INFLUENCE OF VISUAL CONDITION ON RESIDUAL FLEXURAL CAPACITY OF EXISTING PRECAST CONCRETE PANELS

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Abstract. The influence of visual assessment grade on the residual flexural capacity of 46 existing precast concrete ribbed panels from different agricultural buildings was studied. Before the tests the panels were assessed on a 6-point rating scale according to visually distinguishable corrosion deterioration. All panels, the ultimate load of which was lower than the control load, received grade 0 on the visual rating scale. Consequently, attention should be paid to panels where the concrete cover of longitudinal reinforcement has spalled (grade 0) which could be a sign of decreasing load capacity. The majority of panels with grade 0 exhibited larger deflections under load than panels with higher grades. Of the 46 panels tested flexural ductile failure was noticed at 36 panels.

Key words: visual grade, residual flexural capacity, corrosion, precast ribbed panels.

Introduction

In Estonia, the bearing structures of many existing agricultural and industrial buildings constitute a precast concrete skeletal frame. Particularly intensive construction based on industrially produced (precast) elements started in the 1960s when standardized design solutions and reinforced concrete structure designs were employed. However, the initial signs of corrosion of steel reinforcement became evident in agricultural buildings already in 1970s. The Department of Rural Building of the Estonian University of Life Sciences (EMU) has gathered data describing the state of concrete load-bearing structures (columns, beams and ribbed panels) in 258 agricultural buildings from 1974 to 1997 assigning grades for 23 336 ribbed ceiling panels (i.e. about 3.5 % of the total number of panels in agricultural buildings of Estonia) [1].

There are about 4 000 agricultural buildings with an average floor space of 1 800 m^2 in Estonia today. Many of their precast concrete load-bearing members (columns, beams and ribbed panels) are in service with a cracked or spalled concrete cover. Owners of buildings are most likely concerned about the condition and residual strength of their concrete structures. There is an increasing demand for informed decisions about the capability of structure to serve its intended function or, otherwise, the need for repair or demolition.

This paper reports an experimental study of 14 precast non-prestressed concrete ribbed panels of mark PKZH-2 and 32 prestressed concrete panels of mark PNS-3, PNS-12 and PNS-14. The first objective of the research is to find the residual flexural strength of the existing precast concrete ribbed panels. The second objective is to clarify whether it is possible to estimate the load capacity of a ribbed panel according to visually discernible corrosion damage. The marks of panels reflect the former Soviet Union standard GOST. Precast ribbed panels with aforementioned marks are common in the industrial and agricultural buildings of Estonia (but also in other former Soviet Union countries) built from 1950s to 1990s. All tested ribbed ceiling and roof-ceiling panels had a length of 5 970 mm and width of 1 490 mm (Figure 1).

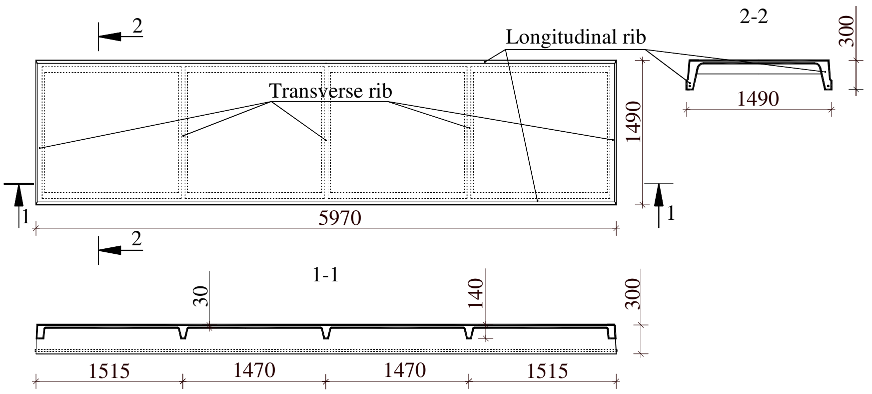

Fig. 1. **Top view, longitudinal and transversal section of a precast ribbed panel**

Non-prestressed concrete panels of mark PKZH were manufactured (in accordance with GOST 7740-55 [2]) from the 1950s until 1964…1965. Prestressed concrete panels of mark PNS were produced from 1964…1965 until at least 1990. Panels PNS-3 were produced in the relatively short period of transition from panel mark PKZH to PNS. Since the mid-1960-ies panels PNS-12 have been produced (a further development of PNS-3) and PNS-14 started [3].

Materials and methods

Before the structural tests, the panels were assessed on a scale developed at the Chair of Structural Mechanics of the former Estonian Agricultural Academy (EMU now) in 1974. The visual assessment scale distinguishes between six different states as shown in Table 1.

Table 1

Classification of deterioration states of the ribbed ceiling panels

Grade	Description of state
5	No corrosion detected
4	1) Less than 20 % of the concrete cover of a slab has spalled; 2) Noticeable longitudinal cracks (0.3-1.0 mm) in transverse ribs.
3	1) More than 20 % of the concrete cover of slab reinforcement has spalled; 2) Less than 20 % of the concrete cover of stirrups in the longitudinal ribs has spalled; 3) In transverse ribs wide (>1.0 mm) cracks have occurred; 4) Less than 20 % of the concrete cover in transverse ribs has spalled.
2	1) More than 20 % of the concrete cover of stirrups in longitudinal ribs has spalled; 2) More than 20 % of the concrete cover of reinforcement in transverse ribs has spalled; 3) Longitudinal micro cracks (0-0.3 mm) due to corrosion in longitudinal ribs.
1	Longitudinal cracks (> 0.3 mm) in longitudinal ribs;
0	Concrete cover of the reinforcement in longitudinal ribs has spalled.

The current study is based on the series of tests of ribbed panels at EMU since 1973. 14 reinforced concrete (RC) panels of mark PKZH-2 and 32 pre-stressed concrete (PC) panels of mark PNS-3, PNS-12 and PNS-14 were tested. The summary of the test series is presented in Table 2. Letter(s) in the first column is associated to the location of panels. RC panels are marked with hyphen between the letter and number, while PC panels are marked without hyphen.

Table 2

Test series of reinforced and prestressed concrete ribbed panels

Panels (amount)	Mark	Object and purpose	Test location	Loading, location	Test year	Age of panels	Test performer
K-1 ... K-7 (7)	PKZH-2	Kärstna pigsty	Kärstna field tests	Sand uniformly, soil	1973	12	J. Miljan
K-8 ... K-10 (3)	PKZH-2	Kärstna pigsty	Tallinn, test hall	Cast iron loads uniformly, RC floor	1974	13	J. Miljan
P11 ... P13 (3)	PNS-3	Pandivere pigsty	Tallinn, test hall	Cast iron loads uniformly, RC floor	1974	10	J. Miljan
VA14 … VA19 (6)	PNS-3	Vao pigsty	Vao field tests	RC foundation blocks uniformly, soil	1975	11	J. Miljan
T-20 … T-23 (4)	PKZH-2	Torma cowshed	Torma field tests	RC curbstones uniformly, soil	1978	15	J. Miljan
L1 … L10 (10)	PNS-12 PNS-14	Luha cowshed	Tartu, EMU lab.	Hydrocylinder, 4-point bending, RC force floor	2000-2001	26	E. Laiakask
R1 … R8 (8)	PNS-12	Raadi garage	Tartu, EMU lab.	Hydrocylinder, 4-point bending, RC force floor	2002	Un-known	M. Kiviste, H. Tomann, M. Tarto
V8 … V12 (5)	PNS-12 PNS-14	Corridor of Vara pigsty	Tartu, EMÜ lab.	Hydrocylinder, 4-point bending, RC force floor	2005	32	R. Halgma, L. Linnus, T. Salu

As shown in Table 2, all tested panels had been in service for at least 10 years. The panels were demounted and singly loaded. The panels were tested in laboratory (K-8…K-10, P11…P13, L1…L10, R1…R8, V8…V12) as well as on the object (K-1…K-7, VA14…VA19, T-20…T-23). The structural tests with pre-stressed ribbed panels of mark PNS-12 and PNS-14 are discussed in more detail in another paper.

The panels were lifted to RC blocks, which acted as sub supports. Singly tested panels were simply supported on a steel pin and roller support. All tested panels were loaded in increments of 10 % of the control load (q_c) which was kept constant for at least ten minutes on each stage [4].

The control load was set to test new panels issued from factory. A few randomly chosen new panels were tested in the factory to check their crack resistance, rigidity and load capacity up to one increment higher than the control load. Repetition tests were due if the ultimate load of a panel issued from the factory was less than the control load but not less than 85 % of the control load. Panels did not meet the strength requirements if a single ultimate load in primary or repetition tests was less then 85 % of the control load [3]. The design load (q_d) was implemented by the structural engineering design of a building.

In all test series, uniformly distributed loads were imitated to compare the results with the control and design load. The panels were tested to failure or limit state whereby deflections of a panel increased without additional load [4]. The maximum load a panel could carry was recorded as the ultimate load (q_u). 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 a panel. Generally, dial gauges of precision 0.01 mm were applied at the corners and compliant measuring gauges (type Maksimov) of precision 0.1 mm and 0.01 mm at mid-span of a rib. The mid-span deflection of a panel was calculated as a difference of the mean mid-span deflection of both longitudinal ribs and of the mean displacement at supports of the panel [4].

Results and discussion

To compare the residual strength of panels of 4 different marks, the ratio (q_u/q_c) of ultimate load and control load was calculated. The *one-way analysis of variance* (*ANOVA*) did not reveal significant difference in the average ratio of the ultimate load and control load by the panel marks (PKZH-2, PNS-3, PNS-12, PNS-14) at the confidence level α=0.05. Also, in purpose of comparison the ultimate load (q_u) of the test panels was divided to design load (q_d).

The control loads of panels PKZH-2, PNS-3 (later PNS-12) and PNS-14 are 387 [2], 750 [3] and 1440 kgf·m^{-2} [3], transformed to kN·m^{-2}, respectively. The design loads of panels PKZH-2, PNS-3 (later PNS-12) and PNS-14 are 270 [2], 460 [3] and 950 kgf·m^{-2} [3], transformed to kN·m^{-2}, respectively. The results of visual assessment and flexural test of ribbed panels are presented in Table 3. The influence of visual condition (grade) on the load capacity (q_u/q_c) of 46 singly tested panels is presented in Figure 2. The box plot in Figure 2 was generated with statistical software R.

Figure 2 shows non-linear decreasing trend of q_u/q_c ratio with decreasing grade of panel. Only a few samples of high grades exist in the current data set. Neither statistical nor substantial reasons exist to assume a trend in q_u/q_c ratio at grade 2 or higher. However, box plots from grade 2 to 0 demonstrate evident decrease of q_u/q_c ratio. The one-way *ANOVA* revealed a significant effect for grades, $F(5,40)=5.35$; $p=0.0007$. The magnitude of the grade to q_u/q_c ratio was computed as $R^2=0.40$. *Tukey's HSD test for multiple comparisons of means* proved the significant difference of q_u/q_c ratio between grade 0 and higher grades.

The ultimate load of only five of the 46 singly tested panels was less than the control load. All of these five panels received grade 0 on the visual rating scale. Consequently, attention should be paid to panels where the concrete cover of longitudinal reinforcement has spalled (grade 0) which could be a sign of decreased load capacity. The visual scale proposed in the paper has the potential to serve as a rational tool for practitioners, operators and asset managers to make decisions about the optimal timing for repairs, strengthening, and/or rehabilitation of corrosion-affected concrete infrastructure. Scale-acquainted engineers can rate reinforced concrete structures relatively quickly and simply to fetch out ribbed ceiling panels (if any) of spalled concrete cover. Later on the residual flexural capacity of panels with grade 0 needs structural expert judgment. It is also worth mentioning that no panels with a corrosion-induced crack in the longitudinal rib (grade 1) were dangerous from the aspect of ultimate residual load capacity. All studied panels (irrespective of their grade) were able to carry the design load (q_u/q_d>1.01).

Table 3

Results of visual assessment and flexural test of ribbed panels

Panel	Mark	Grade	q_u, kN·m^{-2}	q_u/q_c	q_u/q_d	Panel	Mark	Grade	q_u, kN·m^{-2}	q_u/q_c	q_u/q_d
K-1	PKZH-2	0	4.52	1.19	1.71	L1	PNS-12	0	9.00	1.23	2.00
K-2	PKZH-2	1	5.18	1.36	1.96	L2	PNS-12	4	9.20	1.25	2.04
K-3	PKZH-2	0	3.97	1.05	1.50	L3	PNS-12	1	9.25	1.26	2.05
K-4	PKZH-2	1	4.79	1.26	1.81	L4	PNS-12	3	9.70	1.30	2.15
K-5	PKZH-2	1	5.16	1.36	1.95	L5	PNS-12	1	9.75	1.33	2.16
K-6	PKZH-2	0	4.31	1.14	1.63	L9	PNS-12	3	9.04	1.23	2.00
K-7	PKZH-2	1	4.54	1.20	1.71	L6	PNS-14	5	16.95	1.20	1.82
K-8	PKZH-2	0	4.10	1.08	1.55	L7	PNS-14	0	13.56	0.96	1.46
K-9	PKZH-2	0	2.67	0.70	1.01	L8	PNS-14	0	10.17	0.72	1.09
K-10	PKZH-2	0	2.67	0.70	1.01	L10	PNS-14	0	15.82	1.12	1.70
P11	PNS-3	2	11.01	1.50	2.44	R1	PNS-12	0	8.35	1.14	1.85
P12	PNS-3	1	8.07	1.10	1.79	R2	PNS-12	0	7.26	0.99	1.61
P13	PNS-3	2	9.56	1.30	2.12	R3	PNS-12	1	9.12	1.24	2.02
VA14	PNS-3	1	8.79	1.19	1.95	R4	PNS-12	1	9.64	1.31	2.14
VA15	PNS-3	1	8.79	1.19	1.95	R5	PNS-12	1	9.86	1.34	2.19
VA16	PNS-3	2	9.90	1.35	2.20	R6	PNS-12	0	8.59	1.17	1.90
VA17	PNS-3	2	9.90	1.35	2.20	R7	PNS-12	1	7.91	1.08	1.75
VA18	PNS-3	2	9.90	1.35	2.20	R8	PNS-12	0	8.28	1.13	1.84
VA19	PNS-3	2	9.90	1.35	2.20	V8	PNS-12	0	9.00	1.22	2.00
T-20	PKZ-2	1	5.64	1.49	2.13	V9	PNS-12	0	10.53	1.43	2.33
T-21	PKZ-2	2	5.94	1.57	2.24	V10	PNS-12	1	8.80	1.20	1.95
T-22	PKZ-2	1	5.64	1.49	2.13	V11	PNS-12	2	9.30	1.26	2.06
T-23	PKZ-2	1	5.43	1.43	2.05	V12	PNS-14	1	15.28	1.08	1.64

Figure 2 demonstrates that q_u/q_c ratio varies the most in panels with spalled concrete cover (grade 0). This means that panels, which may have just reached grade 0 as well as panels in critical state in terms of their load capacity are both rated as grade 0. Consequently, panels with spalled concrete cover should be differentiated to specify their different residual load capacity. Deterioration states employed for panel classification in the current study (in Table 1) were developed already in 1974 and could be updated. Durham et al. [5]; Heymsfield et al. [6] had tested 33 existing precast non-prestressed channel ribbed panels, which were used in short multi-span bridges in Arkansas in the 1950s through the early 1970s. The panels, constructed without shear reinforcement, were categorized as “good”, “average” or “poor” as a function of percentage and location of exposed longitudinal reinforcing steel. All these three classifications correspond to grade 0 on the visual rating scale of the current study.

The original objective of the study of Heymsfield et al. [6] was to establish a correlation for inspection purposes between the beam’s visual deteriorated state and its corresponding approximate structural capacity. 5.79 m channel ribbed panels with similar cross section were tested also on a four-point loading frame. It was found that the strength of beams was more a function of a concrete compressive strength rather than deterioration state.

Torres-Acosta et al. [7] had proposed a durability model based on experimental load capacity values from various investigations, where results of different structural members (beams, slabs) under accelerated corrosion were presented. Figure 3 represents an illustrative load-capacity model for a reinforced concrete flexural member referred in Torres-Acosta et al. [7] and the current study with the addition of research results by Heymsfield et al. [6] and Li et al. [8].

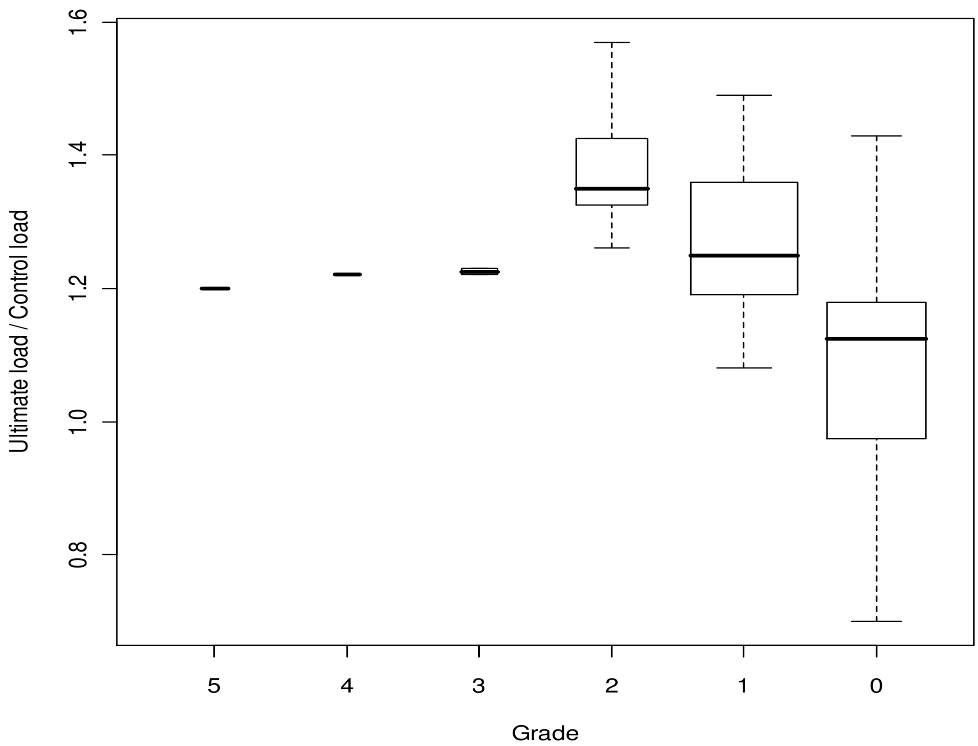

Fig. 2. **Box plot of q_u/q_c ratio for panels of different visual grades**
(The box plots show distribution characteristics: the median (thick horizontal line), upper and lower quartiles (horizontal edges of the box), minimum and maximum values (ends of vertical bar) of the q_u/q_c ratio by different grades)

The model presents the structural load capacity of a flexural member as a function of its lifetime. The lifetime T of the flexural member is defined as:

$$T = T_I + T_P + T_{RL}, \tag{1}$$

where T_I – corrosion initiation stage from the time of construction to the time of corrosion initiation;
T_P – corrosion propagation stage during which the steel corrodes until an unacceptable level of corrosion is reached and;
T_{RL} – residual life stage from serviceability to the ultimate limit state.

As corrosion progresses, there will be an increasing build-up of corrosion products and associated increased radial stresses, causing longitudinal cracking and, eventually, concrete spalling. In this study, the unacceptable level is defined as a corrosion-induced crack in the longitudinal rib of a panel more than 0.3 mm wide (grade 1). This also might be implied as serviceability limit state of a ribbed panel. Li et al. [8] stated that once the structure is considered to be unserviceable due to corrosion-induced cracking, there is considerable remaining lifetime before the structure can be considered to have become unsafe. Residual life stage T_{RL} starts from the time the structure becomes unserviceable until the ultimate limit state is reached, before structural collapse.

The categorization of "good", "average" and "poor" by Heymsfield et al. [6] is also included in Figure 3. An attempt has to be made to add the six detailed phases of the phenomenological model [8] for steel corrosion in concrete. However, the model by Li et al. [8] has a different approach. The latter differentiates six phases (D1, D2, C0, C1, C2, C3) from the mechanics of corrosion applied to the steel bar at a generic cross section of a reinforced concrete member. In addition, the initiation period of the model was based on corrosion induced by chloride attack. It was found that, for practical flexural members subject to chloride attacks, corrosion initiation may start quite early in their service life [8].

As mentioned before, all panels with visual grade 1 or higher overreached the control load, which explains the location of the control load on the time axis. Since the structural engineering designers based their calculations on the design load, the latter is employed as an equivalent of the ultimate limit state in Figure 3. The thick load capacity line in Figure 3 represents the period for the reinforced concrete member covered by current structural tests. As observed from Figure 2 and Figure 3 the structural load capacity remains almost the same during the initiation and propagation period until reaching grade 0 (in residual life period), where the capacity decrease rate is accelerated.

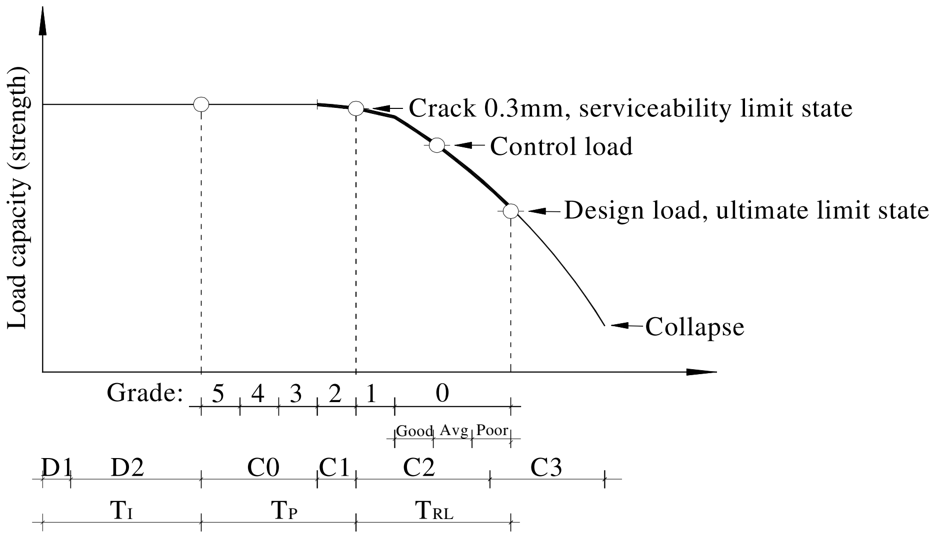

Fig. 3. **Load-bearing capacity model for a reinforced concrete member.**
Based on Torres-Acosta et al. [7] and the current study with the addition of the research results by Heymsfield et al. [6]; Li et al. [8]

Conclusions

Based on the results of the current experimental investigation of the existing precast concrete ribbed panels, the following conclusions are drawn:

1. All panels, the ultimate load of which was lower than the control load, received grade 0 on the visual rating scale. Consequently, attention should be paid to panels where the concrete cover of longitudinal reinforcement has spalled (grade 0) which could be a sign of decreasing load capacity.
2. No panels with a corrosion-induced crack in the longitudinal rib (grade 1) were dangerous from the aspect of the ultimate residual load capacity.
3. All studied panels (irrespective of their grade) were able to carry the design load. Since the structural engineering designers based their calculations on the design load, the latter is employed as an equivalent of the ultimate limit state

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