Evaluation of residual flexural capacity of existing pre-cast pre-stressed concrete panels—A case study

Mihkel Kiviste *, Jaan Miljan
Estonian University of Life Sciences, Institute of Forestry and Rural Engineering, Department of Rural Building, Tartu, 51014, Estonia

A R T I C L E   I N F O

Article history:
Received 26 March 2010
Received in revised form
22 June 2010
Accepted 1 July 2010
Available online 7 August 2010

Keywords:
Flexural capacity
Corrosion
Pre-stressed panels/panel
Chloride
Carbonation

A B S T R A C T

The residual flexural capacity of 28 existing pre-cast, pre-stressed, ribbed concrete ribbed panels was studied. Four-point bending was applied into the middle third of the panel. Fifteen panels of the 28 conformed to the strength and rigidity requirements of new factory-issued panels. In the study, all of the studied existing panels have retained sufficient residual flexural capacity to carry the design loads applied in service. Probably, it is likely that pre-stressing steel of an inferior strength mark was applied onto five of the panels studied. This, which could explain the poor flexural capacity of those panels of in good visual condition and the insignificant corrosion level on the pre-stressing bars. Twenty-two out of 28 panels exhibited flexural ductile failure. Three panels failed as a result of an anchorage (weld) rupture and three panels failed in shear. It is likely that the corrosion deterioration of the panels were probably neither carbonation nor chloride induced.

© 2010 Elsevier Ltd. All rights reserved.

1. Introduction

Reinforcement corrosion is the most widespread cause of premature deterioration in reinforced concrete structures. Corrosion is a concern because of the associated cracking and spalling of concrete cover, reduction in steel cross-sectional area and loss of bond, which over time will decrease the strength and serviceability of concrete structures. Considerable resources are spent on repair and to rehabilitate deteriorating concrete structures. The degree of reinforcement corrosion, and the resulting decrease in the load-carrying capacity of the structural component, needs to be evaluated to develop effective repair strategies.

Most of the studies reporting structural tests on corroded reinforced concrete structures have been conducted under laboratory conditions. However, laboratory studies cannot fully represent all the aspects of the on-site behaviour of concrete structures. Although field studies can help us account for the reduction in strength and serviceability of concrete structures over time, investigations on the strength and condition of the existing reinforced and pre-stressed concrete structures, after long-term exposure to the on-site environment, are scarce in the literature.

Flexural capacity tests have been performed with beams in harbour docks [1] which had been in an aggressive environment for more than 10 years [2]. Wipf et al. [3] conducted an extensive field testing of pre-cast channel bridges as well as laboratory testing of their pre-cast panels. An experimental study was conducted on two 28-year-old reinforced concrete bridge arch ribs to explore the residual load carrying capacity and failure pattern [4]. In the study by Shhid et al. [5], 40-year-old pre-cast pre-stressed concrete bridge piles were rated visually and flexurally tested to failure. Heymsfield et al. [6] examined the flexural load capacity of 33 pre-cast channel beams, which were removed from existing bridges. These were 5.79 m long and 1.07 m wide channel beams and were used in a large number of bridges in Arkansas (USA) from the mid-1950s to the mid-1970s. The length and the loading arrangement of the beams in the study of Heymsfield et al. are similar to these employed in the present study.

In Estonia, the bearing structures of many agricultural and industrial buildings constitute a pre-cast concrete skeletal frame. Intensive construction, based on industrially produced elements, started in the 1960s when standardized design solutions and reinforced concrete structure designs were employed. After a relatively short period of service, the initial signs of corrosion of steel reinforcement became evident in agricultural buildings.

In Estonia today, there are about 4000 agricultural buildings with an average floor space of 1800 m². Many of their pre-cast concrete bearing structures (columns, beams and ribbed panels) have reached an undefined state. The pre-stressed ribbed panels examined in this study are very common in Estonian agricultural buildings. There is an increasing demand for informed decisions about the structures’ capability to serve their intended function or, otherwise, the need for repair or demolition.

* Corresponding author.
E-mail address: mihkel.kiviste@emu.ee (M. Kiviste).

0141-0296/ $ – see front matter © 2010 Elsevier Ltd. All rights reserved.
2. Experimental procedure

2.1. Test panels

The flexural capacity of 28 existing pre-cast, pre-stressed, ribbed concrete panels of mark PNS-12 and PNS-14 were studied experimentally. These panels, 1490 by 5970 mm (Fig. 1), were mass-produced in concrete factories of the former Soviet Union from the mid-1960-ies until at least 1990. The abbreviation PNS denotes that the panels are pre-stressed and the numbers refer to panels of different capacities [7].

In general, the diameter of the pre-stressing bar was used to distinguish between the different marks of panels although on a few panels the factory-painted mark was still readable. Pre-stressed concrete panels PNS-10...14 employed two hot-rolled low-alloyed pre-stressing bars of mark 35 GS (C = 0.3 ... 0.37%, Mn = 0.8% ... 1.2%, Si = 0.6% ... 0.9%, Cr = 0.3%, Ni = 0.3, Cu = 0.3) [8]. The diameter (and pre-stress) of pre-stressing bars of panels PNS-12 and PNS-14 were 16 mm (343 N/mm²) and 20 mm (481 N/mm²), respectively [7]. The ultimate strength of pre-stressing steel (of both PNS-12 and PNS-14) should be at least 5500 kg/f/cm² (539 MPa) to correspond to its mark [7]. Pre-stressing bars are welded to the details at the support ends of a panel [7]. Reinforcing and anchorage of panels PNS-12 and PNS-14 are shown in Fig. 2.

The strength of concrete of panels PNS-12 and PNS-14 should correspond to concrete strength mark M200 = 200 kg f/cm² and M300 = 300 kg f/cm², respectively [7]. Hence, the concrete strength of panels PNS-12 and PNS-14 should be at least 19.6 MPa and 29.4 MPa [7], respectively.

This study is a follow-up for the series of tests on the ribbed panels of type PKZH-2 and PNS-3 performed by J. Miljan since 1973 [9], The panels, transported to the structural laboratory of the Eesti Maailikool (EMU), the Estonian University of Life Sciences, were demounted from the following four different existing and abandoned structures.

1. Ceiling panels from cowshed No. 2, constructed in 1975 according to the design of type TP 891–254 at Luha farm i in the Vara district of Tartu county. Flexural tests on six panels PNS-12 and four panels PNS-14 (panels L1 ... L14) from this cowshed were performed by Laiakask in 2000–2001 [10].

2. Roof-ceiling panels from the truck garage of the former Raadi military airfield the construction time of which is unknown due to their military origin. Flexural tests with eight panels PNS-12 (panels R1 ... R8) from this garage were performed by Kiviste et al. in 2002 [11].

3. Ceiling panels from Vara pigsty, constructed in 1973 and abandoned in 1993, in the Vara district of Tartu county. Flexural tests with six panels PNS-14 (panels V1 ... V6) from the pigsty were performed by Patrael et al. in 2005. Test results of panel V3 are omitted.

4. Roof-ceiling panels from the corridor of Vara pigsty in the Vara district of Tartu county. Flexural tests with four panels PNS-12 and one panel PNS-14 (panels V8...V12) were performed by Halgma et al. in 2005–2006.

2.2. Requirements for a panel

According to former Soviet standard GOST 8829 [12], a randomly chosen pre-cast concrete element from each production batch from the factory must be tested to assess its conformity to strength, rigidity and crack resistance requirements. In this study only the requirements for strength and rigidity were considered. Due to corrosion deteriorations (cracked or spalled cover) most of the studied panels could not meet crack resistance requirements prior to loading.

The control load [12] ($q_c$) was set to check the strength requirements of a panel. A panel meets the strength requirements of its mark if the ultimate load of the tested panel exceeds the control load. Repetition tests were made if the ultimate load of a panel was less than the control load but not less than 85% of the control load. A panel does not meet the strength requirements if a single ultimate load in primary or repetition tests is less than 85% of the control load [7]. The control load ($q_c$) for panels PNS-12 and PNS-14 was set to 7.35 kN/m² and 14.12 kN/m², respectively [7].

The control load ($q_c$) was calculated with the following formula [7]:

$$q_c \geq \frac{s}{t} \cdot (q_d + q_{dead}) - q_{dead},$$

where

- $s$ is the coefficient of overload, 1.4;
- $t$ is the coefficient of working conditions, 1.0;
- $q_d$ is the design load, kN/m²; and
- $q_{dead}$ is the dead load of panel, kN/m².

The value of design load ($q_d$) was derived from catalogues and design drawings [7] of pre-cast concrete elements. The design load was implemented by the structural engineering design of a building.

A panel, including initial deflection, should be rigid enough not to exceed a deflection of 20 mm under a load of 3.73 kN/m² and 7.75 kN/m² for panels PNS-12 and PNS-14, respectively [7].

2.3. Test setup and loading

For the flexural test, a four-point bending test arrangement was set up whereby two loads, acting as an equivalent for the uniformly distributed load, were applied to the middle third of the panel (Fig. 3). The loads were applied by means of a hydraulic cylinder of a nominal maximum capacity of 250 kN. A steel main beam divided the total applied load from the cylinder into two loads. The distributing beams further divided it across the width of the panel, resulting in a total of four concentrated loads on the panel. The main and distributing beams acted as a simple beam with a pin and roller support. The application of load was controlled by a manually activated hydraulic pump. Panels were simply supported with a pin and roller support [12].

Before loading, a stressed wire was held on the supports to measure the initial deflection of each longitudinal rib in the mid-span of the panel, using a ruler with an accuracy of 1 mm. A mean initial deflection of both ribs was calculated as an initial mid-span deflection of a panel [12]. As a general rule, a negative initial deflection of a ribbed panel had been provided by pre-stressing it in the factory.

Panels were loaded in increments of 10% of the control load ($q_c$) which was kept constant for at least 10 min at each stage [12]. Panels were tested to failure or limit state whereby deflections of
a panel increased without adding further load [12]. The maximum load a panel could carry was recorded as the ultimate load \( q_u \). Then, according to the loading test arrangement, the applied force \( (kN) \) from the hydraulic cylinder was converted to bending moment \( (kNm) \) and the latter to the uniformly distributed surface load \( (kN/m^2) \).

Existing cracks and cracks developing during the test were carefully recorded with a marker on the panel surface. Vertical displacements were measured at the four corners (on supports) and on both longitudinal ribs at mid-span of each panel. Dial gauges with a precision of 0.01 mm were applied at the corners and compliant measuring gauges with a precision of 0.1 mm and 0.01 mm at mid-span of a rib. The mid-span deflection of a panel was calculated as the difference of the mean mid-span deflection of both longitudinal ribs and of the mean displacement at the supports of the panel [12].

2.4. Cores, carbonation, cover and chloride content of concrete

The concrete core test was based on Estonian National Standard EVS-EN 12504–1:2003 [13]. Ten cores with a diameter of 54 mm were drilled from the longitudinal rib of each panel. Cores were cut by means of a rotary cutting drill with diamond bits. The ends of the cores were ground or capped with rapid hardening cement. Each core was measured in accordance with EVS-EN 12504–1:2003 [13]. The mean cross-sectional area was calculated from five diameter measurements and the mean height of each core was calculated from five height measurements. The estimated cube strength \( f_{\text{est, cube}} \) was calculated by applying the following Eq. (2) in BS 6089:1981 [14]:

\[
    f_{\text{est, cube}} = \frac{D}{1.5 + 1/\lambda} \cdot f_{\text{core}},
\]

where \( D = 2.5 \) for cores drilled horizontally (perpendicular to the cast direction for pre-cast units), or \( D = 2.3 \) for cores drilled vertically (parallel to the cast direction for pre-cast units), \( \lambda \) is the height/diameter ratio, and \( f_{\text{core}} \) is found by dividing the maximum load sustained by the core with its average cross-sectional area.

Cores with a height/diameter ratio 1 were tested, because cylinders with this ratio have very nearly the same strength as standard cubes [13]. The estimated cube strength was compared to the concrete mark from design drawings [7] to verify if the panels were cast in accordance with the drawings. The concrete mark (operative until 1984) was calculated as a mean compressive strength of standard cube specimens of sides 150 mm in kgf/cm\(^2\) (e.g. concrete of mark M200).

Carbonation depth was measured by the conventional phenolphthalein test according to EVS-EN 14630:2006 [15]. Testing was undertaken by applying a phenolphthalein solution to a freshly opened surface of non-cracked concrete cover of pre-stressing bars after the flexural test. Carbonation depth was measured by means of a ruler at 10 different locations (at least five readings for each location) on the longitudinal rib of a panel. A mean value of carbonation depth \( (D_{\text{carb}}, m) \) of a panel was calculated.

A cover of pre-stressing bars was also measured by means of a ruler at the same location where the carbonation test was performed (at least five readings at each location). A mean value of cover \( (c_m) \) of pre-stressing bars of a panel was calculated.

Chloride content was determined on 20 samples taken from the concrete cover of pre-stressing bars of nearby panels of R1...R8 at the research object Raadi. Chloride content was determined by applying Quantab chloride titrator stripes.
2.5. Yield, ultimate strength and corrosion of steel

Six pre-stressing steel specimens of visually larger corrosion damage were cut from each panel. Specimens were cut to a length of 450 mm, cleaned and weighed. A percentage gravimetric mass loss of each pre-stressing steel sample was calculated. A mean gravimetric mass loss ($\Delta m_{gr,m}$) of six samples was found to show the mean corrosion penetration of pre-stressing bars of a panel. After mass loss determination, each pre-stressing bar was further inspected for evidence of pitting. However, no pits were found on the pre-stressing bars. At least 10 calliper gauge measurements were performed and a minimal diameter of each pre-stressing bar was recorded. A maximum diameter loss ($\Delta d_{max}$) out of six bars was calculated to represent the maximum corrosion penetration of pre-stressing bars of a panel.

All pre-stressing steel specimens were subjected to tensile testing. A universal testing machine R-20 (maximum capacity 200 kN) with the software for test data recording was applied. A mean yield ($f_{y,m}$) and ultimate strength ($f_{u,m}$) of six pre-stressing bars was calculated from test data.

3. Experimental results and discussion

3.1. Panels PNS-12

Results of flexural and material tests of 18 pre-stressed concrete ribbed panels PNS-12 are presented in Table 1. The control load ($q_c$) and the design load ($q_d$) for panels PNS-12 were set to 7.35 kN/m² and 4.51 kN/m², respectively [7].

Except for panel R2, the ultimate load ($q_u$) of all the panels PNS-12 were higher than control load and even for the R2 the capacity constituted 0.98 of the control load. The ultimate load of the studied panels PNS-12 constituted at least 1.60 of the design load. Table 1 shows that panel R2, with the largest initial mid-span deflection ($\Delta_d$) of 53 mm, also had the lowest flexural capacity. Thus, visually detectable initial mid-span deflections of a panel could be a sign of decreasing flexural capacity. The mean yield strength of pre-stressing steel specimens of panel R2 was at least 53 MPa lower than that of other panels PNS-12. This could also have reduced the flexural capacity of R2, because most of the panels (including R2) were tested until deflections increased without further load, meaning that the pre-stressing steel of a panel had begun to yield.

A pre-stressing steel specimen no 3, cut from the mid-span of panel R1, was significantly more corroded ($m_{red} = 19.6\%$, $d_{red,max} = 10.3\%$) and weaker ($f_y = 458$ MPa, $f_u = 531$ MPa) than any other specimen cut from panels PNS-12 (Table 1). The ultimate strength of pre-stressing steel of PNS-12 should be at least 53 MPa to correspond to its strength mark 35GS [7]. Except for specimen no. 3 from R1, the ultimate strength of all pre-stressing steel specimens from panels PNS-12 considerably exceeded their corresponding strength mark. However, the characteristics of that sample have not significantly reduced the flexural capacity ($q_u/q_c = 1.14$) of panel R1.

Sixteen panels PNS-12 had a flexural ductile failure, panel R7 had an anchorage failure and V8 failed in shear. The pre-stressing bars of PNS-12 are welded to the details at the supports of a panel (Section A–A in Fig. 2(a)) [7]. During the failure of panel R7, the weld ruptured and the longitudinal pre-stressing bar slipped inwards at the support (Fig. 4). Panel R7 also showed a relatively low flexural capacity in comparison with the other panels. The ultimate load of R7 was registered shortly before weld rupture.

The critical shear crack of V8 started from the point of load application and ended at the lower end of the last stirrup (Fig. 5). No stirrups were in the critical shear crack, because the last stirrups ended at 1800 mm, and the point of load application was 1973 mm from the support (Fig. 2(a) and Fig. 5). Therefore, the shear resistance of V8 in the critical section depended only on concrete strength. Table 1 shows that the concrete strength of panel V8 ($f_{est.,cube,m} = 18.4$ MPa) was considerably lower than that of other panels of PNS-12. As mentioned earlier, the concrete strength of panels PNS-12 should be at least 19.6 MPa to be in accord with their strength mark [7]. Except for panel V8, the concrete strength of all panels PNS-12 substantially exceeded its strength mark (Table 1). Low concrete strength was probably the main reason for the shear failure of panel V8. This also explains why the cover of only V8 was fully carbonated ($D_{carb,m} = 24.7$ mm > $c_m = 24$ mm). About one-third of the cover was carbonated on the other panels PNS-12.

The load–deflection curves of panels PNS-12, excluding and including initial deflection, are presented in Figs. 6 and 7, respectively. Curves with dashed lines indicate panels of spalled cover in the longitudinal rib. The authors managed to acquire several test documents of new panels (tested in the 1970s) from concrete factories in Estonia. The load–deflection curves of eight new panels, which conformed to the requirements [12] for PNS-12, are plotted with thick solid lines (Fig. 7). It should be noted that new panels were tested only up to the control load, i.e. until the strength requirement was met.

Fig. 7 shows that panels R2, R8 and V9 failed to meet the rigidity requirements, caused by their initial deflection of 18 mm or more. These panels also had a spalled cover. However, other panels with a spalled cover (e.g. panels V8, R1 and R6 in Figs. 6 and 7) met both strength and rigidity requirements. Fig. 7 also shows that the flexural behaviour of the analysed existing panels (which have been in service for at least 25 years) was not substantially different from the new panels PNS-12.

3.2. Panels PNS-14

Results of flexural and material tests of 10 pre-stressed concrete ribbed panels PNS-14 are presented in Table 2. The control load and
the design load of panels PNS-14 was set to 14.12 kN/m² and 9.32 kN/m², respectively [7].

The ultimate load of five panels PNS-14 was less than the control load. The ultimate load of L7, V5 and V6 consisted of more than 0.85 of the control load, which would mean repetition tests for new panels issued from the factory. The strength of L8 and V4 was less than 0.85 of the control load. Hence, these panels failed to meet the strength requirements for new panels.

Nevertheless, the lowest ultimate load of panel L8 constituted 109% of the design load. That means that all analysed panels (both PNS-12 and PNS-14) have retained sufficient residual flexural capacity for design loads applied in service.
Both yield and ultimate strength of pre-stressing steel specimens from panels V1 ... V6 (test series 3) were low in comparison with other samples from panels PNS-14 (Table 2). The ultimate strength of pre-stressing steel 35GS (applied in PNS-14) should be at least 539 MPa [7]. The mean ultimate strength of pre-stressing bars from panels of test series 3 was on the verge (panel V1) or lower (panels V2, V4, V5, V6) than steel mark 35GS. The mean yield strength of pre-stressing bars from panels V1 ... V6 was 442 MPa or less. Therefore, these pre-stressing bars would have started to yield under the pre-stress of 481 N/mm² for panels PNS-14 [7]. The visual condition of these panels was good and the corrosion level on the pre-stressing bars was insignificant ($d_{\text{red, max}} \leq 2.6\%$). Hence, probably the pre-stressing steel of an inferior strength mark was applied to the panels V1 ... V6. This also explains the poor residual flexural capacity of those panels.

Seven panels PNS-14 had a flexural ductile failure. Panels L7 and L8 failed as a result of weld rupture at the support ends. As reported earlier, the ultimate load of both panels was less than the control load. The pre-stressing bar of panel PNS-14 was welded to the washer, which itself was welded to the detail at the support (Section B–B in Fig. 2(b) [7]). Therefore, the condition of anchorage (weld) should be checked at the support ends, because it could reduce the flexural capacity of a panel. The concrete strength mark of panels PNS-14 should be at least 29.4 MPa [7]. Except for panel L8 the concrete strength of all panels PNS-14 corresponded to (or exceeded) its strength mark (Table 2). Even the concrete strength of L8 ($f_{\text{concrete, m}} = 27.1$ MPa) was not considerably lower than the strength mark for panels PNS-14. The mean gravimetric mass loss ($\Delta m_{\text{gr, m}} \leq 7.2\%$) and the maximum diameter loss ($\Delta d_{\text{max}} \leq 3.8\%$) of the pre-stressing bar sample cut from panels PNS-14 was not significant.

Both panels L10 and V12 failed in shear with a large inclined crack at the longitudinal rib starting from the point of load application (panel V12 in Figs. 8a and 8b). This type of failure can be accounted for by the loading arrangement involved. Former Soviet standard GOST-8829 [12] recommends a loading scheme of a concrete structure with a uniformly distributed load (Fig. 9(a)) and its equivalent load (Fig. 9(b)). An equivalent load with four equally concentrated loads can replace the uniformly distributed load if a total uniformly distributed control load exceeds 35 kN [12]. The bending moment (Fig. 9(b1)) and shear (Fig. 9(b2)) of equivalent load [12] (solid line) are compared to those of the current study (dashed line). Equivalent loading arrangement would result in half the shear stresses (P/4 in Fig. 9(b2)) as those (P/2 in Figs. 9(b2) and 3(e)) developed with the chosen loading arrangement at the section of load application.

The load–deflection curves of panels PNS-14, excluding and including initial deflection, are presented in Figs. 10 and 11, respectively. Fig. 11 shows that panels L8 and L10 just failed to meet the rigidity requirements set for the new panels of PNS-14 [7].

3.3. Carbonation and chlorides

Except for panel V8, about one-third of the concrete cover of all studied panels was carbonated (Tables 1 and 2). The cover of the pre-stressing bar of V8 was fully carbonated because of its low concrete strength when compared to the other panels. Chloride content was determined on 20 samples taken from the nearby panels of R1 ... R8 at the research object Raadi. The mean chloride content was 0.20% with a minimum and maximum value of 0.15% and 0.22% by mass of cement, respectively. Chloride-induced corrosion can only take place once the chloride content in concrete in contact with the steel surface has reached a threshold value. The morphology of chloride attack is that typical of pitting
or localised corrosion. However, no signs of pitting corrosion were found visually on any of the pre-stressing bars of panels studied. Tables 1 and 2 also show no serious local cross-section reduction ($d_{red,max} \leq 10.3\%$) of the studied pre-stressing bar samples. These results show that corrosion deteriorations in the studied panels were probably neither carbonation nor chloride induced.

4. Conclusions

This study presents the results of flexural and material tests of 28 existing pre-cast pre-stressed concrete ribbed panels, which have been in service for at least 25 years. Based on test results, the following conclusions were drawn.

1. Fifteen panels of the 28 conformed to the strength and rigidity requirements of new factory-issued panels. The flexural behaviour of the analysed existing panels was not significantly different from new panels.

2. The ultimate load of all studied panels constituted at least 105\% of the design load. That means that all the analysed existing panels have retained sufficient residual flexural capacity to carry design loads applied in service.

3. It is likely that pre-stressing steel of an inferior strength mark was applied to panels V1 ... V6 (PNS-14). Only this could explain the poor flexural capacity of those panels of good visual condition and insignificant corrosion level on pre-stressing bars.

4. Twenty-two of the 28 panels exhibited flexural ductile failure. Three panels failed as a result of an anchorage (weld) rupture and three panels failed in shear.

5. It is likely that the corrosion deterioration of the panels was neither carbonation nor chloride induced.

References


