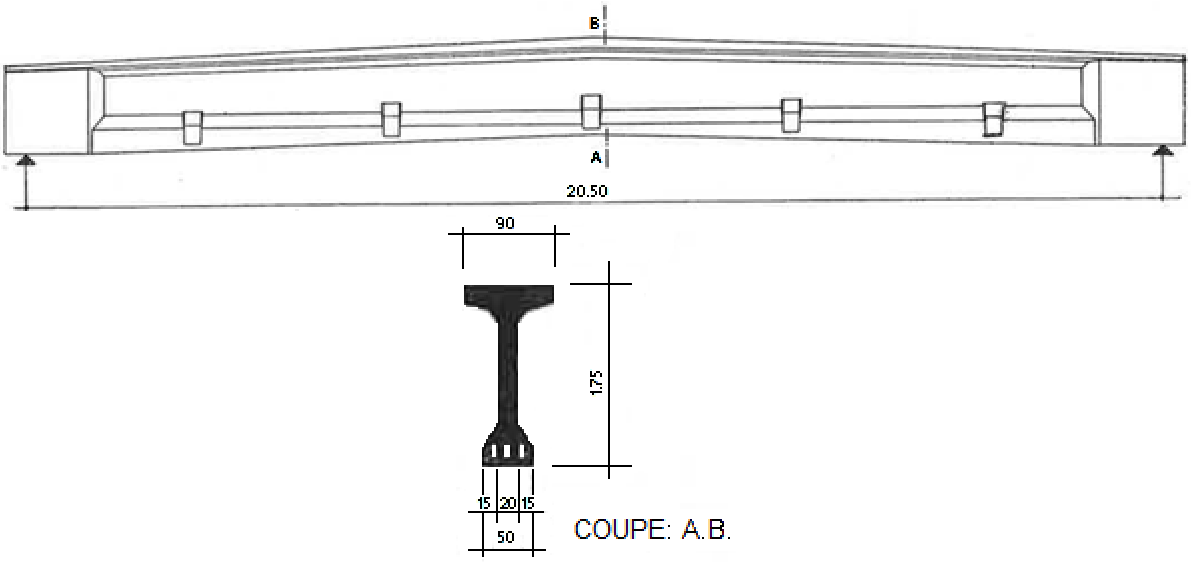

Figure 2: Schematic overview of the primary beams adopted from (Magnel, 1948)

Both the primary and secondary beams are pre-cambered, i.e. they have the form of an inverse V, in order to (Magnel, 1948) (1) facilitate the water flow on the roof and (2) maintain all cables straight.

According to (Magnel, 1948), the design was performed considering a maximum stress in the concrete of 13 MPa, while the compressive strength of concrete at the moment of prestressing was estimated at 40 MPa.

In 2016, part of the factory was demolished in the context of an adaptive refurbishment project. Subsequently, a primary and a secondary beam were transported for testing to the Magnel Laboratory for Concrete Research of Ghent University. The details of the experimental campaign on the primary beam are explained in the next sections.

2. Experimental test set-up

This section provides a description of the test set-up which was used for testing the primary beam. It should be noted that this primary beam differs from the typical primary beam as described in (Magnel, 1948) since the beam originates from a zone where an additional volume was present on the roof resulting in a specific geometry of the end section of the primary beam. In particular, this primary beam is prestressed by means of three cables of 56 Ø 5 mm which deviates from the description provided in (Magnel, 1948) and can be attributed to the previously mentioned specific load configuration and geometry of this particular beam. Furthermore, it should be noted that, apart from the prestress reinforcement and some reinforcement in the end blocks, no other reinforcement was detected in the beam, i.e. there is no supporting reinforcement at the position of the corbels.

Figure 3 shows the test set-up for the primary beam. The beam was simply supported using a hinged support at the left hand side and a roller support at the right hand side in the figure. The distance between the two supports was 20.5 m, which corresponds to the nominal span mentioned in (Magnel, 1948). Based on practical considerations, it was decided to load the primary beam up to failure by means of two point loads located at 7.25 m from the supports. At each loading point, two jacks with a capacity of 500 kN each were used to apply the force on a mortar embedded steel profile in order to properly distribute the two point loads on the top flange of the beam. The vertical reaction forces of the jacks were transmitted to an adjustable reaction frame that was anchored in the laboratory floor. The loads were measured by pressure sensors and the jacks were connected to a servo-controlled hydraulic unit.

Considering the slender geometry of the beam, lateral supports are provided at both end sections and near one of the load application points (see Figure 3).

Figure 3: Overview of the test set-up for the primary beam.

The measurement equipment used for measuring deformations was fixed to adjustable stands that were attached to a stiff aluminium frame. The following equipment was used:

(1) Displacement transducers (LVDTs);
(2) Demountable mechanical strain gauges (DEMECs) with a base length of 200 mm;
(3) Electronic deformation gauges (deformation gauges with a base length of 200 mm);
(4) A potentiometer at mid-span (connected to the bottom flange);
(5) Dial gauges.

The beam was surrounded by steel netting in order to prevent adverse events due to concrete spalling in the compression zone.

3. Test observations

This section presents to processed data obtained during the testing of the primary beam (section 3.2) as well as the results of the characterisation of concrete and prestressing steel properties (section 3.1).

3.1. Material characterisation

The mechanical properties of the concrete and the prestressing steel were determined based on specimens taken from another beam situated in the same building. The concrete compressive strength (according to EN 12390-3) and density were determined based on three cylindrical specimens with a height of 100 mm and a diameter of 100 mm. Furthermore, the characteristics of the prestressing steel were determined based on a tensile test executed on three wires according to EN ISO 15630-3. The results are presented in Table 1 and Figure 4.

Table 1: Mechanical properties of concrete and prestressing steel

	Concrete		Prestressing steel		
	$f_{c,cyl\ 100x100}$ [MPa]	**ρ_c [kg/m³]**	**$F_{p0.2}$ [MPa]**	**F_m [MPa]**	**E_p [GPa]**
1	54.7	2300	1525	1730	194.8
2	56.9	2280	1470	1690	194.9
3	46.3	2340	1440	1690	193.5
Mean	**52.6**	**2310**	**1478**	**1704**	**194.4**
Standard deviation	**5.6**	**30**	**43**	**24**	**0.8**

Notations: $f_{c,cyl100x100}$: compressive strength determined on cylinders with a height and diameter of 100 mm; ρ_c: mass density of concrete; $F_{p0.2}$: 0.2% strain limit; F_m: tensile strength; E_p: Young's modulus

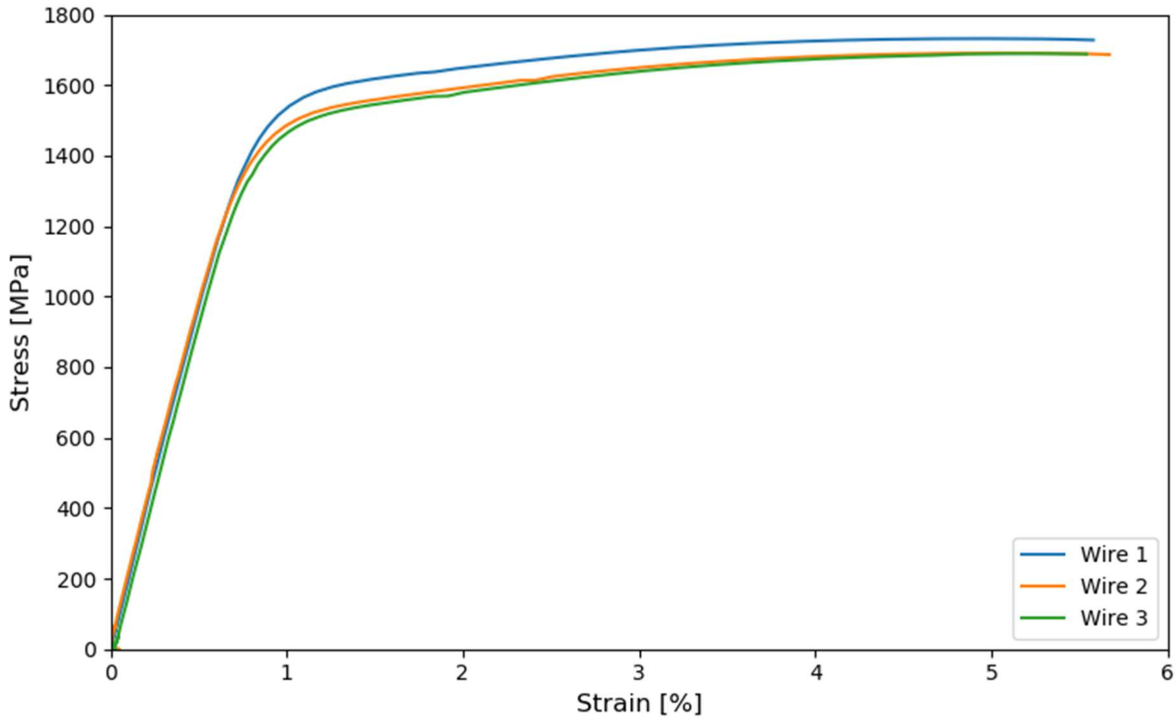

Figure 4: Stress-strain diagram of prestressing steel

Additional cores were drilled from remaining pieces of the test on the primary beams and a tensile splitting test has been performed on these cores, yielding an average value $f_{ct,sp}$ = 4.2 MPa. According to EN1992-1-1 (paragraph 3.1.2(8)) the average tensile strength can be calculated as $f_{ctm} = 0.9\ f_{ct,sp}$, i.e. f_{ctm} = 3.8 MPa.

3.2. Test results related to the primary beam

The test on the primary beam was executed in two phases. In the first phase, the load P applied at each of the two load application points was increased up to a load level higher than the cracking moment and subsequently the beam was fully unloaded. During phase 2 the load P was increased up to failure of the beam. Phase 2 was executed two weeks after phase 1.

It should be noted that due to the limited stroke of the jacks (i.e. 125 mm), the jacks had to be readjusted while the loads were taken over by an auxiliary reaction system. This was achieved by temporarily fixing two transverse spreader beams to the reaction frame. In this way, the jacks could be unloaded and readjusted, thus allowing to restart the load application without significantly influencing the beam's deformation state.

3.2.1. Load-deflection measurements

The load-deflection diagram is shown in Figure 5, showing the load P in each of the two load application points as a function of the vertical deflection at mid-span measured by means of an LVDT. Additionally, the displacements obtained by means of dial gauges at discrete load levels are also shown in Figure 5. The load-deflection curve related to phase 1 shows a linear elastic behaviour up to approximately 450 kN, after which a nonlinear behaviour is observed up to 580 kN. Subsequently, the load is completely removed and a residual deflection of 2.5 mm is found.

The load-deflection curve of phase 2 shows a linear elastic behaviour up to a load level of approximately 400 kN after which a nonlinear behaviour is observed. At a mid-span deflection of 140 mm, the take-over procedure was executed. The beam failed at a load level of approximately 800 kN per load application point (or 1600 kN in total) and a mid-span deflection of 170 mm, at a slightly higher load after the take-over procedure.

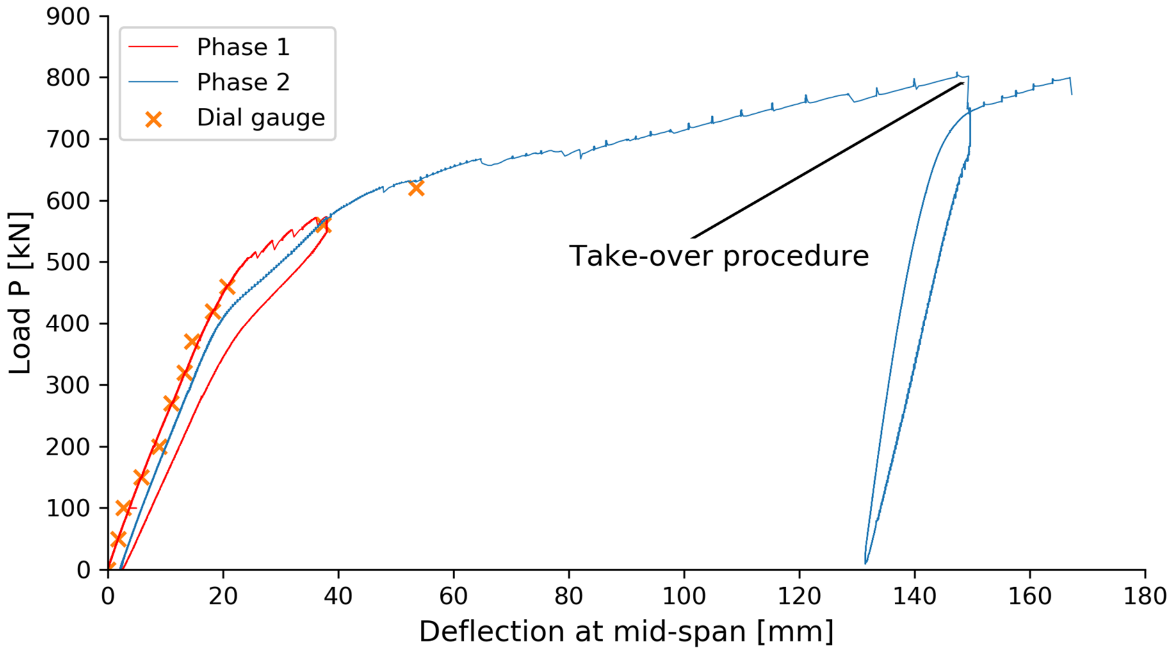

Figure 5: Load applied in one load application point as a function of the displacement at mid-span

3.2.2. Deformation measurements

The continuous deformation measurements obtained by electronic deformation gauges of a bottom fibre near the cross-section located around mid-span are presented in Figure 6 and Figure 7 for phase 1 and 2 (up to 600 kN) respectively.

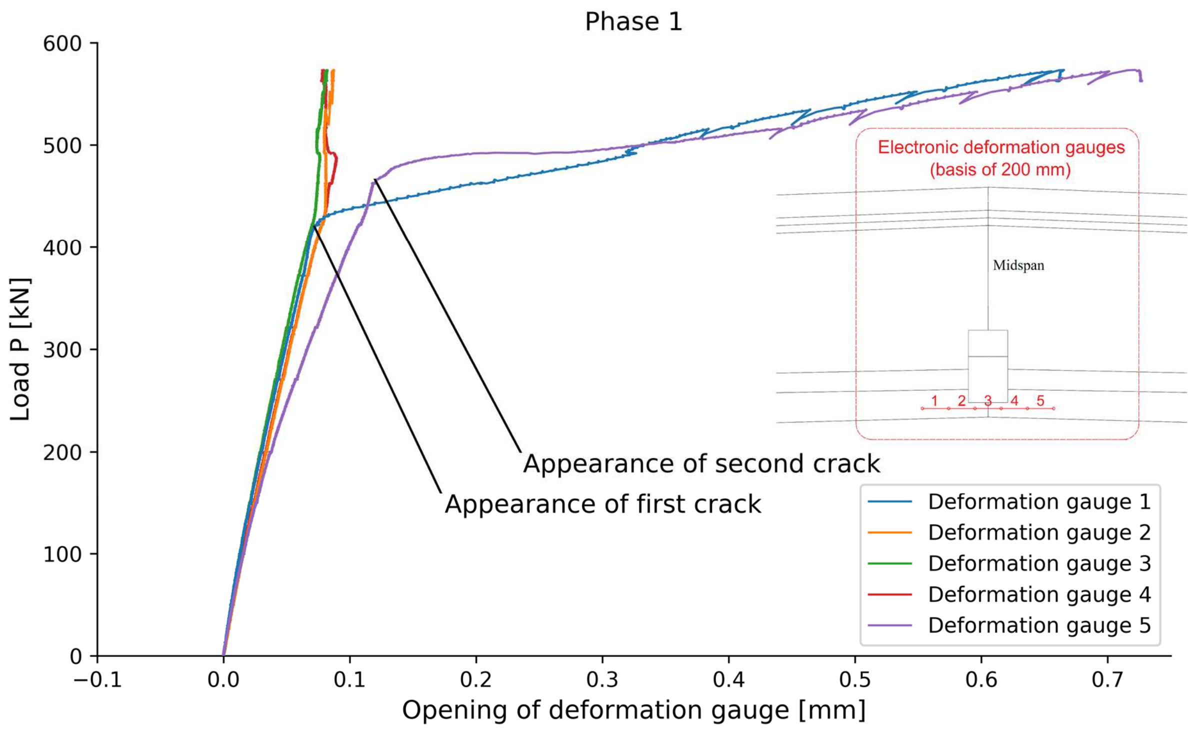

Figure 6: Deformation of the bottom fibre in phase 1 measured by means of electronic deformation gauges (positive values refer to opening of the deformation gauge)

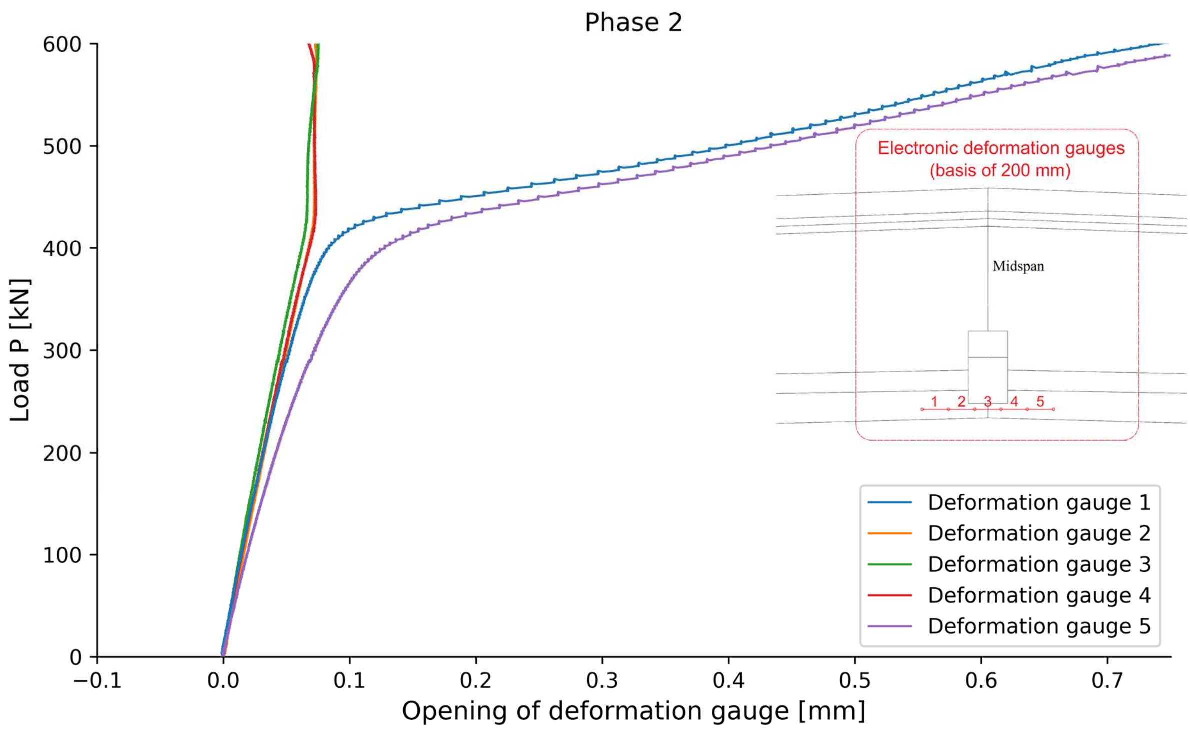

Figure 7: Deformation of the bottom fibre in phase 2 (up to 600 kN) measured by means of electronic deformation gauges (positive values refer to opening of the deformation gauge)

The deformation measurements related to phase 1 allow to detect the moment of cracking. Figure 6 shows that the first and second crack appear in the zone covered by deformation gauges 1 and 5 at a load level of approximately 420 kN and 460 kN, respectively. These load levels correspond approximately to the earlier observed load level at which the load-deflection behaviour becomes nonlinear.

The deformation measurements related phase 2 allow to detect the moment of reopening of the previously formed cracks, i.e. the moment of decompression of the bottom fibre. Figure 7 shows that the first crack reopens at a load level of approximately 300 kN (reflected by the difference in deformation of e.g. deformation gauges 1 and 2).

It is noted that the measurements obtained from gauge 5 show a less stiff behaviour compared to the behaviour of the other deformation gauges. This can be attributed to the fact that a crack was present at that location before testing, probably caused by transport of the beam from the site to the laboratory.

3.2.3. Crack pattern

The crack pattern at a load level of 700 kN is shown in Figure 8. Two types of cracks can be observed:

1) Bending cracks are present between the two load application points. These cracks developed first, i.e. at relatively low load levels;
2) Two large shear cracks extending from the load application points towards the supports. These cracks developed simultaneously at a load level of approximately 650 kN.

The failure of the primary beam was finally caused by propagation of the shear crack, which can mainly be attributed to the lack of shear reinforcement.

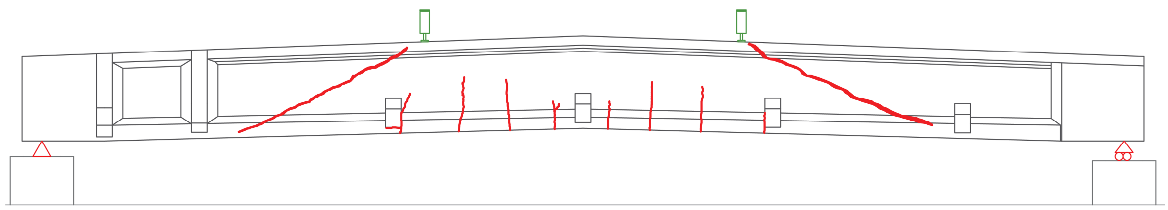

Figure 8: Crack pattern observed at a load level of 700 kN

4. Preliminary analytical analysis

In this section, some preliminary analytical calculations are presented in order to evaluate the results obtained from the experimental test presented in section 3.

4.1. Service load

Considering the loads indicated in section 1 (based on (Magnel, 1948)), the mid-span bending moment due to imposed loads on the primary beam is approximately equal to 2765 kNm (with partial factors being considered). Considering the load configuration adopted in the test set-up presented in section 2, the latter corresponds to a load level P_g of:

$$P_g = \frac{2765 \text{ kNm}}{7.25 \text{ m}} = 381\ kN \tag{1}$$

It is noted that this load level P_g is close to the load level at which the first crack developed (P ≈ 420 kN). This shows that the design of these beams in the pioneering period of prestressed concrete was really at the limit in order to prove the economic advantage of using this new construction type.

4.2. Determination of moment of cracking and residual prestress

Considering the load levels corresponding to decompression of the bottom fibre ($P_0 \approx 300$ kN) and the appearance of the first crack ($P_{cr} \approx 420$ kN) according to the observations made during the test (see section 3.2.2), the remaining prestress $\sigma_{p\infty}$ can be estimated analytically. The stress conditions corresponding to decompression and cracking are, respectively:

$$\sigma_{p1} + \sigma_{g1} + \sigma_{q1} = 0 \tag{2}$$

$$\sigma_{p1} + \sigma_{g1} + \sigma_{q1} = f_{ctm} \tag{3}$$

where σ_{p1} is the stress at the bottom fibre due to prestress, σ_{g1} is the stress at the bottom fibre due to the selfweight of the beam and σ_{q1} is the stress at the bottom fibre due to the load P. These equations can be rewritten as follows:

$$\frac{\sigma_{\mathrm{p}\infty} \cdot A_{\mathrm{p}}}{A} + \frac{\sigma_{\mathrm{p}\infty} \cdot A_{\mathrm{p}} \cdot e \cdot a_1}{I} - \frac{(M_{\mathrm{g}} + M_{\mathrm{P_0}}) \cdot a_1}{I} = 0 \tag{4}$$

$$\frac{\sigma_{p\infty} \cdot A_{\mathrm{p}}}{A} + \frac{\sigma_{\mathrm{p}\infty} \cdot A_{\mathrm{p}} \cdot e \cdot a_1}{I} - \frac{(M_{\mathrm{g}} + M_{\mathrm{P_{cr}}}) \cdot a_1}{I} = f_{\mathrm{ctm}} \tag{5}$$

with A_{p} the cross-section of the prestressing reinforcement (3297 mm²), e the eccentricity of the prestressing reinforcement (781 mm), a_1 the distance between the centre of gravity of the cross-section and the bottom fibre (956 mm), A the transformed cross-section (656,835 mm²), I moment of inertia of the transformed cross-section (242.32 10^9 mm^4), M_{g} the bending moment due to selfweight (0.71 10^9 Nmm), M_{P0} the bending moment due to the load P_0 (2.175 10^9 Nmm), M_{Pcr} the bending moment due to the load P_{cr} (3.045 10^9 Nmm) and f_{ctm} the tensile strength (-3.8 MPa, see section 3.1). The previously indicated cross-sectional properties refer to a cross-section near mid-span.

Based on the latter, both Equation (4) and (5) result in $\sigma_{\mathrm{p}\infty} \approx 750$ MPa approximately. This means that the total prestress losses amount up to 25%. The latter value includes the immediate prestress losses (due to e.g. slip) as well as the time-dependent losses after 75 years in service.

5. Future research

Future research efforts consist of the analysis of a secondary beam and the comparison of both beams. Furthermore, advanced finite element models will be developed in order to allow for more detailed structural analyses of both primary and secondary beams. Finally, Bayesian updating will be used to update material characteristics as well as prestress losses based on the experimental observations.

6. Conclusions

This paper describes the experimental test on a large post-tensioned beam dating from the pioneering period of prestressed concrete. The test consisted of two phases: (1) the goal of phase 1 was to determine both the moment of cracking and the moment of decompression, which allowed to determine the remaining prestress, while (2) in phase 2 the ultimate capacity of the beam was determined. It was observed that failure of the beam occurred due to the propagation of a shear crack. No shear reinforcement is present in the beam.

Analytical calculations showed that the loading of the beam in service conditions was very close to the moment of cracking observed in the experimental test. This shows that the design of these beams in the pioneering period of prestressed concrete was really at the limit in order to prove the economic advantage of using this new construction type.

Finally, it was shown that, based on preliminary analytical calculations, the prestress losses after 75 years in service amount up to 25%.

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