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Evaluation of retention stresses of prestressing bars of a concrete ribbed panel from agricultural building after 20 years of service

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Abstract. In Estonian agricultural buildings there exist a lot of precast concrete load-bearing structures, which were manufactured in the 1970s and 80s. By now, some of these are rather near for reaching their designed service life. 20 years old prestressed concrete ribbed ceiling panels (type PNS-12) with the dimensions of 6 m (length) by 1.5 m (width) from an existing agricultural building (pigsty) are the subject of current study. The objective of current study was to evaluate the retention stress of prestressing bars (PSBs) of a ribbed panel PNS-12. In other words the purpose was to find out how much of the factory-issued initial prestress was left at the PSBs after 20 years of service.
As a novel application in civil engineering strain gauges were applied in the evaluation of retention stress of PSBs in current study. The active strain gauges were glued to the opposite sides of PSBs at the middle-span of the ribbed panel, dummy gauges were glued to the unloaded steel slab. Strain gauges were connected to the half-bridge and measured with strain indicator and recorded. Retention stresses were calculated using the Hooke's law through the measured strains and elastic modulus of steel. Elastic modulus of PSBs was also measured using the universal testing machine Instron 3369 and software Bluehill 2, based on two standards.
The results demonstrated about 20.8% and 10.0% of retention stress of PSBs, respectively. The possible errors created by different aspects in experiment are also discussed.

Key words: retention stress, precast, prestressed, concrete, ribbed panel.

INTRODUCTION

Prestressed concrete is an advanced type of reinforced concrete which has been tensioned before the application of the loading. The compressive strength of concrete is nearly 10 times its tensile strength for a particular grade. Therefore, for structures which are expected to experience significant tensile loads concrete are initially prestressed during manufacture so that it is under compression. This pre-compression will ensure that the concrete section will remain under compression under the application of external tension producing forces. This is achieved by passing highly stressed (in tension) bars through the concrete section in factory. These tensile bars will take reaction from the position at which they are anchored at the ends of the concrete section thereby causing the concrete section to come under compression (Fig. 1) and, therefore, prevent tensile cracks, which characterize conventional reinforced concrete. The summarised compressive stress develop camber (the negative initial deflection) to the prestressed concrete structure.

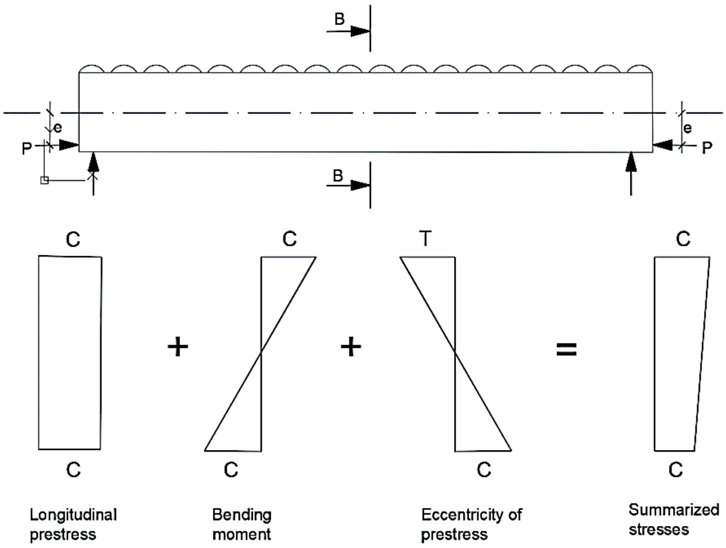

Figure 1. Principle of eccentrical prestressing. The stresses are abbreviated as follows: C – compression, T – tension.

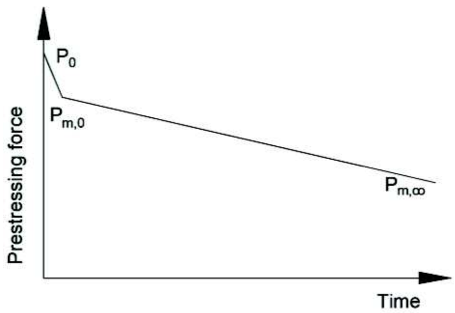

Figure 2. Principle of losses in prestressing force (P_0) in time.

Losses of prestress σ_p and the initial prestressing force P_0 (Fig. 2) occur in prestressing steel through several sources (Fig. 2). The losses of prestress (prestressing force) are divided as instantaneous σ_{pm0} ($P_{m,0}$; during the process of manufacture of prestressed structure) and time-dependent $\sigma_{pm\infty}$ ($P_{m,\infty}$, including long-term) (Fig. 2). The instantaneous losses of prestress σ_{pm0} ($P_{m,0}$) are divided as follows: due to a) the elastic deformation of concrete $\Delta\sigma_{p,el}$; b) fast-developing relaxation $\Delta\sigma_{p,r}$; c) loss of anchorage $\Delta\sigma_{p,r}$; d) temperature change in prestressing steel $\Delta\sigma_{p,temp}$ and e) friction $\Delta\sigma_{p,\mu}$. The time-dependent losses of prestress $\sigma_{pm\infty}$ ($P_{m,\infty}$) are divided as follows: due to a) elastic deformation of concrete $\Delta\sigma_{p,el}$; b) loss of anchorage $\Delta\sigma_{p,r}$; c) temperature change in prestressing steel $\Delta\sigma_{p,temp}$; d) friction $\Delta\sigma_{p,\mu}$ and e) creep and shrinkage of concrete and relaxation of steel $\Delta\sigma_{p,c+s+r}$.

Losses of prestress can be evaluated analytically and experimentally. The experimental techniques used to evaluate the losses of prestress include several typologies (Azizinamini et al., 1996; Labia et al., 1997; Baran et al., 2005; Wu et al., 2011; Caro et al., 2013).

In Estonian industrial and agricultural buildings there exist a lot of precast concrete load-bearing structures, which were mass-produced in concrete factories of the former Soviet Union from the 1960ies until at least 1990.

In this research precast concrete ribbed ceiling panels from an existing agricultural building (pigsty) are studied. These kind of precast concrete ceiling panels were very commonly used in the industrial, agricultural and military buildings across the former Soviet Union and are partially still in service in Eastern European and Baltic countries. By now, due to corrosion deteriorations, many of these ribbed panels are rather near for reaching their designed service life. The owners of the building need to make informed decision whether to further use, refurbish or demolish those ribbed panels.

There is no evidence-based information how much retention stress is left in prestressing bars (PSBs) of ribbed panels after long period (many decades) of service. The objective of this study is to evaluate experimentally the retention stress of PSBs of a prestressed ribbed panel PNS-12. In other words the purpose was to find out how much of the factory-issued initial prestress was left at the PSBs of ribbed panels after 20 years of service.

MATERIALS AND METHODS

The studied ribbed panels were from Vara pigsty (Tartu county, Estonia), which was constructed in 1986, and the ribbed panels were tested in 2006. The top view, longitudinal and transverse section of a ribbed panel PNS-12 is demonstrated in Fig. 3. The abbreviation PNS denote that the ribbed panels are prestressed and the number refer to different load-bearing capacities (PK-01-111, 1961).

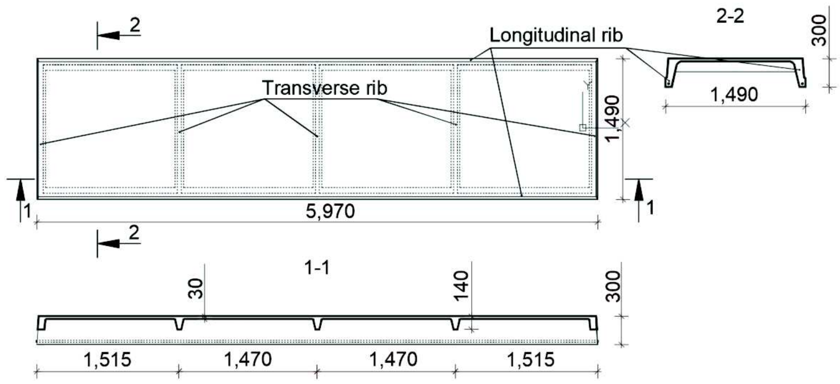

Figure 3. Top view, longitudinal and transverse section of a ribbed panel PNS-12 (PK-01-111, 1961). *Dimensions are in mm.*

Six ribbed panels were assessed on a 6-point rating scale (5, 4, 3, 2, 1 and 0) according to visually distinguishable corrosion deterioration. Grade 6 indicates no corrosion deteriorations, grade 2 indicates cracks in transverse ribs, grade 1 cracks in longitudinal ribs and grade 0 spalled cover of the longitudinal rib of a ribbed panel (Miljan & Kiviste, 2010). Fig. 4 shows the corrosion deteriorations of ribbed panel P8. Due to corrosion-affected spalled cover at the longitudinal ribs, ribbed panel P8 received grade 0 in the visual rating scale.

Five 20 years old prestressed concrete ribbed ceiling panels of type PNS-12 from an existing agricultural building (pigsty) are studied. The prestressing and reinforcing steel details of a ribbed panel PNS-12 are shown in Fig. 5.

According to the design drawings of ribbed panels PNS-12 employed two hot-rolled low-alloyed PSBs of mark of mark 35 GS (C from 0.3 to 0.37%, Mn from 0.8 to 1.2%, Si from 0.6 to 0.9%, Cr = 0.3%, Ni = 0.3%, Cu = 0.3% (PK-01-111, 1961; GOST 5058-65). The diameter and prestress of PSBs of ribbed panels PNS-12 are 16 mm and 343 MPa (N mm^{-2}), respectively. The ultimate strength of prestressing steel of PNS-12 should be at least 5,500 kgf cm^{-2} (539 MPa) to correspond to its mark. The prestressing and reinforcing steel details of ribbed panels PNS-12 are shown in Fig. 5.

Figure 4. The condition of ribbed panel P8 before testing. Due to corrosion-affected spalled cover at the longitudinal ribs, ribbed panel P8 received grade 0 in the visual rating scale.

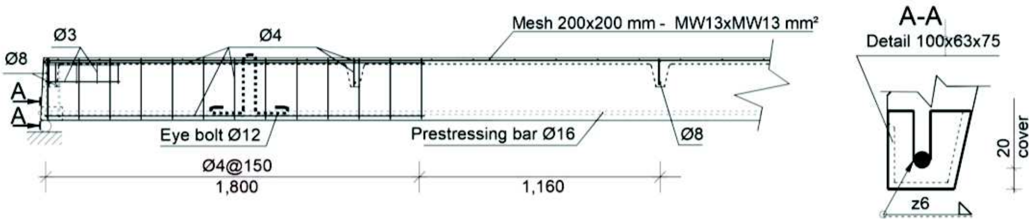

Figure 5. Prestressing and reinforcing steel details of the ribbed panel PNS-12 (PK-01-111, 1961). *Dimensions are in mm.*

A total of twelve PSB specimens with a length 300mm were cut from longitudinal ribs of each studied ribbed panel (P7-P11). The yield strength, ultimate strength and modulus of elasticity of each PSB specimen was determined in tensile test using testing machines P-20 and Instron 3369. The length change for calculating the modulus of elasticity was measured by an optical gauge (Advanced Video Extensometer 2663-821) and software Bluehill 2, which is based on the American (ASTM E8-16a) and European (EVS-EN ISO 6892-1:2016) standards. For modulus of elasticity the approximation line was obtained between the loads (initial F_1 and final F_2) as 10% and 40% of yield strength. The diameter of PSB was calculated according to standard EVS-EN ISO 6892-1:2016.

For the evaluation of compressive strength, ten concrete cores with a diameter of 50 mm were drilled (steel drill with diamond bits Rems Picus S3) from the longitudinal rib of ribbed panels. Later on, the cores were cut to the height of also 50 mm, in order to reach the height and diameter ratio of 1, which is equivalent to the standard cube strength (EVS-EN 13791:2007). The compressive strength of concrete of ribbed panels PNS-12 should correspond to concrete strength mark M200 = 200 kgf cm^{-2}, which corresponds to 19.6 MPa (PK-01-111).

One ribbed panel (P7) with a largest camber (negative initial deflection, -20 mm) and least corrosion deteriorations (grade 3) was chosen to the retention stress evaluation procedure. Ribbed panel was placed upside down in order to prevent the additional stresses developed by the dead-weight of a ribbed panel.

A first attempt was made to evaluate the longitudinal change of length of prestressed bars with Helios Preisser Digi-Met 0 273 7 Digital Caliber 450 mm with an accuracy of 0.01 mm. However, the accuracy of the caliber applied in measurements was not sufficient, and was used only for approximate evaluation later on.

The retention stress of prestressed bars was evaluated at the middle-span of the two longitudinal ribs (A-B and C-D) of a ribbed panel (P7). In those places the concrete was locally removed. As a novel application in civil engineering, strain gauges were applied in the evaluation of retention stress of PSBs. Strain gauge is based on the physical property of electrical conductance and its dependence on the conductor's geometry. The measured electrical resistance of the strain gauge is proportional to the amount of induced strains.

Strain gauge with a basic length 20 mm, resistance 200 W and a gauge factor 2.0 at 20 ± 1 °C was applied for measuring strains. Strain gauges were glued (БФ-2 glue) in the longitudinal direction onto pre-cleaned and flat-sanded surface (the finish roughness of the surface (about 3.2 μm) was obtained with sandpaper) of PSB and joined to form of half-bridge with dummy gauge glued to the unloaded steel plate, which was needed for thermal compensation (Fig. 6).

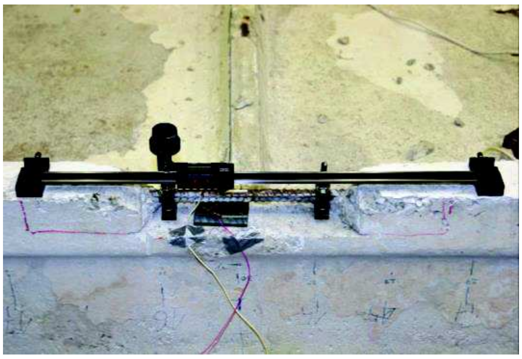

Figure 6. Pre-cleaned and pre-prepared PSB, with strain gauges and dummy gauge on the steel plate. Attached Helios Preisser Digi-Met 0 273 7 Digital Caliber 450 mm had insufficient accuracy (0.01 mm) and was applied only for approximate evaluation of longitudinal change of length of PSB.

After cutting the PSB the signals of strain were recorded by strain indicator (Wemmo-Anti), consisting of a half bridge and a processor which is capable of saving signals and saved data can be sent directly to computer for further processing. Calibration of strain gauges represent separate experiments. The strain gauges were glued onto a cantilever beam of uniform strength, which was loaded step by step mechanically by a screw. The readings of the strain and deflection indicators (dial gauge with accuracy of 0.002 mm) were registered at every step. The strain indicator constant was found by using the program *MS Excel 2016* with the regression analysis function. As a result, the strain indicator constant $C = 5.01 \times 10^{-7}$ (per unit of the strain indicator) was obtained.

RESULTS AND DISCUSSION

The material properties of the studied ribbed panels were found. The results of tensile test of PSB specimens averaged on studied ribbed panels (P7-P11) are presented in Table 1. The tensile test graph of two randomly chosen PSB specimens from each of the studied ribbed panel, thus, a total of ten specimens are presented in Fig. 7. All the studied prestressed bars exhibited yielding as show in Fig. 7.

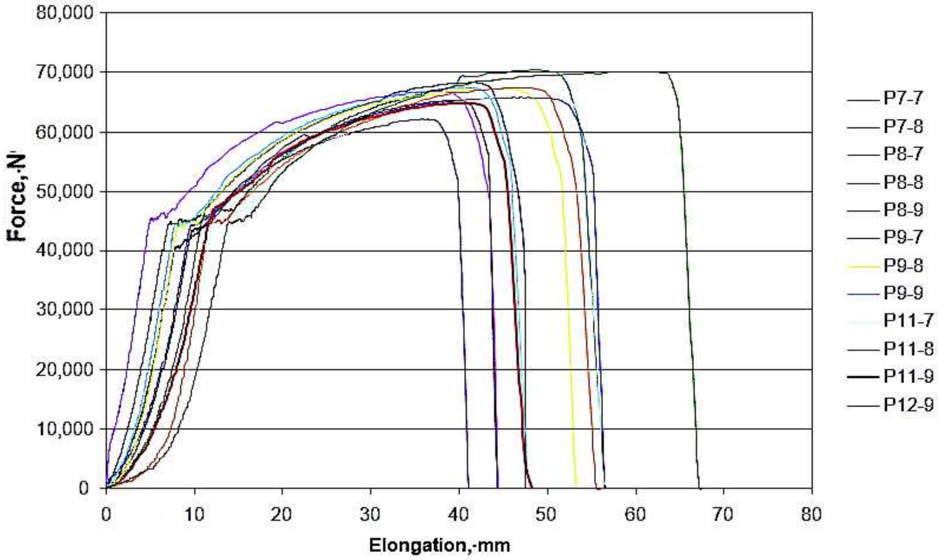

Figure 7. A graph representing the tensile test of two randomly chosen PSB specimens from each studied ribbed panel (P7-P11).

According to construction drawings, ribbed panels PNS-12 employed two hot-rolled low-alloyed PSBs of mark 35 GS. The ultimate strength of pre-stressing steel of ribbed panels PNS-12 should be at least 539 N mm^{-2} to correspond to its mark (PK-01-111, 1961; GOST 5058-65).

Table 1. Tensile test results of the studied PSB specimens (ø16 mm) from ribbed panels PNS-12 (P7-P11). The ribbed panel for the evaluation of retention stresses (P7) is marked as bold

Ribbed panel	Average ultimate strength of PSB specimens, MPa	Average yield strength of PSB specimens, MPa	Average modulus of elasticity of PSB specimens, GPa
P7	**657.1**	**544.3**	**197.0**
P8	*698.9*	620.5	*192.0*
P9	649.0	576.2	221.5
P10	635.8	576.1	175.7
P11	660.6	566.0	174.3

The average ultimate strength of PSB specimens of the studied ribbed panels (in Table 1) was found higher than strength mark. Thus, the ultimate strength of prestressing steel was found to conform the requirements of prestressed ribbed panels PNS-12. However, the characteristic ultimate strength of high-strength wire strands applied in modern prestressed concrete is f_{pk} = 1,500…2,000 MPa (N mm^{-2}, EVS 833-1:2002, EN 10138-1: 2000). Therefore, the strength properties of the studied prestressing steel bars of ribbed panel PNS-12 are about three times lower and thus directly incomparable to high-strength wire strands. Therefore, also much greater losses of prestress due to both fast-developing and long-term relaxation are expected in the studied PSBs.

The results of compressive test of cores from the studied ribbed panels is presented in Table 2. According to construction drawings of ribbed panels PNS-12 the average compressive strength of concrete should correspond to concrete strength mark M200, which corresponds to 19.6 MPa (PK-01-111). Table 2 shows that the average compressive strength of cores from the studied ribbed panel (P7, which was used for evaluation of retention stresses) was 44.3 MPa. Thus, the concrete compressive strength of ribbed panel P7 was more than twice greater than required (strength mark) and has not decreased during 20-years time. For the study of losses of prestresses also concrete strains have to be studied in order to evaluate the instantaneous and time-dependent (shrinkage and creep) strains of concrete. However, that was impossible to perform in current study as comparative (laboratory) study of concrete specimens at the different age would be necessary for that. The instantaneous and time-dependent strains (including shrinkage and creep strains) of concrete prismatic specimens over 1 year were studied in different research (Caro et al. (2013)).

Table 2. Core strength, carbonation and cover depth of concrete of the studied ribbed panels PNS-12 (P7-P11). The ribbed panel for the evaluation of retention stresses (P7) is marked as bold

Ribbed panel /Longitudinal rib	Average of core compressive strength, MPa	Standard deviation of core compressive strength, MPa	Carbonation depth / cover depth of concrete, mm
P7/A-B	**44.3**	**± 4.8**	**7.0 / 23.6**
P7/ D-C			**7.4 / 24.2**
P8/A-B	*18.4*	± 4.9	*27.0 / 24.4*
P8/ D-C			*22.3 / 24.3*
P9/A-B	56.4	± 3.8	8.4 / 25.0
P9/ D-C			8.4 / 24.2
P10/A-B	43.7	± 6.8	9.9 / 25.6
P10/D-C			9.4 / 27.7
P11/A-B	45.8	± 8.7	8.1 / 24.3
P11/D-C			9.1 / 24.6

The ribbed panel (P7) with the least corrosion deteriorations was deliberately chosen for the study as the corrosion deteriorations increase the losses of prestress. The studied ribbed panel (P7) received grade 3 at visual scale, which means no visual corrosion deteriorations were found at the longitudinal ribs. The full cover has to be carbonated (Table 2) for the propagation of carbonation-induced corrosion. However, only about one third of a cover was carbonated at the studied ribbed panel (P7). In comparison, the ribbed panel with the lowest compressive strength of concrete (P8 in Table 2) had also the full cover carbonated in average. Also almost full cover carbonation could lead to corrosion. Also, the cleaned out PSB (in Fig. 6) had also no signs of corrosion.

The ultimate strength, yield strength and modulus of elasticity (E-modulus in Table 1) of PSB specimens of studied ribbed panel (P7) for calculations was found as 657.1, 544.3 and 197×10^3 MPa (N mm^{-2}), respectively. The retention stresses of PSBs were calculated using the Hooke's law: $\sigma = E \cdot \varepsilon$, where $\varepsilon = C(A'' - A')$, A" is the final reading of strain indicator, after the cutting of PSBs and A' is initial reading of strain indicator before cutting.

The retention stress at the middle-span of longitudinal rib A-B of ribbed panel (P7) are presented as follows: A'' = 945 unit, A' = 224 unit and σ = 71.2 MPa (N mm^{-2}) The retention stress of the longitudinal rib C-D of a ribbed panel (P7) is presented as follows: A'' = 756 unit, A' = 412 unit and σ = 34.2 MPa (N mm^{-2}). The retention stresses of PSBs at two longitudinal ribs of a same ribbed panel differ, but the difference is less than a magnitude.

The initial prestressing force P_0 and the initial prestress σ_p of the studied ribbed panel (P7) is not known. Unfortunately, P_0 and σ_p are also untraceable as a part of a batch of precast prestressed ribbed panels in mass-production in concrete factory. Therefore, the actual loss of prestress of the individual ribbed panel could not be found. According to construction drawings the prestress of PSBs of ribbed panels PNS-12 is 343 N mm^{-2} (3,500 kgf cm^{-2}, PK-01-111, 1961). Therefore, the retention stresses are 20.8% (longitudinal rib A-B) and 10.0% (C-D) of the assumed initial prestress. This means about 79–90% loss of prestress of a ribbed panel at the age of 20 years.

In comparison with other experimental research, 25–60% measured losses of prestress were noticed already over 1 year (Caro et al., 2013). Caro et al. (2013) evaluated the losses of prestress in prestressed concrete prismatic specimens of three different cross sections The total measured losses of prestress values ranged from 25% to 60%: 25–40% for specimens with greater cross-sections (100×100 mm^2), 40–50% for specimens with intermediate cross-sections (80×80 mm^2), and 50–60% for specimens with smaller cross-sections (60×60 mm^2).

The determination of losses of prestress usually involves complicated, laborious procedures because time-dependent prestress losses are inter-dependent (Francis & Au, 2011). Prestressing reinforcement relaxation is continuously altered by changes in stress due to concrete shrinkage and creep. Concrete creep, in turn, constantly alters by changes in prestressing steel stress. Moreover, concrete shrinkage and creep movements are partially restrained by the prestressing steel.

It is generally accepted that losses of prestress have little effect on ultimate design strength and on the capacity of pretensioned concrete structures, but can affect service conditions (ACI 318-11). It has been found out in earlier studies that the residual load-bearing flexural capacity of the studied six ribbed-panels was sufficient. All the studied panels, (with ultimate load q_u), irrespective of their grade were able to carry the design load q_d (q_u/q_d. > 1.95) (Kiviste & Miljan, 2010).

CONCLUSIONS

Retention stresses of the 20-year-old prestressed ribbed ceiling panel were experimentally investigated with the strain gauges. For the background information of losses of prestress the strength properties of concrete and prestressing steel bars of five ribbed panels were also evaluated.

The studied prestressing bars have lost most of its initial prestresses and the retention stresses are relatively low (10% and 21%). Although conforming to the requirements, the strength properties of the studied PSBs was greatly lower than that of prestressed wire stands applied in modern prestressed concrete. Therefore, also much greater losses of prestress were found in the studied PSBs.

The residual load-bearing capacity of the ribbed panel evaluated in earlier study was found to be sufficient. Thus, the studied structure was still in accordance with the ultimate limit state. However, the losses of initial prestress (90% and 79%) are indicating that the studied structure (ribbed ceiling panel) at the age of 20 years is inefficient according to service conditions (serviceability limit state). Therefore, careful monitoring of the studied structure is required in the further operation. Undesired cracking or excessive deflections may occur and could impair the serviceablity of a structure in the future.

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