49
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STEPGRAD
СТЕПГРАД

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METHODS FOR STRUCTURAL CONDITION ASSESSMENT AND PROOF LOAD TESTING: A CASE STUDY OF THE CAMPUS GARAGE AT THE UNIVERSITY OF BANJA LUKA

Abstract:

This paper presents a comprehensive condition assessment of the campus garage at the University of Banja Luka. The assessment methodology included visual inspections, non-destructive testing (NDT) of concrete using rebound hammer and ultrasonic pulse velocity methods, and proof load testing of the primary roof truss. Deformations were monitored using dial gauges, deformeters, and geodetic instruments, specifically laser scanning and digital levels. Numerical modeling was conducted to determine the theoretical deflections of the roof truss, with particular emphasis on the specific characteristics of the bottom chord. The key contribution of this work lies in the comparative analysis of results obtained through different methods and measuring devices. This study aims to enhance the condition assessment of similar structures and provide a framework for the future Code of Practice for Structural Assessment in the Republic of Srpska.

Keywords: condition assessment, proof loading, steel truss, degradation, deflection.

МЕТОДЕ ПРОЦЈЕНЕ СТАЊА КОНСТРУКЦИЈА И ИСПИТИВАЊЕ ПРОБНИМ ОПТЕРЕЋЕЊЕМ: ПОРЕЂЕЊЕ РЕЗУЛТАТА НА ПРИМЈЕРУ ГАРАЖЕ У КАМПУСУ УНИВЕРЗИТЕТА У БАЊОЈ ЛУЦИ

Сажетак:

У раду је приказана свеобухватна процјена стања гараже у Кампусу Универзитета у Бањој Луци. Примијењени су визуелни преглед, недеструктивна испитивања бетона склерометром и ултразвуком, као и испитивање пробним оптерећењем главног кровног решеткастог носача. Деформације су праћене угибомјерима, деформетром, те геодетским инструментима – ласерским скенером и нивелиром. Спроведена су и софтверска моделирања ради одређивања вриједности теоријских деформација кровног носача, за који су везане специфичности у погледу доњег појаса решетке. Посебан допринос рада огледа се у упоредној анализи резултата добијених различитим методама и мјерним уређајима, што може допринијети унапређењу процјене стања сличних конструкција и представљати смјерницу за будући Правилник о процјени стања конструкција у Републици Српској.

Кључне ријечи: процјена стања, пробно оптерећење, челична решетка, деградација, угиб.

1. INTRODUCTION

The study encompassed data acquisition regarding the existing structure, a comprehensive visual inspection, and the implementation of selected non-destructive testing (NDT) methods directed at evaluating material quality [1] and structural load-bearing capacity.

In the second phase, the structural behaviour of the steel roof truss was investigated through integrated experimental and numerical analyses. A systematic protocol was established to monitor the deformation of the truss under proof loading. Experimental data obtained from the in-situ structure were analysed and cross-referenced, accounting for the diverse measurement techniques employed, including mechanical dial gauges, a deformeter, and geodetic instrumentation. Concurrently, various numerical models were developed using Tower software, and their outputs were subjected to a comparative evaluation. Finally, the experimentally measured deformations were validated against the corresponding results derived from the numerical simulations.

Figure 1 shows the structure subject to the condition assessment.

Figure 1. ''Campus garage'' at the University of Banja Luka (photography by author).

The estimated age of the structure is approximately 55 years. The structure has a regular rectangular plan with overall dimensions of 12 × 36 m. The roof covering consists of 5 cm thick sandwich panels. The roof covering is supported by steel purlins made of rectangular hollow sections (RHS) 100 × 60 × 3 mm, spaced at a center-to-center distance of 102 cm. According to the static system, the purlins act as simply supported beams resting on the main steel truss girders. The primary load-bearing structure consists of ten transverse frames spaced at 4 m intervals. The upper chord of the steel truss is composed of two equal-leg angle sections L 65 × 65 × 7 mm with a spacing of 22 mm, while the lower chord consists of two equal-leg angle sections L 50 × 50 × 5 mm with the same spacing. The truss web consists of diagonal members formed by bending reinforcing bars with a diameter of 22 mm. The diagonals are welded to the upper and lower chords of the truss. The connection between the truss and the column is pinned, realized by means of a bearing steel plate with stiffening plates and four anchor bolts. The vertical load-bearing structure consists of reinforced concrete columns with an I-shaped cross-section, with overall dimensions of 18.5 × 25.5 cm.

2. STRUCTURAL CONDITION ASSESSMENT

The condition assessment was carried out using the following methods:

- visual inspection of the structure,
- non-destructive testing of surface hardness using a Schmidt rebound hammer in order to determine the compressive strength of concrete,
- measurement of the velocity of ultrasonic longitudinal waves for the purpose of assessing the quality of concrete.

2.1. VISUAL INSPECTION OF THE STRUCTURE

The diameter and arrangement of the reinforcement, as well as the concrete cover, were determined by visual inspection, taking into account the observed level of degradation; therefore, the application of electromagnetic reinforcement detection methods (rebar locator/profometer) was not required. Using a vernier caliper, the diameter of the smooth longitudinal load-bearing reinforcement bars in the column was measured as 16 mm, indicating that the column is reinforced with 6Ø16 bars. In addition, the position and diameter of the transverse reinforcement were clearly visible, and it was established that smooth bars with a diameter of 6 mm were used as stirrups at a spacing of approximately 150 mm, with measured values ranging from 130 to 240 mm. No reduction of stirrup spacing was observed in the end regions of the columns. The average measured concrete cover was 25 mm, with individual measurements ranging from 15 to 35 mm.

Corrosion observed on the steel members and welded joints, as shown in Figure 2, indicates long-term exposure of the structure to adverse environmental actions and insufficient corrosion protection.

Figure 2. Typical examples of corrosion on steel members and welded joints (photography by author).

Spalling of the concrete cover was observed on most columns, primarily localized in the lower region of the column and in the upper region near the anchor plate. Inspection revealed that the concrete contained river aggregate with a nominal maximum size of 32 mm. In many locations, insufficient compaction of the concrete is evident, which, after decades of service, has contributed to the observed level of deterioration. The damage in the lower regions of the columns, shown in Figure 3 (left) from the base up to a height of 1.70 m, is additionally attributed to the combined action of frost and de-icing salts, with the latter being introduced via vehicle tires. The deterioration observed in the upper regions of the columns (Figure 3, right) results from the synergistic effects of inadequate concrete compaction and mechanical actions induced by the operation of the anchors under various service conditions. Localized concrete detachment was typically observed around one to two anchors.

Figure 3. Spalling of concrete cover in column lower zones (left) and upper zones near the anchor plate (right) (photography by author).

Significant concrete segregation was observed in several locations (Figure 4, left), with a complete absence of cement paste in zones extending up to half of the column width. At these locations, a local reduction in concrete strength of over 40% is estimated [2].

Non-structural parts of the structure – the parapet walls were severely damaged at numerous locations. Characteristic examples are shown in Figure 4 (right). Large portions of concrete were detached, the reinforcing steel was exposed and corroded, and vegetation growth was observed on the exterior surfaces of the walls.

Figure 4. Segregation of concrete in the columns (left) and deterioration of non-structural elements (parapet walls) (right) (photography by author).

2.2. SCHMIDT REBOUND HAMMER TEST

The surface hardness of the reinforced concrete columns was assessed using a Schmidt rebound hammer manufactured by SJJW, model HT225-W+ [3].

On each tested column, a total of 16 impacts were performed, with all test points carefully marked [4]. When determining the representative average rebound index, minimum and maximum acceptable rebound values were defined, which should not deviate by more than 25% from the average of all rebounds measured at a single location [5].

2.3. ULTRASONIC TESTING

At the same locations where the Schmidt hammer tests were performed, the quality of the concrete was also assessed using ultrasonic pulse velocity measurements of longitudinal waves [6], using a MATEST device, type C369N.

3. PROOF LOADING

3.1. STRUCTURAL ANALYSIS OF THE STRUCTURE

At the time when the project was executed, the design calculations of structural elements were carried out using simplified procedures (without the application of advanced software tools that are commonly used in current engineering practice). This indicates that the analysis of the truss girders was performed using a model composed of straight bar elements with concentrated loads applied at the joints. However, during the construction phase, deviations from the design model occurred, and the girders were executed with a different geometry, as shown in Figure 5.

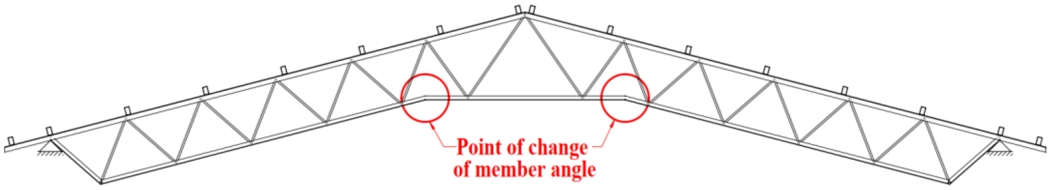

Figure 5. Polygonal configuration of the lower chord of the main truss girder (drawing by author).

In the continuation of the study, the actual (polygonal) structural model is analyzed, which was found not to satisfy the load-bearing capacity criteria for the design loads according to the JUS regulations. This result indicates the need for a more detailed analysis in order to explain how the structure has successfully withstood significant loads during service, particularly snow loads, which on several occasions exceeded the design values (a characteristic example being the snow load recorded in December 2024).

One possible explanation is that the steel girders in reality entered the plastic range of behavior, whereby, through the process of internal force redistribution, a new equilibrium state was achieved, enabling the structure to sustain load levels higher than those assumed in the design.

Plasticization of the main girder was observed in all girders except the two end girders. This can be explained by the fact that the end girders were subjected to approximately half of the load during service, and therefore did not undergo the above-mentioned plastic behavior.

Due to the previously described irregularities, an alternative, idealized calculation model of the main truss girder is introduced in this paper, as shown in Figure 6.

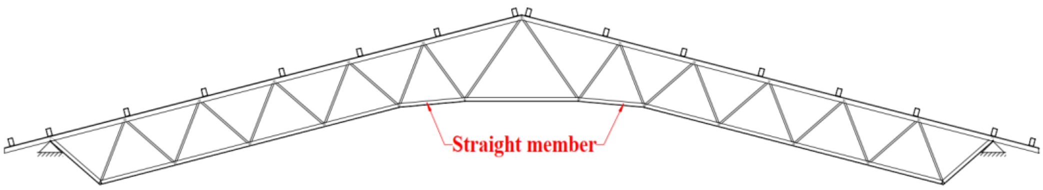

Figure 6. Idealized model of the main truss girder (drawing by author).

For this type of structure, snow load is generally considered the governing action; therefore, it was specified that snow load be adopted as the proof load.

The structure was originally designed for a snow load of 0.75 kN/m² in accordance with the "Privremeni tehnički propisi za opterećenje zgrada" (Temporary Technical Regulations for Building

Loads, in Serbian), published in the Official Gazette of SFRY No. 61/48 and in force at the time of construction [7]. Over time, technical regulations and standards are subject to change, and it is common for newer standards to prescribe higher requirements with regard to structural load-bearing capacity and serviceability under various actions. In this context, it was planned that the proof load be defined in accordance with the National Annex to the Eurocodes.

According to the National Annex BAS EN 1991-1-3/NA:2018 "Karta opterećenja od snijega na tlu u BiH" (Snow Load Map for Bosnia and Herzegovina, in Serbian), the characteristic value of the ground snow load for the location "Campus Garage" at the University of Banja Luka is 1.87 kN/m^2 [8]. When multiplied by the shape coefficient $\mu_i = 0.8$, which depends on the roof slope, this results in a value of 1.496 ≈ 1.50 kN/m^2, which is exactly twice the originally designed snow load.

Nevertheless, taking into account the previously identified structural irregularities (polygonal configuration of the lower chord of the truss girder), it was decided to define the proof load using the lower snow load value, i.e. in accordance with the aforementioned JUS regulation (0.75 kN/m^2). This approach ensures the safe execution of the load test while remaining within the load levels for which the structure was originally designed.

According to the applicable standard JUS.U.M1.047:1987, "Ispitivanje konstrukcija visokogradnje probnim opterećenjem i ispitivanje do loma" (Testing of Building Structures by Proof Loading and Testing to Failure, in Serbian), the position of the proof load under static loading shall correspond to the most unfavorable load configuration considered in the design or shall result in approximately the same internal forces in the characteristic cross-sections. During the application of the proof load, the structure shall be applied in at least four equal increments up to the design load level (with an efficiency factor not less than 1) [9].

In this chapter, the proof load is defined for four testing phases, as follows:

- F1 – 25% of the snow load (0.1875 kN/m^2),
- F2 – 50% of the snow load (0.3750 kN/m^2),
- F3 – 75% of the snow load (0.5625 kN/m^2) and
- F4 – 100% of the snow load (0.7500 kN/m^2).

The calculation models were developed for an isolated representative frame. Four models were analyzed in this study:

- polygonal model with actual load distribution (MP-AL),
- polygonal model with proof load (MP-PL),
- idealized model with actual load distribution (MI-AL) and
- idealized model with proof load (MI-PL).

3.1.1. Effects Obtained from the Structural Analysis

Figures 7 and 8 show the bending moments for the fourth phase of testing (100% of the snow load) for the polygonal and idealized models, respectively. Corrosion of steel elements detected during the visual inspection was not taken into account in the evaluation of the load-bearing capacity of the steel truss, as its influence was considered negligible.

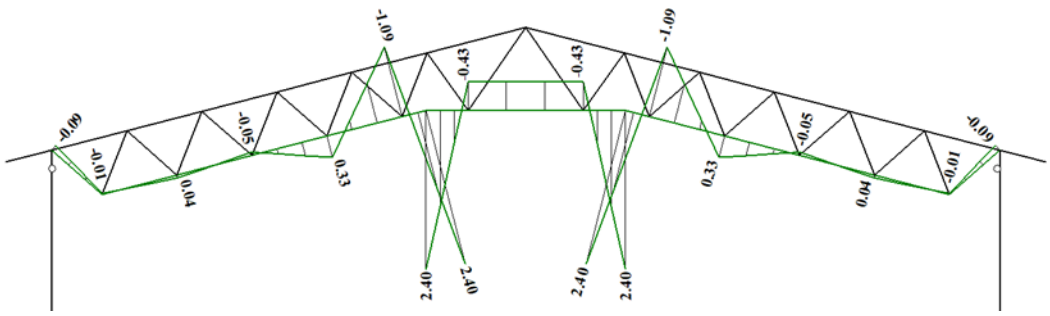

Figure 7. ***MP-PL*** *Bending moments of the bottom chord of the main girder [kNm] for Phase F4 (drawing by author).*

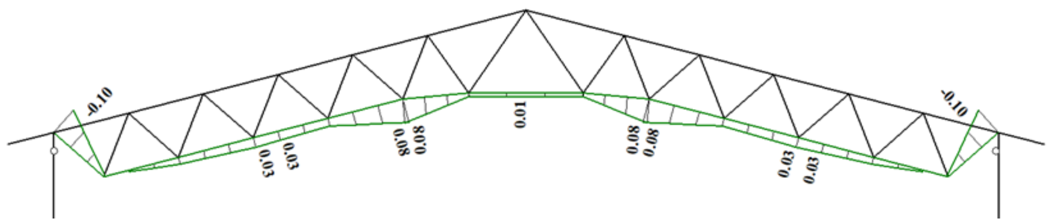

*Figure 8. **MI-PL** Bending moments of the bottom chord of the main girder [kNm] for Phase F4 (drawing by author).*

Figures 9 and 10 present the results of the stability check (without reduction of the allowable stress) for the polygonal and idealized models, respectively. Members of the lower chord of the polygonal model whose bending moments induced significant stresses exceeding the allowable values are marked in red. It was established that these members satisfy the stability criteria for the combination of permanent load plus **44%** of the snow load.

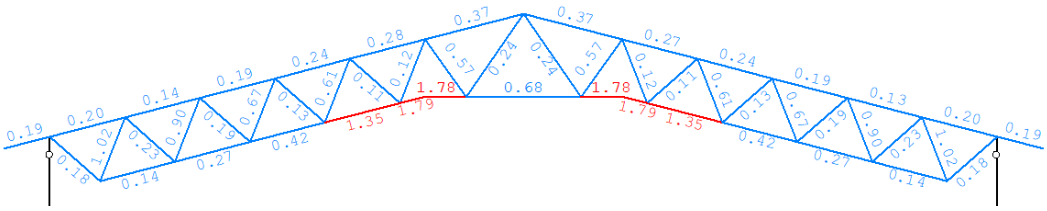

Figure 9. Stability check of the polygonal model (drawing by author).

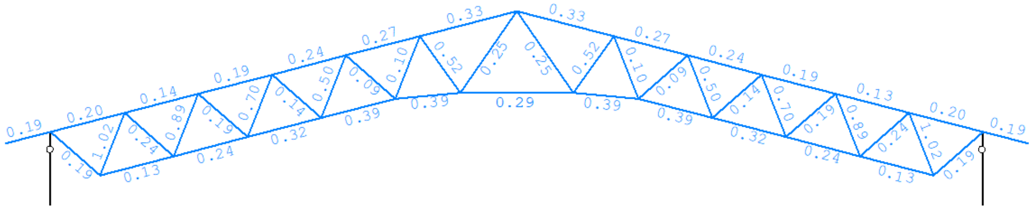

Figure 10. Stability check of the idealized model (drawing by author).

3.2. PROGRAM OF PROOF LOADING

The testing was carried out in accordance with the applicable standard JUS U.M1.047 and the testing program described below.

One day prior to the test, the structure was prepared. For the girder subjected to testing, five plastic tanks with corresponding pallets were suspended in accordance with the scheme shown in Figure 11. Each tank had a capacity of 1000 l, which corresponds to a load of 10 kN when the tank is filled with water.

On the day of testing, prior to the load application procedure, instruments for monitoring the structural behavior of the girder were installed. The following devices were used:

- five mechanical dial gauges of the "Huggenberger" brand, with a measuring range of 0–5 cm and an accuracy of 0.05 mm, for monitoring deflections (vertical displacements) of the girder, installed according to the scheme shown in Figure 11;
- seven deflectometers of the "Meixin" brand, with a measuring range of 0–1 mm and an accuracy of 0.001 mm, for monitoring relative elongations/shortenings of the bottom and top chords of the truss, installed according to the scheme shown in Figure 12;
- a terrestrial laser scanner (FARO Focus M70), with an accuracy of ± 3 mm [10], used for verification of the measured girder deformations;
- a digital precision level of the "Leica" brand, type DNA03, with a reading accuracy at the instrument of 0.05 mm, in combination with precision leveling staffs at measurement points U3 and U4 (Figure 11), used for controlling deflections during unloading of the girder.

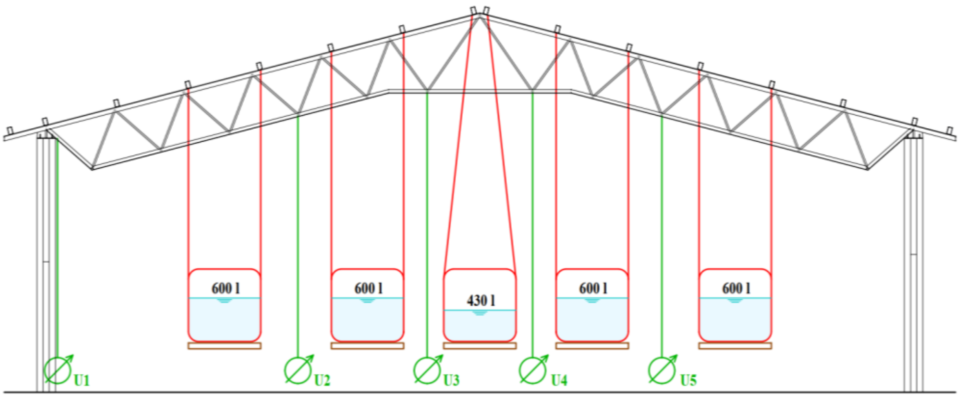

Figure 11. Scheme of proof load for the fourth testing phase (100% load) and arrangement of mechanical dial gauges and leveling measurement points (drawing by author).

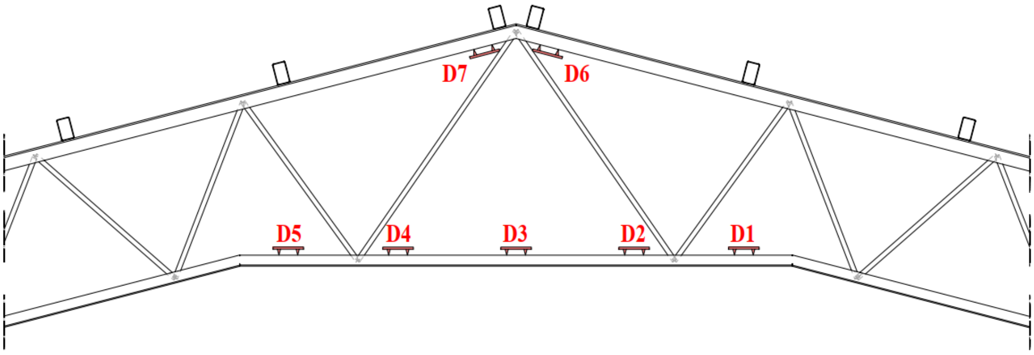

Figure 12. Arrangement of deflectometers (strain gauges) (drawing by author).

The proof load testing was conducted in four controlled loading phases [9], as follows:

- F1 – 25% of the load,
- F2 – 50% of the load,
- F3 – 75% of the load and
- F4 – 100% of the load.

Since it was observed on site that daily temperature increments had a certain influence on the development of girder deformations, a decision was made to monitor the variation of deflections over the full 24-hour temperature cycle under summer conditions. Specifically, the final loading phase was maintained until the following day of testing, up to 12:00.

Unloading was also performed in four phases:

- R1 – 75% of the load,
- R2 – 50% of the load,
- R3 – 25% of the load and
- R4 – complete unloading.

Figure 13 shows the proof load testing of the subject structure.

Figure 13. Proof load testing of the ''Campus garage'': load application on the steel truss (left) and container unloading (right) (photography by author).

3.3. PROOF LOADING RESULTS

3.3.1. Deflection Gauge Measurements

Table 1 presents the final deflection values for each testing phase obtained by deflection gauge measurements.

Figure 14 shows the variation of deflection values at measurement points U2, U3, U4, and U5 over the duration of the test. Measurement point U1 is omitted, as it did not exhibit any increase in deflection.

Table 1. Final deflection values obtained by deflection gauge measurements

Phase	U1 [mm]	U2 [mm]	U3 [mm]	U4 [mm]	U5 [mm]
F1 (25%)	0.000	2.440	3.060	3.040	2.560
F2 (50%)	0.000	5.390	6.210	6.430	5.310
F3 (75%)	0.000	8.400	9.820	10.140	8.450
F4 (100%)	0.000	10.910	12.550	13.190	10.980
R4 (0%)	0.000	0.890	1.420	0.720	0.710

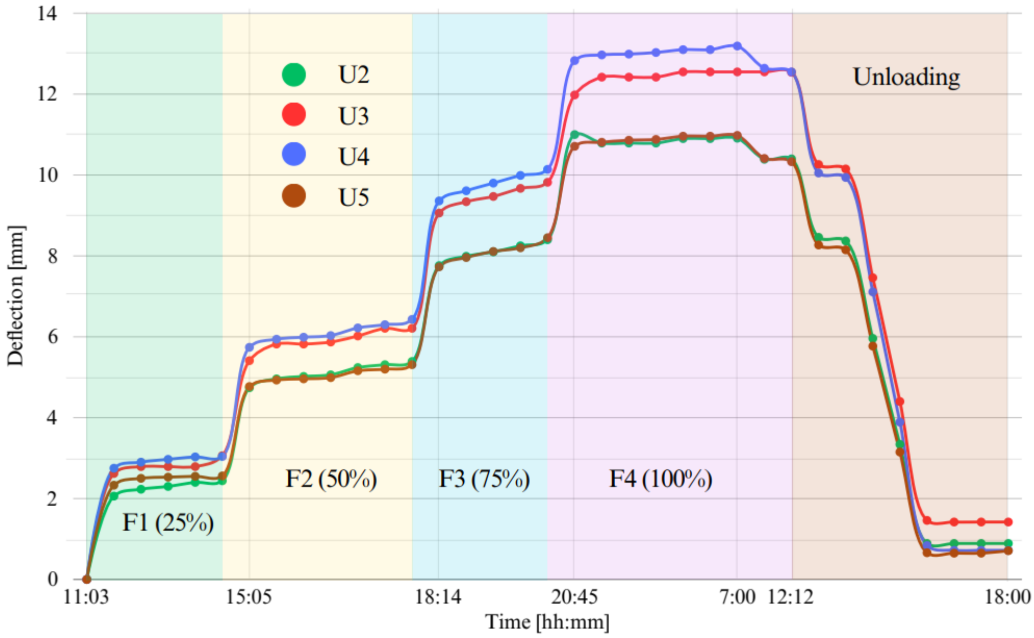

Figure 14. Measured deflections [mm] as a function of time for measurement points U2, U3, U4, and U5 (drawing by author).

3.3.2. Terrestrial laser scanner Measurements

Geodetic scanning was performed to record deformations through the aforementioned four loading phases (Figure 15), with the aim of determining the structural behavior of the girder and comparing the results with the deflection values obtained from deflection gauge measurements.

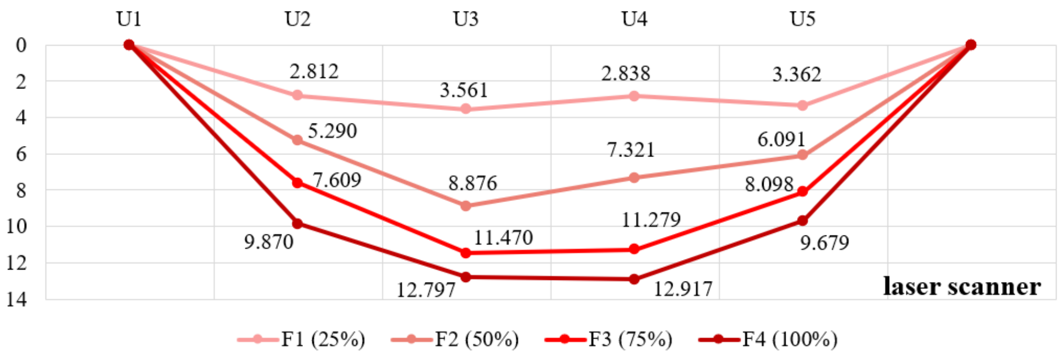

Figure 15. Deflections [mm] measured by the terrestrial laser scanner (drawing by author).

Based on the presented data, it can be concluded that the final deflection values measured by the scanner do not differ by more than 6.3% compared to the results obtained using mechanical dial gauges. Particularly good agreement was observed at measurement points U3 and U4, where the deviations do not exceed 3%. The maximum absolute deviation amounts to 2.666 mm, recorded at measurement point U3 during loading phase F2, which is still within the stated scanner accuracy of ± 3 mm.

The accuracy of the scanner can be improved by adjusting the quality parameter, which enables multiple measurements at the same resolution, thereby allowing the results to be averaged and higher precision to be achieved [11].

It is important to note that during the first three loading phases (F1–F3), scanner measurements, which last approximately 30 minutes, were carried out under conditions where the girder was still exhibiting increasing deflections. In contrast, for the final phase (F4), scanning was performed 15 hours after load application, resulting in significantly smaller measurement deviations compared to the previous phases.

Although the scanner-based deflection measurement method has lower accuracy compared to the use of mechanical dial gauges or leveling instruments, its advantage lies in the ability to capture the entire geometry of the structure rather than only discrete points [11], [12].

3.3.3. Deflectometers Measurements

Based on deflectometer measurements, the stress values presented in Table 2 were calculated.

Measurement point D2 was excluded from the analysis because displacement of the gauge support occurred during the first measurement. Measurement points D1, D4, and D5 were also excluded, since the initial values recorded in phase F0 significantly deviated from the results obtained in the subsequent phases, indicating their unreliability. All excluded measurement points are shaded in the table 4.

It is assumed that these deviations occurred due to slight displacement of the deflectometer supports, caused by insufficiently secured adhesive, most likely as a consequence of elevated temperatures. Difficult access to the aforementioned measurement points may have further affected the accuracy and reliability of the measurements.

Table 2. Stress values obtained from deflectometer measurements

Phase	D1 [MPa]	D2 [MPa]	D3 [MPa]	D4 [MPa]	D5 [MPa]	D6 [MPa]	D7 [MPa]
F1 (0%)	-173.3	–	-16.8	-155.4	172.2	14.7	21.0
F2 (25%)	-259.4	–	-18.9	-170.1	168.0	27.3	50.4
F3 (75%)	-181.7	–	-21.0	-178.5	149.1	50.4	48.3
F3 (100%)	-232.1	–	-35.7	-195.3	138.6	71.4	52.5
R4 (0%)	-137.6	–	-2.1	-151.2	178.5	4.2	12.6

3.3.4. Leveling Measurements

Using a digital precision level in combination with precision leveling staffs installed at measurement points U3 and U4, the deflections obtained from deflection gauge measurements were verified during unloading of the girder.

The values from measurement point U4 were discarded due to displacement of the leveling staff after it was struck by a tank, as a result of swaying caused by wind.

The deflection values obtained by the level and the deflection gauge at measurement point U3 showed close agreement. The deflection during unloading measured by the level was -11.26 mm, while the deflection measured by the deflection gauge was -11.12 mm, resulting in a percentage difference of 1.26%.

3.4. INFLUENCE OF TEMPERATURE ON GIRDER BEHAVIOR

Since the testing was performed under summer temperatures, the effects of temperature variations during the test are not negligible. The following analysis examines the influence of temperature differences at 7:00 and 12:00 on the second day of testing [13], when different deflection values were measured under the same load level (phase F4). It was assumed that no additional deflection due to the proof load occurred during this time period, as the girder had been loaded to phase F4 on the previous day, at which point the deflection values had already stabilized.

For the purpose of determining temperature differences in individual parts of the girder, temperatures of the bottom and top chords of the truss, as well as of the roof covering and purlin, were measured. A temperature difference between the two roof planes was observed, which is a consequence of the sun's position during the day.

Based on the presented data, it was found that the ambient temperature difference averaged about 10°C when comparing temperatures at 7:00 and 12:00. It was also shown that the top chord of the girder had, on average, a temperature 1.5°C higher than that of the bottom chord. On this basis, computational models were developed in the Tower software, in which temperature effects of 10.0°C were applied to the bottom chord and diagonals in the axes of the cross-sections, and 11.5°C to the axes of the cross-sections of the top chord. Figure 16 shows the calculated deflections of the bottom and top chords of the steel truss.

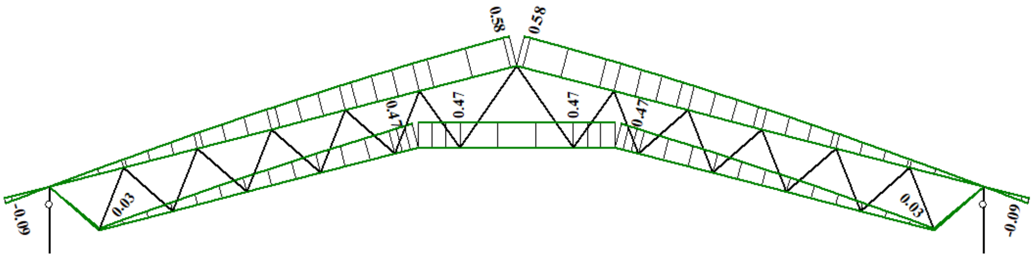

Figure 16. Truss deflection [mm] due to temperature variations (drawing by author).

3.5. ANALYSIS OF TEST RESULTS

3.5.1. Assessment of Condition Using Non-Destructive Methods

Based on the visual inspection of the structure, it can be concluded that most of the observed damage falls into damage level 3 (damage that reduces the long-term durability of the structure and promotes the development of degradation processes in the concrete cover layer) and damage level 4 (damage that may reduce the reliability of the structure in the foreseeable future) [14].

Following the visual inspection, the surface hardness of the columns was tested using a Schmidt hammer in order to determine the compressive strength of the concrete, and the velocity of ultrasonic longitudinal waves was measured to assess the quality of the installed concrete. An overview of the results of these tests is presented in Table 3.

Table 3. Overview of Schmidt hammer and ultrasonic test results

Position	Calculated compressive strength of concrete [MPa]	Wave velocity [m/s]
C/10	35.9	3807
D/8	36.1	3753
D/6	41.0	3593
D/4	42.2	3895
D/2	40.0	3551
C/1	46.8	3783
A/4	61.9	4075
A/6	44.5	3945
A/8	49.5	3830

Analysis of the calculated compressive strength results showed that the value obtained at position A/4 significantly deviated from the other values and from the arithmetic mean, and was therefore considered statistically unreliable and excluded from further analysis.

After eliminating this value, the estimated average calculated compressive strength of the concrete is approximately 42 MPa, referring to a 15 cm cube.

According to the "Pravilnik o tehničkim normativima za beton i armirani beton 1987" (Regulations on Technical Norms for Concrete and Reinforced Concrete, in Serbian), the ratio between the compressive strength of a 200 mm cube and a 150 mm cube specimen is 0.95 [15]. Accordingly, the compressive strength of concrete corresponding to a 200 mm cube specimen is 39.9 MPa, corresponding to concrete class MB 30 (approximately equivalent to class C25/30 according to Eurocode).

Furthermore, analysis of the results of ultrasonic longitudinal wave velocity testing indicated that all measured values fall within the range of 3500 to 4500 m/s. Such values, together with the Schmidt hammer test results, confirm the good quality of the tested concrete [14].

It should be noted that Schmidt hammer and ultrasonic tests, in accordance with the guidelines, were performed at the most favorable locations – flat, clean, and dry surfaces. Accordingly, the obtained results regarding the compressive strength and quality of the concrete should be interpreted critically, taking into account the previously described degree of structural damage.

3.5.2. Deflection Analysis

Deflection measurements were carried out across four loading and unloading phases at five measurement points installed along the bottom chord of the steel lattice girder.

Measurement point U1, located in the support zone of the girder, did not exhibit any deflection during all measurement phases, which is consistent with expectations. This behavior can be explained by the fact that the structure has been in service for approximately 55 years, allowing the ground to fully consolidate. Additionally, no displacement or slip at the support was observed.

Figures 17, 18, 19 and 20 present a comparison between the theoretical and experimental deflection values for load testing phases F1, F2, F3 and F4, respectively.

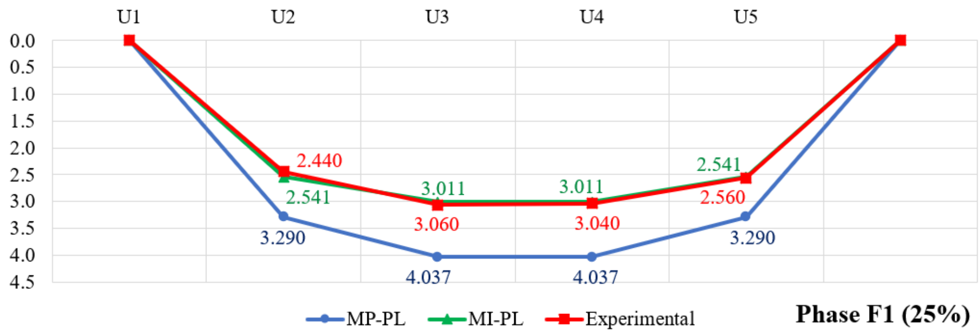

Figure 17. Theoretical and experimental deflection values [mm] for load testing phase F1 (25%) (drawing by author).

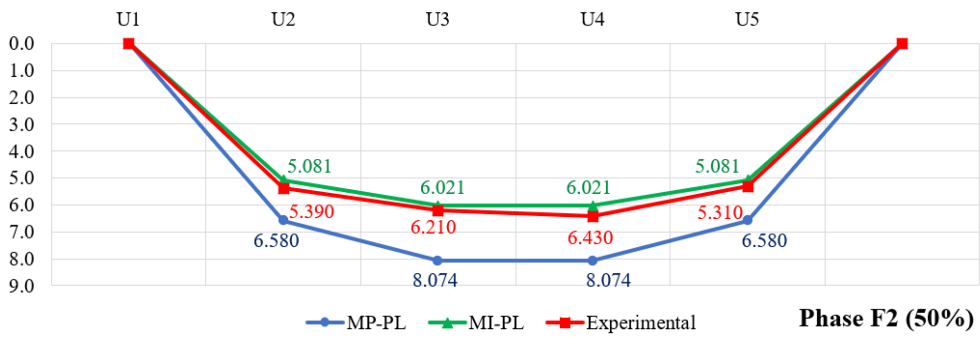

Figure 18. Theoretical and experimental deflection values [mm] for load testing phase F2 (50%) (drawing by author).

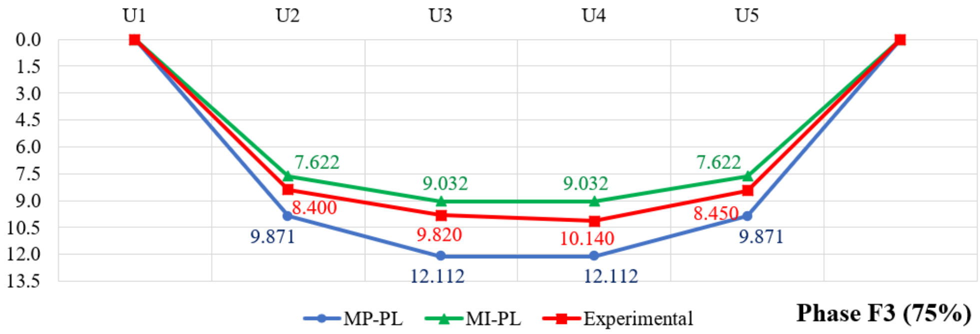

Figure 19. Theoretical and experimental deflection values [mm] for load testing phase F3 (75%) (drawing by author).

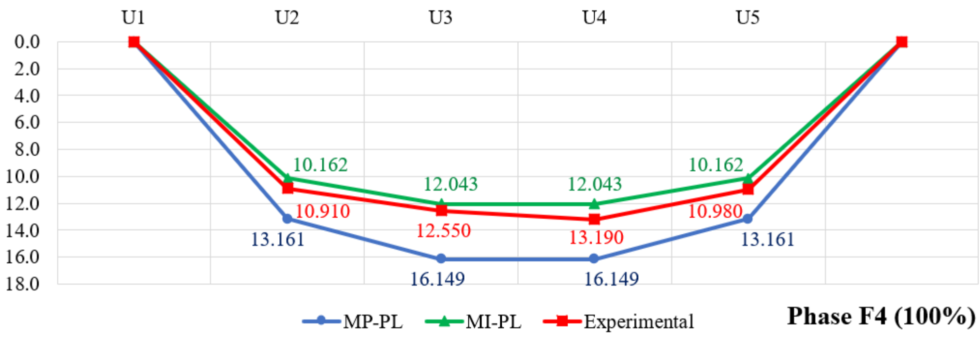

Figure 20. Theoretical and experimental deflection values [mm] for load testing phase F4 (100%) (drawing by author).

Based on the presented data on measured deflections, the following observations can be made:

- The experimentally obtained deflection values under the applied proof load on the lattice girder fall between the deflection values of the polygonal and the idealized models calculated using Tower software;
- For the polygonal computational model, the highest deflection values were obtained, i.e., 20–30% greater than the final deflections measured experimentally, indicating an overestimation of the actual structural response;
- For the idealized computational model, the lowest deflection values were obtained, i.e., 4–8% lower than the experimentally measured final deflections. This suggests that the experimental values are closer to the results predicted by the idealized model;
- The measured residual deflections are below 15% of the maximum measured deflections, indicating predominantly elastic behavior of the lattice girder [9].

3.5.3. Stress Analysis

Stress measurements were carried out across four loading and unloading phases at five measurement points installed on the bottom chord and two measurement points on the top chord of the steel lattice girder.

Four measurement points, D1, D2, D4, and D5, were excluded due to evident deviations in the results. The theoretically calculated and experimentally obtained stress values at the remaining measurement points D3, D6, and D7 are presented in Figures 21, 22 (left), and 22 (right), respectively.

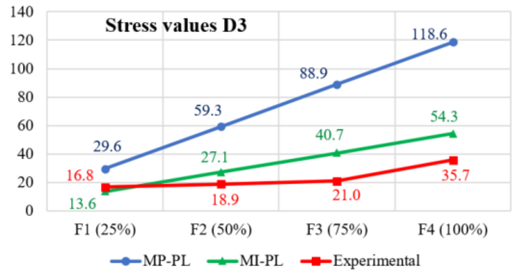

Figure 21. Theoretical and experimental stress values [MPa] at measurement point D3

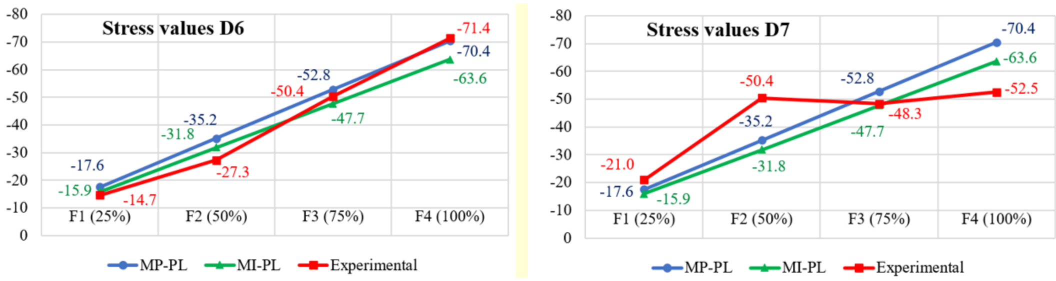

Figure 22. Theoretical and experimental stress values [MPa] at measurement point D6 (left) and D7 (right) (drawing by author).

Based on the presented stress data, the following observations can be made:

- The stress values of both the polygonal and idealized models at the midspan of the girder's bottom chord (measurement point D3), calculated using Tower software, are higher than the experimentally obtained stresses;
- The final stress of the polygonal model (phase F4) deviates by over 200% compared to the experimentally measured stress at D3, indicating that the lattice girder behaves more closely to the idealized model, for which the deviation at this point is approximately 50%;
- The measured stresses at points D6 and D7 (top chord of the truss) show variable deviations relative to the theoretical stresses but are still considerably closer to the predicted values than the stresses in the bottom chord;
- The measured residual stresses at D3 and D6 are 5.88%, while at D7 the residual stress reaches 24%. Considering that the other measurement points exhibit uniform behavior, the results at D7 can be considered unreliable.

3.5.4. Analysis of Temperature Effects

During the fourth phase of testing the steel lattice girder, the occurrence of negative deflection, i.e., upward movement of the girder, was observed under the same applied load. This phenomenon indicates the influence of other factors, primarily temperature variations.

In the theoretical models, based on the temperatures measured during the experiment, a corresponding temperature load was applied. A comparison of the theoretically calculated and experimentally measured deflection values for phase F4 is presented in Table 4.

Table 4. Theoretical and experimental deflections [mm] due to temperature effects at measurement points U2, U3, U4, and U5

Measurement point	Polygonal model	Idealized model	Experimental results
U2	-0.372	-0.362	-0.510
U3	-0.469	-0.461	-0.010
U4	-0.469	-0.461	-0.640
U5	-0.372	-0.362	-0.650

The collected data indicate that the deflections due to temperature effects in the polygonal and idealized models do not differ significantly, with differences on the order of 0.01 mm. However, the deflections measured experimentally show substantially higher values.

The deflection recorded at measurement point U3 significantly deviates from the other values and, therefore, cannot be included in the analysis.

At measurement points U2 and U4, the polygonal and idealized models show deviations of up to 30% relative to the experimental values, while at measurement point U5, this difference is more pronounced, exceeding 40%.

Analysis of the results confirms that the girder behaves as a shallow-arch structural system, with temperature-induced stresses causing its vertical upward movement.

3.5.5. Recommended Level of Intervention

Based on the overall condition assessment, it can be concluded that the tested structure falls under **Activity level 2**, indicating the need for investment measures to preserve its functionality and structural stability [14].

4. CONCLUSION

Condition assessment and proof loading represent essential activities for ensuring the safety, functionality, and durability of structures. These activities form the basis for decision-making regarding potential rehabilitation and/or strengthening of damaged structural elements or parts of a structure.

In this study, a condition assessment of the garage located on the Campus of the University of Banja Luka was carried out, together with proof loading of the main roof girder. The condition assessment was performed using non-destructive testing methods, namely visual inspection of the structure, surface hardness testing using a Schmidt rebound hammer to estimate the compressive strength of concrete, and ultrasonic pulse velocity measurements of longitudinal waves to assess the quality of the in-situ concrete. The proof load test was conducted in accordance with the applicable JUS standard. As the governing load case, a snow load of 0.75 kN/m² was adopted in accordance with the JUS regulations that were in force at the time the structure was designed.

The visual inspection revealed a high degree of structural degradation. Pronounced corrosion of steel girders and welded joints, concrete segregation, as well as significant damage to the concrete cover and cracking at multiple locations were observed. In the lower regions of the columns, concrete cover damage is attributed to insufficient concrete compaction combined with the simultaneous effects of frost and de-icing salts. Damage in the upper regions of the columns is likewise a consequence of inadequate concrete compaction and mechanical effects induced by anchor action under various service conditions.

Most of the observed damage corresponds to damage level 3 (damage that reduces the long-term durability of the structure and accelerates degradation processes in the concrete cover) and damage level 4 (damage that may, in the foreseeable future, reduce the reliability of the structure).

Non-destructive testing using the Schmidt rebound hammer indicated an average calculated concrete compressive strength of 39.9 MPa, with the concrete grade identified as MB 30. Analysis of ultrasonic pulse velocity test results confirmed a good quality of the tested concrete.

Since Schmidt hammer and ultrasonic tests were performed at the most favorable locations in accordance with the testing guidelines, the obtained results regarding concrete strength and quality should be interpreted with a certain degree of caution, taking into account the level of deterioration observed in the remaining parts of the structure.

A correct and accurate assessment of the condition of a structure subjected to degradation processes represents the foundation of any successful rehabilitation strategy. Based on the identified damage and defects, appropriate rehabilitation measures should be defined for the structure under consideration. These measures may include removal of the damaged concrete cover and its replacement with new material, such as new concrete, repair mortar, or another appropriate repair material. Prior to this, thorough cleaning of corroded reinforcement bars is required, followed by the application of protective treatments to prevent further corrosion development. Similarly, corrosion of steel girders may be removed mechanically or by sandblasting, followed by the application of appropriate corrosion protection systems.

Structural analysis was performed for both the polygonal (as-built) model and the idealized model. It was found that the polygonal model satisfies the stability criteria for permanent actions combined with 44% of the snow load. One possible explanation for the structure's ability to withstand higher loads during service is that the steel girders entered the plastic range of behavior, and through redistribution of internal forces reached a new equilibrium state capable of sustaining load levels higher than those assumed in the design. This observation is consistent with findings reported by several authors [16], [17], [18], [19] who analyzed different types of structures, indicating that as-built structures are in most cases stiffer than originally designed.

The static analysis of the main roof girder was used to determine the load distribution for the proof load test. This distribution achieved a good simulation of the actual load distribution, based on the resulting deflections and stresses obtained from numerical analysis.

Analysis of the test results showed that the experimentally measured deflections under proof loading of the truss girder lie between the deflection values obtained from the polygonal and the idealized numerical models implemented in the Tower software. For the polygonal calculation model, final deflection values were 20–30% higher, whereas for the idealized model they were 4–8% lower compared to the actual experimentally measured deflections. Based on these results, it can be concluded that the experimental deflections are closer to the results obtained from the idealized model.

The measured residual deflections were less than 15% of the maximum measured deflections, indicating elastic behavior of the truss girder.

Deflections measured using a terrestrial laser scanner showed certain deviations compared to values obtained using displacement transducers, which is considered reasonable given the measurement accuracy of the terrestrial laser scanner (± 3 mm). The observed deviations did not exceed this specified accuracy range.

During strain gauge measurements, significantly lower stresses were recorded in the lower chord of the girder compared to stresses obtained from numerical analysis for both the polygonal and idealized models. The measured stress values, together with the deflection results, indicate that the girder behavior is closer to that of the idealized model. Experimentally measured stresses in the upper chord of the truss showed variable deviations relative to the numerically obtained values for both models, but were nevertheless considerably closer to the numerical results than the stresses measured in the lower chord. The observed discrepancies between experimental and numerical results were also influenced by air temperature, as strain gauges may be sensitive to temperature variations. Therefore, whenever possible, proof loading should be carried out under stable temperature conditions, and the use of temperature-compensated measuring devices is recommended to achieve higher accuracy.

Analysis of temperature effects indicated that the girder behaves as a shallow arch structural system, with temperature-induced stresses causing vertical uplift of the girder.

No loosening of connections between web members and chords, no slippage of chord splices, and no deformation of support zones were observed during the experiment.

The results of the proof loading indicate that the structure is capable of carrying the snow load defined according to the JUS regulations. However, according to the current requirements of the interactive snow load maps for Bosnia and Herzegovina, adopted as National Annexes to Eurocode 1, the snow load for the analyzed location is twice as high, exceeding the capacity of the existing structure and indicating the need for further intervention or strengthening.

Based on the conducted analyses and proof load test results, the observed column damage does not currently significantly affect the global stability of the structure. However, the identified degradation of the concrete and concrete cover, along with potential reinforcement corrosion, represents a risk to its long-term reliability and stability.

The comparative analysis of the applied measurement techniques performed within the proof load testing programme indicates that mechanical dial gauges provide the most reliable and consistent measurements of structural deflections. Geodetic methods may represent valuable complementary approaches for monitoring deflections; however, they should primarily be considered as supporting techniques rather than principal methods for structural verification. In this respect, deflection measurements obtained using precise levelling proved to be more reliable than those derived from terrestrial laser scanning.

LITERATURE

[1] M. Latinović-Krndija, G. Broćeta, A. Cumbo, and Ž. Lazić, “Istraživanja, ispitivanja i edukacija u Centru za materijale i konstrukcije,” *Savremeno graditeljstvo SAG+*, no. 15, pp. 16–23, 2024.

[2] M. Muravljov, *Osnovi teorije i tehnologije betona*, Beograd, 2000.

[3] Instruction of Digital Concrete Test Hammer, SJJW, model HT225-W+, 2025.

[4] G. Broćeta et al., “Naknadna nedestruktivna ispitivanja građevinskih materijala u konstrukcijama,” in *Proc. XXVI International Fair Conference "GRAMES – DEMI 2023"*, Banja Luka, 2023.

[5] EN 12504-2:2021, *Testing concrete in structures – Part 2: Non-destructive testing: Determination of rebound number*, European Committee for Standardization, 2021.

EN 12504-4:2021, *Testing concrete in structures – Part 4: Determination of ultrasonic pulse velocity*, European Committee for Standardization, 2021.

[6] *Zbirka jugoslovenskih standarda i pravilnika za građevinske konstrukcije, Knjiga 1 – Dejstva na konstrukcije: Privremeni tehnički propisi za opterećenje zgrada (Sl. list SFRJ 61/48)*, Beograd, 1995.

[7] *Nacionalni dodatak – karta opterećenja od snijega na tlu u BiH BAS EN 1991-1-3/NA:2018*. Accessed: Sep. 1, 2025. [Online]. Available: https://www.bas.gov.ba

[8] *JUS.U.M1.047:1987, Ispitivanje konstrukcija visokogradnje probnim opterećenjem i ispitivanje do loma*, Beograd, 1987.

[9] *Laser scanning Europe – FARO Focus M70 scanner information*. Accessed: Sep. 1, 2026. [Online]. Available: https://www.laserscanning-europe.com/en/glossary/faro-focus-m-70

[10] S. Gordon, D. Lichti, M. Stewart, and J. Franke, “Structural deformation measurement using terrestrial laser scanners,” in *Proc. 11th FIG Symposium on Deformation Measurements*, Santorini, Greece, 2003.

[11] M. Latinović-Krndija et al., “Load Testing of Steel Structure in Doboj,” in *Proc. Int. Conf. of the Macedonian Association of Structural Engineers*, Ohrid, North Macedonia, 2025.

[12] Republicki hidrometeoroloski zavod Republike Srpske, *Časovne vrijednosti temperature vazduha za 3. i 4. jul 2025, AMS Kampus*, Banja Luka, 2025.

[13] G. Broćeta, *Prezentacija nastavnog predmeta na master studijama "Trajnost i procjena stanja betonskih konstrukcija"*, Arhitektonsko-građevinsko-geodetski fakultet, Univerzitet u Banjoj Luci, Banja Luka, 2024.

[14] *Zbirka jugoslovenskih standarda i pravilnika za građevinske konstrukcije, Knjiga 2/1 – Betonske konstrukcije: Pravilnik o tehničkim normativima za beton i armirani beton 87*, Beograd, 1995.

[15] J. Radić, *Betonske konstrukcije 4 – Sanacije*, Građevinski fakultet Sveučilišta u Zagrebu, Zagreb, 2010.

[16] A. Cumbo and M. Popović, “Ispitivanje probnim opterećenjem čelične krovne konstrukcije sportske dvorane u Gradišci,” in *Ocjena stanja, održavanje i sanacija građevinskih objekata i naselja, VIII naučno-stručno međunarodno savjetovanje*, Borsko jezero, 2013.

[17] D. Jokić and M. Latinović, “Ispitivanje krovne konstrukcije sportske dvorane na Palama probnim opterećenjem,” *AGG+*, vol. 1, no. 1, pp. 166–171, 2013, doi: 10.7251/aggplus1301166j.

[18] I. Duvnjak, M. Rak, and D. Damjanović, “Probno ispitivanje krovnih konstrukcija velikih sportskih građevina,” *Građevinar*, vol. 62, no. 10, pp. 887–896, 2010.