Structural Analysis of Historic Construction – D'Ayala & Fodde (eds)
© 2008 Taylor & Francis Group, London, ISBN 978-0-415-46872-5

Strengthening of an industrial cylindrical shell damaged by a collision

Wine Figeys, Sven Ignoul & Dionys Van Gemert
Department of civil engineering, KULeuven, Belgium
Triconsult N.V., Lummen, Belgium

ABSTRACT: This case-study concerns an old industrial reinforced concrete shell structure at the port of Antwerp. The building serves as a pilot for stowing of shipped steel and wooden products. A fork-lift hit one of the supporting columns. The impact of the collision caused the concrete to crush and the internal reinforcement to shift considerably. Some of the internal reinforcement yielded or broke. Temporary supports had to be placed to avoid collapse of the total structure. The stress situation is analyzed. A FEM model was built to gain insight into the stress distribution in the shell structure. In comparison with the undamaged situation, membrane forces, shear forces and moments did strongly increase which explains the actual damage. A solution is worked out to repair and strengthen the damaged part of the structure. The FEM-analysis demonstrates that a compression arch will arise in the top of the barrel shell to span the displaced support. This requires a tension member at the bottom side of the concrete barrel. Therefore, externally bonded reinforcement is applied to increase the tensile capacity of the perimeter beam. The web reinforcement has yielded or is broken so that it is replaced by external CFRP reinforcement. After strengthening the structure, a monitoring system is installed. Strain gauges are glued on several laminates. These measurements give feed back on the structural behaviour of the strengthened structure and the applicability of the FEM used. This case-study illustrates the load distribution mechanisms in the shell structure, the assessment of the actual condition and an appropriate intervention based on external reinforcement.

1 INTRODUCTION

An old industrial reinforced concrete shell structure (1955) at the port of Antwerp in Belgium is used as a pilot for stowing of shipped steel and timber products, Figure 1. This shell needs a strengthening after a collision of a fork-lift with a supporting column.

Restoration is generally based on a sequence of anamnesis, analysis, diagnosis, therapy and control (ICOMOS 2003), (Schueremans 2003):

- the **anamnesis**. Significant and objective information of the building is collected. Data can be gathered from literature, direct visual observation and field research;
- a structural **analysis** is performed;
- the causes of the damage and/or decay are determined in the **diagnosis**;
- if necessary, one or more **therapies** are proposed to repair and upgrade the building. In this case study, the technique of externally bonded reinforcement is applied;
- the last step is **control** whereby checks are carried out during and after intervention.

These basic steps are used as a guideline through the strengthening process.

Figure 1. The shell construction.

2 ANAMNESIS

The reinforced concrete structure consists of a sequence of 25 cylindrical barrel shells, supported by reinforced concrete columns. The span of each shell is 15 m. Each shell is supported by 3 columns at both sides. The thickness of the shell decreases from 13 cm at the abutment till 8 cm at mid span. In the mid part of each shell, a longitudinal roof light is located. In

Figure 2. Damaged column, December 2004.

Figure 3. Damaged column in the past.

December 2004, a fork-lift hit one of the 6 meter high concrete columns, Figure 2. This accident was not the first one, Figure 3. In November 2007 a new accident occured, 4.

2.1 *Visual inspection*

The settlement of the column and the crack formation were studied (Figeys and Van Gemert 2006). Measurements showed a horizontal displacement of about 14 cm at the impact location of the damaged column, Figure 2. The corresponding vertical (downward) displacement of the shell support equaled approximately 8 cm. This differential settlement resulted in severe cracking of the supported shell structure. Fortunately, only the two neighboring half parts of the shell were damaged. The roof lights in the top of the shell avoid force transfer to the other half of the shell.

At different places, the full concrete shell cross-section is cracked. The maximum crack opening was almost 20 mm, Figures 5 and 6. The cracks start from the support and run under an angle of 45° to the top of the shell, Figure 5. Due to the large displacements, part of the internal reinforcement yielded.

Figure 4. Recently damaged column, November 2007.

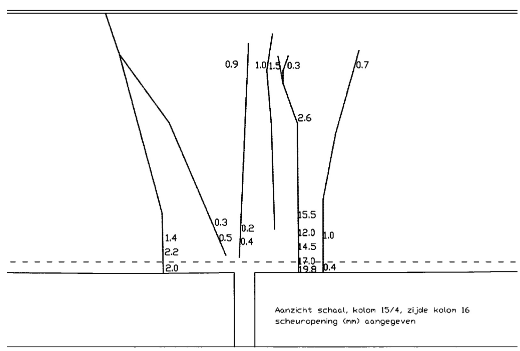


Figure 5. Observed crack formation and measured crack widths [in mm].

2.2 *Field research*

The concrete quality is determined experimentally by means of pull-off-tests (CEN pr EN 1542 1998). A mean tensile strength of 3.8 MPa is measured.

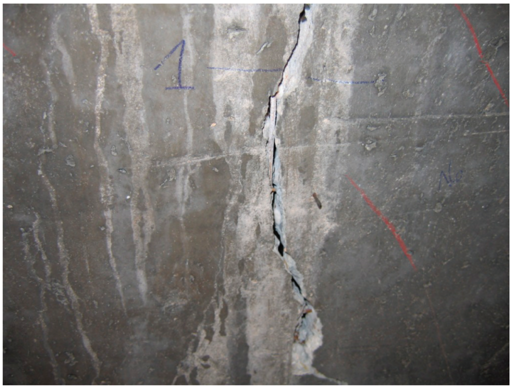


Figure 6. Crack above the damaged column with a maximum crack opening is 19.8 mm.

3 ANALYSIS AND DIAGNOSIS

The stress situation in the concrete shell is studied to determine the necessity and the amount of the strengthening. Therefore the concrete quality and the loads are determined.

3.1 *Material characteristics*

3.1.1 *Concrete*

The compressive strength and the Young's modulus is determined from the measured mean tensile strength according to Eurocode 2, (Eurocode 2 1999). The characteristic tensile strength of the concrete, f_{ctk}, is derived from the results of the pull-off-tests:

$$f_{ctk} = f_{ctm} - 1.64\sigma = 2.7\frac{N}{mm^2} \quad (1)$$

with

f_{ctk} the characteristic tensile strength
f_{ctm} the mean measured tensile strength = 3.8 N/mm^2
σ the dispersion = 0.7N/mm^2

The compressive strength, f_{ck}, is estimated using the inverse of formula 2 of Eurocode 2, 1999). The characteristic compressive strength, f_{ck}, equals 46.0 N/mm^2. The design value found with Equation 3, taking into account a partial safety factor of 1.5.

$$f_{ctm} = 0.30 f_{ck}^{2/3} \quad (2)$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} = 30.6\frac{N}{mm^2} \quad (3)$$

The Young's modulus can be determined using formula 4 and equals 35.9 kN/mm^2:

$$E_{cm} = 9.5(f_{ck} + 8)^{1/3} \quad (4)$$

3.1.2 *Internal reinforcement*

The reinforcement of the concrete shell exists of main steel bars and a web. The location of the bars (ϕ15 mm) correspond with the position on plan. The distance between the bars of the web (ϕ5 mm) is 150 mm in the longitudinal direction and 200 mm in the other.

Close to the column, the mean crack opening is $\Delta l_b = 15.9$ mm. The strain in the steel bars can be estimated by this elongation. Taking the anchorage length as the original length, the strain is equal to Equation 5. The anchorage length can be calculated according to Eurocode 2, Equation 6.

$$\epsilon = \frac{\Delta l_b}{l_b} = \frac{15.9mm}{l_b} \quad (5)$$

$$l_b = \frac{\phi}{4} = \frac{f_{yd}}{f_{bd}} \quad (6)$$

with

ϕ diameter of internal reinforcement
f_{yd} design yielding strength of internal reinforcement
f_{bd} design adhesion strength of concrete, 1.7 N/mm^2

The strain in the main reinforcement equaled 1.54% and in the web 4.62%. The strain of the main reinforcement is high. The bars yielded. However the strain remained below the ultimate stress level (steel BE 22: 18%). The strain in the web is higher, close to the ultimate stress level according to NBN 24-304 (6%). At the main crack, the web reinforcement had yielded and was possibly broken. Visual inspection confirmed the broken web bars.

3.2 *Determination of loads*

The design value of the load is calculated according to Eurocode 0 (Eurocode 0 2002) and 1 (Eurocode 1 1999). Besides of the self-weight of the concrete, also loads from wind (Eurocode 1 1995) and snow (Eurocode 1 2003) are taken into account.

The density of the concrete is determined by measuring the weights of small cubes of broken concrete of the column. The mean density equals 2268 kg/m^3, the characteristic value is 2419 kg/m^3.

The snow load, Equation 7, is counted as an enlargement of the self weight. As the wind load has an opposite and thus positive effect on the shell structure, it is neglected.

$$s = \mu_i C_e C_t s_k = 0.54\frac{kN}{m^2} \quad (7)$$

with

μ_i shape factor, equal to 1.08
s_k characteristic snow load in Antwerp
C_e exposure coefficient, equal to 1.0
C_t thermal coefficient, equal to 1.0.

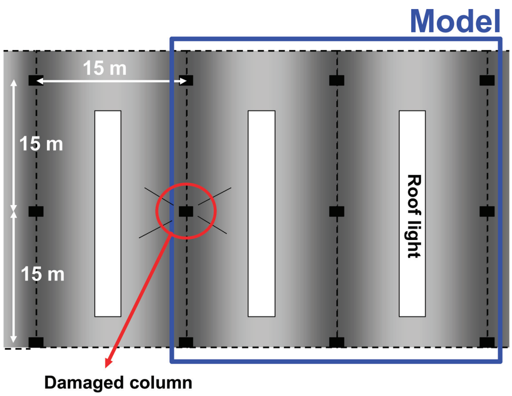


Figure 7. Schematic view of reinforced concrete shell construction.

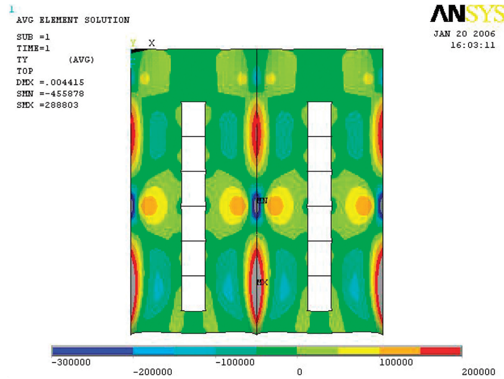


Figure 8. Calculation of the membrane forces in the Y-axis in the shell before the column settlement.

The increased density of the concrete is calculated in the ultimate limit state, Equation 10, and equals 40.76 kN/mm^3.

$$\rho_{UGT} = \gamma_g \rho_{G_k} + \gamma_s \rho_{S_k} \quad (8)$$

$$= 1.35 \cdot 24.19 \frac{kN}{m^3} + 1.5 \cdot \frac{0.54 \frac{kN}{m^2}}{0.1m} \quad (9)$$

$$= 40.76 \frac{kN}{m^3} \quad (10)$$

3.3 *FEM-analysis*

A FEM-model is built to analyse the stress distribution in the shell structure (Ansys 2005). As the damaged column is located in the central part of the building, edge effects can be neglected. Only the two neighboring shells are included in the FEM-model, the damaged column being located at the outer left part, Figure 7. Symmetry boundary conditions are applied to the left and right edge of the model.

The shell structure is modeled with the 8-node quadrilateral SHELL93 shell element, which has 6 degrees of freedom at each node: translation in the X, Y and Z direction and rotations around the X, Y and Z-axis. The beams around the roof lights are modeled by means of 2-node BEAM4 and BEAM44 elements, which have also 6 degrees of freedom at each node. These node displacements are translation in three directions and rotation around the three axes.

Membrane forces, shear forces and moments are calculated (Figeys and Van Gemert 2006). The original situation is studied, Figure 7. Tensile forces in longitudinal direction are present at mid span of the edge beam (with a maximum of 289 kN/m) and in the shell near the central column (150 kN/m), Figure 8.

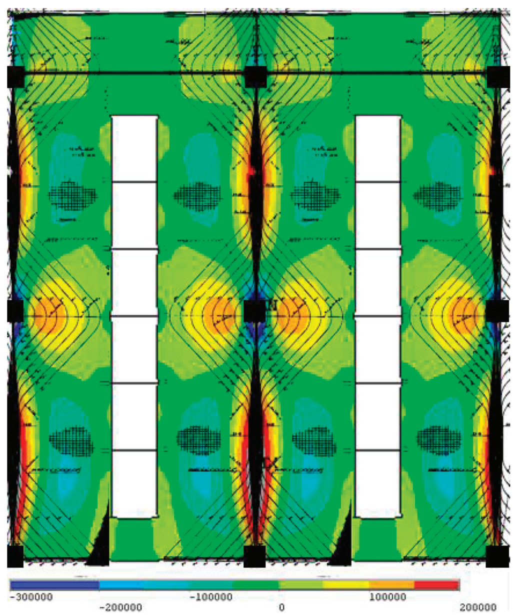


Figure 9. Original reinforcement located at high stress zones.

The original reinforcement plan (1955) show that the internal main reinforcement is bundled in the mid span of the beam and spreads out to the supports. This main reinforcement take care of the tensile stresses in the shell, Figure 9.

In comparison with the undamaged situation, calculations show that the membrane forces, shear forces and moments strongly increase at the imposed deformations. In particular, the membrane tensile forces in the longitudinal direction of the barrel

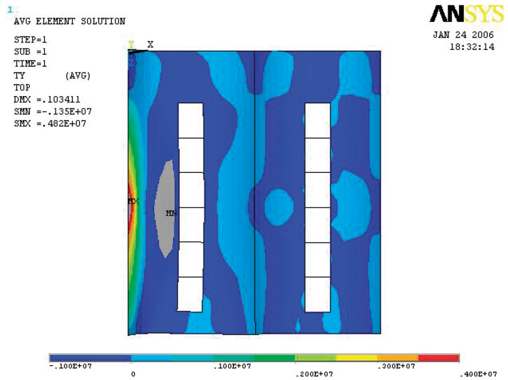


Figure 10. Calculation of the membrane forces in the Y-axis in the shell after the column settlement.

Figure 11. Jack up of the shell structure.

shells (longitudinal axis, Figures 8 and 10) increased extremely.

This caused the actual damage state and, moreover, changed the structural system. The damaged shell seems to be supported by the two outer columns only. High tensile stresses (with a peak of 4820 kN/m^2) appear in the edge beam at the missing support (i.e. damaged column). In the top of the shell compressive stresses in the longitudinal direction are introduced by the arch action. The available reinforcement in the edge beam at the support is insufficient, which led to the observed cracking of the concrete.

4 THERAPY

A method to repair and strengthen the damaged part of the structure was studied and executed. The FEM-analysis demonstrates that, due to the vertical settlement of the support, a compressive arch developed in the shell, that spans the displaced support. In order to restore as much as possible the initial stress-strain situation, the structure is jacked up, Figure 11.

However, it is not possible to neutralize the total vertical displacement. The plastic deformation of the internal reinforcement can not be recovered and chipping off of concrete would arise at excessive lifting. An additional tension member at the bottom edge of the concrete barrel should take up the induced tensile forces. For that purpose, externally bonded reinforcement is applied to increase the tensile capacity of the longitudinal beam.

The tensile force in the edge beam can be calculated as:

$$\frac{pL^2}{8f} = T_{Sd} \tag{11}$$

with

- T_{Sd} Design value of tensile force in the bottom of the barrel at one side
- L Span without damaged column, 30.0 m
- f Height of the shell, 2.95 m
- p Load per meter along the longitudinal axis, that equals:

$$p = \theta R d \rho_d \tag{12}$$

- θ Half of the opening angle of the shell, 43°
- R Radius of the shell trajectory, 10.9 m
- d Mean thickness of the shell, 0.10 m
- ρ_d Design value of load, including proper weight, wind loading, snow loading 40.76 kN/m^3.

The tensile force present at the beam for each half shell equals $T_{Sd} = 1285\,kN$. Taking into account the original reinforcement ($9 \times \phi 15$ mm, $T_{Rd} = 325$ kN), five CFRP laminates $100 \times 1.2\,\text{mm}^2$ are additionally needed to take of this tensile force, Equation 13. A safety factor $\gamma_{M,CFRP}$ of 1.5 is applied.

$$T_{Rd} = \frac{A_{CFRP} f_d}{\gamma_{M,CFRP}} = \frac{600mm^2 2500N/mm^2}{1.5} = 1000kN \tag{13}$$

Also the internal reinforcement mesh near the column is damaged: yielding and cracking are observed. However, because of the crack formation, the calculated stress peaks at the imposed deformations are reduced. Therefore, it is safe to take the stress level of the undamaged situation into account. The stress level, calculated with the FEM-model, is compared with the amount of undamaged reinforcement. In the longitudinal as well as in the transversal direction additional reinforcement is needed.

The tensile force in the longitudinal direction equals about 150 kN/m near the column and decreases to

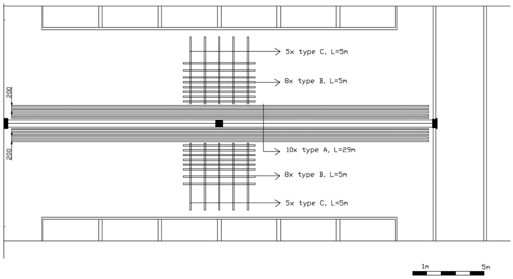


Figure 12. Proposed strengthening of the shell structure.

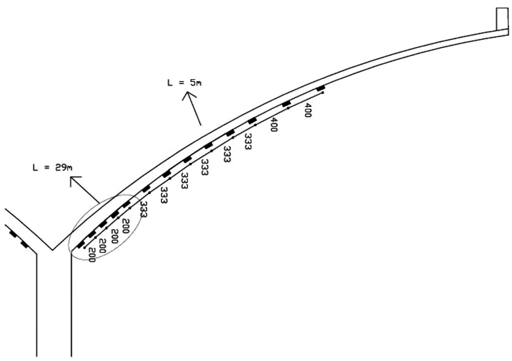


Figure 13. Section of the strengthened structure.

0 kN/m near the roof light. Also in the transversal direction, additional reinforcement is needed. Taking into account the maximal transferable load and the minimal needed anchorage length calculated from the pure shear model described by Brosens (Brosens 2001), the lay out presented in Figures 12 and 13, is chosen.

The strengthened shell construction is shown in Figure 14. After reconstruction of the damaged part of the column, Figure 15, the temporary supports were removed.

5 CONTROL

A monitoring system is installed to follow up the stress situation in the external reinforcement. Strain gauges are glued on several laminates, Figure 16. After repairing of the column, the temporary supports are removed. During this action, the different strain gauges are measured. The strain gauges are measured at different moments: before removing the supports (17.7°C), after removing (20.5° C), after 3 days (16.5° C) and after 31 days (22.6° C). Measurement of the temperature allows correcting the measured values to the effective stresses in the laminate. These stresses are

Figure 14. Strengthened shell construction.

Figure 15. New reinforcement and concrete for damaged part of the column.

presented in Figure 16. In Table 1, the stress levels after one month are given.

As the concrete column is replaced and the structure is jacked up, stresses as predicted with the original structure are expected, Figures 17 and 18. Close to the column, a peak tensile force of 150 kN/m is present. In the first meter near the column, 3 steel bars ϕ15 mm an 5 CFRP ($1.2 \times 100\ mm^2$) are present to take these tensile force. In the ultimate limit state a stress level of $182\ N/mm^2$ is expected. Taking into account the real

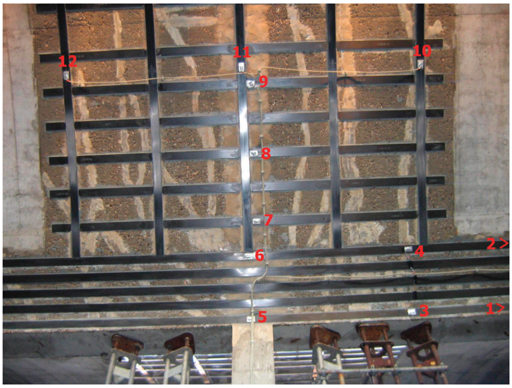


Figure 16. Location of the different strain gauge.

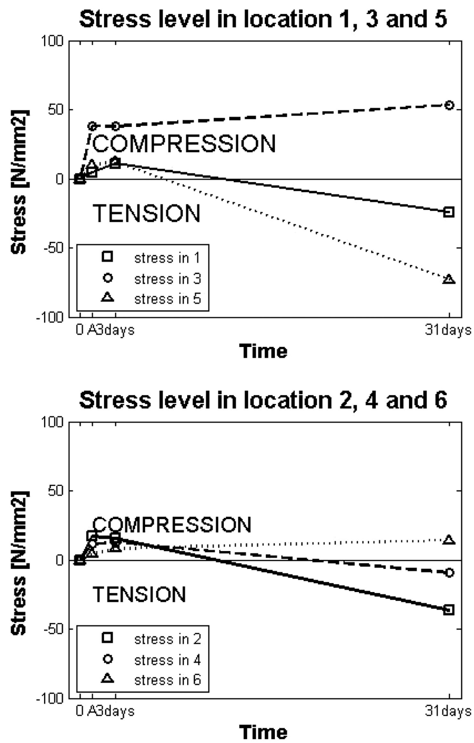
Table 1. Stress level in laminates on different locations.

Location	Stress [MPa]
1	−24.4
2	−36.5
3	65.8
4	12.8
5	−73.0
6	14.5
7	−78.5
8	−12.0
9	19.7
10	11.7
11	11.4
12	−40.3

+: compression; −: tension.

situation, only the mean density acting on the structure ($\rho_d = 22.68\, kN/m^3$ instead of $40.76\, kN/m^3$), the stress level will be reduced to about 90 N/mm^2. These tensile stresses can be observed in these locations (5 and 7), except in location 6.

Strain gauges 1, 2, 3 and 4 are located further on the laminates. In these zone the tensile stress decreases. Except for location 3, the tensile stress levels in these locations decreased. Immediately after removing the supports, a relatively large compressive stress is found in location 3. This location is situated at the bottom CFRP laminate, two meters from the concrete column. The measuring point is close to the injected crack with the largest opening of about 20 mm. After removing of the supports, the original situation is approached in which compressive stresses will appear in the bottom of the edge beam in the zone of the crack. These stresses will compress the epoxy resin in the crack. The laminate is compressed as well, which can be observed in the compressive stress curves.



0: before removing of the supports A: after removing of the supports

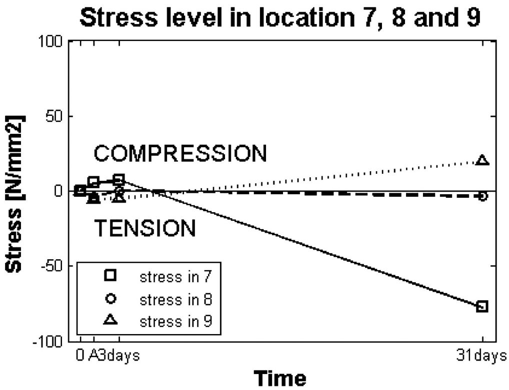
Figure 17. Stress level on different location derived from measured strains.

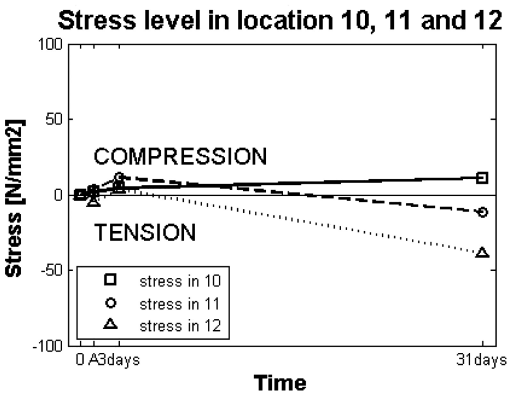
Location 8 and 9 are situated at the upper part of the shell, Figure 16. The tensile stresses decrease into the upper part to change in small compressive stresses at the top, as observed at location 9.

Three strain gauges measure the strain in the transverse direction. According to the calculations only small tensile stresses are observed, Figure 18.

6 CONCLUSIONS

Externally bonded CFRP-laminates are used to strengthen and repair a damaged shell structure. FEM analyses are used to determine stresses and strains in both the undamaged and damaged shell. This FEM-model allowed to identify the formation of a compressive arch in the damaged shell structure and to determine the required additional reinforcement, to be applied by means of externally bonded CFRP laminates on the edge beam and on the shell structure in the vicinity of the damaged column. Monitoring of stresses in the laminates during different phases of the





0: before removing of the supports A: after removing of the supports

Figure 18. Stress level on different location derived from measured strains.

repair showed the effectiveness of the repair procedure. As it was possible to jack up the shell and to restore the support, stress levels as predicted for the undamaged situation are observed.

ACKNOWLEDGEMENT

The authors would like to thank the Flemish Institute for Promotion of Scientific and Technological Research in the Industry (IWT – Vlaams Instituut voor de Bevordering van Wetenschappelijk-Technologisch Onderzoek in de Industrie) for their financial support.

REFERENCES

Ansys, I. (2005). Ansys version 10.0.

Brosens, K. (2001). Anchorage of externally bonded steel plates and CFRP laminates for strengthening of concrete element. Ph. D. thesis, Katholieke Universiteit Leuven, Leuven.

CEN pr EN 1542 (1998). Products and systems for the protection and repair of concrete structures – test methods-measurements of bond strength by pull-off.

Eurocode 0 (2002). Basis of design: Design principles.

Eurocode 1 (1995). Basis of design and actions on structures-part 2–4: Actions on structures – wind actions.

Eurocode 1 (1999). Basis of design and actions on structures-part 1 – 1: Actions on structures – densities, self-weight and imposed loads.

Eurocode 1 (2003). Eurocode 1 – basis of design and actions on structures – part 1–3: General actions – snow loads.

Eurocode 2 (1999). Design of concrete constructions – part 1–1 (in dutch).

Figeys, W. and D. Van Gemert (2006). Report d/00413/05 dd. 13-03-2006. shell repair. hybrid solution (in Dutch). Technical report, Triconsult N.V.

ICOMOS (2003). Recommendations for the analysis, conservation and structural restoration of architectural heritage. International Scientific Committee for Analysis and Restoration of Structures of Architectural Heritage.

Schueremans, L. (2003). Evaluation of bearing capacity and design of consolidation and strengthening by means of injections. In WTATagung, Leuven.